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Flood Control District of Maricopa County

Volume I - Report

Hydrology Study

Dambreak Analyses for Powerline, Vineyard, and Rittenhouse Flood Retarding Structures (FCD 88-37)

August 1989

Registered Professional Engineer (Civil)
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ARIZONA, U.S.A.

JMM James M. Montgomery
Consulting Engineers Inc.

FLOOD CONTROL DISTRICT OF
MARICOPA COUNTY

HYDROLOGY STUDY

DAMBREAK ANALYSES
FOR
POWERLINE, VINEYARD, AND RITTENHOUSE
FLOOD RETARDING STRUCTURES

Prepared by

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August 1989

Final Report

Serving the World's
Environmental Needs

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JMM James M. Montgomery
Consulting Engineers Inc.

August 7, 1989

Flood Control District
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ATTN: Mr. J. M. Rumann
Hydrologist

SUBJECT: Final Hydrology Report - Dambreak Analyses for Powerline, Vineyard, and Rittenhouse Flood Retarding Structures (FCD 88-37)

Gentlemen:

James M. Montgomery, Consulting Engineers, Inc. (JMM) is pleased to present ten (10) copies of the Final Hydrology Report for the drainage areas above the Powerline, Vineyard, and Rittenhouse Flood Retarding Structures (FRS). The final report was prepared after receiving review comments on the draft report from the District during our meeting of July 13, 1989.

As specified in the contract scope of work, hydrologic models using the HEC-1 computer program were prepared for the drainage areas above the Vineyard and Rittenhouse FRS's. Five storms were analyzed for these drainage areas, including the 25-, 50-, and 100-year events along with the 6-hour and 72-hour probable maximum precipitation (PMP) events. For the Powerline drainage area, the existing HEC-1 model developed by the District was used after revisions to analyze the five storms.

Also submitted with the ten (10) Final Hydrology Reports are original mylar overlays showing the contract-specified drainage information and computer diskettes containing the hydrologic models.

It has been a pleasure to work with the District on this interesting project.

Very truly yours,

Fred K. Duren, Jr.

Fred K. Duren, Jr., P.E., P.G.
Project Manager

/jw

cc: 1212.0020.3.1.2

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[a] Full-page figures are on page following number shown in this list.

ABBREVIATIONS

Abbreviation	Definition
AMC	antecedent moisture condition
CAP	Central Arizona Project
cfs	cubic feet per second
csm	cubic feet per second per square mile
DARF	drainage-area-reduction factor
District, FCD	Flood Control District of Maricopa County
FRS	Flood Retarding Structure
HEC	Hydrologic Engineering Center, Corps of Engineers
HMR	Hydrometeorological Report
JMM	James M. Montgomery, Consulting Engineers, Inc.
l.ft.	lineal feet
NOAA	National Oceanic and Atmospheric Administration
NWS	National Weather Service
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RCP	reinforced concrete pipe
SCS	Soil Conservation Service
sq. mi.	square mile
T_c	time of concentration
USGS	U.S. Geological Survey

CHAPTER 1

INTRODUCTION

The contract for this project includes two separate elements: hydrologic studies for two drainage basins and dambreak analyses for three flood retarding structures (FRS) which are operated by the Flood Control District of Maricopa County (District). Complete, new hydrologic models were developed for drainage basins tributary to the Vineyard and Rittenhouse FRS's, while an existing hydrologic model was utilized for performing hydrologic evaluations for the basin tributary to the Powerline FRS.

The hydrologic studies undertaken in the project are the subjects of this report. The dambreak analyses are covered in a separate report.

A brief introduction of the hydrologic studies is provided in this chapter of the report. Later chapters describe the technical analyses and results in more detail.

PROJECT AUTHORIZATION

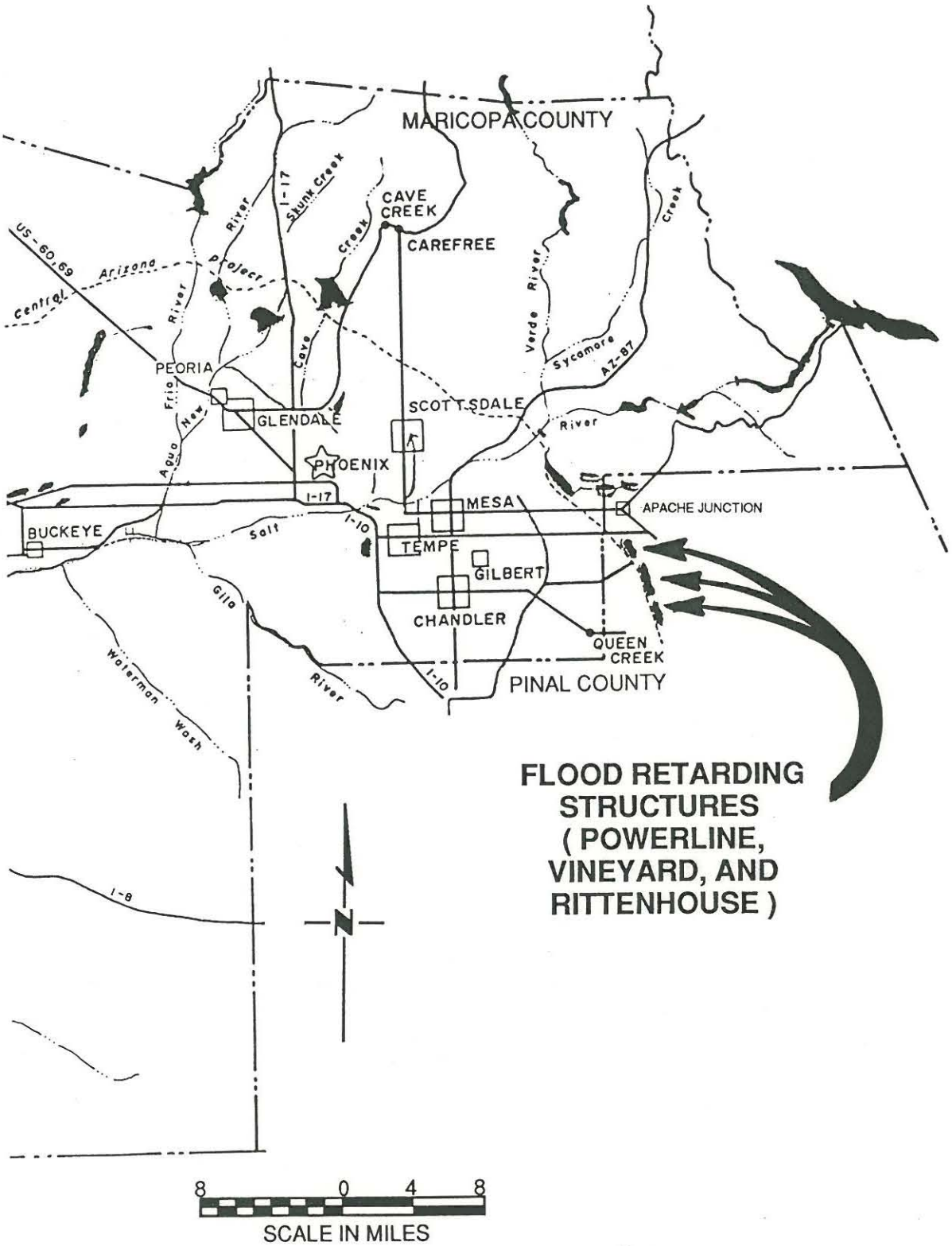
The project was authorized by a contract between the District and James M. Montgomery, Consulting Engineers, Inc. (JMM) dated February 22, 1989, and is designated FCD Project No. 88-37.

PROJECT DESCRIPTION

The hydrologic studies project element is described in the following paragraphs.

Location

As shown in Figures 1-1 and 1-2, the three FRS's are located in northwestern Pinal County, south of Apache Junction. The combined watershed above the three FRS's covers 146.9 square miles, extending from the desert floor at a minimum elevation of 1,560 feet northeastward to the peaks of the Superstition Mountains at a maximum elevation of 5,057 feet.



VICINITY MAP

FIGURE 1-1

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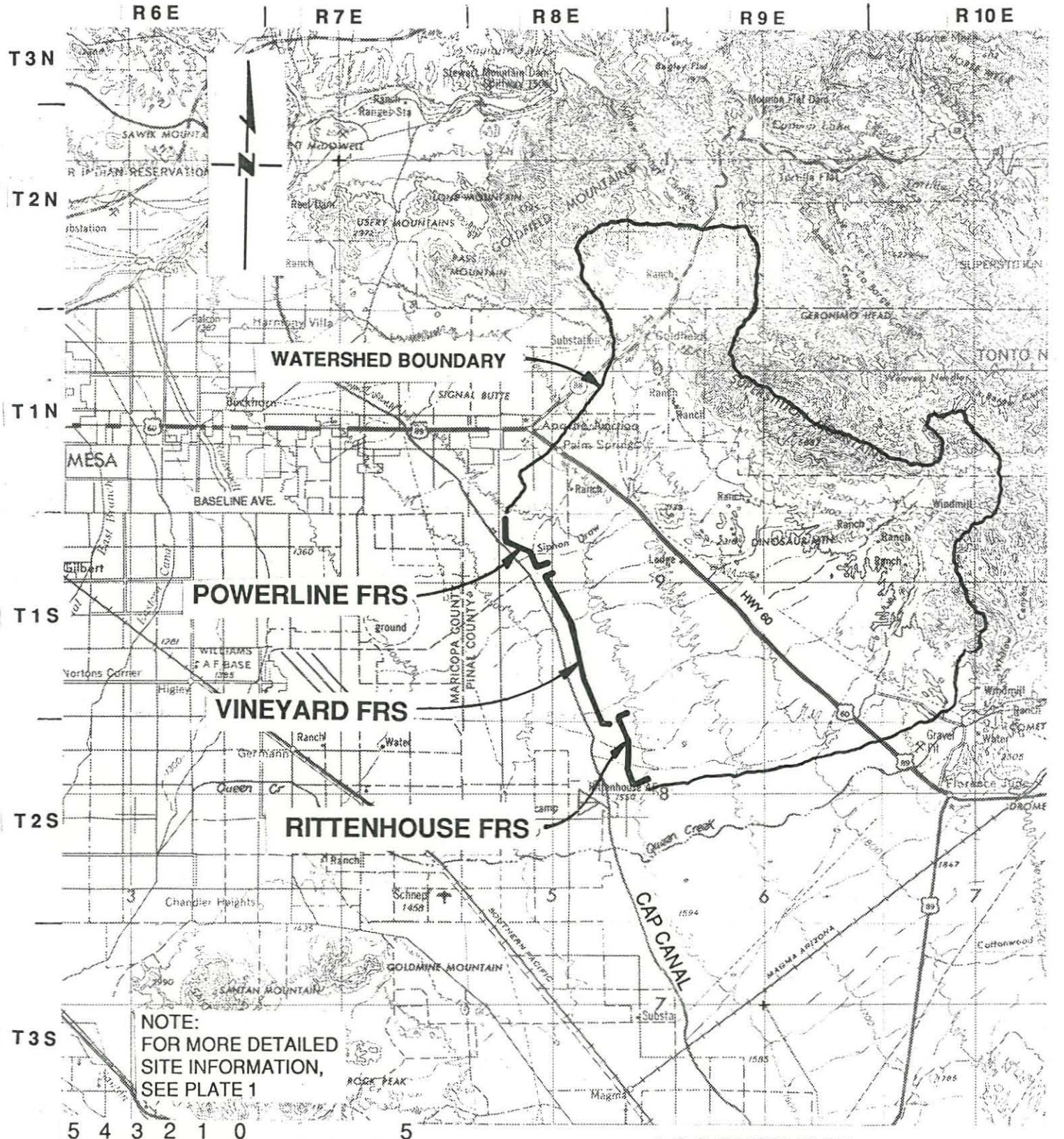


LOCATION MAP

FIGURE 1-2

JMM

Purposes

There are two purposes for the project. The first is to develop hydrologic models for the drainage basins above the Vineyard and Rittenhouse FRS's. These models are to be delivered to the District on flexible diskettes for use in performing subsequent hydrologic evaluations of the basins for later periods when conditions change. The models are also to be used during this study to assess the capabilities of each FRS under the selected flooding conditions. A secondary task of the hydrologic studies, which was added to the scope after the project began, was to prepare revisions to the existing Powerline hydrologic model and run the revised model to evaluate the selected storms. Discussions of the preparation of the hydrologic models for the Vineyard and Rittenhouse basins, the revisions to the Powerline model, and the evaluations of FRS performance are contained in this report.

The second purpose of the project is to develop inundation information for assumed failures of the FRS's. This information is to be submitted by the District to the Arizona Department of Water Resources (ADWR) Dam Safety Division as part of the dam certification requirements for the three FRS's. Presentation of inundation information is contained in the dambreak analyses report.

Scope of Work

The scope of work for the hydrologic studies includes development of inflow hydrographs for five flooding events: the 25-, 50-, and 100-year floods and the 6- and 72-hour probable maximum floods (PMF). These inflow hydrographs were to be prepared for the three FRS's, and the performance of each FRS determined by routing the floods through the reservoirs.

Hydrologic models were to be developed for the drainage basins above the Vineyard and Rittenhouse FRS's using the HEC-1 program. In the original scope of work, an existing HEC-1 hydrologic model for the Powerline FRS basin was to be utilized without modification to develop hydrographs for the 25- and 50-year floods. The existing Powerline model included precipitation data for the 100-year and two PMF flooding events and no new analyses were to be performed for these three events. Precipitation data for the 25- and 50-year events were to be input and the model run to estimate runoff from these two additional storms in the Powerline basin.

After the project was underway, the scope of work was expanded to include revisions to the original Powerline hydrologic model. These revisions included inputting precipitation data that were the same as used for the Vineyard and Rittenhouse hydrologic model and changing the loss rates to reflect existing watershed conditions.

A summary of the seven tasks that were performed as part of the hydrologic study element of the project is provided as follows:

Task 1: Study and Document Flooding History.

Several elements were involved in this task, including the initial kickoff meeting, collecting data for all study tasks, field reconnaissance of the watersheds, subbasin selections, and documentation of available flooding history within the area of the watersheds and the flood retarding structures.

Task 2: Review Drainage Studies Completed Within the Area and Summarize in Report.

Several reports that describe drainage studies within the project area were reviewed and pertinent information from them summarized.

Task 3: Provide Specs and Prepare the Maps Required to Analyze Watershed Characteristics.

In accordance with the contract, selected methodologies were used to perform the hydrologic analyses. These methodologies included the use of Corps of Engineers program HEC-1 to develop basin hydrology and the use of the Soil Conservation Service's (SCS) unit hydrograph method. A more complete discussion of the methodologies stipulated by the contract is presented in Chapter 2.

All hydrologic specifications necessary to perform the analyses by the stipulated methodologies were developed during this task. These specifications included subbasin drainage areas, curve numbers, lag times, precipitation data, and routing parameters. Also as part of this task, several maps were prepared and submitted, including a topographic/hydrologic base map, a photographic map, and a soil map.

Task 4: Check and, if Needed, Modify Capacity Rating Curve (i.e., Storage/Outflow Curve) for the Dams, Based on the Latest As-Built Plans.

This task involved revisions of the original reservoir rating curves based on a comparison between the construction as-built topographic plans and recent topographic plans. The completion of this task depended on the availability of recent topographic information which was not known at the time the scope of work was developed.

Task 5: Develop Estimates of Peak Discharges and Volumes at Specified Concentration Points.

The HEC-1 models developed for the three drainage basins were run with the hydrologic criteria developed in Task 3 to develop inflow hydrographs for the five flooding events and to route these flood hydrographs past the FRS's. The performance of each FRS for each flooding event was evaluated based on the capability of the structure to pass the flood without using the emergency spillway and without overtopping.

Task 6: Prepare Hydrology Report as Separate and Independent from the Dambreak Report.

As part of Task 6, this report was prepared in draft form for District review. The final report was prepared after receipt of comments from District staff. Also as part of this task, several final submittals were made, including sample input and output from the HEC-1 runs, flexible diskettes containing the input and output HEC-1 files, HEC-1 output file hard copy, and map overlays.

Task 7: Prepare and Submit Monthly Progress Reports to District.

As part of this task, project progress was documented in monthly reports submitted to the District.

COMPLETION SCHEDULE

The original contract requires completion of both elements of the project (i.e., the hydrology and dambreak elements) within 150 calendar days of issuance of Notice to Proceed, exclusive of District review time, which was set at a maximum of 90 days in the contract. Notice to Proceed was received at JMM on March 8, 1989. Thus, project completion was to be achieved by August 5, 1989, exclusive of District review time. Addition of the Powerline model revisions work to the original scope of work resulted in an extension of the completion date to August 31, 1989.

PRIOR STUDIES

As described earlier in this report, several prior hydrologic studies have been completed for the watershed area. These include an SCS study for Weekes Wash (SCS, 1985), which is a main drainage for the Powerline basin; the "Superstition Freeway Comprehensive Offsite Drainage Plan," (A-N West, 1987); the "Final Drainage Study Report, Superstition Freeway," (Tudor, 1987); and the "Hydrologic Analysis of the Powerline F.R.S.," (Flood Control District of Maricopa County, 1989). These reports were reviewed during the hydrologic study, as were other available information, including "Design Computations" for the Vineyard and Rittenhouse FRS's and as-built drawings for the Powerline Floodway (SCS, 1966) and the Vineyard (SCS, 1967) and Rittenhouse (SCS, 1968) FRS's.

The list of references contained in Appendix A in Volume II provides documentation information on these reports and other materials. A more detailed discussion of the contents of selected prior reports used in the study is presented in Chapter 2.

CHAPTER 2

WATERSHED HYDROLOGY

INTRODUCTION

This chapter of the report provides a description of the analyses performed to develop hydrologic models for the Vineyard and Rittenhouse drainage areas and to revise the existing hydrologic model for the Powerline drainage. Included are discussions of the methods and assumptions used to prepare the models.

SOURCES OF INFORMATION

Three sources of information were utilized during development of the models. These sources included prior studies, review guidance from the District staff, and field reconnaissance of the FRS's and lower watershed area.

Prior Studies

As described earlier, portions of the watershed have been studied previously and were discussed in prior reports. Summaries of these reports are provided below.

Weekes Wash Study (SCS, 1985). The purpose of the Weekes Wash Study was to determine if the Powerline FRS would safely pass the design Freeboard Storm, either with or without the construction of a dam on Weekes Wash, at a site approximately six miles upstream of the Powerline FRS. It was concluded that although the probability of overtopping the Powerline FRS would decrease with a dam on Weekes Wash, in either case the FRS would not safely pass the design Freeboard Storm.

The following parameters and results were identified in the study report:

Hydrologic Method: TR-20
Routing Method: Modified Attenuated Kinematic
Precipitation Data Source: NOAA Atlas
Depth-Area-Reduction Factor: None

Land Use: Existing conditions

Peak Inflow to Powerline FRS (24-hr, 100-yr, SCS Type II) without Weekes Wash Dam: 18,279 cfs

Drainage Area: 46.33 sq. mi.

100-yr Peak Flow per Square Mile: 395 csm

It was observed that some of the subbasins selected for the study contained a mixture of rugged mountainous regions and gentler alluvial fan areas, which may affect the accuracy of the corresponding hydrographs. Additionally, one subbasin was comparatively large, having a drainage area of approximately 14 square miles. In performing routing of hydrographs in the SCS study, it was assumed that the bridges and culverts under Highway 60 would pass the design storm without attenuation.

A Hydrologic Analysis of the Powerline FRS (District, 1989). The purposes of this study were to convert the Weekes Wash Study from TR-20 to HEC-1 format and to modify the converted model to reflect certain physical characteristics not addressed in the Weekes Wash Study. These characteristics include the Superstition Freeway and Weekes Wash breakout at the intersection of Weekes Wash and Junction Drive.

As a result of the hydrologic analysis, it was concluded that the Superstition Freeway does not have a significant impact on the Powerline FRS. This was attributed to the fact that the magnitude of flows carried by Weekes Wash was small compared to that carried by the Siphon Draw.

The following parameters and results were identified in the study report:

Hydrologic Method: HEC-1

Routing Method: Muskingum

Precipitation Data Source: NOAA Atlas

Depth-Area-Reduction Factor: None

Land Use: Future conditions

Peak Inflow to Powerline FRS (24-hr, 100-yr, SCS Type II): 17,913 cfs

Drainage Area: 47.10 sq. mi.

100-yr Peak Flow per Square Mile: 304 csm

The subbasins selected for the Weekes Wash Study were utilized by the District to maintain compatibility.

Superstition Freeway Comprehensive Offsite Drainage Plan (A-N West, Inc., 1987). The purpose of this study was to perform offsite hydrology and recommend structures to transport runoff from the 50-year storm under a section of the Superstition Freeway. The 100-year flood was also developed to define floodplain limits and to size detention basins.

The following parameters and results were identified in the study report:

Hydrologic Method: HEC-1

Routing Method: Kinematic Wave (urban areas) and Modified Puls Storage (undeveloped areas)

Precipitation Data Source: Addendum to "Hydrologic Design for Highway Drainage in Arizona"

Depth-Area-Reduction Factor: None

Land Use: Existing conditions

Peak Flow at the Superstition Freeway (24-hr, 100-yr, SCS Type II): 9,831 cfs

Drainage Area: 16.56 sq. mi.

100-yr Peak Inflow per Square Mile: 594 csm

The subbasin areas and corresponding peak flows were not summarized in this study. To obtain the peak flow at the Superstition Freeway, Sheet 3 of Figure 3 in the report was utilized; and peak flows for subbasins 27 through 43 were summed to produce a flow of 9,831 cfs. Contributing drainage areas were identified from Figure 6 in the report, and the drainage areas were taken from the HEC-1 input model of the Superstition Freeway offsite drainage analysis, proposed condition, Highway 60 to the North Diversion Dam (File SUPROE).

The subbasins selected for the Weekes Wash and the District studies were utilized by A-N West. However, the subbasins were truncated at the proposed Superstition Freeway.

Final Drainage Study Report, Superstition Freeway: Ironwood Drive to U.S. 60 (Tudor Engineering Company, 1987). No hydrologic information was contained in this report. The "Superstition Freeway Comprehensive Offsite Drainage Plan" was referenced as the document containing all hydrologic information on which the proposed drainage alternatives were based. That report was discussed above.

District Review Guidance

The second source of information was review guidance from District staff. This guidance consisted of two elements. First, selection of certain methodologies was made by the District in the scope of work. The contract-specified methodologies included use of the HEC-1 model and the SCS unit hydrograph technique, among others. Secondly, the guidance also consisted of reviewing submittals and ongoing review comments provided during progress meetings.

Field Reconnaissance

Field reconnaissance was conducted of the FRS's and lower watershed areas on February 28, 1989. During field reconnaissance, estimation of Mannings "n" values was made, information for use in selection of cross sections was obtained, and vegetative cover in the watershed was observed.

WATERSHED DESCRIPTION

Characteristics of the drainage basins above the Powerline, Vineyard, and Rittenhouse FRS's vary markedly from the steep, rugged upper reaches of the Superstition Mountains to the gently sloping alluvial fans immediately upstream of the FRS's. Vegetative cover is sparse, consisting of desert shrubs and cacti. Typical vegetation found in the non-mountainous watershed areas include palo verde, creosote bush, cholla, saguaro, and grasses.

Topography and Soils

In the upper reaches of the watershed, the ground slopes are as great as 70 percent, with values typically ranging from 20 and 45 percent. When storms occur, runoff initially travels overland, but quickly concentrates in narrow canyons and washes with steep sideslopes. Channels in this area are formed in hard rock and contain little to no alluvial material in the streambed.

In the central portion of the watershed, which is that portion below the mountainous area and upstream of Highway 60, slopes range from 1 to 35 percent, with typical values between 1 and 3 percent. The washes are well-defined, but channel bottoms are wider and sideslopes are less steep than those in the upper reaches of the watershed. The channels in this portion of the

watershed are principally formed on alluvial fans deposited from the Superstition Mountains. The channel material consists of sand- to boulder-sized sediments.

The terrain changes markedly near the location where Highway 60 crosses the watershed. Downstream of Highway 60, a broad floodplain occurs. In this lower section of the watershed, slopes are consistent and fairly mild, ranging from 0.5 to 1 percent, and the washes are shallow and laterally unstable. Silt and sand have been deposited in the channel bottoms, which are formed in the finer sand and gravel alluvial deposits from the Superstition Mountains.

Soils in the drainage area generally consist of gravelly sandy loans. A description of soil properties related to infiltration characteristics is provided later in this chapter.

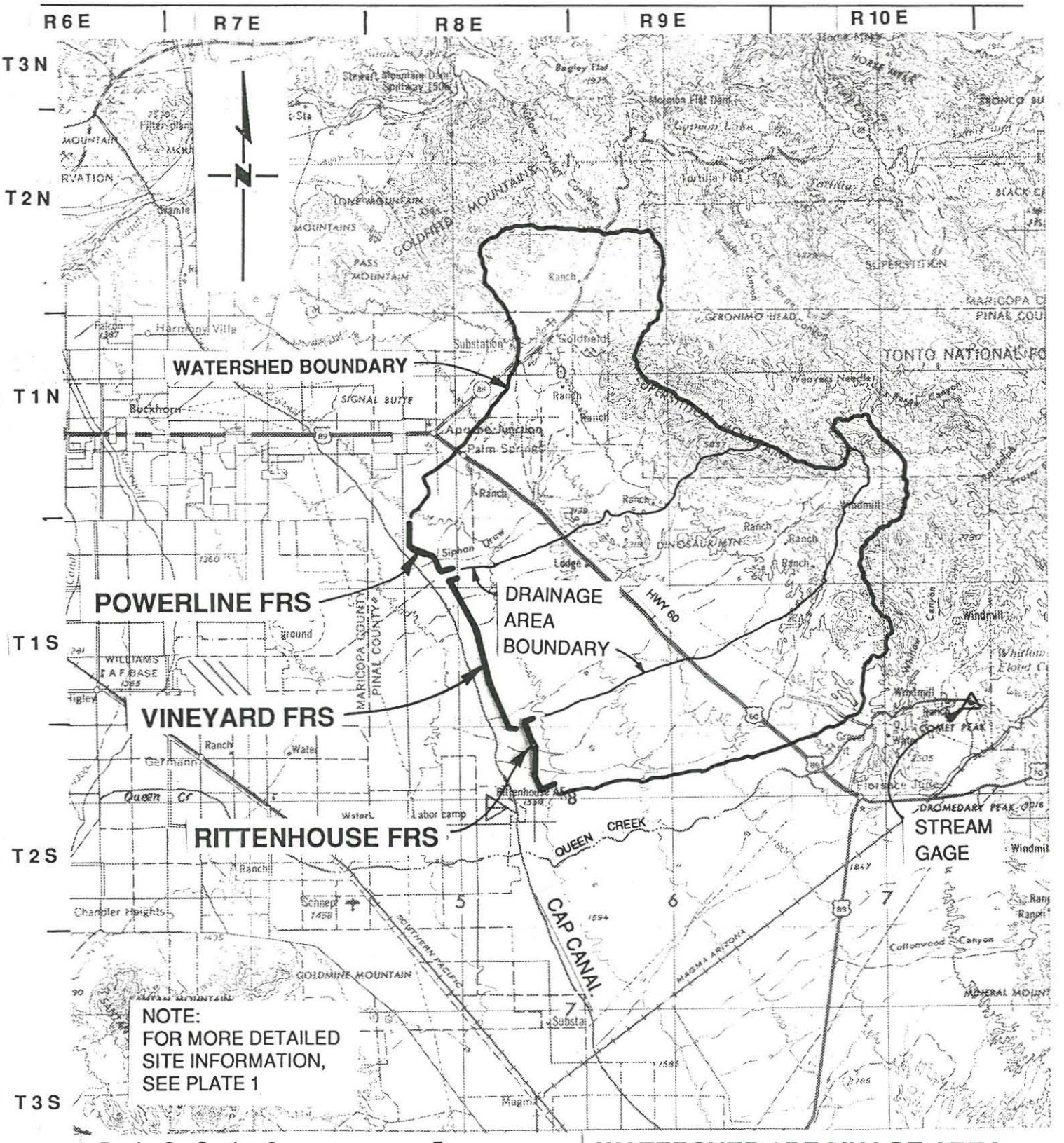
Flood Retarding Structures

Runoff from the watershed is captured by the three adjacent FRS's. The watershed area of 146.9 square miles has been subdivided in this study into three drainage basins, which are delineated by that which contributes runoff to the three FRS's. Figure 2-1 presents the relative location of the FRS's and their respective drainage basins.

Construction of the three FRS's was sponsored by the Soil Conservation Service. Table 2-1 contains descriptions of the physical features of each structure, including their drainage area.

Stored water can be released from each of the FRS's through several different facilities, including outlet pipes and an emergency spillway. Some of these facilities are interrelated in that they share common discharge conveyance works or empty into the reservoir behind an adjacent FRS.

The outlet facilities at each FRS consist of gated pipes; an ungated pipe, termed the principal spillway; and an emergency spillway. The emergency spillway at each FRS is an unlined earthen berm, which serves as a weir over which reservoir water can be released. The emergency spillway empties into an unlined, earthen, trapezoidal channel.



5 4 3 2 1 0 5

SCALE IN MILES

WATERSHED / DRAINAGE AREA MAP

FIGURE 2 - 1

JMM

TABLE 2-1

PHYSICAL FEATURES

Powerline, Vineyard, and Rittenhouse FRS's

Project Features	Powerline	Vineyard	Rittenhouse
Drainage Area (sq. mi.)	47.1	52.1	47.7
Type of Structure	Compacted Earthfill	Compacted Earthfill	Compacted Earthfill
Top of Structure Elevation (ft.)	1589.1	1579.5	1602.3
Length of Structure	13,358 l.ft.	28,829 l.ft.	3.6 mi.
Maximum Height (ft.)	21	16.5	24.3
Top Crest Width (ft.)	14	14	14
Emergency Spillway Crest Elevation (ft.)	1583.3	1574.8	1597.6
Emergency Spillway Capacity (cfs)	16,600	12,800	12,800
Storage Capacity (ac-ft)	4,194	4,310	4,060
Principal Spillway Discharge Rate (cfs)	203	368	143
Total Spillway Capacity	16,803	13,168	12,943
Principal Outlet Structure	36" RCP (ungated)	54" RCP (ungated)	33" RCP (ungated)

Notes:

1. All data are from as-builts and other documentation provided by the District, except as described below.
2. Spillway capacities for Vineyard and Rittenhouse are based on a one-foot freeboard.
3. Principal and emergency spillway capacities for Powerline FRS are estimated from HEC-1 model input data prepared by District.
4. Principal and emergency spillway capacities for Vineyard and Rittenhouse FRS's are calculated based on orifice and weir coefficients utilized in HEC-1 models developed in this study.
5. Total spillway capacity does not include gated outlets.

At the Powerline and Vineyard FRS's, the earthen channels that serve the principal and emergency spillways empty into a common outlet canal, which drains toward the CAP canal. At the CAP canal, a concrete channel section (i.e., CAP overchute) conveys flow across the CAP canal and into the Powerline Floodway. At the Rittenhouse FRS emergency spillway, flow is conveyed around the southern end of the FRS toward the CAP canal. This flow will pond behind the CAP canal because an overchute channel is not provided at this site. Flow from the

Rittenhouse principal spillway empties into the reservoir behind Vineyard FRS, where it is released through one of the outlet facilities at Vineyard.

FLOODING HISTORY

During investigation of flooding history for the three FRS's, it was found that little direct historical flooding information exists. The relevant flooding information that was uncovered consists of an oblique aerial photograph showing water contained within the reservoirs and streamgage records for Queen Creek. An SCS hydrology study was performed on the 1954 flooding event in the watershed; however, this study, which served as the basis for the design of the three FRS's, is not retrievable from SCS file storage.

Flooding Photograph

The aerial photograph of the three FRS's showing flood water stored in the reservoirs was taken on October 20, 1972. Peak reservoir inflows or volumes are not available for this flood; however, an estimate of water in storage for the Powerline FRS was made based on the photograph. By estimating the length of the stored water along the south wingwall of the FRS, an approximate storage volume of 2,100 acre-feet was calculated. Total storage at the Powerline FRS is 4,194 acre-feet; thus, the reservoir at the time the photograph was taken was at about one-half capacity. It is not known if the photograph was taken at the time of maximum storage.

Queen Creek Streamflow Record

The Queen Creek watershed, which is immediately adjacent to the southern boundary of the Rittenhouse drainage basin (see Figure 2-1), is similar in many respects to the watershed draining the three FRS's. A streamgage was maintained on Queen Creek at the Whitlow damsite, as located on Figure 2-1, intermittently for 17 years, including the years 1917 through 1920, 1939, and 1948 through 1959. The drainage area above the gage is 144 square miles. The peak discharge recorded at the gage was 42,900 cubic feet per second (cfs) in 1954.

A flow frequency analysis by log-Pearson Type III was performed of the annual maximum flows recorded at the Whitlow damsite gage. The results of this analysis are presented in Table 2-2.

TABLE 2-2

FLOW-FREQUENCY ANALYSIS
QUEEN CREEK AT WHITLOW DAMSITE GAGE

Log-Pearson Type III
(17 Annual Maximums)

Flooding Event	Estimated Discharge		Confidence Limits			
			0.05 Limit		0.95 Limit	
	(cfs)	(csm)	(cfs)	(csm)	(cfs)	(csm)
10-year	19,000	132	43,500	302	11,000	76
25-year	33,800	235	93,500	649	18,000	125
50-year	49,200	342	156,000	1,083	24,600	171
100-year	69,200	481	248,000	1,722	32,400	225
500-year	139,000	965	648,000	4,500	57,000	396

As indicated in Table 2-2, there is a wide range in the confidence limits for the individual estimated discharges. For example, the 90 percent confidence interval for the 100-year flood is 225 to 1,722 cfs per square mile (csm).

It has been demonstrated that arid west hydrology is characterized by highly fluctuating streamflow, including some years having zero flow and some years having high peak flows. This characteristic increases the uncertainty in hydrologic statistics generated from arid west streamflow records, especially for records of short duration.

A recent paper (Paulson and Duren, 1989) documented the impact of changing lengths of streamflow record on the computed statistics for streams in the arid west. This paper demonstrated that, for 18 arid west streamgages having records from 17 to 69 years in length, the 100-year peak flow estimate changed by an average of 32 percent when 10 years of data were subtracted from the period of record. The paper also showed that the largest changes in estimated discharges typically occurred at gages having 25 years of record or less, where changes in the estimated 100-year peak flow were found to change by as much as 66 percent.

Based on the above discussions, it can be concluded that there is little direct quantitative flooding history record that can be used to compare with the discharges generated in this study for the drainage areas above the three FRS's. The direct watershed information consists of an aerial photograph of the reservoir during a flooding event and an adjacent streamgage record of such short duration that the peak flow estimates have a low level of certainty.

SUBBASIN SELECTIONS

In preparing input for the HEC-1 model, the drainage areas above the Vineyard and Rittenhouse FRS's were subdivided into numerous subbasins as shown on Plate 1, at the back of this report. For the Powerline FRS, the subbasins developed in the earlier studies were utilized.

The selection of subbasin boundaries for the Vineyard and Rittenhouse drainage basins was based on the following criteria:

1. Where hydrologically and hydraulically feasible, each subbasin area was limited to a maximum size of four square miles in order to ensure that the hydrologic model adequately represented the drainage areas. (In a few instances it was necessary to exceed the four-square-mile criterion due to the physical arrangement of the washes.)

2. Areas of expected development were isolated for ease in future updating of the model.

3. Areas containing mountainous terrain were separated from those displaying alluvial fan characteristics.

4. Highway 60 was established as a subbasin boundary to allow for routing of hydrographs across the highway, thereby accounting for the effects of ponding behind the highway embankment.

HYDROLOGIC CRITERIA

In conformance with the contract scope of work, HEC-1 hydrologic models, utilized with the SCS unit hydrograph and loss rate methods, were prepared for the Vineyard and Rittenhouse drainage basins. For the Powerline basin, the existing HEC-1 model was revised, as previously indicated. The IBM 512K version (revised February 1985) of HEC-1 was utilized and run on an IBM PS-2 Model 50Z. Computer diskettes containing both input and output files for all runs were provided to the District for future use. A hard copy of the output files for the five flood analyses for the Powerline, Vineyard, and Rittenhouse FRS drainage basins is found in Appendix B.

Runoff from the 25-, 50-, and 100-year, 24-hour storms and from the 6-hour and 72-hour probable maximum precipitation (PMP) was generated using the new HEC-1 model for both the Vineyard and Rittenhouse drainage basins. The 25- and 50-year events were analyzed for the Powerline basin after modifying the District hydrologic model to adjust curve numbers from future watershed conditions to current conditions. New precipitation data for these storms was input into the existing model. The 100-year and probable maximum flood (PMF) analyses for Powerline were also generated based on current conditions and after also revising the model precipitation data to be consistent with that used for the Vineyard and Rittenhouse model.

The hydrograph computation interval for all storms analyzed in the Vineyard and Rittenhouse HEC-1 model was 5 minutes, except for the 72-hour PMP. The computation time interval for the 72-hour PMP was increased to 15 minutes in order to model the watershed through the duration of the storm. The increase in the computation time interval violated model stability criteria in approximately half of the subbasins; however, it was found from review of the HEC-1 output that 5- and 10-minute intervals caused the hydrograph to be shortened to a dimensioning limitation such that an unacceptable hydrograph length past the peak period was obtained. Thus, a longer computation duration was necessary.

In the existing Powerline model, the hydrograph computation interval was 10 minutes for the 25-, 50-, and 100-year floods. For the 6-hour PMP, a five-minute interval was used, while a 15-minute interval was used for the 72-hour PMP.

The following sections of this chapter describe the development of input data for the Vineyard and Rittenhouse HEC-1 hydrologic model, including (1) precipitation, (2) lag times, (3) loss rates, and (4) routing coefficients. In these descriptions the methodologies and assumptions that were utilized are discussed.

Calculation sheets showing derivation of selected hydrologic criteria (e.g., PMP, curve numbers, times of concentrations, and lag times) are presented in Appendices C and D. For more detailed information on methodologies and procedures than found in this chapter of the report, the Appendices should be reviewed.

Precipitation

Two different types of precipitation data were required in order to develop runoff hydrographs for the design storms. These types of data are described in the following paragraphs.

Return-Interval Precipitation. Total precipitation values for the 25-, 50-, and 100-year, 24-hour duration storm events were estimated from the "NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume VIII - Arizona" (NWS, 1973). In developing the total precipitation amounts for the three storms, it was assumed that the storm covered the entire watershed of 146.9 square miles. This area was plotted on the appropriate isopluvial maps within the NOAA Atlas. Due to the size of the watershed, several isopluvials were intersected. Therefore, for each storm a weighted precipitation was calculated based on the area within the watershed covered by each isopluvial.

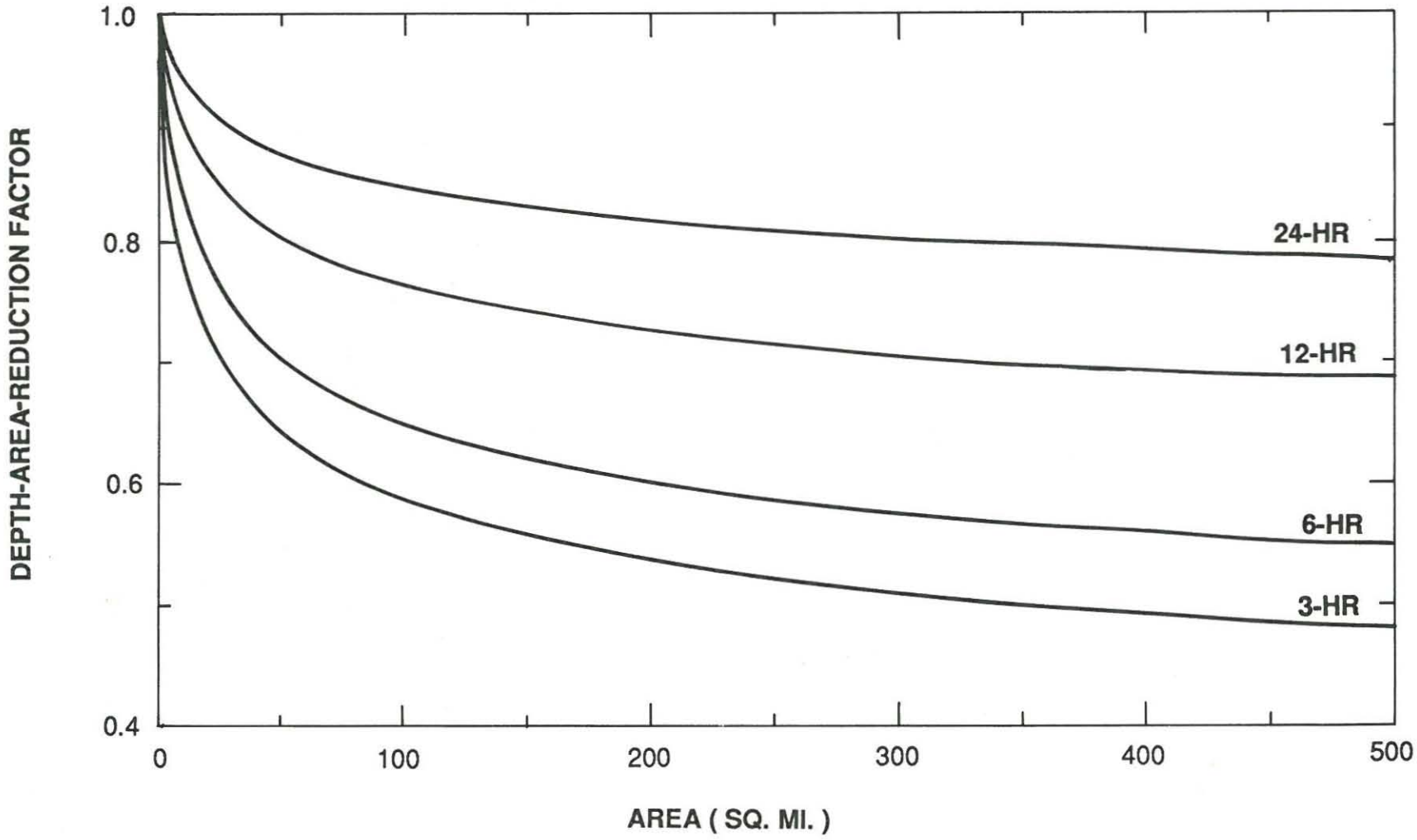
The precipitation amounts for each storm as determined from the NOAA Atlas are shown in Table 2-3.

TABLE 2-3

UNADJUSTED STORM PRECIPITATION TOTALS
WATERSHED ABOVE POWERLINE, VINEYARD, AND RITTENHOUSE FRS'S
NOAA ATLAS 2

Frequency/Duration	Unadjusted Precipitation (in)
25-year, 24-hour	3.29
50-year, 24-hour	3.76
100-year, 24-hour	4.14

The precipitation amounts in Table 2-3 are "unadjusted" values since they have not yet been modified to account for watershed size. To perform this adjustment, the NOAA Technical Memorandum, NWS Hydro-40 (NWS, 1984), was utilized to develop depth-area-reduction factors (DARF's). The Hydro-40 method was developed to extend the Osborn method to include a 24-hour duration storm, extend DARF relationships to areas larger than 79.5 square miles, and define regions over which the Osborn methods apply. The appropriate DARF's were selected from Figure 15 of Hydro-40, which is shown as Figure 2-2 in this report. Table 2-4 presents the DARF's obtained from Hydro-40 for this study. A maximum drainage area of 53 square miles was selected because none of the concentration points in the watershed network has a larger tributary area.



DEPTH-AREA-REDUCTION CURVES (HYDRO-40)

FIGURE 2 - 2

JMM

TABLE 2-4

DRAINAGE-AREA-REDUCTION FACTORS

Drainage Area (sq. mi.)	24-Hour DARF
0.26	1.00
5.0	0.98
15.0	0.93
30.0	0.90
53.0	0.87

Using the DARF's presented in Table 2-4, the unadjusted storm precipitation amounts shown in Table 2-3 were converted to appropriate storm precipitation for the tributary area above each concentration point. The depth-area-storm option within HEC-1 was used to accomplish this task. The program computes hydrographs for individual subbasins utilizing the table of reduction factors. As subbasins are subsequently combined downstream, the combined hydrograph is recomputed with a new DARF corresponding to the increased drainage area.

Probable Maximum Precipitation (PMP). Two PMP events were evaluated in this study, as prescribed in the contract scope of work. These were the 6- and 72-hour storms. A general storm analysis was performed for the 72-hour PMP, and a local storm analysis was performed for the 6-hour PMP.

The general storm PMP was computed for the watershed for several different months. The intent was to determine the month having the maximum precipitation contained in the 6- and 72-hour intervals of the storm. Utilizing Figures 2.11, 2.12, and 2.13 of Hydrometeorological Report No. 49 (HMR 49), (NWS, 1979), the months of July, August, and September were selected because they displayed the highest 72-hour convergence values.

The results of the general storm PMP computation for the 6- and 72-hour events are shown in Table 2-5.

TABLE 2-5

GENERAL STORM PMP VALUES

Month	Convergence and Orthographic PMP 6-hr. Interval	72-hr. Interval
July	7.6 in	15.5 in
August	8.7 in	17.8 in
September	8.6 in	17.6 in

Based upon the above results, the general storm PMP for the month of August was selected to represent the 72-hour flood event.

Utilizing HMR 49's procedures for local storm analysis, the local storm 6-hour PMP was determined to be 7.6 inches.

Precipitation Distribution. The precipitation for the 25-, 50-, and 100-year events was distributed within the HEC-1 model according to the SCS Type II distribution. The computation time interval was chosen as five minutes, which enabled hydrograph computation for the duration of the storm without compromising the computational stability of the model.

The 6-hour PMP was distributed hourly according to the procedures outlined in HMR 49, Table 4.7. The hourly precipitation was further distributed evenly into 15-minute increments with the exception of the maximum hourly, which was distributed according to HMR 49, Table 4.8.

The 72-hour PMP precipitation was distributed hourly by first plotting a depth-duration curve from the 6-, 18-, 36-, 48-, and 72-hour cumulative precipitation values calculated from HMR 49 procedures. Cumulative precipitation was then interpolated from the depth-duration curve for each hourly increment. The hourly precipitation distribution was subsequently obtained using the U.S. Bureau of Reclamation procedure described in its Flood Hydrology Manual (USBR, 1989). In this procedure the maximum hourly precipitation is placed at two-thirds the storm duration (i.e., the forty-eighth hour for the 72-hour storm).

Calculation sheets for the 25-, 50-, and 100-year precipitation and for the PMP analyses are presented in Appendix C. The step-by-step procedure for determining PMP amounts and distributions are provided therein.

SCS Lag

The lag time, per SCS methods, was computed for each subbasin on the basis of a relationship to subbasin time of concentration. Time of concentration, T_c, is defined as the runoff travel time from the hydrologically most distant point of the subbasin to the point of concentration at the subbasin outlet. The methods presented in TR-55 (SCS, 1986) were utilized to calculate T_c. Three methods are outlined in TR-55, and the use of each is based on watershed characteristics.

For the watershed above the three FRS's, only two of the SCS methods for computing T_c were applicable. One method (i.e., Method No. 1 for sheet flow) was limited to flow paths of less than 300 feet; and there were no flow paths this short in the watershed model. Therefore, this method was not applicable in the analysis.

Model No. 2 is for shallow concentrated flow. This method was used for flow paths not in defined washes and involved calculation of the average slope. For a given channel slope, velocity was estimated by a nomograph provided in TR-55. Using the nomograph velocity and measured flow path, T_c was computed for each reach as:

$$T_c \text{ (hrs)} = \frac{L}{3600\,V}$$

where:

L = measured flow path length
V = nomograph velocity

Method No. 3, which applies to open channels, was developed for defined washes such as those identified in blue on the USGS topographic maps. This method is based on determining flow velocity by Manning's equation for open channel flow, assuming full bank flow conditions. In determining bank flow conditions, the bank widths and depths were not obtainable from the available USGS topo maps due to the contour interval being larger than wash depths. Therefore, assumptions were made based on expected depths and channel sections for the steep mountainous regions and based on field reconnaissance for the washes in the middle and lower regions of the watershed.

The Mannings "n" coefficient of roughness in channels where the slope did not exceed 0.04 ft/ft was estimated utilizing a nomograph found in a USGS report (USGS, 1984). In channels were the slope exceeded 0.04 ft/ft, an "n" value of 0.04 was assumed.

Critical velocities based on the flow capacity of the assumed channel cross sections and computed slopes were calculated and used as a limiting value in developing T_c estimates. This procedure was based on engineering judgement. In a few reaches, the computed channel velocities exceeded critical velocity, indicating supercritical flow. In these reaches, the critical velocity was used to calculate T_c.

T_c was then calculated from the velocity and length of flow path relationship defined previously in Method No. 2 of calculation. T_c was later converted to SCS lag by the following relationship:

$$\text{SCS lag} = 0.6\ T_c$$

Calculations of the times of concentration and lag times are presented in Appendix D.

Loss Rates

The SCS Curve Number method was utilized to estimate loss rates, based on TR-55 suggested curve numbers. Curve numbers for undeveloped areas were taken from the TR-55 chart for arid and semiarid rangelands, with a cover type of desert shrub with less than 30% ground cover.

Two subbasins in the Vineyard drainage area, V4 and V8, contain developed areas. Rather than increase the associated curve numbers to reflect the impervious areas, the percent of impervious area within each subbasin was estimated and incorporated into the loss rate portion of the HEC-1 model. As a result, no losses were computed for the impervious portions of these subbasins.

The antecedent moisture condition (AMC) was assumed to be II for the 25-, 50- and 100-year events and III for the PMF. The higher AMC for the PMF was used to be consistent with the severity of the PMF event. Selected curve numbers by hydrologic soil group are as shown in Table 2-6.

TABLE 2-6

SELECTED SCS CURVE NUMBERS

Hydrologic Soil Group	Curve Numbers AMC II	AMC III
A	63	81
B	77	92
C	85	97
D	88	98

The "Soil Survey of Aguila-Carefree Area, Parts of Maricopa and Pinal Counties, Arizona, " (SCS, 1986) was obtained from the SCS, and soil types by hydrologic group were delineated on an overlay submitted to the District as a study product. This detailed soil survey covered a major portion of the Powerline drainage basin, but only a minor portion of the Vineyard basin and none of the Rittenhouse basin was covered.

The Arizona General Soil Map (USDA, U of A) was obtained in an effort to establish hydrologic soil groups of the areas outside of the detailed soil survey boundary. However, the soil classifications identified on the map were not of sufficient detail to obtain hydrologic soil groups. Therefore, soil groups were extrapolated from the Gila-Carefree Soil Survey and trends were compared to those within the survey boundary. Generally, Group D soils were used for the mountainous regions and Groups B and C were assumed for the alluvial fan areas. For the largest washes which also contained denser vegetation, Group A soils were assumed.

Utilizing the existing soil survey and soil zone extrapolations described above, curve numbers were calculated for each subbasin based on the area of the subbasin that is covered by a specific soil zone and curve numbers presented in Table 2-6. Curve numbers used for each subbasin within the Vineyard and Rittenhouse drainage basins are presented in Appendix D.

The existing HEC-1 model for the Powerline drainage area was based on anticipated future watershed conditions. To obtain consistency with the hydrologic model for the Vineyard and Rittenhouse model, the curve numbers in the existing Powerline model were revised to reflect current watershed conditions. Calculations of the revised curve numbers is presented in Appendix E.

Routing

Two routing techniques were utilized to compute downstream hydrographs. Channel routing techniques were utilized in reaches where storage was not a factor, while storage routing techniques were used for reservoir routing.

Routing schematics for the Vineyard and Rittenhouse basins are presented in Figures 2-3 and 2-4. Standard District symbols are used in the schematics to describe the routing operations. The interconnected principal spillway from the Rittenhouse FRS to the Vineyard FRS is depicted in the routing schematic at the bottom of the figures.

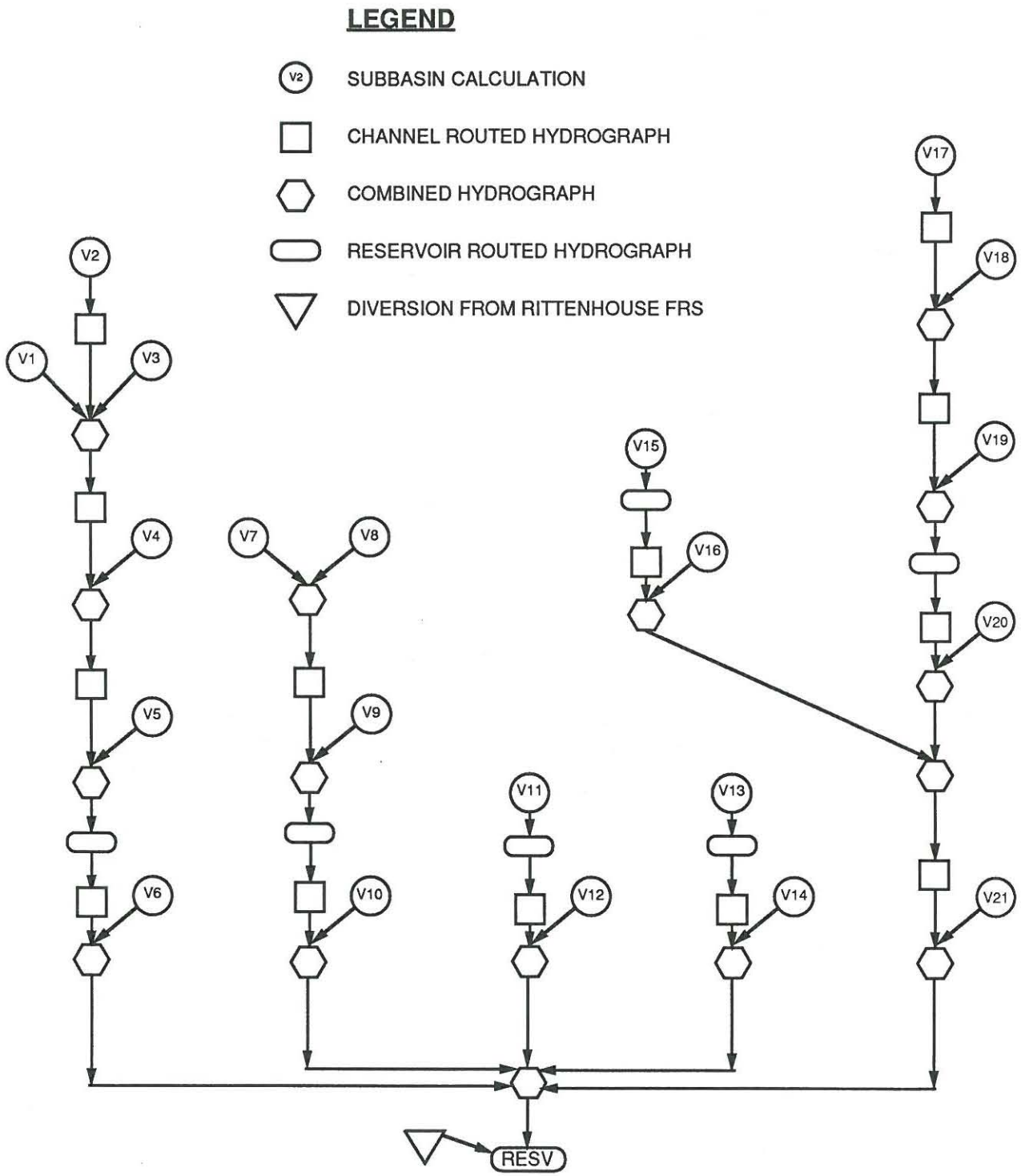
Channel Routing. The Muskingum method within HEC-1 was utilized for channel routing as specified in the contract scope. This method was selected at the request of the District to maintain consistency with the methodology of the Powerline basin hydrologic model.

Two parameters are used to simulate routing by the Muskingum method. The first, the Muskingum "X" coefficient, is a measure of storage. The limits of this coefficient are 0.0 for a reservoir condition and 0.5 for minimal flow attenuation. A value of 0.3 was selected to be consistent with the District's HEC-1 model of the Powerline basin.

The second parameter, Muskingum "K", is an estimate of travel time within the channel. To identify this parameter, Method No. 3 previously outlined for calculating T_c was utilized to estimate channel velocity and subsequently determine the travel time along the measured flow paths. As above, the limiting channel velocity was assumed to be the critical velocity of the channel.

Computed values for "X" and "K" for each reach are provided in Appendix F.

Highway 60 Routing. U.S. Highway 60 crosses the drainage basins of Powerline, Vineyard, and Rittenhouse FRS's approximately 3 to 6 miles upstream of these structures. In developing the hydrologic models for the Vineyard and Rittenhouse basins, the highway embankment was modeled as a dam; and reservoir routing was performed to develop hydrographs downstream of the highway. The results of the routing were reviewed to determine if the embankment caused a significant attenuation in the hydrograph peak flow.



HEC - 1 ROUTING SCHEMATIC FOR VINEYARD BASIN
FIGURE 2-3

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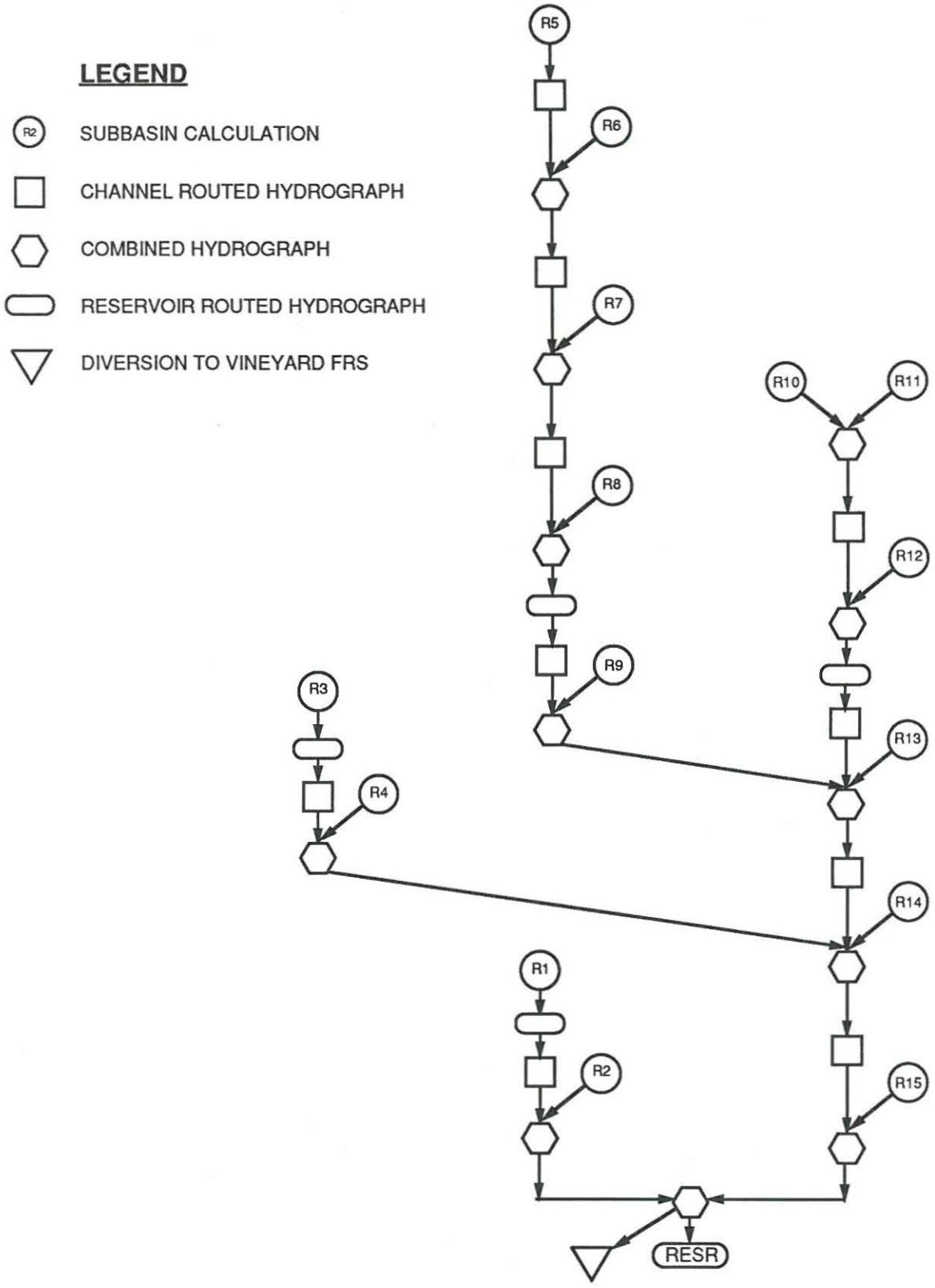

HEC - 1 ROUTING SCHEMATIC FOR RITTENHOUSE BASIN

FIGURE 2-4

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The procedure utilized in this analysis for the highway routing involved computing a rating curve for each "reservoir" behind the highway embankment. A reservoir was modeled at the lower concentration point of each subbasin that ended at the upstream side of the highway embankment. The rating curve was computed by calculating flows through the culverts and bridges and over the top of the road. Because there are typically numerous culverts or bridges located within the limits of the subbasin boundaries at the highway, a composite bridge/culvert outlet section was developed for each subbasin. A table of elevation versus outflow from each reservoir was thus computed, based on the composite culvert and top of road elevations, and input into HEC-1.

A listing of the drainage structures crossing Highway 60 is presented in Appendix G.

Bridge/Culvert Outflows. In preparing a composite bridge/culvert section for each subbasin, a factor was developed for converting the sum of the individual bridge/culvert open areas into a composite bridge/culvert open area. As shown on the calculation sheets contained in Appendix H, a factor of 92.5 percent was developed for converting the sum of the individual bridge/culvert open areas to a composite open area. The composite culvert was calculated to have the same flow capacity as the sum of the individual culverts and bridges.

Invert and crown elevations of the composite culvert were calculated as the weighted averages (by open area) of the respective elevations for the individual bridges and culverts located within the subbasin. Using the composite area of the culvert, a width was computed for each culvert. Open-channel and orifice flow calculations were utilized to develop the rating curve for flow through the composite culvert. The calculations for computing composite culvert sections at each subbasin are shown in Appendix I, as is a sample calculation of the composite culvert rating curve for subbasin V10A.

Weir Flow Over Top of Road. The top of road elevation at the composite culvert was set at 2.0 feet above the culvert crown elevation. At this elevation, weir flow was assumed to begin to occur across the highway. A horizontal top of dam (i.e., road) was assumed, and the width of this weir was calculated on the basis of the road profile within the subbasin limits. For computing weir flow at higher water surface elevations, the width of the weir was extended as dictated by the road profile. A sample calculation of the rating curve for weir flow for subbasin V10A is shown on the same calculation sheets in Appendix I.

Reservoir Routing. Routing of the five inflow flood hydrographs (i.e., the 25-, 50-, and 100-year events and the 6-hour and 72-hour PMF) was performed at the three FRS's.

Routing criteria were developed for the Vineyard and Rittenhouse FRS's from as-built drawings for the two structures. Only the principal spillway was considered to convey flow through the dams as a low-level outlet. There are several gated outlets through both dams; however, per District instructions, no flow was assumed to be conveyed by these outlets. The emergency spillway and top of dam data from the as-built drawings were utilized to provide the remaining input.

In routing the inflow hydrographs for the 25-, 50-, and 100-year floods through the three FRS's, the reservoirs were assumed to be empty. For routing the PMF, the reservoir water level was assumed to be at the elevation of the emergency spillway. These routing criteria were developed based on discussions with District staff.

As described earlier in Chapter 1, the contract scope of work included a task to develop a new rating curve for the three FRS's. At the time of scope development, it was not known if current topographic information was available to use as a basis for preparing a revised rating curve. Upon investigation during this study, no new topographic information was found with which to complete this task. Consequently, revised rating curves for the three FRS's were not able to be prepared.

CHAPTER 3

FLOOD HYDROGRAPH GENERATION

INTRODUCTION

This chapter describes the results of the hydrologic analyses conducted of the drainage basins above the Powerline, Vineyard, and Rittenhouse FRS's. It presents the peak flows for each of the storms studied, which includes the five floods at each FRS (i.e., the 25-, 50-, and 100-year events and the 6-hour and 72-hour PMF). Also presented in this chapter are dam performance evaluations for the inflow floods at each structure.

PEAK FLOWS

The peak flows for the inflow floods are presented in Table 3-1. All floods represent existing watershed conditions and consistent precipitation amounts.

TABLE 3-1

PEAK FLOWS
INFLOW FLOOD HYDROGRAPHS
POWERLINE, VINEYARD, AND RITTENHOUSE FRS'S

Flooding Event	Flood Retarding Structure					
	Powerline Peak Flow		Vineyard Peak Flow		Rittenhouse Peak Flow	
	(cfs)	(csm)	(cfs)	(csm)	(cfs)	(csm)
25-year	9,101	193	8,895	171	6,669	140
50-year	11,534	245	11,057	212	8,438	177
100-year	12,073	256	12,843	247	9,925	208
PMF						
6-hour	62,044	1,317	59,484	1,142	46,895	983
72-hour	53,807	1,142	57,349	1,101	47,190	989

As can be determined from Table 3-1, the unit peak flows for the Powerline drainage area are slightly greater than those for the Vineyard and Rittenhouse drainages. This occurrence is expected since there is more developed area within the Powerline drainage, which results in decreased loss rates for Powerline compared to the other two drainages.

Table 3-1 also shows that the peak flows for the 6-hour PMF storms are greater than those for the 72-hour storms for the Powerline and Vineyard drainages. For the Vineyard drainage, the 6-hour event peak flow is only slightly larger (i.e., 4 percent) than the peak flow for the 72-hour event. However, due to the greater impervious area within the Powerline drainage, the shorter, 6-hour storm on Powerline produces a greater relative peak flow, amounting to a 15 percent increase over the 72-hour event. On the Rittenhouse drainage the 72-hour peak flow is slightly greater (i.e., 1 percent) than the 6-hour peak flow. The occurrence of a slightly larger 72-hour peak flow on Rittenhouse, compared to a larger 6-hour peak for Vineyard, could be attributed to the watershed shape. Based on these findings, it can be concluded that the peak flows for the 6-hour and 72-hour PMF events for watersheds of this size are nearly equivalent. Differences in the relative magnitude of the peak flows are due to watershed cover and, possibly, watershed shape.

The csm values presented in Table 3-1 can also be compared to those calculated for the adjacent Queen Creek streamgage. As can be determined from review of Table 2-2 in Chapter 2 of this report, all but one of the unit discharges developed in this study are within, but toward the lower end of, the 90 percent confidence limits of the respective flooding events for the Whitlow Damsite gage on Queen Creek. The 100-year unit peak discharge of 208 csm for the Rittenhouse drainage is slightly less than the 225 csm value at the lower end of the confidence limit for this flooding event. However, due to the short length of record for the Queen Creek gage, the peak discharges developed in this study cannot be considered inappropriate due to the uncertain accuracy of the flow frequency analysis of the Queen Creek streamflow record, which results from its short duration.

In performing the routing of flow across U.S. Highway 60, it was found that negligible attenuation of peak flows occurred. The reduction in peak flows routed across the highway ranged from 1 to 10 percent, with most flow reductions being between 1 and 3 percent. These reductions were determined to have an insignificant effect on the reservoir inflow hydrographs.

DAM PERFORMANCE

The HEC-1 output for the floods analyzed in this study revealed that the 25-, 50-, and 100-year floods could be passed through the Vineyard and Rittenhouse reservoirs without causing flow over the emergency spillway or top of dam. However, the 100-year flood at the Powerline FRS would cause flow to occur over the emergency spillway. For the PMF events at Powerline, Vineyard, and Rittenhouse, flow was found to occur across the emergency spillway and the top of dam.

CHAPTER 4

SUMMARY

This chapter of the report presents a summary of the information contained in the previous chapters. More detailed discussions of the results and analyses undertaken during the study can be obtained by reviewing the appropriate preceding chapter.

ANALYSES UNDERTAKEN

Hydrologic models, using the HEC-1 computer program, were prepared for the drainage areas above the Vineyard and Rittenhouse FRS's. In accordance with the project contract, the SCS unit hydrograph and loss rate criteria were utilized. The Muskingum routing method was used for all channel routing, as also specified by the contract. Storage routing was used to route flows across Highway 60 and through the reservoirs behind the FRS's. These models were used to develop runoff hydrographs for the 25-, 50-, and 100-year floods and the PMF.

An existing HEC-1 hydrologic model for the drainage area above the Powerline FRS was revised and used to develop flood hydrographs for the five floods events for this watershed. A prior study (District, 1989) provided the HEC-1 model used for estimating the flood events for the Powerline drainage.

RESULTS OF ANALYSES

Peak discharges for the specified flooding events are presented in Table 3-1. These values are reasonably consistent with streamgage records for the nearby Whitlow Damsite gage on Queen Creek. Both the Vineyard and Rittenhouse FRS's were able to pass the all floods except the PMF without using the emergency spillway and the top of dam.

The 25- and 50-year floods developed in this study for the Powerline FRS were passed without using the emergency spillway. However for the Powerline FRS, the 100-year flood would result in flow being passed over the emergency spillway.

Both the 6-hour and 72-hour PMF's would produce flow across the emergency spillway and over the top of dam at all three FRS's.