CHAPTER 9

HYDROLOGIC ANALYSIS

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METHODS AND APPLICATIONS

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1.1 INTRODUCTION

Due to the complexity of the natural terrain, orographic effects of the Rocky Mountains, and semi-arid climate of the region, the type and duration of storm events vary substantially within the State. However, rainstorm events can be generally defined as either short-duration convective storms or long-duration general rainstorms.

The short-duration convective storms (cloudbursts/thunderstorms) can produce high rainfall intensities for a short period and generally cover smaller watersheds. Convective storms are commonly known to be responsible for high peak flows and flooding problems for many small drainage basins. The long-duration general rainstorms can produce rain coverage over a large watershed area for a period ranging from several hours up to several days. General rainstorms can produce large amounts of total rainfall runoffs and sometimes generate higher peak flows than the convective storms especially in larger watersheds. Depending on the purpose of the hydrologic analysis, it may be necessary to analyze both types of rainstorms in order to estimate the high peak flow rate and the high runoff volume for a given drainage basin.

There are many different flow estimation analysis methods available. However, not all of the methods can be effectively utilized in Colorado. Some methods are not applicable for the hydrologic conditions that exist in Colorado, and other methods cannot be utilized easily or accurately due to the lack of measured data. Also, the computed flow estimates may vary considerably depending on the methods utilized for a given watershed. Therefore, it is necessary to define minimum standards for hydrologic analysis in order to promote accuracy and consistency in the computed flow rates.

Presented in this section are a list of accepted hydrologic analysis methods and approaches and their appropriate uses in the State of Colorado. The detailed descriptions of the listed analysis methods are provided in Sections 2 thru 5 of Chapter 9. The information presented in this chapter is the most current information available at the time of preparation of this Manual and should be updated as better techniques and new rainfall and stream gage data become available in the future.

1.2 HYDROLOGIC ANALYSIS

The following hydrologic analysis methods have been used and accepted widely throughout the State to estimate flow rates and hydrographs resulting from surface runoffs for various design storm events:

- Statistical analysis of recorded stream gage data (Section 2, Chapter 9)
- Regional regression analysis (Section 3, Chapter 9)
- Synthetic rainfall-runoff modeling (Sections 4 & 5, Chapter 9)

Detailed discussions on the above analysis methods and example applications are provided in the subsequent sections of this chapter. Flow rates and hydrograph estimates for the purpose of floodplain analysis and drainage design in Colorado should be computed using one or more of the listed hydrologic analysis methods.

1.2.1 ANALYSIS REQUIREMENTS

For detailed floodplain/floodway delineation projects, the hydrologic analysis should include, at a minimum, calculations for the 10-, 50-, 100-, and 500-year frequency discharges. It is optional but recommended that the peak discharge for 2- and 5-year flood events be calculated in addition to the other discharges. If using a rainfall-runoff modeling, 500-year flow rates may be estimated by multiplying the 100-year flow rates by a factor of 1.7 (FHWA, HEC-18).

For approximate floodplain delineation projects, the 100-year frequency discharge should be estimated at a minimum.

Flow hydrographs (total flow volume, timing of peak flows, etc.) should also be computed if it is necessary to determine effects of flow routings, detentions/dam storages, diversion flows, etc.

1.2.2 PREVIOUS STUDIES

Where appropriate, previously approved hydrologic studies should be used so that previous work by federal, state, or local agencies is not duplicated. When such data is not available, conditions have changed significantly, or the methodologies or data used in previous studies are not appropriate, a new hydrologic analysis for each stream should be prepared.

If a new hydrologic analysis is prepared, a comparison of new discharges with all available published or not published discharge data that exist for the study area should be provided. If the new hydrologic analysis results are significantly different than the previously adopted flows, the following criteria should be used in deciding which flow estimate should be used. However, the site-specific limitations/conditions may warrant a deviation from the evaluation criteria below. The project engineer should coordinate with the appropriate agencies in deciding which flow estimate should be used.

If a new hydrologic analysis is prepared, a comparison of new discharges with all available published or not published discharge data that exist for the study area should be provided.

For streams with at least 50 years of stream-flow gage records, the following general FEMA evaluation criteria should be used.

- The latest discharges should be adopted if the previously established discharges do not fall within the 95 and 5 percent confidence limits (90 percent confidence interval) of the most recent estimates; the previously established discharges should be adopted if they fall within the 75 and 25 percent confidence limits (50 percent confidence interval) of the most recent estimates.

- For all other cases, the new hydrologic analysis results should be used if the new analysis is proven to be technically superior and if the resulting peak flow rate change is greater than 10%.

1.3 APPLICATIONS

The following guidelines should be used in determining the appropriate hydrologic analysis method for a given waterway. A method selection matrix table is provided at the end of this section as Table CH9-T101.

- When at least 50 years of stream-flow gage records are available, a flow frequency statistical analysis should be performed to determine the flood peaks of the selected recurrence intervals.

- When 25 to 50 years of stream-flow gage records are available, the hydrologic analysis should include a statistical analysis and a comparison with established flow rates for similar watersheds. Similar watersheds are defined as watersheds that have similar hydrologic characteristics (precipitation depth and distribution, slope, size, elevation, vegetation cover, etc.) as the watershed being studied.

  If the estimated flow rates using the statistical method are determined inaccurate after comparison with similar watersheds, additional hydrologic analysis should be performed using a regional regression analysis and/or synthetic rainfall-runoff modeling methods to validate the flow rates.

- When 10 to 24 years of stream-flow gage records are available, the hydrologic analysis should include a statistical analysis, comparisons with similar watersheds, and flood hydrograph estimates using synthetic hydrologic models and precipitation records.

  The estimated flow rates for 2-, 5-, and 10-year design storm events using the statistical method should be reasonably accurate. However, the estimated flow rates for 50-, 100-, and 500-year storm events using the statistical method may not be reliable, since only 10 to 24 years of stream flow gage records are used in the analysis.

  A rainfall-runoff model should be prepared and calibrated to the estimated 10-year flow rates using the statistical method. Then, the calibrated rainfall – runoff model may be used to estimate flow rates for other design storm events.

  All drainage basin characteristics that affect the rainfall-runoff relationship should be documented, including, but not limited to, delineation of basin and subbasin boundaries, size, shape, length, slope, general aspect, elevation extremes, time of concentration, land use, and soil types and compositions. When actual precipitation records of major recorded storm events are available from area rain gage stations, such data should be used in conjunction with rainfall data.

- When less than 10 years of stream-flow gage records are available, the hydrologic analysis should include a regional regression analysis and flood hydrograph estimates using synthetic hydrologic models and precipitation records.

  The computed flows using a rainfall-runoff model should be compared to the confidence limits of the existing flows on estimated flow rates using a regional regression analysis.

1.3.1 ADDITIONAL REQUIREMENTS

Depending on the floodplain analysis and drainage design requirements, it may be necessary to develop a synthetic rainfall-runoff model, even when sufficient amount of stream-flow gage records are available. The synthetic rainfall-runoff models should be calibrated to match the statistical analysis results. The calibrated synthetic model can then be used to generate hydrographs for various design storm events. The following is a list of some of these cases:

- Various flood frequency hydrographs are required, but the statistical analysis alone cannot generate the necessary hydrographs.
- The subject watershed is undergoing or projected to undergo a substantial amount of new development.

The synthetic rainfall-runoff models should be calibrated to match the statistical analysis results. The calibrated synthetic model can then be used to generate hydrographs for various design storm events.

- Comparison of before and after development hydrographs to quantify potential increase in flows due to the proposed developments.

1.4 WATERSHED DEVELOPMENT CONDITIONS

Hydrologic analysis should be performed, at a minimum, to reflect the existing watershed development conditions. Public works projects in progress that are planned to be completed within 12 months following the hydrology study completion should be included in the analysis. Where construction of a publicly owned, operated and maintained flood control facility will not be completed within 12 months following completion of the study, but adequate progress has been made, the impact/benefit of the project may be included in the hydrologic analysis. The project engineer should coordinate with the public agency in charge of the facility design and construction, affected local agencies and Colorado Water Conservation Board (CWCB) to determine whether to include the subject facility in the existing conditions hydrologic analysis or not.

As new developments occur, the estimated existing conditions peak flow rates may change substantially, depending on the nature and amount of new developments within a watershed. Therefore, local communities are encouraged to develop future (built-out) conditions flow rates and floodplain information in addition to the existing conditions, especially when the area plan indicates substantial amount of future developments.

1.5 DETENTION FACILITIES

The hydrologic analysis should include detention facilities designed and constructed with the purpose of impounding water for flood detention that are owned, operated, and maintained by a government body. Detention structures that are randomly located, privately owned, or privately maintained should not be included in the hydrologic analyses unless it can be shown that they exacerbate downstream peak discharges.

If existing detention basins are not included in the hydrologic analysis, discussions should be provided in the report describing the detention basins and reasons why they were not considered in the analysis.

1.5.1 STORAGE ROUTING METHOD

Only the storage specifically reserved for the flood attenuation purposes should be included in the analysis.

The flow attenuation effect of a detention basin can be determined using the Modified Puls routing method. The Modified Puls routing method can be used in HEC-1, HEC-HMS, and UDSWM computer programs to route hydrographs through dams and reservoirs. Only the storage specifically reserved for the flood attenuation purposes should be included in the analysis. Detailed discussions on the Modified Puls routing method and the use of the hydrologic computer programs are provided in Section 5, Chapter 9.

1.6 SPECIAL HYDROLOGIC CONDITIONS

The following hydrologic conditions may exist within the study watersheds:

- Rain on snow
- Vegetation cover loss due to fire
- Flow diversion structures
- Frozen soils
- Effects of dams and reservoirs

These hydrologic conditions should not be ignored in the watershed analysis. The practical ways to deal with the listed conditions vary depending on the selected analysis method, and they are described in the subsequent sections of this chapter. Discussions on other uncommon drainage conditions including alluvial fan, mud and debris flow, irrigation-stormwater interaction, and Ice Jam are provided in Chapter 12.

1.6.1 DYNAMIC FLOW ROUTING MODEL

For certain flow routing conditions, it may be desired or necessary to route flows using more comprehensive hydraulic flow routing models (i.e. HEC-RAS Unsteady, FLO-2D, etc.) in place of simplistic flow routing methods utilized by the rainfall-runoff programs. The engineer should coordinate with appropriate local, state, and federal agencies in determining the appropriate dynamic flow routing models for a given waterway.

SELECTION OF ANALYSIS METHODS

METHOD OF ANALYSIS	GAGE RECORD (> 50 YEARS)	GAGE RECORDS (25 TO 50 YEARS)	GAGE RECORD (10 TO 24 YEARS)	GAGE RECORDS (< 10 YEARS)
STATISTICAL ANALYSIS (USING AVAILABLE STREAM GAGE RECORDS)	YES	YES	YES	
COMPARISONS WITH SIMILAR WATERSHEDS (ESTABLISHED FLOWS)		YES	YES	
RAINFALL-RUNOFF MODEL**	*	*	YES	YES
REGIONAL REGRESSION ANALYSIS				YES

* Rainfall-Runoff modeling may be needed in addition to statistical analysis in order to generate various storm hydrographs

** Rainfall-Runoff models should be calibrated

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SECTION 2

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2.1 INTRODUCTION

For basins with reliable stream gage records, the preferred method of estimating various frequency flow rates is the Statistical Analysis Method using recorded stream flow gage data. The reliability of the statistical approach is generally better than rainfall-runoff modeling, provided that the period of gage record is sufficiently long. A minimum of 10 years of reliable stream gage data should be used in the flow frequency analysis.

The statistical analysis method acceptable for use in Colorado is the one that utilizes Log Pearson Type III Distribution as described in "Guidelines for Determining Flood Flow Frequencies," Bulletin 17B, Water Resources Council (March 1982). The following two computer programs may be used to assist in the flow-frequency analysis using Log Pearson Type III Distribution:

- U.S. Army Corps of Engineers, "Flood Flow Frequency Analysis," Computer Program HEC-FFA, Hydrologic Engineering Center
- U.S. Geological Survey, "Annual Flood Frequency Analysis," Computer Program PEAKFQ

2.2 METHODOLOGY

Detailed analysis procedures and guidelines for determining peak flow frequency curves using Log Pearson Type III Distribution are provided in the following publications:

- Water Resources Council, "Guidelines for Determining Flood Flow Frequency," Bulletin 17B, Hydrology Committee, Washington, D.C., March 1982

All flow frequency statistical analysis should be performed in accordance with the procedures and guidelines outlined in the Bulletin 17B. The main purpose of statistical analysis is to use the recorded runoff events for a given period of record as means of extrapolating to a longer period of time. In the statistical approach to determining the size of flood peaks, the assumption involved is that nature over a period of years has defined a flood magnitude-frequency relationship that can be derived by studying actual occurrences. A period of record of a particular basin where the floods have been measured and recorded is considered to be a representative period. For any given year, the probability of a flood of any given frequency happening in that year is the same as the probability of it happening in any other year. Thus, the 100-year flood has a 1 percent chance of being equaled or exceeded in any given year.

The statistical analysis has the greatest applicability to natural streams where the basins will remain in a natural state. Such streams include those with large basins where the urbanization effects on runoff will be negligible, and on small streams where the basin primarily consists of undevelopable land or land comprising greenbelt areas. In urban areas, the use of statistical analysis approach can be limited 1) by almost total lack of adequate runoff records, 2) by the effects of rapid urbanization, and 3) by man-induced changes in the watershed which may include reservoirs, flow diversion structures, canalization of natural streams, etc.

2.2.1 WEIGHTED SKEW COEFFICIENT

The skew coefficient value computed based on a small sample of gage records is not reliable. Therefore, the skew coefficient should be estimated by weighting the computed station skew coefficient with a generalized skew coefficient. The following skew weighting equation is presented in the Bulletin 17B:

$$Gw = MSE_{\overline{G}}(G) + MSE_G(\overline{G})) \;/\; (MSE_{\overline{G}} + MSE_G)$$

Where :

G_W = weighted skew coefficient

G = computed (station) skew

$\overline{G}$ = generalized skew

$MSE_{\overline{G}}$ = mean-squared error of generalized skew

MSE_G = mean-squared error of computed (station) skew

The previously referenced computer programs HEC-FFA and PEAKFQ can be used to compute weighted skew coefficients to meet the guidelines provided in the Bulletin 17B.

2.3 EVALUATION OF GAGE DATA

> Only the annual maximum records should be used in determining the peak flow frequency curves using Log Pearson Type III Distribution.

The reliability and accuracy of the estimated peak flow frequency curves depend greatly on the duration and accuracy of the measured gage data. There are different types of gage records that may be available through various agencies (i.e. USGS, CWCB, UDFCD, etc.) including annual maximum peak flows and stages, flow volumes, mean daily flows, daily peak flows, etc. However, only the annual maximum records should be used in determining the peak flow frequency curves using Log Pearson Type III Distribution.

The collected stream gage data should be carefully evaluated by a qualified professional engineer to determine the reliability and uniformity of the data. The measured data should represent homogeneous watershed hydrologic conditions throughout the record period. The following factors and conditions may result in non-homogeneous gage records:

- Significant urbanization of the watershed
- Construction of reservoirs, dams, and other flood control facilities
- Substantial changes in the flow storage and diversion regulations
- Changes in the shape and capacity of the channel at the gaging station
- Loss of vegetation due to fire over large portions of the watershed

If any of the above conditions existed within the watershed during the gage record period, the data should be adjusted to make the entire record homogeneous, or the statistical analysis method should not be used.

2.4 ADJUSTMENT OF COLLECTED DATA

One of the basic assumptions incorporated into the frequency statistical analysis is that the recorded peak flows are homogeneous. However, in recent years, many watersheds have experienced substantial changes including urbanization, manmade flood control facilities, reservoirs, etc. Therefore, the peak flows during a record period may have resulted from different hydrologic watershed conditions. The recorded data should be evaluated and adjusted to reflect uniform watershed hydrologic conditions.

If the gage data during a record period reflects both natural and altered watershed conditions, then the flow rates based on the altered conditions should be adjusted to reflect the unregulated natural conditions to make the entire population uniform. Professional engineering judgment should be exercised in determining whether the adjustment should be made or not. If the changes in the subject watershed are relatively minor, the adjustment may not be necessary.

General discussions on the common conditions that may require adjustment of the recorded data are provided in this section. For detailed discussions on how to adjust the recorded gage data for various altered watershed conditions, readers are referred to the following publications:

- U.S. Army Corps of Engineers (USCOE), Engineer Manual No. 1110-2-1415, Engineering and Design, Hydrologic Frequency Analysis, March 1993
- Water Resources Council, Guidelines for Determining Flood Flow Frequency, Bulletin 17B, Hydrology Committee, Washington, D.C., March 1982

2.4.1 URBANIZATION

Urbanization of a watershed can substantially alter the resulting peak flows by reducing pervious areas, natural depressions, and the flow concentration time. In most cases, urbanization results in increased flood peak flows for downstream locations. Generally, the increases in peak flows are greater for more frequent flood events compared to less frequent events. Also, urbanization often results in increased base flows. Many streams that used to be dry most of time may experience continuous base flows due to irrigation return flows.

Adjustment of the recorded peak flow data skewed by urbanization is usually made utilizing a calibrated rainfall-runoff model. Detailed discussions on the development of a rainfall-runoff model are provided in Section 5, Chapter 9.

2.4.2 MANMADE FACILITIES

Manmade flood control facilities are usually designed to reduce and/or confine peak flood flows in order to protect human lives and private and public structures. Consequently, these facilities (channels, detention basins, flood control reservoirs, levees, flow diversion structures, etc.) can substantially alter the resulting downstream peak flow rates. The resulting changes in the peak flows for a given watershed may vary considerably depending on the location and size of the facilities and magnitude and distribution (storm-centering) of storm events.

The effects of manmade facilities on flood peak flows can usually be quantified by routing several representative floods through the facilities. Using the routing analysis results, relations between with-facilities peak flows vs. without-facilities peak flows can be determined and plotted on a graph.

2.4.3 LOSS OF VEGETATION DUE TO FIRE

Vegetation loss due to fire over a large portion of a watershed can significantly change the flooding characteristics. Without the benefits provided by vegetation cover including rainfall interception, absorption, and erosion protection, the resulting flood flows can be increased substantially with high concentration of sediment and debris.

Similar to the urbanization adjustment procedures, the recorded peak flows altered by temporary vegetation loss due to fire can be adjusted by using a calibrated rainfall-runoff model.

2.4.4 STAGE-DICHARGE RELATIONS

Many gaging stations are equipped to measure flood stages, and the peak flow rates are estimated using a pre-determined stage-discharge relations of the channel section. Consequently, if the channel section at a gaging station experiences substantial scour or sediment deposition (gradual or rapid), the stage-discharge relations need to be updated to reflect the “changed” channel conditions.

2.5 SNOWMELT AND RAINFALL FLOOD EVENTS

The collected peak flow gage data should be examined to determine the need to segregate the data. Two distinctively different types of flood events may cause stream peak flows in any give year; spring snowmelt and rainstorm. The largest annual peak flows for each flooding conditions should be selected. Peak flow frequency curves should be determined separately using annual snowmelt flood peaks and annual rainfall event flood peaks. The final flow frequency curve should be generated by combining (see Bulletin 17B) the two curves determined using the segregated annual peak flows.

If the gage data cannot be separated into two annual peak flows due to lack of data, then, the mixed population data should be treated as if the data is homogeneous.

Peak flow frequency curves should be determined separately using annual snowmelt flood peaks and annual rainfall event flood peaks. The final flow frequency curve should be generated by combining (see Bulletin 17B) the two curves determined using the segregated annual peak flows.

2.6 HISTORIC FLOOD EVENTS

Historic flood events that occurred prior to the systematic record period can be used to extend the gage record period. The reliability of the historic flood information should be carefully evaluated by a qualified professional engineer. The procedures outlined in the Bulletin 17B should be followed to compute a historically adjusted flow frequency curves.

2.7 CONFIDENCE LIMITS

The flow frequency curve represents "expected' flow rates for various recurrence intervals as computed based on the sample gage peak flow data. The accuracy of the computed flow frequency estimates can be illustrated by defining the confidence limits. In general, there is a 5 percent chance that the true flow value for a given frequency flood event is greater than the value estimated from the 5 percent confidence curve, and a 5 percent chance that the true value is smaller than the value estimated from the 95 percent confidence curve. In other words, there is 90 percent chance that the true flow value can be found between the two curves. By understanding the reliability of the computed flow frequency curves, engineers and planners can make informed decisions on the appropriate uses of the computed flow rates (i.e. additional freeboard requirements, etc.).

The 5 percent and 95 percent confidence limits should be established using the "Non-Central T Distribution". Detailed discussions on the determination of confidence limits can be found in the previously referenced Bulletin 17B and USCOE publications.

2.8 FLOOD HYDROGRAPHS

If flood hydrographs are needed for the floodplain analysis or drainage facilities design, it may be necessary to develop a synthetic rainfall-runoff model. Detailed discussions on the development of a rainfall-runoff model are provided in Section 5, Chapter 9. Synthetic rainfall-runoff models should be calibrated to match the statistical analysis results. The calibrated rainfall-runoff model can then be used to generate hydrographs for various design storm events. The following is a list of some of these cases:

- Various flood frequency hydrographs are required, but the statistical analysis alone cannot generate the necessary hydrographs.
- The subject watershed is undergoing or projected to undergo a substantial amount of new development.
- Comparison of before and after development hydrographs to quantify potential increase in flows due to the proposed developments.

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SECTION 3

REGIONAL REGRESSION ANALYSIS

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REGIONAL REGRESSION ANALYSIS

3.1 INTRODUCTION

The Regional regression analysis method is a simplified procedure of estimating peak flow rates for various frequency storm events for unregulated streams with short or no streamflow records. In Colorado, the regional regression equations presented in this section may be used for the following purposes:

- Computation of peak flow rates for use in delineation of 100-year floodplain boundaries
- As a check to validate the computed flow rates using rainfall-runoff models for detailed floodplain delineation

The State of Colorado has been divided into seven major hydrologic regions as shown on Figure CH9-F301 and the regression equations are assigned for each region. The regression equations are based upon unregulated streamflows and regulated streamflows adjusted to unregulated conditions. The subjected watershed should be carefully evaluated by a qualified professional engineer to determine the applicability of the regression method. If natural or manmade features exist within the watershed (i.e., reservoirs, dams, etc.) that could have substantial impacts on the resulting peak runoff, the regression equations should be used only for validation of an unregulated rainfall/runoff model of the watershed. The use of regression equations should be limited to watersheds with minimal flow regulations and no significant urban developments.

The use of regression equations should be limited to watersheds with minimal flow regulations and no significant urban developments.

3.2 REGIONAL REGRESSION ANALYSIS

Regional regression flood hydrology in Colorado is based upon delineation of seven (7) major hydrologic regions as shown on Figure CH9-F301. The hydrologic regions presented in this section were defined based on the basin boundaries documented in the USGS and CWCB publications described in Section 3.3, Chapter 9.

The western half of the state was divided into four major regions using the Mountain, Rio Grande, Southwest, and Northwest regions from the USGS publication. The regression equations for the four western regions shown on Table CH9-T301 were taken from the USGS study.

The eastern half of the state was divided into three major regions using Arkansas River, South Platte River, and Republican River basin boundaries from the 2004 CWCB publication. The 100-year regression equations presented in the 2004 CWCB study for the subregions within the three major eastern regions are summarized in Table CH9-T301.

3.2.1 LIMITATIONS

The regression equations presented in Table CH9-T301 can be used to estimate peak flow rates for unregulated streams and for validation of rainfall/runoff models based upon unregulated conditions. The following general limitations apply to the use of these regression equations:

- The computed peak flow rates, without validation with a rainfall/runoff model, may only be used for delineation of approximate 100-year floodplain boundaries. Flow rates should be determined using either statistical analyses (Section 2, Chapter 9) or rainfall-runoff models (Section 5, Chapter 9) with validation with regional regression equation results for detailed floodplain studies and design and analyses of drainage facilities.
- The regression equations may be used as a check to validate the computed flow rates using rainfall-runoff models when recorded gage data for the stream is not available.
- The regression equations should only be used for unregulated rural natural streams with minimal flow regulations and no significant urban developments. If natural or manmade features exist within the watershed (i.e., reservoirs, dams, etc.) that could have substantial impacts on the resulting peak runoff, the regression equations can be used to validate rainfall/runoff modeling using unregulated conditions.
- The applicable minimum and maximum drainage basin area, slope, and mean annual precipitation limitations summarized in Table CH9-T301 should be adhered to.

Readers are referred to the USGS and CWCB publications for detailed discussions on the specific limitations for each regions and subregions.

3.3.2 PROCEDURE

The general guidelines for using the regression equations for a given ungaged natural stream are provided below.

- Using Figure CH9-F301, identify the major hydrologic region and subregion, if applicable, for the given stream design point.
- Evaluate the study watershed to determine the applicability of the regression analysis method.
- From Table CH9-T301, select the appropriate regression equation for the identified region or subregion.
- Estimate the contributing total drainage area at the hydrologic point of interest.
- If applicable, estimate the mean annual precipitation (P, in inches) and mean drainage basin slope (S, in foot per foot) for the contributing watershed.
- Calculate the peak flow values for the hydrologic point of interest by applying the appropriate regression equation.

If the contributing drainage basin for a given stream design point lies in two regions or subregions, the flow rates should be computed using regression equations for both regions/subregions, and the weighted discharge should be computed using the following equation (USGS, 2000):

$$Q_w = (Q1 * Area\ 1) + (Q2 * Area\ 2) / Total\ Area \qquad \text{(Eq. CH9-301)}$$

Where

Q_w	= Weighted discharge (cfs)
Q1	= Region 1 computed discharge (cfs) – using total area
Q2	= Region 2 computed discharge (cfs) – using total area
Area1	= Contributing drainage area in region 1 (sq. mi.)
Area2	= Contributing drainage area in region 2 (sq. mi.)
Total Area	= Total drainage area in both regions (sq. mi.)

3.3 BASIS OF REGIONAL REGRESSION EQUATIONS FOR COLORADO

The information provided in the following two regression analysis studies for Colorado should be used to determine the hydrologic regional boundaries and regression equations presented in this section:

For Western Colorado:

- U.S. Geological Survey (USGS), Water-Resources Investigations Report 99-4190, Analysis of the Magnitude and Frequency of Floods in Colorado, 2000

For Eastern Colorado:

- Colorado Water Conservation Board (CWCB), Guidelines for Determining 100-Year Flood Flows for Approximate Floodplains in Colorado, Version 5.0, June 2004

3.3.1 USGS REGRESSION EQUATIONS

The USGS study divided the state into five distinct hydrologic regions (Mountain, Rio Grande, Southwest, Northwest, and the Plains), and presented separate regression equations for each of the five regions. The Ordinary Least-Squares (OLS) and Generalized Least-Square (GLS) regression analyses were performed utilizing the recorded streamflow gage data through water year 1993 to develop the regression equations for various design storms ranging from 2- to 500-year recurrence intervals. The regression equations and their limitations are presented in the USGS publication.

The USGS regression equations for Mountain, Rio Grande, Southwest, and Northwest regions have standard error of estimates ranging from 41 to 85 percent. The accuracy of the regression equations is generally represented by the percent standard error of estimates. These equations are deemed to have an acceptable percent of standard error. However, the regression equations for the Plains region have very high standard error of estimates ranging from 204 to 306 percent due to the lack of measured gage data for the region. Thus, for eastern Colorado, the CWCB equations are deemed to produce more reasonable flow values than the USGS regression equations.

3.3.2 CWCB REGRESSION EQUATIONS

The CWCB developed 100-year recurrence interval regression equations using 100-year flow values from published FEMA and other detailed floodplain studies. In addition to the flow values computed based on flow-frequency analyses of streamflow gage data, the published flow values based on various rainfall-runoff models were utilized to develop the regression equations. The regression equations and their limitations are documented in the CWCB publication.

The state was divided into nine (9) major hydrologic basins (Arkansas River, South Platte River, Republican River, Colorado River, Green River, Dolores River, San Juan River, Rio Grande, and North Platte River). The major basins were further divided into subregions and separate regression equations were developed for each of the subregions. The standard error of estimates for the 100-year regression equations for the subregions within Arkansas River, South Platte River, and Republican River basins ranged from 6 to 50 percent. These equations are deemed to have an acceptable standard error of estimate. For western Colorado, the USGS regression equations are deemed to produce more reasonable flow values than the CWCB equations.

3.4 SITE SPECIFIC REGIONAL ANALYSIS

Regional regression analyses may also be performed on a case-by-case basis using selected stream gages, in the vicinity of a hydrologic point of interest, that re deemed to be appropriate for a more detailed or site specific purpose.

3.5 EXAMPLE APPLICATION

Problem:

Determine the weighted 100-year flow rate for the following watershed:

Total drainage area = 120 sq. mi.
Drainage area in Southwest region = 40 sq. mi.
Drainage area in Northwest region = 80 sq. mi.
Mean annual precipitation = 20 inches

Solution:

Southwest region regression equation, $Q_{100} = 118.4\ (A)^{0.715}$ (Table CH9-T301)

$$Q_{100} = 118.4\ (120)^{0.715} = 3630 \text{ cfs}$$

Northwest region regression equation, $Q_{100} = 104.7\ (A)^{0.624}$ (Table CH9-T301)

$$Q_{100} = 104.7\ (120)^{0.624} = 2077 \text{ cfs}$$

Weighted 100-year discharge, Equation CH9-301

$$Q_w = (Q_{southwest} * Area_{southwest}) + (Q_{northwest} * Area_{northwest}) / Total\ Area$$

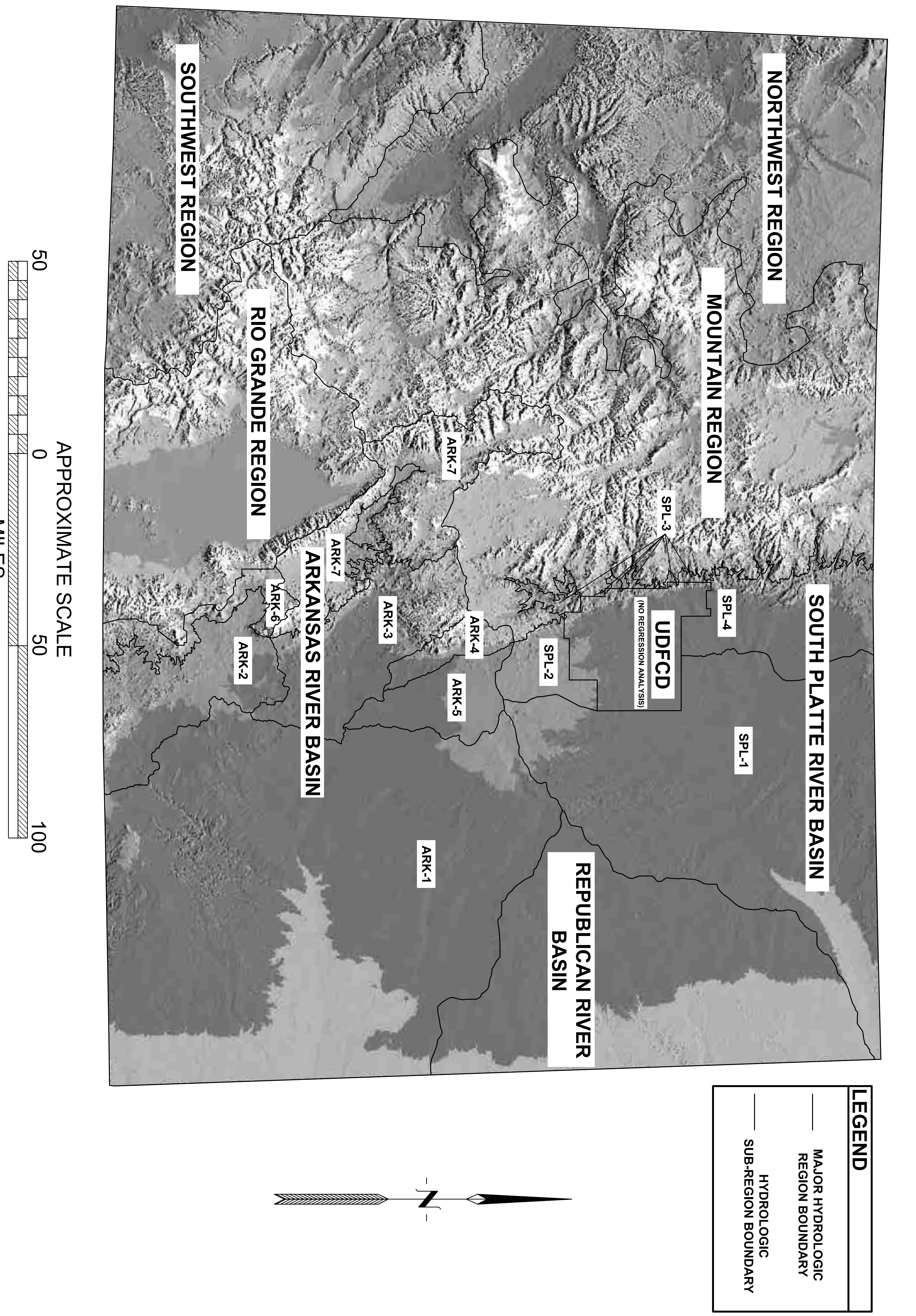
$$Q_w = (3630 * 40) + (2077 * 80) / 120 = 2595 \text{ cfs}$$

REGIONAL REGRESSION EQUATIONS

Region	Sub-region Name	100 Year Recurrance Interval Regression Equation*	Limitations		Std. Error of Estimate
			Min. Area	Max. Area	
Arkansas	ARK - 1	$Q = 1572.8(A)^{.547}$	25	1125	25%
Arkansas	ARK - 2	$Q = 3959.2(A)^{.366}$	12	280	16%
Arkansas	ARK - 3	$Q = 1089.3(A)^{.653}$	1	930	15%
Arkansas	ARK - 4	$Q = 1408.2(A)^{.654}$	1	26	25%
Arkansas	ARK - 5	$Q = 1343.4(A)^{.578}$	4	75	30%
Arkansas	ARK - 6	See Text Section 3.2	N/A	N/A	N/A
Arkansas	ARK - 7	$Q = 46.0(A)^{.717}$	4	330	6%
S. Platte	SPL - 1	$Q = 707.9(A)^{.654}$	2	1090	34%
S. Platte	SPL - 2	$Q = 1005.5(A)^{.638}$	1	170	18%
S. Platte	SPL - 3	$Q = 762.4(A)^{.546}$	1	175	23%
S. Platte	SPL - 4	$Q = 800.8(A)^{.478}$	1	445	48%
S. Platte	SPL - 5	$Q = 39.4(A)^{.776}$	2	480	29%
Republican	REP - 1	$Q = 289.1(A)^{.667}$	1	1300	36%
Mountain Region	-	$Q = 39.5(A)^{.706}(S+1.0)^{1.577}$	5.5	945	42%
Rio Grande Region	-	$Q = 1.19(A)^{.846}(P)^{1.074}$	10.5	595	50%
Southwest Region	-	$Q = 118.4(A)^{.715}$	8.2	720	76%
Northwest Region	-	$Q = 104.7(A)^{.624}$	5	988	75%

* A = Area in Acres
P = Mean Annual Precipitation in Inches (Min. = 7, Max. = 49)
S = Slope in Foot/Foot (Min. = 0.126, Max. = 0.554)

See Figure CH9-F301 for Region Boundaries



VERSION: JANUARY 2006	REFERENCE: COLORADO WATER CONSERVATION BOARD	**FIGURE CH9-F301** COLORADO HYDROLOGIC REGIONS AND SUBREGIONS

CHAPTER 9

HYDROLOGIC ANALYSIS

SECTION 4

RAINFALL

CHAPTER 9
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4.1 INTRODUCTION

Presented in this section are the rainfall depths and distributions for various design storm events to be utilized with the four selected deterministic runoff modeling methods. The four deterministic methods include the Rational Method, the NRCS TR-55 Method, the NRCS Unit Hydrograph Method, and the CUHP/UDSWM Method. The criteria to be used in selection of the appropriate rainfall-runoff model for a given drainageway are provided in this section. Detailed discussions on the runoff modeling methods and guidelines are provided in Section 5, Chapter 9.

The information presented in this section is the current information available at the time of preparation of this Manual and should be updated as better techniques and new rainfall data become available in the future.

4.2 RAINFALL DATA

The rainfall data published by National Oceanic and Atmospheric Administration (NOAA) in their "Precipitation-Frequency Atlas of the Western United States, Volume III – Colorado, 1973" should be used to perform necessary rainfall-runoff calculations within the State of Colorado, unless site-specific rainfall studies have been performed and adopted by the local government agency having jurisdiction over the study area.

The NOAA Atlas 6-hour and 24-hour precipitation frequency maps for various storm events for the State of Colorado are included as Figures CH9-F401 through CH9-F412. The 6-hr and 24-hr point precipitation values can be estimated directly from Figures CH9-F401 through CH9-F412, and if needed, these point rainfall values can then be used to develop 5-minute, 10-minute, 15-minute, 30-minute, 1-hour, 2-hour, 3-hour, and 12-hour rainfall depths using the procedures outlined in Section 4.4 of this chapter.

4.3 METHOD SELECTION

There are many different rainfall-runoff deterministic models available. However, not all of the methods can be effectively utilized in Colorado. Some methods are not applicable for the hydrologic conditions that exist in Colorado, and other methods cannot be utilized easily or accurately due to the lack of data. Also, the computed flow estimates may vary considerably depending on the methods utilized for a given watershed. Therefore, it is necessary to define minimum standards for the analysis in order to promote accuracy and consistency in the computed flow rates.

The following four deterministic rainfall-runoff modeling methods have been selected for use in Colorado:

- Rational Method
- NRCS TR-55 Method

- NRCS Unit Hydrograph Method
- CUHP and UDSWM Method

In addition, other models or methods approved by local, state, and federal study partners may be used on a case-by-case basis.

Detailed discussions on the above runoff modeling methods are provided in Section 5, Chapter 9. The recommended rainfall depths and distributions to be used with the listed runoff methods are presented in this section. Due to the incorporated assumptions and limitations of the listed modeling methods, not all of the selected deterministic methods can be used for all hydrologic conditions. The methodology used to generate the rainfall-runoff data should be selected based on the size and location of the drainage basin to be studied and the intended use of the computed flow rates and hydrographs.

The Rational Method for determining runoff is widely accepted as providing a sufficient level of detail for generating runoff from relatively small basins and can be used for drainage basins with a total contributing area of less than 160 acres.

The Rational Method for determining runoff is widely accepted as providing a sufficient level of detail for generating runoff from relatively small basins and can be used for drainage basins with a total contributing area of less than 160 acres. The Rational Method utilizes rainfall data in the form of time-intensity-frequency curves. Since the assumptions used in the Rational Method become less valid for larger areas, larger basins require a more rigorous analysis to generate runoff data.

For drainage basins with an area greater than 90 acres, NRCS TR55 Method, NRCS Unit Hydrograph Method, or CUHP/UDSWM Method should be used depending on the location and hydrologic complexity of the drainage basin to estimate the runoff data. For drainage basins with an area between 90 acres and 160 acres, any of the four selected methods maybe utilized.

The NRCS TR55 Method was first developed and documented in 1975 to provide a simplified procedure for estimating runoff and peak discharges from small urban and urbanizing watersheds. Two peak runoff determination techniques are available: Graphical Method and Tabular Method. The Graphical Peak Discharge method estimates only the peak runoff. The Tabular Hydrograph method can produce a runoff hydrograph. A synthetic 24-hour regional design rainfall distribution is used in the TR-55 runoff computations.

The NRCS TR-55 Method can be effectively used for simple watershed runoff modeling. However, the NCRS Unit Hydrograph Method utilizing the US Army Corps of Engineer's HEC-1 or HEC-HMS computer program should be utilized for rainfall-runoff modeling of watersheds that involve multiple sub-basins and routing elements and reaches.

NRCS Unit Hydrograph Method utilizing the US Army Corps of Engineer's HEC-1 or HEC-HMS computer program should be utilized for rainfall-runoff modeling of watersheds that involve multiple sub-basins and routing elements and reaches.

To promote consistency in the computed results and to simplify the analysis, a 24-hour balanced storm distribution should be used with the NRCS Unit Hydrograph Method.

The NRCS Unit Hydrograph Method can be used with a wide range rainfall distribution types and durations (6-hour, 24-hour, etc.). However, to promote consistency in the computed results and to simplify the analysis, a 24-hour balanced storm distribution should be used with this method.

The CUHP (Colorado Urban Hydrograph Procedure) method has been used exclusively for the Urban Drainage and Flood Control District (UDFCD) jurisdictional area. The CUHP method was developed and calibrated to effectively model short duration convective storms within the Denver Metro area. Therefore, the CUHP method should only be used for urban areas that have similar hydrologic characteristic as the Denver Metro area. The CUHP model can be used to generate sub-basin hydrographs and the UDSWM (Urban Drainage Storm Water Management) computer program can be used to route and combine hydrographs. For detailed discussions on the CUHP and UDSWM methods and programs, please refer to the latest UDFCD Drainage Criteria Manual.

The CUHP method should only be used for urban areas that have similar hydrologic characteristic as the Denver Metro area.

4.4 RAINFALL DEPTHS

The 6-hr and 24-hr point precipitation values for 2-, 5-, 10-, 25-, 50-, and 100-year storm events can be estimated directly from Figures CH9-F401 through CH9-F412. The point precipitation values for each storm duration (6- and 24-hr) obtained from the isopluvial maps should be plotted on the return-period diagram (Figure CH9-F413), and a straight line of best fit should be drawn. If any rainfall value deviates substantially from the best-fit line, the value read from the line should replace the original point precipitation value from the map.

Once the 6- and 24-hr rainfall values have been obtained and adjusted (if necessary), the rainfall depths for other durations can be estimated using the following procedures from the NOAA Atlas II, Volume III, 1973. The State of Colorado has been divided into four (4) geographic regions by NOAA and they are shown on Figure CH9-F414. Before applying the empirical methods outlined below, it is necessary to determine the region and apply appropriate equations for the drainage basin. If the drainage basin is located within few miles of a regional boundary, computations should be made using equations for both regions and the average rainfall values should be used for the rainfall-runoff analysis.

The 1-hour frequency values for 2- and 100-year storm events can be estimated utilizing the appropriate regional equations from Table CH9-T402. Once computed, the 2-year and 100-year, 1-hour values can be plotted on Figure CH9-F413, and a straight line between the two values can be drawn. Then, the 1-hour values for return periods between 2- and 100-year events can be obtained from the line.

Rainfall depths for the 2-hour and 3-hour events can be estimated using the following formulas (NOAA Atlas 2, 1973).

Region 1 $D_{X,2}= 0.342^{*}D_{X,6} + 0.658^{*}D_{X,1}$ (Eq. CH9-400)
Region 2 $D_{X,2}= 0.341^{*}D_{X,6} + 0.659^{*}D_{X,1}$ (Eq. CH9-401)
Region 3 &4 $D_{X,2}= 0.250^{*}D_{X,6} + 0.750^{*}D_{X,1}$ (Eq. CH9-402)

Where $D_{X,2}$= "X"-year, 2-hour rainfall depth (Inches)
$D_{X,1}$= "X"-year, 1-hour rainfall depth (Inches)
$D_{X,6}$= "X"-year, 6-hour rainfall depth (Inches)

Region 1 $D_{X,3}= 0.597^{*}D_{X,6} + 0.403^{*}D_{X,1}$ (Eq. CH9-403)
Region 2 $D_{X,3}= 0.569^{*}D_{X,6} + 0.431^{*}D_{X,1}$ (Eq. CH9-404)
Region 3&4 $D_{X,3}= 0.467^{*}D_{X,6} + 0.533^{*}D_{X,1}$ (Eq. CH9-405)

Where $D_{X,3}$= "X"-year, 3-hour rainfall depth (Inches)
$D_{X,1}$= "X"-year, 1-hour rainfall depth (Inches)
$D_{X,6}$= "X"-year, 6-hour rainfall depth (Inches)

Based on Figure 17 in the NOAA Atlas 2, the 12-hour duration rainfall depth for the desired recurrence frequency is essentially the average of the 6-hour and 24-hour storm events (NOAA, 1973).

$D_{X,12}= (D_{X,6} + D_{X,24})/2$ (Eq. CH9-406)

Where $D_{X,12}$= "X"-year, 12-hour rainfall depth (Inches)
$D_{X,6}$= "X"-year, 6-hour rainfall depth (Inches)
$D_{X,24}$= "X"-year, 24-hour rainfall depth (Inches)

Rainfall depths for durations less than 1-hour can be estimated using the adjustment ratios supplied in Table CH9-T401 and the estimated "X"-year, 1-hour rainfall depth (NOAA, 1973). These adjustment ratios were originally published in the US Weather Bureau Technical Paper No. 40 in 1961, and later evaluated and adopted by NOAA.

$D_{X,Y}= D_{X,1}{}^{*}\ RATIO_{X,Y}$ (Eq. CH9-407)

Where $D_{X,Y}$= "X"-year, Y-minute rainfall depth (Inches)
$D_{X,1}$ = "X"-year, 1-hour rainfall depth (Inches)
$RATIO_{X,Y}$ = Ratio to convert "X"-year, 1-hour rainfall depth to the "X"-year, Y-minute depth

4.5 RAINFALL

Time-intensity-frequency curves can be used with the Rational Method to produce rainfall-runoff data for drainage basins of less than 160 acres. The time-intensity-frequency curves can be developed for a desired location utilizing the rainfall depths of durations less than 1-hour for storm events between 2- and 100-year.

Utilizing the estimated rainfall depths of the 5-, 10-, 15-, 30-, and 60-minute durations for a given recurrence frequency, rainfall intensities can be estimated by dividing the rainfall depth by the duration of the storm.

$I_{X,Y}= D_{X,Y}/Duration_{Y}$ (Eq. CH9-408)

Where $I_{X,Y}$= "X"-year, Y-minute rainfall intensity (Inches/Hour)
$D_{X,Y}$ = "X"-year, Y-minute rainfall depth (Inches)
$Duration_{Y}$ = Duration Y minute divided 60 (Hour)

A time-intensity curve for a given recurrence frequency can be developed by plotting the intensity values versus their corresponding storm duration values. An example showing the development of a time-intensity-frequency curve is given in Section 4.9.2 of this chapter.

4.6 RAINFALL DISTRIBUTION FOR NRCS TR-55 METHOD

The 24-hr point precipitation values of 2-, 5-, 10-, 25-, 50-, and 100-year storm events for a study basin can be estimated directly from Figures CH9-F407 through CH9-F412. The estimated 24-hour rainfall depths should be used in conjunction with the NRCS Type II rainfall distribution to determine various frequency flow rates utilizing either the Graphical Method or the Tabular Method. Detailed discussions on the application of the TR-55 Graphical and Tabular methods are provided in Section 5, Chapter 9. Readers are referred to the Natural Resources Conservation Service (NRCS), Technical Release 55, dated June 1986 for detailed discussions on the NRCS Type II rainfall distribution.

The estimated 24-hour rainfall depths should be used in conjunction with the NRCS Type II rainfall distribution to determine various frequency flow rates utilizing either the Graphical Method or the Tabular Method.

4.7 RAINFALL DISTRIBUTION FOR NRCS UNIT HYDROGRAPH METHOD

The rainfall data used with the NRCS Unit Hydrograph Method should be a centrally distributed storm event with rainfall depths at time intervals of 5-minutes, 15-minutes, 60-minutes, 2-hours, 3-hours, 6-hours, 12-hours, and 24-hours for the desired recurrence frequency. The NOAA procedures to determine these rainfall depth values were discussed previously in Section 4.4 of this chapter. The recommended 24–hour rainfall distribution is centered around the midpoint of the design storm (time = 12 hours), and is commonly known as “Balanced Storm” distribution.

The computed rainfall values can be entered into either HEC-1 or HEC-HMS programs. When using the PH record in HEC-1 to input the rainfall depths, a value of 0.001 should be input into Field 2 to prevent the program from using an internal point rainfall reduction adjustment.

4.7.1 DEPTH-AREA REDUCTION FACTORS (DARF)

The NOAA Atlas 2 precipitation depths are related to rainfall frequency at an isolated point. Storms, however, can cause rainfall to occur over extensive areas simultaneously, with more intense rainfall typically occurring near the center of the storm. Standard precipitation analysis methods require adjusting point precipitation depths downward in order to estimate the average depth of rainfall over the entire storm area. This is normally performed using depth-area reduction factors (DARF) relating to a point precipitation reduction factor to storm area and duration.

Figure CH9-F415 provides the depth-area reduction curve for the 24-hour storm event (NOAA, 1973). The application of DARF for large watersheds is complicated by the necessity to determine the "storm centering" which produces the greatest peak flow and/or volume at the selected design point. In order to obtain consistent results and to simplify the application of DARF, the flow value at a given concentration point should be determined using the depth-area reduction value for the total watershed area tributary to the subject point of interest. As runoff flows through a subject watershed, the contributing drainage area increases and the associated depth-area reduction factor will vary. To account for this, a range of depth-area reduction factors may need to be estimated for large watersheds that have several sub-basin design points. For example, if a total watershed area is 15 square miles, three depth-area reduction values may be used to estimate runoff for a design point at 5 square miles, one at 10 square miles, and one at 15 square miles. The respective depth-area reduction values would be 0.992, 0.985, and 0.978

In order to obtain consistent results and to simplify the application of DARF, the flow value at a given concentration point should be determined using the depth-area reduction value for the total watershed area tributary to the subject point of interest.

4.8 RAINFALL DISTRIBUTION FOR CUHP METHOD

The CUHP (Colorado Urban Hydrograph Procedure) computer model has been used within the Urban Drainage and Flood Control District (UDFCD) jurisdictional area to estimate urban sub-basin hydrographs. The CUHP method was developed and calibrated to simulate short duration convective storms in the Denver Metro area and other similar urban drainage environments. Convective storms are commonly known to be responsible for high peak flows and flooding problems for many small drainage basins.

4.8.1 CUHP STORM DISTRIBUTION

The rainfall intensity and distribution analysis performed by UDFCD using 73-years of rainfall record data at the Denver rain gage revealed that the majority of the past intense rainstorms produced their largest rainfall within the first hour of the storm. The analysis further discovered that out of the 73 storm events analyzed, 68 events produced the most intense rainfall beginning and ending within the first hour of the storm and 52 events produced the most intense rainfall beginning and ending within the first half hour of the storm. The UDFCD analysis concluded that these "leading intensity" convective storms were the main cause of most of the flooding problems in the Denver Metro Region (UDFCD, 2001).

The rainfall distributions recommended to be used with CUHP were developed to reflect the "leading intensity" characteristics of the previously recorded convection storms in the Denver Region, and they vary from 2- to 6-hours depending on the size of the drainage basin. The rainfall distributions for 2-, 3- and 6-hour storm durations can be developed using the following procedures from the UDFCD Drainage Criteria Manual, 2001.

For drainage basins less than 10 square miles but greater than 90 acres, two-hour storm distribution rainfall values without area adjustments of the values should be used with CUHP. For drainage basins between ten and twenty square miles, three-hour storm distribution rainfall values with the area-adjustment should be used. For basins equal to and larger than 20 square miles, six-hour storm distribution values with the area-adjustment should be used. Area adjustments of the rainfall values for drainage basins equal to or greater than 10 square miles are necessary to determine the average depth of precipitation over the entire drainage basin being analyzed.

The 1-, 3-, and 6-hour point rainfall depths estimated using the NOAA Atlas 2 procedure described previously in Section 4.4 of this chapter can be used to develop storm distributions for a given recurrence frequency. The estimated NOAA point precipitation values can be distributed to develop 2-, 3- or 6-hour temporal distribution values using a 5-minute time increment following the distribution procedures from the UDFCD Drainage Criteria Manual, 2001.

The 2-hour temporal distribution for a given recurrence frequency can be developed by multiplying the NOAA 1-hour rainfall depth by the incremental distribution percentages (0 to 120 minutes) given in Table CH9-T403. The 2-hour design storm distribution can be used without further modifications with CUHP for drainage basins less than 10 square miles.

The 3-hour storm distribution can be developed by adding incremental precipitation values for the period between 125 minutes and 180 minutes to the 2-hour distribution discussed above. The incremental precipitation values for the period between 125 minutes and 180 minutes can be determined by evenly distributing the difference between the NOAA 3-hour rainfall depth and the 2-hour total precipitation developed using Table CH9-T403. In a similar approach, the 6-hour distribution can be developed by evenly distributing the difference between the NOAA 3-hour and 6-hour rainfall depths over the period of 185 minutes to 360 minutes. The first three hours of the 6-hour distribution is same as the three-hour distribution discussed above. More detailed discussions on the CUHP storm distribution generations can be found in the UDFCD Drainage Criteria Manual, 2001.

4.8.2 DEPTH-AREA ADJUSTMENT

Rainfall depth-area adjustment is necessary to determine the average depth of precipitation over the entire drainage basin being analyzed.

The NOAA precipitation depths are related to rainfall frequency at an isolated point. Storms, however, can cause rainfall to occur over extensive areas simultaneously, with more intense rainfall typically occurring near the center of the storm. Rainfall depth-area adjustment is necessary to determine the average depth of precipitation over the entire drainage basin being analyzed. This is normally performed using depth-area reduction curves relating point precipitation reduction factor to drainage basin area and storm duration.

In order to assist engineers with the depth-area adjustment application procedures, UDFCD developed an adjustment factor table for drainage basins between 10 and 75 square miles. The UDFCD table is included in this section as Table CH9-T404. The 3- and 6- hour storm distribution values can be adjusted by multiplying each incremental rainfall depth by the appropriate adjustment factor from Table CH9-T404 for a given time increment and the size of the drainage basin. An example showing the development of a CUHP design storm distribution is given in Section 4.9.3 of this chapter.

4.9 EXAMPLES

The following examples are provided to guide the readers through the rainfall distribution development procedures outlined in this section by analyzing a hypothetical drainage basin.

4.9.1 EXAMPLE 1: RAINFALL DISTRIBUTION FOR NRCS UNIT HYDROGRAPH METHOD

Problem:

Develop a 100-year, 24-hour balanced storm distribution for Basin A.

Solution:

Step 1: Determine the average 100-year, 6-hour rainfall depth and the average 100-year, 24-hour rainfall depth for Basin A from Figures CH9-F401 and CH9-F412, respectively (Basins that have highly variable rainfall depths for a given frequency and duration may need to be subdivided into areas of common rainfall depth. A weighted average of the rainfall depth can then be calculated using the areas and rainfall depths of the sub-basins).

$D_{100,6}$ = 3.6 inches (assumed for this example)
$D_{100,24}$ = 5.0 inches (assumed for this example)

The average 6-hr and 24-hr rainfall depths in Basin A for 2-, 5-, 10-, 25-, and 50-year storm events should also be estimated from Figures CH9-F401 through CH9-F412. The average rainfall values of the six recurrence frequencies for each storm duration (6- and 24-hr) should be plotted on the return-period diagram (Figure CH9-F413), and a straight line of best fit should be drawn. If any recurrence frequency rainfall value deviates substantially from the best-fit line, the value read from the line should replace the original rainfall value from the map. For the purpose of this example, it is assumed no adjustments of the rainfall values are necessary.

Step 2: Calculate the average 100-year, 1-hour rainfall depth, Y_{100}.

From Figure CH9-F414, determine the geographic region of Basin A. For the purpose of this example, Basin A is located in Region 1 and the average basin elevation is 6,000 ft.

From Table CH9-T402,

Region 1, $Y_{100} = 1.897 + 0.439[(3.6)(3.6/5.0)] – 0.008(60) = 2.56$ inches

Step 3: Determine the 100-year, 5 minute and the 100-year, 15-minute rainfall values. The conversion ratios provided in Table CH9-T401 are multiplied by the 1-hour rainfall depth.

RATIO5 = 0.29
RATIO15= 0.57

$D_{100,5}$ = RATIO5*$D_{100,1}$ = 0.29*2.56 = 0.74 inches
$D_{100,15}$ = RATIO15*$D_{100,1}$ = 0.57*2.56 = 1.46 inches

Step 4: Compute the 100-year, 2-hour, 100-year, 3-hour, and the 100-year, 12-hour rainfall values using Equations CH9-400 and CH9-403, respectively.

$D_{100,2}$ = 0.342*$D_{100,6}$ + 0.658*$D_{100,1}$
= 0.342*3.6+0.658*2.56 = 2.92 inches
$D_{100,3}$ = 0.597*$D_{100,6}$ + 0.403*$D_{100,1}$
= 0.597*3.6+0.403*2.56 = 3.18 inches
$D_{100,12}$ = 0.5 *$D_{100,6}$ + 0.5 *$D_{100,24}$
= 0.5*3.6+0.5*5.0 = 4.3 inches

Step 5: Estimate the depth-area reduction factor from Figure CH9-F415.

Assuming the drainage area for Basin A is 2,140 acres, or 3.34 square miles, the area-reduction factor is approximately 0.995.

The computed rainfall depths for durations of 5 minutes, 15 minutes, 1 hour, 2 hours, 3 hours, 6 hours, 12 hours, and 24 hours can be entered into either HEC-1 or HEC-HMS to define the 24-hour balanced storm distribution.

4.9.2 EXAMPLE 2: TIME-INTENSITY-FREQUENCY CURVE FOR RATIONAL METHOD

Problem:

Develop a 100-year time-intensity-frequency curve for Rocky Subdivision located in Basin A.

Solution:

Step 1: Calculate the 100-year, 1-hour rainfall depth for Rocky Subdivision as explained in the Example 1.
$D_{100,1}$ = 2.56 inches (Assumed for example purposes only)

Step 2: Generate 100-year rainfall depths for storm durations of 5 minutes, 10 minutes, 15 minutes and 30 minutes.

$D_{100,5}$ = RATIO5*$D_{100,1}$ = 0.29*2.56 = 0.74 inches
$D_{100,10}$= RATIO10*$D_{100,1}$= 0.45*2.56= 1.15 inches
$D_{100,15}$= RATIO15*$D_{100,1}$= 0.57*2.56= 1.46 inches
$D_{100,30}$= RATIO30*$D_{100,1}$= 0.79*2.56= 2.02 inches

Step 3: Calculate 100-year rainfall intensity values for storm durations of 5 minutes, 10 minutes, 15 minutes, 30 minutes, and 60 minutes.

$I_{100,5}$ = $D_{100,5}$/Duration = 0.74/(5/60) = 8.88 inches/hour
$I_{100,10}$= $D_{100,10}$/Duration = 1.15/(10/60)= 6.90 inches/hour

$I_{100,15}$= $D_{100,15}$/Duration = 1.46/(15/60)= 5.84 inches/hour
$I_{100,30}$= $D_{100,30}$/Duration = 2.02/(30/60)= 4.04 inches/hour
$I_{100,60}$= $D_{100,60}$/Duration = 2.56/(60/60)= 2.56 inches/hour

Step 4: Plot the time-intensity-frequency curve for the 100-year storm for Rocky Subdivision. (See Figure CH9-F416).

4.9.3 EXAMPLE 3: CUHP STORM DISTRIBUTION

Problem:

Develop a 100-year rainfall distribution to be used with CUHP model for a 17 square mile drainage basin.

100-year, 1-hr rainfall depth = 2.20 inches
100-year, 3-hr rainfall depth = 2.75 inches

Solution:

Step 1: Since the drainage basin is less than 20 square miles but greater than 10 square miles, a three-hour storm distribution should be used with CUHP. First, a two-hour temporal distribution should be developed by multiplying the 100-year, 1-hour rainfall value of 2.2 inches by the incremental distribution percentages from Table CH9-T403.

Step 2: Calculate incremental rainfall depths for the period between 125 and 180 minutes by evenly distributing the rainfall depth difference between the 100-year, 3-hour rainfall depth of 2.75 inches and the 2-hour total precipitation.

Step 3: Apply the depth-area reduction factors from Table CH9-T404 to the calculated incremental rainfall depths for the entire storm duration.

Results of the above three steps are shown in Table CH9-T405.

NOAA ATLAS 2 MAP REFERENCE
(OVERSIZE DRAWINGS)

COLORADO

FLOODPLAIN AND STORMWATER CRITERIA MANUAL

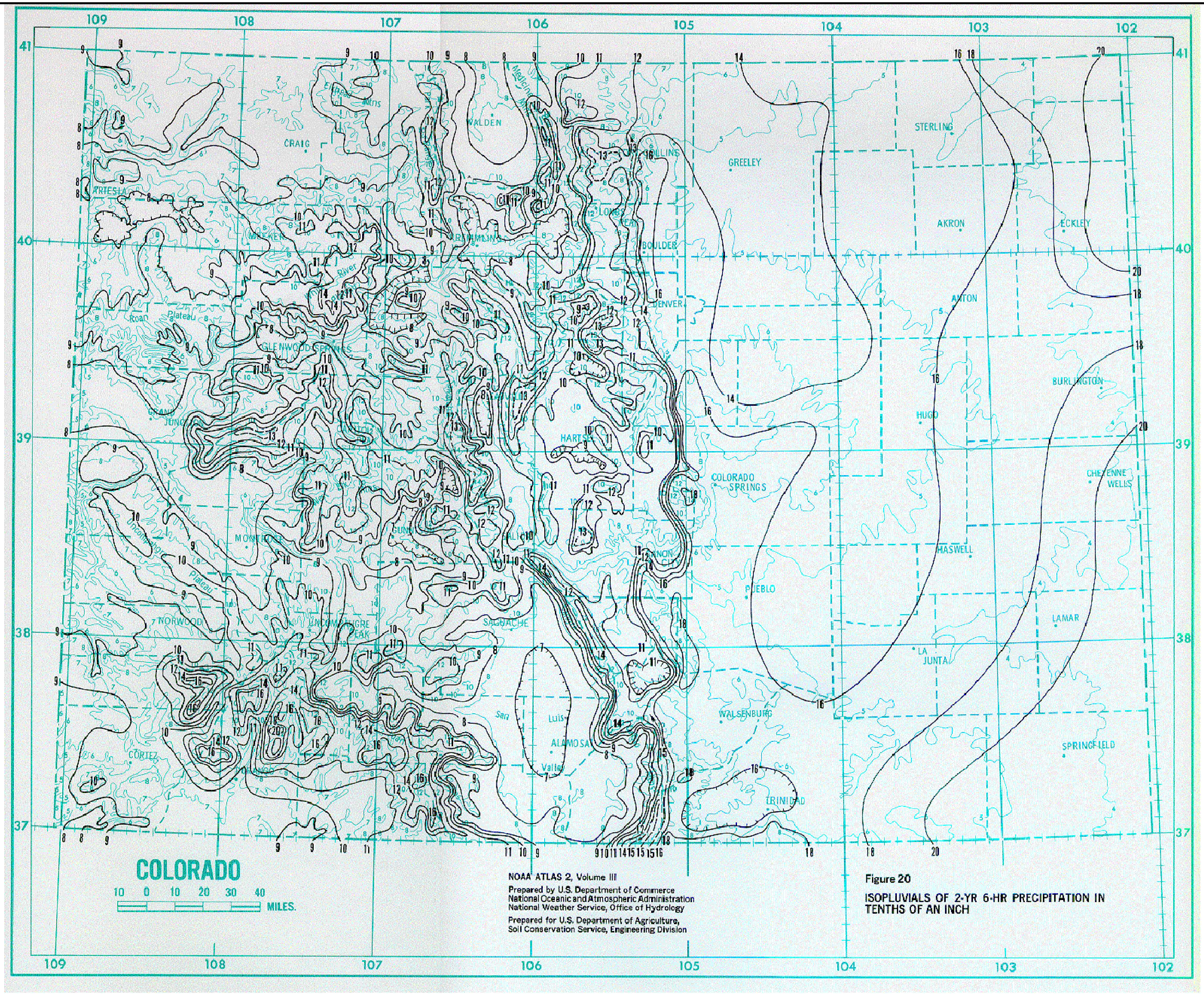

FIGURE CH9-F401

2-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

FIGURE CH9-F402
5-YEAR, 6-HOUR PRECIPITATION MAP

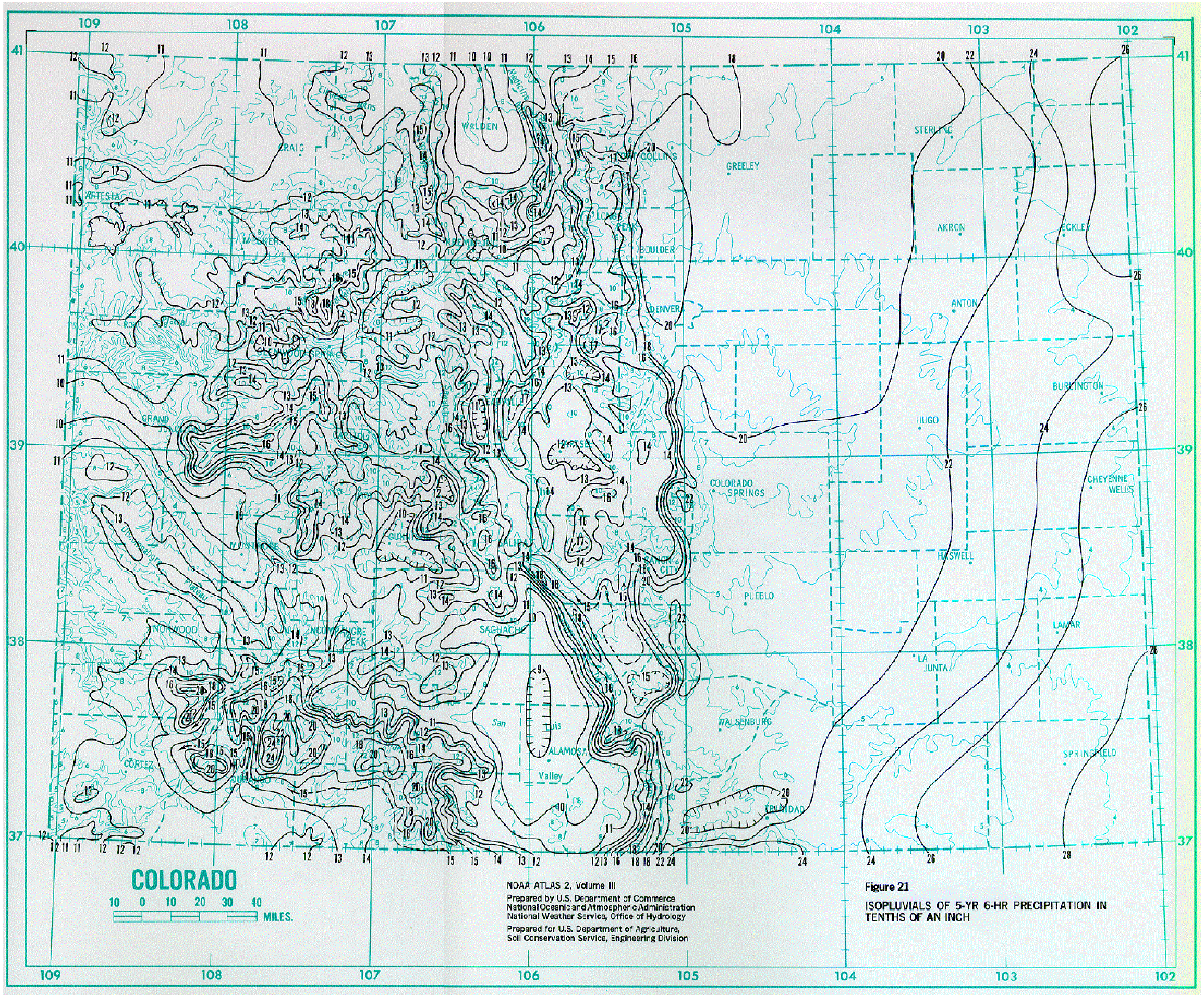

Figure 21
ISOPLUVIALS OF 5-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

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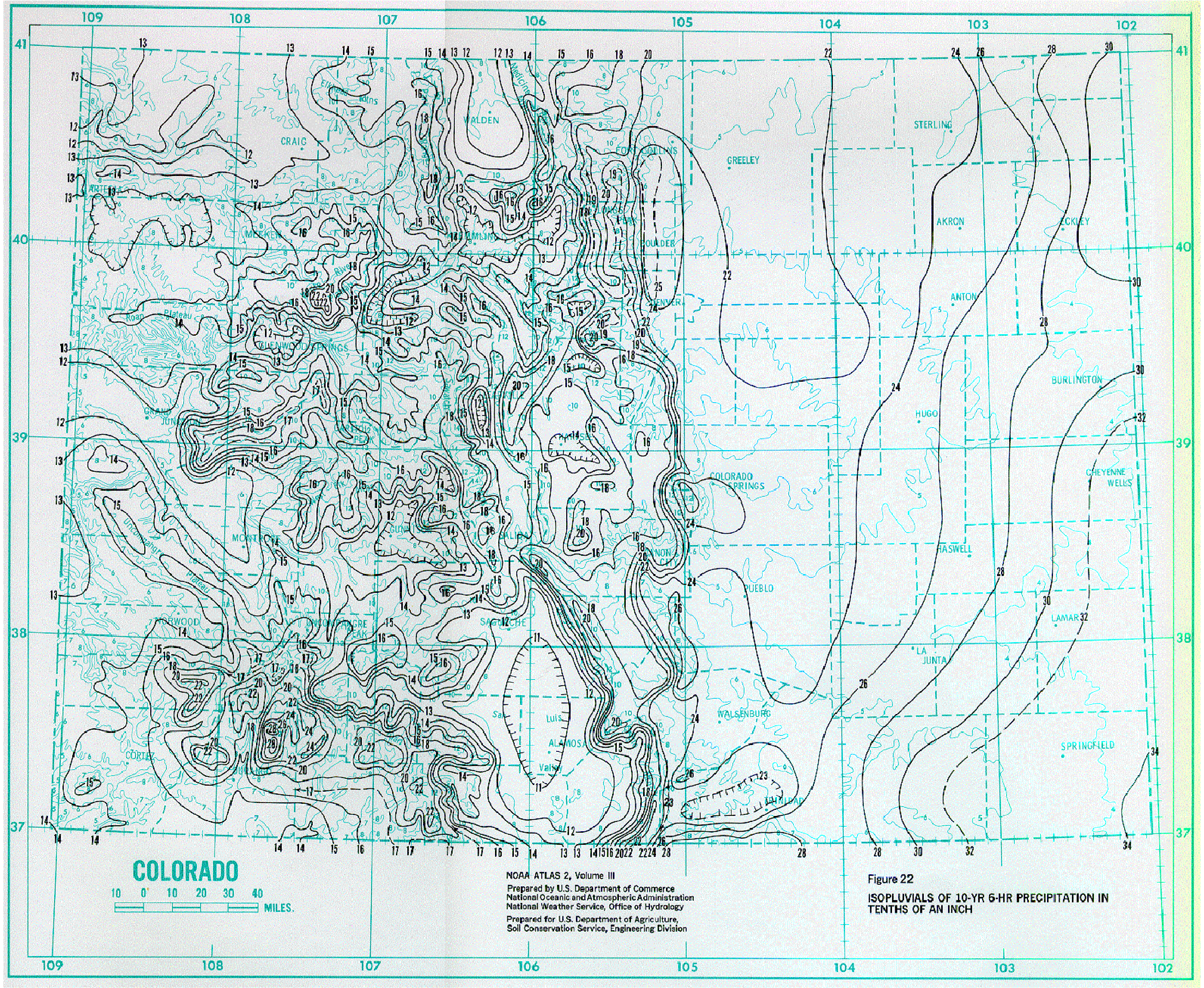

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FIGURE CH9-F403

10-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

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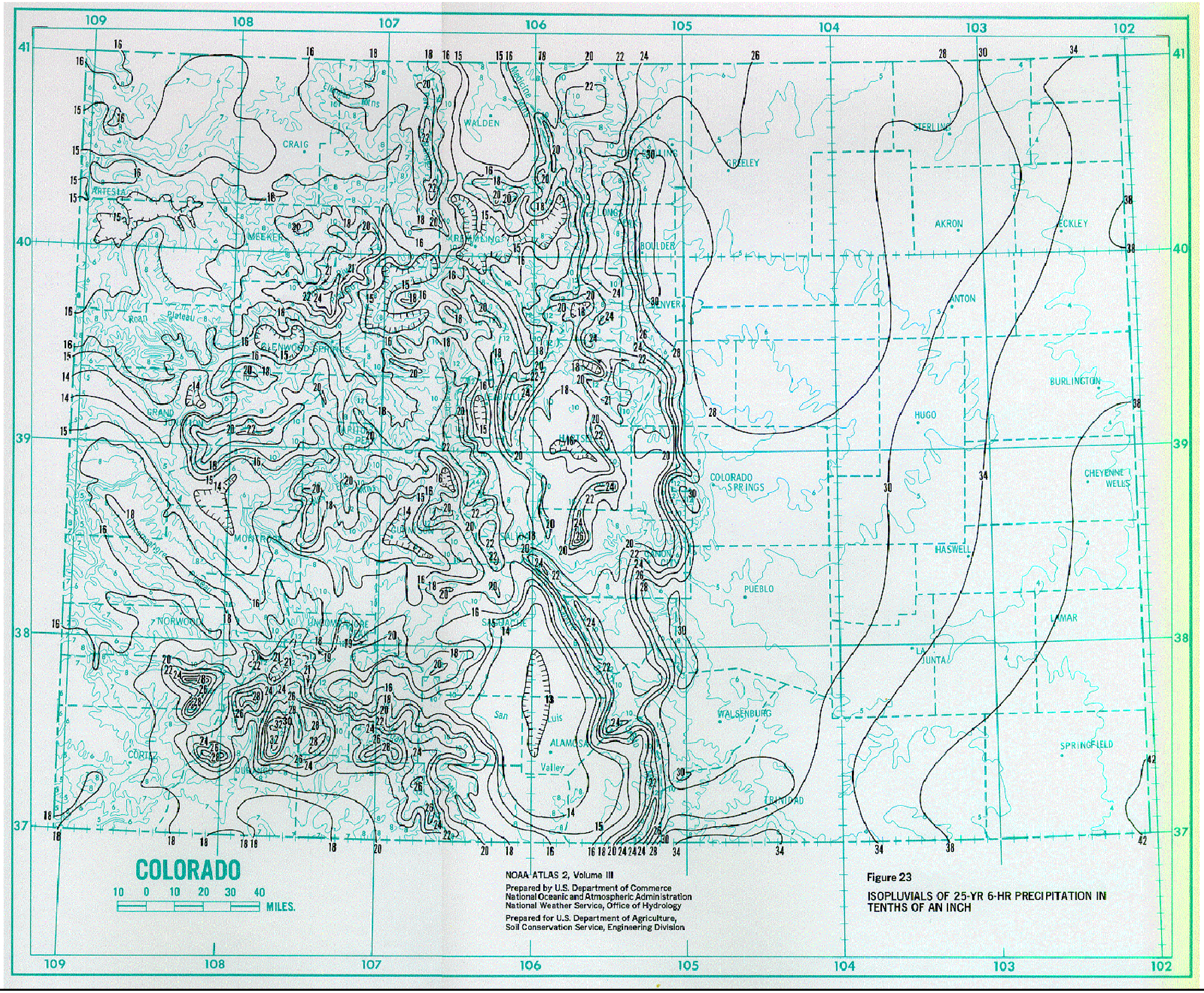

FIGURE CH9-F404

25-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

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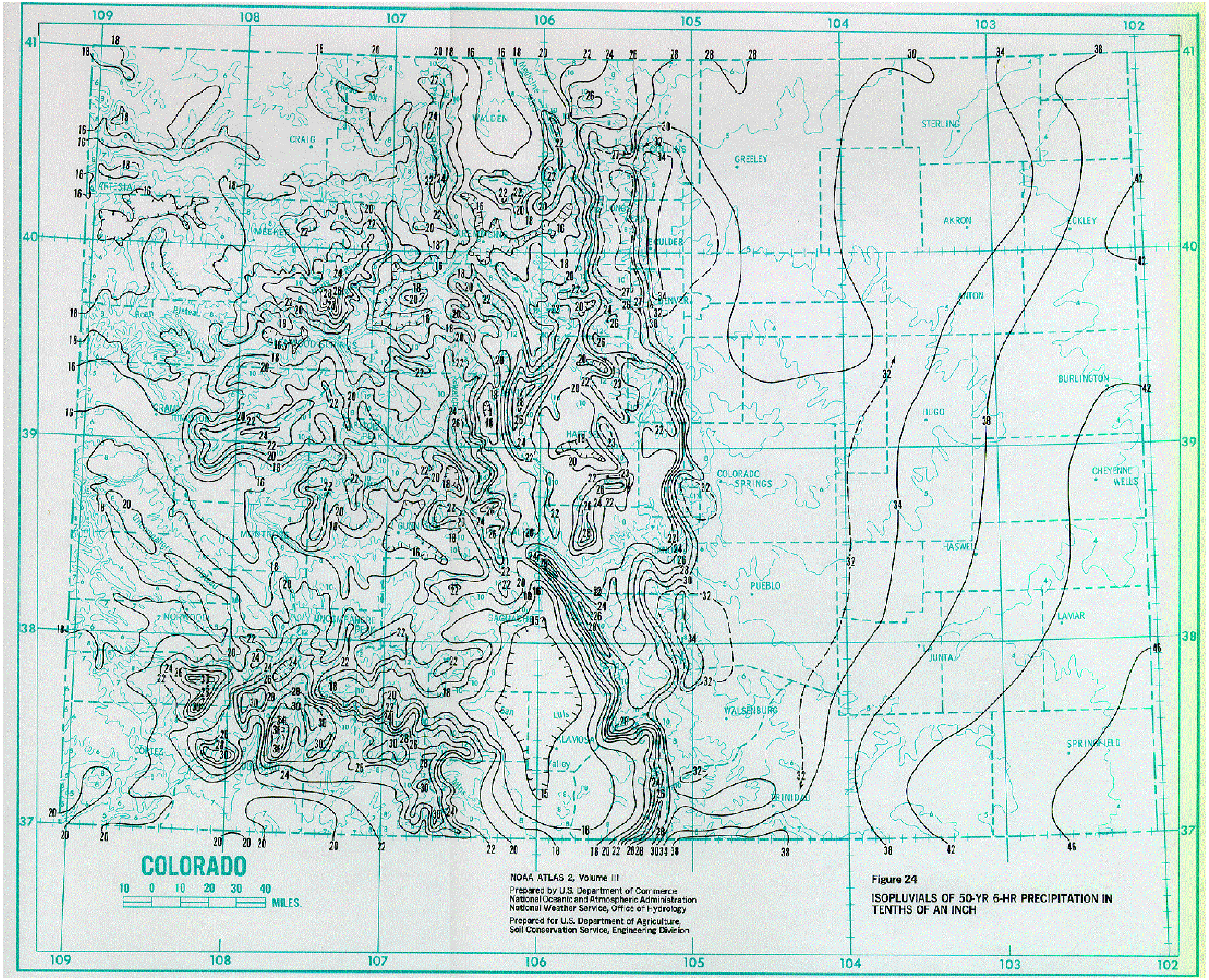


Figure 24

ISOPLUVIALS OF 50-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

FIGURE CH9-F405

50-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

O:/2120/FIGURES/CHAP9-1.DWG, CH9-F405 - 1/6/06 - GPB

COLORADO

FLOODPLAIN AND STORMWATER CRITERIA MANUAL

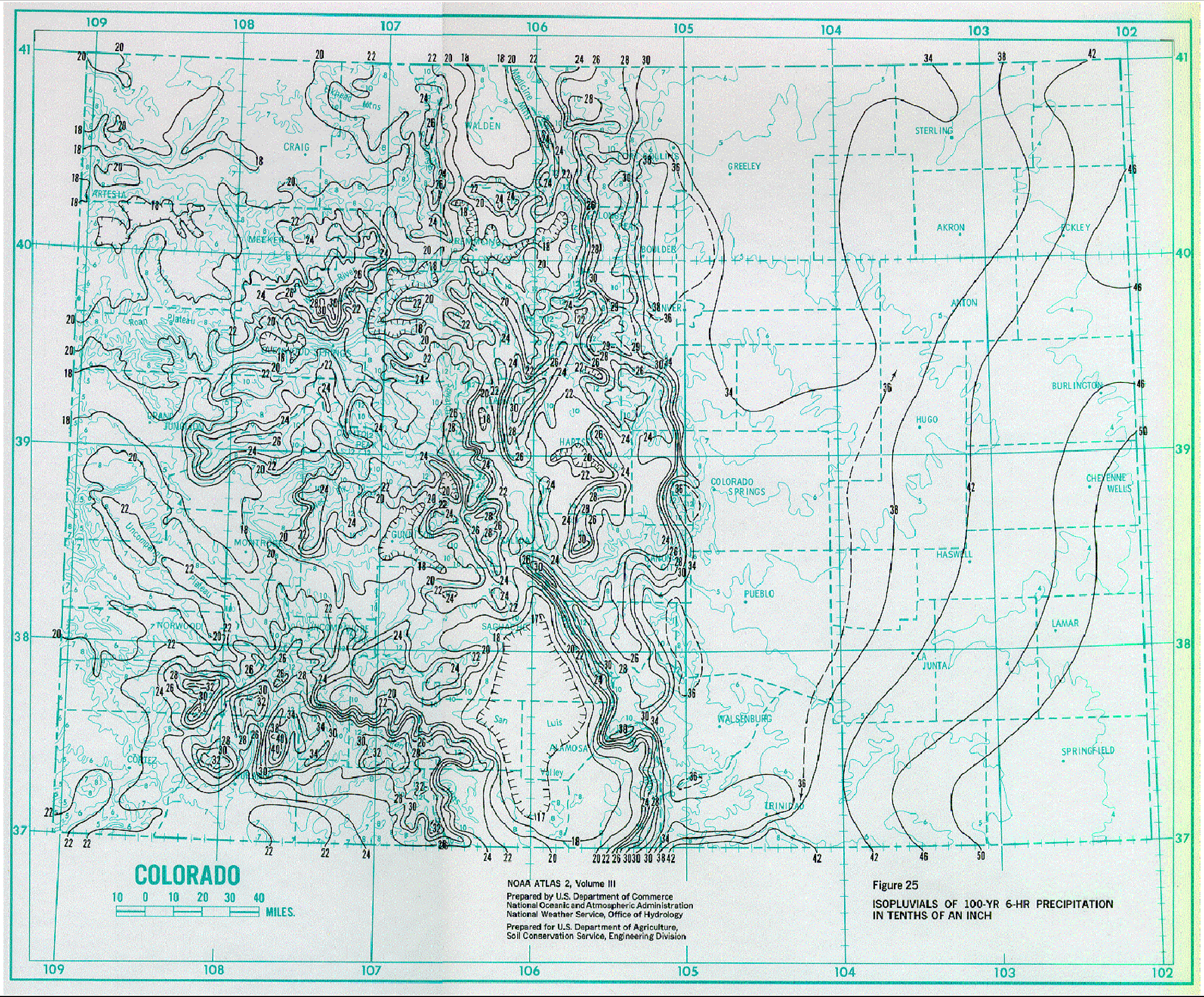


FIGURE CH9-F406

100-YEAR, 6-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

COLORADO

FLOODPLAIN AND STORMWATER CRITERIA MANUAL

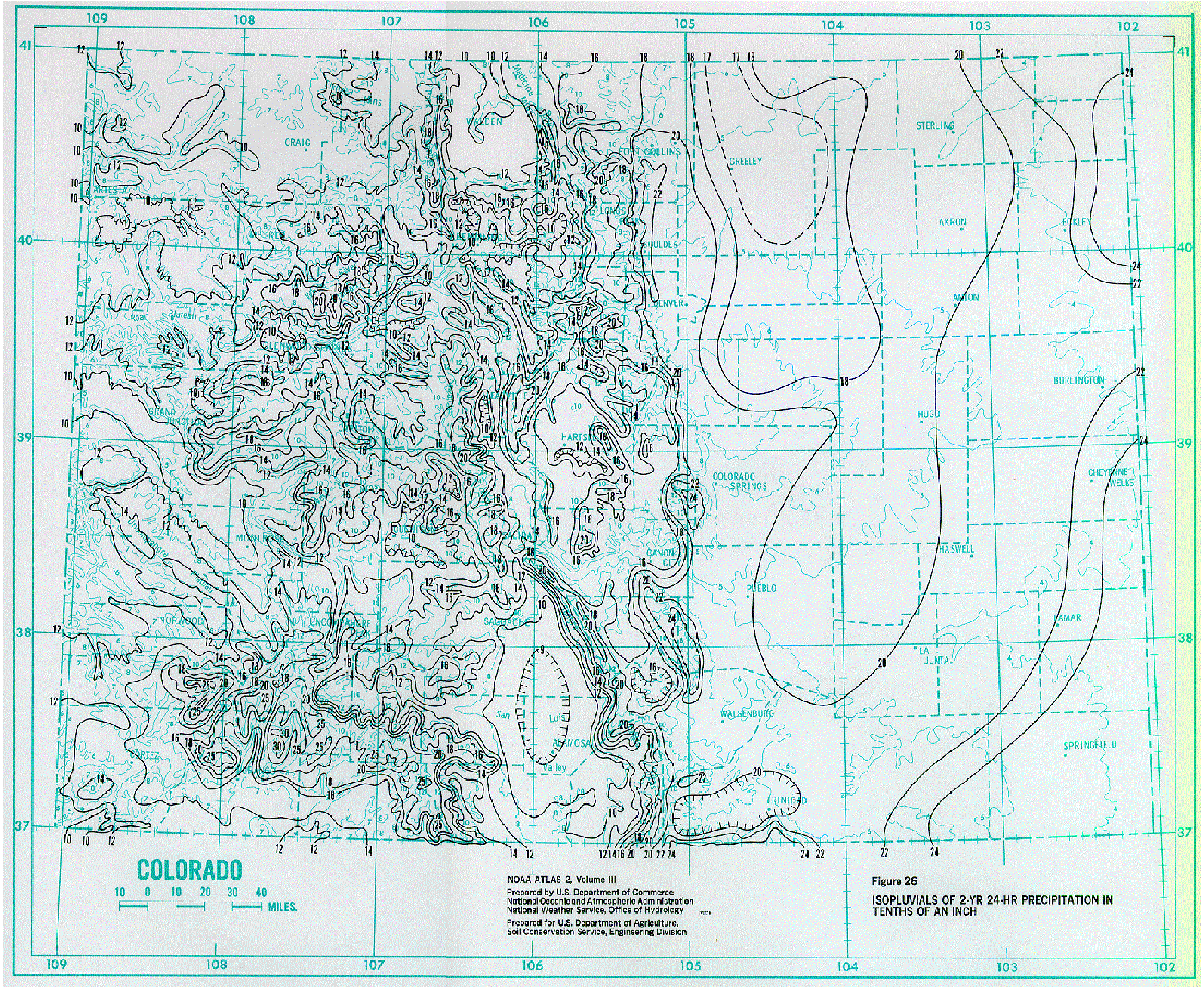


FIGURE CH9-F407

2-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE:

NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

FIGURE CH9-F408

5-YEAR, 24-HOUR PRECIPITATION MAP

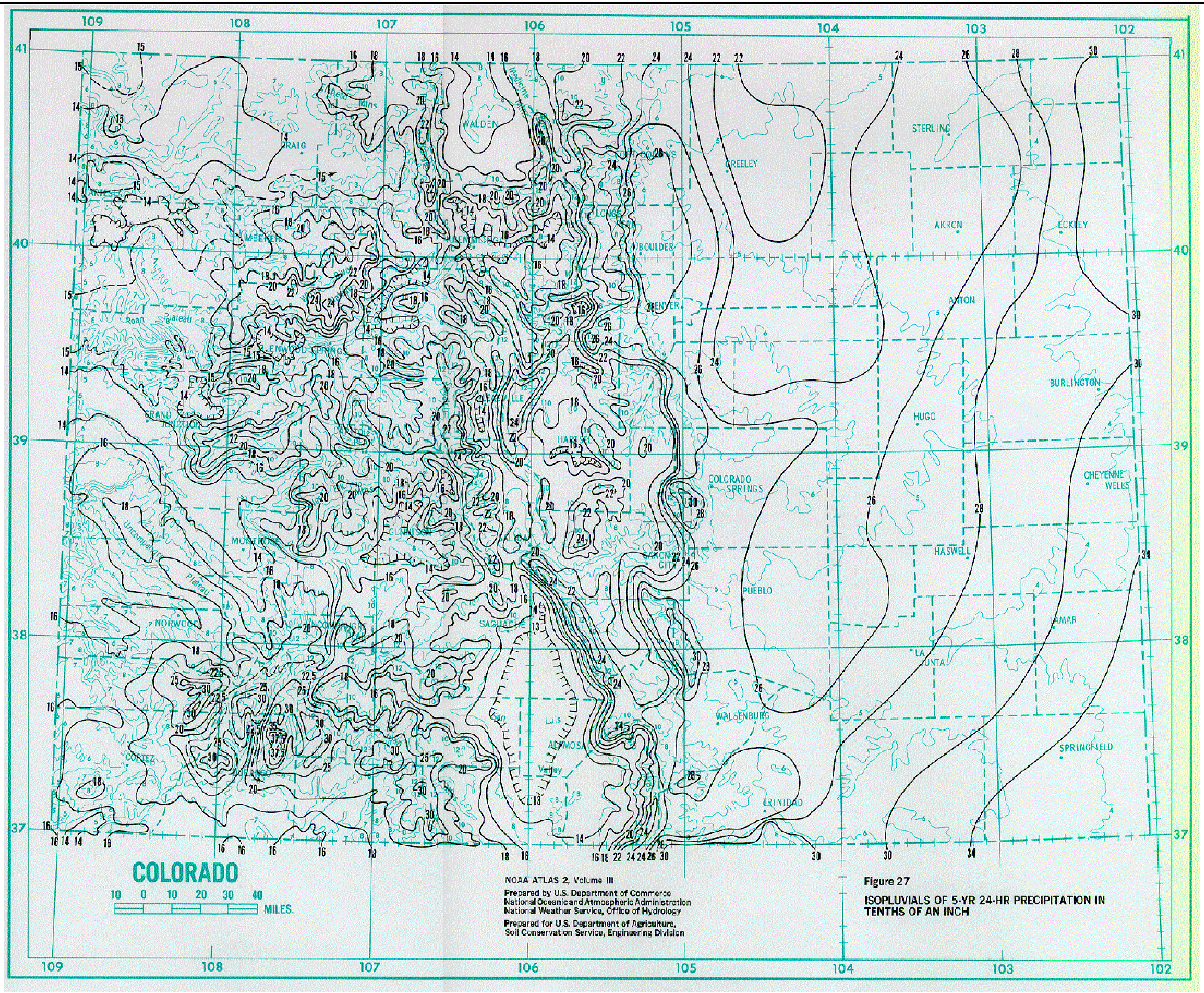
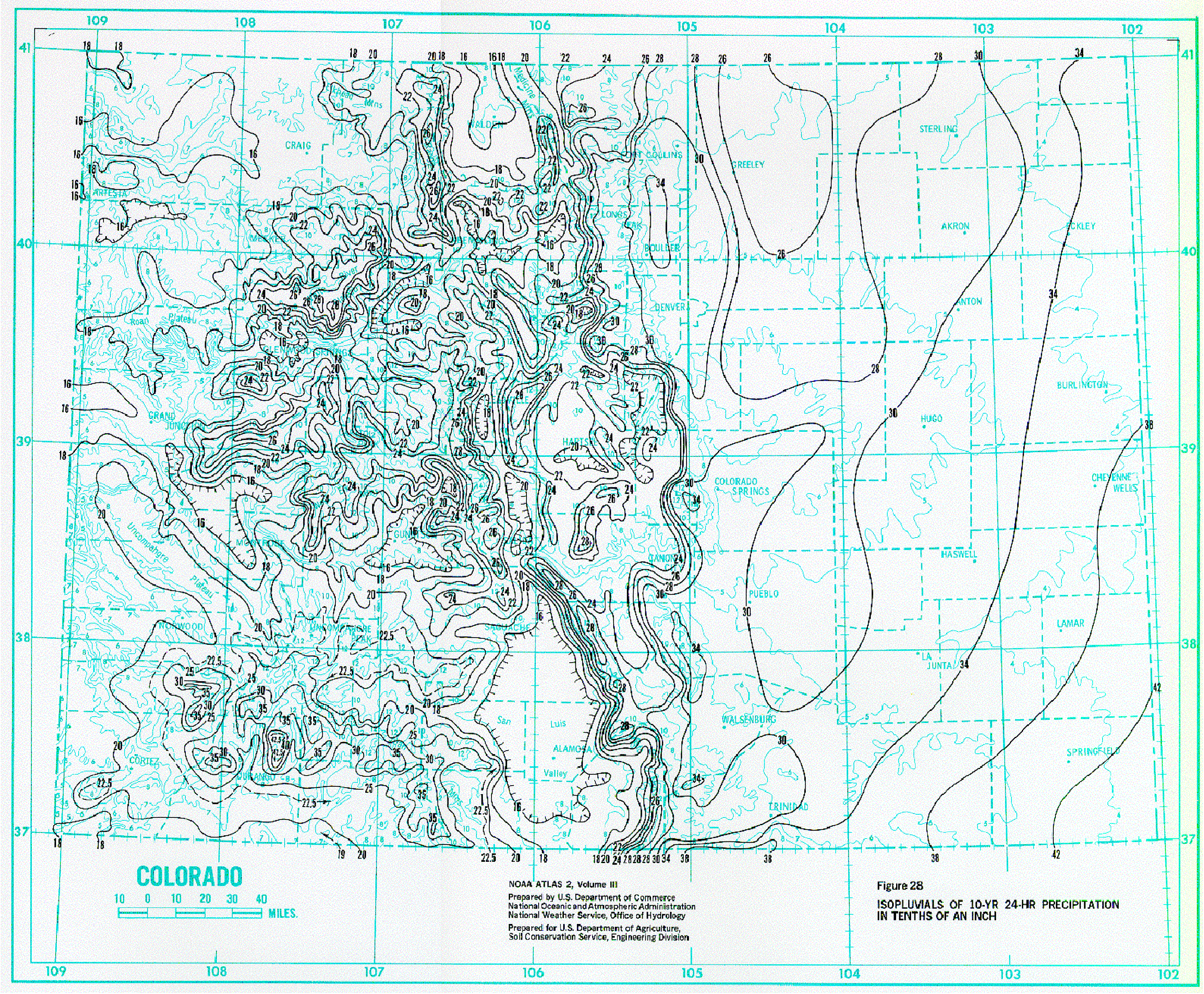


Figure 27
ISOPLUVIALS OF 5-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

FIGURE CH9-F409
10-YEAR, 24-HOUR PRECIPITATION MAP



REFERENCE: NOAA ATLAS 2, VOL III, 1973

COLORADO
FLOODPLAIN AND STORMWATER CRITERIA MANUAL

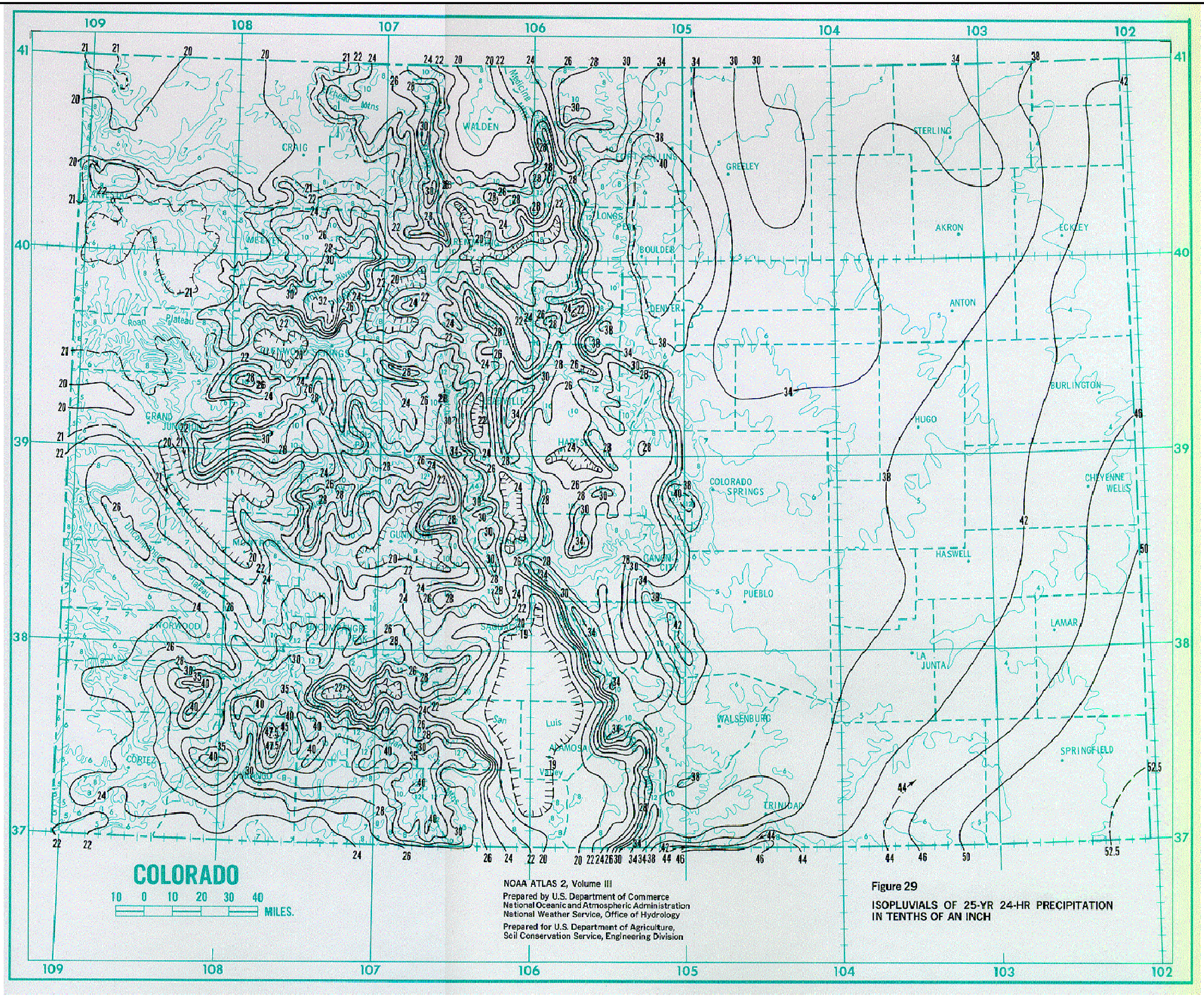


FIGURE CH9-F410
25-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

FIGURE CH9-F411

50-YEAR, 24-HOUR PRECIPITATION MAP

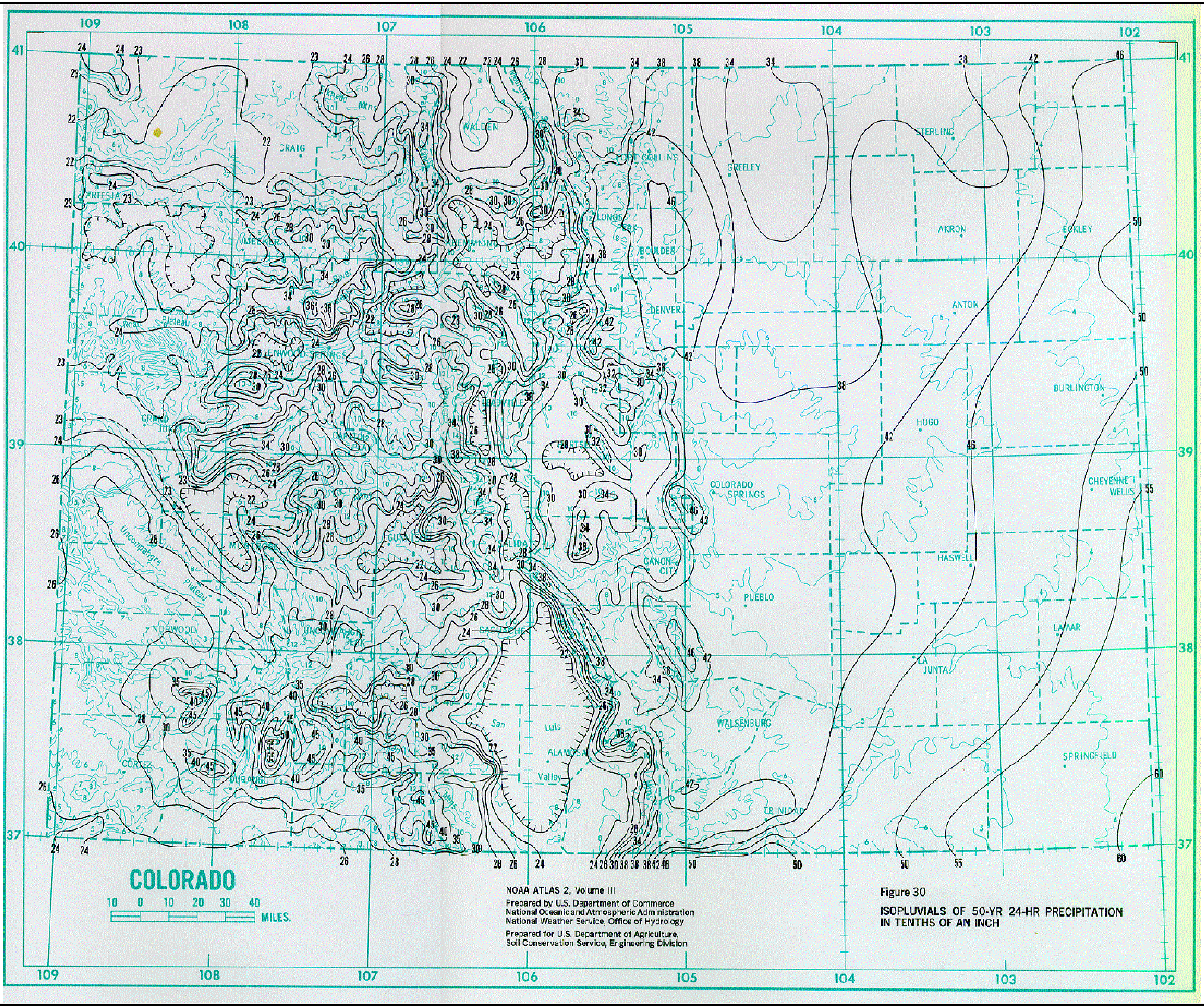


Figure 30
ISOPLUVIALS OF 50-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

COLORADO

FLOODPLAIN AND STORMWATER CRITERIA MANUAL

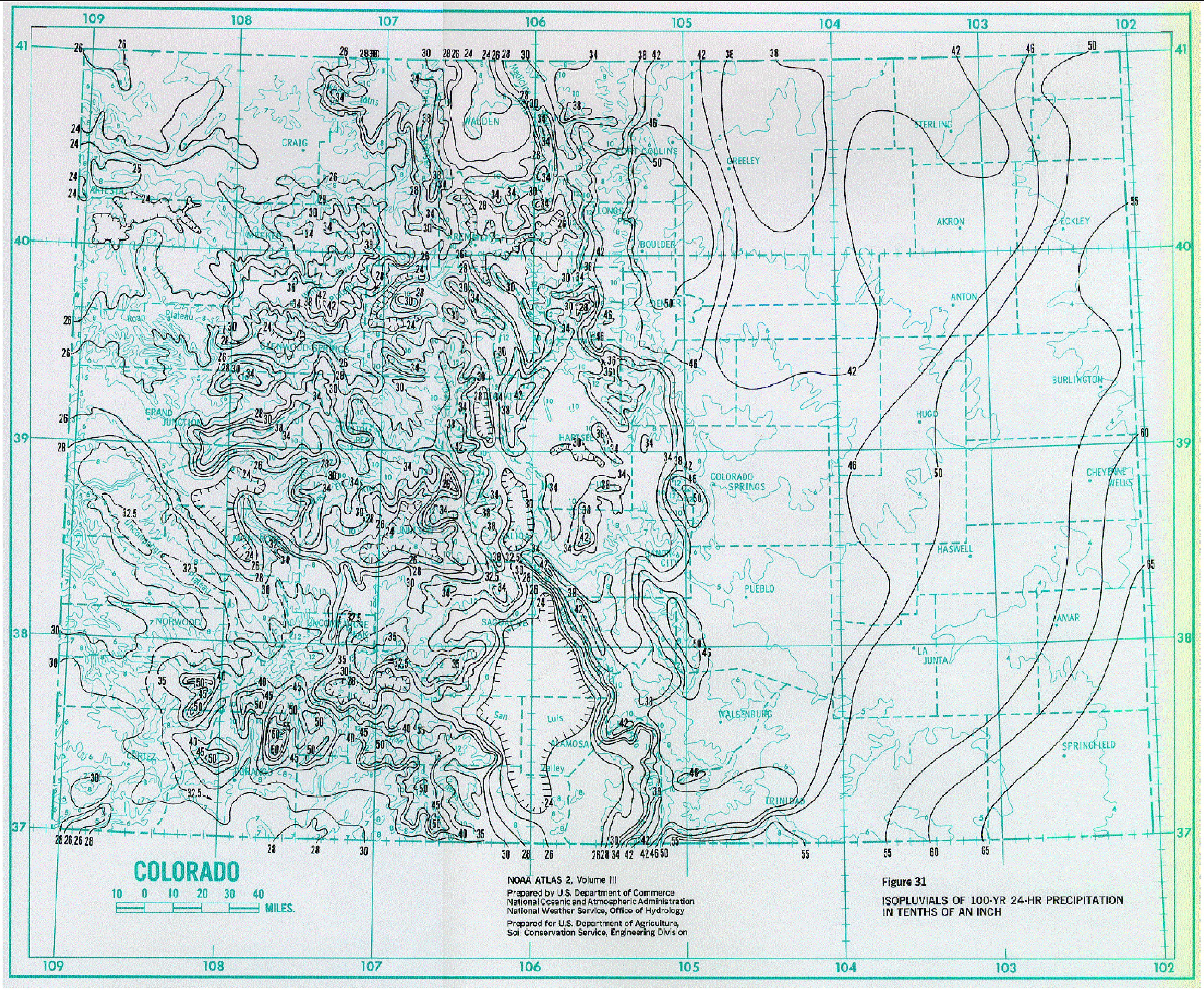


FIGURE CH9-F412

100-YEAR, 24-HOUR PRECIPITATION MAP

REFERENCE: NOAA ATLAS 2, VOL III, 1973

VERSION: JANUARY 2006

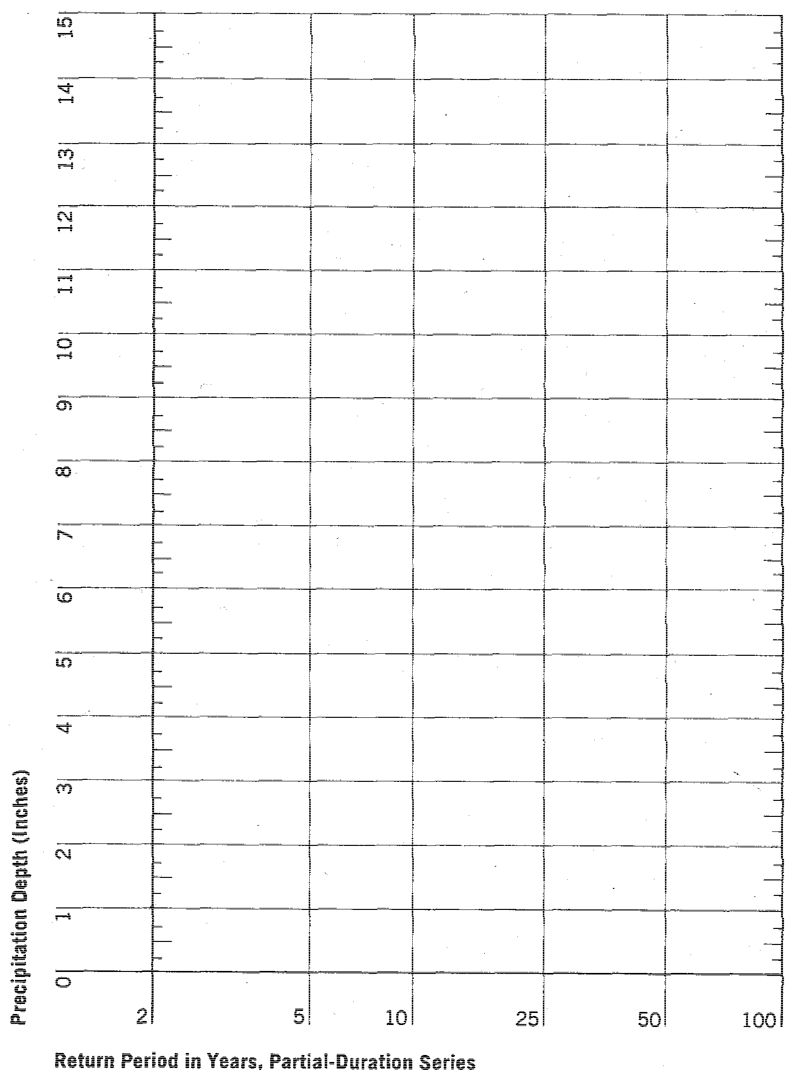


Figure 6. *Precipitation depth versus return period for partial-duration series.*

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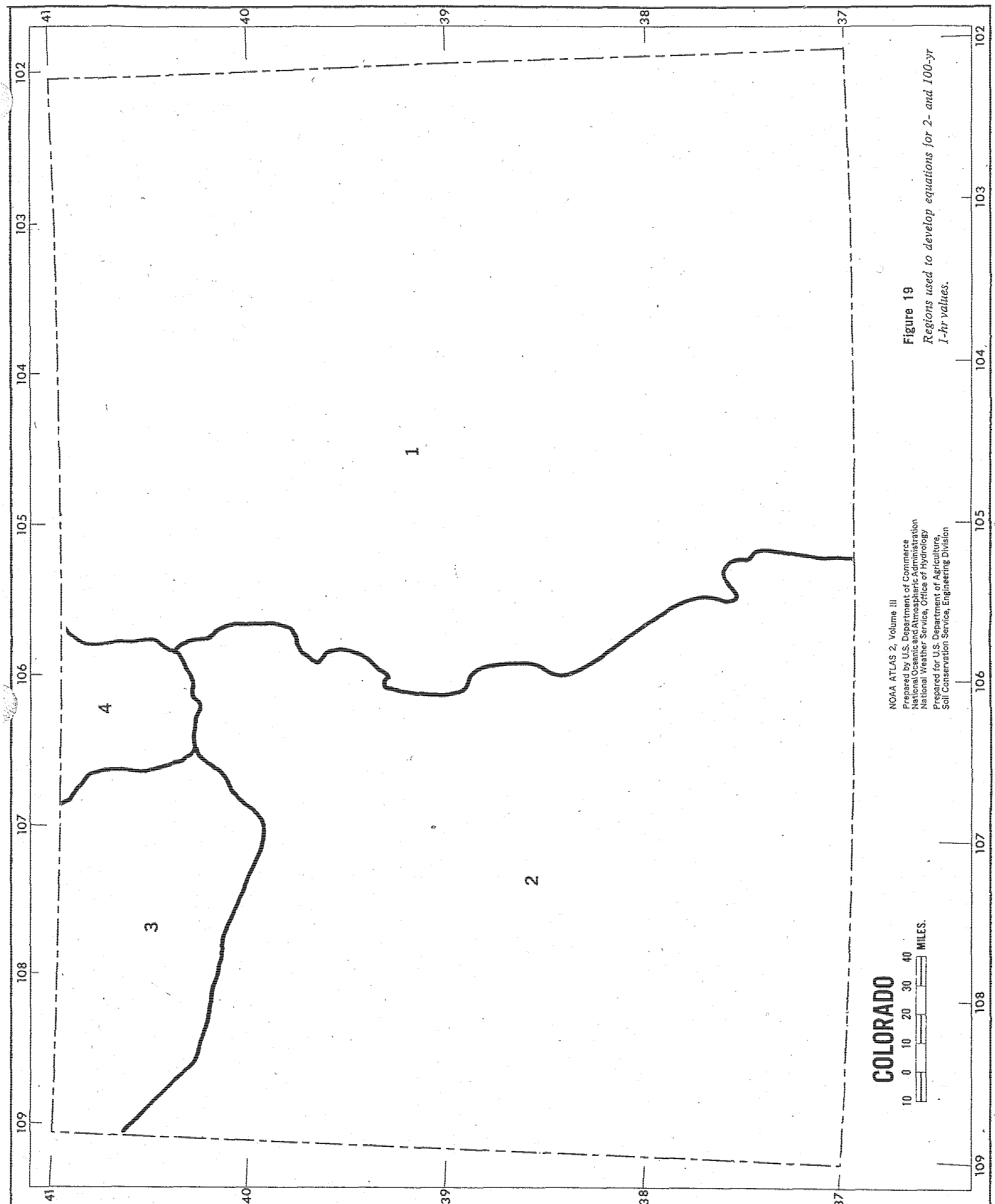
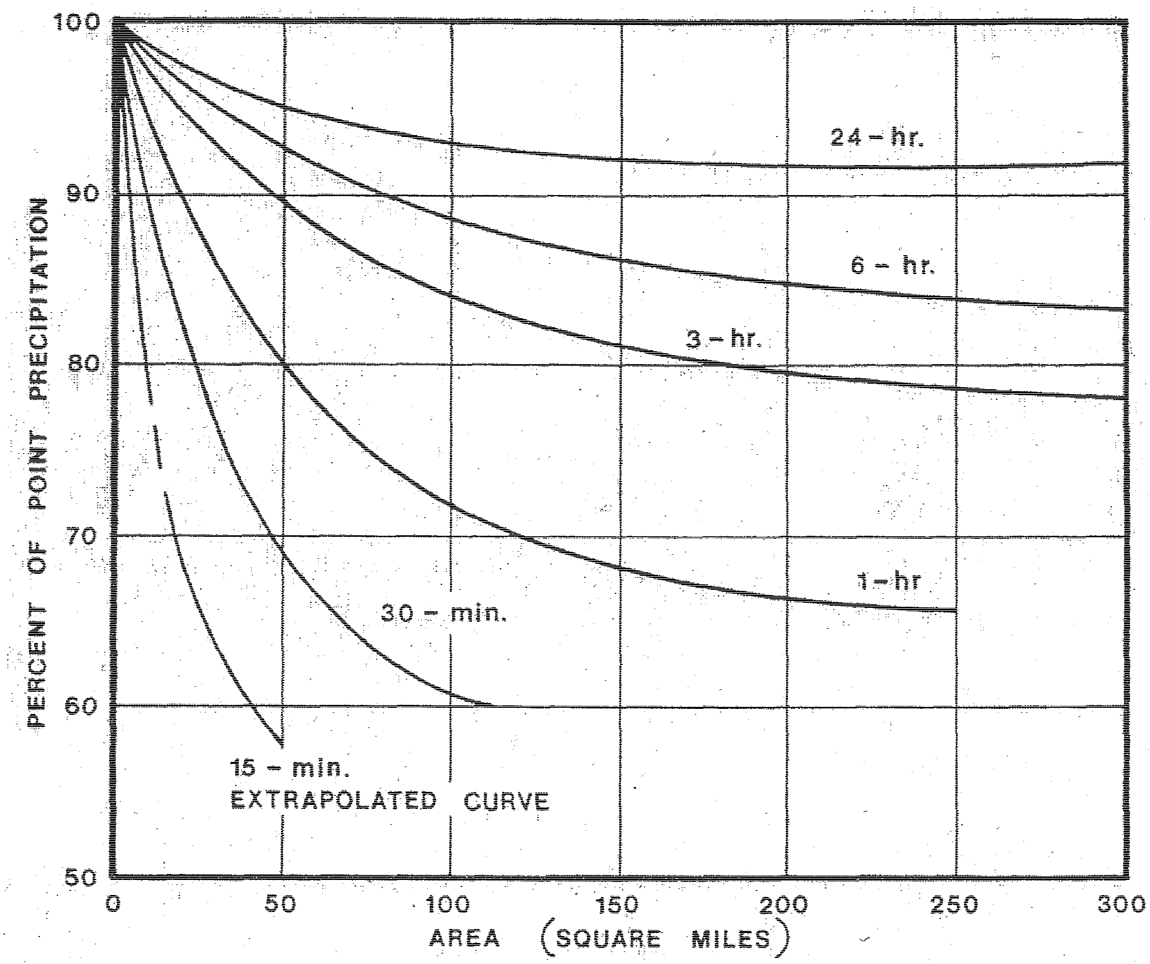
COLORADO

FLOODPLAIN AND STORMWATER CRITERIA MANUAL



VERSION: JANUARY 2006	REFERENCE: NOAA ATLAS 2, VOL II, 1973	**FIGURE CH9-F414** NOAA COLORADO GEOGRAPHICAL REGIONS

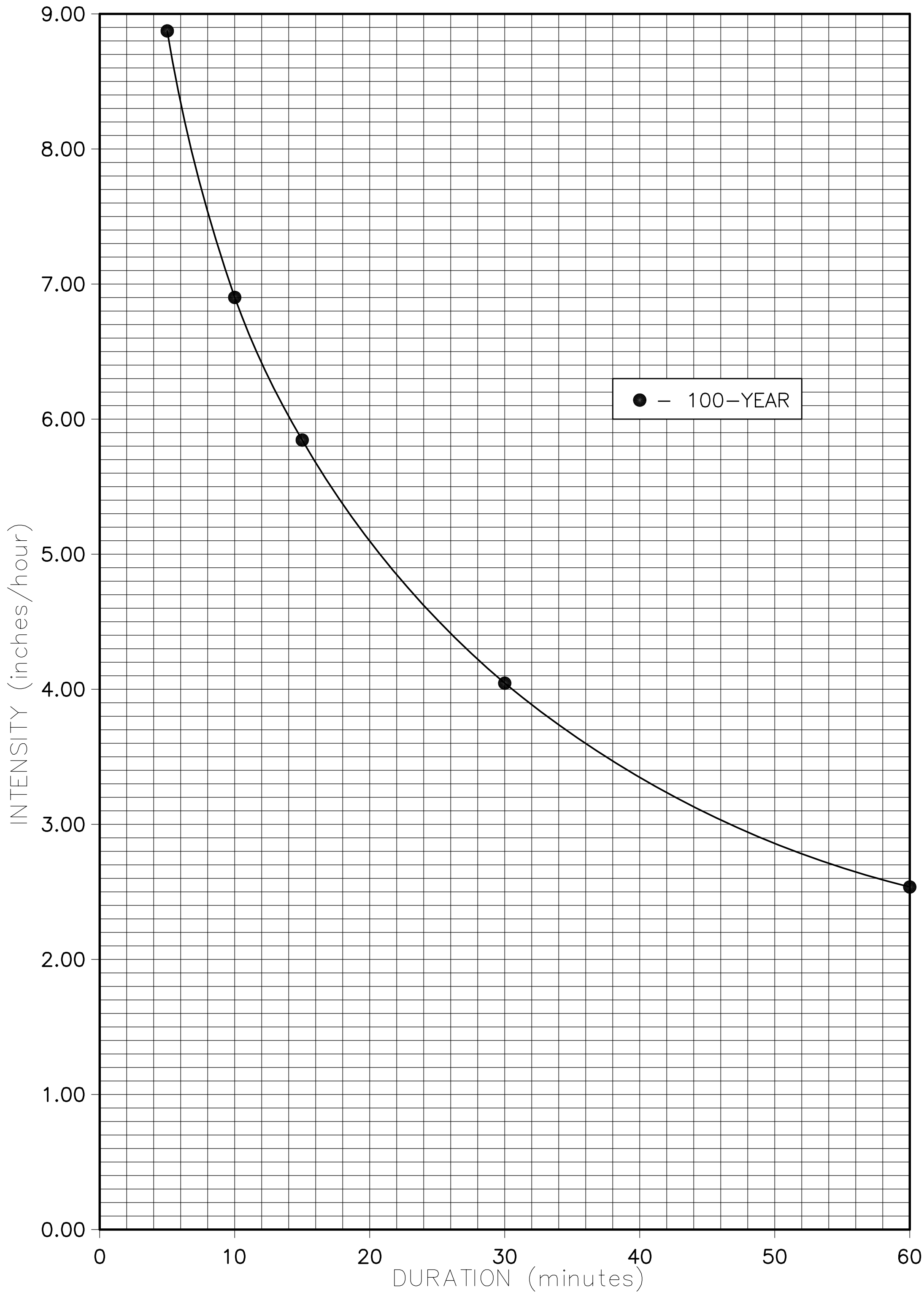
O:/2120/FIGURES/CHAP9-2.DWG, CH9-F414 – 1/6/06 – GPB



VERSION: JANUARY 2006	REFERENCE: NOAA ATLAS 2, VOL II, 1973 UDFCD	FIGURE CH9-F415 DEPTH-AREA ADJUSTMENT CURVES

FIGURE CH9-F415

DEPTH-AREA ADJUSTMENT CURVES



VERSION: JANUARY 2006	REFERENCE:	FIGURE CH9-F416
		EXAMPLE 2: IDF CURVE RESULTS

Duration (min)	5	10	15	30
Ratio to 1-hr	0.29	0.45	0.57	0.79

(Adopted from *U.S. Weather Bureau Technical Paper* No. 40, 1961.)

REFERENCE:
NOAA ATLAS 2, VOL III, 1973

TABLE CH9-T401
ADJUSTMENT FACTORS FOR DURATIONS LESS THAN ONE HOUR

Region of applicability*	Equation	Corr. coeff.	No. of stations	Mean of computed stn. values (inches)	Standard error of estimate (inches)
South Platte, Republican, Arkansas, and Cimarron River Basins (1)	$Y_2 = 0.218 + 0.709[(X_1)(X_1/X_2)]$	0.94	75	1.01	0.074
	$Y_{100} = 1.897 + 0.439[(X_3)(X_3/X_4)] - 0.008Z$	.84	75	2.68	.317
San Juan, Upper Rio Grande, Upper Colorado, and Gunnison River Basins and Green River Basin below confluence with the Yampa River (2)	$Y_2 = -0.011 + 0.942[(X_1)(X_1/X_2)]$	.95	86	0.72	.085
	$Y_{100} = 0.494 + 0.755[(X_3)(X_3/X_4)]$	.90	85	1.96	.290
Yampa and Green River Basins above confluence of Green and Yampa Rivers (3)	$Y_2 = 0.019 + 0.711[(X_1)(X_1/X_2)] + 0.001Z$	.82	98	0.40	.031
	$Y_{100} = 0.338 + 0.670[(X_3)(X_3/X_4)] + 0.001Z$	.80	79	1.04	.141
North Platte (4)	$Y_2 = 0.028 + 0.890[(X_1)(X_1/X_2)]$	.93	90	0.60	.062
	$Y_{100} = 0.671 + 0.757[(X_3)(X_3/X_4)] - 0.003Z$	.91	88	1.71	.236

* Numbers in parentheses refer to geographic regions shown in figure 19. See text for more complete description.

List of variables

Y_2 = 2-yr 1-hr estimated value
Y_{100} = 100-yr 1-hr estimated value
X_1 = 2-yr 6-hr value from precipitation-frequency maps
X_2 = 2-yr 24-hr value from precipitation-frequency maps
X_3 = 100-yr 6-hr value from precipitation-frequency maps
X_4 = 100-yr 24-hr value from precipitation-frequency maps
Z = point elevation in hundreds of feet

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Time Minutes	Percent of 1-Hour NOAA Rainfall Atlas Depth				
	2-Year	5-Year	10-Year	25- and 50-Year	100- and 500-Year
5	2.0	2.0	2.0	1.3	1.0
10	4.0	3.7	3.7	3.5	3.0
15	8.4	8.7	8.2	5.0	4.6
20	16.0	15.3	15.0	8.0	8.0
25	25.0	25.0	25.0	15.0	14.0
30	14.0	13.0	12.0	25.0	25.0
35	6.3	5.8	5.6	12.0	14.0
40	5.0	4.4	4.3	8.0	8.0
45	3.0	3.6	3.8	5.0	6.2
50	3.0	3.6	3.2	5.0	5.0
55	3.0	3.0	3.2	3.2	4.0
60	3.0	3.0	3.2	3.2	4.0
65	3.0	3.0	3.2	3.2	4.0
70	2.0	3.0	3.2	2.4	2.0
75	2.0	2.5	3.2	2.4	2.0
80	2.0	2.2	2.5	1.8	1.2
85	2.0	2.2	1.9	1.8	1.2
90	2.0	2.2	1.9	1.4	1.2
95	2.0	2.2	1.9	1.4	1.2
100	2.0	1.5	1.9	1.4	1.2
105	2.0	1.5	1.9	1.4	1.2
110	2.0	1.5	1.9	1.4	1.2
115	1.0	1.5	1.7	1.4	1.2
120	1.0	1.3	1.3	1.4	1.2
Totals	115.7	115.7	115.7	115.6	115.6

TABLE CH9-T403
CUHP 2-HOUR DESIGN STORM DISTRIBUTION TABLE

	2-, 5-, and 10-Year Design Rainfall Area—Square Miles				25-, 50-, 100-, and 500-Year Design Rainfall Area—Square Miles			
Time Minutes	10-20	20-30	30-50	50-75	10-20	20-30	30-50	50-75
5	1.00	1.00	1.10	1.10	1.00	1.00	1.05	1.10
10	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
15	1.00	1.00	1.05	1.00	1.00	1.00	1.05	1.10
20	0.90	0.81	0.74	0.62	1.00	1.00	1.05	1.00
25	0.90	0.81	0.74	0.62	0.90	0.81	0.74	0.60
30	0.90	0.81	0.74	0.62	0.90	0.81	0.74	0.60
35	1.00	1.00	1.05	1.00	0.90	0.81	0.74	0.70
40	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.00
45	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
50	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
55	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
60	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
65 - 120	1.00	1.00	1.05	1.10	1.00	1.00	1.05	1.10
125 - 180	1.00	1.15	1.20	1.40	1.00	1.15	1.20	1.40
185 - 360	N/A	1.15	1.20	1.20	N/A	1.15	1.20	1.20

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Time (min.)	2-hr Dist. %	2-hr Design Storm Dist. (inches)	3-hr Design Storm Dist. (inches)	Area Adjustment Factors	Adjusted 3-hr Design Storm Dist. (inches)
0		0.00	0.00		0.00
5.0	1.0	0.02	0.02	1.00	0.02
10.0	3.0	0.07	0.07	1.00	0.07
15.0	4.6	0.10	0.10	1.00	0.10
20.0	8.0	0.18	0.18	1.00	0.18
25.0	14.0	0.31	0.31	0.90	0.28
30.0	25.0	0.55	0.55	0.90	0.50
35.0	14.0	0.31	0.31	0.90	0.28
40.0	8.0	0.18	0.18	1.00	0.18
45.0	6.2	0.14	0.14	1.00	0.14
50.0	5.0	0.11	0.11	1.00	0.11
55.0	4.0	0.09	0.09	1.00	0.09
60.0	4.0	0.09	0.09	1.00	0.09
65.0	4.0	0.09	0.09	1.00	0.09
70.0	2.0	0.04	0.04	1.00	0.04
75.0	2.0	0.04	0.04	1.00	0.04
80.0	1.2	0.03	0.03	1.00	0.03
85.0	1.2	0.03	0.03	1.00	0.03
90.0	1.2	0.03	0.03	1.00	0.03
95.0	1.2	0.03	0.03	1.00	0.03
100.0	1.2	0.03	0.03	1.00	0.03
105.0	1.2	0.03	0.03	1.00	0.03
110.0	1.2	0.03	0.03	1.00	0.03
115.0	1.2	0.03	0.03	1.00	0.03
120.0	1.2	0.03	0.03	1.00	0.03
125.0			0.02	1.00	0.02
130.0			0.02	1.00	0.02
135.0			0.02	1.00	0.02
140.0			0.02	1.00	0.02
145.0			0.02	1.00	0.02
150.0			0.02	1.00	0.02
155.0			0.02	1.00	0.02
160.0			0.02	1.00	0.02
165.0			0.02	1.00	0.02
170.0			0.02	1.00	0.02
175.0			0.02	1.00	0.02
180.0			0.02	1.00	0.02
Total	**115.6**	**2.54**	**2.75**		**2.64**

TABLE CH9-T405

EXAMPLE 3: 100-YEAR RAINFALL DISTRIBUTION RESULTS