Application of Hydrologic Methods In Maryland

A Report Prepared by the Hydrology Panel
Convened by
The Maryland State Highway Administration
and
The Maryland Department of the Environment

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I. INTRODUCTION

The Maryland State Highway Administration (SHA) has been using deterministic models, primarily the Natural Resources Conservation Service (NRCS)-TR-20, to synthesize hydrographs and to estimate peak discharges for both existing and ultimate development conditions for some time. However, there has been a belief among SHA and other designers that the NRCS-TR-20 tends to overpredict peak flow in many cases. This belief is supported by U.S. Water Resources Council (1981) tests on ten procedures that found that the TR-20 had a mean bias of approximately 60% high on attempts to reproduce the 100-year peak discharges. A report entitled "Analysis of the Role of Storm and Stream Network Parameters on the Performance of the SCS-TR-20 and HEC-1 Under Maryland Conditions," by Ragan and Pfefferkorn (1992), concluded that the TR-20 could produce good results, but it was quite sensitive to the values selected for input parameters including the Manning roughness coefficients, representative cross sections, curve numbers, storm structure and storm duration. If the TR-20 was to continue to be used, the SHA wanted guidance that would lead to more dependable performance and results that were more consistent with Maryland stream flow records.

In addition, the SHA wanted to make greater use of the updated hydrologic estimating procedures developed for the State of Maryland by the U.S. Geological Survey (USGS) "Technique for Estimating Magnitude and Frequency of Peak Flows in Maryland," by Dillow (1996). The Water Management Administration (WMA), Maryland Department of the Environment (MDE), has selected the TR-20 model or its equivalent as a standard method for computing flood flows in Maryland. The WMA has been reluctant to accept general use of the USGS Regression Equations for the following reasons:

- they do not account for ultimate development
- they do not reflect recent land use changes, and
- they do not account for changes in storage and times of concentration.

The USGS regression equations have been classified as non-standard models by the WMA. The WMA requires that for a non-standard model to be considered for use in estimating flood peaks the model must meet the following conditions:

- Be in the public domain.
- Be generally accepted by the hydrologic community.
- Be verifiable.

However, the USGS regression equations meet all three of the above criteria. First, the regression equations developed by the USGS are, by definition, in the public domain. Second, the regression methodology is widely used and recognized as acceptable by the hydrologic community. And third, the original data, regression methodologies, and the resulting equations are published and, therefore, readily verifiable.

The use of the uncalibrated TR-20 is not recommended. Where sufficient actual, measured rainfall and runoff data are available, the TR-20 model should be calibrated and, if possible, verified prior to its application. However, the availability of on-site rainfall and runoff data is rarely the case in actual practice. In these more typical circumstances, **the USGS regression equations may be used as a basis to "calibrate"** the **TR-20 model providing the watershed conditions are consistent with those used to develop the USGS equations.**

Because of the need to address the NRCS-TR-20 and USGS regression equation issues outlined above and an array of other concerns being faced by the two organizations, the Maryland Water Management Administration and the Maryland State Highway Administration appointed a special hydrology panel. The following members were selected by the WMA and the SHA:

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1.1 RECOMMENDATIONS

1.1.1 Overview of the Modeling Process and the Calibration Requiements

The hydrologic analysis of Maryland State Highway Administration bridges and culverts must evaluate the behavior of the structure and local stream under both existing and ultimate development watershed conditions. Because two land cover and flow path conditions are involved, the basis for these hydrologic analyses must be a deterministic model that can simulate the runoff processes that occur during and after the storm. The recommended first step is to calibrate the deterministic model using field and map defined input parameters so that it adequately describes the runoff processes under existing watershed conditions. After the designer is satisfied that the model provides a realistic representation of the existing watershed conditions, the impact of ultimate development will be simulated by adjusting the input parameters to reflect the planned land cover and flow path modifications.

The panel discussions focused on watersheds having drainage areas larger than one square mile. Hydrologic analyses for all watersheds having drainage areas larger than one square mile will be supported by field investigations and the design discharges will be determined utilizing two hydrologic models: (1) a USGS probabilistic method based on stream flow records and (2) a flood hydrograph deterministic procedure such as the NRCS- TR-20 or its equivalent. The objective is to use the USGS probabilistic method that is based on long-term stream flow records collected in the State to ensure that the TR-20 produces peak discharges that are consistent with Maryland conditions. As described in Chapters 3 and 4 of this report, the sensitivity of the TR-20 to the values assigned to its input parameters and the uncertainties associated with the selection of these parameters are such that this calibration against USGS historical data is considered mandatory. The USGS methodology will be utilized in the following order if priority to determine peak flow:

1. A gage located at the site with the record being weighted with the regional regression estimates as presented by Dillow (1996) or future studies once they became available. The discharges reported will be the weighted estimate and an error bound of plus one standard error of prediction.

2. If there is no gage at the site, but there is a gage on the same stream that can be transposed, (the gage's data can be transposed ± half the gaged area up or downstream). The gaged record will be transposed to the site following the approach recommended by Dillow (1996). The discharges reported will be the estimate and an error bound of plus one standard error of prediction.

3. If there is no gage on the stream and the watershed characteristics are within the bounds of those used to derive the USGS regression equations, the regression equations will be applied to the watershed. The discharges reported will be the regression equation estimate and an error bound of plus one standard error of prediction.

The term, "± one standard error of prediction" is equivalent to 68% prediction limits.

The NRCS-TR-20 or its equivalent will be applied to the existing watershed conditions and calibrated against one of the three USGS methodologies presented on the previous page. The TR-20 input parameters defining the existing watershed land cover and drainage characteristics will be based on careful field reconnaissance and map investigations. The, TR-20 model will be run using the latest IDF curves and center-peaking NRCS Type II hyetographs as design storms. The volumes of these design storms will be defined from the isohyetal maps presented in Appendix 6. **Until new research on storm structure is complete, the 100-, 50-, and 25-year storm events should be derived using the NRCS 24-hour Type II design storm duration. The 10-, 5-, and 2-year storm events should be derived using either a 6 or 12 hour duration based on the most intense periods of the NRCS 24-hour Type II design storm duration.** [For watersheds having a total time of concentration of less than six hours, the 6-hour design storm duration is appropriate. For watersheds having a total time of concentration greater than six hours, the 12-hour design storm duration is appropriate.]

If the peak discharge of the hydrograph synthesized for the design storm is bounded by the USGS estimate and the upper limit of the standard error of prediction, then the analysis will be accepted as a reasonable representation of the runoff for existing watershed conditions. The model then forms the basis for simulating the watershed under ultimate development conditions.

If the discharge estimated by the hydrograph model is outside the window defined by the USGS estimate and an upper bound of plus one standard error of prediction, additional investigations and simulations will be conducted to determine:

1. Are the watershed conditions consistent with those in the USGS sample?

2. Are the USGS procedures appropriate for use on this watershed?

3. Even though the averaged watershed characteristics are consistent with the USGS sample, are there specific conditions such as extensive stream valley wetlands or a deeply incised channel or other factors that would cause unusually low or high peak discharges?

4. Are the hydrograph model parameters defining the curve number, time of concentration and storage attenuation appropriate for the field conditions being simulated? If not they can be adjusted in accordance with Chapter IV. Any adjustments must be justified with supporting documentation and **MUST BE WITHIN THE BOUNDS OF SOUND HYDROLOGIC PRACTICE**. Some parameter adjustment is allowed because the TR-20 is quite sensitive to the assigned values and it is very difficult to select quantities that best represent the watershed conditions.

If the existing watershed condition is more than 15% urbanized, the USGS regression equations do not apply. Thus, the TR-20 calibration process for existing conditions will be a two step process. First, the designer will estimate the predeveloped land cover distribution and calibrate to the USGS regression equations for this predeveloped condition. These TR-20 discharges will then be adjusted to reflect the increased curve numbers and the drainage network of the existing condition. The process is described in section 4.6 and illustrated in Appendix 4 of this report. The Panel believes that the uncertainties associated with a "predeveloped calibration" are less than those associated with an approach that requires the designer to select TR-20 input parameters without any opportunity for calibration.

If the USGS and hydrograph results cannot be reconciled, the designer should explain why the existing watershed conditions are significantly different from those defining the USGS sample or why the hydrograph model is not applicable to this particular watershed. The analyst will then pick the most appropriate method for the specific watershed. In western Maryland (Appalachian Plateau as defined in Dillow (1996)), there are indications that flood producing rainfalls may be shorter duration than those further east. Therefore, if the flood estimates using the 24-hour storm do not lie between the regression estimate and the upper 68% limit, in the Appalachian Plateau the analyst should use the 12-hour storm for the 25-, 50- and 100-year events and the 6-hour storm for the the 2-, 5- and 10-year events.

1.1.2 Issues Concerning the Selection of TR-20 Input Parameters

First step is to use map and field investigations to select input parameters that are consistent with established hydrologic practice and give a reasonable simulation of existing watershed conditions. If inputs give results that are outside USGS bounds, the designer will review the parameters used as inputs to define the TR-20 simulation. If the review indicates that a parameter may be incorrect, additional field and map investigations will be used to support any corrections. In no instance will TR-20 inputs be accepted that are outside the bounds of standard hydrologic practice.

Before attempting revise input parameters in a TR-20 calibration against one of the USGS statistical approaches, the designer should carefully study the report, MD-SHA AWO92-351-046, "Analysis of the role of storm and stream parameters on the performance of SCS-TR-20 and HEC-1 under Maryland conditions".

Normally, watersheds having drainage areas larger than one sq. mile will be delineated on 1:24000 quad sheets. Special care must be taken in locating the ridge line on the eastern shore or in other areas of low relief.

The designer should perform a map check of the results of automatic boundary delineation using the USGS digital terrain data. Different data resolutions will give different results and the low relief of the eastern shore remains a problem area.

The NRCS presents runoff curve numbers for many hydrologic soil-cover complexes as a range covering "good", "fair" and "poor". Also, Figure 10.2 of USDA-SCS-NEH-4 (1985) presented in this report as Figure 3.2 shows that there is scatter in the data used to develop the Runoff Curve Number (RCN) tables. Thus, the Panel recommends that the designer be granted a reasonable degree of latitude in the selection of RCN values for individual land parcels during the calibration process providing the values remain within the range recommended by NRCS and that the decision be justified in writing. Adjustments must be made on a parcel by parcel basis and cannot be made by simply changing the overall watershed RCN.

The commonly used peak rate factor of 484 in NRCS dimensionless unit hydrograph (DUHG) is known to vary for different terrain. The regional DUHGs for Maryland are currently being updated. Until new peak rate factors are published by NRCS, the designer may use those of Table 3.1.

Designers are encouraged to explore time-area curve derived DUHGs that can be developed from the digital terrain data in the SHA geographic information system, GISHYDRO-2000.

The use of the NRCS lag equation to estimate the time of concentration should not be used on watersheds having drainage areas in excess of five square miles. The hydraulic length in the equation should not be less **than 800 feet because** shorter lengths result in artificially short lag times. The slopes in the equation can be estimated using digital terrain data, but, caution must be observed because the 100 meter data will give different results than that obtained using the 30 meter data.

Because the quantity of flow and, therefore, the hydraulics are different for each storm frequency it is logical to expect that the time of concentration will be different for a two year storm than for a 100 year storm. The Panel supports the recommendation of NRCS that the 2-year storm conditions be used to estimate the time of concentration.

NRCS kinematic wave equation should be used to estimate time of overland flow travel with a maximum **flow length of 100 feet.**

Use the 1:24000 scale USGS 7.5 minute quadrangle sheets to estimate channel length. It is recognized that this scale cannot adequately represent meanders and, therefore, estimated length may be too short and slope too steep. When field investigations or more detailed maps indicate that such is the case, the designer may increase the estimated length, **providing the increase is justified in writing.**

As illustrated by Equation 3.16, it is difficult to estimate <u>the</u> correct Manning roughness coefficient. Variations in the estimate of the Manning roughness can produce significant changes in the estimated peak discharge. The designer should exercise extreme care in the estimate of the roughness and use comparisons with the USGS statistical approaches to improve the estimates.

Velocities at "bank full" conditions are to be used in estimating the time of travel through the main channel. Selection of the representative bank full hydraulic radius is difficult because the bank full cross section varies along the length of the channel. A "best estimate" should be made using field and map investigations and then brought into agreement with the USGS models through corrections justified by additional field and/or map investigations.

When the watershed is divided into sub-basins, the routing cross sections and the channel and overbank roughness coefficients are difficult to estimate and can have a significant impact on the attenuation simulated by the routing procedure.

In situations where errors can result is loss in life or major economic damage, routing cross sections should be developed through detailed mapping along the stream.

When the economics of a project do not justify detailed surveys along the length of a stream, reasonable modeling results can be produced with:

> Bank-full cross sections developed from regional regression equations that relate channel depth and width to the drainage area above the cross section;
>
> Routing sections developed by drawing perpendicular transects to the channel across the contours;
>
> In both cases field investigations should be made to ensure that the sections are realistic for the watershed involved.

When subdividing a watershed into sub-basins, the designer should carefully review the guidelines of the NRCS-TR-20 Manual to avoid the mistake of making too many subdivisions and, therefore, producing "kinematic translation" which results in no peak flow attenuation by the channel. The TR-20 manual states that the main time increment "should be about 0.1 or 0.2 of the shortest time of concentration…generally not smaller than 0.1 hours". The travel time between cross sections should be greater than one half of the main time increment?

Changes in the duration and/or structure of the design storm used as an input to the TR-20 produces major changes in the magnitude of the peak discharge and shape of the runoff hydrograph. More research is needed to finalize synthetic storm structure and duration to be used for specific frequencies and locations in Maryland. Until new research on storm structure is complete, the designer should continue to use design storms developed from the structure of the NRCS Type II distribution modified for different durations as furnished by the State Highway Administration. With the exception of watersheds in the Appalachian Plateau, flood estimates will be developed for:

The 25, 50 and 100 year events using a 24 hour duration storm;

The 2, 5 and 10 year events using either a 6 or 12 hour duration storm; If the time of concentration is greater than six hours, the 12 hour duration storm must be used.

Preliminary analyses indicate that the flood producing rainfalls in the Appalachian Plateau are considerably shorter than those in the rest of the State. Until the completion of further studies, if reasonable agreement with the USGS approaches cannot be achieved, flood estimates may be developed for:

The 25, 50 and 100 year events using a 12 hour duration storm;

The 2, 5 and 10 year events using a 6 hour duration storm.

In all instances, the hyetograph time increment, Δt, shall not exceed six minutes. It must be emphasized that the decision on the duration of the design storm must be supported in writing. The other TR-20 input parameters must be consistent with accepted practice and the TR-20 results should fall between the USGS expected value and plus one standard error of prediction.

IDF curves and the isohyetal maps of Appendix 6 are developed from point measurements. The spatial distribution of the design storm should be reflected by reducing the rainfall intensities as a function of duration and watershed area using the graph of USWB-TP-40 reproduced in this report as Figure 3.22.

If there are culverts or other storage producing structures along the stream, the attenuation should be reflected in the inputs to the TR-20.

Where available, comprehensive planning maps, as opposed to zoning maps, should be used to predict future land use. The planning maps incorporate key elements of time and spatial distribution that are not apparent on zoning maps.

1.1.3 Need for Continuing Research

As described in Chapter V of this report, there are many areas of hydrology that require additional research if we are to improve our confidence in the modeling process. It is imperative that a continuing, well conceived and adequately funded research program be implemented to address a number of problems, especially,

Improving he structure and duration of the design storms;

Using the time-area curve available from the digital terrain data to generate geomorphic unit hydrographs that are unique for the watershed being modeled;

Until procedures for the future use of geomorphic unit hydrographs can be implemented, research must continue on the regionalized peak factors to be used with the NRCS dimensionless unit hydrograph;

Improving methods for estimating times of travel through the watershed;

Peak discharge transposition of gaging station data;

Estimating confidence levels that are appropriate for TR-20 adjustments;

Proving improved statistical alternatives to develop estimates of the 2 – 500 year peak discharges for rural and urban streams in Maryland;

Defining guidelines for the application of the new Muskingum-Cunge routing module in the NRCS-TR-20 on watersheds above roadway drainage structures.

Developing guidelines for estimating NRCS runoff curve number from information on zoning maps.

1.2 RATIONALE

1. **Each watershed will be analyzed by two widely accepted approaches,** one statistical (USGS) and one deterministic (TR-20 or equivalent). In the past the effort associated with such an approach would make it prohibitive. **With the current GIS supported capabilities** that includes automatic delineation of the watershed boundaries, the tasks can be performed in considerably less time than was required by conventional techniques.

2. Studies have shown that the TR-20 often predicts peak discharges that are not consistent with the peaks that have been measured at Maryland stream gages. A major contributor to this problem is the fact that it is very difficult to select <u>the</u> curve number, <u>the</u> Manning roughness coefficients and <u>the</u> "typical" cross sections that represent the watershed conditions. Small errors in the selection of these parameters can lead to incorrect estimates of the volume of runoff, the time of concentration and the storage attenuation and, therefore, lead to peak flow predictions that are too high or too low. Calibration against USGS gages or USGS regression equations that are based on statistical analyses of 219 stream gages located in Maryland and adjacent states can **aid the designer in the selection of appropriate hydrograph input parameters and,** therefore, produce estimated peaks that are consistent with Maryland conditions. The calibration will also provide a confidence that the TR-20 is not overpredicting to cause unnecessary construction costs and not underpredicting to cause unnecessary flooding risks.

3. The recommended procedures are consistent with accepted practice, especially with AASHTO that states, "What needs to be emphasized is the need to calibrate to local conditions. This calibration process can result in much more accurate and consistent estimates of peak flows and hydrographs… Should it be necessary to use unreasonable values for variables in order for the model to produce reasonable results, the model should be considered suspect and its use carefully considered." An example of an inappropriate use of the TR-20 would be to use an NRCS dimensionless hydrograph peak factor of 484 on the Eastern Shore of Maryland, where the recommended peak factor is 284.

4. The recommended procedure is to make use of the USGS stream flow gages as the cornerstone for calibrating the hydrograph model. The USGS methods are utilized to ensure that the deterministic model provides a realistic representation of existing watershed conditions. Once confident that the deterministic model represents the existing conditions, the designer can vary the input parameters to simulate changes in the land cover and drainage networks associated with ultimate development and be fairly confident in the final results.

5. It is not the intent of this report to recommend that the calibration of the hydrograph model be accomplished at the upper bound of the prediction interval. Rather, the prediction limits can be used to provide an indication of the level of risk associated with the discharge selected. Assuming that the USGS estimates are unbiased, 50% percent of the peaks measured on watersheds having these characteristics will be higher and 50% will be lower than the expected value. Approximately 68% of the peak discharges will fall between plus and minus one standard error of prediction of the expected value. Thus, there is an approximately 84% probability that the peak for this type of watershed will not exceed that indicated by the upper bound. Similarly, there is an 84% chance that a measured peak for this type of watershed will be greater than that indicated by the lower bound. For purposes of "calibrating" the TR-20 model, the model parameters will be adjusted (if necessary) so the estimated flood discharge falls between the regression estimate (expected value) and the upper 68 percent prediction limit.

II. U.S. GEOLOGICAL SURVEY METHODS

2.1 FLOOD DISCHARGES AT GAGING STATIONS

Flood discharges at gaging stations located at the site of interest are weighted with the regional regression estimates as presented by Dillow (1996). The weighted discharges and an error bound of ± plus error of prediction are reported.

Estimates of design discharges, such as the 100-year flood discharge, are made at gaging stations where there is at least 10 years of annual peak discharges by using Bulletin 17B, Guidelines For Determining Flood Flow Frequency (Interagency Advisory Committee on Water Data (IACWD), 1982). These guidelines are used by all Federal agencies and several state and local agencies for flood frequency analysis for gaged streams. Bulletin 17B guidelines include fitting the Pearson Type III distribution to the logarithms of the annual peak discharges using the sample moments to estimate the distribution parameters and provides for (1) outliner detection and adjustment, (2) adjustment for historical data, (3) development of generalized skew, and (4) weighting of station and generalized (regional) skew.

Computer programs for implementing Bulletin 17B guidelines are available from the U.S. Army Corps of Designers (USACE) (Program HEC-FFA User's Manual, USACE, 1992) and the U.S. Geological Survey (USGS) (Program PEAKFQ User's Manual, Thomas et al., 1999). Annual peak discharges are available for gaging stations in Maryland and surrounding states from the USGS over the World Wide Web at http://waterdata.usgs.gov/nwis-w/us/. The annual peak data and the available computer programs can be used to estimate design discharges for Maryland streams.

If the gaged watershed has undergone significant change during the period of record, the annual peak data may not be homogeneous. The user should insure that the data are homogeneous, and exhibit no significant trends due to land-use change before performing the frequency analysis. A simple way to evaluate this is to plot the annual peak discharges versus time and determine if there are any noticeable trends in the data. Statistical procedures for performing a more quantitative evaluation of trends and nonhomogeneity in flood data are discussed by Pilon and Harvey (1992), McCuen and Thomas (1991) and McCuen (1993).

In the most current regional flood frequency study, conducted in cooperation with the Maryland State Highway Administration, Dillow (1996) used Bulletin 17B procedures to estimate selected design discharges at gaging stations in Maryland and surrounding states in the development of regional regression equations. Dillow (1996) used the generalized skew map in Bulletin 17B in computing a weighted skew. Dillow (1996) provided estimates of the 2-, 5-, 10-, 25-, 50-, 100- and 500-year peak discharges at 219 rural gaging stations in Maryland and surrounding states that were used in his regional analysis. He also provided design discharges for an additional 17 gaging stations (mostly urban watersheds) that were not used in the regional analysis. Estimates of design discharges provided by Dillow (1996) are available to those users who choose not to perform their own Bulletin 17B analysis.

If the watershed characteristics of the gaging station are similar to those used in deriving the regression equations, then the best estimate of design discharges at the gaging station is considered to be a weighted estimate based on gaging station data and regional regression estimates. Dillow (1996) describes the watershed and climatic characteristics used in the development of the regional regression equations. Watershed and climatic characteristics evaluated but not used in the published regression equations are given in Appendix 1. Data provided by Dillow (1996) and data given in Appendix 1 can be used to determine if the gaging station of interest has similar watershed characteristics as those gaging stations used in developing the regression equations. The procedures for weighting the gaging station and regression estimates are described below.

In accordance with Bulletin 17B guidelines (IACWD, 1982), it is assumed that an estimate at a single gaging station is independent of the regional regression estimate. Assuming independence of estimates, Hardison (1976) has shown that a weighted estimate, obtained by weighting each estimate inversely proportional to its variance, has a variance less than either of the individual estimates. Hardison (1976) further demonstrated that weighting two estimates inversely proportional to their variances was comparable to weighting by the equivalent years of record. The following weighting equation proposed by Dillow (1996) is used:

$$\log(Qw) = (\log(Qg) * Ng + \log(Qr) * Nr) / (Ng + Nr) \qquad (2.1)$$

where

log (.) is the logarithm of the peak discharge,
Qw is the weighted peak discharge at the gaging station, in cubic feet per second (cfs),
Qg is the peak discharge at the gaging station based on observed data, in cfs,
Qr is the peak discharge computed from the appropriate regional regression equation, in cfs,
Ng is the years of record at the gaging station, and
Nr is the equivalent years of record for the regression estimate.

The equivalent years of record of the regression estimate is defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of prediction of the regional regression equation. The equivalent years of record (Nr) for each watershed are computed as (Hardison, 1971):

$$Nr = (S/SEp)^2 \; R^2 \qquad (2.2)$$

where

S is the standard deviation of the logarithms of the annual peak discharges at the ungaged site,
SEp is the standard error of prediction of the regression estimate in logarithmic units, and
R^2 is a function of recurrence interval and skewness and is computed as (Stedinger et al., 1993):

Piedmont
50 sites
G = 0.569
S = 0.307

$$R^2 = 1 + G*Kx + 0.5 *(1+0.75*G^2)*Kx^2 \qquad (2.3)$$

where

G is the average skewness for a given hydrologic region, and
Kx is the Pearson Type III frequency factor for recurrence interval x and skewness G. Kx can be obtained from Appendix 3 of Bulletin 17B or can be estimated as (IACWD, 1982):

$$Kx = 2/G * [1 + (G*Zx)/6.0 - G^2/36]^3 - 2/G \qquad (2.4)$$

where

Zx is the standard normal deviate for recurrence interval x (for example, Zx = 2.32635 for the 100-year recurrence interval).

Average skewness values G were defined using design discharges from Dillow (1996) and are as follows: 0.46 for the Appalachian Region, 0.49 for the Blue Ridge Region, 0.53 for the Piedmont Region, 0.69 for the Western Coastal Plain Region and 0.67 for the Eastern Coastal Plain Region. The average skewness values were computed from the regression estimates (Dillow, 1996) for the 2-, 10- and 100-year flood discharges for the 219 gaging stations used in the regional analysis and an average skew was computed for the five hydrologic regions.

For an ungaged site, the standard deviation of the logarithms of the annual peak discharges (S in Equation 2.2) is estimated on the basis of drainage area. Regression equations for estimating S based on drainage area were developed by Dillow (1996) as part of his regional flood frequency analysis and are incorporated into the computer program described below. The standard error of prediction (SEp) for the ungaged site is computed as the sum of the model and sampling error as described by Hodge and Tasker (1995). The model error is a measure of how well the explanatory variables in the regression equation explain the variability in flood hydrology in the region assuming infinite record lengths for an infinite number of gaging stations. The sampling error is a measure of the error in the regression coefficients and is determined as a function of the watershed characteristics of the ungaged site.

A computer program, developed by Tasker (USGS), can be used to compute the weighted estimate given in Equation 2.1 and for determining the equivalent years of record, standard error of prediction and prediction intervals for these estimates. This computer program is a modification and extension of the program described by Hodge and Tasker (1995) and can be obtained from the Maryland State Highway Administration. The equivalent years of record for the weighted estimate is assumed to be Ng+Nr (see Equation 2.1), the sum of the years of gaged record and equivalent years of record for the regression estimate. The standard error of prediction is computed by substituting the equivalent years of record for the weighted estimate in Equation 2.2 and solving for SEp. The prediction intervals are then computed as a function of SEp using procedures described in the section "Estimates at Ungaged Sites." 2.

Examples 1 and 2 in Appendix 2 illustrate how Tasker's program can be used to compute a weighted estimate at a gaging station, Little Patuxent River at Guilford Downs (station 01593500), a 38-square-mile watershed in the Piedmont Region. The flood discharges for station 01593500 (Qg) based on 58 years of record are taken from Dillow (1996) and are given in Table 2.1. Also provided in Table 1 are the Piedmont Region regression estimates (Qr) at station 01593500, which are taken from Example 1 in Appendix 2.

Table 2.1. Flood frequency estimates for Little Patuxent River at Guilford Downs based on gaging station data (Station), regression equations (Regression) and a weighted (Weighted) estimate. $R = Q_w/Q_r$

Return period (Qw) (years)	Station (Qg) (cfs)	Regression (Qr) (cfs)	Weighted (cfs)	
2	1340	1670	1350	.308
5	2480	2990	2520	.343
10	3620	4160	3690	.337
25	5670	5960	5720	
50	7780	7550	7730	
100	10500	9380	10240	1,092
500	20300	14900	19000	

The regression estimates (Qr) are weighted with the station estimates (Qg) using Equation 2.1. The weighting factors are the years of record at station 01593500 (Ng = 58) and the equivalent years of record (Nr) for the regression equations given in Example 1 in Appendix 2. The weighted estimates are shown in Example 2 in Appendix 2 and in Table 2.1. For example, the 100-year weighted estimate is computed from Equation 2.1 as

$$\text{Log } Qw = (\log Qg * Ng + \log Qr * Nr) / (Ng + Nr) = (4.02119*58 + 3.97220*16.34) / (58+16.34) = 4.01042 \text{ log units}$$

$$Qw = 10^{4.01042} = 10{,}240 \text{ cfs}$$

The equivalent years of record for the weighted estimate is assumed equal to the sum of the observed record length (58 years) and the equivalent years of record from the regression equations. For the 100-year weighted estimate, the equivalent years of record is 74.34 years as shown in Example 2 in Appendix 2.

2.2 TRANSPOSITION OF GAGING STATION DATA

If there is no gaging station at the site of interest, but there is a station on the same stream, the gaging station data can be transposed up or downstream to within 50% of the gaged area. Flood discharges at the gaging station are weighted with regional regression estimates as described in Section 2.1 and transposed to the site of interest following the approach described by Dillow (1996). The transposed flood discharges and ± plus error of prediction are reported.

Procedures described by Dillow (1996) are recommended for obtaining estimates of design discharges for ungaged sites that are on the same stream as the gaging station, have similar watershed characteristics as the gaging station and are within 50% of the drainage area of a gaging station. Data provided by Dillow (1996) and in Appendix 1 can be used to determine if the gaged stream has watershed characteristics similar to those used in developing the regression equations. The procedure involves three steps:

1. Compute the ratio (R) of the weighted estimate (equation 1) to the regression estimate at the gaging station

2.5

$$R = Qw/Qr \qquad (5)$$

where Qw and Qr are defined in equation 1.

2. Scale the ratio R based on the difference in drainage area between the ungaged site and the gaging station using the following equation (Sauer, 1974):

2.6

$$Rw = R - ((2|Ag\text{-}Au|)/Ag) *(R\text{-}1) \qquad (6)$$

where

Rw is the scaled ratio,
Ag is the drainage area in square miles at the gaging station, and
Au is the drainage area in square miles at the ungaged location.

3. Compute the final estimate (Qf) at the ungaged site as

$$Qf = Rw * Qu \qquad (7) \quad 2.7$$

where

Qu is the regression estimate at the ungaged site.

An example of using Tasker's program to compute an extrapolated estimate is given in Example 3 of Appendix 2. The weighted estimates at the Little Patuxent River gaging station at Guilford Downs (shown in Table 2.1) where the drainage area is 38 square miles, are extrapolated downstream to an ungaged location where the drainage area is 50 square miles. For example, the weighted (Qw) and regression (Qr) 100-year flood discharge at station 01593500 are 10,240 and 9,380 cfs, respectively, and the regression estimate (Qu) at the ungaged location is 11,050 cfs. The 100-year flood discharge at the ungaged location on the Little Patuxent River is computed to be 11,400 cfs using Equations ~~2.4-2.6~~, as follows: ? ~ 2.5 to 2.6

$$R = Qw/Qr = 10{,}240/9{,}380 = 1.092$$

$$Rw = R - [((2|Ag\text{-}Au|)/Ag) * (R-1)] = 1.092 - [((2|38\text{-}50|)/38) * (0.092)] = 1.034$$

$$Qf = Rw * Qu = 1.034 * 11{,}050 = 11{,}400 \text{ cfs}$$

The equivalent years of record is 37.71 years for the 100-year flood discharge at the ungaged location and is shown in Example 3 in Appendix 2. This value is interpolated between 74.34 years for the weighted station data at 38 square miles and 16.34 years for the regression equation estimate at 50 square miles. The computation is 74.34 – ((74.34-16.34)*12/19) = 37.71 years.

As noted earlier, the gaging station data should only be extrapolated to 0.5 to 1.5 times the drainage area at the gaging station. Example 5 illustrates an attempt to extrapolate the station flood frequency estimates for the Little Patuxent River at Guilford Downs (drainage area 38 square miles) downstream to an ungaged location of 68 square miles (about 1.8 times the drainage area). Note the message "Difference in drainage area for Station 1593500 too great: NO ADJUSTMENT MADE." The results provided in Example 4 represent the regression estimate with no weighting with station data.

Equation 2.6 6 was developed with the limiting assumption that estimates would only be extrapolated upstream and downstream on the same stream to 0.50 or 1.50 times the drainage area of the gaging station. If Equation 2.6 is used beyond these limits, then irrational results may be obtained. If the gaged watershed has undergone significant change during the period of record, then the annual peak data may not be homogeneous and the extrapolation procedure may not be appropriate.

In the case where the ungaged site is between two gaging stations, Dillow (1996) recommends that Qg be estimated by interpolating between the two gaging stations on the basis of a logarithmic plot of peak discharge versus drainage area. An estimate of Ng is obtained as an arithmetic average of the record length at the two gaging stations using the differences in drainage area between the ungaged site and the gaging stations as the weighting factor. The values of Qg and Ng so obtained should be used in Equation 1 to get a final weighted estimate for the ungaged site.

Tasker's computer program can be used to obtain estimates of design discharges for ungaged sites on the same stream within 50% of the drainage area of a gaging station. The equivalent years of record, standard errors of prediction and prediction intervals are also computed for these estimates. The equivalent years of record are interpolated on the basis of drainage area using the years of record for the weighted estimate at the gaging station (Ng+Nr) and the equivalent years of record for the regression estimate (Nr) at a 50% increase or decrease in drainage area. The standard error of prediction (SEp) is computed by substituting the equivalent years of record for the extrapolated estimate into Equation 2 and solving for SEp. The prediction intervals are then computed as a function of SEp using procedures described in the section "Estimates at Ungaged Sites."

2.3 FLOOD DISCHARGES AT UNGAGED SITES

If there is no gaging station on the stream, regional regression equations are applied to the watershed if the watershed characteristics are within the range of those used to develop the equations. The flood discharges and ± plus error of prediction are reported.

Regression equations developed by Dillow (1996) can be used for estimating the 2-, 5-, 10-, 25-, 50-, 100- and 500-year peak discharges for rural watersheds in Maryland which are not significantly affected by detention storage, urbanization, tidal marshes or changing land-use conditions such as mining, excavation or landfill activities. As described in Sauer et al. (1983), significant urbanization is assumed if more than 15% of the watershed land use is characterized as commercial, industrial or residential development (does **not** mean that 15% of watershed is impervious).

In addition, the watershed characteristics for the site of interest should be within the range of the watershed characteristics of the gaging stations used in the regional analysis. Dillow (1996) describes the watershed and climatic characteristics used in the development of the regional regression equations. Watershed and climatic characteristics evaluated but not used in the regression equations are given in Appendix 1. These data can be used to determine if the ungaged site has similar watershed characteristics as those used in developing the regression equations.

Tasker's computer program can be used to obtain flood discharge estimates at ungaged sites using the regional regression equations documented by Dillow (1996). The equivalent years of record, the standard errors of prediction and prediction intervals are also computed for these estimates.

The prediction intervals are then computed as:

$$\log Qx + t(c/2, n\text{-}p)*(SE^2(1+ho))^{0.5} \qquad \text{upper value} \qquad (2.8a)$$

$$\log Qx - t(c/2, n\text{-}p)*(SE^2(1+ho))^{0.5} \qquad \text{lower value} \qquad (2.8b)$$

where

Qx is the flood discharge for recurrence interval x,
t is the critical value of t for a 100 (1-c)% prediction interval with n-p degrees of freedom,
n is the number of gaging stations used in the regression analysis,
SE is the standard error of estimate in logarithmic units from the regression analysis,
p is the number of explanatory variables in the regression equation, and
ho is the leverage of the site.

The leverage expresses the distance of the site's explanatory variables from the center of the convex data set (called the Regressor Variable Hull) defined by the explanatory variables in the regression analysis (Montgomery and Peck, 1982). The prediction intervals are directly related to the magnitude of the leverage for a given site. The leverage is computed as (**bold letters** denote a matrix):

$$ho = xo\ (\mathbf{X}^T\mathbf{W}^{-1}\mathbf{X})^{-1}\ xo^T \qquad (2.9)$$

where

xo is a row vector of the logarithms of the explanatory variables at a given site,
$(\mathbf{X}^T\mathbf{W}^{-1}\mathbf{X})^{-1}$ is the covariance matrix of the regression parameters (T means transpose),
W is a weighting matrix used in the Generalized Least Squares regression program (Tasker and Stedinger (1989)), and
xo^T is a column vector of the logarithms of the explanatory variables at a given site.

An example of a Regressor Variable Hull (RVH) is given in Figure 2.1 for the Piedmont Region where forest cover +10% is plotted versus drainage area. The boundaries are defined by straight-line segments whose convex angles are less than 180 degrees. RVH's for the other regions are given in Appendix 3 where all explanatory variables for the given region are plotted against drainage area.

Just plots of explanatory variables

If an ungaged site has a leverage value greater than the maximum value for any gaging station in the regression analysis, then this indicates the regression equations are being extrapolated beyond the limits of the data and the point falls outside the RVH as defined in Figure 2.1 (Montgomery and Peck, 1982). Tasker's computer program prints a warning message if this occurs to alert the user that the regression equations are being extrapolated. It is possible to be within the RVH and still be outside the limits of one of the explanatory variables. Therefore, Tasker's computer program also alerts the user if any watershed characteristic is beyond the limits of the data used to derive the equations.

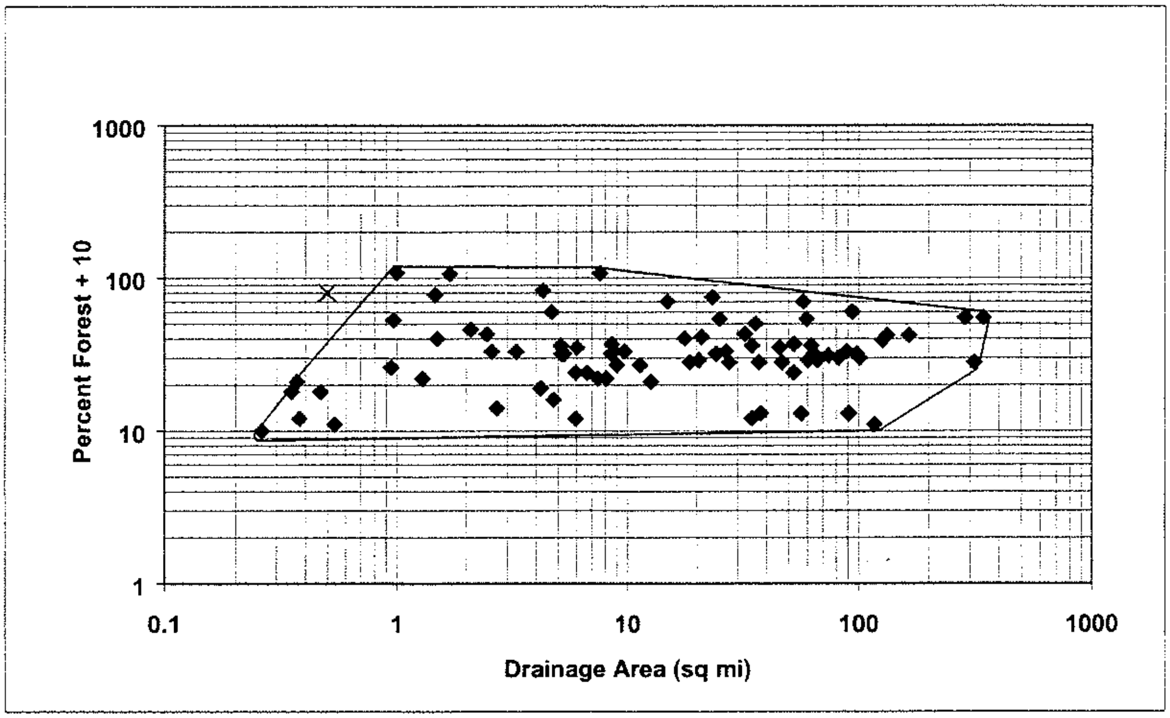


Figure 2.1. Regressor variable hull for the Piedmont Region in Maryland.

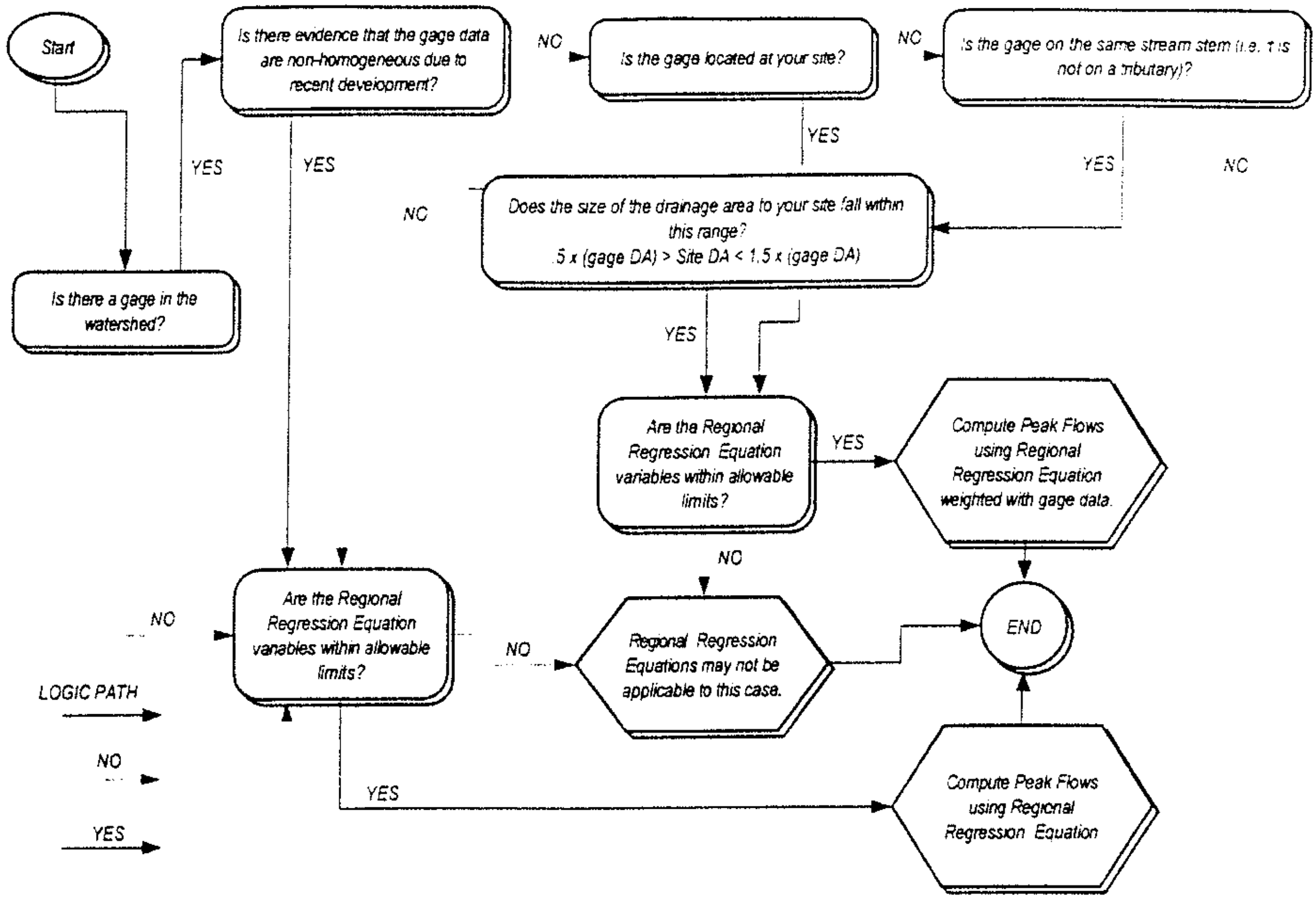
The regression equations developed by Dillow (1996) should not be used beyond the limits of the data used to derive them. The RVH plots of Figure 2.1 and in Appendix 3 can be used to determine if the ungaged site is within the applicable range of the regression equations.

An ungaged watershed in the Piedmont Region with a drainage area of 0.5 square miles and a forest cover of 70% (80% with constant of 10 added) is outside the RVH shown in Figure 2.1 and is an extrapolation of the regression equations. Note however that the drainage area and forest cover are within the limits of the data but the combination of a small watershed with high forest cover is not represented in the data set.
Example 5 in Appendix 2 illustrates applying Tasker's program to the 0.5-square-mile watershed with 70% forest cover. Note the message "WARNING - - Prediction beyond

observed data." This indicates to the user that the regression equations are being extrapolated beyond their applicable limits.

As described in Dillow (1996), if a basin lies in more than one region, the discharge for the basin is computed twice, as if the basin were entirely within each region. A weighted average discharge is then calculated with the weighting factors being the percentage of the total basin that is within each region. **The following flow chart was developed to illustrate the logic in applying the USGS Methods when the stream is gaged and ungaged.**

flow chart missing



FLOW CHART FOR USGS METHODS

FIGURE 2.2

III. BEHAVIOR OF THE NRCS-TR-20 MODEL IN RESPONSE TO UNCERTAINTIES IN THE INPUT PARAMETERS

3.1 OVERVIEW

The NRCS-TR-20 model is a deterministic hydrologic model that synthesizes a single event runoff hydrograph as a function of a rainfall input and watershed characteristics. The model is designed to operate on a time varying rainfall to produce a hydrograph that simulates the role of the watershed area; land cover; hydrologic soils types; antecedent moisture conditions; topography; characteristics of the overland, shallow confined, and channel flow paths; and, storage attenuation such as that created by flood plains, wetlands, structures, and ponds. A single watershed can be modeled by inputting the drainage area, time of concentration, curve number and a time-intensity rainfall distribution such as the NRCS-Type II 24-hour duration design storm. If the watershed is large or heterogeneous, it can be divided into a number of subwatersheds with their hydrographs attenuated by routing through the stream network that the user defines in terms of length, slope, roughness, cross-section and any storage elements or structures that may be distributed along its length.

Because the TR-20 model can simulate watershed conditions and changes in these conditions in terms of relatively simple input parameters, it continues to be the baseline for SHA hydrologic analyses that require hydrographs for both existing and ultimate development conditions on watersheds larger than one square mile. The first step is to select model parameters that are consistent with established hydrologic practice and give a reasonable simulation of existing hydrologic conditions. After the user is satisfied that the model is satisfactory for existing watershed conditions, the curve number and flow network parameters can be changed to simulate the hydrologic response of the watershed to a future, or ultimate development, condition.

Experience has shown that, like most deterministic hydrologic models, the NRCS-TR-20 model is quite sensitive to the values chosen for the input parameters. The SHA-sponsored study by Ragan and Pfefferkorn (1992) provides some examples of these sensitivities. There is also a belief among SHA and other designers that the TR-20 model tends to over- predict in many cases. This belief is supported by U.S. Water Resources Council (1981) tests on ten procedures for estimating peak discharges for ungaged watersheds. Each procedure was applied by five persons at gaging stations with at least 20 years of observed peak-flow record. Based on 105 applications at 21 gaging stations in the Midwest and Northwest Regions of the country, it was found that the TR-20 model overestimated the 100-year flood discharge by about 55%, the 10-year discharge by about 60% and the 2-year discharge by about 55%.

The Panel recognizes the parameter sensitivities of the NRCS-TR-20 model and its tendency to overpredict. However, the Panel has concluded that these problems can be overcome and that the TR-20 model can be a sound, dependable model for simulating existing and ultimate conditions for most watersheds provided that it is calibrated for local conditions. Calibration of all deterministic models is strongly recommended by AASHTO (1991, pgs. 7-17, 7-18). The Panel recommends that it become standard practice to require that the TR-20 be calibrated for existing watershed conditions against one of the USGS gage-based procedures of Chapter II, provided that the watershed conditions are consistent with those above the USGS gage or the sample used to derive the USGS regression equations. The regression equations are based on statistical analyses of 219 stream gages in Maryland and adjacent states having record lengths between 10 and 70 years. Thus, a successful calibration following the procedures outlined in Chapter IV and Appendix 4 can produce reliable TR-20 peak discharges that are consistent with Maryland conditions.

The remaining sections of Chapter III discuss the issues that the Panel examined with respect to defining the input parameters to the TR-20 model. Chapter IV and the appendices discuss procedures that will assist the TR-20 model user in the selection of input parameters during the calibration process.

3.2 DRAINAGE AREA

The scale of the map can create an error in the estimate of the drainage area. Delineating on a small scale map, such as 1:100,000, probably will not give the same drainage area as one would obtain from a 1:24,000 or 1:4,800 scale map. Normally, watersheds having drainage areas larger than one square mile of interest to the SHA will be delineated on a 1:24,000 scale USGS 7.5 minute quadrangle sheet. Special care must be exercised in flat terrain such as the Eastern Coastal Plain because of the wide spacing of contours and lack of definitive of ridge lines.

Recently, there has been considerable interest in generating watershed boundaries automatically from the digital terrain data now available from the USGS. The new version of GISHYDRO is built around automatic boundary delineation. Two issues must be recognized with the new GISHYDRO or any region growing method. First, the person using these techniques must be thoroughly trained. The procedures can give excellent results; but, if the user does not know what he or she is doing, significant errors can result. For example, if one tries to delineate a watershed that is too small - one containing only a few elevation points - the results will be very questionable. Figure 3.1, developed from a study by Fellows (1983), shows the percent difference between watershed areas manually delineated on paper 1:24,000 scale maps and those grown from digital terrain data as a function of the number of elevation points inside the boundary. A_m is the area determined "manually" by visually tracing the ridge lines on 1:24,000 scale maps. A_G is the area "grown" using the digital terrain data. There are two levels of detail in the current digital terrain data that are available, 30-meter and 90-meter. The 90-meter data may not give the same level of accuracy as the 30-meter data. If the area of the watershed is overestimated, the peak discharge will be overestimated as well.

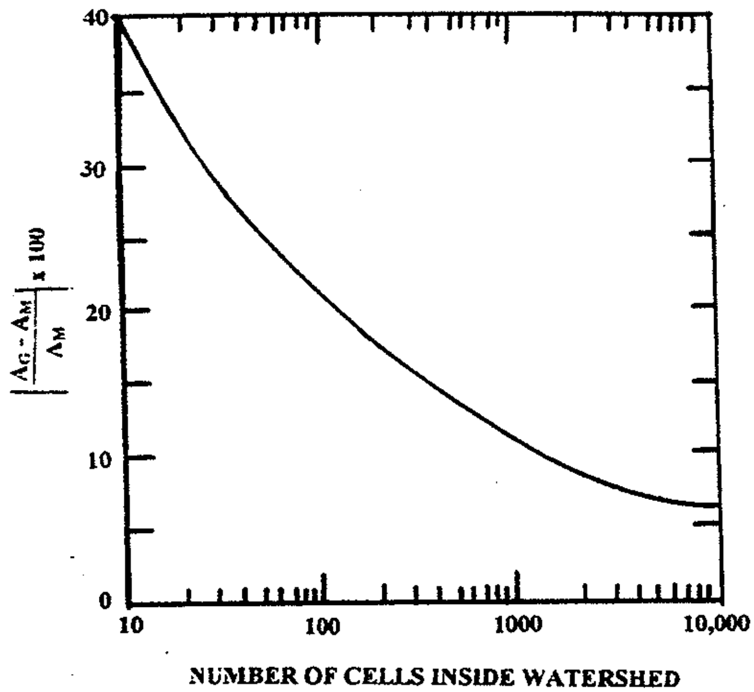


FIGURE 3.1
99% CONFIDENCE ERROR ENVELOPE FOR DIFFERNECE BETWEEN MANUALLY AND AUTOMATICALLY DEFINED AREAS

3.3 VOLUME OF RUNOFF

A deterministic model must have a component that estimates the rainfall excess that becomes the volume of the runoff hydrograph. Thus, there must be a means to account for the interception, infiltration and depression storage processes that occur in the watershed. In the NRCS family of models, the rainfall excess is estimated by a Curve Number (CN) that is a function of the land cover, the underlying soil type, and antecedent moisture conditions (AMC). Tables 2-2a, b, c, and d from NRCS TR-55 (1986) are recommended for use in SHA hydrologic analyses using TR-20.

The rainfall excess, or volume of runoff under the hydrograph, is given by Equation 3.1

$$Q = (P - .2S)^2 / (P + 0.8S) \qquad (3.1)$$

where

$$S = (1000/CN) - 10 \qquad (3.2)$$

Tables 2a through d in NRCS-TR-55 assign curve numbers in terms of "good," "fair," or "poor" condition in some of the land cover categories. First, it may be difficult for the designer to determine which of the conditions is appropriate for each land parcel in the watershed. Further, the curve numbers were derived from watershed data collected from across the United States. The specific numbers may or may not be appropriate for the particular Maryland watershed under investigation. Finally, Equation 3.1 is a simplification of

$$Q = (P - I_a)^2 / ((P - I_a) + S) \qquad (3.3)$$

where it is assumed that

$$I_a = 0.2S. \qquad (3.4)$$

The data on which the assumption of Equation 3.4 is based, presented as Figure 10.2 in SCS-NEH-4 (1984), are shown here as Figure 3.2.

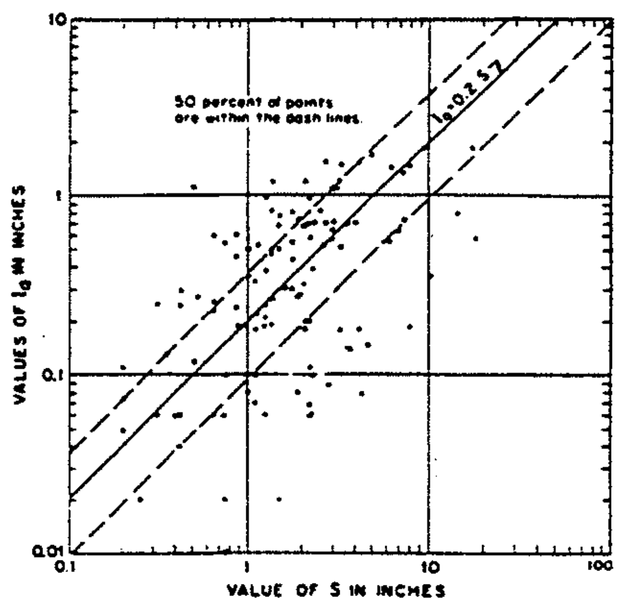


FIGURE 3.2
RELATIONSHIP BETWEEN I_a AND S
Plotted Points are Derived from Experimental Watershed Data
(Figure 10.2 of USDA-SCS-NEH-4)

The consequences of making an error in the determination of the weighted curve number for a natural watershed is illustrated by Figure 3.3 from Ragan and Pfefferkorn (1992).

The purpose of this volume of runoff section is to encourage users of the NRCS-TR-20 to recognize that estimating the volume of surface runoff using the curve number approach is an imperfect process. Thus, as described in Chapter IV, the Panel recommends that the user exercise a degree of flexibility in the selection of curve numbers to represent specific land/soil complexes provided that basis for the decision is explained.

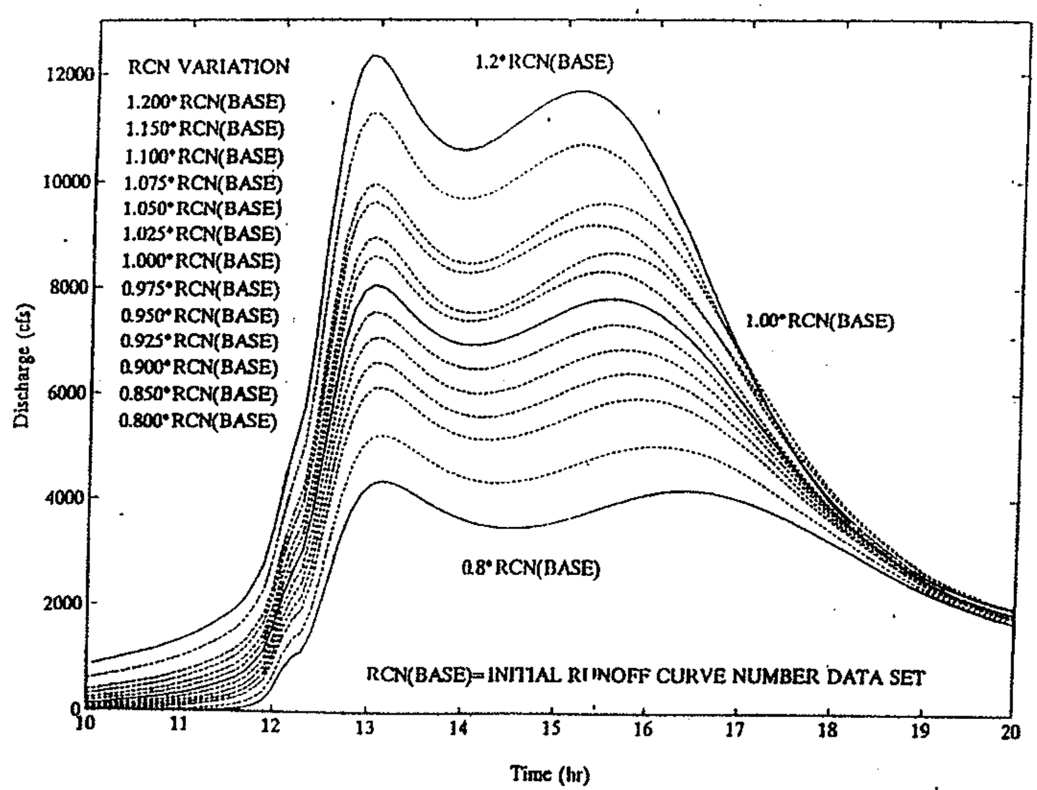


FIGURE 3.3
HYDROGRAPHS USING INDICATED INCREASES AND DECREASES IN WATERSHED CURVE NUMBERS

3.4 PEAK DISCHARGE AND SHAPE OF THE RUNOFF HYDROGRAPH

3.4.1 The Dimensionless Unit Hydrograph

A storm occurring on a low relief watershed with wide, flat streams will produce a long duration hydrograph with a low peak discharge in comparison with that generated by a high relief mountain basin having steep narrow channels. Many deterministic models, including the TR-20, simulate the interrelationships among the runoff processes through a unit hydrograph (UHG). If stream flow records are available for the subject watershed, the TR-20 allows a site specific UHG to be input. If possible, the derived UHG should be used. However, the usual circumstance is to use the default dimensionless UHG built into the TR-20. While the NRCS dimensionless UHG is thoroughly discussed in Chapter 16 of SCS-NEH-4 (1985), several issues are presented here for completeness.

The dimensionless UHG controls the shape and peak discharge of the runoff hydrograph using the drainage area, the volume of runoff, and the time of concentration as input parameters. SCS-NEH-4 (1985) gives the peak discharge of the unit hydrograph that the TR-20 convolutes with the time-distribution of rainfall excess as

Lag

$$q_p = 484AQ / 0.60T_c \tag{3.5}$$

where T_c is the time of concentration. In Equation 3.5, Q is 1.0 inches because it is a unit hydrograph.

The constant value of 484 is the "peak rate factor." SCS-NEH-4 points out that "this factor has been known to vary from about 600 in steep terrain to 300 in very flat swampy country." A UHG with a peak rate factor of 284 has been used for some time on the flat watersheds of the Maryland Eastern Coastal Plain.

In the case of the Maryland Eastern Coastal Plain UHG, the lower peaking factor accounts for the greater storage and longer travel times of the flat wetlands often found on streams in that area. However, one must be aware that a peak flow rate can sometimes be lowered by subdividing the watershed into sub-basins and then routing the sub-basin hydrographs through the storage provided by the network of connecting streams. In general, models that have larger (more than one square mile) sub-basins should use the regional dimensionless unit hydrograph. **In Maryland, these regional dimensionless unit hydrographs are currently being updated by the NRCS.** Until other values are published, the designer may use the new peaking factor values for the Maryland Dimensionless Unit Hydrographs, shown in Table 3.1.

TABLE 3.1

Unit Hydrograph Peak Factors

Add DimHYD

REGION	PEAK FACTOR
Eastern Coastal Plain	284
Western Coastal Plain	284
Piedmont	484
Blue Ridge	484
Appalachian	~~460~~ 434

Note

If a watershed falls within more than one region boundary, the TR-20 model can be split into appropriate parts with corresponding regional dimensional unit hydrographs.

In addition to the probable variation of the peak rate factor as a function of the watershed topography, it can also be seen from Equation 3.5 that the peak discharge of the UHG is set by the time of concentration, T_c. As described later in this chapter, the time of concentration is difficult to define. Thus, the NRCS dimensionless or any other "nationally-derived" synthetic UHG defined in terms of a few parameters can create errors in the runoff estimate. In the future there may be approaches that allow the use of more site specific UHG's, even when no stream flow records are available. Because of the availability of the USGS digital terrain data, the "geomorphic" UHG using a time-area-curve concept that tracks the flow path of each grid cell in the watershed should be a practical approach in the near future.

3.4.2 Time of Concentration and Lag

Definitions

Travel time is the time it takes for runoff to travel from one location in a watershed to another location downstream. Estimating travel time is complicated by the fact that it may occur on the surface of the ground or below it or a combination of the two. The *Time of Concentration* is the time required for runoff to travel from the hydraulically most distant part of the watershed to the outlet of the watershed. Recall that it is the time of concentration that is input to the TR-20 to define the peak discharge of the unit hydrograph from the dimensionless UHG. The *Lag* can be thought of as a weighted time of concentration. If the watershed is divided into increments, and the travel times from the centers of the increments to the watershed outlet are determined, then the lag is calculated as

$$L = \frac{\Sigma (a_i Q_i T_{ti})}{\Sigma (a_i Q_i)} \qquad (3.6)$$

where

L	is the lag time, in hours;
a_i	is the the ith increment of the watershed area, in square miles;
Q_i	is the the runoff from area a_i, in inches;
T_{ti}	is the the travel time from the center of a_i to the point of reference, in hours.

SCS-NEH-4 provides the empirical relation

$$L = 0.6\ T_c \qquad (3.7)$$

Lag, as defined by NRCS, is the time from the center of mass of the rainfall excess to the peak rate of runoff as shown by Figure 3.4. Similarly, the time of concentration is the time from the end of the rainfall excess to the point on the falling end of the hydrograph where the recession curve begins, as shown in Figure 3.5. It is quite difficult to determine the time that the rainfall excess begins and ends. Where sufficient rainfall and runoff data are not available, the usual procedures for determining L and T_c are outlined in the following sections.

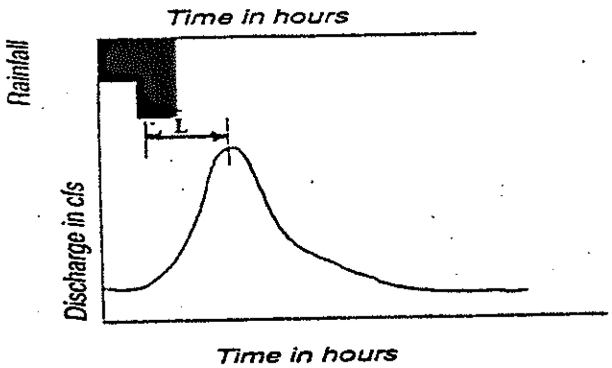


FIGURE 3.4
DEFINITION OF LAG TIME

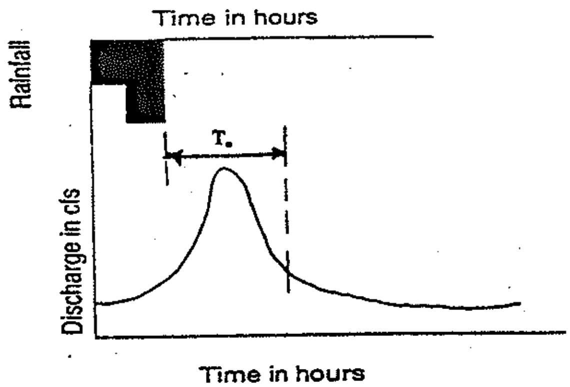


FIGURE 3.5
TIME OF CONCENTRARTION

SCS-NEH-4 discussed two methods for estimating time of concentration and lag when hydrograph data are not available. These methods, the curve number method and the flow path hydraulics method, are discussed in the following sections.

3.4.3 Curve Number Method to Estimate Lag and Time of Concentration

One parameter that is needed for input to the TR-20 is the time of concentration. The designer may use Lag Equations or graphs instead of calculating the individual overland/sheet flow and shallow concentrated flow separately. <u>The Lag Equation may not be used when the drainage area is greater than five square miles. The minimum length used in the Lag Equation shall be 800 ft.</u> Shorter lengths will result in artificially low lag time. The time-of-concentration is calculated as:

$$Tc = 1.67\ L \qquad (3.8)$$

$S = \left(\frac{1000}{CN}\right) - 10$

where both Tc and Lg are in either hours or minutes.

$\frac{S+10}{1000} = \frac{1}{CN}$

The NRCS Lag Equation is:

$CN = \frac{1000}{S+10}$

$$L = \frac{L_h^{\,0.8}\,(S+1)^{\,0.7}}{1900\ Y^{\,0.5}} \qquad (3.9)$$

where:

L is the Lag, in hours

L_h is the hydraulic length of watershed, in feet

S is the $\left(\frac{1000}{CN}\right) - 10$ (3.10)

Y is the average watershed land slope (perpendicular to flow), in percent

There are several ways to estimate the watershed slope, Y, and they may not agree with each other. The original version of the SHA GISHYDRO used the average slope categories assigned to the soil types. This is probably the weakest approach. The optimal approach is to use the 30-meter resolution digital terrain data that are available for some parts of Maryland in the new version of GISHYDRO that is now available. Slopes estimated with the 90-meter data will not agree with the 30-meter data. Another approach is to digitize the areas between "heavy line" contours, assign average elevations to these enclosed areas and then weight them for the watershed. The "heavy line" contours are those such as 100 feet, 200 feet, etc. Finally, the lengths of the heavy line contours can be measured and the watershed slope estimated as:

$$\text{Watershed Slope} = MN/A_{sf} \qquad (3.11)$$

where

M is the total length of heavy line contours, in feet
N is the contour interval, in feet
A_{sf} is the drainage area in, square feet

The hydraulic L_h length in feet can be estimated from a map or the following relation can be used

$$L_h = 209(A)^{0.6} \qquad (3.12)$$

where A is in acres.

In summary, there are several issues in the use of the empirical lag equation approach that impact the time of concentration and, thereby, the peak discharge of the storm hydrograph. The uncertainties in the value of the curve number discussed in Section 3.3 represent one problem. Estimating the hydraulic length is another. And the value assigned to the slope depends on the estimation approach adopted.

3.4.4 Estimating the Time of Concentration from Flow Path Hydraulics

The time of concentration is the cumulative flow time required for a particle of water to travel overland from the hydraulically most remote point, through the shallow concentrated flow channels, and through the main stream network to the watershed outlet. The time may increase as a consequence of flow through natural storage such as lakes or wetlands or ponding behind culverts or other man-made structures. Estimating the time of concentration by simulating the hydraulics of each flow path component is treated in this section. Because the quantity of flow and, therefore, the hydraulics are different for each storm frequency, it is logical to expect that the time of concentration will be different for a 2-year storm than for a 100-year storm. Recognizing this, the NRCS recommends that 2-year storm conditions be used to estimate the time of concentration.

3.4.5 Overland Flow

At the upper reaches of a watershed, runoff does not concentrate into well-defined flow paths, such as rills, gullies, or swales. Instead it probably flows over the surface at reasonably uniform, shallow depths as sheet flow. It is evident on long, sloping streets during rainstorms. After some distance, sheet flow begins to converge into concentrated flow paths that have depths noticeably greater than that of the shallow sheet flow. The distance from the upper end of the watershed or flow surface to the point where significant concentrated flow begins is termed the sheet-flow, or overland flow, length. For impervious surfaces the sheet-flow length can be several hundred feet. For pervious erodoble surfaces and surfaces with vegetation, concentrated flow will begin after relatively short sheet-flow lengths.

In the upper reaches of a watershed, sheet-flow runoff during the intense part of the storm will flow as a shallow layer with a reasonably constant depth. An equation, referred to as the kinematic wave equation for the equilibrium time, can be developed using Manning's equation with the assumption that the hydraulic radius equals the product of the rainfall intensity and the travel time, i.e., $R_h = i\,T_o$ which is the depth for a wide channel approximation. Using the velocity equation with the travel time (minutes) equal to the time of concentration, Manning's equation becomes: R_h = depth

$$V = \frac{L}{T_t(60)} = \frac{1.49}{n} R_h^{2/3} S^{1/2} = \frac{1.49}{n}\left(\frac{i\,T_t}{60(12)}\right)^{2/3} S^{1/2} \qquad (3.13)$$

In which i [=] in./hr, T_t [=] min, S [=] ft/ft, and L [=] ft. Solving for the travel time yields:

$$T_t = \frac{0.938}{i^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6} \qquad (3.14)$$

Equation 3.14 requires the rainfall intensity i for the time of concentration. Since T_t is not initially known, it is necessary to assume a value of T_t to obtain i from a rainfall IDF curve and then compute T_t. If the initial assumption for T_t is incorrect, then a new estimate of i is obtained from the IDF curve using the computed value of T_t. The iterative process should be repeated until the value of T_t does not change. Generally, only one or two iterations are required.

To bypass the necessity to solve Equation 3.14 iteratively, Welle and Woodward (1986) assumed a power-model relationship between rainfall intensity and rainfall duration. Using a return period of two years, they substituted the 2-year, 24-hour rainfall depth for the rainfall intensity i and derived the following alternative model for Equation 3.14:

n_o

$$T_t = \frac{0.42}{P_2^{0.5}}\left(\frac{nL}{S^{0.5}}\right)^{0.8} \tag{3.15}$$

in which L is the flow length (ft), S is the average slope (ft/ft), P_2 is the 2-yr. 24-hr rainfall depth (in.), and T_t [=] min. Equation 3.15, which is presented in USDA-SCS-TR-55 (1986), has the important advantage that an iterative solution is not required.

In addition to the previously mentioned assumptions, these two kinematic wave equations make the following assumptions: (1) constant rainfall intensity, i; (2) no backwater effects; (3) no storage effects; (4) the discharge is only a function of depth, for example $q = ay^b$; and (5) planar, non-converging flow. These assumptions become less realistic as the slope decreases, the surface roughness increases, or the length of the flow path increases.

n_o

The n values for use with Equations 3.14 and 3.15 are given in Table 3.2 and are for very shallow flow depths, 0.1 inch or so. These values reflect the effects of rain drop impact; drag over plane surfaces; obstacles such as litter, crop ridges, and rocks; and, erosion and transportation of sediment. The 24-hour rainfall depth P_2 for Equation 3.15 can be computed as the product of 24 and a 24-hour intensity obtained from an IDF curve for the 2-year return period.

Table 3.2
Manning's Roughness Coefficients "n_o" for Sheet Flow

Surface Description	n_o
Concrete, Asphalt, bare smooth ground	0.011
Gravel, rough ground	0.02
Cultivated Soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
No-till cultivated (corn – mature	0.40
growth)	0.30
Cultivated (corn-mature growth)	0.50
Cultivated – fallow (no residue)	
Soybeans (full growth)	
Grass:	
Short and sparse	0.15
Dense turf (residential lots & lawns)	0.24
Very dense, tall, rough surface, uncut	0.41
Pasture grasses (grazed)	0.20
Woods:	
Light undergrowth	0.40
Dense undergrowth	0.80

3.4.6 Shallow Concentrated Flow

The shallow concentrated flow portion of the time of concentration is generally derived using Figure 3.1 of the TR-55 manual or similar graphs. The flow velocities are computed using the Manning's equation; n = 0.05 and R = 0.4 for non paved areas; and n = 0.025 and R = 0.2 for paved areas. These selected values of n are those normally expected for channel flow.

Use of the TR-55 graph (and the values of n and R listed above) may underestimate the travel time by overestimating the flow velocity for upper reaches of the shallow concentrated flow path. For shallow depths the hydraulic radius approaches the depth of flow. For depths of flow between the 0.1 feet ± implied for sheet flow and the implied depths of 0.2 feet ± (paved) and 0.4 feet ± (unpaved) for shallow concentrated flow, the designer is not given transitional values of n. In this shallow flow range the n value should represent a higher resistance than that which would be used for channel flow. For example, a wide grass swale with flow depths of less than 0.5 feet and grass 6-inches high or more, the n value may fall between the 0.24 value for sheet flow and the 0.05 value for channel flow. In this case the designer might select an n value of 0.10 which better represents this shallow concentrated flow.

For more insight on the behavior of the Manning n in grassed channels, the reader should examine pages 179-188 in Chow (1959) which discuss the extensive experimental work of W.O. Ree (1949). Ree's experiments showed that Manning roughness coefficients varied with the type, density and height of grass and the product of the velocity and hydraulic radius. Shallow depths with low velocities produced roughness coefficients as high as 0.5.

3.4.7 Open Channel Flow

Estimating the travel time through the main stream requires the user to model the length, slope, roughness and the typical bankfull cross-section associated with the two-year flood. While a good map is assumed to provide a reasonable estimate of the length and slope of the stream, it is very difficult to select the Manning roughness coefficient and the "typical" cross section. Even if one uses stream gaging to determine a roughness coefficient at a point, the coefficient is likely to be different at another discharge or at another point along the stream. The cross section varies significantly along the stream, so it is difficult to determine which is the "typical" section. Errors in these selections can lead to incorrect estimates of the time of concentration and storage conditions and, therefore, lead to peak predictions that are too high or too low. Although several figures from Ragan and Pfefferkorn (1992) illustrating the sensitivity of NRCS-TR-20 to channel parameters are included in this section, it is recommended that the reader review the complete report.

3.4.8 Length and Slope

The Panel recommends that the USGS 1:24,000 quadrangle sheets be the standard for determining the length and slope of streams used to estimate part of the time of concentration. It is recognized that the 1:24,000 scale cannot adequately represent the meanders of many streams and, therefore, the estimated length may be too short and, therefore, the slope too steep. When field investigations indicate that this may be a problem, the user should seek a larger scale map or support changes through additional field investigations or aerial photography.

3.4.9 Manning Roughness Coefficient

Figure 3.6 illustrates the changes in the peak discharges estimated by the TR-20 in

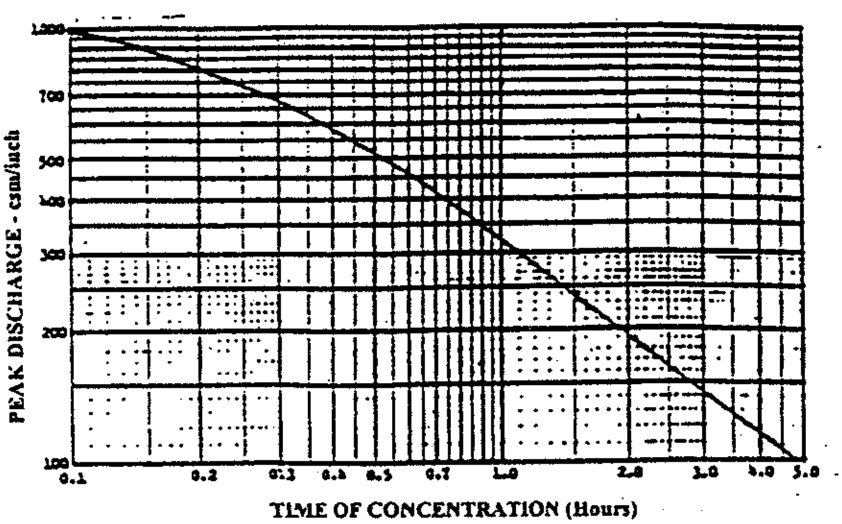


FIGURE 3.6
PEAK DISCHARGE IN CSM PER INCH OF RUNOFF VS. TIME OF CONCENTRATION FOR A 24-HOUR TYPE II STORM DISTRIBUTIONS

response to a 24-hour, Type II synthetic storm as a function of the time of concentration. Suppose the Curve Number of a 2.0 square mile watershed is such that the volume of runoff for a storm is 1.5 inches. The time of concentration is set by the time of travel down the main channel that is 12,000 feet long, has a hydraulic radius of 1.5 feet and a slope of 0.0075 feet/foot. We will define q* as the discharge in cubic feet per second per square mile per inch of runoff found from Figure 3.6. The change in the peak discharge, Q_p, estimated by the TR-20 as the Manning roughness coefficient of the main channel is changed is shown by Table 3.3.

If the channel roughness is actually 0.04, and assuming the other parameters are correct, the peak discharge is 1140 cfs. Underestimating the roughness as 0.03 would result in 1380 cfs, a peak that is 21% higher than the "correct" 1140. Overestimating the roughness as 0.05 would predict a peak of 960 cfs, 16% lower than the "correct" 1140.

The Manning roughness coefficient is a very difficult parameter to estimate and, as illustrated by Table 3.3, it can cause significant changes in the estimates of peak discharges. Even if estimates are based on carefully made field measurement, the "n" would probably change if the measurements are made at a different discharge or at another cross section.

TABLE 3.3
PEAK DISCHARGE VARIATION AS A FUNCTION OF MANNING ROUGHNESS IN THE MAIN STREAM OF AN EXAMPLE WATERSHED

N	Channel Velocity ('/sec)	Time of Concentration (hrs)	q*	Q_p (cfs)
0.03	5.64	0.7	460	1380
0.04	4.23	0.8	380	1140
0.05	3.38	1.0	320	960

A study conducted by the US Army Corps of Engineers Hydrologic Engineeering Center (USACE-HEC, 1986) explored the question of uncertainty in roughness coefficient estimates by asking their staff and training course participants to estimate roughness coefficients for several natural streams given photographs and descriptions of the streams. This effort found that the estimates by the participants were approximately log normally distributed with a standard deviation given by the equation

$$SD = n(e^{(0.582+.10\ \ln(n))2} - 1)^{0.5} \qquad (3.16)$$

The equation indicates that an average estimate of n = 0.04 has a standard deviation of 0.011. Thus, if the average estimate of a group of experienced designers is n = 0.04, we can anticipate that their estimates will scatter, with approximately 68% of their predictions being between n = 0.029 and n = 0.051. The consequences of different roughness estimates are further illustrated by Figure 3.7 where the peaks vary between 7941 cfs and 9872 cfs. Figure 3.7 comes from a study conducted in the Anacostia watershed by Ragan and Pfefferkorn (1992).

A number of tables list Manning roughness coefficients for different types of man-made and natural channels. The table presented by Chow (1959) in his Chapter V is an excellent source. Chow points out that these values should be adjusted to reflect local conditions such as channel irregularity, alignment, silting and scouring, obstructions, meandering, suspended material and bed load. These and other corrections are discussed in considerable detail in Chow's Chapter V.

Still another problem arises when field investigations indicate that the roughness varies significantly from one section of the stream to another. In these instances it may be necessary to break the stream into segments and compute the flow time for each.

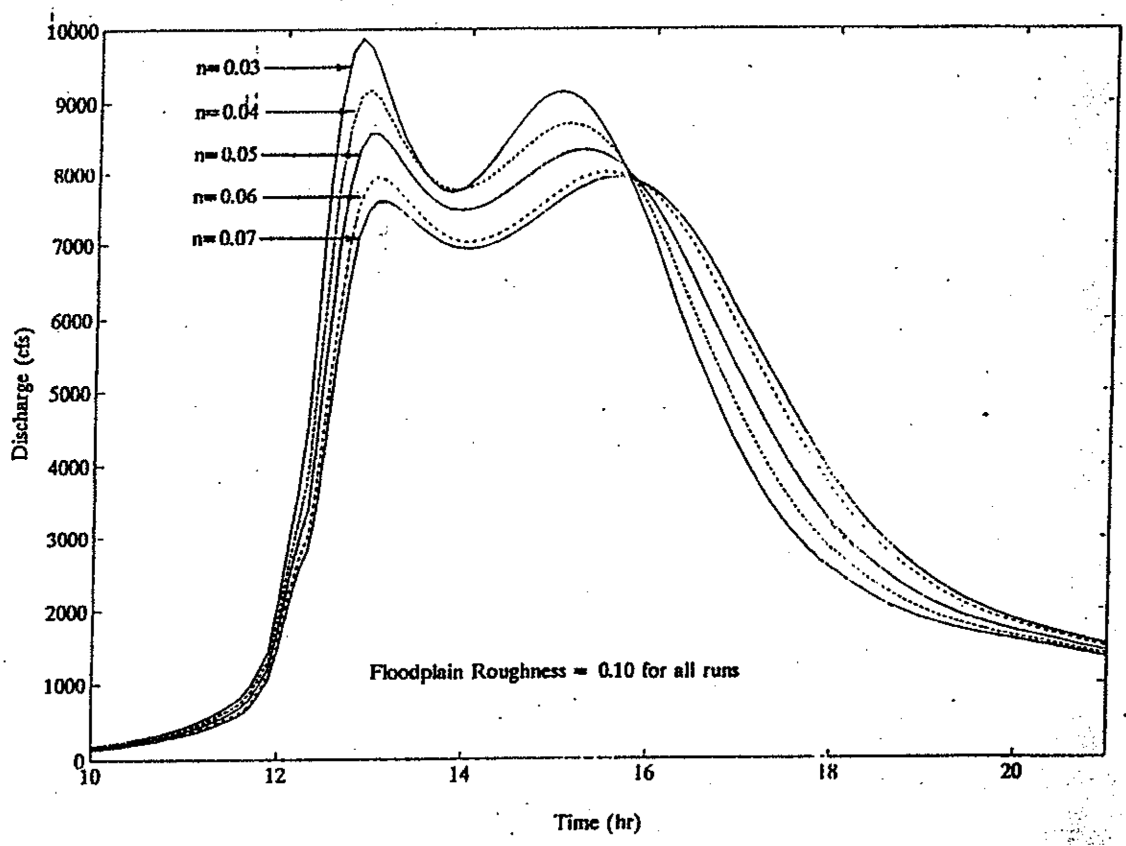


FIGURE 3.7
HYDROGRAPHS FOR A RANGE OF IN-BANK CHANNEL ROUGHNESS COEFFICIENTS

3.4.10 Bank Full Cross Section

Another factor contributing to changes in the peak flow prediction is the "typical" bank full cross section selected to determine the velocity and, therefore, one part of the time of concentration. For example, selection of a section near the outlet of the watershed may result in a channel velocity that is significantly different from that predicted by the use of a section chosen from a point about half-way up the stream. The larger the hydraulic radius, the higher the velocity and the shorter the time of concentration. Because the section varies from point to point along the channel, it is quite difficult to decide which is the representative section. Thus, the user must recognize the importance of the representative section when calibrating against the USGS methods.

If it is not practical to survey bank full cross sections, an alternative is to use regional regression equations that relate the bank full depth, width and cross sectional area to the area of the upstream drainage basin. Figure 3.8 showing preliminary results obtained from a current SHA study, is an example of these regional regression equations. Dunne and Leopold (1978) present a similar set of relations and Rosgen (1996) includes several examples of findings similar to Figure 3.8.

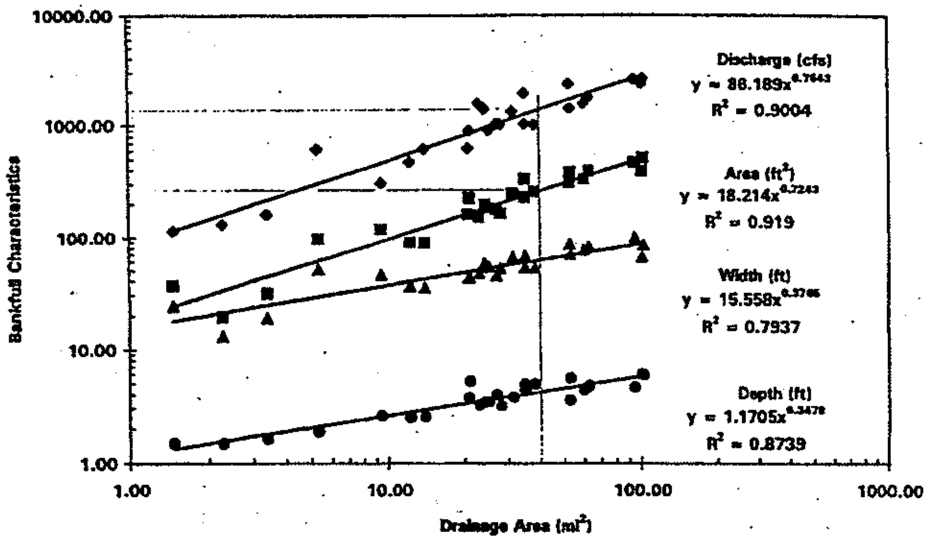


FIGURE 3.8
BANKFULL CHARACTERISTICE FOR SELECTED USGS SITES IN THE MARYLAND PIEDMONT

Figures 3.9 and 3.10 indicate that time of concentration differences associated with cross-sections defined through the use of regional regression equations, as opposed to surveyed cross sections, may be less than the differences associated with different roughness coefficients. In Figure 3.9, the Siebach (1987) S-curve (time-area curve) defining time of concentration used travel times computed with surveyed, bank full cross sections. The Dunne and Leopold curve used cross sections that were defined with their regional regression equations that estimated bank full width, area and depth as a function of the watershed area. The S-curves used to estimate the time for concentration in Figure 3.10 used surveyed cross sections with the Manning roughness coefficient being varied. The two figures indicate that errors in the Manning roughness coefficient can cause larger errors in the time of concentration than the changes associated with differences between surveyed and regression defined bank full cross sections. This is to be expected because the channel velocity varies linearly with the roughness coefficient and with the 0.667 power of the hydraulic radius.

As can be seen from the above discussion, accurate estimates of the time of concentration are difficult to obtain because of the large uncertainty in the parameters used to compute the time of concentration. Thus, there needs to be an alternative approach that can serve to define upper and lower bounds for time of concentration. Regression models that estimate time of concentration based on watershed characteristics provide an attractive approach. Limited tests with a model developed by W.O. Thomas, Jr. and described in Appendix 5 have been very encouraging. The Panel recommends that designers be

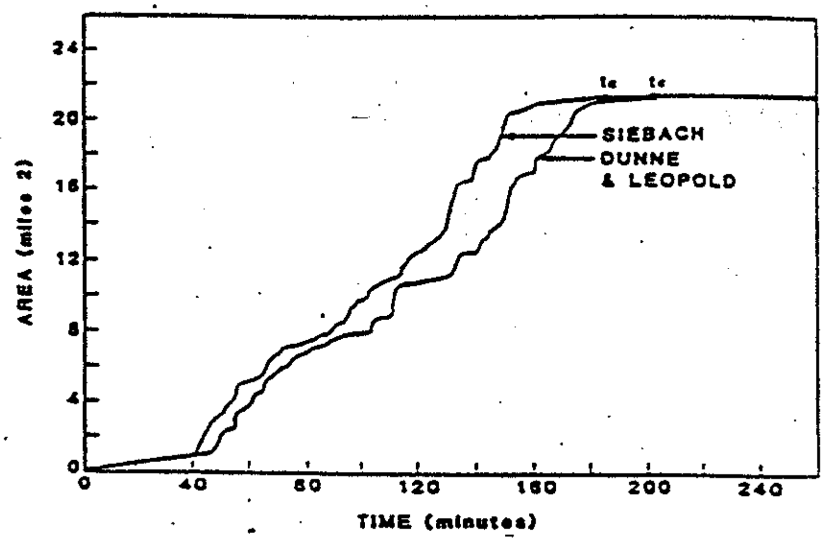


FIGURE 3.9
TIME-AREA CURVES USING SURVEYED AND REGRESSION
EQUATION DEFINED IN-BANK CROSS SECTIONS
(n = 0.04)

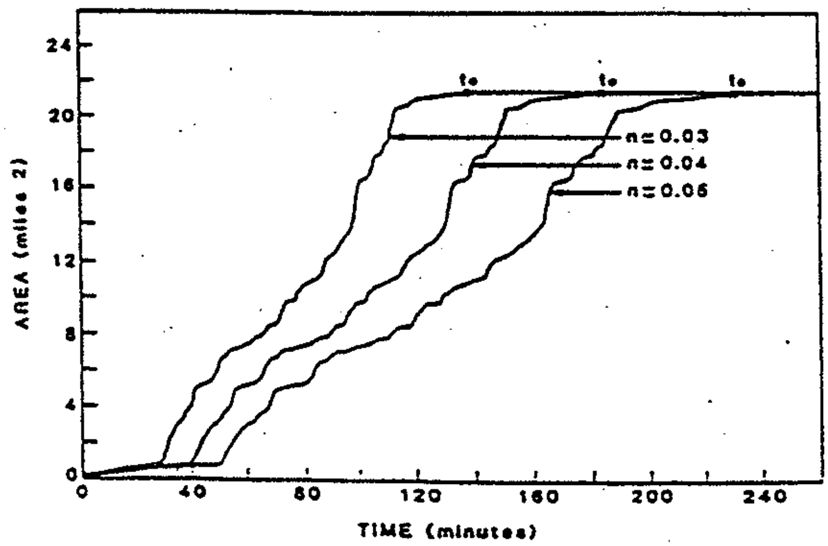


FIGURE 3.10

SECTIONS AND INDICATED MANNING ROUGHNESS COEFFICIENTS

encouraged to apply the Thomas model in their studies to determine realistic bounds for the time of concentration. The Panel also recommends that a regional regression research project described in Chapter 5 be given one of the highest priorities.

3.5 SUBDIVIDING INTO SUB-WATERSHEDS AND ROUTING

If the watershed is large or has tributary drainage areas that have land/soil complexes that differ from each other, the watershed may be divided into sub-watersheds. In this subwatershed are then routed through the stream network to the outlet of the overall watershed. Even if the watershed is not especially large or heterogeneous, calibrating to the USGS methods may require subdivision in order to model the attenuation provided by the flood plain. An example of this situation is presented in Appendix 4.

No "magic number" exists to define a small versus a large watershed. A watershed might be considered small if the land phase processes - overland and shallow confined flow - dominate the peak discharge and the shape of the runoff hydrograph. A watershed might be large if the translation and storage provided by the stream network provides significant attenuation or modification to the storm hydrograph. A large watershed by this definition will require subdividing and flood routing.

3.5.1 How Many Sub-watersheds

Part of the decision controlling the subdivision of the watershed is tied to the heterogeneous nature of the watershed. Other aspects of the decisions controlling the subdivision of the watershed and the location or spacing of the typical cross sections along the stream are inter-related with the selection of the main time increment. The NRCS-TR-20 Manual states that the main time increment "should be about 0.1 or 0.2 of the shortest time of concentration ..... generally not smaller than 0.1 hour."

The current TR-20 uses the "Modified Att-Kin" method to simulate the role of the channel network by routing sub-watershed hydrographs from one cross section to another. Selecting cross sections that are too closely spaced, "kinematic translation" will result, in which the hydrographs are simply off-set in time with no attenuation. To avoid this problem, Appendix H of NRCS-TR-20 states, "The travel time (between cross sections) should be greater than one half of the main time increment."

There does not appear to be a "rule" that one can apply to confirm that there is an optimal number of subdivisions for a watershed of a given size or set of topographic characteristics. Ragan and Pfefferkorn (1992) broke the 21-square-mile Northwest Branch of the Anacostia River into 26, 13, 4 and 2 sub-watersheds and input a 100-year, 24-hour Type II NRCS design storm. The resulting hydrographs are shown in Figure 3.11. Using 26 cross sections results in a higher peak than using 13 sections or 2 sections, but four sections produces a peak that is higher than the others. Designers must calibrate against the USGS methods to ensure that their subdividing approach is appropriate.

3.5.2 The Representative Routing Section

Bank full and over-bank cross sections often show tremendous variations along a stream reach. Selecting the representative section to use to develop the stage-area-discharge relation for the routing reach is a very difficult task. If the flood plain is too narrow, the peak will be too high and if it is too wide, the peak will be subject to too much attenuation. Figure 3.12, from Ragan and Pfefferkorn (1992), shows four representative

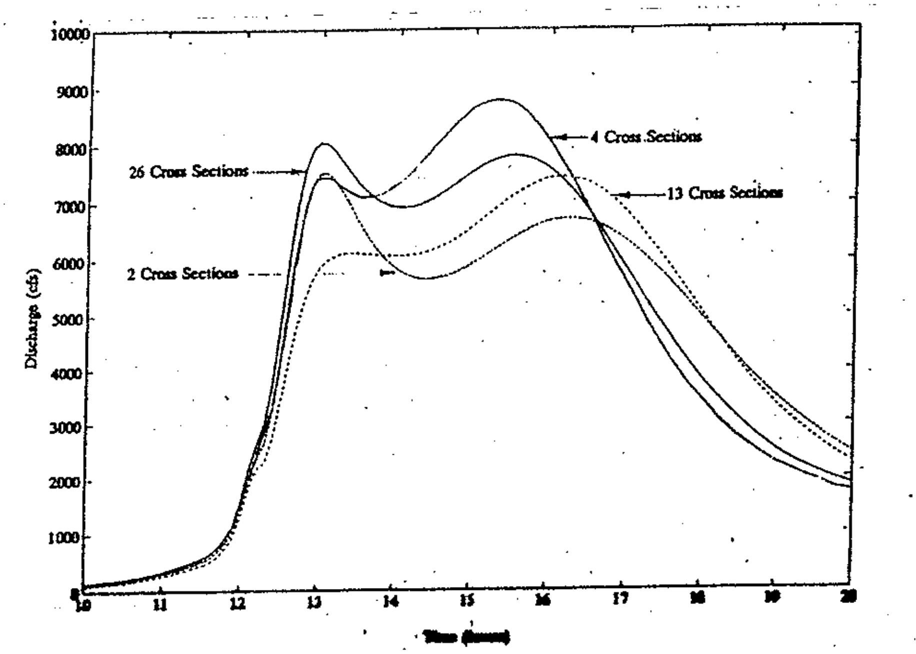


FIGURE 3.11
HYDROGRAPHS WITH STREAM NETWORK ATTENUATION DEFINED WITH INDICATED NUMBER OF REPRESENTATIVE CROSS SECTIONS

cross sections and Figure 3.13 illustrates the hydrographs that can be produced by routing through each of these cross sections.

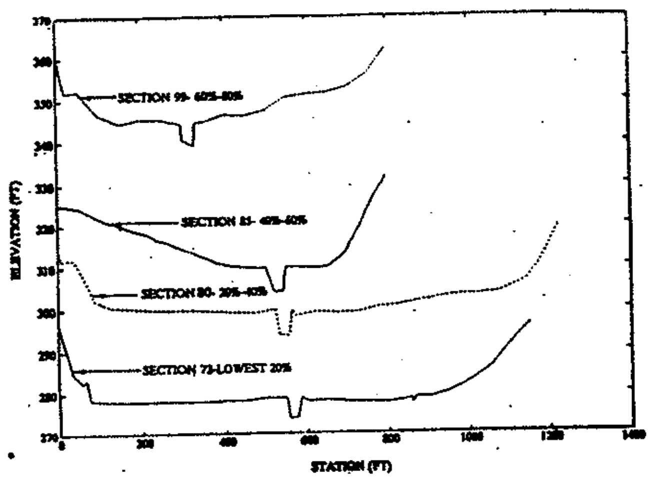


FIGURE 3.12
REPRESENTATIVE CROSS SECTIONS AT INDICATED LOCATIONS ALONG MAIN STREAM

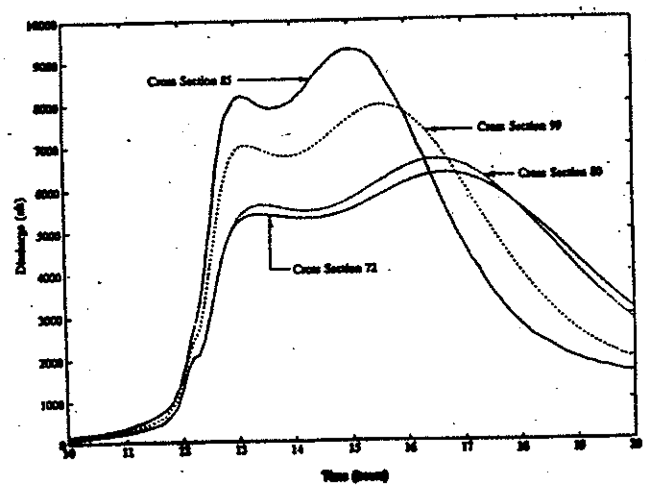


FIGURE 3.13
HYDROGRAPHS GENERATED USING A SINGLE CROSS SECTION TO REPRESENT THE STREAM NETWORK

Another alternative to the use of field surveys to define typical cross sections is to digitize along transects drawn on maps, perpendicular to the stream. In many areas, 1:2,400 or similar scale maps are available. Transects on these maps can provide an excellent base for routing sections. The bank full portion of the section is generated by the regression equations discussed in Section 3.5. As shown by Figure 3.14, even a 1:24,000 scale map can be used in areas where there is good topographic definition.

Figure 3.15 shows storm hydrographs generated with 26 surveyed sections and synthetic sections generated from transects drawn on 1:2,400 and 1:24,000 maps.

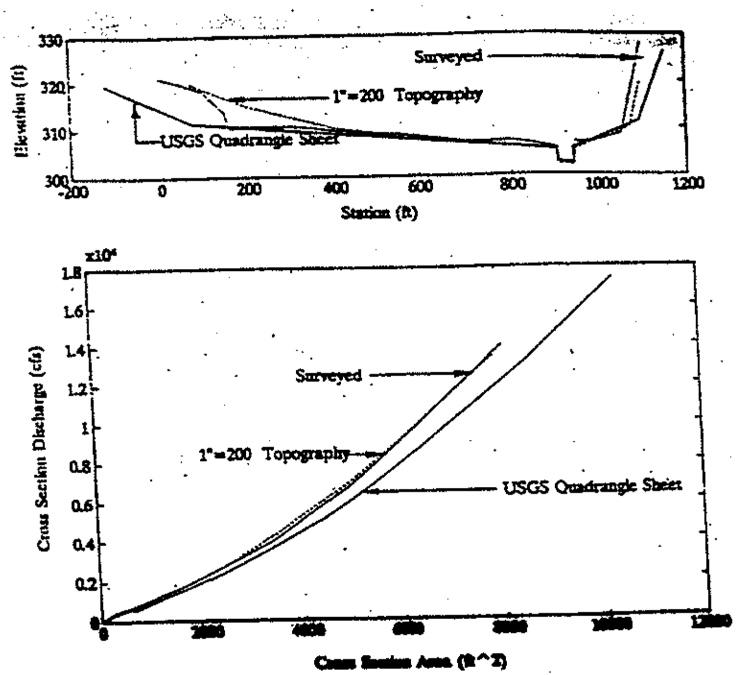


FIGURE 3.14
DISCHARGE-AREA CURVES FOR SURVEYED AND SYNTHETIC CONTOUR DEFINED CROSS SECTIONS

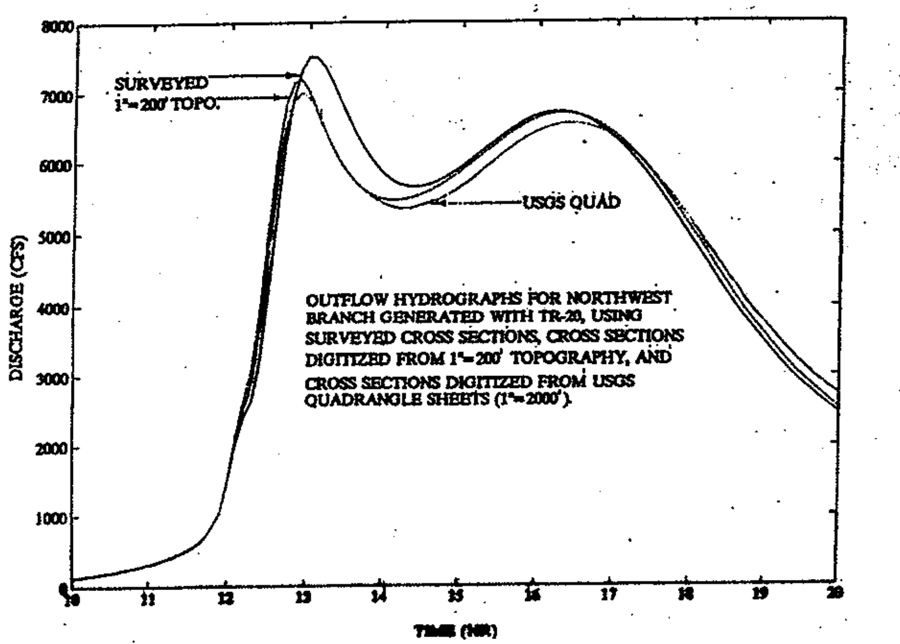


FIGURE 3.15
HYDROGRAPHS USING SURVEYED AND CONTOUR-DEDFINED CROSS SECTIONS

3.5.3 Manning Roughness Coefficients

Assume that we are confident that the "correct" representative sections for the flood routing component of the TR-20 have been chosen. We are now faced with the problem of selecting the Manning roughness coefficients required for the stage-area-discharge relations. Section 3.5 discussed the difficulties associated with the definition of the in-bank roughness and illustrated the impact of the roughness on the time of concentration. Figure 3.7 in that section showed the impact of different bank full roughness coefficients on the storm hydrograph. Twenty-six surveyed cross sections were used in that example where the overbank roughness was 0.1 in each section.

Estimating the over-bank roughness involves more uncertainty than the bank full coefficient because of the extremely limited amount of data collected for flow in a flood plain. Chow's (1959) table suggests flood plain Manning roughness coefficients that range from 0.02 to 0.20. Figure 3.16 shows the impact of selecting over-bank roughness coefficients ranging from 0.1 to 0.2 while holding the bank full roughness at 0.05.

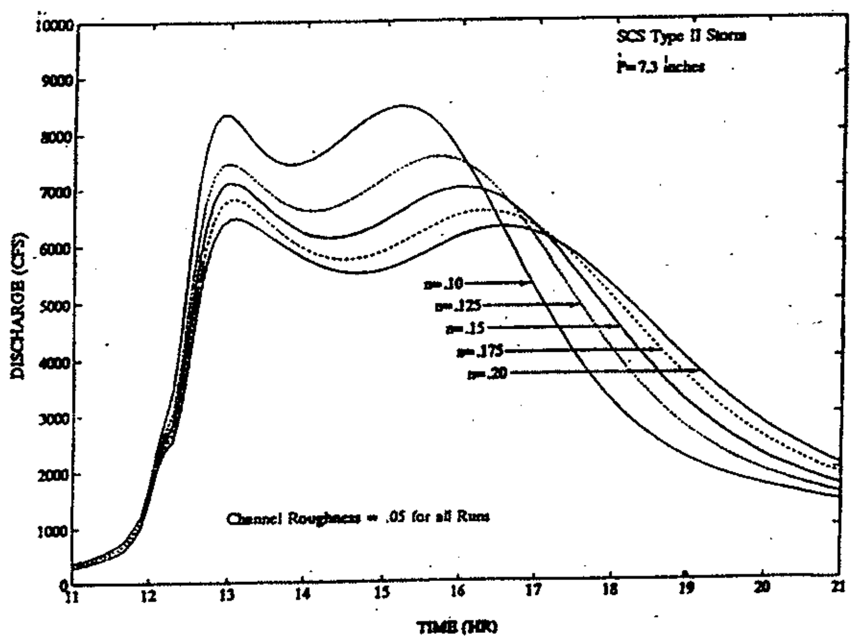


FIGURE 3.16
HYDROGRAPHS FOR A RANGE OF
OVERBANK ROUGHNESS COEFFICIENTS

The impact of changing the over-bank roughness or, for that matter, any parameter in the representative cross section, is a function of the length of the routing section. Figure 3.17 illustrates this situation. As the length of the routing reach increases, the consequences of the details of the routing section become greater.

3.6 THE DESIGN STORM

The NRCS-TR-20 requires that the user define the total volume of rainfall, the duration of the storm, and time distribution of the rainfall intensities within the storm. The usual

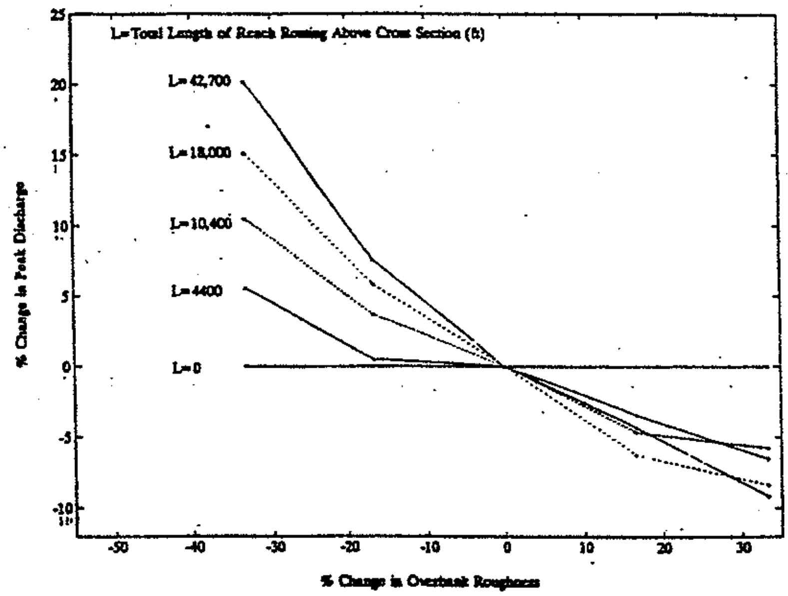


FIGURE 3.17
PERCENT CHANGE IN PEAK DISCHARGE AS A FUNCTIONS OF CHANNEL LENGTH AND THE PERCENT CHANGE IN THE MANNING ROUGHNESS COEFFIEICNT

approach is to accept one of the "standard" design storms such as the NRCS Type II, 24-hour storm. Rainfall intensities within the design storm then are convoluted with the dimensionless UHG that has been defined by the watershed area, curve number and time of concentration to produce a storm hydrograph. If the 100-year, 24-hour volume of rainfall is used to define the intensities of the Type II storm, the "design expedient" typically accepts the peak discharge generated by the TR-20 as an estimate of the 100-year frequency peak discharge to be used in design. It must be emphasized that the TR-20 is computing an estimate of the peak discharge caused by a synthetic 100-year storm that is based on rainfall records and not an estimate of the peak discharge based on stream flow records. The two discharges may differ significantly. The Panel's recommended calibration against one of the USGS methods described in Chapters II and IV of this report is intended to reconcile some of the disagreement.

Decisions that define the storm input are very important because the performance of the TR-20 is very sensitive to the structure of the rainfall input. Figure 3.18 (from Ragan and Pfefferkorn (1992)) provides an example of the sensitivity of NRCS-TR-20 to storm input structure. An 8.5-inch, 12-hour duration rainfall was used as the input storm volume for the 21.3-square-miles of Anacostia watershed. The TR-20 produced the lower hydrograph when the storm was uniform with an intensity of 0.708 inches per hour for the 12-hour duration. The upper hydrograph resulted when the rainfall intensities were varied in accordance with the center 12-hours of the NRCS Type II design storm.

Segments of the NRCS Type II, 24-hour design storm should be used to develop synthetic storms having different durations. When developing a synthetic storm having a

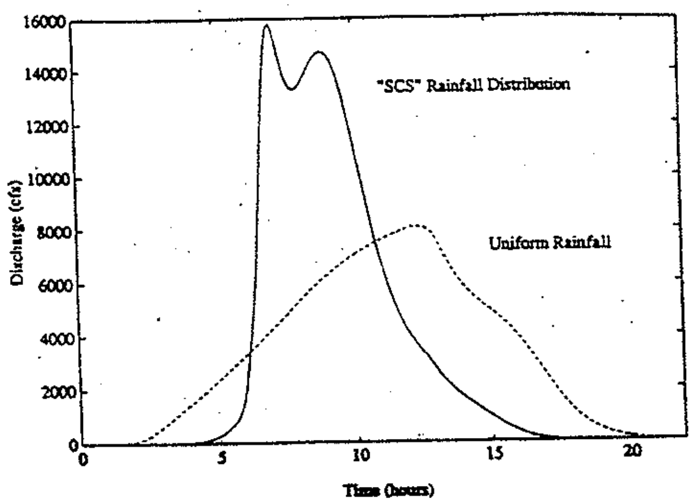


FIGURE 3.18
HYDROGRAPHS PRODUCED BY NRCS-TR-20 USING
12-HOUR STORMS OF 8.5 INCHES AD INPUT

duration that is shorter than 24 hours, one should use the period that is distributed equally on each side of the steepest portion of the Type II mass curve. For example, a four-hour storm would be based on the dimensionless intensities between T = 9.8 and T= 13.8 hours on the Type II distribution. Figure 3.19 illustrates the portions of the Type II storm used to generate the storms having the indicated durations of Figure 3.20. Each storm matches the IDF curves used in central Maryland.

Design storms having similar structures, but different durations, produce significantly different hydrographs and peak discharges when input to the TR-20. This behavior is illustrated by Figure 3.21 from Ragan and Pfefferkorn (1992)). As a consequence, there is uncertainty as to what storm duration should be used. The traditional practice in Maryland has been to use the 24-hour Type II storm in all cases. Some writers recommend a duration "at least equal to the time of concentration." For example, the NRCS Emergency Spillway Hydrograph method summarized by Viessman, Lewis, and Knapp (1989) uses a length of storm of 6-hour duration or t_c, whichever is greater.

Experiments conducted by the Panel demonstrate that the 25-, 50-, and 100-year flood peaks predicted by the TR-20 model, using the 24-hour design storm duration and appropriate estimates of watershed parameters, agree reasonably well with the flood peaks predicted by the USGS regression equations. However, such is not the case for more frequent storm events. The Panel's experiments indicate that the 2-, 5-, and 10-year flood peaks generated by the TR-20 model using the 24-hour design storm duration are often significantly higher than those predicted by the USGS regression equations. When shorter duration design storms, based upon center-peaking period of the NRCS Type II storm and meeting all of the conditions imposed by the Maryland IDF curve, are used for the 2-, 5-, and 10- year flood peaks, the TR-20 and USGS estimates may be brought into close agreement. Obviously, more research is warranted. In the interim, the 10-, 5-, and 2-year storm events should be derived using either the 6-hour or 12-hour design storm duration. For watersheds having a total time of concentration of less than six hours, the

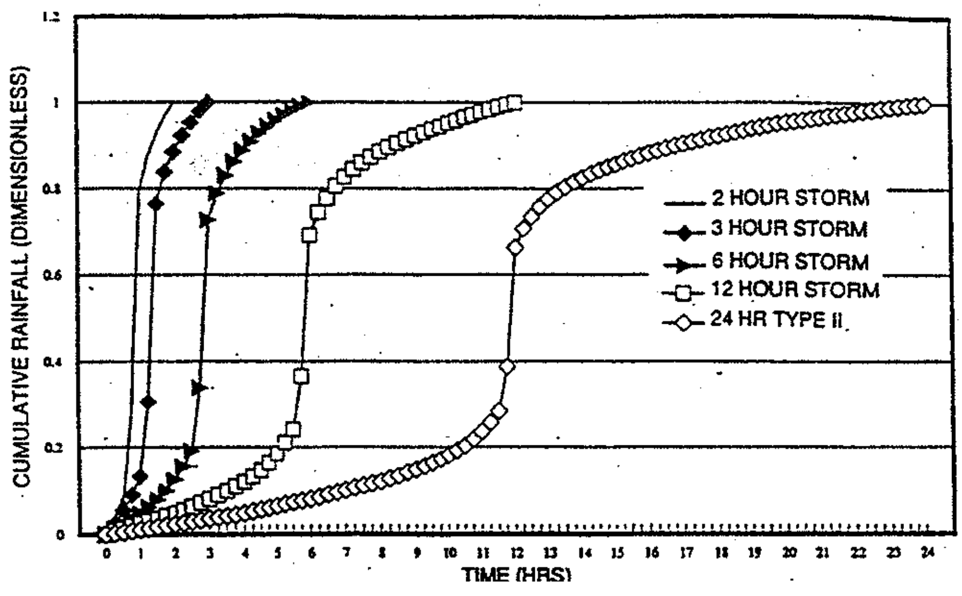


FIGURE 3.19
CUMULATIVE DIMENSIONLESS DEPTHS FOR DESIGN STORMS OF INDICATED DURATIONS

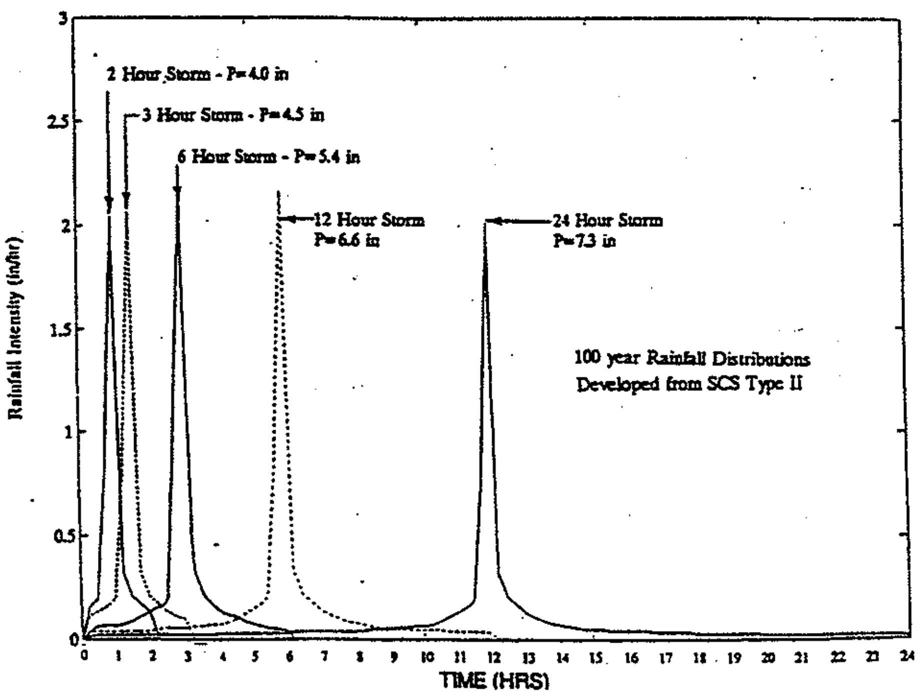


FIGURE 3.20
INTENSITIES FOR DESIGN STORMS OF INDICATED DURATIONS

6-hour design storm duration is appropriate. For watersheds having a total time of concentration greater than six hours, the 12-hour design storm duration is appropriate.

The volumes of rainfall of a given frequency and duration vary considerably across Maryland. As illustrated by the last map of Appendix 6, the volume of precipitation in a 100-year 24-hour storm varies from 5.7 inches in western Maryland to 8.1 inches in the vicinity of Ocean City. The rainfall volumes that are to be used to define the intensities

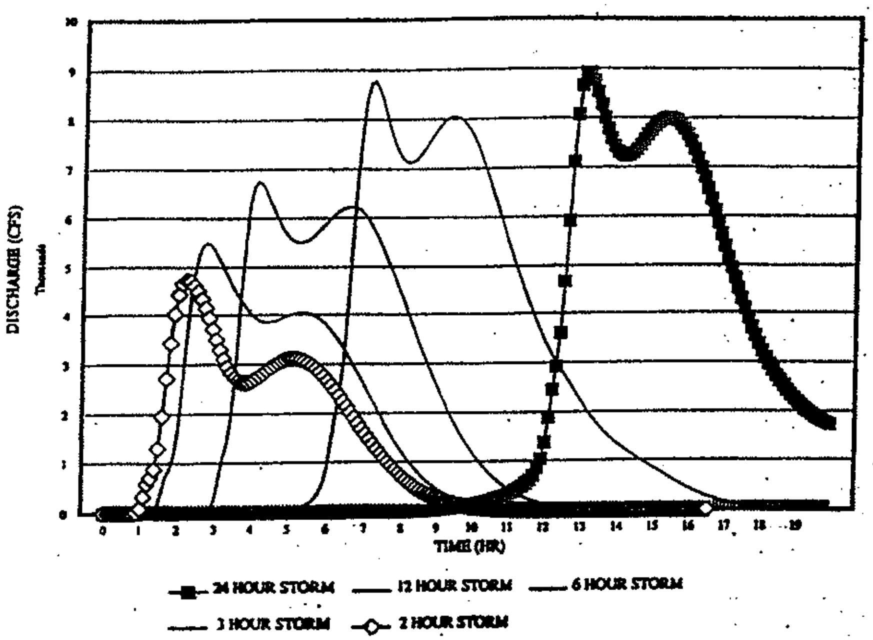
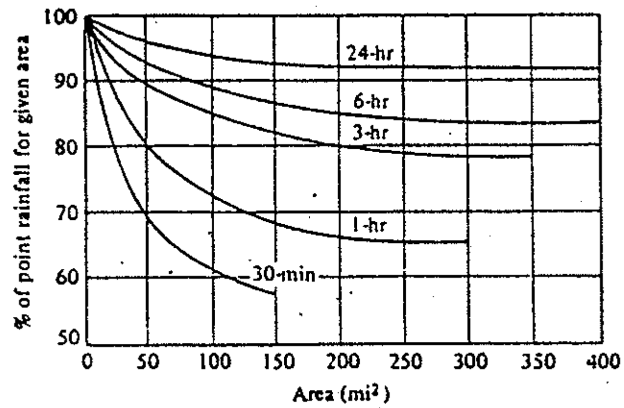


FIGURE 3.21
HYDROGRAPHS USING 100-YEAR DESIGN STORMS OF INDICATED DURATIONS

of the TR-20 input design storms are to be interpolated for the watershed location from the maps of Appendix 6.

There appears to be a problem in applying TR-20 models in western Maryland. Peak flood flows predicted by TR-20 are often significantly higher than the estimates provided by the USGS regression equations.. Many of the USGS stream gages have operated in that region for more than 70 years. These gages simply have not measured peak flows as high as those measured in the central portion of the State. Analysis of eleven USGS gages in the Maryland Appalachian Plateau and Valley and Ridge provinces demonstrates that the observed maximum flows range between 83 and 300 cubic feet per second per square mile, with an average of 167 cubic feet per second per square mile. The minimum length of record is 17 years and the maximum length is 50 years. The average watershed area is 23 square miles. The same analysis conducted on six gages in the Maryland Piedmont indicates that the maximum flows vary from 319 to 780 cubic feet per second per square mile, with an average of 452 cubic feet per second per square mile. The minimum length of record is 12 years and the maximum length is 60 years. The average watershed area is 22.3 square miles. Based upon watershed characteristics alone, one would expect the steep mountain areas in western Maryland would yield higher peak flows than the Piedmont. However, indications are that flood producing rainfalls in western Maryland may be shorter in duration than those further east. More specific research is warranted in this regard. Therefore, if the flood estimates using the 24-hour storm do not lie between the regression estimate and the upper 68% limit, the analyst should use the 12-hour storm for the 25-, 50- and 100-year events and the 6-hour storm for the 2-, 5- and 10-year events.

The rainfall intensities of IDF curves and the volumes of Appendix 6 are from point measurements. The typical storm is spatially distributed with a center area having a maximum rainfall and a decay in intensities and volumes away from the storm center. The Panel recommends that the design storm rainfall intensities used as TR-20 inputs be reduced as a function of the storm duration and drainage area in accordance with Figure 3.22.



FIIGURE 3.22
DEPTH-AREA CURVES FOR USE WITH IDF VALUES
(From USWB-TP-40)

IV. CALIBRATION OF NRCS-TR-20 WITH USGS METHODS

4.1 OVERVIEW

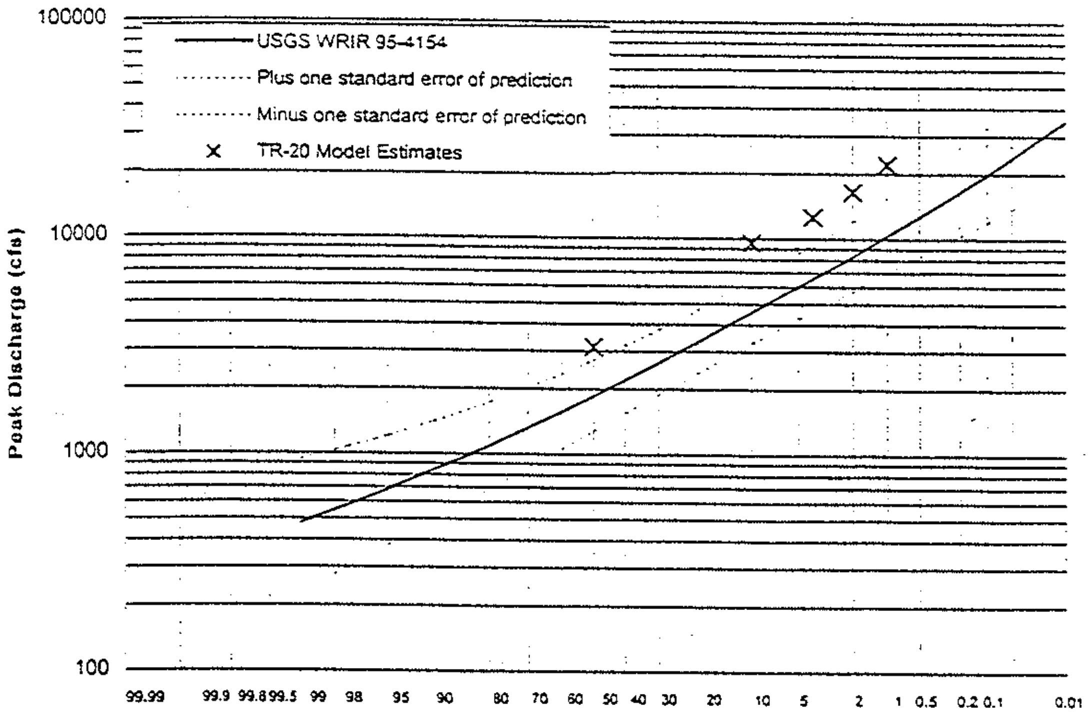
The hydrologic analysis of SHA bridges and culverts must examine the behavior of the structure and local stream conditions under both existing and ultimate development watershed conditions. Because two land cover and flow path conditions are involved, the basis for these hydrologic analyses must be a deterministic model that can simulate the major runoff processes that occur during and after the storm. The recommended approach is to first select field and map defined parameters that describe the runoff processes for existing watershed conditions. After the designer is satisfied that the model provides a realistic representation of the existing watershed conditions, the impact of ultimate conditions can be simulated by adjusting the input parameters to reflect the land cover and flow path modifications that are planned.

The NRCS-TR-20 is a well established deterministic model that has an extensive history of use in Maryland. However, the TR-20, as with all deterministic models, is sensitive the values of the input parameters. In most instances, the input parameters are difficult to determine. As discussed earlier, the TR-20 has a tendency to over predict at all return periods. This behavior is illustrated by Figure 4.1. The Panel has concluded that this tendency to over predict can be overcome through calibration. Thus, in order to provide the designer with confidence that the input parameters selected are representative of the existing watershed conditions, the Panel recommends that the TR-20 peak discharges for existing watershed conditions be calibrated against one of the USGS methods described in Chapter II. The TR-20 will be accepted as calibrated if the peak discharges are in the window between the USGS best estimate and an upper limit of plus plus error of prediction as defined by the **Hodges Tasker (1995)**. If the watershed conditions are such that a calibration cannot be achieved in accordance with the procedures defined below, the designer will explain why the calibration cannot be accomplished and what approach will be followed to generate the required flows.

The Panel emphasizes that all input parameters to the TR-20 must be consistent with accepted hydrologic practice. Thus, all TR-20 computations will be supported by documentation that lists the values of (1) category curve numbers; (2) the quantities used to define the time of concentration, and (3) the watershed segmentation and stage-area-discharge relations if routing is involved. This documentation will explain the decision making process behind the selection of each input quantity.

The following sections examine the types of errors that may occur in the definition of inputs to the TR-20 and the procedures to follow in making adjustments to achieve calibration. Because so few watersheds of concern to the SHA are located at a USGS gage or at a point that will allow gage transposition, the emphasis of this chapter is on calibration against the USGS regression equations. Figure 4.1 illustrates the situation that often occurs where the TR-20 model estimates are greater than the USGS regression estimates. The TR-20 estimates in Figure 4.1 are actually greater than the USGS regression estimates plus one standard error of prediction. The objective of the calibration of the TR-20 model is to modify the model input parameters to produce estimates of the flood discharges that are between the regression line and the upper limit represented by plus one standard error of prediction. This chapter provides guidance on modifying the model input parameters.

Comparison of USGS Regression Estimates to TR-20 Model Estimates



Annual Chance of Exceeding Discharge (percent)

Comparison of USGS Regression Estimates to TR-20 Model Estimates

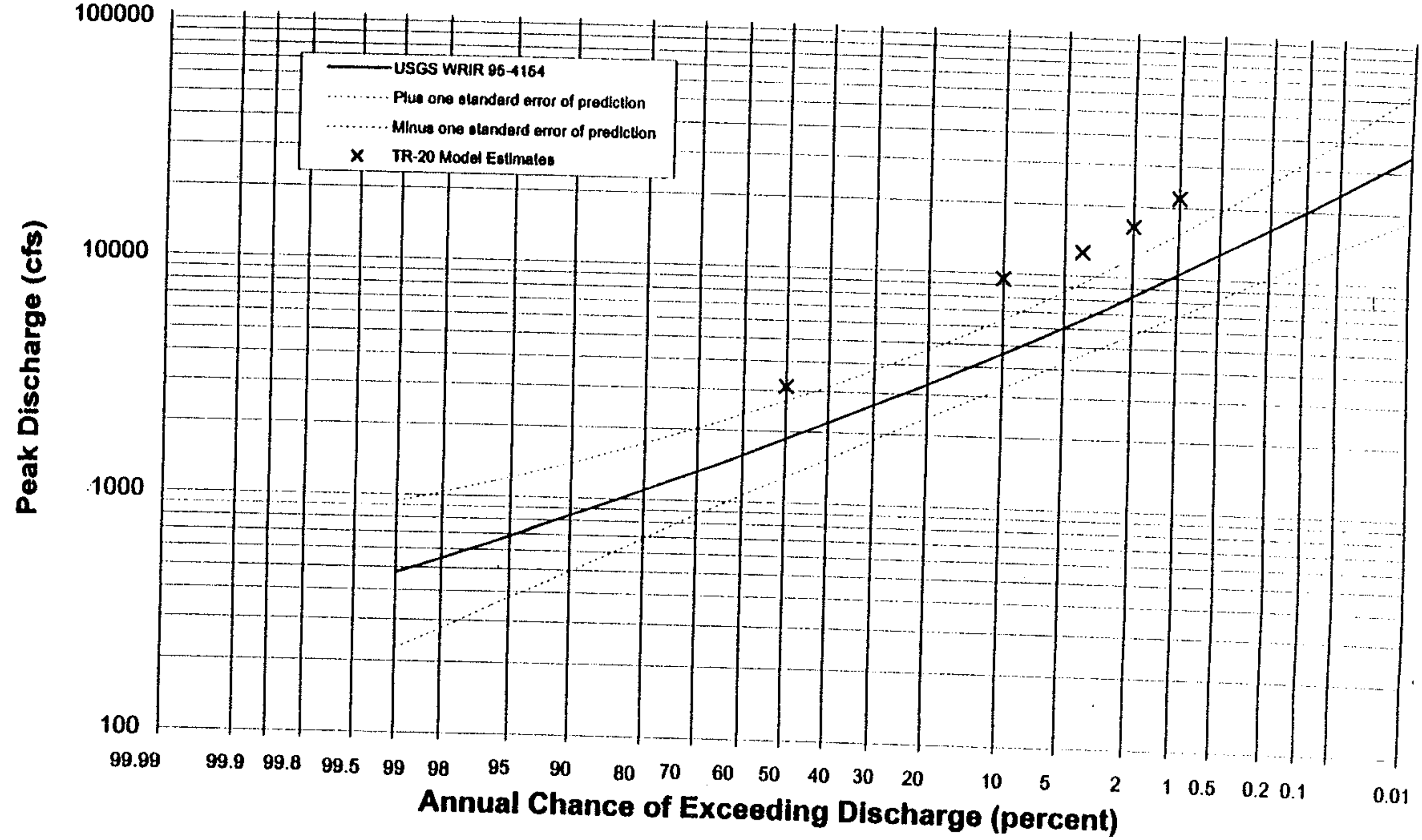


Figure 4.1

4.2 SIZE AND CHARACTERISTICS OF THE WATERSHED

For watersheds greater than about 300 square miles in size, TR-20 models are not recommended. The NRCS developed the dimensionless UHG from data collected on relatively small watersheds. On most large watersheds, significant peak flow attenuation caused by the channel network may not be incorporated into the NRCS dimensionless UHG. Thus, the validity of TR-20 applications on large watersheds is questionable. Moreover, the effects of ultimate land use conditions on peak flows generally are muted on very large watersheds.

For watersheds smaller than about 300 square miles, the first step is to calibrate each sub-basin as an individual unit. Thereafter, the calibrated sub-basins should be incorporated into a TR-20 model of the entire watershed. Only then should the TR-20 model of the entire watershed be used as the basis for any iterations needed to adjust the routing parameters. This approach is illustrated in Appendix 4.

note

Before any calibration is attempted, care should be exercised to ensure that the characteristics of the watershed are within the bounds of the USGS sample used to develop the regression equations. Calibration will not be valid if there are ponds, wetlands storage, or structures that significantly change the natural flow characteristics of the watershed. If existing urbanization exceeds approximately 15%, calibration still may be possible provided the approach recommended in section 4.6 is followed.

4.3 UNDERSTANDING ERRORS

The construction of any deterministic model involves the selection of certain input values. The selection estimate or measurement of any value includes the possibility of several types of errors. These can be labeled: Random (non-systematic), Systematic (always more or always less), and Cumulative (small systematic errors that add up to large systematic errors). Each variable entered in the TR-20 model can have one or more of these errors. As with the regional equations, the selected value for any TR-20 input variable represents the "best educated guess." Unfortunately, unlike the standard error of the regional equation, the standard error of TR-20 input variables is unknown. However, with experience, designers can estimate the range of reasonable values and confine their choices to those within this range. For example, a Manning's roughness coefficient for a natural stream channel might be 0.05. Estimates that are 0.07 and 0.03 still appear to be within a reasonable range while 0.1 and 0.20 are not. In general, the designer should select the variables with systematic errors as the most likely values to calibrate or adjust.

The TR-20 input variables and a description of the types of errors that are inherent in their estimate follows, along with recommendations regarding adjustments for calibration to more closely simulate the results of the USGS regional equation.

4.3.1 **Drainage Area** (square miles)

Assuming that both the map used to delineate the drainage area and the measuring devices are accurate, the estimation of the drainage area includes a random error. When digitizing areas, the designer should check for random errors by ensuring that the sum of all sub-areas equals the digitized total area. Adjusting the size of a drainage area is seldom justified unless the watershed includes Karst topography or non-contributing drainage areas. In some cases, depressional areas will not contribute to watershed runoff at the 2-year event and may contribute at the 50- or 100-year event. Careful evaluation of depressional areas normally is required.

4.3.2 Runoff Curve Number

The error in selection of an RCN value is random. The NRCS manuals (TR-55) show the acceptable range of values for each land cover. Those for croplands and natural ground cover are based on hydrologic conditions such as fair, poor, or good. In cases where one land cover is predominate, a potential for a systematic error exists because of the impact of the selection of one significant value rather than the distribution of small random errors in a varied land cover model.

RCN value(s) can be adjusted to match a measured runoff volume provided that the resulting RCN falls within the logical limits of their respective ARC (Antecedent Runoff Conditions) limits. Consideration should be given to the use of ARC = 1 for the frequent events (1- to 5-year storms). The reasoning is that these small storms are usually the result of short duration summer thunderstorms without the preceding ground wetting light rain. Greater storms (10-year and larger) are generally related to cyclonic storms of 12- to 48-hour duration where several hours of rain preceded that of the flood producing rain intensities.

4.3.3 Time-of-Concentration (overland/sheet flow component)

The application of several methods to calculate the overland component to the time-of-concentration can contain both random and systematic errors. This overland flow variable, by experience, has shown to be the most difficult to quantify of any of the input variables. The potential for a systematic error is high, which may be related to the experience or application techniques of the designer. This is one of the variables that should be examined for adjustment, especially if the sub-basins are small and the time-of-concentration is short.

4.3.4 Time-of-Concentration (shallow concentrated flow component)

Calculation of this portion of the Tc often will generate a systematic error that will result in underestimation of the flow time. The shallow concentrated flow portion of the time-of concentration is generally derived using Figure 3.1 of the TR-55 manual or similar graphs. The flow velocities are computed using the Manning equation within = 0.05 and R = 0.4 for non-paved areas; and n = 0.025 and R = 0.2 for paved areas. These selected values of n are those normally expected for channel flow.

Use of the TR-55 (Figure 3.1) graph (and the values of n and R listed above) may underestimate the travel time by overestimating the flow velocity for upper reaches of the shallow concentrated flow path. For shallow depth, the hydraulic radius approaches the depth of flow. For depths of flow between the upper limits of sheet flow and the implied depths of 0.2 feet ± (paved) and 0.4 feet ± (unpaved) for shallow concentrated flow, the designer is not given transitional values of n. In this shallow flow range the n value should represent a higher resistance than that which would be used for channel flow. Consider, for example, for a wide grass swale with flow depths of less than 0.5 feet and grass 6-inches high or more. The n value may fall between the 0.24 value for sheet flow and the 0.05 value for channel flow. In this case the designer might select an n value of 0.10 which better represents this shallow concentrated flow. In unique conditions, the designer can develop a new relationship of velocity to slope for assumed values of n and the hydraulic radius.

4.3.5 Time-of Concentration (channel flow component)

The selection of the channel component of the time-of-concentration can produce a systematic error that will shorten the travel time. This can be attributed to three factors: incorrect estimates of the channel length, the Manning roughness co-efficient and the bank full cross-section.

Measuring the length of channel flow generally involves a scale error. Larger scale maps such as the USGS quad maps at 1:24,000 do not account for all the bends or meanders of a natural stream channel. Using a smaller scale map (1 inch = 200 feet) will help reduce this error, but it will always be systematic. **Adjustments in channel lengths up to 25% when measuring from a USGS 1:24,000 map can be reasonable providing the designer supports the decision in writing.**

A single Manning n value selection to represent channel flow should be higher than an n value used for the channel in a hydraulics model like HEC-2, HEC-RAS, or WSPRO. This single n value must account for all hydraulic losses including high resistance bank expansion and contraction losses, gradient changes, debris in flow, and local obstructions such as culverts. An increase of up to 25% in the n value is appropriate when using a simple trapezoidal cross section and single n value as is most often done when calculating the channel flow portion of the travel time. Most references for n values assume clear water flow.

The NRCS recommends that the velocity defined by the bank full, cross section be used to estimate the channel component of the time of concentration. The channel velocity is a function of the two-thirds power of the hydraulic radius. Because the cross section and, therefore, the hydraulic radius changes from point to point along the channel, it may be difficult to determine the "typical" bank-full section. Care must be taken in the definition of the "typical" section because an error can lead to a significant over estimate or under estimate of the time of concentration in a large watershed that has a relatively long main stream.

4.3.6 Representative Reach Cross Section for Reach Routing

The selection of a representative cross section for reach routing can produce large systematic errors. TR-20 models with many reaches may exhibit cumulative systematic errors that will significantly affect the peak flow estimation. Since the TR-20 model is sensitive to the timing of hydrographs routed through long reaches, the typical routing section is a likely choice for adjustment.

Cumulative errors can be the product using a series of short reaches in which the hydrographs are translated downstream rather than attenuated. The TR-20 manual and other references include several methods to address the short reach problem including accounting for the reach timing as a portion of each subbasin's time-of-concentration.

Systematic errors in the selection of a "representative cross section" usually produce reach routing that underestimates the hydrograph travel and underestimates the attenuation. The n value selection and length of reach are again suspect as in the time-of-concentration channel flow component described earlier.

The effect of stream storage is often underestimated. A good method to derive a representative cross section is using the results of multiple HEC-RAS runs. For each flow rate the cumulative volume in the reach is divided by the total reach length. This results in a representative cross sectional area for each flowrate. However, cross sections for a hydraulic model are usually taken so as to eliminate ineffective flow areas. These ineffective flow areas, while not contributing to the stream conveyance in the hydraulic model, do affect the attenuation of the hydrograph in the reach routing computation. This is most common in reaches that are characterized by wide, flat flood plains and wetlands. If stream storage is expected to be underestimated, the designer may be justified in increasing the area for each flowrate value on the cross section table by an amount up to 15% and still remain within reasonable limits of reach modeling.

4.3.7 Reach Length

Reach lengths measured on large-scale maps (USGS Quad, 1:24,000) commonly underestimate the true length of a stream. Topographic maps of a scale of (1:2400) and smaller will show more meanders and yield longer measurements. The effective stream length may not be the same for minor and severe events (2-year vs. 100-year). This is due to the fact that the more extreme events are conveyed over floodplains rather than in the channel, resulting in shorter flow paths. For minor events, such as 5-year and less events, a longer reach length is appropriate due to the longer flow path in the meandering channel.

4.3.8 Storage at Culverts

Experience shows that if the storage behind a culvert is less than 15% of the volume of runoff of the contributing drainage area, storage routing may be ignored without significant impact in the peak flow rate prediction. However, an accumulation of several culverts, each having storage potential near 15%, could affect the peak flow prediction and should be examined.

The measurement of storage behind culvert is sometimes subject to systematic error, which tends to underestimate storage, especially for low flows. Topographic maps with large contours (10 or 20 feet) will not show small depressions and ditches that may contain storage that can effect the peak flow prediction of small storms.

4.3.9 Antecedent Runoff Condition (ARC) (See also discussion of RCN)

Most applications will use the recommended value of ARC=2 to represent the preliminary wetting of the ground surface and filling of small depressions. The ARC = 2, which represents the average watershed conditions when flooding occurs, is appropriate for severe storms such as the 10-, 25-, 50-, and 100-year events because they are generally related to the longer duration cyclonic events such as hurricanes and tropical storms with a longer duration. ARC = 1, which is the dry soil condition, may be more applicable to short duration summer thunderstorms in dry weather for the more frequent 2- and ~~5-year~~ rainfall events.

4.3.10 Dimensionless Unit Hydrograph – The dimensionless unit hydrograph varies by region. Refer to Table 3.1

4.3.11 Rainfall Tables

The Type II, 24-hour rainfall distribution found in the TR-20 model has been shown to approximate closely most of the Maryland statistical rainfall data for large cyclonic storms. However, there is justification for selecting durations of less than 24 hours in certain circumstances. Until new research on storm structure is complete, the 25-, 50-, and 100-year storm events should be derived using the

24-hour design storm duration. The 2-, 5-, and 10-year storm events should be derived using either the 6-hour or 12-hour design storm duration. For watersheds having a total time of concentration of less than six hours, the 6-hour design storm duration is appropriate. For watersheds having a total time of concentration greater than six hours, the 12-hour design storm duration is appropriate. In western Maryland (Appalachian Plateau as defined in Dillow (1996)), there are indications that flood producing rainfalls may be shorter duration than those further east. Therefore, if the flood estimates using the 24-hour storm do not lie between the regression estimate and the upper 68% limit, the analyst should use the 12-hour storm for the 25-, 50-, and 100-year events and the 6-hour storm for the 2-, 5- and 10-year events.

Rainfall maps for the 6-, 12-, and 24-hour storm durations for return periods of 2-, 5-, 10-, 25-, 50-,10- and 500-years are given in Appendix 6. These maps can be used to develop design storms for input to TR-20. The maps for the 2- to 100-year frequencies were adapted from TP-40. Estimates of the 500-year rainfall were made at about 20 locations around the State by extrapolating a rainfall frequency curve based on the 2-, 10-, and 100-year rainfall values. The 500-year rainfall values were then contoured using the 100-year contours as a guide.

TABLE 4.1
Table of TR-20 Variable Adjustment Limits for Calibration

Variable	Error Type	Likely Error Source Variable	Common Error Trend	Effect On Peak Q	Note	Adjustment Limits of variable in column 3
Area	**Random**	**Area**	**High or Low**	**Increase or Decrease**		**Not Recommended**
RCN	Random	Table Selection	High or Low	Increase or Decrease	4	± 10% for each category and within the limits of the NRCS guidelines.
Tt (Overland)	Systematic	N_0	Low	Increase	3	Up to 25%
Tt (shallow conc.)	Systematic	Length, n	Low	Increase	3	L and n Up to 25%
Tt (channel)	Systematic	Length, n	Low	Increase	3	L and n Up to 25%
Representative X-sect.	Systematic	Area, n	Low	Increase	3	Area and n Up to 25%
Reach Routing Length	Systematic	Length	Low	Increase	3	Up to 30% for 1;24,000 maps, up to 19% for 1:2,400 maps
Storage at culverts	Systematic	Volume	Low	Increase	1	Up to 15%
ARC	Random	N/A	N/A	N/A	2	Not Recommended – use ARC = 2
Dimensionless Unit Hydrograph	Systematic	Peak Factor K	High or Low	Increase or Decrease		Regional values of K in Maryland
Rainfall Tables	Systematic	Increment, intensity, and duration	High or Low	Increase or Decrease		24, 12 and 6 hr. distributions

Definitions: Random (errors) = either high or low from an expected mean value
Systematic (errors) = always higher or always lower than the calculated value
Low = calculated value lower than probable "actual" value
High = calculated value higher than probable "actual" value

ARC = ? is recommended

Notes:
1. If the total volume of "reservoir" storage in the watershed is less than 15% of the total rainfall volume, the effects of storage may be ignored.
2. ARC = 1 may be more appropriate for estimating the 2-yr storm runoff. <? or more frequent
3. Primary calibration variable.
4. Do not adjust the weighted RCN number.

- **Table 4.1 is presented as a guide to assist the designer as he or she reevaluates TR-20 input parameters that might be causing the peak discharges to fall outside the recommended USGS bounds. The table is a guide suggesting that, because of the difficulties in the estimation process, the parameters of column 3 could be in error by as much as the value listed in the last column. The selected values of all parameters in column 3 must be supported by field and map investigations, be consistent with standard hydrologic practice and justified in writing.**

4.4 SENSITIVITY OF TR-20 RESULTS TO VARIATION IN INPUT VARIABLES

Experience has shown that the variables that affect hydrograph timing contain the greatest potential error of estimation and are, therefore, those that should be calibrated first. The hydrograph timing variables include each of the T_t components, the Representative Reach Cross Section, and the reach length.

If further calibration is necessary, re-evaluate the watershed storage by adding storage routing at culverts and other structures that create backwater. In particular, railroad culverts and embankments frequently cause backwater and reservoir storage. In very flat areas only a small rise in backwater may generate substantial amounts of storage that should be included as reservoirs in the TR-20 model. Occasionally, urban watersheds may experience a cumulative effect of storage from multiple road culverts. It may be practical to combine a series of small culverts with backwater into one reservoir to simplify modeling if accurate flows between these culverts are not needed.

Calibration of RCN values involves selecting values within the range recommended by NRCS for each land cover and soil type. Generally, the designer will be changing the RCN value for woods, meadows, or croplands from average to good or poor condition to adjust the peak discharge. **However, these changes must be documented.** In limestone regions, there may be some justification for a further reduction in RCN values or changing the soil classification from B to A, C to B, etc.

The designer must compare the appropriate USGS regression equation with the peak flow rates computed by the TR-20 model. In some circumstances, a decision may be made to adjust the TR-20 model input variables to yield peak flows that are closer to the results of the regional equation. In most instances, the adjustment of the TR-20 input variables should fall within the ranges shown in Table 4.1. However, the following factors should be evaluated before adjusting the TR-20 input:

1. Does the TR-20, using map and field study defined input parameters that are within the bounds of sound hydrologic practice, estimate peak discharges that fall between the best estimate plus one standard error of prediction? If it does, adjustment of the TR-20 may not be necessary.

2. Are the values of the input variables used for the regional equation within the limits prescribed by USGS? Do the study watershed conditions lie within the bounds of the data from which the regional equation was derived? If the answer to either of these equations is no, then the regional equation results may not be valid.

3. If part of the study watershed lies within different regions, has the proportional regional equation been computed using the recommended USGS procedures?

4. Have the regional equation input variables been measured from the same scale maps used in the derivation of the regional equations (i.e., USGS 1:24,000 Quadrangle maps)? If not, the designer should determine if there is a possible bias by calibrating the map used with the USGS map for the same area. For example, a 200 scale map may show many small clusters of trees that are not shown as green shaded areas on the USGS quadrangle maps from which the forest cover percentage was derived. Use of the 200 scale map to measure forest cover may result in a higher area of forest or a bias toward this variable that will affect the peak flow estimate of the regional equation.

5. Are there reservoir storage, wetlands, quarries, or other features that may invalidate the regional equations? If these areas have been accounted for in the TR-20 model, there would be no benefit in a comparison to regional equation estimates.

6. Is the study area more than 15% urbanized? If so, then the regional equation may not be valid. The term "urbanized" refers to the areas on the USGS Quadrangle Topographic Maps that are shown as pink, purple (updates), or areas surrounding industrial, commercial, or institutional buildings shown as black or purple rectangles on the map.

If it is determined that the regional equation has been applied correctly and is valid for the study watershed, these results then may be used to adjust the input parameters of the TR-20 program. However, these TR-20 input parameter adjustments must be map and/or field justified and within the range of sound hydrologic practice. The designer will provide documentation that explains the selection and adjustment of each input parameter.

4.5 DERIVING ULTIMATE DEVELOPMENT PEAK FLOW RATES USING THE ADJUSTED TR-20 MODEL

In most cases, the designer/designer will derive the "Ultimate Development" peak flow rates **by only changing the RCN values in the adjusted** Existing Land Cover model. The new RCN values for each sub-basin are computed to reflect the future conditions using zoning maps or comprehensive planning maps. The other existing Land Cover model parameters usually remain unchanged. Preserving the hydrograph timing parameters can usually be justified in watersheds over one square mile since it is unlikely that a significant length of existing stream channels will be hydraulically improved under current regulations. **However, there may be instances where there is ultimate development channelization or enclosure that will result in velocities that are significantly different from those under existing conditions. In that situation the changed time of concentration would have to be incorporated.** The focus on stream water quality, stormwater management, wetland and habitat preservation in Maryland and the relatively few large river flood prone areas has inhibited the construction of major channel improvements, long large diameter pipe systems, and flood conveyance channel-levee systems. Of course, there may be exceptions to this assumption that should be examined on a case-by-case basis. If justified, the hydrograph timing parameter can be also modified to reflect expected significant changes to stream channel hydraulic characteristics.

4.5.1 Ultimate Development as Defined Under COMAR Title 08, Subtitle 05

This paragraph in "Chapter 03, Construction on Non-Tidal Waters and Floodplains" states:

"F. Unless waived by the Administration, hydrologic calculations shall be based on the ultimate development of the watershed assuming existing zoning."

In the creation of a TR-20 hydrologic model for ultimate conditions, it is common practice for the designer to derive RCN values for each zoning type for the jurisdiction of the watershed. These "ultimate development" RCN values are substituted for the "existing" RCN values and an "ultimate development" model is constructed. This model, when the regulatory 2-, 10-, and 100-year Type II rainfall is applied, results in "ultimate development" peak flow rates. These peak flow rates then are used for structure design or floodplain delineation and become the benchmark for regulatory evaluation. However, there are several pitfalls that both the practitioner and regulator should consider in its application. They are:

1. Many zoning districts cover a wide range of permitted uses that have significant variability in hydrologic characteristics. There are two methods of accounting for the wide variation: (1) use more subdivision of the zoning divisions into more homogeneous areas; (2) use weighted RCN for the zoning district based on the actual land use and hydrologic soils.

2. Existing agricultural areas that are zoned for large multi-acre lots may yield lower RCN values under "ultimate development" than under the existing conditions of active croplands. Common practice has been to select the higher of the two RCN values. In some cases this situation may be realistic if the hydrologic conditions of the area was poor. However, this case is often unidentified or ignored in large, variable land use models.

3. Many modern zoning types do not lend themselves to simple conversion to an RCN value. Several of these zoning types are related to ecological and historic preservation or recreation that have a wide range of possible future RCN values.

4. Many jurisdictions permit clustered or planned unit development that typically creates high density mixed development interspersed with natural preservation areas. The resulting land cover then bears no resemblance to the originally described zone type; hence, the ultimate RCN value derived from it is unreliable.

5. The creation and editing of zoning maps is a political process and is not intended to represent future hydrologic conditions. A jurisdiction wishing to promote industrial development, for example, may designate large areas for that zoning classification to attract industry, yet have no realistic expectation that all such zoned land will be developed. Similarly, rural jurisdictions may find it politically preferable to label vast areas as "agricultural" or "conservation" but expect to re-zone specific sites if a non-conforming, intensive use is deemed desirable. In all such cases the direct conversion from zoning type to RCN is invalid as a prediction of future peak flow rates.

6. Current environmental regulations inhibit full build out of many residential and other intensive use zoning districts. For example, a district that may permit 16 units per acre seldom achieves full density. This is due to restrictions such as wetlands, road systems, forest conservation, and recreational or open space reservations.

While these pitfalls are known to many in the hydrologic profession, the common rationalization of the use of zoning is that it is the best, or simplest, way to derive a future development model that will ensure that newly designed hydraulic structures are not underdesigned. In other words, the regulation requiring the use of "ultimate development" peak flow rates for design is simply an hydrologic safety factor. Unfortunately, because of the unreliable nature of the future land use – zoning relationship, the use of existing zoning to derive "ultimate" peak flow rates will result in undefined and highly variable factors of safety for different watersheds. This is not a correct application of factors of safety in an hydrologic analysis.

The selection of a factor of safety to apply to a calculated peak flow rate should be based on the following considerations:

1. The potential for land use changes

2. The timing of land use change

3. The potential risk of failure of the hydraulic structure

4. The economic life and useful life of the hydraulic structure

5. The reliability of the computational method

Item number 5 is usually addressed in the selection of input values for each method and is discussed in other chapters of this report. Items 3 and 4 are often considered by selecting the flow or storm frequency. In general, large expensive structures or ones that could endanger the public are designed for flows of lesser frequency such as the 100-year (1% annual change of occurrence) for major interstate highways. Minor drainage systems are designed using the 10-year (10% annual chance of occurrence) event.

Item numbers 1 and 2, as discussed above, are not reliably estimated by zoning district. A better estimate of Items 1 and 2 can be derived from comprehensive planning maps.

Comprehensive planning maps are prepared for most major jurisdictions in the state. In Maryland the latest revision is July 1, 1997. Most plans include a 20-year projection and are available in both map and digital GIS form.

4.5.2 Using Comprehensive Planning Maps For Future Hydrologic Conditions

Comprehensive planning maps, if available, offer a better tool for the designer to predict the future land use of a watershed than the zoning map. They incorporate the key elements of time and spatial distribution that are not apparent in zoning maps. The designer can compare these maps to the zoning maps to determine the following:

1. Does the 20-year comprehensive plan approach complete build-out as defined by the zoning maps? If not, it may be better to use the comprehensive plan as the more realistic future projection.

2. Does the comprehensive plan define specific land use within a general zoning type? Comprehensive plans will show areas of likely growth based on existing and planned transportation networks, proximity to growth centers, and water and sewer service areas. They will also account for special environmental or historic areas and buffers, critical areas, unfavorable terrain, proximity to uninviting land use such as landfills and airports, and similar conditions that are likely to inhibit growth.

3. Will the intensively urbanized areas induce in-fill type development according to zoning or will the general character of the urban area change? Comprehensive plans may account for the trends for more urban green space or the conversion from heavy industrial to office parks, recreation/tourism, or mixed residential/commercial use.

The current regulation permits the Administration (now Maryland Department of the Environment) to waive the requirement of current zoning to define ultimate development. This requirement should be waived in favor of the Comprehensive Planning Maps, wherever available.

4.6 ADJUSTMENT OF TR-20 USING THE USGS REGIONAL EQUATIONS WHEN EXISTING URBANIZATION IS GREATER THAN 15%

The USGS Regression Equations were developed using stream flow data from watersheds that had no more than 15% of their areas characterized by commercial, industrial and/or residential land cover. Residential land use is classified as " urban" if the lot size is less than one acre. If the developed portion of the watershed was greater than 15%, it was assumed that significant alteration of the basin had occurred and the watershed was dropped from the sample.

Even though the present report is concerned only with basins having drainage areas in excess of one square mile, it can be anticipated that many hydrologic analyses in Maryland will be conducted on watersheds having more than 15% urbanization. Thus, there needs to be an approach that will help guide the designer in the section of input parameters that will ensure results that are realistic for the existing watershed condition. The steps to follow are listed below, described by flow chart of Figure 4.2 and demonstrated through example problems presented in Appendix 4. After the calibration is complete for existing conditions, the calibrated model is applied for ultimate conditions as shown in Figure 4.3.

1. Delineate the area, determine the curve number for the existing land cover, estimate the travel times through the flow elements and the time of concentration.

2. Use the undeveloped watersheds in the region or Table 4.2, to estimate the pre-development land cover distribution and computations such as that illustrated by Table 4.3 to estimate a curve number for the pre-developed condition. Table 4.2 was developed by using a GIS to tabulate the 1985 areas of forest, cropland, grass and brush in each county.

3. Apply the USGS regression equations and the Tasker approach to estimate a prediction window for this pre-developed condition.

4. Use the drainage network on the 1:24000 USGS quad sheets to estimate a time of concentration for the pre-developed condition. If the channel cross section has changed as a consequence of urbanization, use the regional regression equations that relate width and depth to the upstream drainage area, such as those illustrated by Figure 3.9, to estimate the pre-development bank full cross sections.

5. Calibrate the TR-20 to the predevelopment USGS window of step 3.

6. Estimate a set of TR-20 discharges for the actual existing watershed conditions by multiplying the TR-20 discharges of step 5 by the ratio of the pre-developed and existing condition curve numbers. In some cases it can be anticipated that the

should it be:

$\frac{\text{Pre Volume}}{\text{Existing Volume}}$ $\frac{\text{Pre CN}}{\text{Existing CN}}$ *

drainage network may have been so modified by development that timing may also have to be adjusted.

7. The TR-20 discharges of step 6 become the estimate for existing watershed conditions and, thus, the base for simulating the ultimate conditions.

The Panel recognizes the uncertainties associated with estimating a pre-development land cover and drainage network and then extrapolating model results for those conditions to estimate a synthetic flood frequency series for a watershed having more than 15% urbanization. However, the Panel believes that the uncertainties associated with a "pre-development calibration" are less than those associated with an approach that requires the designer to select input parameters without any opportunity for calibration.

The Panel also recognizes that this pre-development calibration adds a step that could be considered labor intensive. However, GIS support for hydrologic modeling can allow the steps to be done without a great deal of effort. The MD-SHA GIS for hydrologic modeling, both the original GISHYDRO, and the new GISHydro 2000 automatically checks the existing watershed condition to determine the percent urbanization. If it exceeds 15%, it checks undeveloped areas in the county to estimate a pre-developed land cover condition. The problem in Appendix 4 illustrates the approach.

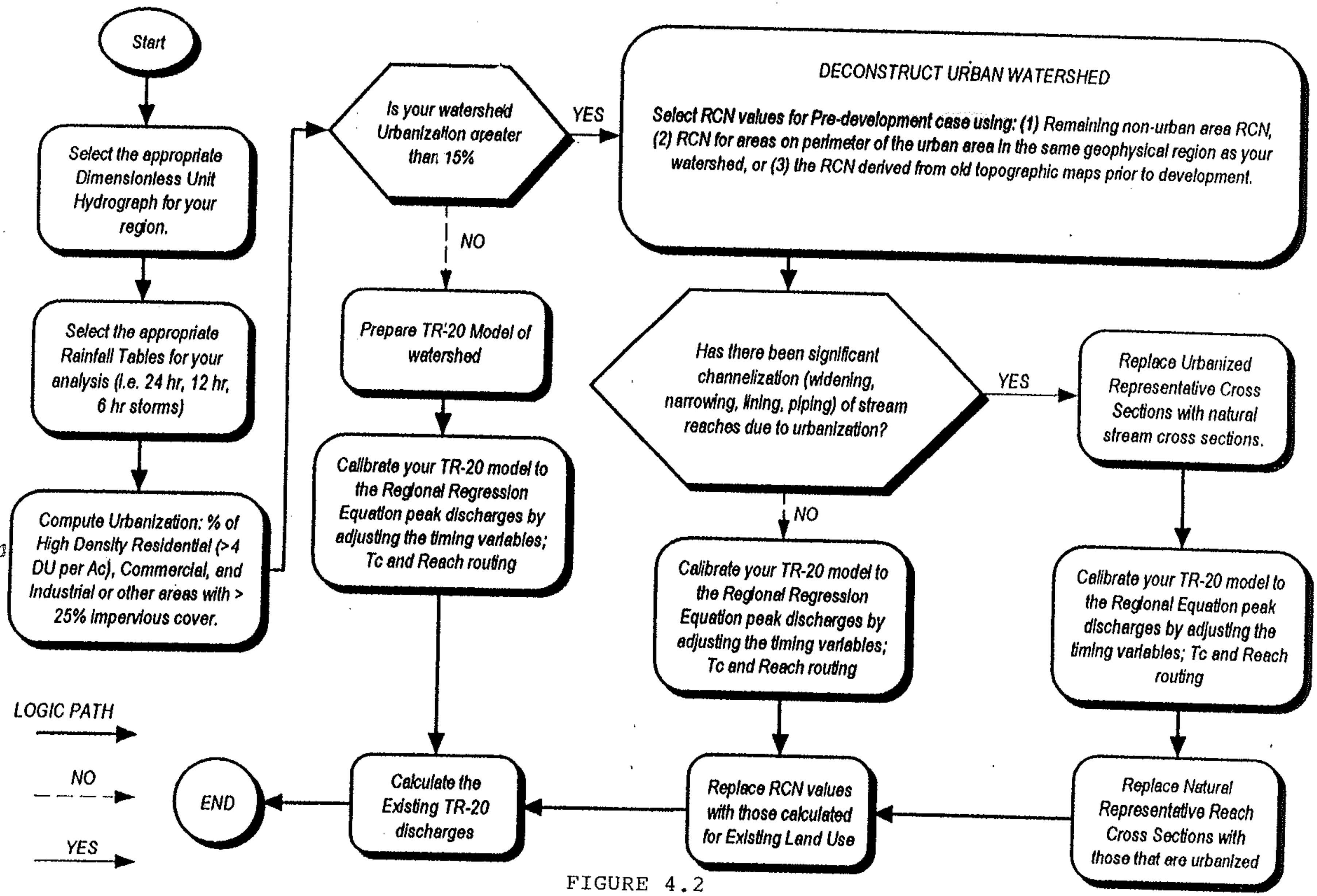


FIGURE 4.2

FLOW CHART
TR-20 CALIBRATION WITH REGIONAL REGRESSION EQUATION
EXISTING LAND USE

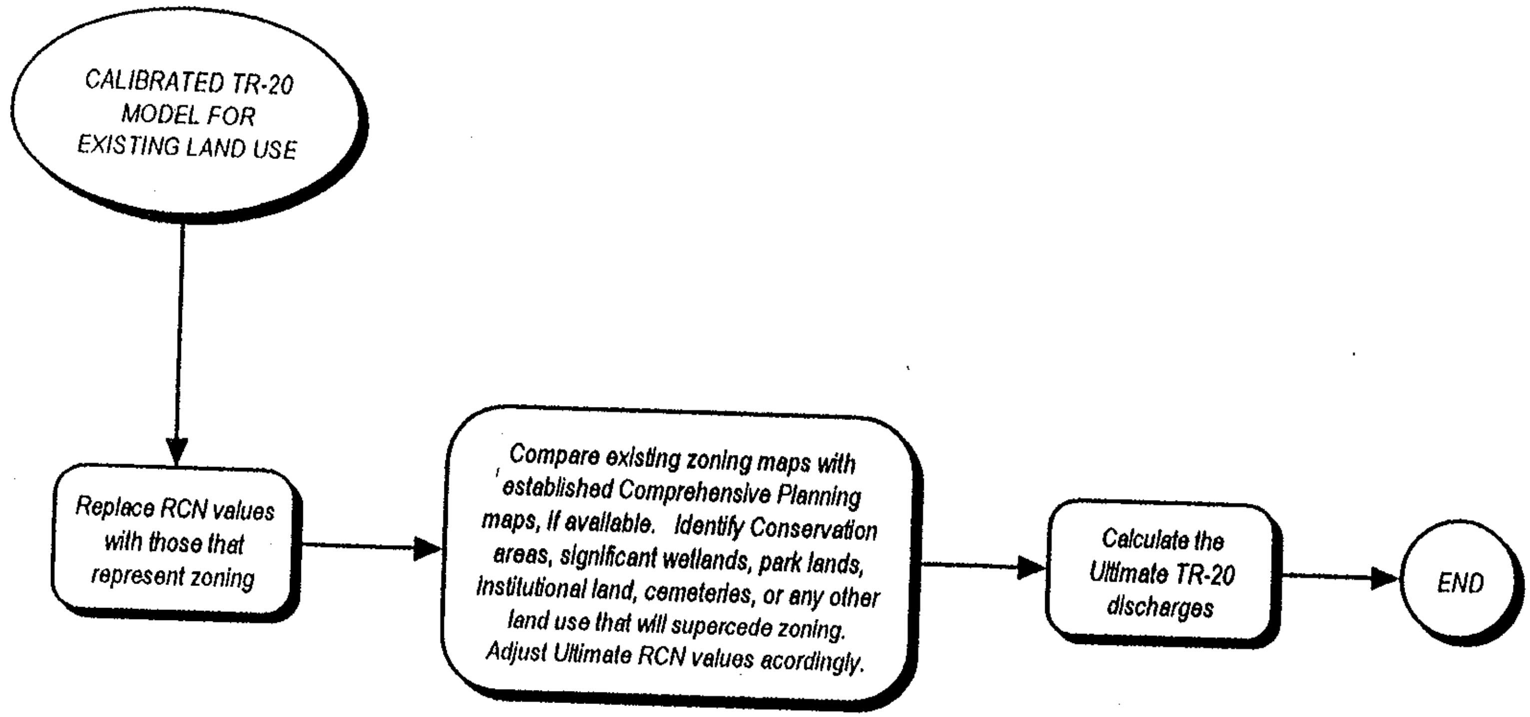


FIGURE 4.3

FLOW CHART
TR-20 ULTIMATE LAND USE

TABLE 4.2
LAND COVER DISTRIBUTIONS IN UNDEVELOPED AREAS

ALLEGANY COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	209326.95	84.11
BRUSH	5567.67	2.24
CULTIVATED	21848.40	8.78
GRASS	12126.78	4.87

ANNEARUN COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	126463.69	67.44
BRUSH	3304.80	1.76
CULTIVATED	48204.18	25.71
GRASS	9547.20	5.09

BALTIMOR COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	146499.03	52.21
BRUSH	6618.78	2.36
CULTIVATED	99621.37	35.50
GRASS	27852.12	9.93

CALVERT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	85213.35	71.05
BRUSH	1271.43	1.06
CULTIVATED	30335.31	25.29
GRASS	3121.20	2.60

CAROLINE COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	66761.55	34.49
BRUSH	2483.19	1.28
CULTIVATED	122273.02	63.16
GRASS	2070.09	1.07

CARROLL COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	66348.45	25.57
BRUSH	3203.82	1.23
CULTIVATED	161021.80	62.05
GRASS	28949.13	11.15

CECIL COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	90322.02	44.22
BRUSH	3033.99	1.49
CULTIVATED	105629.67	51.72
GRASS	5255.55	2.57

CHARLES COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	193909.14	74.11
BRUSH	4167.72	1.59
CULTIVATED	59128.38	22.60
GRASS	4443.12	1.70

TABLE 4.2- Continued

DORCHEST COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	124545.06	48.61
BRUSH	9202.95	3.59
CULTIVATED	120859.30	47.18
GRASS	1583.55	0.62

FREDERIC COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	134739.45	33.58
BRUSH	2359.26	0.59
CULTIVATED	228306.61	56.90
GRASS	35838.72	8.93

GARRETT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	288991.00	71.35
BRUSH	13251.33	3.27
CULTIVATED	75679.92	18.68
GRASS	27122.31	6.70

HARFORD COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	105996.88	45.32
BRUSH	2923.83	1.25
CULTIVATED	110123.28	47.08
GRASS	14839.47	6.34

HOWARD COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	57466.80	46.02
BRUSH	4511.97	3.61
CULTIVATED	51274.89	41.06
GRASS	11617.29	9.30

KENT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	46795.05	27.63
BRUSH	688.50	0.41
CULTIVATED	120854.70	71.37
GRASS	1005.21	0.59

MONTGOME COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	91432.80	40.65
BRUSH	8945.91	3.98
CULTIVATED	80453.52	35.77
GRASS	44096.13	19.60

PRINCE-G COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	152580.78	68.59
BRUSH	1142.91	0.51
CULTIVATED	50669.01	22.78
GRASS	18061.65	8.12

TABLE 4.2- Continued

QUEEN-AN COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	66977.28	29.99
BRUSH	1000.62	0.45
CULTIVATED	153232.56	68.62
GRASS	2106.81	0.94

ST-MARYS COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	133679.17	65.68
BRUSH	1748.79	0.86
CULTIVATED	63915.75	31.41
GRASS	4172.31	2.05

SOMERSET COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	73210.50	50.91
BRUSH	15009.30	10.44
CULTIVATED	54267.57	37.74
GRASS	1308.15	0.91

TALBOT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	44596.44	28.06
BRUSH	1041.93	0.66
CULTIVATED	110164.59	69.31
GRASS	3134.97	1.97

WASHINGT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	117233.20	42.76
BRUSH	3635.28	1.33
CULTIVATED	132320.53	48.26
GRASS	20999.25	7.66

WICOMICO COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	101002.95	48.68
BRUSH	13462.47	6.49
CULTIVATED	91878.03	44.28
GRASS	1152.09	0.56

WORCESTE COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	150097.59	56.80
BRUSH	15564.69	5.89
CULTIVATED	96692.95	36.59
GRASS	1918.62	0.73

TABLE 4.3
ESTIMATING PRE-DEVELOPMENT CURVE NUMBER

The existing land cover of a 1000 acre watershed being investigated in Baltimore County is more than 15% urban. As part of the calibration process against the USGS regression equations, we need to estimate a pre-development curve number. The hydrologic soil distribution for the 1000 acre watershed is:

Group A	100 acres
Group B	700 acres
Group D	200 acres

Table I shows the distributions of undeveloped areas in Baltimore County as:

Forest	52.21%
Brush	2.36%
Cultivated	35.50%
Grass	9.93%

The curve numbers for the land/soil complexes are:

	Hydrologic Soil			
Category	A	B	C	D
Forest	36	60	79	89
Brush	35	56	70	77
Cultivated	72	81	88	91
Grass	48	69	79	89

An estimate of a pre-development curve number is obtained by assuming the land cover is equally distributed over the existing A, B and D soil groups as:

Forest	.5221[(100)(36) + (700)(60) + (200)(79)]	=	32,057
Brush	.0236[(100)(35) + (700)(56) + (200)(77)]	=	1,371
Cultivated	.3550[(100)(72) + (700)(81) + (200)(91)]	=	29,148
Grass	.0993[(100)(48) + (700)(69) + (200)(89)]	=	7,040
	Total	=	69,616

Pre-development Curve Number = 69,616/1000 = 69.6

V. RECOMMENDATIONS FOR RESEARCH

5.1 INTRODUCTION

In spite of the volume of research reported in professional literature, knowledge of many aspects of applied engineering hydrology is lacking. In this section, some aspects of design hydrology that require additional research are identified along with the potential benefits that could result from better knowledge about these topics. Research on the topics below would possibly enable better decisions to be made with respect to the use of hydrologic methods in hydrologic design.

5.2 TIME OF CONCENTRATION

The time of concentration is a principal input to most hydrologic design methods. The velocity method and curve number method are two primary tools used in estimating times of concentration. The velocity method generally uses Manning's equation to compute the velocity. The NRCS (SCS) TR-55 kinematic wave equation is frequently applied for computing travel time for shallow sheet flow.

When the velocity is computed using Manning's equation, estimates of the roughness coefficient, the hydraulic radius, and the slope are required. Each of these inputs is important, and error or uncertainty in the inputs reduces the accuracy of estimates of the time of concentration. Roughness varies considerably with river stage. Since the river stage for a design discharge is related to the return period of the flow, it is likely that the roughness used to compute a velocity should depend on the cross section that reflects the discharge rate for the design return period. Research on the effects of depth dependent Manning roughness coefficients on time of concentration is needed. If only the roughness of the channel bottom or that for bankfull flow is used when the design return period would suggest out-of bank flow conditions, the estimated velocity and, therefore, the computed t_c could be significantly different than the most appropriate value.

An estimated velocity is sensitive to the assumed hydraulic radius. The hydraulic radius can be a function of the stage of flow, which as indicated above depends on the return period. The hydraulic radius also depends on the shape of the cross section, which can vary considerably along a channel reach. Therefore, it is necessary to understand the sensitivity of computed velocities when using a single supposedly representative hydraulic radius for a stream in which the cross section changes noticeably over the channel length. Research on the effects of variation in both the return period and cross-section characteristics as they relate to the hydraulic radius could improve the estimation of t_c.

If a representative cross section is difficult to select because of excessive variation in cross section characteristics throughout a channel reach, the Dunne and Leopold (1978) equations can be used to compute the cross-section characteristics. While preliminary analyses suggest that these equations provide reasonable estimates in Maryland, the underlying data base is sparse. More analyses of these equations using data from Maryland are needed.

The slope of a channel section is computed using the elevation drop and the reach length. Generally, the variation in reach length for different scale maps is considerably greater than variation in the elevation drop. Where the reach length is estimated from a map, the accuracy of the length will influence the accuracy of the computed slope. If a large map scale is used and the scale of the map prevents accurate depiction of the meanders, then the overall length could be underestimated, which leads to an overestimate of the slope and velocity and an underestimate of the t_c. The significance of this factor needs investigation.

Empirical models, such as the NRCS lag equation, are possible alternatives to the velocity method. While a number of studies indicate that some empirical models provide reasonable estimates of t_c, the accuracy of empirical models for use in Maryland has not been evaluated. Useful research could result from using times of concentration obtained from rainfall-runoff data to assess the accuracy of empirical equations. As additional research, t_c values estimated from rainfall-runoff data could be used with measured physiographic data to calibrate empirical equations for different regions of Maryland and develop a synthetic hydrograph in conjunction with these times.

Another alternative to the velocity method is to define the time of concentration from observed rainfall hyetographs and discharge hydrographs. Using this approach, the time of concentration is defined as the time from the ending of rainfall excess to the first inflection point on the recession of the discharge hydrograph. Regression analysis can be used to relate the computed time of concentration to watershed and climatic characteristics for the gaged watershed. Estimates of the time of concentration can be made at ungaged locations by simply determining the watershed and climatic characteristics and applying the regression equation.

A regression equation for estimating time of concentration for Maryland streams is described in Appendix 5. Limited tests with this regression equation have been promising. The regression approach is easy to use and provides reproducible estimates, but the time of concentration is generally in excess of that detemined by the velocity method. Several questions have been raised as to whether it is appropriate to use estimates of the time of concentration determined from observed rainfall-runoff data in conjunction with the NRCS unit hydrograph. Furthermore, the computed times of concentration given in Appendix 5 were generally based on runoff events less than the 2-year flood. **Research is needed to determine if the time of concentration from observed rainfall-runoff data**

should be used with NRCS hydrograph theory and to determine if the time of concentration varies significantly with the magnitude and frequency of peak discharge.

5.3 UNIT HYDROGRAPH PEAK RATE FACTORS

While some research on the peak rate factor for the NRCS unit hydrograph has been completed, additional work is still needed. Most importantly, peak rate factors need to be estimated from hydrograph data, not just peak discharge data. It is important to estimate the peak rate factor from unit hydrographs computed from measured hyetographs and hydrographs. This research could show the geographic variation of peak rate factors, as well as the extent of their uncertainty. Additionally, peak rate factors computed from unit hydrographs obtained from rainfall-runoff data could be compared to the peak rate factors computed using geomorphic unit hydrographs derived from time-area curves. This would enable geomorphic unit hydrographs to be combined with hyetograph – hydrograph generated unit hydrographs in selecting regional peak rate factors. Improving estimates of the peak rate factor for Maryland watersheds will improve design accuracy.

5.4 PEAK DISCHARGE TRANSPOSITION

While various forms of peak discharge transposition are widely used, surprisingly little understanding of their accuracy exists. The results provided by McCuen and Levy (1999) for Pennsylvania, Virginia, and Maryland appear to be the only empirical assessment of the transposition procedure. The PA/VA/MD data base is sparse; therefore, these results need to be verified for other data sets. Additionally, the variation of the weighting functions, both of the area-ratio and USGS methods, needs to be assessed over a broader range of data. The structures of the weighting functions need to be specifically evaluated.

Research on the alternative transposition methods should be performed to assess the accuracy of the methods. The results would increase the confidence that could be placed in their use. Without this additional research, transposition methods should be used with caution.

5.5 TRANSFORMATION OF ZONING-MAP INFORMATION INTO HYDROLOGIC MODEL INPUT

Some designs require assessment for ultimate-development watershed conditions. The input to hydrologic models for ultimate-development conditions often requires obtaining information from zoning maps. Zoning maps delineate areas assigned to different land use categories. However, these categories are not consistent across political boundaries and, more importantly, a systematic method for transforming the land use categories into inputs for hydrologic models is lacking. For example, different jurisdictions use different notations for the various densities of residential development, and measures of the corresponding impervious area, which is important input to hydrologic design methods, are not provided or are ambiguously assessed.

While it would be useful to have standard zoning classifications for all jurisdictions in Maryland, this is unlikely to happen. Even this would not eliminate the need for a procedure for transforming zoning map classifications into input parameters for hydrologic design methods. Research could provide a procedure for estimating model inputs such as impervious areas and curve numbers from zoning classifications. This would improve the reproducibility of designs.

5.6 ADJUSTING TR-20 USING REGRESSION EQUATION ESTIMATES

When applying the TR-20 adjustment procedure using the confidence limit on the USGS regression equation, the best estimate plus plus error of prediction window is recommended herein. This value is based on the judgement and hydrologic experience of the Panel members.

Research needs to be undertaken on the most accurate and appropriate confidence level, which will probably vary with geographic region, return period, drainage area and project. A systematic research effort should provide confidence levels that can make TR-20 adjustments more accurate.

5.7 THE DESIGN STORM

The traditional approach followed in Maryland is to use the NRCS Type II 24-hour duration storm as the input to the TR-20. The volume of precipitation is selected from the appropriate IDF curve. If the TR-20 over-predicts, a major portion of the problem may originate from the severity of this design storm input. Twenty-four hours may be too long and the structure may not be appropriate for all parts of Maryland. The 24-hour duration coupled with the NRCS Type II storm distribution may be especially inappropriate for Western Maryland where gaged discharges tend to be much lower than those estimated by the TR-20 model.

A flood hydrograph study for the State of Maryland by the U.S. Geological Survey (Dillow, 1997) identified 278 rainfall-runoff events at 81 gaging stations throughout Maryland. These rainfall-runoff events were used to develop dimensionless hydrographs for three hydrologic regions in Maryland and to estimate the average basin lagtime for each of the 81 gaging stations.

These rainfall-runoff data were used to investigate the duration of rainfall to provide insight into whether the 24-hour duration storm used with the TR-20 model was reasonable. Rainfall events were analyzed for 10 gaging stations where one of the runoff events exceeded a 10-year event. The time from the beginning of rainfall to the ending of rainfall, including intermittent periods of rainfall, was tabulated. The longest duration storms tended to be tropical depressions such as the November 1985 Hurricane Juan that caused severe flooding in Western Maryland or the June 1972 Hurricane Agnes that caused extensive flooding across central Maryland and Delaware. The duration of these tropical depressions ranged from 14 to 24 hours.

Spring and summer rainfall events were generally less than 10 hours in duration. A few spring or summer rainfall events in Western Maryland exceeded 10 hours in duration but the rainfall was intermittent with long periods of no rainfall. Based on a limited sample of events, it appears that rainfall events in Western Maryland are less intense than in Central and Eastern Maryland and this may contribute to the lower peak discharges per square mile that have been observed in this region.

The study currently underway by NWS in the Ohio River Basin should provide information on rainfall amounts and design storm temporal structure. Additional research is needed to determine the most appropriate storm duration and structure for use with TR-20.

5.8 GEOMORPHIC UNIT HYDROGRAPHS

Standard unit hydrograph shapes are used in hydrologic design. For Maryland, the NRCS 484-UHG and 284-UHG are accepted. Research suggests that the most appropriate unit hydrograph for a watershed is one that is based on the geomorphic characteristics of the watershed. Recent research in the professional literature suggests that time-area based unit hydrographs accurately regenerate observed storm runoffs. With the capability of GIS to generate watershed boundaries and internal drainage structures from digital terrain data, it is feasible to use GIS to develop a unit hydrograph that is unique to a watershed, thus improving the accuracy of design hydrographs.

A study of Maryland watersheds should be undertaken to evaluate the accuracy of geomorphic unit hydrographs. Predictions of storm runoff based on these should be compared with predictions based on the 484-UHG and 284-UHG. Both the NRCS and geomorphic unit hydrographs could be compared with measured runoff events in Maryland to assess their accuracy.

5.9 STATISTICAL ALTERNATIVES

The need to evaluate an alternative statistical method that may provide more accurate estimates of the 2- to 500-year flood peak discharges for rural and urban streams in Maryland should be evaluated. Research should compare three statistical methods for watersheds with small drainage basins (1 to 20 square miles should be compared). The three promising methods that should be tested are: (1) Bulletin 17B at-site estimates coupled with weighted and generalized least square regression analysis (updated version of Dillow (1996)); (2) the Region of Influence method based on Bulletin 17B procedures; and (3) an index flood approach based on L-moment analysis.

5.10 MUSKINGUM-CUNGE ROUTING MODULE

The new version of NRCS-TR-20 will replace the Att-Kin routing module with a Muskingum-Cunge (M-C) approach. The M-C method is a spinoff of the Muskingum method that has been used for may years in river forecast operations by the National Weather Service, U.S. Army Corps of Designers and similar organizations. Both the M-C and Muskingum methods use a series of routing coefficients that are defined by the routing period, dt, a travel time constant for the routing reach, K, and a weighting factor, x. In the traditional river forecast environment, there are usually recorded inflow and outflow hydrographs that can be used to define K and x and earlier experiences on the river can evolve the optimal value of dt. Concise summaries of the two routing methods can be found in Bedient and Huber (1992).

In the SHA environment, there will be no records of inflow and outflow hydrographs at the point of interest that can be used to determine K and x. Without historic records of inflow and outflow hydrographs, K is estimated by the length of the routing reach and the celerity of a small gravity wave moving through the reach. The length of the routing reach is a decision made by the user. The celerity of the small gravity wave requires an estimate of the average velocity, width and depth of flow through the routing reach. The value of x is defined from the routing reach length, average width, average slope, celerity of a gravity wave, and the peak discharge entering the reach.

The M-C method was selected by NRCS because it was concluded that it would overcome some of the problems associated with the ATT-KIN module. Before the M-C method can be used with confidence by SHA, a research project similar to that conducted by Ragan and Pfefferkorn (1992) is needed to examine the impact of different input decisions that have to be made by the user. Note that all the parameters in the previous paragraph have feedbacks involving many of the same issues that impact the performance of the current Att-Kin method. For example, to get the coefficients x and K, the user has to have decided on the length of the routing section and must still make judgment decisions on the

Manning n and "average cross section" so that the celerity can be computed. The values for each of these elements are difficult to determine.

As illustrated in Chapter 3, the SHA has evolved an experience and research base that allows the users of the Att-Kin method to assess the consequences of different Manning roughness coefficients and cross sections that are based on field and map investigations. A research project similar to that of Ragan and Pfefferkorn (1992) is needed to indicate the changes in the runoff hydrograph caused by different decisions on the input parameters to the M-C method. The project will need to provide more guidance to the user on the selection of the input parameters than is currently available. The experiences gained on major rivers will be of limited value in the smaller watershed arena of interest to SHA and similar users. Such a project should be undertaken by the NRCS because their state offices are going to face the same problems as SHA. If a major NRCS effort does not materialize, the SHA and MDE should conduct a joint project that focuses on Maryland conditions. Until this recommended research is completed, the SHA should be allowed the option of continuing use of the Att-Kin approach.

5.11 DEVELOPMENT OF A MODEL FOR USE ON MIXED URBAN-RURAL WATERSHEDS

An increasing number of watersheds of concern to the SHA are going to have some portions that are highly urbanized and other areas that are in agricultural or forest land cover. The TR-20 can adjust the structure of the runoff flow paths to reflect man-made drainage, and urban curve number categories can define the land covers. However, the TR-20 was not designed for this type of watershed. The dimensionless UHG, as one example, was derived from rural watershed data.

The SHA needs a deterministic model that can handle a rational partitioning of the watershed into urban and rural segments. Such a model would not have to be a totally original system. It could be a combination of two models, one of which would be implemented on the urbanized portions and the other on the rural portions. The urban component might draw on the EPA Storm Water Management Model as a base and the rural component could be a revision of the TR-20. The mechanics of this approach could be done today. However, a significant level of research would have to be conducted to put the components into a package that would give consistent results and would be relatively easy to run.

5.12 RECOMMENDATIONS FOR UPDATING THE HYDROLOGY PANEL REPORT

The recommendations provided in this report are based on a combination of hydrologic judgement, existing reports and methodologies, and limited testing and evaluations of new concepts. The centerpiece of the recommendations is to quasi-calibrate the TR-20 deterministic watershed model using the USGS regional regression equations where these equations are applicable. This approach has not been tested extensively but appears to be a logical approach for improving estimates of flood discharges for Maryland and for combining the strengths of TR-20 modeling and regional regression equations. As more experience is gained with this approach and as technology changes, this approach may need to be revised. Similarly, as new research is completed, new technology should be incorporated into this report.

This report should be considered a dynamic report with updates as needed. MSHA and MDE should jointly pursue the recommended research to improve the estimation of flood discharges for Maryland streams.

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APPENDIX 1

Watershed and Climatic Characteristics Evaluated
But Not Used in the U.S.G.S. Regression Equations

Appendix 1. Watershed and Climatic Characteristics Evaluated But Not Used in the U.S.G.S. Regression Equations.

SL = Main Channel Slope in feet per mile (ft/mi)
Y224 = 2-yr, 24-hr precipitation in inches(in)
MAP = Mean Annual Precipitation in inches(in)
MBE = Mean Basin Elevation in feet(ft)
SOILA = Percent of watershed area in Hydrologic Soil Type A (%)
SOILD = Percent of watershed in Hydrologic Soil Type D (%)
HYPAR = Relative area under the hypsometric curve (dimensionless) (0<=HYPAR<=1)
HYPSL = Maximum slope of the hypsometric curve (dimensionless)

Gage No.	SL	Y224	MAP	MBE	SOILA	SOILD	HYPAR	HYPSL
01485000	1.49	3.4	47.	44	2	97		
01485500	3.56	3.35	47.	46	8	83		
01486000	5.47	3.3	46.	32	0	98		
01486100	8.47	3.45	47.	59	6	74		
01489000	7.65	3.5	45.	46	0	49		
01490000	4.53	3.5	45.	28	5	39		
01490800	7.67	3.45	44.5	56	0	70		
01491000	3.01	3.5	45.	60	8	64		
01491050	6.06	3.45	45.	54	2	36		
01492000	14.8	3.45	44.5	55	8	20		
01492050	9.81	3.5	45.5	43	0	13		
01492500	8.80	3.4	44.5	59	0	21		
01492550	16.9	3.4	44.5	54	4	20		
01493000	6.06	3.45	43.5	61	3	40		
01493500	9.15	3.35	43.	60	1	9		
01495000	17.9	3.25	44.	398	0	8		
01495500	23.7	3.25	44.5	359	0	7		
01496000	24.0	3.25	43.5	380	0	11		
01496080	125.	3.25	44.	229	0	0		
01496200	29.0	3.25	43.	375	0	5		
01578500	8.84	3.2	42.	497				
01578800	69.8	3.25	42.5	363	0	10		
01579000	37.0	3.25	42.5	348	0	3		
01580000	17.7	3.15	44.5	657				
01581500	38.2	3.25	43.	390	0	26		
01581700	30.0	3.2	44.	488	0	8		
01582000	33.8	3.1	45.	658	0	3		

01582510	94.5	3.2	45.	612	0	0		
01583000	46.1	3.25	45.	591	0	0		
01583495	181.	3.25	45.	402	0	0		
01583500	24.5	3.2	45.	544	0	4		
01583580	123.	3.25	45.	556	0	1		
01584500	21.1	3.25	45.	542	0	4		
01585100	48.2	3.4	44.					
01585300	54.7	3.4	44.					
Gage No.	SL	Y224	MAP	MBE	SOILA	SOILD	HYPAR	HYPSL
01585400	27.1	3.4	44.				0.66	2.6
01585500	56.0	3.15	45.	823	29	6		
01586000	23.9	3.15	45.	747	21	6		
01586500	16.7	3.2	44.	702	19	5		
01587000	14.6	3.35	44.5	665	24	4		
01587050	104.	3.3	43.	723	0	0		
01587500	26.2	3.3	43.	642	15	3		
01588000	38.4	3.3	43.5	665				
01589300	21.0	3.3	44.	555				
01589440	31.7	3.35	44.5	491	0	7		
01590000	18.1	3.7	43.5	112	0	13	0.44	2.5
01590500	19.8	3.7	43.5	120	0	12	0.44	2.4
01591000	28.2	3.35	42.5	589	0	1		
01591500	26.2	3.35	43.	559	3	5		
01592000	17.7	3.4	42.5	508	1	6		
01593350	80.6	3.5	43.5	422	0	0		
01593500	22.1	3.45	43.	409	0	7		
01594000	9.8	3.45	43.	439	0	7		
01594400	32.9	3.55	44.5	262	8	25	0.69	2.5
01594445	28.7	3.7	44.5	124	0	2	0.58	2.0
01594500	7.44	3.6	44.	147	0	9	0.44	2.2
01594600	22.8	3.75	44.	106	8	14	0.39	2.3
01594800	20.7	3.5	44.	108	21	11	0.36	2.5

01595000	30.5	2.75	48.	2450		
01595500	48.7	2.75	46.	2820		
01596000	47.6	2.75	45.5	2670		
01596500	65.1	2.75	44.	2510	0	3
01597000	137.	2.75	46.	2510	0	0
01598000	69.8	2.75	44.5	2410	0	2
01599000	62.7	2.75	41.5	2170	0	7
01600000	36.3	2.75	43.	2270		
01601500	55.0	2.75	36.	1880		
01603000	39.1	2.75	42.	2160		
01609000	13.5	2.85	36.	1310		
01609500	61.2	2.85	36.5	818		
01610105	345.	2.85	36.	1330		
01610150	46.3	2.9	37.	1060	0	1
01610155	17.5	2.85	38.	1120		
01612500	93.6	2.9	37.5	851	0	0
01613000	7.93	2.85	35.5	1800		
01613150	47.5	2.9	37.5	720	0	1
01613160	132.	2.9	37.5	683	0	0
01614500	11.2	3.0	39.5	1050		
01617800	23.8	3.10	39.5	509		
01618000	5.98	2.95	37.	1520		

Gage No.	SL	Y224	MAP	MBE	SOILA	SOILD	HYPAR	HYPSL
01619475	401.	3.15	39.5	512	0	0		
01619500	10.8	3.1	40.	781				
01637000	210.	3.2	42.	1010	0	1		
01637500	47.5	3.15	42.5	1110	0	2		
01637600	262.	3.2	40.5	826				
01638500	5.56	3.05	39.5	1360				
01639000	18.9	3.05	43.5	597				
01639095	77.8	3.1	43.	525	0	0		
01639500	12.8	3.1	43.5	625	51	4		
01640000	58.5	3.15	43.5	709	66	0		
01640500	201.6	3.1	47.5	1460	0	0		
01640700	51.7	3.1	41.5	411	0	2		
01641000	135.	3.1	46.	1100				
01641500	263.	3.2	46.5	1450	0	0		

01642000	6.91	3.1	44.	731				
01642400	46.0	3.2	42.5	551				
01642500	19.2	3.25	42.5	576	16	0		
01643000	5.57	3.15	44.	621				
01643500	23.8	3.25	42.	521				
01644420	132.	3.3	41.5	532	0	14		
01645000	15.1	3.35	42.	468				
01646500	4.36	3.1	40.5	1240				
01647720	25.6	3.4	43.	457	0	16		
01650050	42.2	3.4	43.	455	0	7		
01650085	130.	3.45	43.	484	0	0		
01650190	117.	3.4	43.	486	0	6		
01650500	19.3	3.45	43.5	415	0	10		
01653600	16.1	3.5	44.	197	5	10	0.25	4.5
01658000	10.5	3.5	45.5	183	9	32	0.22	4.1
01660900	22.8	3.65	44.5	214	5	27	0.24	1.8
01660930	20.5	3.65	44.	160	9	12	0.28	3.2
01661000	21.2	3.6	42.5	133	17	7	0.22	3.3
01661050	12.4	3.55	44.5	112	24	10	0.36	2.7
01661430	61.5	3.4	44.5	80	0	7	0.23	2.7
01661500	12.9	3.45	41.	101	8	12	0.35	3.0
03075450	99.4	2.75	47.5	2520	0	30		
03075500	6.09	2.7	50.	2610				
03075600	204.	2.7	49.5	2470	0	5		
03076505	415.	2.65	48.	1930	0	0		
03076600	65.6	2.7	48.	2460	0	2		
03077700	135.	2.7	46.5	2820	0	5		
03078000	32.4	2.7	45.5	2610	0	12		
03078500	39.3	2.6	43.	2550				

APPENDIX 2

Examples of Computing Design Discharges
Using Tasker's Regression Program

Appendix 2 - Examples of Computing Design Discharges Using Tasker's Regression Program

This Appendix illustrates the application of Tasker's computer program to actual examples using data for Maryland streams.

Examples 1-4 are examples of applying Tasker's program in computing a weighted estimate at the Little Patuxent River at Guilford Downs (station 01593500).

Example 1 is an application of the regression equation at gaging station 01593500.

Example 2 is a weighted estimate at gaging station 01593500. The equivalent years of record are the sum of the equivalent years for the regression estimate and the years of actual record at station 01593500.

Example 3 is an example of extrapolating the weighted estimate at station 01593500 (38 square miles) downstream to a drainage area of 50 square miles. The equivalent years of record is interpolated by drainage area between the gaging station at 38 square miles and at 57 square miles (50% increase) where the regression equations are applicable.

Example 4 is an example to illustrate that the one cannot extrapolate beyond the 50% limit. The estimates in Example 4 are simply the regression estimates for station 01593500, no weighting was performed because the drainage area is outside the 50% limit.

Example 5 is an example of extrapolating the regression equations beyond their applicable limits for an ungaged site in the Piedmont Region.

Appendix 2 - Examples of computing design discharges using Tasker's regression program

Example 1 - Regression equation estimates at Little Patuxent River at Guilford (station 01593500).

REGION: Piedmont region
area= 38.00: forest = 33.00 :skew= 0.53

Return Period	Discharge (cfs)	Standard Error of Prediction (percent)	Equivalent Years of Record	Standard Error of Prediction (logs)
2	1670.	38.9	2.59	0.1629
5	2990.	35.0	5.97	0.1478
10	4160.	34.5	9.31	0.1458
25	5960.	36.1	13.14	0.1522
50	7550.	38.5	15.12	0.1616
100	9380.	41.6	16.34	0.1737
500	14900.	51.0	17.14	0.2088

P R E D I C T I O N   I N T E R V A L S

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
2	1290.	2150.	1150.	2430.	895.	3110.	785.	3550.
5	2370.	3760.	2130.	4200.	1700.	5260.	1510.	5920.
10	3310.	5220.	2970.	5820.	2380.	7260.	2120.	8170.

25	4700.	7560.	4200.	8460.	3330.	10700.	2950.	12100.
50	5860.	9710.	5200.	11000.	4070.	14000.	3570.	15900.
100	7150.	12300.	6290.	14000.	4830.	18200.	4200.	21000.
500	10700.	20600.	9180.	24000.	6690.	33000.	5650.	39000.

Appendix 2 - Examples of computing design discharges using Tasker's regression program

Example 2 - Computing a weighted estimate at Little Patuxent River at Guilford (station 01593500).

REGION: Piedmont region
area= 38.00: forest = 33.00 :skew= 0.53

Return Period	Discharge (cfs)	Standard Error of Prediction (percent)	Equivalent Years of Record	Standard Error of Prediction (logs)
2	1350.	7.8	60.60	0.0337
5	2520.	10.4	64.00	0.0451
10	3690.	12.5	67.30	0.0542
25	5720.	15.1	71.10	0.0654
50	7730.	17.0	73.10	0.0735
100	10300.	18.9	74.30	0.0815
500	19000.	23.3	75.20	0.0997

P R E D I C T I O N I N T E R V A L S

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
2	1280.	1420.	1250.	1460.	1190.	1530.	1150.	1580.
5	2350.	2700.	2270.	2790.	2120.	2990.	2040.	3100.

10	3390.	4010.	3250.	4180.	3000.	4540.	2870.	4740.
25	5170.	6340.	4920.	6650.	4460.	7350.	4230.	7740.
50	6890.	8670.	6520.	9150.	5840.	10200.	5500.	10900.
100	9030.	11700.	8500.	12400.	7510.	14000.	7040.	15000.
500	16300.	22300.	15100.	24000.	13000.	27900.	12000.	30200.

Estimates adjusted for proximity to station 1593500

Appendix 2 - Examples of computing design discharges using Tasker's regression program

Example 3 - Extapolating a weighted estimate at Little Patuxent River at Guilford (station 01593500) downstream to an ungaged site of 50 square miles.

REGION: Piedmont region
area= 50.00: forest = 31.00 :skew= 0.53

Return Period	Discharge (cfs)	Standard Error of Prediction (percent)	Equivalent Years of Record	Standard Error of Prediction (logs)
2	1870.	12.2	23.91	0.0528
5	3360.	15.8	27.24	0.0682
10	4740.	18.4	30.52	0.0794
25	6950.	21.6	34.27	0.0929
50	8990.	24.0	36.22	0.1029
100	11400.	26.5	37.42	0.1132
500	19300.	32.6	38.24	0.1379

P R E D I C T I O N I N T E R V A L S

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
2	1720.	2030.	1660.	2110.	1530.	2290.	1460.	2390.
5	3020.	3740.	2870.	3940.	2590.	4370.	2450.	4610.

10	4190.	5370.	3950.	5690.	3500.	6420.	3280.	6850.
25	6010.	8040.	5610.	8610.	4870.	9910.	4520.	10700.
50	7660.	10600.	7090.	11400.	6070.	13300.	5580.	14500.
100	9580.	13600.	8810.	14800.	7420.	17600.	6770.	19300.
500	15500.	23900.	14000.	26500.	11400.	32600.	10200.	36500.

Estimates adjusted for proximity to station 1593500

Appendix 2 - Examples of computing design discharges using Tasker's regression program

Example 4 - Extrapolating a flood discharge to an ungaged site on the Little Patuxent River that is more than 50 percent of the drainage area at gaging station 01593500 (38.0 square miles).

REGION: Piedmont region
area= 68.00: forest = 27.50 :skew= 0.53

Return Period	Discharge (cfs)	Standard Error of Prediction (percent)	Equivalent Years of Record	Standard Error of Prediction (logs)
2	2500.	38.9	2.43	0.1632
5	4400.	35.1	5.61	0.1479
10	6060.	34.6	8.76	0.1459
25	8600.	36.2	12.35	0.1523
50	10800.	38.6	14.22	0.1618
100	13400.	41.7	15.36	0.1739
500	21100.	51.1	16.10	0.2092

P R E D I C T I O N   I N T E R V A L S

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
2	1940.	3230.	1720.	3650.	1340.	4670.	1180.	5330.

5	3490.	5540.	3130.	6180.	2500.	7740.	2220.	8720.
10	4830.	7610.	4330.	8480.	3470.	10600.	3080.	11900.
25	6780.	10900.	6050.	12200.	4800.	15400.	4250.	17400.
50	8410.	13900.	7460.	15700.	5840.	20100.	5120.	22900.
100	10200.	17600.	8980.	20000.	6890.	26100.	5990.	30000.
500	15200.	29200.	13000.	34100.	9470.	46900.	8000.	55500.

Difference in drainage area for Station 1593500 too great: NO ADUSTMENT MADE

Appendix 2 - Examples of computing design discharges using Tasker's regression program

Example 5 - Estimating flood discharges for an ungaged site in the Piedmont Region that is beyond the applicable limits of the regression equations.

Note that the drainage area and forest cover are within the limits of the data but the combination of a small watershed and high forest are outside the limits of the Regressor Variable Hull in Figure 2.1.

REGION: Piedmont region
area= 0.50: forest = 70.00 :skew= 0.53

Return Period	Discharge (cfs)	Standard Error of Prediction (percent)	Equivalent Years of Record	Standard Error of Prediction (logs)
2	90.	42.4	3.42	0.1766
5	186.	39.2	7.51	0.1642
10	279.	39.2	11.38	0.1644
25	428.	41.7	15.66	0.1737
50	564.	44.7	17.84	0.1854
100	724.	48.5	19.20	0.1998
500	1220.	59.8	20.17	0.2400

P R E D I C T I O N I N T E R V A L S

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper

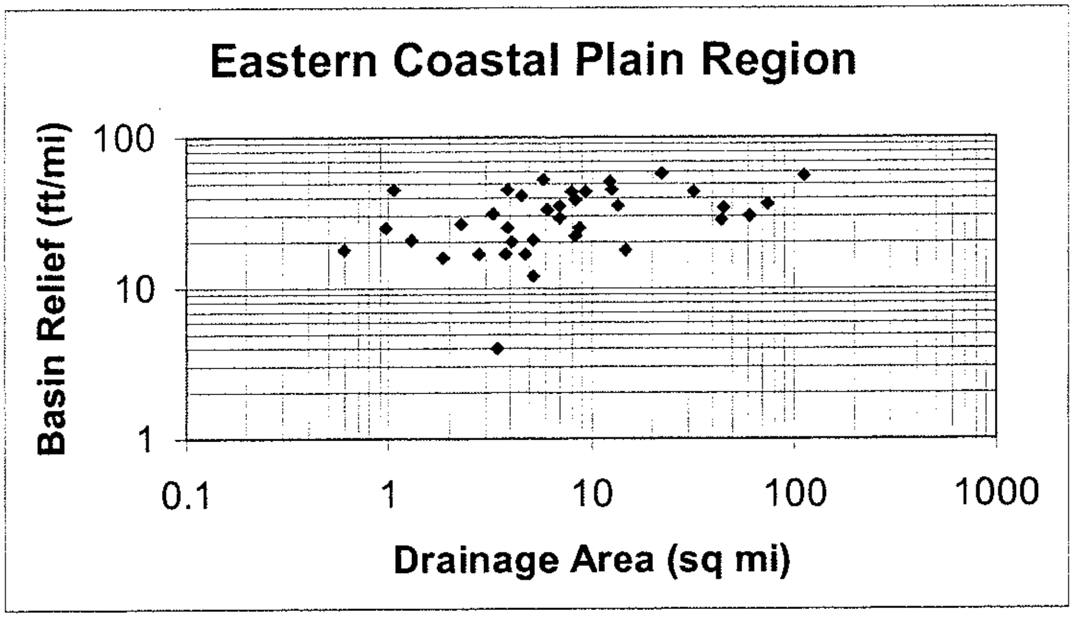
2	68.	119.	60.	135.	46.	177.	40.	204.
5	144.	240.	127.	271.	99.	348.	87.	397.
10	216.	361.	191.	407.	149.	523.	130.	597.
25	327.	562.	287.	639.	221.	832.	192.	957.
50	422.	753.	368.	864.	278.	1150.	239.	1330.
100	530.	988.	457.	1150.	337.	1550.	287.	1820.
500	836.	1770.	700.	2110.	486.	3040.	400.	3690.

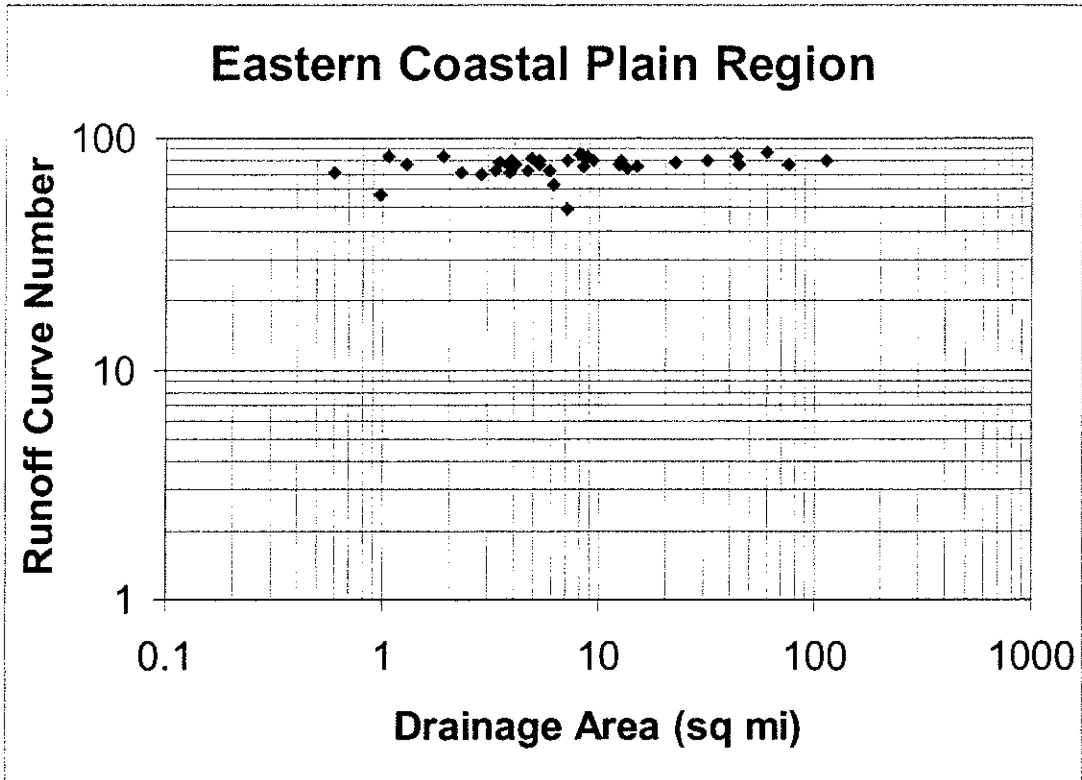
WARNING -- Prediction beyond observed data

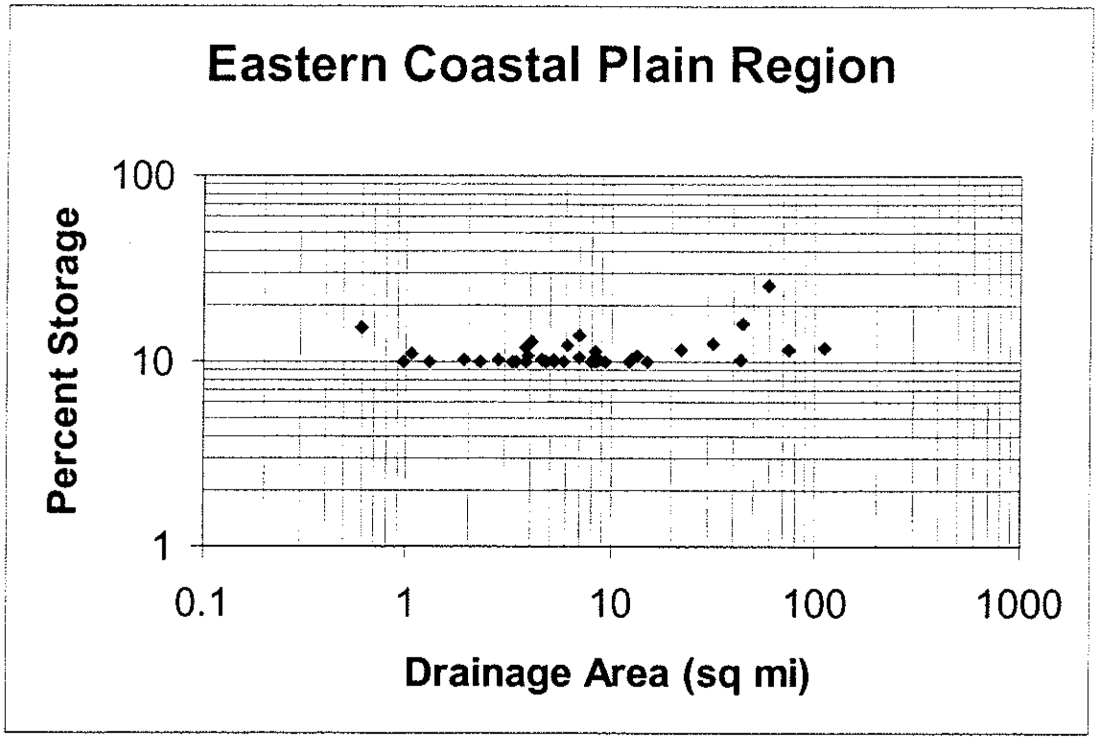
APPENDIX 3

Regressor Variable Hulls for USGS Regression Equations

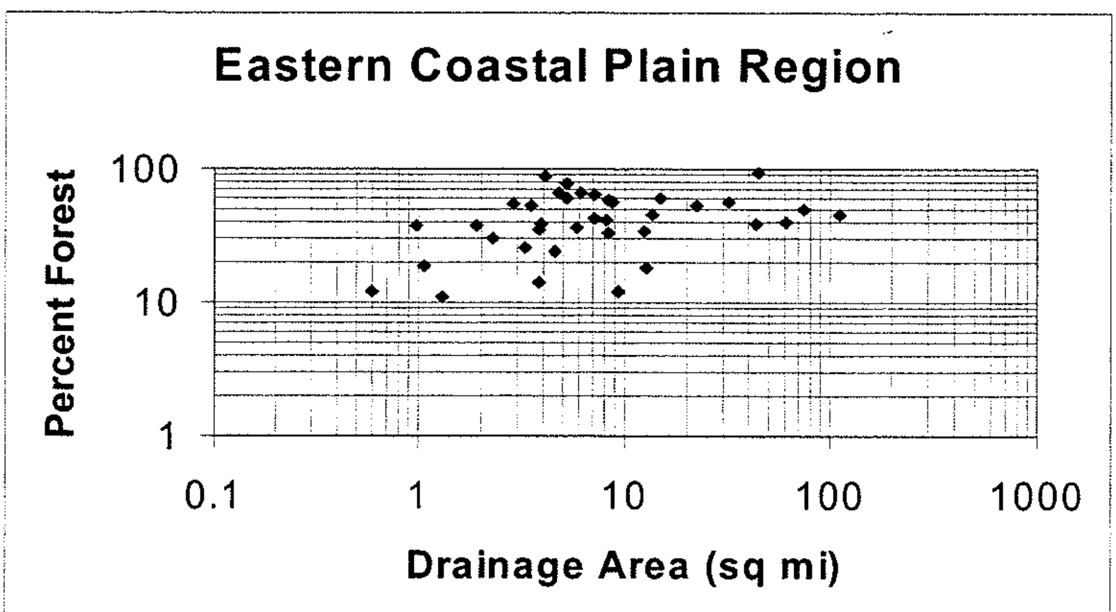
Appendix 3. Plots of the Regression Variable Hulls for four hydrologic regions (excluding the Piedmont Region) in Maryland.



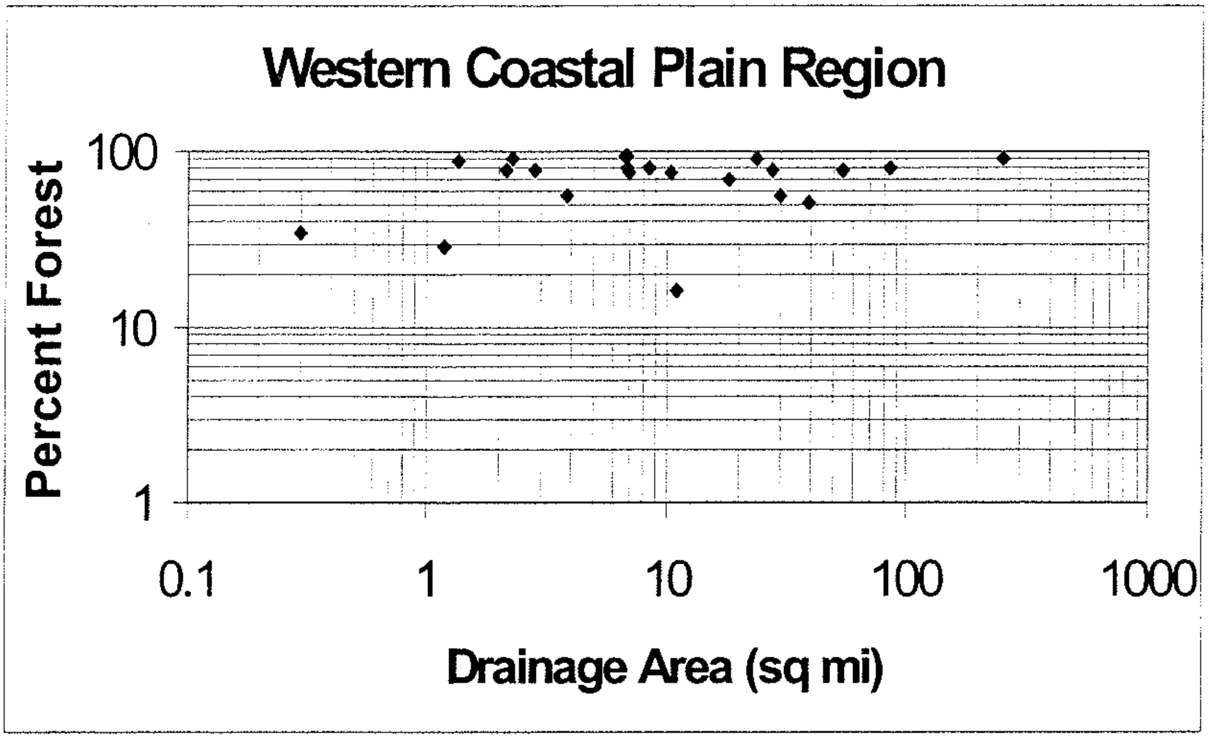

Eastern Coastal Plain Region
Runoff Curve Number
100
10
1
0.1
1
10
100
1000
Drainage Area (sq mi)

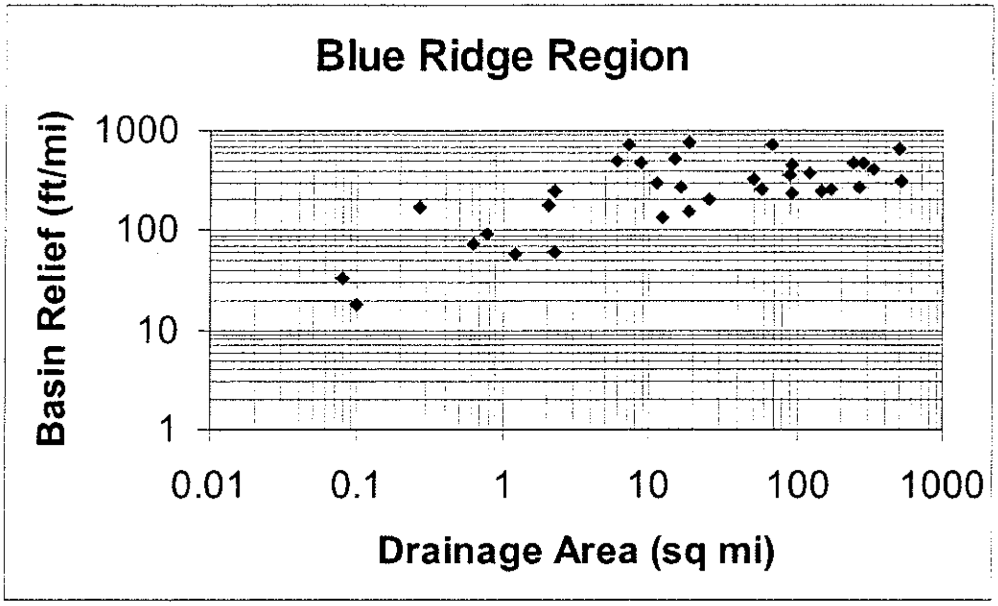


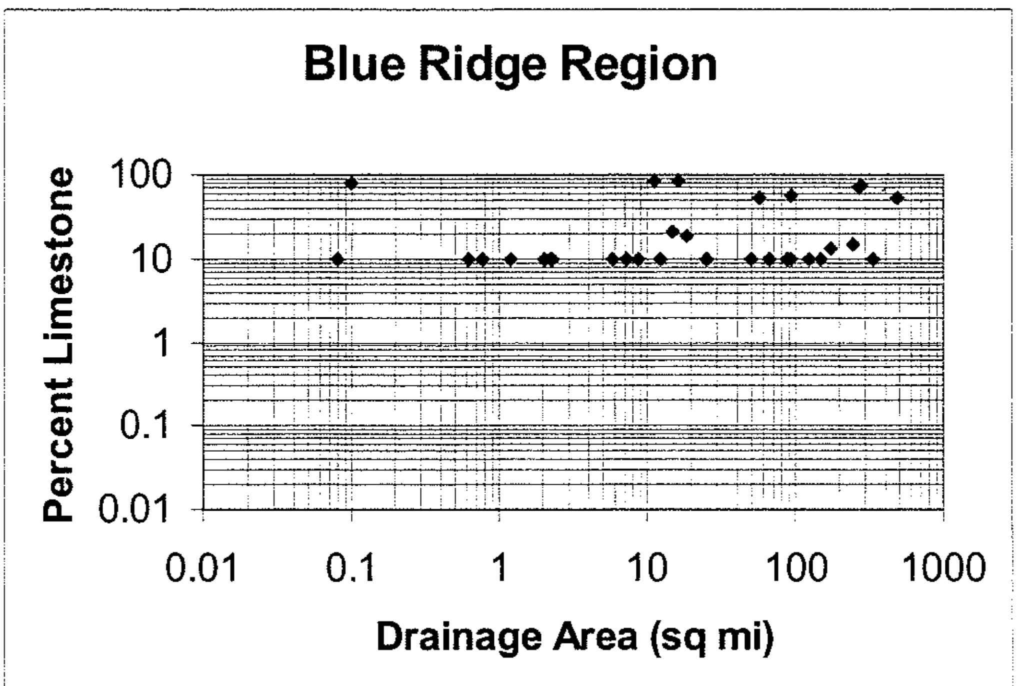
Eastern Coastal Plain Region
Percent Storage
100
10
1
0.1
1
10
100
1000
Drainage Area (sq mi)

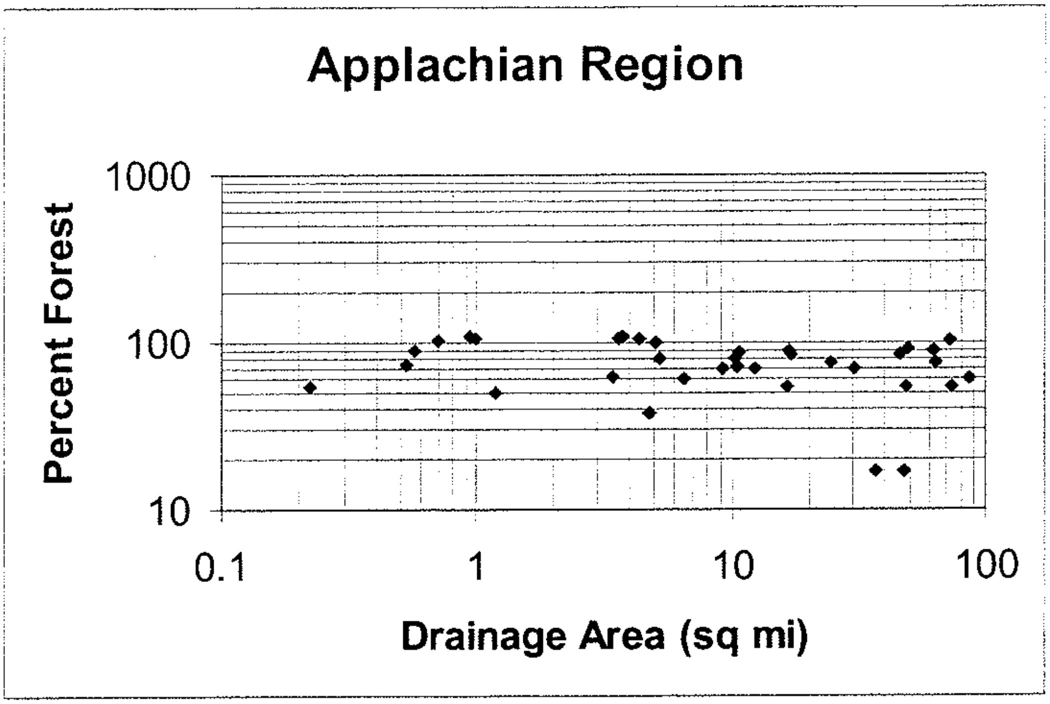


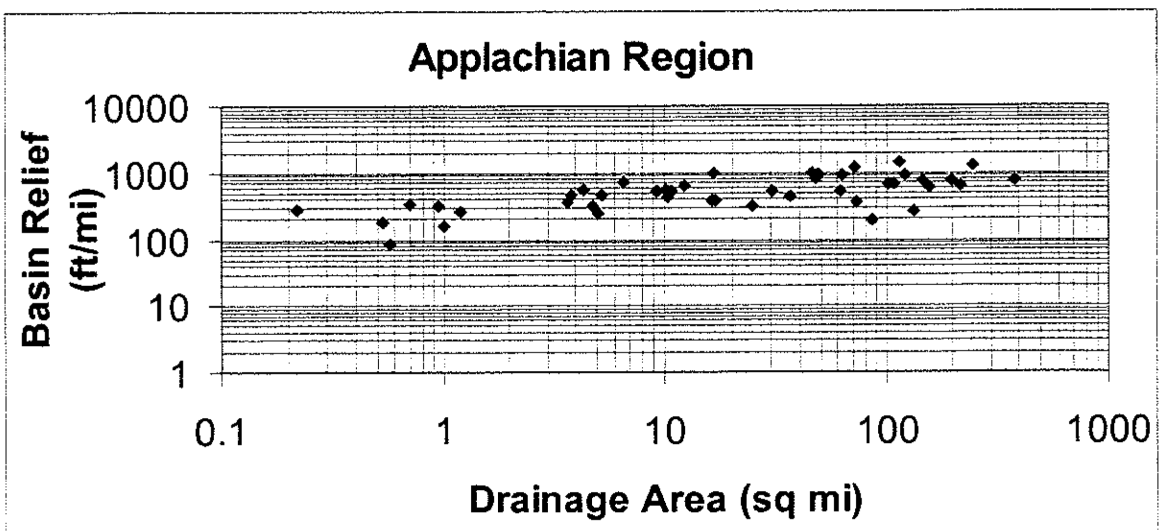
Eastern Coastal Plain Region
Percent Forest
100
10
1
0.1
1
10
100
1000
Drainage Area (sq mi)


Western Coastal Plain Region
Percent Forest
100
10
1
0.1
1
10
100
1000
Drainage Area (sq mi)


Blue Ridge Region
Basin Relief (ft/mi)
1000
100
10
1
0.01
0.1
1
10
100
1000
Drainage Area (sq mi)


Blue Ridge Region
Percent Limestone
100
10
1
0.1
0.01
0.01
0.1
1
10
100
1000
Drainage Area (sq mi)


Applachian Region
Percent Forest
1000
100
10
0.1
1
10
100
Drainage Area (sq mi)


Applachian Region
Basin Relief (ft/mi)
10000
1000
100
10
1
0.1
1
10
100
1000
Drainage Area (sq mi)

APPENDIX 4

Example of Calibration of TR-20 To The Regional Regression Equation When Urbanization is Greater Than 15%

OVERVIEW

Appendix 4 presents three example problems that illustrate the type of analysis that the Panel encourages when using the USGS regression equations to ensure that TR-20 discharges are consistent with Maryland historical streamflow records. Appendix 4A is an example of a watershed that is less than 15% urbanized. Appendices 4B and 4C are watersheds that have more than 15% of their areas in urban land cover categories.

The discussions in Chapters III and IV emphasize that it is difficult to estimate many of the TR-20 input parameters. Errors in these parameter estimates can produce discharges that are much higher or lower than streamflow records indicate are reasonable for Maryland. The steps in refining the initially selected TR-20 input parameters so that the TR-20 discharges are consistent with those of the USGS regression equations cannot be rigorously defined. The steps are unique for each watershed. However, Table 4.1 in Chapter IV provides an indication of the uncertainties associated with some of the parameter estimates and their consequences on hydrograph timing and volumes of runoff. Selection of a parameter to be refined requires the hydrologist to examine his or her data set for indications such as the velocities may be too high or low, the channel section defining over-bank storage is not representative; field investigations indicate that the stream has more meanders and, therefore, is longer than that indicated on a 1:24,000 map; one or more of the curve numbers is too high or low for specific land cover categories in the watershed; there is culvert storage that was not considered in the initial estimate of main stream characteristics; etc.

The three examples of Appendix 4 are intended to illustrate the need to examine your data and, if necessary, return to the field as you refine your parameters. The discussions attempt to explain the thinking that leads to the selection of a parameter for refinement in terms of data inconsistencies and indications that more field work might be required. It is recognized that the approaches followed in the two examples are not the only strategies that could have been adopted.

APPENDIX 4A
CALIBRATION OF 100 YEAR NRCS-TR-20 DISCHARGE AGAINST THE USGS REGRESSION EQUATIONS

STATEMENT OF THE PROBLEM

The purpose of this example is to illustrate an approach that can be followed when calibrating the discharges produced by the NRCS-TR-20 against those predicted by the USGS regression equations. The objective of the calibration is to ensure that the input parameters used to drive the TR-20 define flows that are consistent with the historical stream flow records of Maryland and surrounding states. This example watershed is an 8.67 square mile tributary in the upper end of Seneca Creek in Montgomery County, Maryland. Because of the stream network and the spatial distribution of the land cover, the watershed was divided into three sub-basins as shown in Figure A4.1. The times of concentration were estimated using the segmentation approach in which the travel times of overland, swale, tributary and main channel flows were computed individually and added together.

In this example, the objective is to bring the NRCS-TR-20 discharge produced by the 100 year Type II 24 hour storm into the window defined by the predicted discharge of the USGS regression equations and one standard error above that discharge.

CALIBRATION

Step 1 - Use map and field investigations to define the area, curve number and time of concentration for each of the three sub-watersheds. You must also select a typical cross section and develop the stage-discharge-area table to be used to rout the hydrographs from Sub-watersheds 1 and 2 through the stream reach passing through Sub-watershed 3. The characteristics of each sub-watershed are shown in Table A4.I. The routing sections that were considered are shown in Figure A4.2.

TABLE A4.I
SUBWATERSHED CHARACTERISTICS

Sub-watershed Number	Area (sq mi)	RCN	Time of Conc.(hrs)	Characteristics of Flow Network					
				Main Channel		Tributary		Swale	Overland
				n	V('/sec)	n	V('/sec)		n
1	4.3	72	1.702	.035	4.1	.040	4.5	------	.40
2	3.0	71	.494	.035	11.1	-----	----	Unpaved	----
3	1.4	71	.659	.035	6.1	040	3.8	------	----

Step 2 - Run the USGS regression equations and determine the standard error on Sub-watershed 1 using the Tasker program..

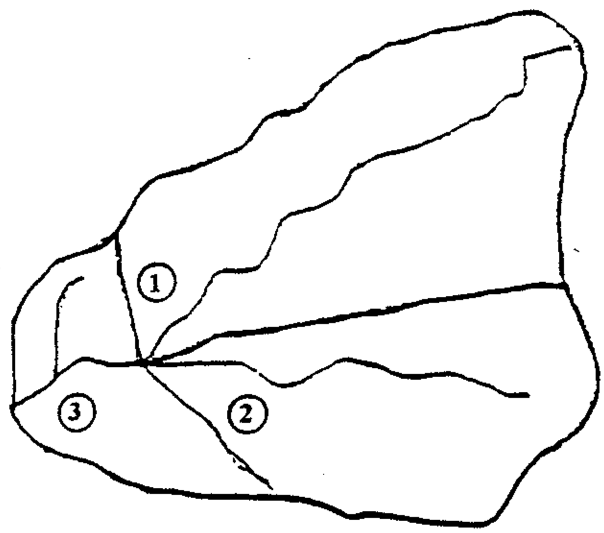
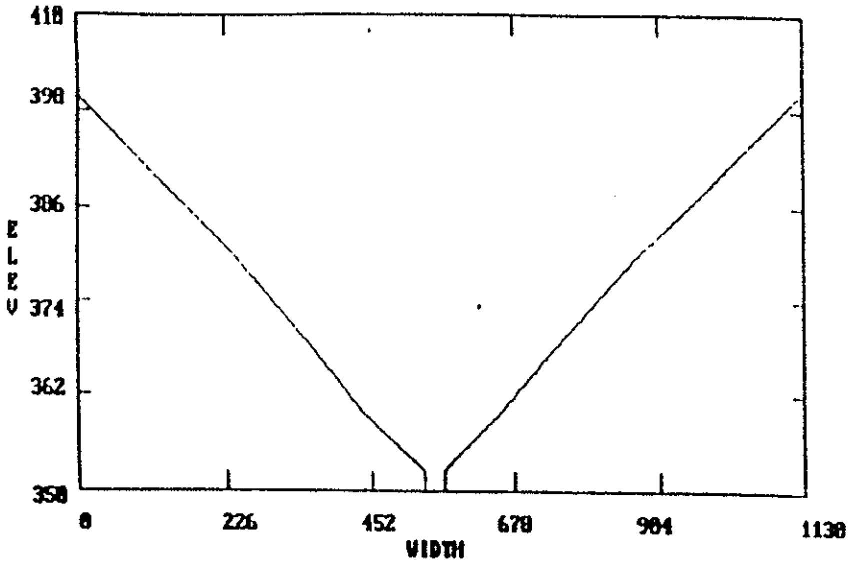
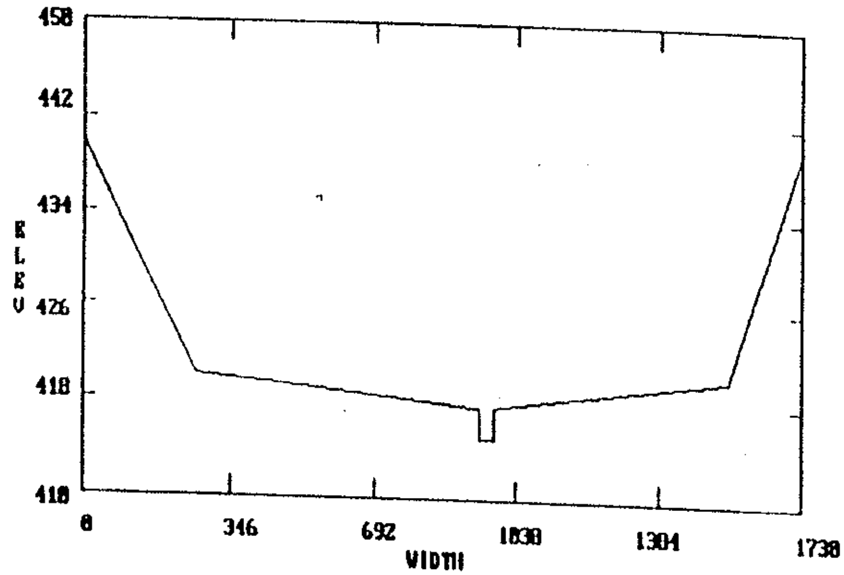


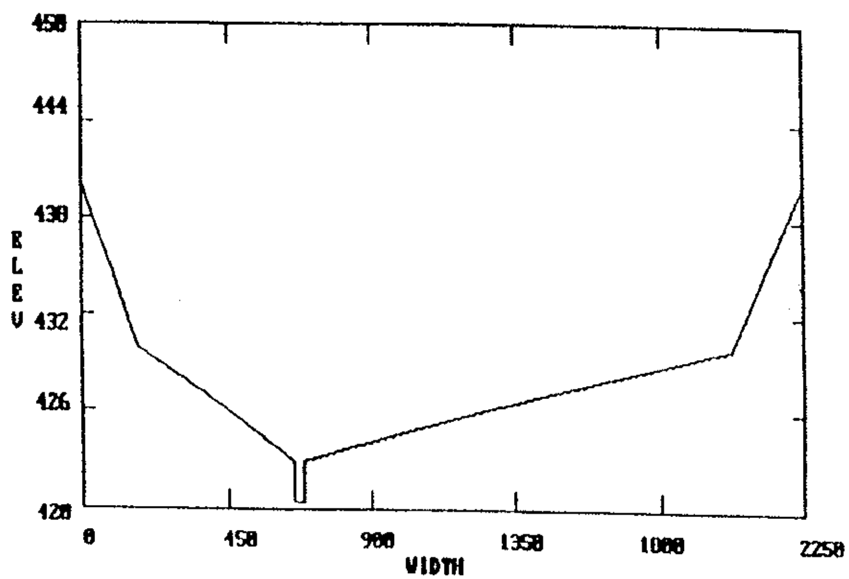
FIGURE A4.1
EXAMPLE WATERSHED DIVIDED INTO THREE SUB-BASINS



TYPICAL SECTION A



TYPICAL SECTION B



TYPICAL SECTION C

FIGURE A4.2
SECTIONS CONSIDERED FOR DEVELOPMENT
OF STAGE-DISCHARGE-AREA RELATION ALONG
ROUTING REACH THROUGH SUB-WATERSHED 3

Step 3 - Run the TR-20 on Sub-watershed 1 using the area, curve number and time of concentration developed in Step1 and shown in Table A4.I.

Step 4 - The synthetic frequency series for the USGS and the TR-20 are shown in Table A4.II. Note that the TR-20 100 year peak of 3816 cfs is less than the 4730 cfs upper bound of the window.

TABLE A4.II
SUB-WATERSHED #1 EXISTING LAND USE
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNS. & TR-20
TEST 1

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS)			NRCS
		LOW Q	ESTIMATE	HIGH Q	TR-20
2	39.5	325	475	695	930
5	36.0	635	899	1270	1561
10	35.6	916	1290	1830	1905
25	37.5	1330	1920	2750	2638
50	40.0	1680	2480	3640	3217
100	43.3	2070	3130	4730	3816

Step 5 - Even though the 100 year TR-20 peak meets the USGS window criteria, check the flow characteristics in Table A4.I to ensure that the conditions are realistic. Note that the in-bank main channel and tributary velocities generated by the roughness coefficients and cross section selected are 4.1 and 4.5 feet per second, reasonable estimates for the type of bank material found in the field investigations.

Step 6 - Repeat Steps 2 - 5 for Sub-watershed 2. The results are shown in Table A4.III.

TABLE A4.III
SUB-WATERSHED #2 EXISTING LAND USE
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNS. & TR-20
TEST 1

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS)			NRCS
		LOW Q	ESTIMATE	HIGH Q	TR-20
2	39.8	271	397	582	1472
5	36.3	533	757	1080	2492
10	36.1	772	1100	1550	3053
25	38.0	1130	1630	2350	4247
50	40.6	1430	2110	3120	5193
100	43.9	1760	2680	4070	6172

Step 7 - The 100 year 6172 cfs TR-20 discharge is much higher than the upper bound 4070 cfs upper bound of the USGS equations. Without some effort toward calibration, the design process would go forward with the 6172 cfs discharge from Sub-watershed 2. As shown on Table 4.3 in Chapter IV, incorrect estimates of any number of parmeters could be causing the high TR-20 discharge. A review of Table A4.I shows a main channel velocity of 11.1 feet per second. Although the channel slope in Sub-watershed 2 is 0.013731 as opposed to 0.0077458 in Sub-watershed 1, your field investigations indicate that the channel could not support an in-bank velocity this high without massive erosion. Also, as described in Chapter III and summarized in Table 4-3 of Chapter IV, there is a fairly high risk of error in the estimation of the Manning roughness coefficient. Based on the lack of serious erosion found in the field investigation and a reexamination of the channel lining, you conclude that your Manning roughness is probably too low.

Step 8 - Change the main channel roughness to n = 0.045 and repeat steps 2 - 5. The lower time of concentration produces the results are shown in Table A4.IV.

TABLE A4.IV
SUB-WATERSHED #2 EXISTING LAND USE
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNS. & TR-20
TEST 2

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS) LOW Q	ESTIMATE	HIGH Q	NRCS TR-20
2	39.8	271	397	582	1352
5	36.3	533	757	1080	2289
10	36.1	772	1100	1550	2803
25	38.0	1130	1630	2350	3900
50	40.6	1430	2110	3120	4769
100	43.9	1760	2680	4070	5668

Step 9 - Note that the 100 year peak has been brought down to 5668 cfs, but, this is still above the desired window. Your TR-20 data shows that the main channel velocity has only been reduced to 8.7 feet per second, which you think is still too high. You are confident that the bank full roughness cannot be higher than 0.045. Thus, you have to suspect that the "typical" cross section is producing the unrealistic velocity. You check your notes and find that you selected your "typical" in-bank cross section very close to the outlet of the sub-watershed and, therefore, conclude that it might not be "typical". You go back to the field and to your maps and select a cross section that is located about half way up the tributary. This section has a much lower hydraulic radius and, consequently, should produce a lower velocity than that produced by the section near the outlet.

Step 10 - You use the new section to compute the channel velocity and repeat steps 1 - 5.. The results of this change are shown in Table A4.V.

Step 11 - Now the TR-20 100 year peak is 3863 cfs - well under the 4070 USGS upper bound. Your data show that the channel velocity is now 4.9 feet per second which is consistent with the other two sub-watersheds and realistic for the channel lining. The time of concentration has been increased from 0.494 hours to 1.022 hours. You accept the new n = 0.045 and the new cross section as being more appropriate for Sub-watershed 2. Because our objective was to calibrate the 100 year peak, the NRCS-TR-20 input parameters are accepted as satisfactory for the existing watershed conditions. Simulation of the watershed under ultimate development will be accomplished by changing the RCN and flow network parameters to reflect those future conditions

TABLE A4.V
SUB-WATERSHED #2 EXISTING LAND USE
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNS. & TR-20
TEST 3

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS) LOW Q	ESTIMATE	HIGH Q	NRCS TR-20
2	39.8	271	397	582	921
5	36.3	533	757	1080	1560
10	36.1	772	1100	1550	1911
25	38.0	1130	1630	2350	2658
50	40.6	1430	2110	3120	3251
100	43.9	1760	2680	4070	3863

-- NOTE --

[Typically, you may get a good calibration for the 100 year discharge, but, you may still have a significant over prediction at the 2, 5, and 10 year events. This problem is shown in Table A4.V. However, if a six hour design storm, instead of the 24 hour storm, is used as the TR-20 input for the more frequent events, the agreement can be significantly improved as shown by comparing the 2, 5, and 10 events in Tables A4.V and A4.VI]

TABLE A4.VI
SUB-WATERSHED 2 EXISTING LAND USE
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNS. & TR-20
(Impact of using 6 hr storm for 2,5,& 10 yr. Events)

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS) LOW Q	ESTIMATE	HIGH Q	NRCS TR-20
2	39.8	271	397	582	404
5	36.3	533	757	1080	951
10	36.1	772	1100	1550	1363
25	38.0	1130	1630	2350	2658
50	40.6	1430	2110	3120	3251
100	43.9	1760	2680	4070	3863

Step 12- Repeat steps 1 - 5 for sub-watershed 3 and make any necessary adjustments as in steps 7 - 9. In this example, calibration procedure shows that the NRCS-TR-20 input parameters for Sub-watershed 3 in Table A4.I are acceptable.

Step 13 - The final NRCS-TR-20 input parameters following the calibration are shown in Table A4.VII.

TABLE A4.VII
NRCS-TR-20 INPUT PARAMETERS
AS REFINED THROUGH CALIBRATION

Sub-watershed Number	Area (sq mi)	RCN	Time of Conc.(hrs)
1	4.3	72	1.702
2	3.0	71	1.022
3	1.4	71	.659

Step 13 - Run the USGS regression equations for the entire watershed. The results are shown in Table VIII The TR-20 peak should be under the 7070 cfs upper bound.

TABLE VIII
ENTIRE WATERSHED EXISTING LAND USE
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNUATIONS

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS) LOW Q	ESTIMATE	HIGH Q
2	39.2	516	754	1100
5	35.8	989	1400	1970
10	35.2	1410	1990	2800
25	37.0	2030	2910	4160
50	39.5	2550	3730	5460
100	42.7	3120	4690	7070

Step 14 - Run the TR-20. The three routing sections shown in Figure A4.2 were explored. Typical sections B and C have very broad flood plains and, therefore would provide significant peak flow attenuation through over bank storage. However, map investigations indicated that these two sections were isolated and could not be considered typical of the stream reach. After careful consideration Section A that has minimal over bank storage for attenuation was deemed to be most representative. In the first effort, the left and right over bank roughnesses were n = 0.09 and 0.15, respectively. These values gave a peak 100 year discharge of 7623 cfs. When the left over bank n = 0.09 was increased to 0.15, the 100 year peak decreased to 7032 cfs, slightly below the 7070 upper bound of Table VIII . Table A4.IX is a portion of the TR-20 output produced by the sub-watershed parameters verified by Steps 1 - 12 and Typical Routing Section A.

TABLE A4.IX
PORTION OF TR-20 OUTPUT
FOR ENTIRE WATERSHED

```
****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

B TR-20                          FULLPRINT                            NOPLOTS
TLE       CURRENT EXAMPLE
TLE
 XSECTN    003                       1.0       352.543
                                    350.           0.0          0.0
                                 351.271       121.292       40.147
                                 352.543       367.132       80.294
                                    360.      4780.404     1042.746
                                    380.      56733.58     9422.301
                                    400.      190798.2     26823.63
) ENDTBL
5 RUNOFF 1    1       5     4.261364       72.1633     1.702367                  1
5 SAVMOV 5 169    5   1                                                          1
5 RUNOFF 1    2       5     3.048955      71.43059     1.022005                  1
5 ADDHYD 4    3   1 5 6                                                          1
5 REACH  3    3   6   7     4977.227                                             1
5 RUNOFF 1    3       6     1.355888      70.97884     .6592048                  1
5 ADDHYD 4    3   6 7 5                                        101010101010      1
  NDATA                                                                          1
7 INCREM 6                        .1                                             2
7 COMPUT 7    1     3            0.0      7.100001         1.02 2    1    1      2
  ENDCMP 1                                                                       2
  ENDJOB 2                                                                       2
*******************************END OF 80-80 LIST*******************************

UMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AN

SECTION/              DRAINAGE
TRUCTURE                AREA       STORM NUMBERS..........
 ID                   (SQ MI)            1

XSECTION    3            8.67
___________________________________
 ALTERNATE      1                   7032.25
ND OF   1 JOBS IN THIS RUN
```

If one attempts to accomplish these calibration steps using traditional methods, it will be very difficult and expensive except in the case of very small watersheds. The above steps were aided using GIS technology that has been developed by the Maryland State Highway Administration. Such technology is becoming increasingly available. With GIS approaches, refining the TR-20 input parameters through the above calibration steps can be completed in a fraction of the time that used to be required to produce an unverified set of TR-20 inputs by traditional paper map-based means.

APPENDIX 4B
CALIBRATION OF TR-20 WITH USGS REGRESSION EQUATIONS WHEN MORE THAN 15% OF THE WATERSHED IS IURBANIZED

STATEMENT OF THE PROBLEM

In this example, a 7.4 sq. mile watershed in Montgomery County that has 45.97% of its area in urban land cover categories is under investigation. The land cover percentages are shown in Table B4.I and the distribution of the NRCS Hydrologic Soil Categories is shown in Table B4.II. The TR-20 input parameters that have been estimated from field and map investigations are presented in Table B4.III.

TABLE B4.I
EXISTING LAND COVER

Category	Acres	Percent
Residential (.2-2 DU/ac)	36.73	0.78
Residential (>2-8 DU/ac)	803.49	16.97
Residential (>8 DU/ac)	835.63	17.65
Commercial/Industrial	385.67	8.15
Institutional	114.78	2.42
Forest	886.13	18.72
Brush	243.34	5.14
Water	9.18	0.19
Cropland	18.37	0.39
Grass	1400.37	29.58
Total	4733.70	100.00

TABLE B4.II
HYDROLOGIC SOIL GROUPS

Group	Percent of Watershed
A	0.00
B	89.82
C	2.23
D	7.95

TABLE B4.III
PARAMETERS USED TO DEFINE TR-20 INPUTS FOR EXISTING CONDITONS

Area (sq mi)	RCN	Time of Conc.(hrs)	Characteristics of Flow Network					
			Main Channel		Tributary		Swale	Overland
			n	V('/sec)	n	V('/sec)		n
7.4	69.6	1.36	035	5.6	-----------		Unpaved	.50

If, instead of calibrating with the USGS regression equations, we simply input the 100 year 24-hour Type II NRCS design storm to the TR-20 using the characteristics of Table B4.II, we get the synthetic frequency series of Table B4.IV. The 100 year flood for existing conditions is estimated at 7140 cfs. At this point, there is not way to know if this is consistent with Maryland conditions even though the input parameters appear to be well chosen.

TABLE B4.IV
TR-20 ESTIMATED DISCHARGES
WITHOUT CALIBRATION

Return Interval	Discharge (cfs)
2	1607
5	2788
10	3443
25	4851
50	5974
100	7140

We want to use the USGS regression equations as guides to ensure that the TR-20 model Produces results that are representative of Maryland conditions. The problem is that the USGS equations were developed using data from watersheds that had a maximum of 15% urbanization. The procedure described in this example performs the calibration in the following steps.

Step 1 – The land cover for the watershed is converted to an estimated pre-developed condition. In this example, there are no "near by" undeveloped watersheds. The natural condition land cover will be selected from Table 4.2 in Chapter IV.

Step 2 – A "predevelopment estimate" of the stream cross section developed from regression equations that estimate the width, depth and area of bank full channel conditions to the drainage area of a natural watershed.

Step 3 – The USGS regression and Tasker equations are applied to the watershed for this pre-developed condition.

Step 4 – The TR-20 is implemented using an estimated curve number for the pre-developed condition and calibrated against the USGS peaks following the same procedure as illustrated in Appendix 4-A.

Step 5 – After the pre-developed calibration is acceptable, an estimate of the existing, or current, watershed condition TR-20 peaks is developed by multiplying the pre-developed peaks by the ratios of the runoff volumes defined by the two curve numbers and adjusting the time of concentration to reflect the current drainage network. In this example, the existing condition bank full cross sections have been changed from their natural conditions as a consequence of urbanization. Thus, in addition to the rationing of the runoff volumes.

we will have to adjust to reflect the shorter time of concentration of the urbanized existing condition.

As in the example of Appendix 4-A, we will calibrate only on the 100 year peak.

CALIBRATION

Because there is no near-by watershed that remains in a natural condition, the land covers for Montgomery County in Table 4.4 are assigned to the watershed as shown by Table B4.V.

TABLE B4.V
PREDEVELOPMENT LAND COVER

Category	Sq. miles	Percent
Forest	3.00	40.6
Brush	.30	4.0
Cultivated	2.65	35.8
Grass	1.45	19.6
Total	7.40	100.0

Following the example of Table 4.5 in Chapter IV and using the soil and land cover data of Tables B4.II and B4.V, the predevelopment curve number is estimated at 64.2. A regression equation similar to that displayed in Figure 3.9 of Chapter III is used to define a predevelopment bank full discharge that will be used to estimate the time of concentration. The initial roughness estimate will remain at n = 0.35. This gives a channel velocity of 4.8 feet/sec and a time of concentration of 1.48 hours.

Table B4.VI shows the USGS and TR-20 predevelopment flood frequency series.

TABLE B4.VI
ESTIMATED PREDEVELOPMENT CONDITION
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNS. & TR-20
TEST 1

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS) LOW Q	ESTIMATE	HIGH Q	NRCS TR-20
2	39.3	387	565	825	1101
5	35.6	754	1070	1510	2095
10	35.3	1080	1530	2150	2646
25	37.0	1570	2240	3210	3851
50	39.5	1970	2880	4210	4828
100	42.7	2410	3620	5460	5856

The 100 year TR-20 estimate of 5856 cfs is above the upper USGS bound of 5460 cfs. As indicated by Table 4.3 in Chapter IV, incorrect estimates of any number of parameters could be causing the high TR-20 discharge. Because of the high degree of uncertainty associated with estimates of the Manning roughness coefficients and its large impact on the time of concentration, we will raise to roughness of the channel to n = 0.40 as a first step to reconciling the USGS and TR-20 100 year peaks. Changing the roughness to n = 0.040 raises the time of concentration from 1.48 to 1.60 hours. The results of the change are shown in Table B4.VII.

TABLE BE, VII
ESTIMATED PREDEVELOPMENT CONDITION
FLOOD FREQUENCY SERIES GENERATED
WITH USGE REGRESSION EQNS. & TR-20
TEST 2

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS) LOW Q	ESTIMATE	HIGH Q	NRCS TR-20
2	39.3	387	565	825	1037
5	35.6	754	1070	1510	1973
10	35.3	1080	1530	2150	2491
25	37.0	1570	2240	3210	3626
50	39.5	1970	2880	4210	4546
100	42.7	2410	3620	5460	5513

The 100 year TR-20 estimate is brought down to 5513 cfs, still higher than the recommended 5460 cfs USGS upper bound. The channel velocity has been reduced from 4.8 feet/sec to 4.1 feet/sec through the roughness adjustment. Let us say that we are reluctant to raise the roughness any higher.

We recognize that the cultivated agriculture of Montgomery has been of high quality for many decades. The "Cultivated"default curve numbers we are using are for "row crops, good condition" RCN = 67, 78, 85, and 89 for the A, B, C and D Groups, respectively. As discussed in Chapter III and summarized in Table 4.3 of Chapter IV, these RCN's were derived from a national data set that has considerable scatter. Historically, a considerable amount of Montgomery County cropland has been in "small grain" with a significant acreage being contoured. Table 3.2 of the NRCS-TR-55 Manual list RCN's as low as 58, 69, 77 and 80 respectively for the higher quality agricultural practices. Table B4. VIII summarized the RCN's we are using and the lower values associated with high quality agriculture practices.

Because we know that the Montgomery County practices are quite high, we are justified in making some adjustment to the RCN defaults that we are using. Taking a conservative approach of making relatively small changes, we adjust the RCN's of the "Cropland" category to 67, 75, 83 and 87. The TR-20 is implemented with these new RCN's to give the frequency series of Table B4. IX.

TABLE B4.VIII
RANGE OF ROW CROP/SMALL GRAIN
CURVE NUMBERS FROM NRCS-TR-55

Category	Curve Number for Soil Group A	B	C	D
Straight Row Crops, good condition	67	78	85	89
Small Grain, contoured & terraced with crop residue	58	69	77	80

TABLE B4.IX
ESTIMATED PREDEVELOPMENT CONDITION
FLOOD FREQUENCY SERIES GENERATED
WITH USGS REGRESSION EQNS. & TR-20
TEST 3

RETURN INTERVAL	STD ERROR OF ESTIMATE (%)	USGS DISCHARGE (CFS) LOW Q	ESTIMATE	HIGH Q	NRCS TR-20
2	39.3	387	565	825	935
5	35.6	754	1070	1510	1859
10	35.3	1080	1530	2150	2361
25	37.0	1570	2240	3210	3467
50	39.5	1970	2880	4210	4367
100	42.7	2410	3620	5460	5315

We have moved the TR-20 100 year flood below the 5460 USGS upper bound. The narrow objective of simply being below the upper bound of 5460 cfs been met. Because we reduced the RCN's by only a small amount, one could probably lower the 5315 cfs TR-20 flood closer to the 3620 cfs "USGS Best Estimate" by further reducing the RCN's. Whether to proceed with steps to further reduce the 5315 cfs is a decision that the hydrologist must make based on his or her judgment and the risks associated with the consequences of a design failure.

Table B4.X lists the frequency series for the predevelopment and the existing, or current, watershed conditions. The existing condition frequency series was developed with a new main stream n = .040 that was indicated by the predevelopment condition and the typical bank full cross section that had been developed through field observations. If the cross section had not been impacted by urbanization, the existing frequency series would had been created by simply ratioing the runoff volumes defined by the two curve numbers. Note that the 6462 cfs of Table B4.X is somewhat less than the 7140 cfs of Table B4.IV that would have been used if there had been no calibration.

Item A

May 15, 2002

Len: Just got around to looking into the issues that you raise. First I agree with Will that Step 6 on page 61 needs revising. I went a little too fast on that one. A more appropriate reading might be :

> Estimate a set of TR-20 discharges for the actual existing watershed conditions by multiplying the TR-20 discharges of step 5 by the ratios of the pre-developed and existing condition volumes of runoff defined by the two curve numbers. In some cases........

The problem that you and Will recognize can be illustrated by a watershed with an exiting RCN = 70. Suppose the land covers suggested by Table 4.2 on page 62C give a predeveloped RCN = 65 by the procedure of page 62F. The ratio of the RCN's is 1.08. Thus, with the current Step 6 recommendation, we would multiply all discharges from 2 year to 500 year by a constant 1.08. However, the volumes of runoff associated with different volumes of rainfall are not a constant. For example:

24 Hr. 2 year rainfall = 3 inches
Volume of Runoff for RCN = 65 is 0.51 inches
Volume of Runoff for RCN = 70 is 0.71 inches
The multiplying ratio for this two year flood should be .71/.51 = 1.39

24 Hr. 100 year rainfall = 7 inches
Volume of Runoff for RCN = 65 is 3.11 inches
Volume of Runoff for RCN = 70 is 3.62 inches
The multiplying ratio for this 100 year flood should be 3.62/3.11 = 1.16

Also, can you send me a new set of the attachments that were in one of your emails. I downloaded them, but cannot find them.

I am not sure how much I will be able to do. We plan to be on the road almost all of June and there are "in and out" trips throughout the fall.

Bob Ragan

Item B

Message to Thomas May 21 or 22

Will:

You are correct in your email of May 16. There are two methods in the Task Force Report for calibration when the percent of urbanization is greater than 15%. The method described in pages 61-62F was programed into the old GISHYDRO during the task force discussions. In that version of GISHYDRO, the existing land cover was tabulated and existing curve number computed. If the cumulative urban classes were greater than 15% the user was asked if the lowest density residential was to be considered "urban" in this particular watershed. Even if this category was dropped as an urban class, suppose the percentage still remained above 15%. If so, a "predevelopment" land cover was assigned in accordance with the county that the watershed was in and the distributions of Table 4.2 beginning on page 62. These percentages were displayed to the user for the Tasker computations and GISHYDRO computed the "predeveloped" RCN as illustrated in Table 4.3 on page 62F. The Tasker discharges were input to GISHYDRO and the "adjusted" calibrations presented by the (now shown to be incorrect) ratioing of step 6 on page 61. (Step 6 should now be ..."

I do not believe the new GISHYDRO has this subroutine. But the computations can be done by hand fairly easily as shown in Table 4.3 on page 62F. Page 62F follows several of the examples from TR-20 and TR-55 manuals. The distributions of Table 4.2 on page 62C were developed with the old GISHYDRO by scanning and tabulating all of the non-urban land covers in each county using the 1985 database.

One item that is not being considered is the time of concentration. With 15% urbanization, it may have changed and should be considered. Also, some of the relatively small watershed data sent to us had times of concentration of 16 hours or so "based on the manual" and around 4 hours using the SCS lag equation. I assume that these high times of concentration are being obtained with the equations of Appendix 5. I know you really believe in this regression approach. With more data and peer review it should be an excellent approach. For the present, however, I think the users should stay with the hydraulic or SCS lag equation approaches.

I have no strong feelings about THE solution to the 15% problem. We can keep both approaches or we can drop one of them. I am 100% behind your statement that the RCN should not go down for the "predeveloped" case, even though, there could be cases where the peak discharge could go down because of good design and storm water management. These changes would be picked up in the hydraulic computations.

===========================

May 15, 2002

Len: Just got around to looking into the issues that you raise. First I agree with Will that Step 6 on page 61 needs revising. I went a little too fast on that one. A more appropriate reading might be :

> Estimate a set of TR-20 discharges for the actual existing watershed conditions by multiplying the TR-20 discharges of step 5 by the ratios of the pre-developed and existing condition volumes of runoff defined by the two curve numbers. In some cases........

The problem that you and Will recognize can be illustrated by a watershed with an exiting RCN = 70. Suppose the land covers suggested by Table 4.2 on page 62C give a predeveloped RCN = 65 by the procedure of page 62F. The ratio of the RCN's is 1.08. Thus, with the current Step 6 recommendation, we would multiply all discharges from 2 year to 500 year by a constant 1.08. However, the volumes of runoff associated with different volumes of rainfall are not a constant. For example:

24 Hr. 2 year rainfall = 3 inches
- Volume of Runoff for RCN = 65 is 0.51 inches
- Volume of Runoff for RCN = 70 is 0.71 inches
- The multiplying ratio for this two year flood should be .71/.51 = 1.39

24 Hr. 100 year rainfall = 7 inches
- Volume of Runoff for RCN = 65 is 3.11 inches
- Volume of Runoff for RCN = 70 is 3.62 inches
- The multiplying ratio for this 100 year flood should be 3.62/3.11 = 1.16

Also, can you send me a new set of the attachments that were in one of your emails. I downloaded them, but cannot find them.

I am not sure how much I will be able to do. We plan to be on the road almost all of June and there are "in and out" trips throughout the fall.

Bob Ragan

TABLE B4.X
ESTIMATED TR-20 FLOOD FREQUENCY SERIES
FOR PREDEVELOPMENT AND EXISTING CONDITIONS

Return Interval (Yrs)	Predevelopment Calibration (cfs)	Existing Condition Calibration (cfs)
	RCN = 64.1	RCN = 69.6
	T_c = 1.60 hrs.	T_c = 1.56 hrs.
2	935	1454
5	1859	2523
10	2361	3116
25	3467	4390
50	4367	5486
100	5315	6462

APPENDIX 4C

EXAMPLE OF CALIBRATION OF TR-20 TO THE REGIONAL REGRESSION EQUATION WHEN URBANIZATION IS GREATER THAN 15%

Definitions: Pre-Urban Development = Watershed land use with urbanized area removed (used for calibration to Regional Regression Equations), Existing Development = Current land use, Ultimate Development = Future land use derived from zoning and comprehensive planning maps.

Given: Watershed location: Western Coastal Plain
- Drainage area to point of study = 10.44 square miles
- No gage on the watershed
- Forest cover is 32% existing
- T_c for TR-20 input calculated using the lag-RCN method of HEH-4 by NRCS
- Regional Equation from USGS Report 95-4154, 1996
- RCN values and forest cover derived from GIS-Hydro by University of MD
- Ultimate land use from County zoning maps and comprehensive plan

Find: Q_{100} peak flow for existing and ultimate development

STEP 1: Prepare base TR-20: subdivide watershed so that urbanized areas are isolated, compute existing RCN values, ultimate RCN values, construct representative cross sections for reach routings (Figure E1), compute Tc using the lag-RCN method (Figure E2). Figure E3 shows the Base TR-20 schematic diagram and data.

STEP 2: Derive Pre-Urban RCN values for those sub-areas that are urbanized (high density residential of 4 du/ac or more, commercial, industrial, or other high impervious land uses). Pre-Urban RCN values can be calculated from the remainder of the watershed or from areas surrounding the urban areas. The Pre-Urban RCN values are shown in Figure E3. Figure E4 shows the input data for the Base TR-20 – Pre-Urban Watershed.

STEP 3: Determine the applicability of the Regional Regression Equation. Compute the Regional Regression Equation expected value and prediction limits using the USGS computer program (Figure E5).

STEP 4: Calibrate the Pre-Urban TR-20 model so that the Q_{100} peak falls within the Regional Regression Equation prediction limit. (Figure E6)

STEP 5: Replace the Pre-Urban calibrated model RCN values and reach representative cross sections (if appropriate) with those computed for Existing Development and calculate existing Q_{100} peak flow. Figure E7 shows the TR-20 input data for the existing model with calibration results.

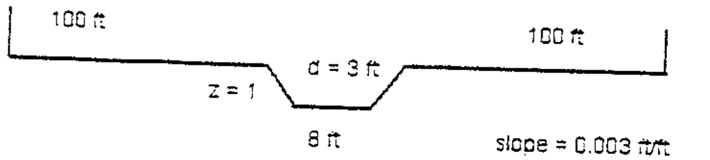
STEP 6: Replace the existing calibrated model RCN values with those RCN values computed for ultimate development and compute the Q_{100} peak flow (Figure E8). The final results are shown in the Summary Table below.

SUMMARY TABLE
TR-20 Results

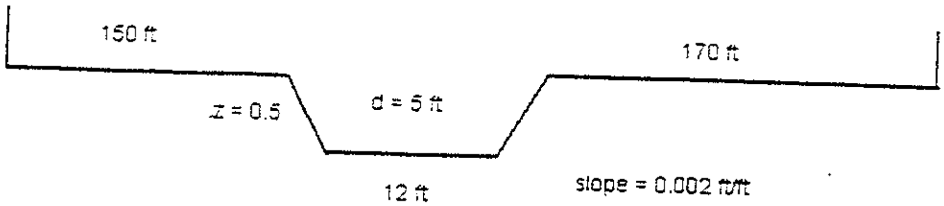
Q_{100} Target Range = 3020 to 5670 cfs (for Pre-Urban watershed)

Base TR-20 model for Pre-Urban watershed	*6220 cfs*
Calibration: increase base tc by 5%	6006 cfs
Calibration : increase base tc by 10%	5809 cfs
Calibration: increase base tc by 10% and n to 0.07 from 0.06	5787 cfs
Calibration: increase base tc by 15% and n to 0.07 from 0.06 (*final calibration*)	**5573 cfs**
Replace RCN values and reach cross section for existing land use and calculate Q_{100}	5977 cfs Existing Q
Replace RCN values for ultimate land use and calculate Q_{100}	6222 cfs Ultimate Q

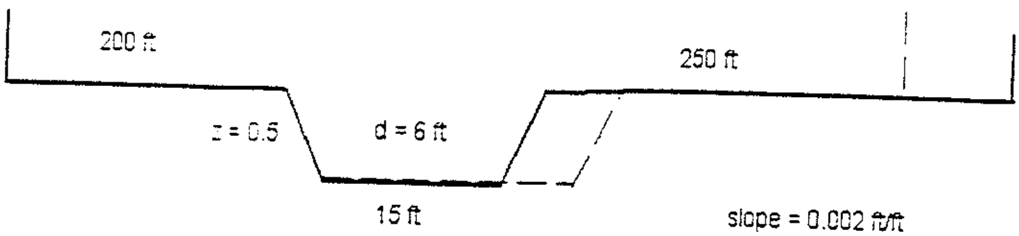
FIGURE E1



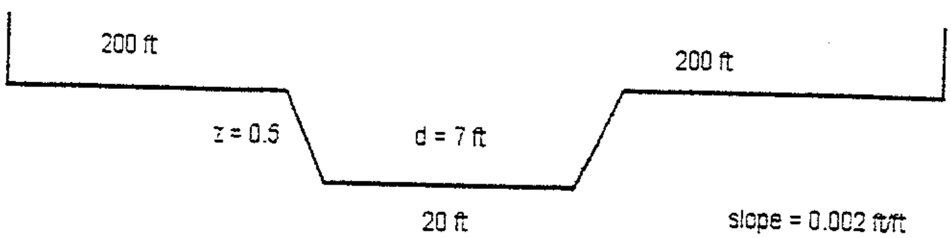
Existing X-Sec No. 1



Existing X-Sec No. 2



Existing X-Sec No. 3PU



Existing X-Sec No. 3

TR-20 REACH REPRESENTATIVE CROSS SECTIONS
EXAMPLE CALIBRATION TO REGIONAL REGRESSION EQUATIONS

FIGURE E2

NRCS LAG EQUATION

Lg = watershed lag in hours
Lh = watershed hydraulic length in feet
RCN = weighted runoff curve number

S = function of RCN (not entered)
Y = average watershed slope in %
Tc = time of concentration = 1.67 x Lg

WATERSHED	*Example TR-20*	Case:
JOB NO.	*1111-00*	***Existing Development***
DATE:	########	***RCN values***

NRCS LAG EQUATION COMPUTATION

Sub Area No.	Lh	RCN	S	Y	Lg	Tc
1	13200	81	2.35	2.6	**1.50**	**2.51**
2	15430	80	2.50	2.3	**1.87**	**3.12**
3	9750	82	2.20	2.8	**1.10**	**1.84**
4	12700	84	1.90	2.2	**1.44**	**2.40**
5	12000	88	1.36	3.1	**1.00**	**1.67**
6	16250	87	1.49	3.4	**1.26**	**2.11**
7	9350	79	2.66	3.0	**1.13**	**1.89**

WATERSHED	*Example TR-20*	Case:
JOB NO.	*1111-00*	***Pre-Urban Development***
DATE:	########	***RCN values***

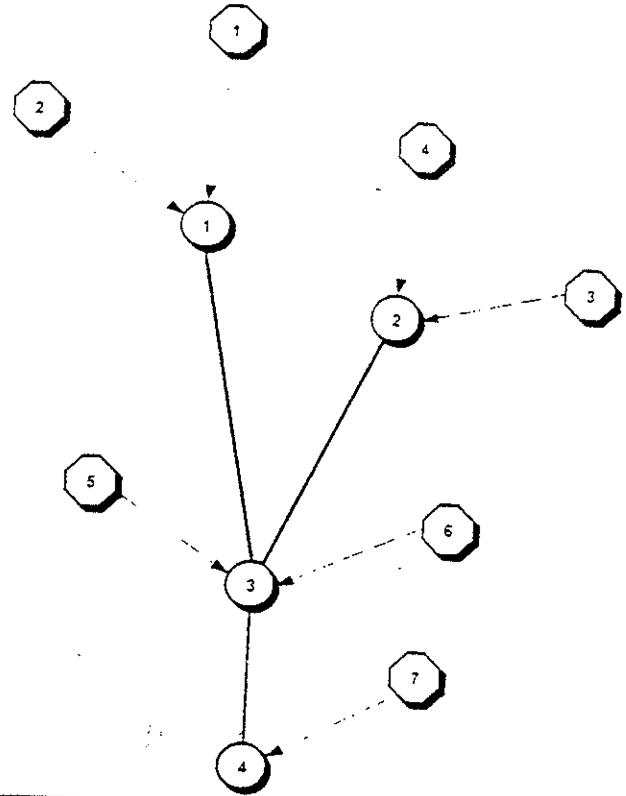
NRCS LAG EQUATION COMPUTATION

Sub Area No.	Lh	RCN	S	Y	Lg	Tc
1	13200	81	2.35	2.6	**1.50**	**2.51**
2	15430	80	2.50	2.3	**1.87**	**3.12**
3	9750	82	2.20	2.8	**1.10**	**1.84**
4	12700	84	1.90	2.2	**1.44**	**2.40**
5	12000	83	2.05	3.1	**1.20**	**2.00**
6	16250	81	2.35	3.4	**1.55**	**2.59**
7	9350	79	2.66	3.0	**1.13**	**1.89**

FIGURE E3

BASE TR-20 MODEL

TR-20 SCHEMATIC DIAGRAM AND DATA



Drainage Sub Area

Stream Reach

Dimensionless Unit Hydrograph K = 284
Storm = BALT *, Rainfall depth = 7.6 inches

* new NWS IDF rainfall tables for Baltimore, MD

Initial Watershed Data (base TR-20 model):

Sub Area No.	*DA Sq. mi.*	*% Urban*	*Existing RCN*	*Existing Tc*	*Pre-Urban RCN*	*Pre-Urban Tc*	*Ult. Devel RCN*
1	1.56	8	81	2.51	→	→	82
2	2.03	12	80	3.12	→	→	83
3	0.95	10	82	1.84	→	→	85
4	1.47	24 14	84	2.40	→	→	84
5	1.34	26	88	1.67	83	2.00	90
6	2.21	42	87	2.11	81	2.59	90
7	0.88	14	79	1.89	→	→	80
Totals	10.44	21.2					

→ No change from Existing Land Use value

Urbanization is 21.2 % which is greater than 15% therefore the existing RCN values should be adjusted to Pre-Urban conditions before calibrating with regional regression equations.

Reach Data:

Reach No.	*Length (ft)*	*Existing X-Sect No.*	*Pre-Urban X-Sect No.*
1-3	4500	1	1
2-3	3450	2	2
3-4	1750	3	3PU

FIGURE E4
Page 1

Base TR-20 Model for Pre-Urban Land Use (Before Calibration)

K = 284

48 values .5 hr increments

← New IDF data for Baltimore, MD

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20 EXAMPLE1                FULLPRINT PASS=001  SUMMARY
TITLE      EXAMPLE TR-20 CALIBRATION USING REGIONAL REGRESSION EQUATION
TITLE      CHANGING EXISTING RCN WHEN WATERSHED IS > 15% URBANIZED
 4 DIMHYD                0.02
 8            0.0        0.111       0.356       0.655       0.996
 8            1.00       0.929       0.828       0.737       0.656
 8            0.584      0.521       0.465       0.415       0.371
 8            0.331      0.296       0.265       0.237       0.212
 8            0.190      0.170       0.153       0.138       0.123
 8            0.109      0.097       0.086       0.076       0.066
 8            0.057      0.049       0.041       0.033       0.027
 8            0.024      0.021       0.018       0.015       0.013
 8            0.012      0.011       0.009       0.008       0.008
 8            0.006      0.005       0.005       0.005       0.000
 9 ENDTBL
 5 RAINFL 7              0.1
 8            0.000000   0.000727    0.001460    0.002199    0.002944
 8            0.003696   0.004455    0.005220    0.005992    0.006771
 8            0.007557   0.008351    0.009151    0.009959    0.010775
 8            0.011598   0.012429    0.013269    0.014116    0.014972
 8            0.015836   0.016709    0.017591    0.018482    0.019382
 8            0.020291   0.021210    0.022139    0.023078    0.024028
 8            0.024987   0.025958    0.026939    0.027932    0.028936
 8            0.029952   0.030980    0.032020    0.033073    0.034139
 8            0.035218   0.036311    0.037417    0.038538    0.039673
 8            0.040824   0.041990    0.043171    0.044370    0.045584
 8            0.046816   0.048066    0.049334    0.050621    0.051927
 8            0.053253   0.054600    0.055968    0.057358    0.058770
 8            0.060206   0.061666    0.063150    0.064661    0.066198
 8            0.067763   0.069357    0.070981    0.072635    0.074322
 8            0.076042   0.077797    0.079588    0.081416    0.083284
 8            0.085193   0.087145    0.089142    0.091186    0.093279
 8            0.095424   0.097623    0.099879    0.102196    0.104575
 8            0.107022   0.109540    0.112133    0.114806    0.117564
 8            0.120412   0.123356    0.126404    0.129563    0.132841
 8            0.136248   0.139794    0.143490    0.147351    0.151392
 8            0.155630   0.160085    0.164781    0.169746    0.175012
 8            0.180618   0.186610    0.193047    0.200000    0.207557
 8            0.215836   0.224987    0.235218    0.246816    0.260206
 8            0.276042   0.295424    0.320412    0.355630    0.415836
 8            0.500000   0.584164    0.644370    0.679588    0.704576
 8            0.723958   0.739794    0.753184    0.764782    0.775013
 8            0.784164   0.792443    0.800000    0.806953    0.813390
 8            0.819382   0.824988    0.830254    0.835219    0.839915
 8            0.844370   0.848608    0.852649    0.856510    0.860206
 8            0.863752   0.867159    0.870437    0.873596    0.876644
 8            0.879588   0.882436    0.885194    0.887867    0.890460
 8            0.892978   0.895424    0.897804    0.900121    0.902377
 8            0.904576   0.906721    0.908814    0.910858    0.912855
 8            0.914807   0.916716    0.918584    0.920412    0.922203
 8            0.923958   0.925678    0.927365    0.929019    0.930643
 8            0.932237   0.933802    0.935339    0.936850    0.938334
 8            0.939794   0.941230    0.942642    0.944032    0.945400
 8            0.946747   0.948073    0.949379    0.950666    0.951934
 8            0.953184   0.954416    0.955630    0.956829    0.958010
 8            0.959176   0.960327    0.961462    0.962583    0.963689
 8            0.964782   0.965861    0.966927    0.967980    0.969020
 8            0.970048   0.971064    0.972068    0.973061    0.974042
 8            0.975013   0.975972    0.976922    0.977861    0.978790
 8            0.979709   0.980618    0.981518    0.982409    0.983291
 8            0.984164   0.985028    0.985884    0.986731    0.987571
 8            0.988402   0.989225    0.990041    0.990849    0.991649
 8            0.992443   0.993229    0.994008    0.994780    0.995545
 8            0.996304   0.997056    0.997801    0.998540    0.999273
 8            1.000000   1.000000    1.000000    1.000000    1.000000
 9 ENDTBL
```

FIGURE E4
Page 2

```
2 XSECTN   001          1.0                                          n = 0.06
8                             200.0        0.0        0.0
8                             200.6        4.6        5.2
8                             201.2       14.7       11.0
8                             201.8       29.3       17.6
8                             202.4       48.1       25.0
8                             203.0       71.3       33.0
8                             204.0      364.6      247.0
8                             206.0     1924.8      675.0
8                             208.0     4312.8     1103.0
8                             210.0     7363.2     1531.0
8                             212.0    10978.7     1959.0
8                             214.0    15090.7     2387.0
8                             216.0    19646.8     2815.0
8                             218.0    24604.8     3243.0
9 ENDTBL
2 XSECTN   002          1.0                                          n = 0.06
8                             180.0        0.0        0.0
8                             181.0       12.7       12.5
8                             182.0       39.2       26.0
8                             183.0       75.3       40.5
8                             184.0      119.8       56.0
8                             185.0      172.3       72.5
8                             186.0      509.7      409.5
8                             188.0     2560.9     1083.5
8                             190.0     5692.2     1757.5
8                             192.0     9705.7     2431.5
8                             194.0    14485.4     3105.5
8                             196.0    19949.9     3779.5
8                             198.0    26036.8     4453.5
8                             200.0    32695.9     5127.5
9 ENDTBL
2 XSECTN   003          1.0                                          n = 0.06   X-Sec 3PU
8                             160.0        0.0        0.0
8                             161.2       18.7       18.7
8                             162.4       57.6       38.9
8                             163.6      110.7       60.5
8                             164.8      176.2       83.5
8                             166.0      253.1      108.0
8                             167.0      630.7      579.0
8                             169.0     3138.0     1521.0
8                             171.0     6969.9     2463.0
8                             173.0    11894.2     3405.0
8                             175.0    17775.2     4347.0
8                             177.0    24518.3     5289.0
8                             179.0    32051.5     6231.0
8                             181.0    40316.7     7173.0
9 ENDTBL
6 RUNOFF 1 001         1 1.56      81.        2.51      1      1
6 RUNOFF 1 001         2 2.03      80.        3.12      1      1
6 ADDHYD 4 001   1 2 3                                  1      1
6 REACH  3 001   3     4 4500.                          1      1
6 RUNOFF 1 002         1 0.95      82.        1.84      1      1
6 RUNOFF 1 002         2 1.47      84.        2.40      1      1
6 ADDHYD 4 002   1 2 3                                  1      1
6 REACH  3 002   3     5 3450.                          1      1
6 ADDHYD 4 003   4 5 6                                  1      1
6 RUNOFF 1 003         1 1.34      83.        2.00      1      1 <--------Values for Pre-
Urban
6 RUNOFF 1 003         2 2.21      81.        2.59      1      1 <--------Values for Pre-
Urban
6 ADDHYD 4 003   1 2 3                                  1      1
6 ADDHYD 4 003   3 6 7                                  1      1
6 REACH  3 003   7     1 1750.                          1      1
6 RUNOFF 1 004         2 0.88      79.        1.89      1      1
6 ADDHYD 4 004   1 2 3                                  1      1
  ENDATA
7 INCREM 6               0.25
7 COMPUT 7 001   004     0.        7.6        1.        7 2  01  99
  ENDCMP 1
  ENDJOB 2
```

Pre urban RCNs
" " TCs

*****************************END OF 80-80 LIST*****************************

FIGURE E4
Page 3

SUMMARY TABLE 3

STORM DISCHARGES (CFS) AT XSECTIONS AND STRUCTURES FOR ALL ALTERNATES
QUESTION MARK (?) AFTER: OUTFLOW PEAK - RISING TRUNCATED HYDROGRAPH.

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 99	
XSECTION 1	3.59		
ALTERNATE 1		1920	
XSECTION 2	2.42		
ALTERNATE 1		1626	
XSECTION 3	9.56		
ALTERNATE 1		5669	
XSECTION 4	10.44		
ALTERNATE 1		6220	← **Uncalibrated Pre-Urban Peak Q_{100}**

FIGURE E5

REGIONAL REGRESSION EQUATION DATA

Western Coastal Plain Region:

$Q_{100} = 2140\ A^{0.770}\ (F + 10)^{-0.391}$ (p. 12 of USGS Report 95-4154)

Where: A = Drainage area in sq. mi.
F = Forest cover in percent of drainage area

For the whole watershed: A = 10.44 sq.mi. F = 32% existing

should it be pre existing?

Using the USGS Computation Program the following is the output:

Flood frequency estimates for
EXAMPLE TR-20 WATERSHED - CALIBRATED TO THE REGIONAL REGRESSION EQ.
REGION: Western Coastal Plain region
area= 10.44: forest = 32.00 :skew= 0.69

Return Period	Discharge (cfs)	Standard Error of Prediction (percent)	Equivalent Years of Record	Standard Error of Prediction (logs)
2	450.	59.1	1.68	0.2375
5	828.	54.9	4.04	0.2228
10	1180.	55.4	6.30	0.2247
25	1760.	59.1	8.97	0.2378
50	2320.	63.8	10.43	0.2536
100	**3020.**	69.7	11.41	0.2732
500	5360.	88.0	12.30	0.3288

P R E D I C T I O N I N T E R V A L S

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
2	309.	656.	260.	778.	174.	1170.	142.	1430.
5	582.	1180.	496.	1380.	339.	2020.	281.	2450.
10	824.	1680.	701.	1970.	478.	2890.	395.	3500.
25	1210.	2570.	1020.	3040.	679.	4560.	554.	5590.
50	1550.	3470.	1300.	4170.	841.	6420.	677.	7960.
100	1960.	4660.	**1610.**	**5670.**	1010.	9030.	801.	11400.
500	3180.	9030.	2510.	11400.	1440.	20000.	1080.	26500.

The range of acceptable Q_{100} values for the Pre-Urban watershed are shown in bold above. They should be expected to fall between 3020 cfs and 5670 cfs.

FIGURE E6
Page 1

Calibrated TR-20 Model for Pre-Urban Land Use (After Calibration)

```
****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20 EXAMPLE1            FULLPRINT PASS=001  SUMMARY
TITLE     EXAMPLE TR-20 CALIBRATION USING REGIONAL REGRESSION EQUATION
TITLE     CHANGING EXISTING RCN WHEN WATERSHED IS > 15% URBANIZED
```

(See above for 284 DIMHYD and New IDF RAINFL Table for Baltimore, MD)

```
 2 XSECTN   001        1.0                                        n = 0.07
 8                           200.0        0.0        0.0
 8                           200.6        3.9        5.2
 8                           201.2       12.6       11.0
 8                           201.8       25.1       17.6
 8                           202.4       41.3       25.0
 8                           203.0       61.1       33.0
 8                           204.0      312.5      247.0
 8                           206.0     1649.9      675.0
 8                           208.0     3696.7     1103.0
 8                           210.0     6311.3     1531.0
 8                           212.0     9410.3     1959.0
 8                           214.0    12934.9     2387.0
 8                           216.0    16840.1     2815.0
 8                           218.0    21089.8     3243.0
 9 ENDTBL
 2 XSECTN   002        1.0                                        n = 0.07
 8                           180.0        0.0        0.0
 8                           181.0       10.9       12.5
 8                           182.0       33.6       26.0
 8                           183.0       64.5       40.5
 8                           184.0      102.7       56.0
 8                           185.0      147.7       72.5
 8                           186.0      436.9      409.5
 8                           188.0     2195.1     1083.5
 8                           190.0     4879.0     1757.5
 8                           192.0     8319.2     2431.5
 8                           194.0    12416.1     3105.5
 8                           196.0    17099.9     3779.5
 8                           198.0    22317.2     4453.5
 8                           200.0    28025.0     5127.5
 9 ENDTBL
 2 XSECTN   003        1.0                                        n = 0.07     X-Sec
3PU
 8                           160.0        0.0        0.0
 8                           161.2       18.5       18.7
 8                           162.4       57.0       38.9
 8                           163.6      109.6       60.5
 8                           164.8      174.4       83.5
 8                           166.0      250.5      108.0
 8                           167.0      624.2      579.0
 8                           169.0     3105.8     1521.0
 8                           171.0     6898.4     2463.0
 8                           173.0    11772.2     3405.0
 8                           175.0    17592.8     4347.0
 8                           177.0    24266.8     5289.0
 8                           179.0    31722.7     6231.0
 8                           181.0    39903.2     7173.0
 9 ENDTBL
```

FIGURE E6
Page 2

```
 6 RUNOFF 1 001       1 1.56      81.       2.89       1       1   ← 115% x  all  tc
values
 6 RUNOFF 1 001       2 2.03      80.       3.59       1       1
 6 ADDHYD 4 001   1 2 3                                1       1
 6 REACH  3 001   3.  4 4500.                          1       1
 6 RUNOFF 1 002       1 0.95      82.       2.12       1       1
 6 RUNOFF 1 002       2 1.47      84.       2.76       1       1
 6 ADDHYD 4 002   1 2 3                                1       1
 6 REACH  3 002   3   5 3450.                          1       1
 6 ADDHYD 4 003   4 5 6                                1       1
 6 RUNOFF 1 003       1 1.34      83.       2.30       1       1   ← Values for Pre-
Urban
 6 RUNOFF 1 003       2 2.21      81.       2.98       1       1   ← Values for Pre-
Urban
 6 ADDHYD 4 003   1 2 3                                1       1
 6 ADDHYD 4 003   3 6 7                                1       1
 6 REACH  3 003   7   1 1750.                          1       1
 6 RUNOFF 1 004       2 0.88      79.       2.17       1       1
 6 ADDHYD 4 004   1 2 3                                1       1
   ENDATA
 7 INCREM 6             0.25
 7 COMPUT 7 001   004   0.        7.6       1.         7 2  01  99
   ENDCMP 1
   ENDJOB 2

*****************************END OF 80-80 LIST*****************************
```

```
                          SUMMARY TABLE 3
                          ---------------
     STORM DISCHARGES (CFS) AT XSECTIONS AND STRUCTURES FOR ALL ALTERNATES
     QUESTION MARK (?) AFTER: OUTFLOW PEAK - RISING TRUNCATED HYDROGRAPH.

XSECTION/           DRAINAGE
STRUCTURE             AREA      STORM NUMBERS..........
  ID                (SQ MI)       99

XSECTION    1          3.59
---------------------------
   ALTERNATE    1                1725

XSECTION    2          2.42
---------------------------
   ALTERNATE    1                1465

XSECTION    3          9.56
---------------------------
   ALTERNATE    1                5103

XSECTION    4         10.44
---------------------------
   ALTERNATE    1                5573  ← Calibrated Pre-Urban Peak Q100
```

FIGURE E7
Page 1

Calibrated TR-20 Model for Existing Land Use (Calibrated Model)

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20 EXAMPLE1              FULLPRINT PASS=001  SUMMARY
TITLE     EXAMPLE TR-20 CALIBRATION USING REGIONAL REGRESSION EQUATION
TITLE     CHANGING EXISTING RCN WHEN WATERSHED IS > 15% URBANIZED
```

(See above for 284 DIMHYD and New IDF RAINFL Table for Baltimore, MD)

```
2 XSECTN   001         1.0                                              n = 0.07
8                            200.0        0.0        0.0
8                            200.6        3.9        5.2
8                            201.2       12.6       11.0
8                            201.8       25.1       17.6
8                            202.4       41.3       25.0
8                            203.0       61.1       33.0
8                            204.0      312.5      247.0
8                            206.0     1649.9      675.0
8                            208.0     3696.7     1103.0
8                            210.0     6311.3     1531.0
8                            212.0     9410.3     1959.0
8                            214.0    12934.9     2387.0
8                            216.0    16840.1     2815.0
8                            218.0    21089.8     3243.0
9 ENDTBL
2 XSECTN   002         1.0                                              n = 0.07
8                            180.0        0.0        0.0
8                            181.0       10.9       12.5
8                            182.0       33.6       26.0
8                            183.0       64.5       40.5
8                            184.0      102.7       56.0
8                            185.0      147.7       72.5
8                            186.0      436.9      409.5
8                            188.0     2195.1     1083.5
8                            190.0     4879.0     1757.5
8                            192.0     8319.2     2431.5
8                            194.0    12416.1     3105.5
8                            196.0    17099.9     3779.5
8                            198.0    22317.2     4453.5
8                            200.0    28025.0     5127.5
9 ENDTBL
2 XSECTN   003         1.0                                n = 0.07    X-Sect 3
replaces 3PU
8                            160.0        0.0        0.0
8                            161.4       32.1       29.0
8                            162.8       98.9       59.9
8                            164.2      190.3       92.8
8                            165.6      302.6      127.7
8                            167.0      434.2      164.5
8                            168.0      688.4      591.5
8                            170.0     3034.5     1445.5
8                            172.0     6539.8     2299.5
8                            174.0    11005.8     3153.5
8                            176.0    16313.9     4007.5
8                            178.0    22380.6     4861.5
8                            180.0    29141.7     5715.5
8                            182.0    36545.4     6569.5
9 ENDTBL
```

FIGURE E7
Page 2

```
6 RUNOFF 1 001       1 1.56      81.      2.89      1      1   ←115% x  tc value
6 RUNOFF 1 001       2 2.03      80.      3.59      1      1   ←115% x  tc value
6 ADDHYD 4 001   1 2 3                              1      1
6 REACH  3 001   3   4 4500.                        1      1
6 RUNOFF 1 002       1 0.95      82.      2.12      1      1   ←115% x  tc value
6 RUNOFF 1 002       2 1.47      84.      2.76      1      1   ←115% x  tc value
6 ADDHYD 4 002   1 2 3                              1      1
6 REACH  3 002   3   5 3450.                        1      1
6 ADDHYD 4 003   4 5 6                              1      1
6 RUNOFF 1 003       1 1.34      88.      1.92      1      1   ←Replace RCN, 115% x Tc
existing tc
6 RUNOFF 1 003       2 2.21      87.      2.43      1      1   ←Replace RCN, 115% x Tc
existing tc
6 ADDHYD 4 003   1 2 3                              1      1
6 ADDHYD 4 003   3 6 7                              1      1
6 REACH  3 003   7   1 1750.                        1      1
6 RUNOFF 1 004       2 0.88      79.      2.17      1      1   ←115% x  tc value
6 ADDHYD 4 004   1 2 3                              1      1
  ENDATA
7 INCREM 6            0.25
7 COMPUT 7 001   004  0.         7.6      1.        7 2  01  99
  ENDCMP 1
  ENDJOB 2

******************************END OF 80-80 LIST******************************
```

SUMMARY TABLE 3

STORM DISCHARGES (CFS) AT XSECTIONS AND STRUCTURES FOR ALL ALTERNATES
QUESTION MARK (?) AFTER: OUTFLOW PEAK - RISING TRUNCATED HYDROGRAPH.

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 99
XSECTION 1	3.59	
ALTERNATE 1		1725
XSECTION 2	2.42	
ALTERNATE 1		1465
XSECTION 3	9.56	
ALTERNATE 1		5460
XSECTION 4	10.44	
ALTERNATE 1		5977 ← Calibrated Existing Peak Q_{100}

FIGURE E8
Page 1

Calibrated TR-20 Model for Ultimate Land Use (Calibrated Model)

```
****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20 EXAMPLE1              FULLPRINT PASS=001  SUMMARY
TITLE      EXAMPLE TR-20 CALIBRATION USING REGIONAL REGRESSION EQUATION
TITLE      CHANGING EXISTING RCN WHEN WATERSHED IS > 15% URBANIZED
```

(See above for 284 DIMHYD and New IDF RAINFL Table for Baltimore, MD)
(Use same cross sections as Existing model)

```
2 XSECTN    001          1.0
8                             200.0        0.0        0.0
8                             200.6        3.9        5.2
8                             201.2       12.6       11.0
8                             201.8       25.1       17.6
8                             202.4       41.3       25.0
8                             203.0       61.1       33.0
8                             204.0      312.5      247.0
8                             206.0     1649.9      675.0
8                             208.0     3696.7     1103.0
8                             210.0     6311.3     1531.0
8                             212.0     9410.3     1959.0
8                             214.0    12934.9     2387.0
8                             216.0    16840.1     2815.0
8                             218.0    21089.8     3243.0
9 ENDTBL
2 XSECTN    002          1.0
8                             180.0        0.0        0.0
8                             181.0       10.9       12.5
8                             182.0       33.6       26.0
8                             183.0       64.5       40.5
8                             184.0      102.7       56.0
8                             185.0      147.7       72.5
8                             186.0      436.9      409.5
8                             188.0     2195.1     1083.5
8                             190.0     4879.0     1757.5
8                             192.0     8319.2     2431.5
8                             194.0    12416.1     3105.5
8                             196.0    17099.9     3779.5
8                             198.0    22317.2     4453.5
8                             200.0    28025.0     5127.5
9 ENDTBL
2 XSECTN    003          1.0
8                             160.0        0.0        0.0
8                             161.4       32.1       29.0
8                             162.8       98.9       59.9
8                             164.2      190.3       92.8
8                             165.6      302.6      127.7
8                             167.0      434.2      164.5
8                             168.0      688.4      591.5
8                             170.0     3034.5     1445.5
8                             172.0     6539.8     2299.5
8                             174.0    11005.8     3153.5
8                             176.0    16313.9     4007.5
8                             178.0    22380.6     4861.5
8                             180.0    29141.7     5715.5
8                             182.0    36545.4     6569.5
```

Use same model as Existing Land Use but substitute Ultimate RCN values

```
 9 ENDTBL
 6 RUNOFF 1 001         1 1.56       82.        2.89        1         1
 6 RUNOFF 1 001         2 2.03       83.        3.59        1         1
 6 ADDHYD 4 001     1 2 3                                   1         1
 6 REACH  3 001     3   4 4500.                             1         1
 6 RUNOFF 1 002         1 0.95       85.        2.12        1         1
 6 RUNOFF 1 002         2 1.47       84.        2.76        1         1
 6 ADDHYD 4 002     1 2 3                                   1         1
 6 REACH  3 002     3   5 3450.                             1         1
 6 ADDHYD 4 003     4 5 6                                   1         1
 6 RUNOFF 1 003         1 1.34       90.        1.92        1         1
 6 RUNOFF 1 003         2 2.21       90.        2.43        1         1
 6 ADDHYD 4 003     1 2 3                                   1         1
 6 ADDHYD 4 003     3 6 7                                   1         1
 6 REACH  3 003     7   1 1750.                             1         1
 6 RUNOFF 1 004         2 0.88       80.        2.17        1         1
 6 ADDHYD 4 004     1 2 3                                   1         1
   ENDATA
 7 INCREM 6             0.25
 7 COMPUT 7 001     004 0.           7.6        1.          7 2  01   99
   ENDCMP 1
   ENDJOB 2
```

****************************END OF 80-80 LIST****************************

SUMMARY TABLE 3

STORM DISCHARGES (CFS) AT XSECTIONS AND STRUCTURES FOR ALL ALTERNATES
QUESTION MARK (?) AFTER: OUTFLOW PEAK - RISING TRUNCATED HYDROGRAPH.

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 99
XSECTION 1	3.59	
ALTERNATE 1		1807
XSECTION 2	2.42	
ALTERNATE 1		1506
XSECTION 3	9.56	
ALTERNATE 1		5686
XSECTION 4	10.44	
ALTERNATE 1		6222 ← **Calibrated Ultimate Peak Q_{100}**

TABLE I
LAND COVER DISTRIBUTIONS IN UNDEVELOPED AREAS

ALLEGANY COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	209326.95	84.11
BRUSH	5567.67	2.24
CULTIVATED	21848.40	8.78
GRASS	12126.78	4.87

ANNEARUN COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	126463.69	67.44
BRUSH	3304.80	1.76
CULTIVATED	48204.18	25.71
GRASS	9547.20	5.09

BALTIMOR COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	146499.03	52.21
BRUSH	6618.78	2.36
CULTIVATED	99621.37	35.50
GRASS	27852.12	9.93

CALVERT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	85213.35	71.05
BRUSH	1271.43	1.06
CULTIVATED	30335.31	25.29
GRASS	3121.20	2.60

CAROLINE COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	66761.55	34.49
BRUSH	2483.19	1.28
CULTIVATED	122273.02	63.16
GRASS	2070.09	1.07

CARROLL COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	66348.45	25.57
BRUSH	3203.82	1.23
CULTIVATED	161021.80	62.05
GRASS	28949.13	11.15

CECIL COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	90322.02	44.22
BRUSH	3033.99	1.49
CULTIVATED	105629.67	51.72
GRASS	5255.55	2.57

CHARLES COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	193909.14	74.11
BRUSH	4167.72	1.59
CULTIVATED	59128.38	22.60
GRASS	4443.12	1.70

TABLE I - Continued

DORCHEST COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	124545.06	48.61
BRUSH	9202.95	3.59
CULTIVATED	120859.30	47.18
GRASS	1583.55	0.62

FREDERIC COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	134739.45	33.58
BRUSH	2359.26	0.59
CULTIVATED	228306.61	56.90
GRASS	35838.72	8.93

GARRETT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	288991.00	71.35
BRUSH	13251.33	3.27
CULTIVATED	75679.92	18.68
GRASS	27122.31	6.70

HARFORD COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	105996.88	45.32
BRUSH	2923.83	1.25
CULTIVATED	110123.28	47.08
GRASS	14839.47	6.34

HOWARD COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	57466.80	46.02
BRUSH	4511.97	3.61
CULTIVATED	51274.89	41.06
GRASS	11617.29	9.30

KENT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	46795.05	27.63
BRUSH	688.50	0.41
CULTIVATED	120854.70	71.37
GRASS	1005.21	0.59

MONTGOME COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	91432.80	40.65
BRUSH	8945.91	3.98
CULTIVATED	80453.52	35.77
GRASS	44096.13	19.60

PRINCE-G COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	152580.78	68.59
BRUSH	1142.91	0.51
CULTIVATED	50669.01	22.78
GRASS	18061.65	8.12

TABLE I - Continued

QUEEN-AN COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	66977.28	29.99
BRUSH	1000.62	0.45
CULTIVATED	153232.56	68.62
GRASS	2106.81	0.94

ST-MARYS COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	133679.17	65.68
BRUSH	1748.79	0.86
CULTIVATED	63915.75	31.41
GRASS	4172.31	2.05

SOMERSET COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	73210.50	50.91
BRUSH	15009.30	10.44
CULTIVATED	54267.57	37.74
GRASS	1308.15	0.91

TALBOT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	44596.44	28.06
BRUSH	1041.93	0.66
CULTIVATED	110164.59	69.31
GRASS	3134.97	1.97

WASHINGT COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	117233.20	42.76
BRUSH	3635.28	1.33
CULTIVATED	132320.53	48.26
GRASS	20999.25	7.66

WICOMICO COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	101002.95	48.68
BRUSH	13462.47	6.49
CULTIVATED	91878.03	44.28
GRASS	1152.09	0.56

WORCESTE COUNTY
SUMMARY OF NON-URBAN LAND

CLASS	1985 ACRES	PERCENT OF TOTAL
FOREST	150097.59	56.80
BRUSH	15564.69	5.89
CULTIVATED	96692.95	36.59
GRASS	1918.62	0.73

TABLE II
ESTIMATING PRE-DEVELOPMENT CURVE NUMBER

The existing land cover of a 1000 acre watershed being investigated in Baltimore County is more than 15% urban. As part of the calibration process against the USGS regression equations, we need to estimate a pre-development curve number. The hydrologic soil distribution for the 1000 acre watershed is:

Group A	100 acres
Group B	700 acres
Group D	200 acres

Table I shows the distributions of undeveloped areas in Baltimore County as:

Forest	52.21%
Brush	2.36%
Cultivated	35.50%
Grass	9.93%

The curve numbers for the land/soil complexes are:

	Hydrologic Soil			
Category	A	B	C	D
Forest	36	60	79	89
Brush	35	56	70	77
Cultivated	72	81	88	91
Grass	48	69	79	89

An estimate of a pre-development curve number is obtained by assuming the land cover is equally distributed over the existing A, B and D soil groups as:

Forest	.5221[(100)(36) + (700)(60) + (200)(79)] =	32,057
Brush	.0236[(100)(35) + (700)(56) + (200)(77)] =	1,371
Cultivated	.3550[(100)(72) + (700)(81) + (200)(91)] =	29,148
Grass	.0993[(100)(48) + (700)(69) + (200)(89)] =	7,040
	Total =	69,616

Pre-development Curve Number = 69,616/1000 = 69.6

This approach to the estimation of a pre-development curve number is the same as that used in the MD-SHA's hydrologic modeling GIS, GISHYDRO.

APPENDIX 5

Regression Equation for Estimating the Time of Concentration

Appendix 5: Regression Equation for Estimating the Time of Concentration

Time of concentration (TC) can be defined from an observed rainfall hyetograph and the resulting discharge hydrograph. TC is estimated as the time between the end of rainfall excess and the first inflection point on the recession of the runoff hydrograph. TC values were computed from rainfall-runoff data compiled by Dillow (1998) as part of a flood hydrograph study for the Maryland State Highway Administration.

Dillow (1998) compiled data for 278 rainfall-runoff events at 81 gaging stations in Maryland. Not all of the 278 events were suitable in defining TC for our study. For some rainfall-runoff events, it was not possible to detect an inflection point on the recession of the hydrograph. On average, about three events were used in determining the average TC for a watershed. For three gaging stations, there were no rainfall-runoff events suitable for determining TC. Therefore, data for 78 gaging stations are used in developing a regression equation for estimating TC for ungaged watersheds. The average TC values and watershed characteristics are given in Table A5.1.

Stepwise regression analysis is used to relate the average TC value at 78 gaging stations to the watershed characteristics given in Table A5.1. The watershed characteristics used in this analysis were taken from Dillow (1998). Some of the watershed characteristics that are highly correlated with TC are also highly correlated with each other. For example, drainage area has a correlation coefficient of 0.98 with channel length. Since these two variables are highly correlated, both variables are not significant in the regression analysis because they are essentially explaining the same variation in TC. The regression equation based on channel length has a slightly lower standard error than the one with drainage area and so channel length is used in the final equation. Channel length also is a better predictor of travel time for a variety of watershed shapes.

Using Dillow's approach (1998), qualitative variables are used in the regression analysis to identify gaging stations in different hydrologic regions in Maryland. Dillow (1998) determined that there are three hydrologic regions for estimating flood hydrographs for Maryland streams: Appalachian Plateau, Piedmont and Coastal Plain. These same regions are assumed applicable in our analysis and are shown in Figure A5.1. The qualitative-variable approach is superior to defining different regression equations for each geographic region because there are only 10 gaging stations in the Appalachian Plateau.

The qualitative variables AP and CP are used in the regression equation to account for variability in TC not explained by the available explanatory variables. In Table A5.1, a CP value of 1 specifies the watershed is in the Coastal Plain Region, a AP value of 1 specifies the watershed is in the Appalachian Plateau and zero values for both CP and AP specify the watershed is in the Piedmont Region. The TC values for watersheds in the Appalachian Plateau and Coastal Plains are larger than watersheds in the Piedmont for a given set of watershed characteristics. The qualitative variables also account for regional differences in TC related to watershed characteristics not available for analysis. Both AP and CP are highly significant in the regression analysis.

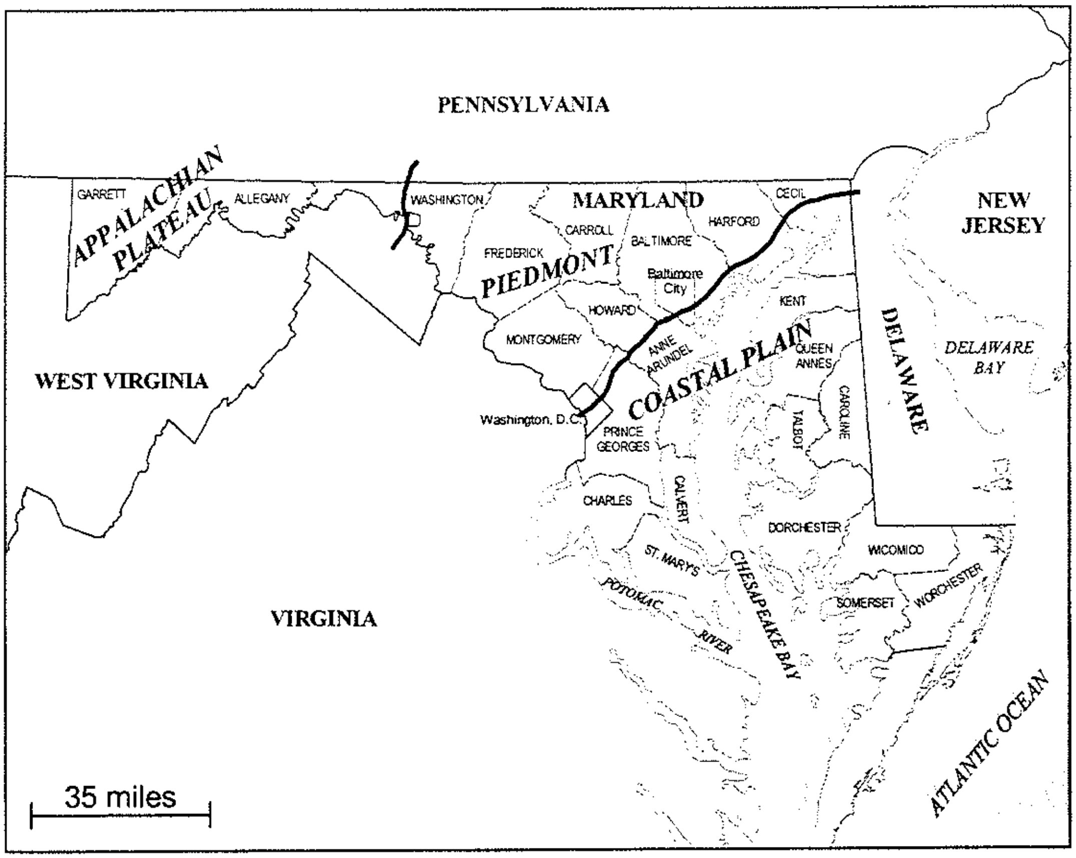


Figure A5.1. – Hydrologic regions in Maryland used in developing a regression equation for estimating the time of concentration for ungaged watersheds.

There is considerable variation in hydrology from the Coastal Plains of Maryland to the mountainous Appalachian Plateau. Therefore, several watershed characteristics are statistically significant in predicting TC. In the following equation, all explanatory variables are significant at the 5% level of significance. The coefficient of determination (R^2) is 0.888% implying the equation is explaining 88.8% of the variation in the observed value of TC. The standard error of estimate is 30.0%.

$$TC = 0.133\,(CL^{.475})\,(SL^{-.187})\,(101\text{-}FOR)^{-.144}\,(101\text{-}IA)^{.861}\,(ST+1)^{.154}\,(10^{.194AP})\,(10^{.366CP})$$
(A.1)

1.563 2.323

where

- TC = time of concentration in hours,
- CL = channel length in miles, measured from the point of interest to the watershed divide.
- SL = channel slope in feet per mile, measured as the slope between points 10 and 85% of the distance upstream from the point of interest.

$\frac{2.323}{1.563} = 1.49$

FOR = forest cover in percentage of the watershed,
IA = impervious area in percentage of the watershed,
ST = lakes and ponds in percentage of the watershed,
AP = 1 if the watershed is in the Appalachian Plateau, 0 otherwise,
CP = 1 if the watershed is in the Coastal Plain, 0 otherwise,
AP and CP = 0 for watersheds in the Piedmont Region.

Equation 1 was computed by transforming the TC values and watershed characteristics to logarithms and fitting a linear regression model to the transformed data. This transformation is somewhat standard in hydrologic analyses since the logarithmic transformation tends to stabilize the variance of the residuals, normalize the distribution of the residuals about the regression equation and linearize the equation.

The percentages of forest cover (FOR), impervious area (IA) and storage (ST) can be zero for a given watershed. Therefore, it is necessary to add constants to these variables prior to the logarithmic transformation or to substract these variables from a constant to avoid taking the logarithm of zero. For our analysis, substracting the percentages from 101 provided more reasonable estimates of the regression coefficients and slightly reduced the standard error of the regression equation.

The above equation can be used to estimate TC for rural and urban watersheds in Maryland. The percentage of impervious area (IA) is a measure of the urbanization or development in the watershed. In addition, urban watersheds would have a reduced amount of forest cover.

The TC values in Table A5.1 are generally longer than computed by SCS (1986) procedures for a given watershed area. One possible hypothesis is that this is related to size of the flood event used to determine TC. In general, the recurrence intervals of peak discharges were less than a 2-year event. There were only about 30 events across the 78 gaging stations where the peak discharge of the runoff hydrograph was a 5-year event or greater. An evaluation of the TC values as a function of recurrence interval revealed that the TC values did not vary with recurrence interval in any consistent pattern. In some instances, the larger flood events had smaller TC values and at other stations the converse was true. Therefore, it is not conclusive that the use of larger flood events would result in smaller TC values.

Dillow (1998) computed basin lagtime using essentially the same rainfall-runoff data used in our study to determine TC. Lagtime as used by Dillow (1998) was the time from the centroid of rainfall excess to the centroid of the direct runoff hydrograph. These lagtime values are given in Table A5.1. On average, the lagtime values computed by Dillow (1998) are about 5% less than the TC values. This difference is consistent with the definitions of lagtime and TC as used in the two analyses. For 90% of the gaging stations, the ratio of the lagtime to TC is between 0.70 and 1.40. A comparison of lagtime to TC values is given in Figure A5.2. As can be noted in Figure A5.2, lagtime and TC values tend to be nearly equal for smaller values of the parameters and deviate the most for larger values of the two parameters.

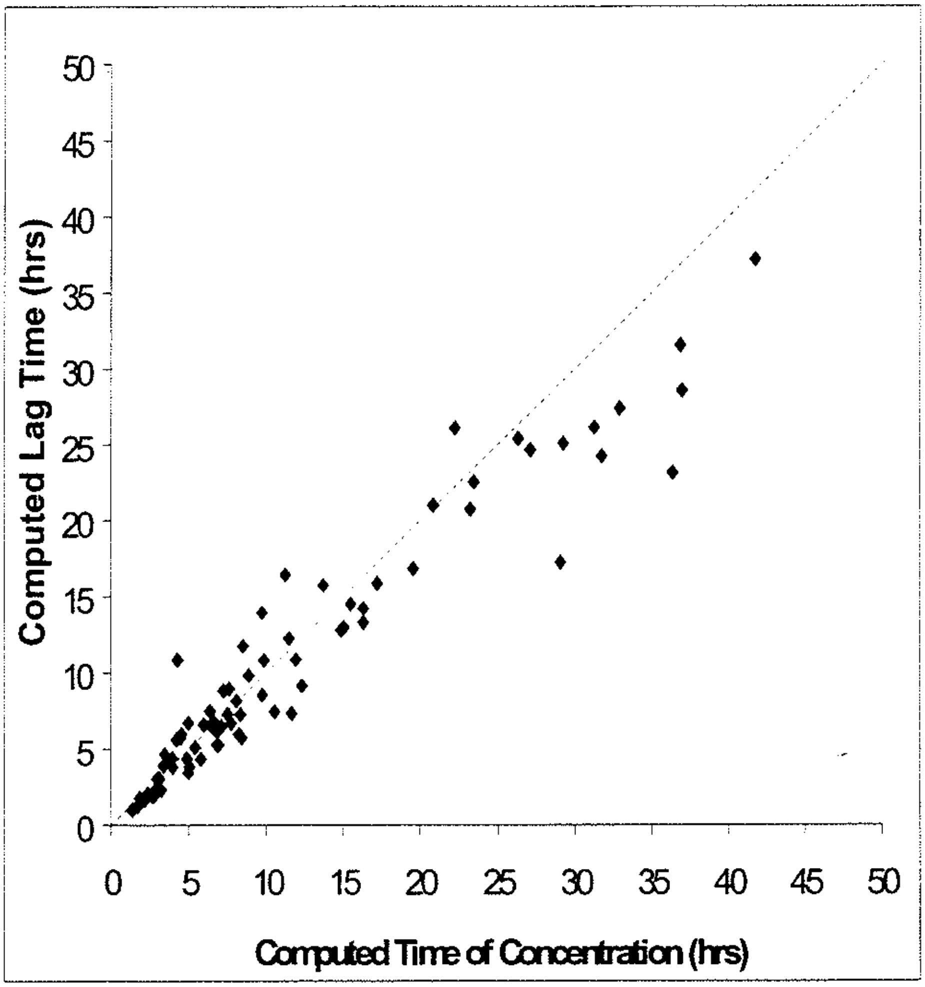


Figure A5.2. Comparison of basin lagtime to time of concentration using rainfall-runoff data in Maryland.

A comparison also was made between estimates of TC computed from Equation 1 and procedures in SCS (1986) based on travel time. The travel times shown in Table A5.2 were computed by MSHA personnel as a combination of overland flow, shallow concentrated flow and channel flow (SCS, 1986).

Table A5.2. A comparison of time of concentration (TC) estimated from a regression equation based on watershed characteristics to those based travel time.

Drainage area (mi^2)	**Site**	Region	Regression TC (hours)	Travel Time TC (hours)
6.66	West Branch @ MD 165	Piedmont	3.45	2.98
25.70	Middle Creek @ MD 17	Piedmont	5.89	5.22
5.01	Mill Creek @ MD 7	Piedmont	4.41	4.30
43.73	Little Gunpowder Falls @ U.S. 1	Piedmont	7.94	9.01
3.16	Little Monacacy River @ MD 109	Piedmont	2.74	1.52
6.26	Blockston Branch @ MD 481	Coastal Plain	10.75	8.70
3.24	Middle Branch @ U.S. Route 113	Coastal Plain	9.10	7.17
6.05	Church Branch @ U.S. Route 113	Coastal Plain	10.99	10.60
1.61	Carey Branch @ U.S. Route 113	Coastal Plain	6.02	5.66
6.64	Birch Branch @ U.S. Route 113	Coastal Plain	10.96	7.65

There is close agreement for TC estimates for several of the sites shown in Table A5.2. When there are significant differences, the values based are travel times (also known as the segmental approach) are less than those from the regression equation. Based on this limited comparison, it appears that Equation 1 can be used to determine realistic bounds on TC estimated by the travel time or segmental approach.

Any regression equation, such as Equation 1, should only be used at ungaged watersheds that have watershed characteristics within the range of those used to develop the equation. The upper and lower limits for the watershed characteristics are given in Table A5.3 for each hydrologic region to define the applicability of Equation 1. Therefore, Equation 1 should not be used for watersheds having characteristics outside the limits of those shown in Table A5.3.

Table A5.3. Upper and lower limits for watershed characteristics for the time of concentration regression equation for each hydrologic region.

Region	Variable	**Upper limit**	**Lower limit**
Appalachian Plateau	Drainage area (mi^2)	295	1.6
Appalachian Plateau	Channel length (mi)	40.8	2.1
Appalachian Plateau	Channel slope (ft/mi)	195	6.1
Appalachian Plateau	Storage (%)	3.2	0.0
Appalachian Plateau	Forest cover (%)	89	54
Appalachian Plateau	Impervious area (%)	1.25	0.0
Piedmont	Drainage area (mi^2)	494	2.1
Piedmont	Channel length (mi)	70	2.2
Piedmont	Channel slope (ft/mi)	336	11
Piedmont	Storage (%)	1.16	0.0
Piedmont	Forest cover (%)	92	2.0
Piedmont	Impervious area (%)	41	0.0
Coastal Plain	Drainage area (mi^2)	113	2.0
Coastal Plain	Channel length (mi)	18.3	2.0
Coastal Plain	Channel slope (ft/mi)	41.8	1.5
Coastal Plain	Storage (%)	26.0	0.0
Coastal Plain	Forest cover (%)	79	5.0
Coastal Plain	Impervious area (%)	35	0.0

In summary, Equation (A.1) is based on estimates of TC computed from rainfall-runoff events at 78 gaging stations in Maryland. The computed values of TC tend to be larger than similar estimates based on SCS (1986) procedures. However, Equation 1 can be used to evaluate the reasonableness of TC estimates from SCS (1986) procedures. Further research is needed to improve the estimation of TC in Maryland that would ultimately provide for more accurate estimates of design discharges from hydrological models such as TR-20.

Table A5.1. Watershed characteristics and times of concentration for rural and urban watersheds used in developing the regression equations.

STANO is the station number
DA is the drainage area in square miles
SL is the channel slope in feet per mile
CL is channel length in miles
SIN is the channel sinuosity detemined by dividing channel length by basin length
BL is the basin length in miles
ST is the percentage area of the drainage area covered by lakes, ponds and swamps
SH is the basin shape defined as channel length squared divided by drainaga area
FOR is forest cover in percentage of the drainage area
IA is impervious area expressed as percentage of the drainage area
BDF is the basin development factor
LT is the lagtime in hours
AP = 1 if the watershed is in the Applachian Plateau, CP = 1 if the watershed is in the Coastal Plains, CP and
AP = 0 implies the watershed is in the Piedmont Region
TC is the time of concentration in hours

STANO	DA	SL	CL	SIN	BL	ST	SH	FOR	IA	BDF	LT	AP	CP	TC
01594930	8.23	26.4	4.4	1.14	3.86	0.000	1.81	86	0.00	0	7.50	1	0	6.38
01594934	1.55	161.9	2.1	1.07	1.95	0.000	2.45	82	0.00	0	6.43	1	0	4.00
01594936	1.91	130.9	2.7	1.16	2.33	0.000	2.84	87	0.00	0	6.62	1	0	6.00
01594950	2.30	194.6	2.7	1.24	2.18	0.000	2.07	89	0.00	0	6.74	1	0	5.00
01595000	73.0	30.5	16.5	1.30	12.70	0.186	2.21	78	0.49	0	12.27	1	0	11.50
01596500	49.1	65.1	19.0	1.41	13.44	0.066	3.68	80	0.06	0	13.97	1	0	9.75
03075500	134.	6.09	19.3	1.59	12.12	0.493	1.10	54	0.88	0	22.57	1	0	23.50
03076500	295.	22.2	40.8	1.45	28.11	3.180	2.68	66	0.24	0	25.10	1	0	29.25

03076600	48.9	65.6	15.3	1.89	8.11	0.000	1.35	62	1.25	0	16.47	1	0	11.25
03078000	62.5	28.2	19.5	1.61	12.13	1.005	2.35	75	0.66	0	16.88	1	0	19.58
01614500	494.	11.2	69.5	2.44	28.45	0.101	1.64	37	1.43	0	25.42	0	0	26.33
01617800	18.9	23.8	9.4	1.08	8.69	0.000	4.00	2	2.32	0	15.53	0	0	.
01619500	281.	10.8	49.9	1.55	32.26	0.123	3.70	30	2.67	0	24.66	0	0	27.12
01637500	66.9	47.5	23.3	1.50	15.50	0.000	3.59	38	1.01	0	8.98	0	0	7.62
01639000	173.	18.9	30.8	1.92	16.05	0.114	1.49	20	0.69	0	15.91	0	0	17.25
01639375	41.3	75.4	12.2	1.40	8.70	0.207	1.83	70	0.87	0	3.47	0	0	5.00
01639500	102.	13.5	26.9	1.43	18.75	0.000	3.45	14	0.13	0	11.80	0	0	8.50

STANO	DA	SL	CL	SIN	BL	ST	SH	FOR	IA	BDF	LT	AP	CP	TC
01640965	2.14	336.4	2.2	1.12	1.96	0.000	1.80	92	0.00	0	1.78	0	0	1.88
01641000	18.4	145.2	9.7	1.57	6.18	0.373	2.08	80	1.93	1	5.11	0	0	5.44
01483700	31.9	4.66	12.3	1.38	8.89	11.927	2.48	21	4.46	2	27.41	0	1	32.92
01484000	13.6	6.26	5.9	1.01	5.87	0.626	2.53	34	0.33	0	21.04	0	1	20.85
01484500	5.24	4.87	4.4	1.19	3.70	0.000	2.61	39	3.24	0	12.82	0	1	14.88

01484548 13.6 4.39 7.9 1.22 6.48 26.055 3.09 33 1.13 0
24.28 0 1 31.75

01485000 60.5 1.49 14.6 1.18 12.42 18.396 2.55 25 0.08 0
28.58 0 1 37.00

01485500 44.9 3.56 12.2 1.11 10.98 1.326 2.69 79 0.30 0
37.21 0 1 41.75

01487000 75.4 3.23 13.7 1.20 11.44 0.000 1.74 40 0.85 0
20.80 0 1 23.25

01488500 44.8 2.65 11.7 1.17 10.00 0.000 2.23 39 0.14 0
12.99 0 1 15.08

01489000 8.50 7.65 5.3 1.46 3.64 0.000 1.87 24 0.00 0
5.78 0 1 8.44

01491000 113. 3.01 18.3 1.36 13.41 6.910 1.59 38 0.66 0
31.57 0 1 36.88

01493000 19.7 6.06 9.7 1.09 8.89 8.777 3.54 20 0.35 0
26.10 0 1 22.25

01493500 12.7 9.15 5.9 1.10 5.38 0.199 2.28 5 0.25 0
13.35 0 1 16.38

01483200 3.85 15.8 3.5 1.04 3.37 1.298 2.95 45 0.38 0
7.37 0 1 11.67

01484100 2.83 7.12 2.5 1.07 2.33 0.000 1.92 43 0.00 0
14.54 0 1 15.50

01486000 4.80 5.47 4.1 . . 0.000 . 57 0.00 0 .
0 1 10.50

01590500 6.92 19.8 4.7 1.14 4.12 0.000 2.45 65 1.87 0
10.90 0 1 11.94

01594526 89.7 8.2 16.1 1.18 13.60 0.037 2.06 30 7.84 4
23.16 0 1 36.38

01594670 9.38 16.9 5.2 1.30 3.99 0.000 1.70 70 3.85 0
9.17 0 1 12.33

01653600 39.5 16.1 14.4 1.64 8.79 0.176 1.96 38 8.25 2
17.29 0 1 29.05

01660920 79.9 10.6 16.6 1.15 14.48 5.051 2.62 56 3.60 0
26.17 0 1 31.25

01661050 18.5 12.4 7.2 1.22 5.92 0.000 1.89 56 3.09 0
14.26 0 1 16.38

01594710 3.26 41.8 2.9 1.08 2.68 0.000 2.20 52 9.24 0
3.86 0 1 5.08

01661500 24.0 12.9 8.0 1.28 6.25 0.000 1.63 78 2.46 0
15.78 0 1 13.75

01583600 20.9 52.0 8.2 1.43 5.72 0.309 1.57 29 18.6 4
5.63 0 0 4.25

01585100 7.61 48.2 6.0 1.12 5.38 0.000 3.80 28 27.5 7
2.11 0 0 2.75

STANO DA SL CL SIN BL ST SH FOR IA BDF LT
AP CP TC

01585200 2.13 72.7 2.2 1.12 1.97 0.000 1.82 7 33.0 8
1.02 0 0 1.38

01585300 4.46 54.7 4.6 1.25 3.68 0.558 3.04 28 23.6 6
2.06 0 0 2.38

01585400 1.97 27.1 2.0 1.22 1.64 0.000 1.37 24 35.1 2
2.33 0 1 3.25

01589100 2.47 87.1 3.2 1.22 2.62 0.000 2.78 19 37.0 4
1.67 0 0 2.17

01589300 32.5 21.0 13.7 1.37 9.99 0.000 3.07 31 18.6 4
3.95 0 0 3.38

01589330 5.52 52.1 3.2 1.12 2.86 0.000 1.48 4 40.8 12
2.26 0 0 2.83

01589500 4.97 24.8 4.4 1.17 3.75 0.000 2.83 44 21.9 3
8.19 0 1 .

01589512 8.24 23.5 5.9 1.17 5.04 1.092 3.08 31 30.8 3
6.72 0 1 7.75

01593500	38.0	15.8	15.5	1.40	11.04	0.623	3.21	23	18.7	6	7.48	0	0	10.58
01645200	3.70	67.4	2.7	1.16	2.33	0.000	1.47	14	28.0	6	1.91	0	0	2.75
01649500	72.8	27.2	15.3	1.33	11.54	0.192	1.83	33	22.0	5	8.85	0	0	7.25
01651000	49.4	19.7	19.1	1.36	14.05	0.047	4.00	19	22.0	6	6.45	0	0	6.58
01495000	52.6	17.9	22.2	1.41	15.80	0.053	4.75	14	1.92	0	9.87	0	0	8.88
01496200	9.03	29.0	5.9	1.36	4.33	0.000	2.08	4	0.00	0	4.38	0	0	5.81
01580000	94.4	17.7	27.3	1.52	17.92	0.039	3.40	27	0.42	0	7.29	0	0	7.50
01581657	4.16	74.2	3.7	1.19	3.12	0.000	2.34	33	5.25	0	4.08	0	0	3.83
01581658	5.22	56.1	4.8	1.28	3.74	0.000	2.68	31	4.78	0	4.38	0	0	4.92
01581700	34.8	30.0	15.8	1.60	9.89	0.000	2.81	21	2.37	2	4.68	0	0	3.50
01582000	52.9	33.8	15.0	1.35	11.14	0.015	2.35	32	0.91	0	6.84	0	0	6.62
01583100	12.3	50.9	7.8	1.08	7.25	0.092	4.27	26	0.29	0	5.77	0	0	4.50
01583500	59.8	24.5	15.9	1.40	11.36	0.064	2.16	22	0.16	0	8.20	0	0	8.08
01584050	9.40	70.0	4.8	1.11	4.32	0.000	1.99	13	1.00	0	3.05	0	0	3.00
01585105	2.65	65.2	3.6	1.14	3.16	0.000	3.77	16	5.22	0	3.86	0	0	4.00

```
01585500 3.29   56.0   3.5 1.11   3.14   1.165 3.00   21 0.45   0
3.08 0 0    3.12

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8.56 0 0    9.75

01586210 14.0   44.0   8.1 1.38   5.86   0.000 2.45   19 1.77   0
4.39 0 0    4.00

01586610 28.0   30.9 10.0 1.47   6.81   0.000 1.66   20 0.38   0
5.97 0 0    4.58

STANO      DA     SL    CL    SIN    BL     ST     SH   FOR   IA BDF   LT
AP CP   TC

01589440 25.2   38.2   9.5 1.37   6.95   0.000 1.92   34 9.92   2
5.29 0 0    6.92

01591000 34.8   28.2 12.2 1.22 10.02   0.000 2.89   21 0.21   0
6.51 0 0    7.12

01591400 22.9   28.0   8.7 1.35   6.44   0.097 1.81   16 1.52   0
6.16 0 0    6.83

01591700 27.0   26.5 10.9 1.28   8.52   0.141 2.69   19 2.08   0
5.28 0 0    6.83

01593710 48.4   17.8 14.7 1.28 11.45   0.000 2.71   24 2.16   0
5.99 0 0    8.25

01594000 98.4   13.6 23.5 1.33 17.62   0.134 3.16   26 6.52   4
10.83 0 0    9.88

01641510 0.40 817.8   0.9 1.09   0.83   0.000 1.72 100 0.00   0
4.26 0 0      .

01643495 0.15 1000.   0.5 1.13   0.44   0.000 1.29 100 0.00   0
1.26 0 0    1.75

01643500 62.8   23.8 15.6 1.43 10.89   0.000 1.89   23 1.19   0
7.30 0 0    8.35

01645000 101.   14.0 21.2 1.56 13.61   0.120 1.83   25 3.15   4
10.88 0 0    4.31
```
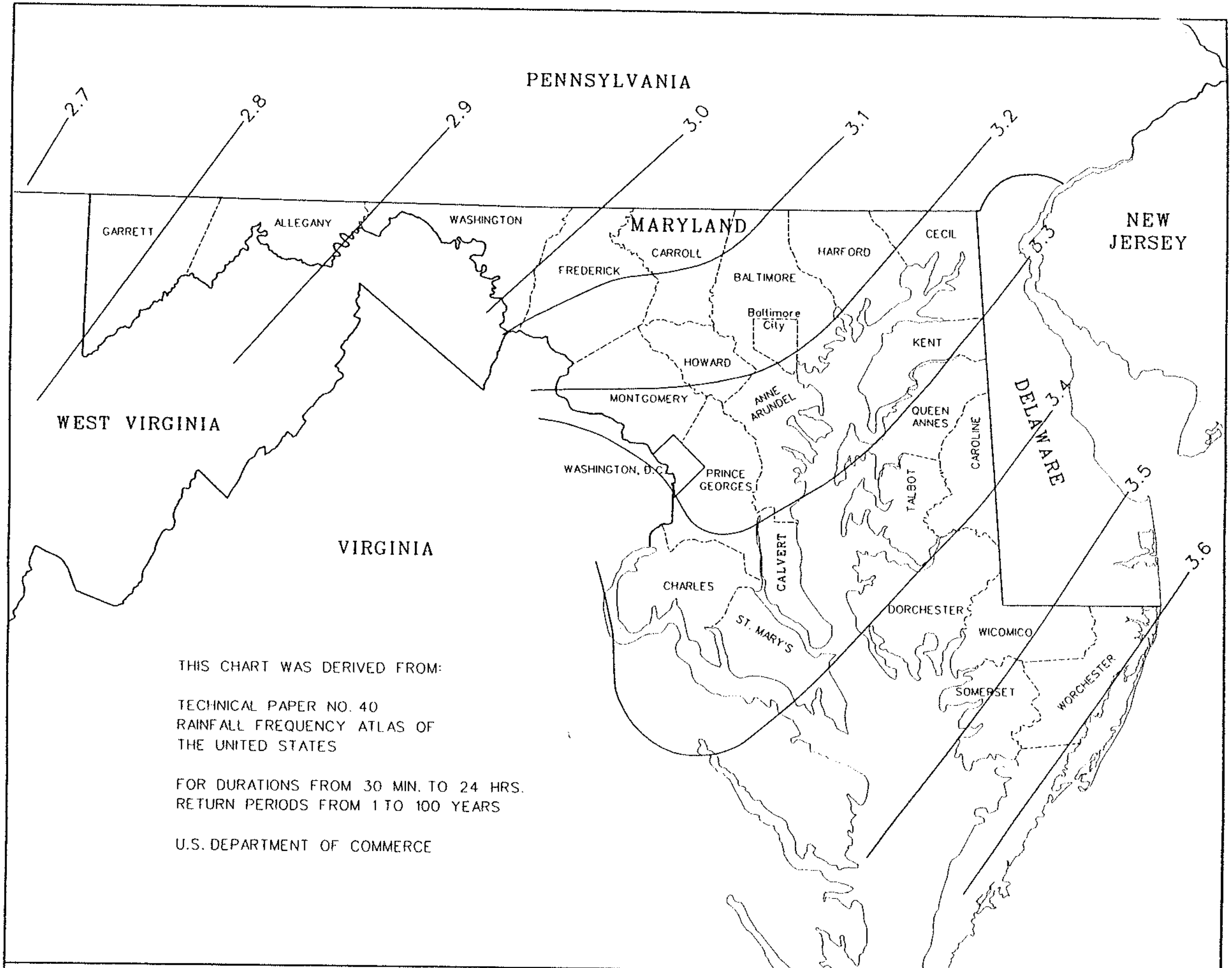
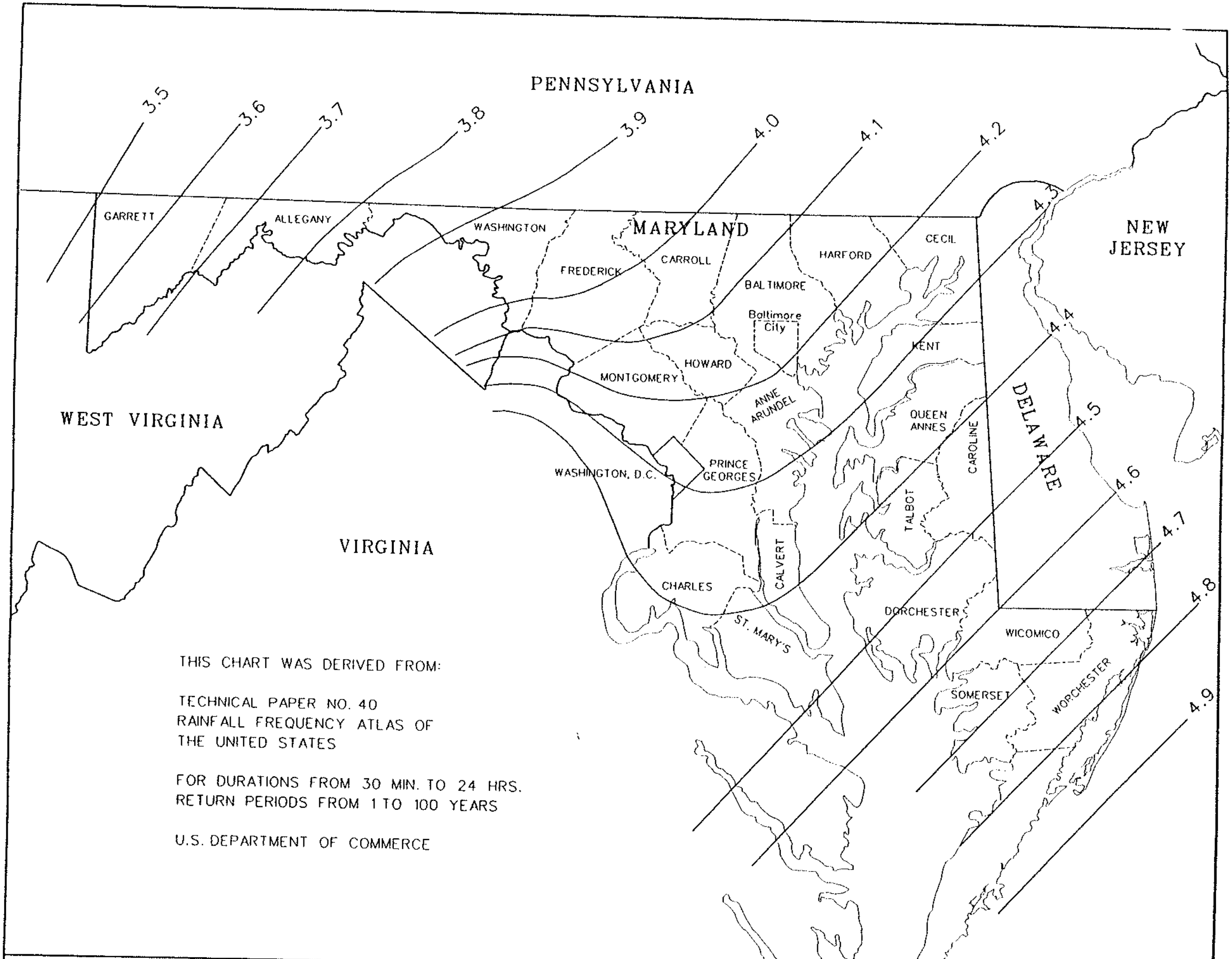

APPENDIX 6

Rainfall Maps For Maryland
TP-40



2 YEAR - 24 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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5 YEAR - 24 HOUR RAINFALL - INCHES

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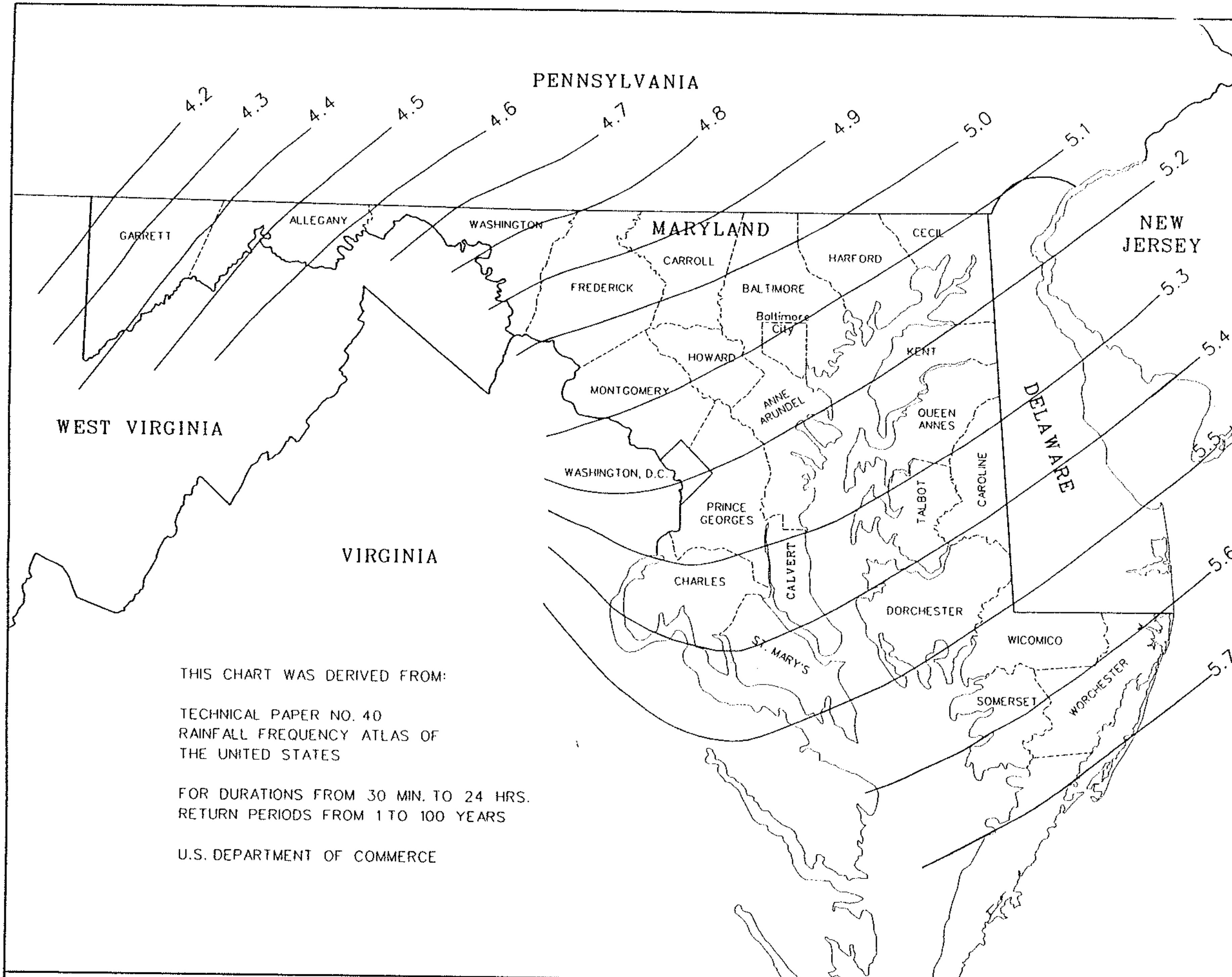

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10 YEAR - 24 HOUR RAINFALL - INCHES

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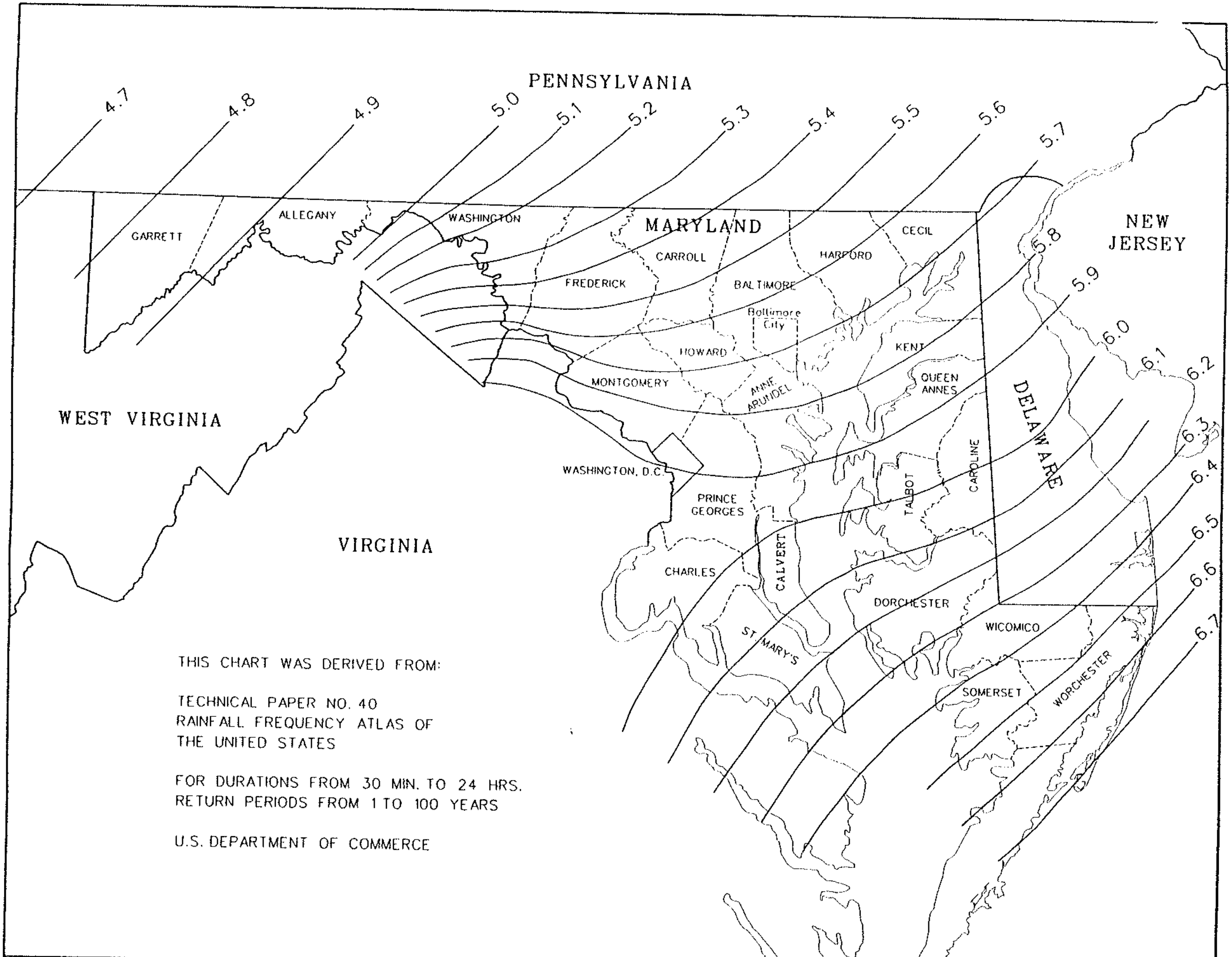

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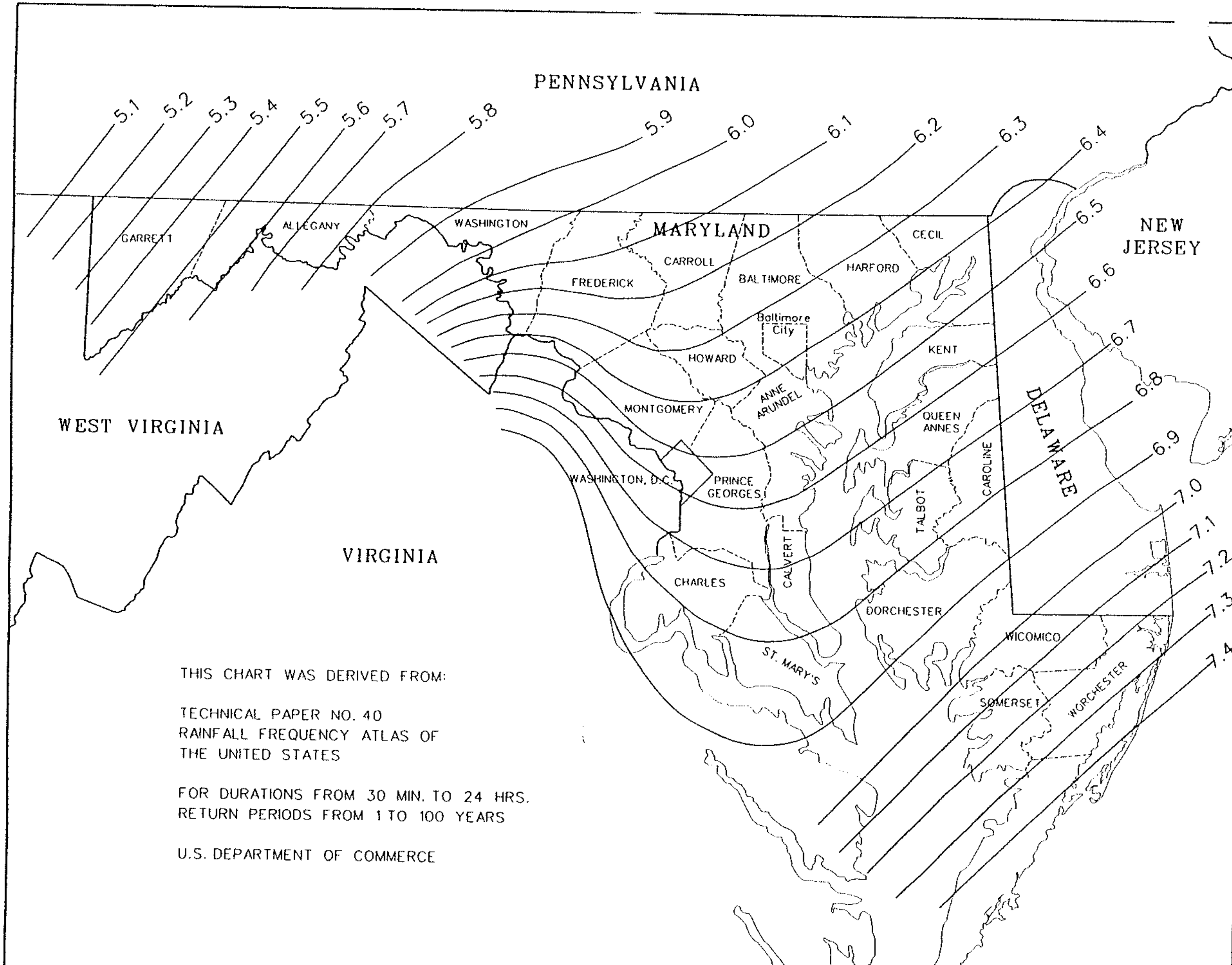

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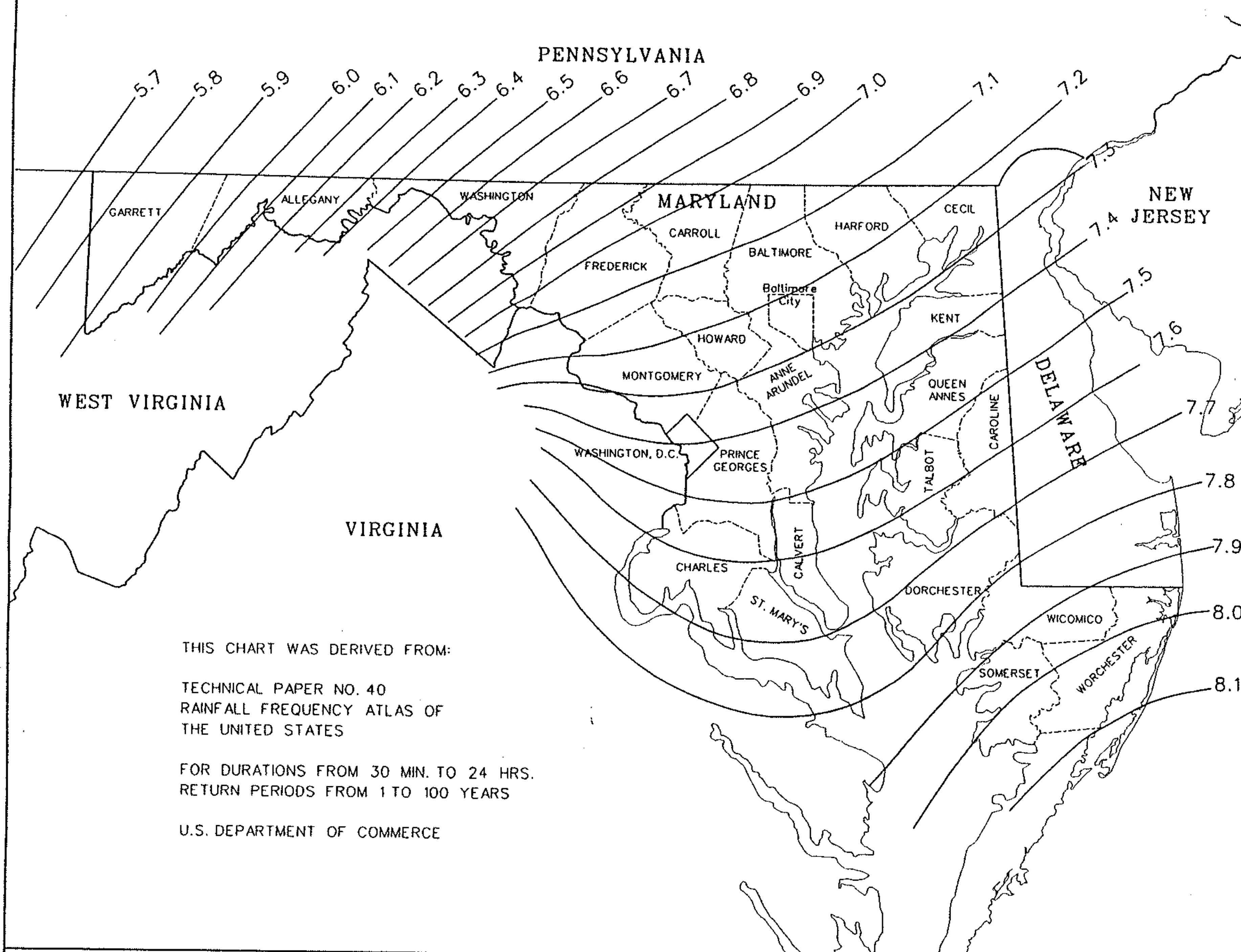
50 YEAR - 24 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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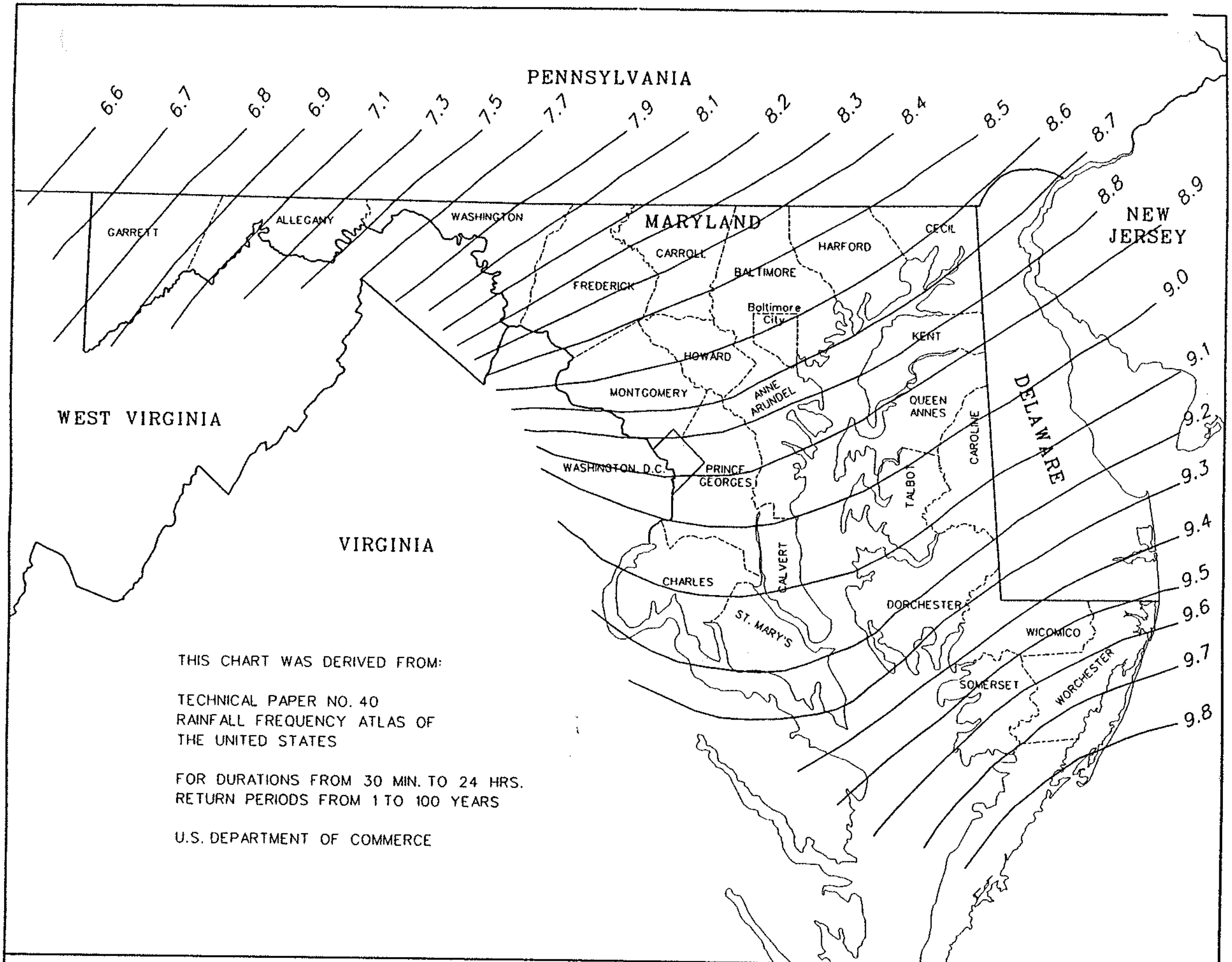
RETURN PERIODS FROM 1 TO 100 YEARS

U.S. DEPARTMENT OF COMMERCE

100 YEAR - 24 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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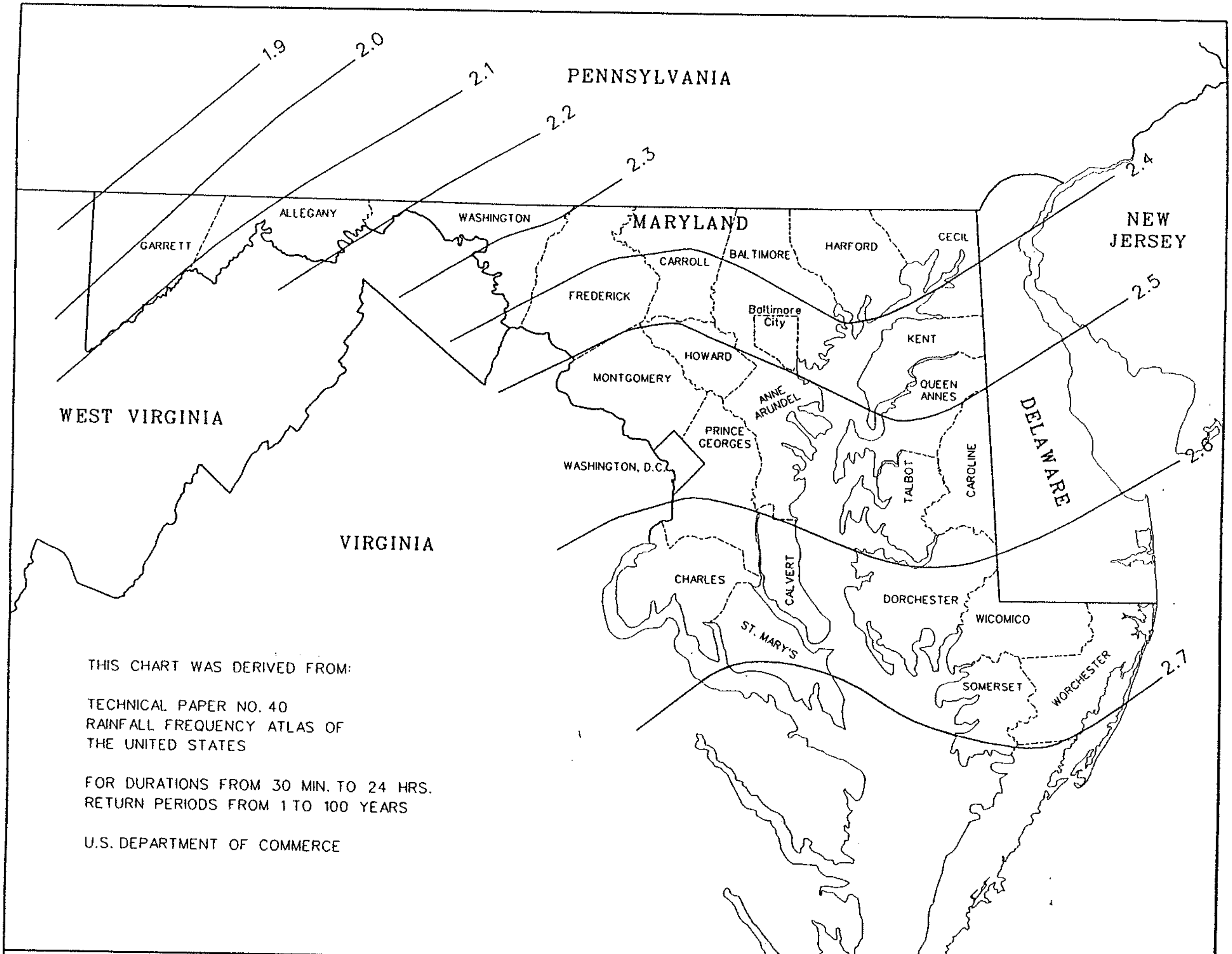
FOR DURATIONS FROM 30 MIN. TO 24 HRS.
RETURN PERIODS FROM 1 TO 100 YEARS

U.S. DEPARTMENT OF COMMERCE

500 YEAR - 24 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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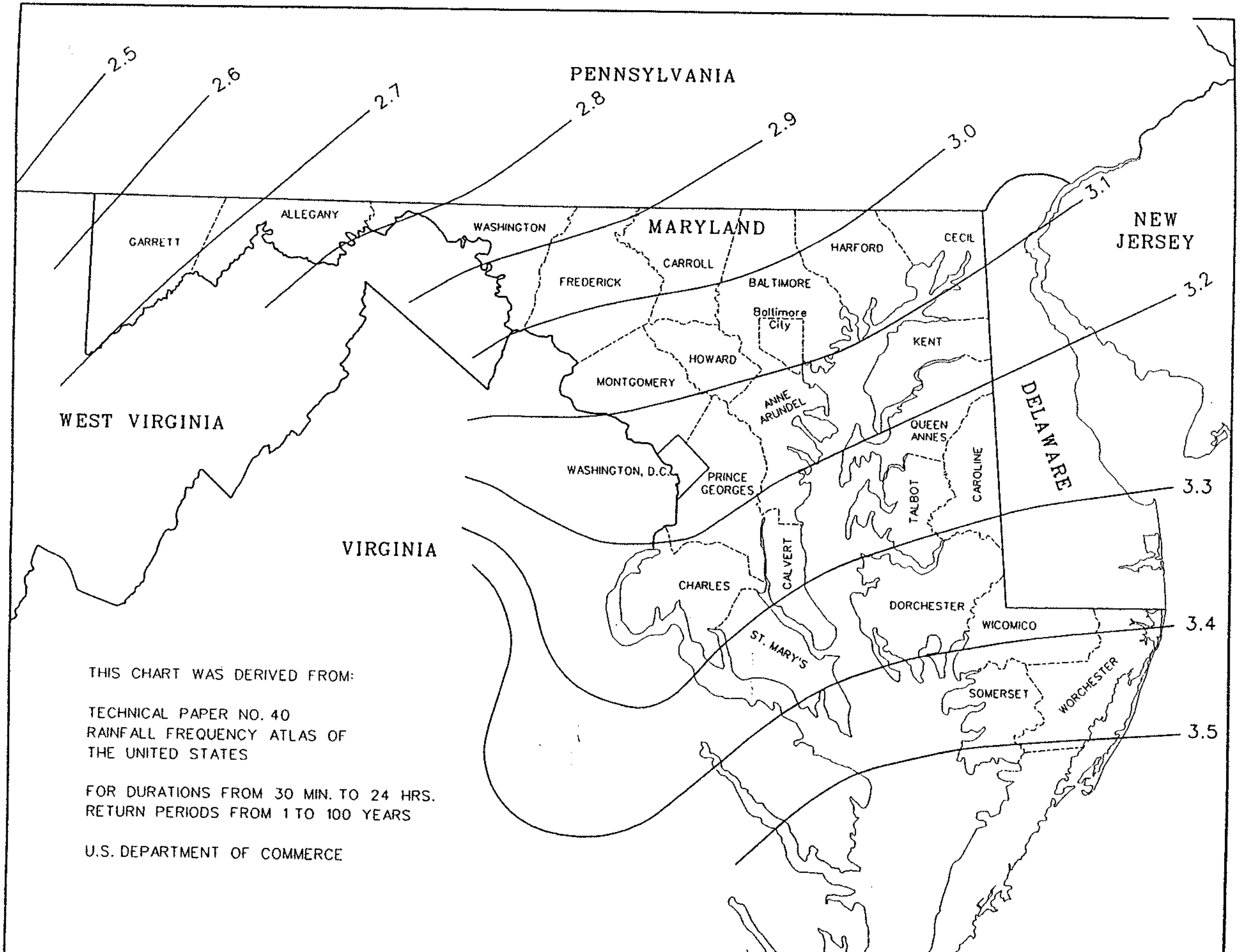
THE UNITED STATES

FOR DURATIONS FROM 30 MIN. TO 24 HRS.
RETURN PERIODS FROM 1 TO 100 YEARS

U.S. DEPARTMENT OF COMMERCE

2 YEAR - 6 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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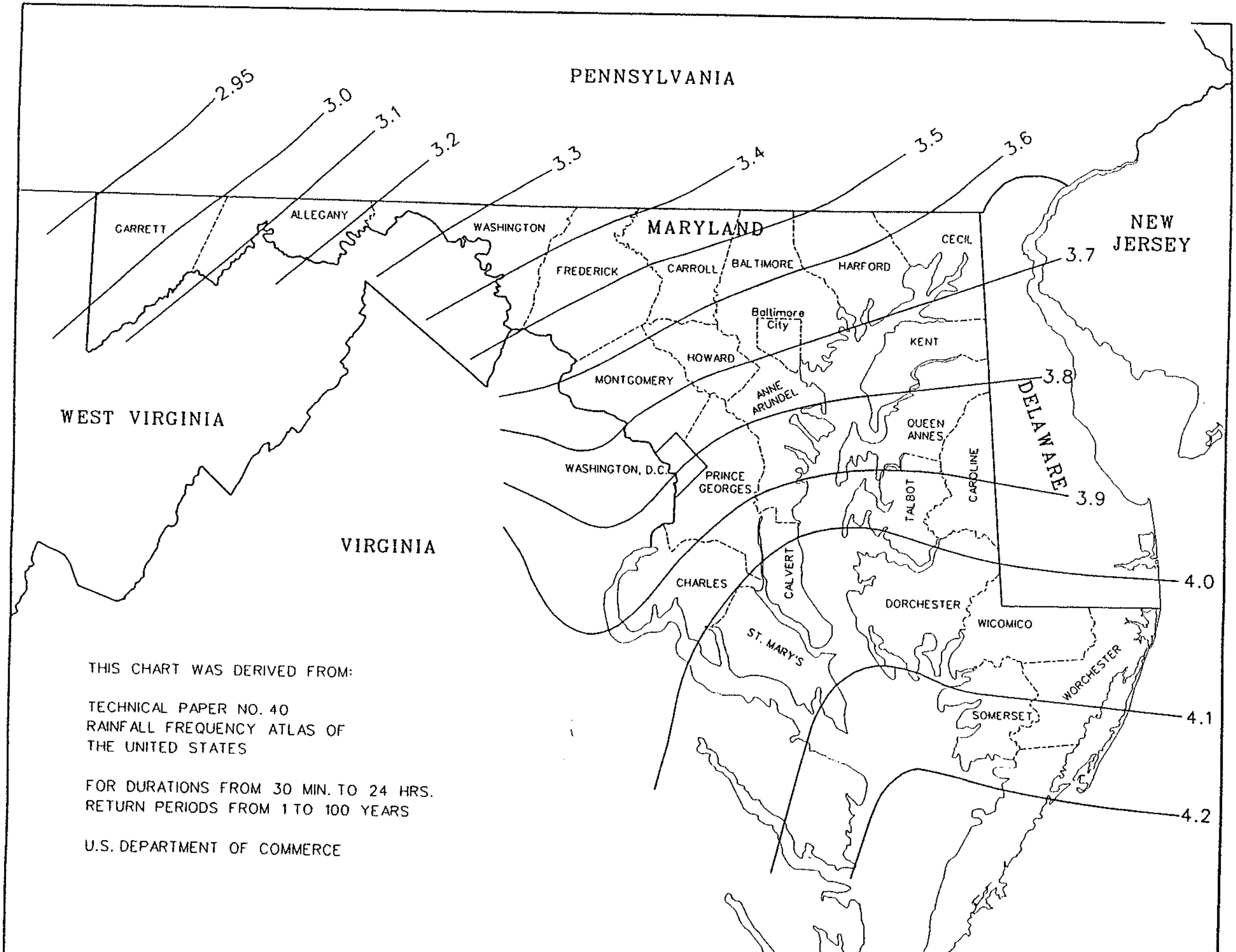
RAINFALL FREQUENCY ATLAS OF
THE UNITED STATES

FOR DURATIONS FROM 30 MIN. TO 24 HRS.
RETURN PERIODS FROM 1 TO 100 YEARS

U.S. DEPARTMENT OF COMMERCE

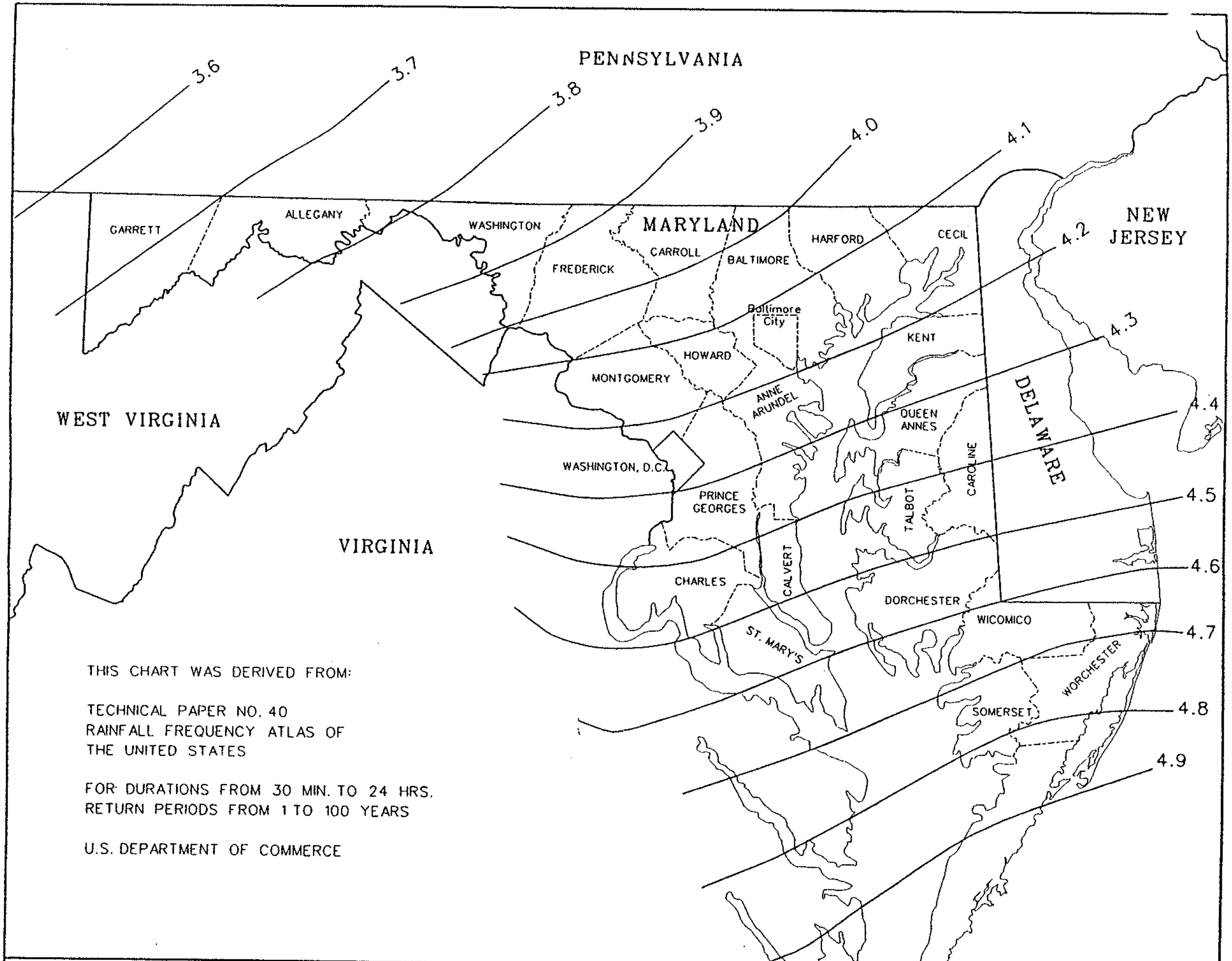
5 YEAR - 6 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



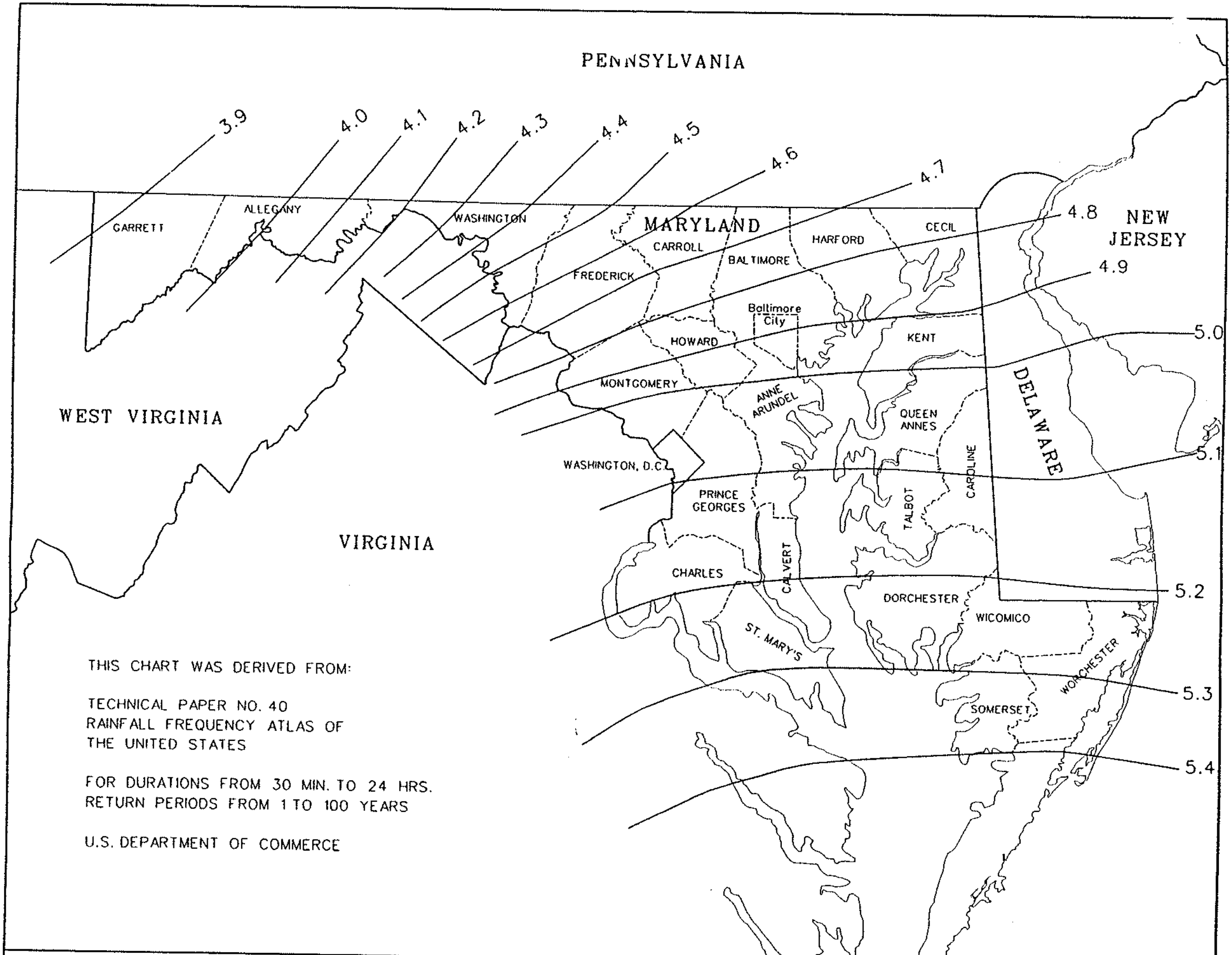
10 YEAR - 6 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



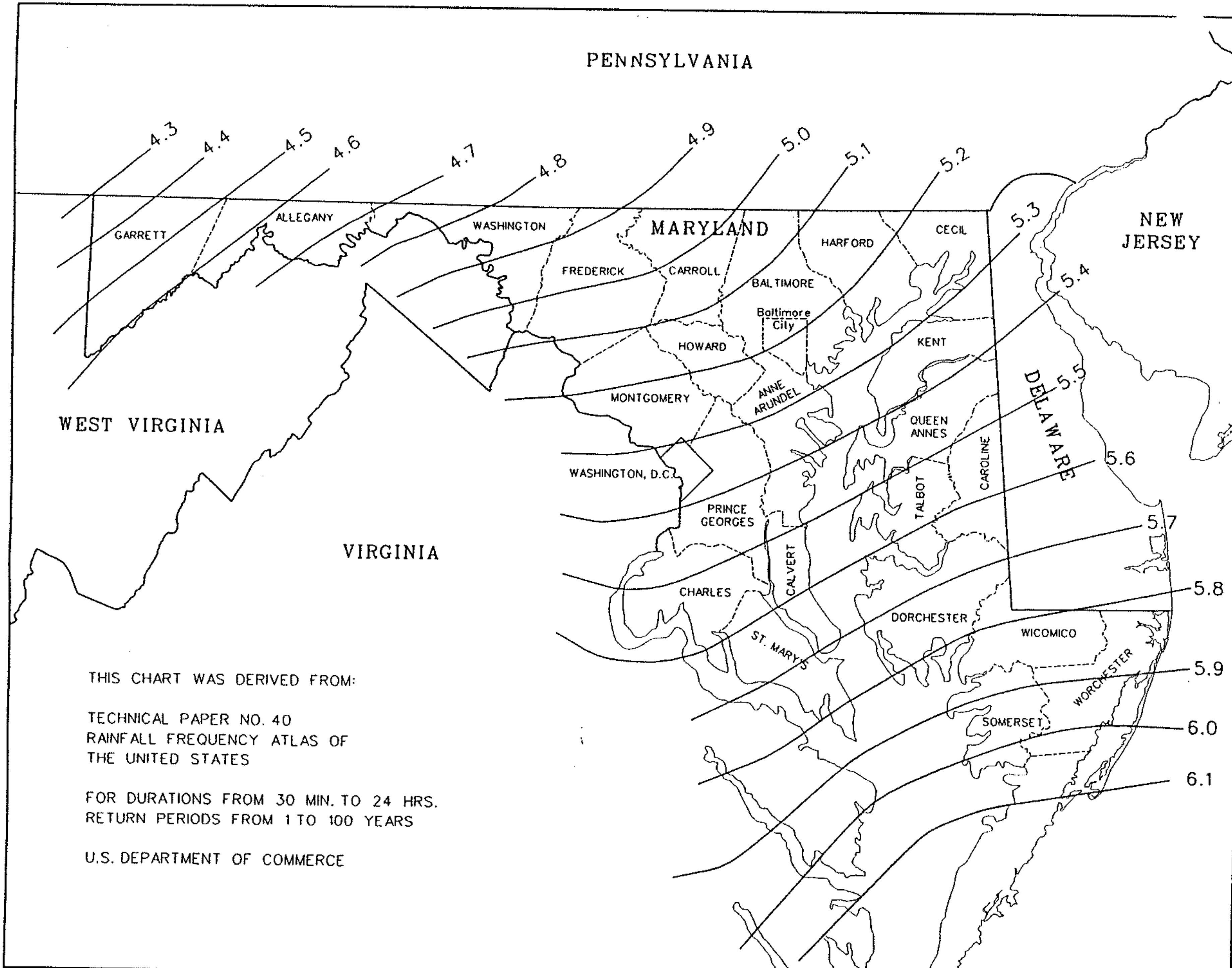
25 YEAR - 6 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



50 YEAR - 6 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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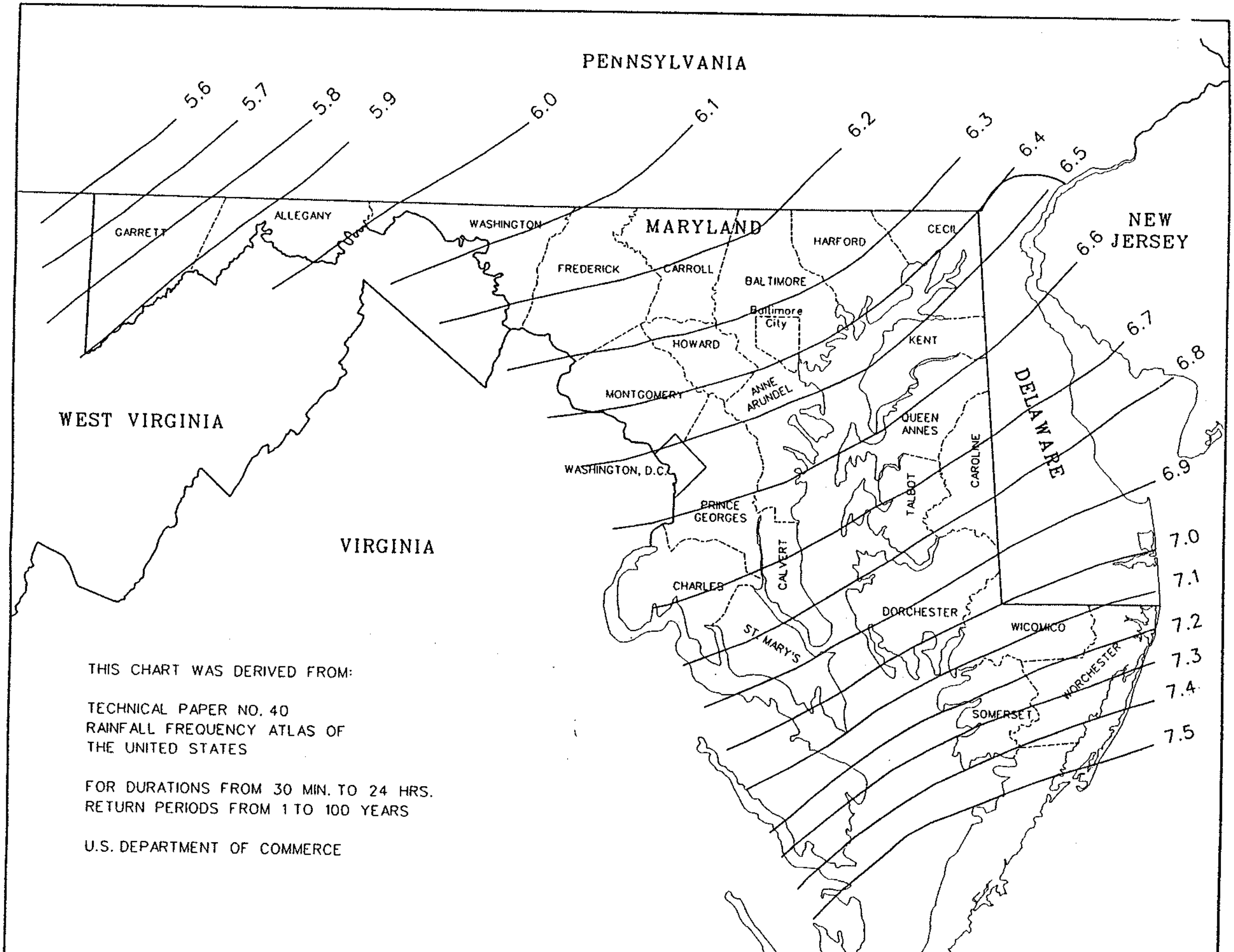
FOR DURATIONS FROM 30 MIN. TO 24 HRS.
RETURN PERIODS FROM 1 TO 100 YEARS

U.S. DEPARTMENT OF COMMERCE

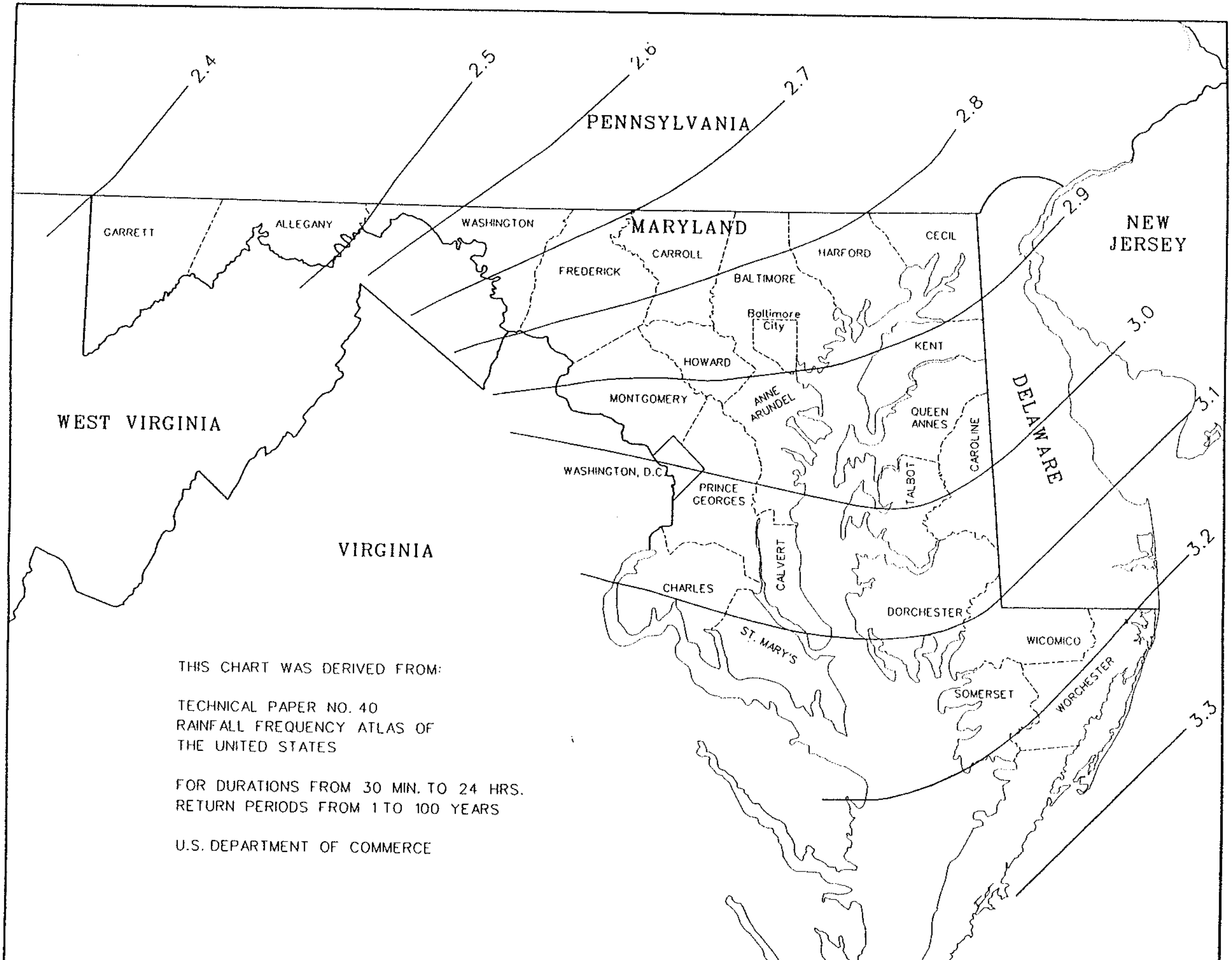
100 YEAR - 6 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



500 YEAR - 6 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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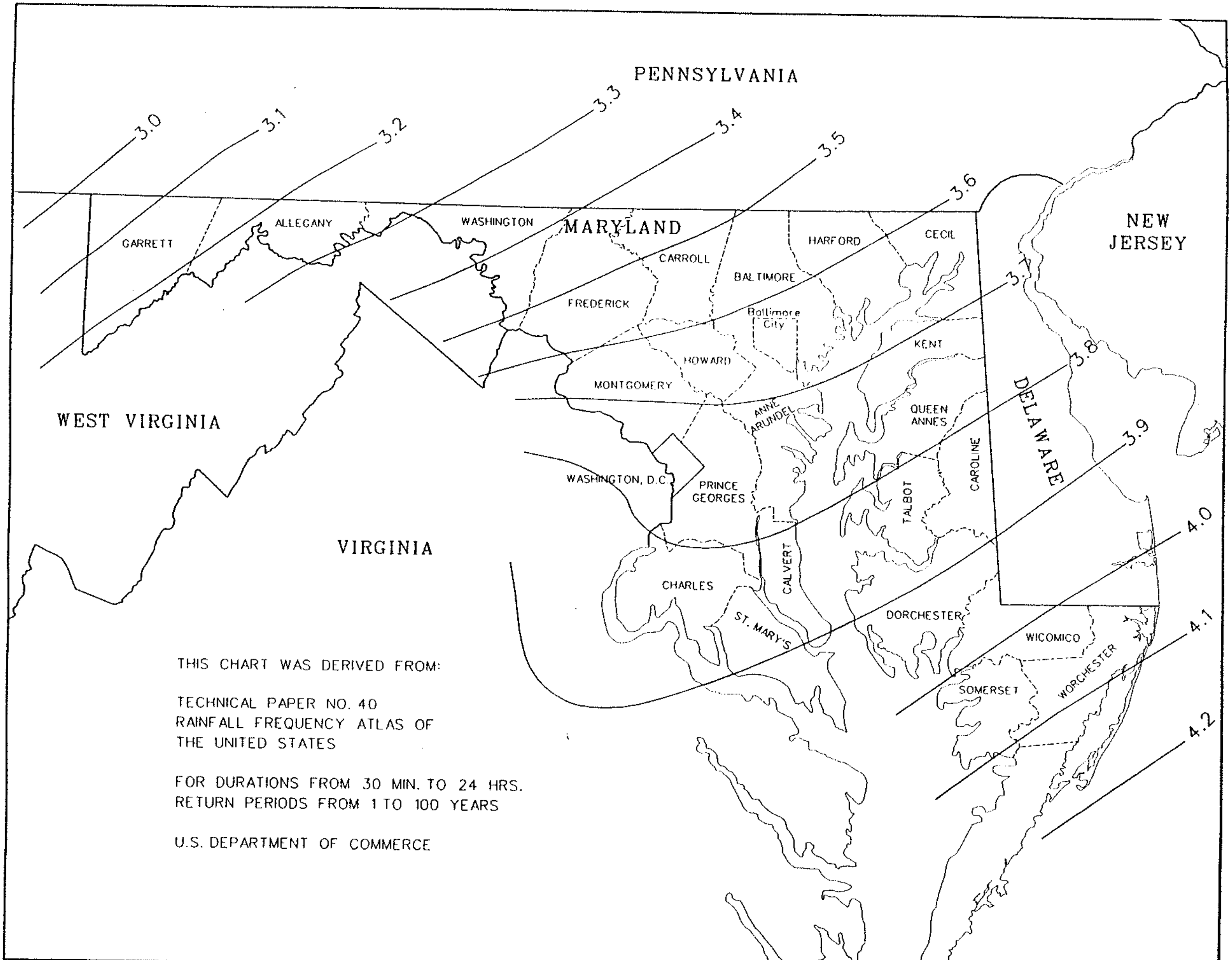
U.S. DEPARTMENT OF COMMERCE

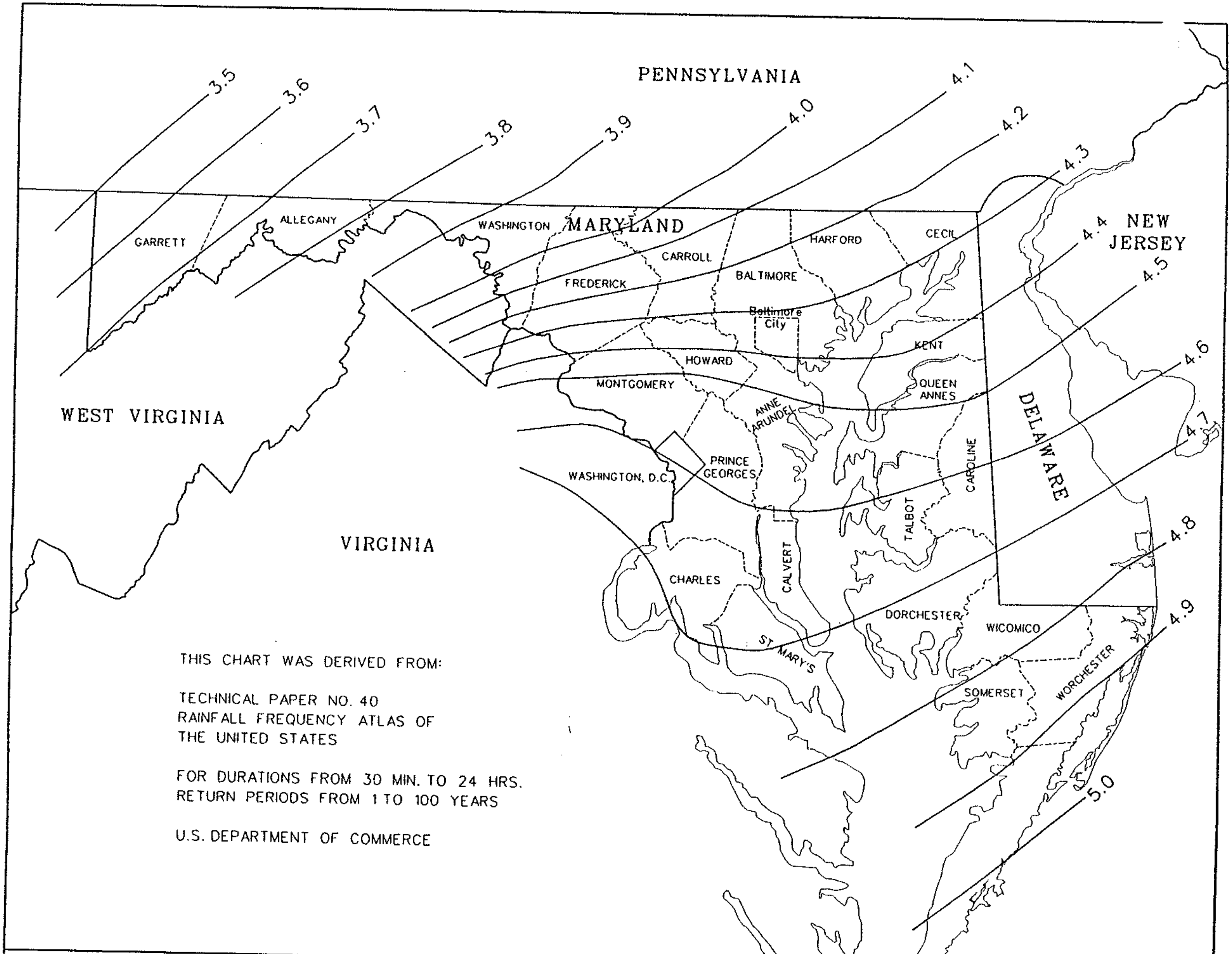
2 YEAR - 12 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



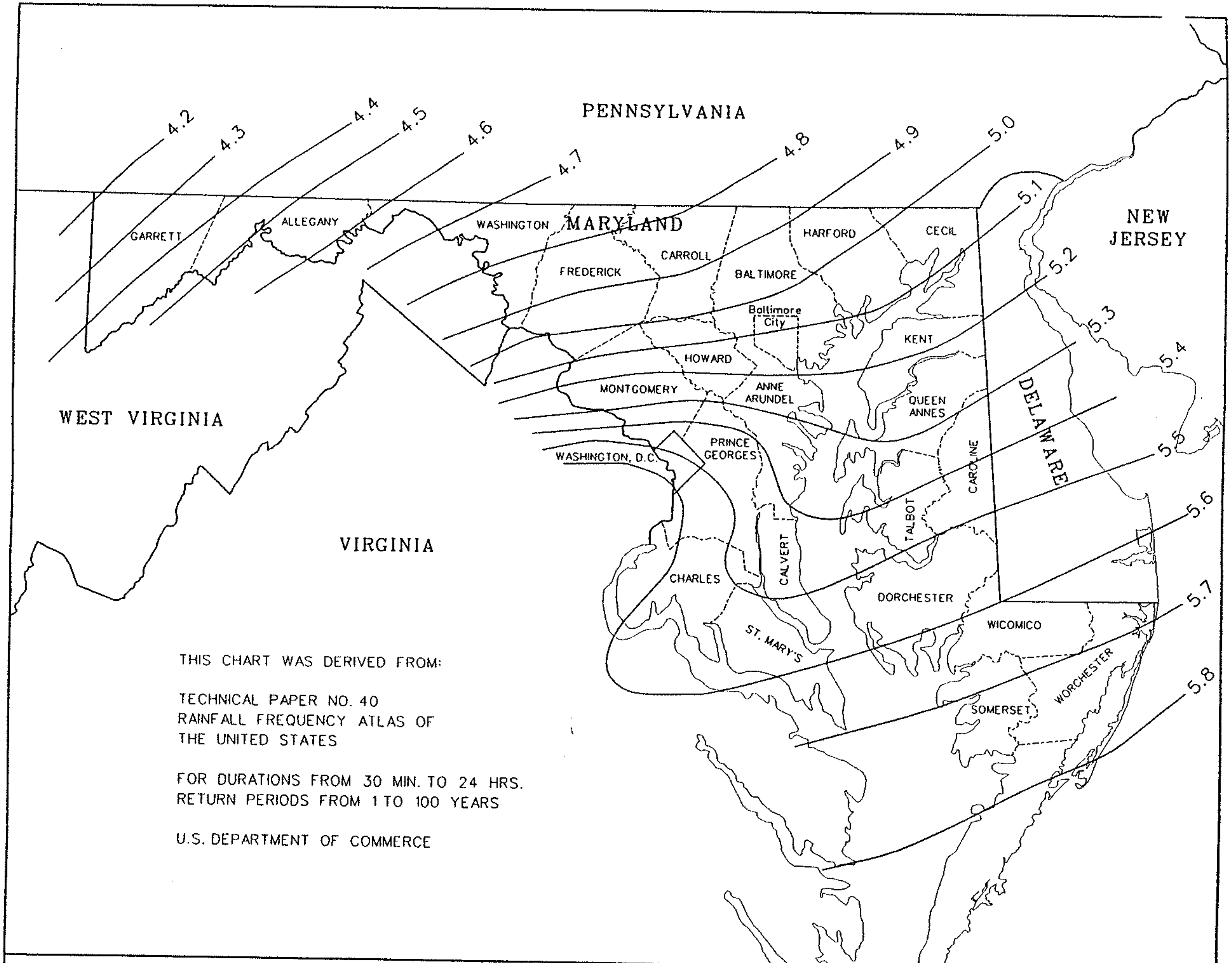
5 YEAR - 12 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



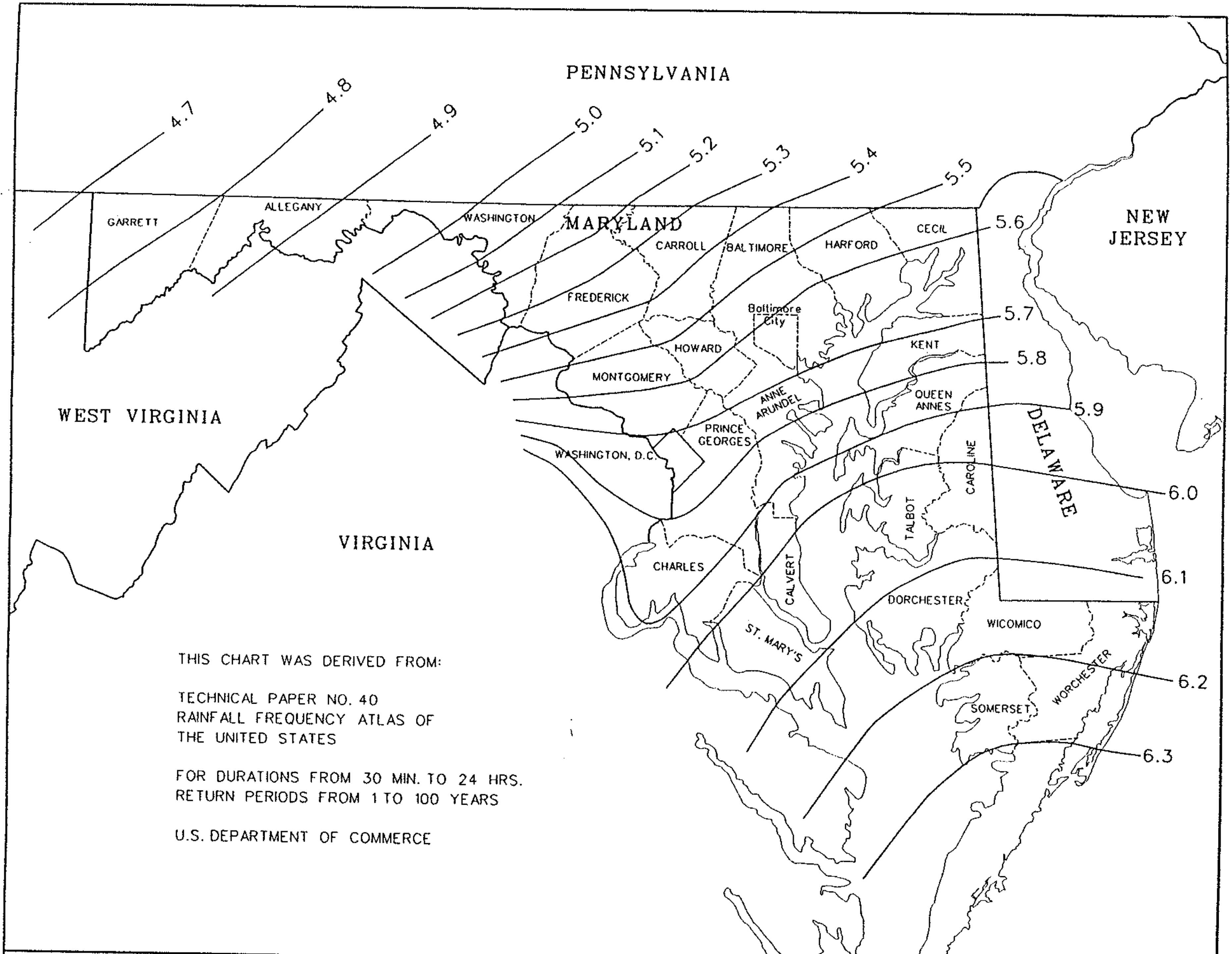
10 YEAR - 12 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



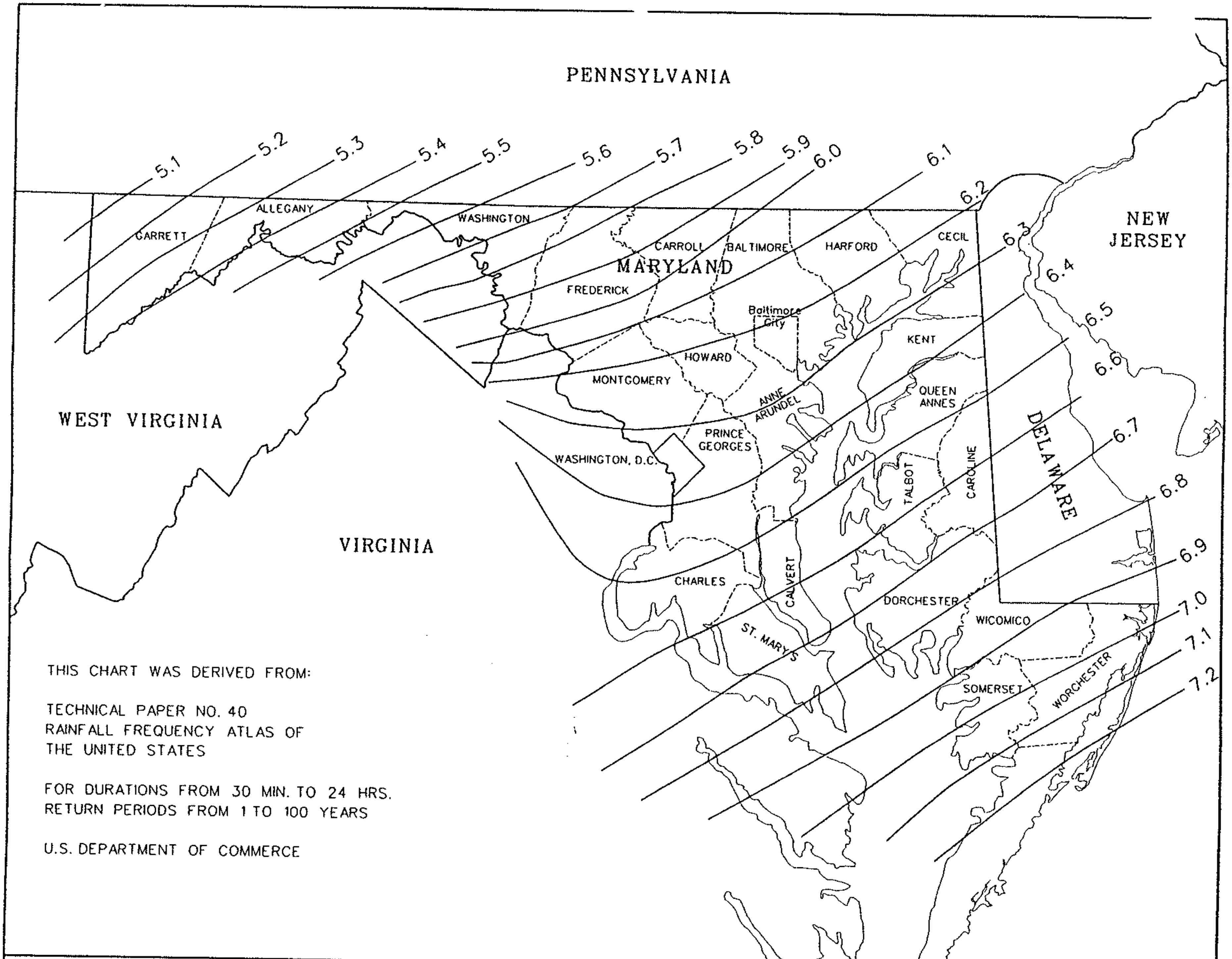
25 YEAR - 12 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



50 YEAR - 12 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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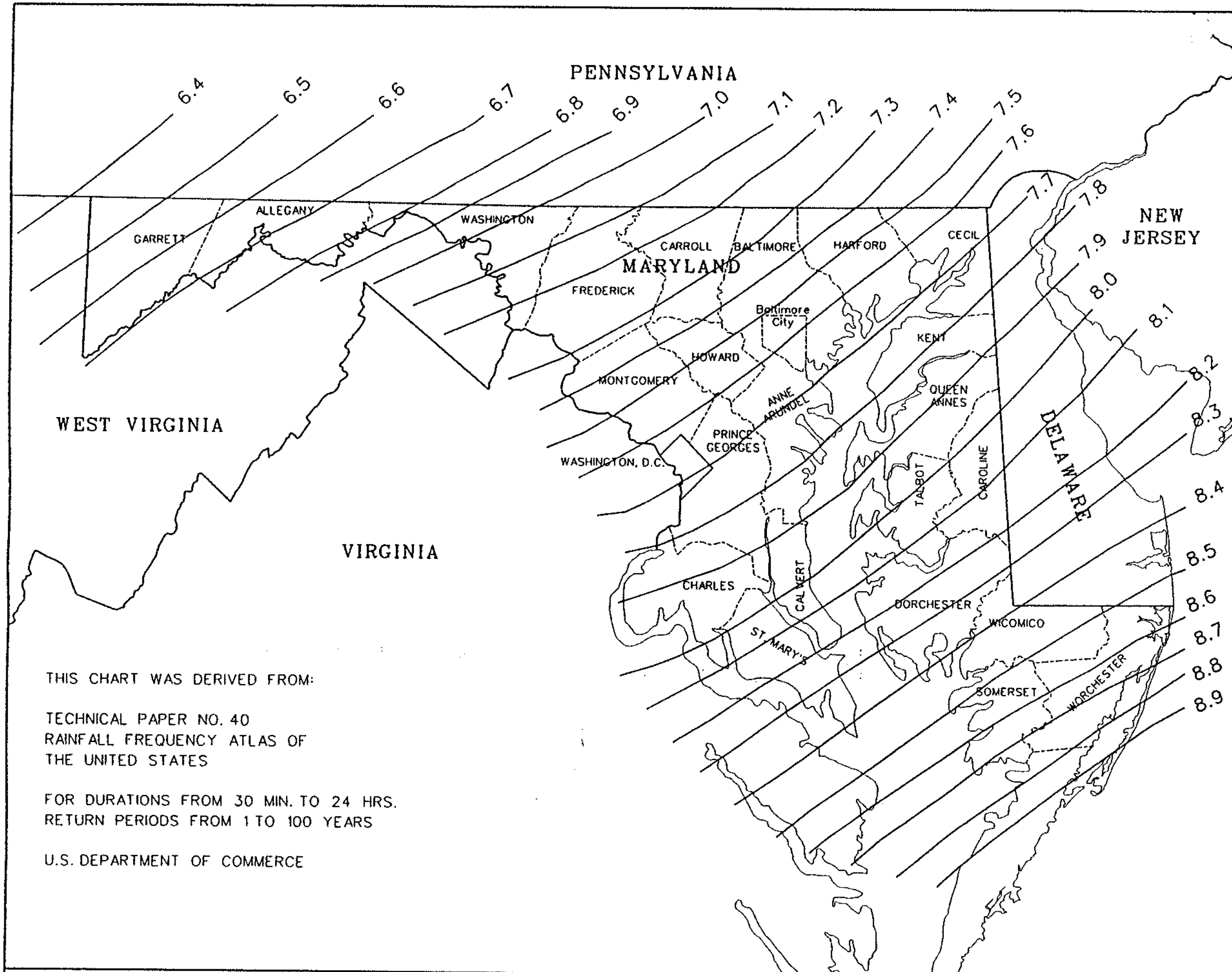
U.S. DEPARTMENT OF COMMERCE

100 YEAR - 12 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION



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U.S. DEPARTMENT OF COMMERCE

500 YEAR - 12 HOUR RAINFALL - INCHES

MARYLAND STATE HIGHWAY ADMINISTRATION

ADDITION TO REPORT:

Please add to the Hydrology & Hydraulic Panel Report
6-hour and 12-hour Duration Rainfall Tables new figures.

If you have any questions, please give Mr. Len Podell of SHA a call at (410) 545-8363

6-hour and 12-hour Duration Rainfall Tables

2-6hr.txt

```
5 RAINFL  5         0.1
8         0.00000   0.00455   0.00910   0.01365   0.01820
8         0.02274   0.02741   0.03230   0.03741   0.04276
8         0.04833   0.05419   0.06039   0.06693   0.07381
8         0.08103   0.08870   0.09695   0.10576   0.11514
8         0.12509   0.13601   0.14829   0.16194   0.17695
8         0.19333   0.22722   0.29477   0.40341   0.59826
8         0.73351   0.76046   0.78417   0.80464   0.82187
8         0.83586   0.84785   0.85911   0.86963   0.87941
8         0.88845   0.89693   0.90500   0.91268   0.91995
8         0.92683   0.93337   0.93963   0.94560   0.95128
8         0.95669   0.96190   0.96701   0.97202   0.97693
8         0.98175   0.98645   0.99107   0.99558   1.00000
9 ENDTBL
```

12hr.txt

```
5 RAINFL  5         0.1
8         0.00000   0.00216   0.00434   0.00654   0.00877
8         0.01102   0.01330   0.01560   0.01792   0.02027
8         0.02264   0.02504   0.02746   0.02990   0.03236
8         0.03486   0.03737   0.03991   0.04247   0.04506
8         0.04767   0.05035   0.05315   0.05607   0.05910
8         0.06226   0.06554   0.06894   0.07245   0.07609
8         0.07984   0.08365   0.08747   0.09128   0.09509
8         0.09890   0.10281   0.10691   0.11120   0.11568
8         0.12035   0.12526   0.13046   0.13594   0.14171
8         0.14776   0.15420   0.16111   0.16850   0.17636
8         0.18470   0.19385   0.20415   0.21559   0.22817
8         0.24190   0.27031   0.32693   0.41801   0.58135
8         0.69472   0.71731   0.73719   0.75435   0.76879
8         0.78051   0.79057   0.80001   0.80883   0.81703
8         0.82460   0.83171   0.83848   0.84491   0.85101
8         0.85678   0.86226   0.86750   0.87251   0.87727
8         0.88180   0.88618   0.89045   0.89466   0.89877
8         0.90281   0.90676   0.91063   0.91441   0.91811
8         0.92172   0.92526   0.92871   0.93208   0.93535
8         0.93856   0.94167   0.94471   0.94765   0.95052
8         0.95330   0.95603   0.95872   0.96139   0.96402
8         0.96663   0.96921   0.97176   0.97427   0.97676
8         0.97922   0.98165   0.98404   0.98642   0.98875
8         0.99106   0.99334   0.99559   0.99781   1.00000
9 ENDTBL
```