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# FINAL

# HYDROLOGY REPORT

# OUTER LOOP FREEWAY

# NORTH OF THE CAP AQUEDUCT

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FINAL

HYDROLOGY REPORT

OUTER LOOP FREEWAY

NORTH OF THE CAP AQUEDUCT

FLOOD CONTROL DISTRICT
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Submitted to:

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April 1987

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# I. INTRODUCTION

The purpose of this report is to document the assumptions and methodologies used to develop the hydrology for the Concept Drainage Plan for an approximate 8.2 mile reach of the Outer Loop Freeway located north of the Central Arizona Project Aqueduct (CAP).

The following sections of this report present a discussion of drainage area characteristics, meteorological conditions, and the results of the hydrologic modeling process that was undertaken to develop the rainfall/runoff data required for the design of a cross-drainage system for this reach of the freeway.

## II. DRAINAGE BASIN CHARACTERISTICS

### 2.1 Drainage Area

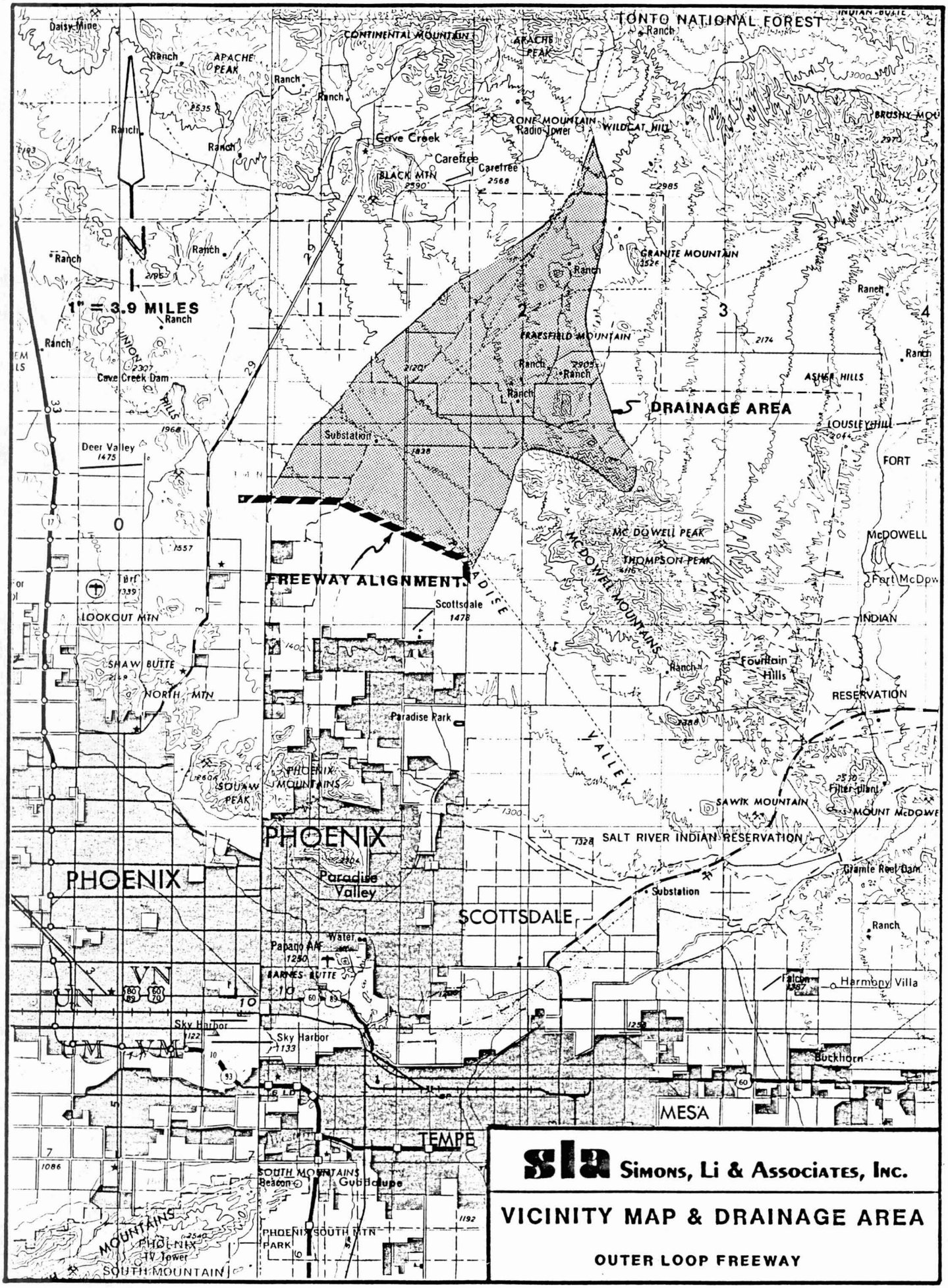
Figure 2.1 presents a vicinity map showing the preliminary freeway alignment and the perimeter of the drainage area contributing runoff to this segment of the freeway. The total contributing drainage area shown on Figure 2.1 is approximately 60 square miles.

Although the basin includes some abrupt mountainous terrain, with peak elevations of about 3,900 feet MSL, the average topographic relief in the watershed ranges from approximately 1,600-feet MSL near the proposed freeway alignment to about 3,100-feet MSL at the northern end of the watershed. Excluding the mountain peaks, the basin exhibits a gradual increase in slope from 0.014 ft/ft near the freeway to approximately 0.030 ft/ft at the upper end of the watershed.

The lower half of the drainage area, which encompasses the proposed freeway alignment, consists of a large alluvial fan type formation which is referred to as a fan terrace in a recently published SCS soil survey. A fan terrace is defined as an inactive remnant of an old alluvial fan, which is no longer a site of active deposition. This type of landform is difficult to analyze from a hydrologic perspective because of the absence of well-defined channels and the transitory flow patterns across the fan. Along the proposed freeway alignment, there are literally hundreds of small, braided washes which intersect the alignment at a nearly perpendicular angle. The flow characteristics across the fan can best be described as exhibiting a wide, shallow, sheetflow pattern rather than the riverine pattern that is descriptive of a more well-defined watercourse such as nearby Cave Creek. The channel patterns and inundation zones on a fan can be very unstable because of limited channel capacity and alterations to channel geometry that often accompany the rapid erosion and sediment deposition processes that occur during flash floods.

### 2.2 Land Use

The vast majority of the watershed is presently undeveloped. Plate 1 is an aerial photograph of the project area which shows the degree of development existing as of December 1985. This level of development is not considered to

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**VICINITY MAP & DRAINAGE AREA**

**OUTER LOOP FREEWAY**

**FIGURE 2.1**

be of a high enough density to alter the hydrologic response of the watershed (along the proposed freeway alignment) from that expected under totally undeveloped land use conditions.

It is important to consider future land use plans for the watershed because of the impact that land use changes may have on the hydrologic response of the area. The increase in the amount of impervious surface area and the concentration of flows in streets or improved channels can significantly increase both runoff volumes and peak discharge rates. In order to investigate the potential for such changes, contact was established with the City of Scottsdale. The City provided a copy of the Tonto Foothills General Plan, Scottsdale, Arizona (2/18/86). This plan provides development guidelines for approximately 75% of the contributing drainage area. Proposed land uses include residential (single family townhouses, apartments, tourist and resorts), commercial, special use (cultural, utilities, and institutional), office, employment (light industry, major office, and warehouse) and open space areas. The most predominant land use proposed in the Tonto Foothills General Plan is "single family residential", with maximum densities ranging from 0.3 to 4.8 dwelling units per acre.

Discussions with the City of Scottsdale indicated that their drainage ordinance will require any future development to comply with the requirement that "peak rates of outflow from developed property be no greater than the peak rates of runoff from the same property under natural conditions with no development." Although this ordinance will provide an upper limit on the peak discharge leaving a specific parcel of land, the timing of the outflow hydrograph should also be controlled in order to prevent a delayed downstream combination of hydrographs that might have the potential to produce a more severe flood peak at some location than would occur under natural conditions. The future installation of numerous detention basins in the watershed may lead to this type of problem if some type of master drainage plan is not adopted to coordinate and interface future drainage improvements. For the purpose of preparing this hydrology study for the Outer Loop, it was assumed that future development of the watershed will not be allowed to occur in a manner that would cause an increase in flooding potential beyond that presently existing in the watershed. This assumption was utilized in selecting SCS curve numbers for the HEC-1 model that was used to generate the rainfall/runoff response for

the watershed.

## 2.3 Soil Type and Vegetation

Soils information is needed in order to model the infiltration characteristics of the watershed. Such information is generally available from Soil Survey Reports published by the Soil Conservation Service (SCS). The watershed for this project was included in the Soil Survey of Aguila-Carefree Area, Parts of Maricopa and Pinal Counties, Arizona, U.S. Department of Agriculture, April 1986.

Using the standard SCS hydrologic soil group classification system, an estimate can be made of the runoff potential of the soils within any given sub-basin of the project watershed. The SCS system is based on four hydrologic soil groups, A through D. Soils in group A have very low runoff potential (i.e., high infiltration rate), those in group B have moderately low runoff potential, those in group C have moderately high runoff potential, and those in group D have high runoff potential (i.e., very slow infiltration rate).

Plate 2 illustrates the composition of the project watershed in terms of hydrologic soil groups. The information in this figure is based on the previously referenced soil survey. As can be seen from Plate 2, the watershed is composed of all four hydrologic soil groups (A, B, C, and D).

From the SCS soil survey, the following descriptions are provided for some of the major soil classifications comprising the drainage area.

1. Soil Group A - Anthony-Arizo Complex

   The Anthony soil is deep and well drained. It formed in alluvium derived dominantly from acid and basic igneous rock. Typically, the surface layer is light brown, sandy loam about 2 inches thick. The upper 38 inches of the underlying material is light brown and brown calcareous, gravelly, sandy loam. The lower part, to a depth of 60 inches or more, is light reddish, brown calcareous loam. In some areas this soil is noncalcareous to a depth of 10 inches or more.

   Permeability of the Anthony soil is moderately rapid. Available water capacity is moderate. Effective rooting depth is 60 inches or more. Runoff is slow, and the hazard of water erosion is moderate.

Where unprotected, this soil is subject to rare periods of flooding.

The Arizo soil is deep and excessively drained. It formed in alluvium derived dominantly from acid and basic igneous rock. Typically, the surface layer is pink gravelly, sandy loam about 1 inch thick. The upper 7 inches of the underlying material is light brown, very gravelly, sandy loam. The lower part, to a depth of 60 inches or more, is reddish yellow and pink calcareous, very gravelly, loamy sand.

Permeability of the Arizo soil is very rapid. Available water capacity is low. Effective rooting depth is 60 inches or more. Runoff is very slow, and the hazard of water erosion is severe. Where unprotected, this soil is subject to occasional periods of flooding. Channeling, deposition, and streambank erosion occur during periods of flooding.

2. Soil Group B - Momoli gravelly sandy loam

Typically, the surface layer is strong brown, gravelly, sandy loam about 3 inches thick. The subsoil is strong brown, very gravelly, sandy loam about 23 inches thick. The substratum to a depth of 60 inches or more is brown, calcareous very gravelly, sandy loam.

Permeability of the Momoli soil is moderately rapid. Available water capacity is low. Effective rooting depth is 60 inches or more. Runoff is slow, and the hazard of water erosion is slight.

3. Soil Group C - Eba very gravelly loam

Typically, the surface layer is pink, very gravelly loam about 3 inches thick. The subsoil is reddish yellow, very gravelly and extremely gravelly clay about 33 inches thick. The substratum to a depth of 60 inches or more is reddish yellow, extremely gravelly sandy loam. The soil is calcareous below a depth of about 11 inches.

Permeability of the Eba soil is slow. Available water capacity is low. Effective rooting depth is 60 inches or more. Runoff is slow to medium, and the hazard of water erosion is slight.

4. Soil Group D - Gran-Wickenburg Complex

The Gran soil is shallow and well drained. It is formed in alluvium and colluvium derived dominantly from granite, latite, and gneiss. Typically, the surface is covered with pebbles and cobbles. The surface layer is reddish yellow, very gravelly sandy loam about 1 inch thick. The subsoil is light red, extremely gravelly, sandy clay about 11 inches thick. Below this to a depth of 36 inches is light red, partially weathered granite. Unweathered bedrock is at a depth of 36 inches. In a few areas, this soil has less clay than is typical.

Permeability of the Gran soil is moderately slow. Available water capacity is very low. Effective rooting depth is less than 20 inches. Runoff is slow to medium, and the hazard of water erosion is slight.

The Wickenburg soil is shallow and well drained. It formed in alluvium and colluvium derived dominantly from granite, latite, and gneiss. Typically, the surface is covered with pebbles and cobbles. The surface layer is light reddish brown, very gravelly, sandy loam about 11 inches thick. Below this to a depth of 60 inches or more, is reddish brown, calcareous, partially weathered granite.

Permeability of the Wickenburg soil is moderately rapid. Available water capacity is very low. Effective rooting depth is less than 20 inches. Runoff is slow, and the hazard of water erosion is slight.

Due to its predominantly undeveloped nature, the vegetation pattern in the watershed is typical desert brush consisting of mesquite, catclaw, creosote bush, palo verde, ironwood, cacti, etc. For those portions of the watershed that have been developed, there has been an attempt to preserve, as

much as possible, the natural desert landscape. Preservation of the natural character of the land is in concert with the Tonto Foothills General Plan adopted by the City of Scottsdale.

The soils and vegetation data discussed in this section were used to select Soil Conservation Service (SCS) curve numbers. These curve numbers, which model the hydrologic abstractions and infiltration characteristics of the watershed, are discussed in more detail in Section 4.2 of this report.

## 2.4 Existing Drainage Facilities

Due to the relatively undeveloped nature of the watershed, there are no drainage facilities or flood control structures in the area that would have any significant impact on peak discharge rates expected along the proposed Outer Loop Freeway alignment. The drainage ditches that exist along the side of some of the major roads (Scottsdale Road, Pima Road, Pinnacle Peak Road, etc.) may create some minor diversion of runoff during the more frequent floods, such as a 2-year event. However, severe floods, such as a 50- or 100-year event, will greatly exceed the capacity of these small channels and will continue to flow along their natural drainage path. During field inspections of the watershed, it was also observed most of the roads utilize "dip" sections at their intersection with the natural desert washes. This practice promotes the tendency for floodwater runoff to continue along its natural course rather than being diverted by the roadways.

It is assumed that Scottsdale's efforts to regulate development in this area will continue to preserve the natural drainage pattern of the watershed. This is an important assumption and one that should continue to be enforced by regulatory agencies, since any unnatural diversion of stormwater runoff could change the hydrologic design discharges that will ultimately be used for design of a cross-drainage system for this reach of the Outer Loop.

## III. RAINFALL CHARACTERISTICS

The hydrologic response of a watershed is dependent upon rainfall characteristics such as depth, duration, and the spatial and temporal distribution of the rainfall event. The rainfall depth is a function of the probability of occurrence and the duration of the event. This probability is expressed as a recurrence interval (50-year, 100-year, etc.), which is defined as the average interval of time within which the magnitude of an event will be equaled or exceeded once. Mathematically, recurrence interval is defined as the reciprocal of the probability of occurrence.

Evaluating storms with different recurrence intervals is required when considering the risk and economic factors associated with the design of a drainage system for a specific meteorological event. In order to incorporate a risk analysis into the freeway design process, ADOT has requested that both the 50- and 100-year storms be evaluated as part of the hydrologic analysis for the Outer Loop.

Rainfall depths for the project drainage area were developed using isopluvial maps and regression equations presented in the Precipitation-Frequency Atlas of the Western United States, Volume VIII - Arizona. Table 3.1-A summarizes point precipitation values for the 50- and 100-year storms for durations of 1-, 2-, 3-, 6-, 12-, and 24-hours.

When using the hypothetical storm distribution in HEC-1, point rainfall values are automatically reduced (by the computer program) in accordance with procedures outlined in Weather Bureau Technical Paper No. 40, May 1961, to account for areal reduction of rainfall depths. For other distributions, such as the SCS Type 2A, a basin-average rainfall depth must be specified by the user. Under this condition, the user must make any desired corrections for areal reduction in rainfall depth. Accordingly, Table 3.1-B shows the areal rainfall reductions that were made for the Type 2A distribution. Reductions for each of the seven sub-drainage areas were made for the Type 2A storm in order to provide an equivalent comparison of peak discharge values with the HEC-1 results for the hypothetical storm.

In order to analyze the sensitivity of the watershed to the temporal distribution of rainfall, both 3- and 24-hour storms were evaluated. The hypothetical storm subroutine in HEC-1 was used to distribute the rainfall for

TABLE 3.1

SUMMARY OF RAINFALL DATA

A. RAINFALL DEPTHS AS A FUNCTION OF RECURRENCE INTERVAL AND DURATION

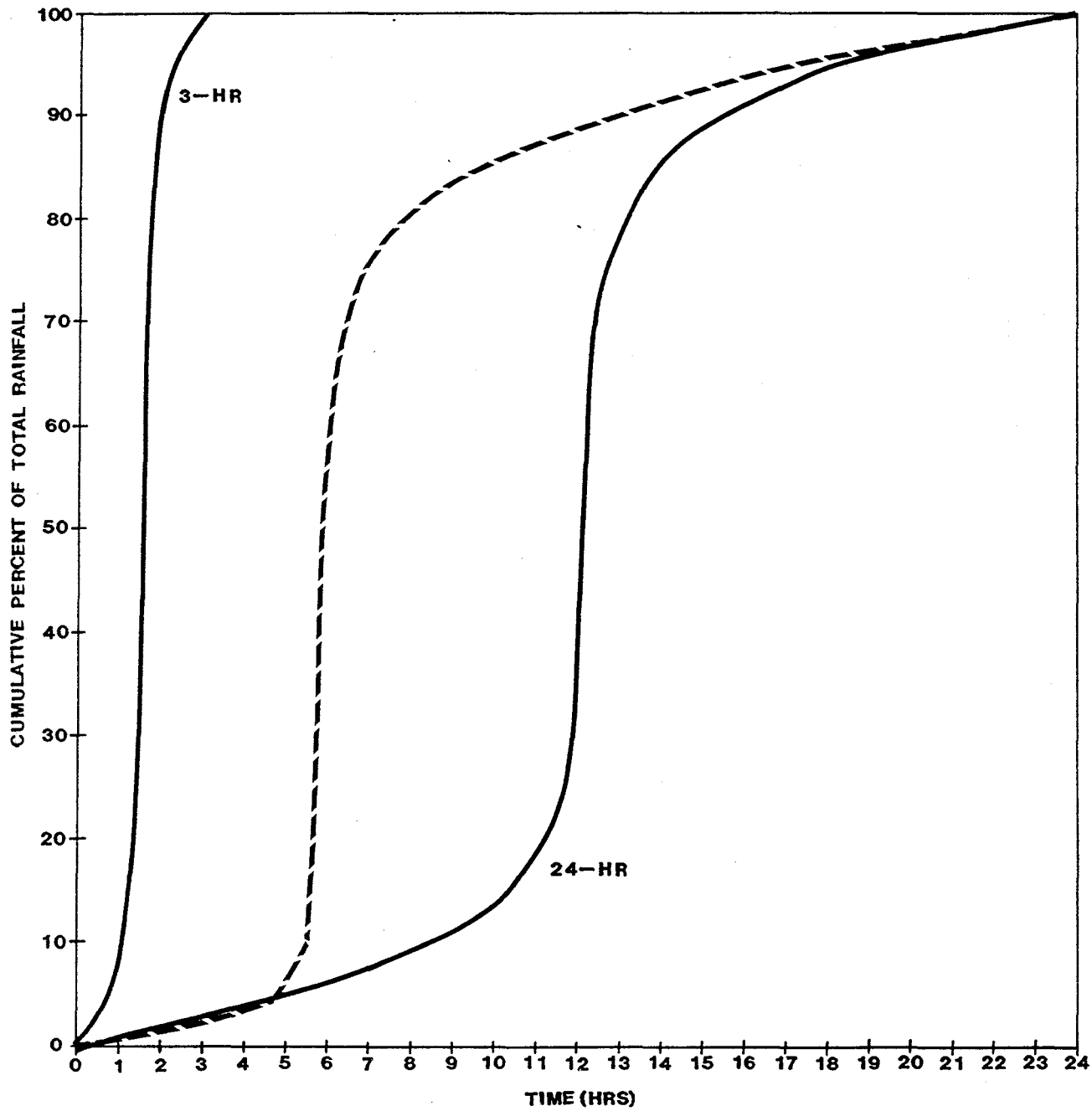
| Recurrence Interval (Years) | Point Precipitation Values (inches) | | | | | |
|---|---|---|---|---|---|---|
| | Duration (hours) | | | | | |
| | 1 | 2 | 3 | 6 | 12 | 24 |
| 50 | 2.23 | 2.49 | 2.67 | 3.01 | 3.41 | 3.82 |
| 100 | 2.47 | 2.77 | 2.97 | 3.35 | 3.81 | 4.28 |

B. AREAL RAINFALL REDUCTION FOR TYPE 2A, 24-HOUR STORM

| Sub-Drainage Area | Size (sq. mi.) | 24-Hour Point Rainfall (in.) | Areal Reduction Factor | Areal Rainfall (in.) |
|---|---|---|---|---|
| | | 100-Year | | |
| A | 8.25 | 4.28 | 0.987 | 4.22 |
| B | 5.33 | 4.28 | 0.991 | 4.24 |
| C | 4.26 | 4.28 | 0.994 | 4.25 |
| D | 18.09 | 4.28 | 0.975 | 4.17 |
| E | 5.53 | 4.28 | 0.990 | 4.24 |
| F | 6.03 | 4.28 | 0.989 | 4.23 |
| G | 12.37 | 4.28 | 0.982 | 4.20 |
| | | 50-Year | | |
| A | 8.25 | 3.82 | 0.987 | 3.77 |
| B | 5.33 | 3.82 | 0.991 | 3.79 |
| C | 4.26 | 3.82 | 0.994 | 3.80 |
| D | 18.09 | 3.82 | 0.975 | 3.72 |
| E | 5.53 | 3.82 | 0.990 | 3.78 |
| F | 6.03 | 3.82 | 0.989 | 3.78 |
| G | 12.37 | 3.82 | 0.982 | 3.75 |

each of these events. Figure 3.1 presents a graphical comparison of the temporal distribution for each of these durations. For comparative purposes, the standard 24-hour SCS Type 2A distribution is also shown on Figure 3.1.

All of these distributions were evaluated using the HEC-1 model. The selection of the design storm is discussed in Section 4.4.2.




**LEGEND**

HEC—1 HYPOTHETICAL

SCS 24—HR TYPE IIA

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**RAINFALL DISTRIBUTIONS**

FIGURE 3.1

## IV. HYDROLOGIC MODEL (HEC-1)

A computerized rainfall/runoff model was developed for the watershed using the U.S. Army Corps of Engineers Flood Hydrograph Package (HEC-1). HEC-1 uses numerical parameters to describe the amount and temporal distribution of rainfall, the runoff characteristics of the watershed, and the hydraulic properties of channels that collect and convey the direct runoff to concentration points. The computer output provides a runoff hydrograph at user selected locations. These hydrographs can be used to design drainage channels, detention/retention basins, or to evaluate the capacity of existing drainage facilities.

This section of the report presents a detailed discussion of specific components of the computer model that were created to simulate the rainfall/runoff response of the watershed. Table 4.1 summarizes the values used to simulate the overland flow elements that are discussed in the following subsections.

The results of the modeling process are also presented (Section 4.4) along with a brief discussion of model verification.

### 4.1 Delineation of Drainage Sub-Basins

This 8.2 mile segment of the Outer Loop intercepts approximately 60 square miles of drainage area. Since the proposed freeway alignment crosses the natural drainage pattern of the alluvial fan terrace at a nearly perpendicular angle, it was necessary to subdivide the total contributing watershed area into seven independent sub-drainage drainage areas in order to analyze the distribution of intercepted runoff at different points along the freeway. These seven sub-drainage areas (A, B, C, D, E, F, G) were then subdivided into 53 sub-basins that were considered to have relatively homogeneous hydrologic properties within their boundaries. Plate 1 is a December 1985 aerial photograph illustrating the delineation of the sub-drainage and sub-basin areas.

A major factor in the delineation of the watershed sub-basins was the alluvial fan terrace which becomes a prominent geologic feature through the lower half of the watershed. Above this fan, the desert washes are fairly well defined and tend to exhibit a drainage network characterized by a tributary pattern that feeds a dominant or main channel within a sub-drainage area.

TABLE 4.1

SUMMARY OF SUB-BASIN COMPONENTS USED TO COMPUTE OVERLAND FLOW RUNOFF

| Sub-Drainage Designation | Sub-Basin Number | Drainage Area (sq. mi.) | Curve Number | Overland Flow Length (ft) | Overland Flow Slope (ft/ft) | Overland Flow Roughness |
|---|---|---|---|---|---|---|
| A | 1 | 0.34 | 79 | 257 | 0.0149 | 0.20 |
| | 2 | 1.22 | 83 | 217 | 0.0148 | 0.20 |
| | 3 | 0.54 | 81 | 204 | 0.0189 | 0.20 |
| | 4 | 1.51 | 75 | 84 | 0.0213 | 0.10 |
| | 5 | 1.87 | 75 | 81 | 0.0213 | 0.10 |
| | 6 | 2.77 | 75 | 146 | 0.0213 | 0.10 |
| B | 7 | 0.67 | 79 | 101 | 0.0135 | 0.20 |
| | 8 | 1.09 | 75 | 116 | 0.0213 | 0.10 |
| | 9 | 1.46 | 75 | 95 | 0.0213 | 0.10 |
| | 10 | 2.11 | 75 | 107 | 0.0213 | 0.10 |
| C | 11 | 0.59 | 75 | 104 | 0.0234 | 0.20 |
| | 12 | 1.33 | 75 | 95 | 0.0213 | 0.10 |
| | 13 | 1.22 | 75 | 86 | 0.0213 | 0.10 |
| | 14 | 1.12 | 75 | 101 | 0.0213 | 0.10 |
| D | 15 | 1.33 | 83 | 168 | 0.0279 | 0.20 |
| | 16 | 1.48 | 83 | 242 | 0.0147 | 0.20 |
| | 17 | 1.15 | 83 | 275 | 0.0519 | 0.20 |
| | 18 | 1.27 | 83 | 322 | 0.0962 | 0.20 |
| | 19 | 0.18 | 85 | 222 | 0.0110 | 0.20 |
| | 20 | 0.57 | 85 | 501 | 0.0630 | 0.20 |
| | 21 | 3.68 | 81 | 201 | 0.0235 | 0.20 |
| | 22 | 1.39 | 85 | 138 | 0.0221 | 0.20 |
| | 23 | 0.39 | 85 | 186 | 0.0188 | 0.20 |
| | 24 | 0.46 | 80 | 125 | 0.0127 | 0.20 |
| | 25 | 1.52 | 85 | 429 | 0.0197 | 0.20 |
| | 26 | 0.61 | 76 | 149 | 0.0129 | 0.20 |
| | 27 | 0.76 | 75 | 159 | 0.0213 | 0.10 |
| | 28 | 1.42 | 75 | 96 | 0.0213 | 0.10 |
| | 29 | 1.88 | 75 | 88 | 0.0213 | 0.10 |

TABLE 4.1 (continued)

SUMMARY OF SUB-BASIN COMPONENTS USED TO COMPUTE OVERLAND FLOW RUNOFF

| Sub-Drainage Designation | Sub-Basin Number | Drainage Area (sq. mi.) | Curve Number | Overland Flow Length (ft) | Overland Flow Slope (ft/ft) | Overland Flow Roughness |
|---|---|---|---|---|---|---|
| E | 30 | 1.33 | 81 | 165 | 0.1211 | 0.20 |
| | 31 | 1.23 | 75 | 102 | 0.0213 | 0.10 |
| | 32 | 1.40 | 75 | 67 | 0.0213 | 0.10 |
| | 33 | 1.57 | 75 | 89 | 0.0213 | 0.10 |
| F | 34 | 0.84 | 81 | 178 | 0.0984 | 0.20 |
| | 35 | 1.29 | 80 | 247 | 0.0149 | 0.20 |
| | 36 | 1.45 | 75 | 116 | 0.0213 | 0.10 |
| | 37 | 1.19 | 75 | 61 | 0.0213 | 0.10 |
| | 38 | 1.26 | 75 | 84 | 0.0213 | 0.10 |
| G | 39 | 1.94 | 85 | 399 | 0.0315 | 0.20 |
| | 40 | 0.58 | 83 | 567 | 0.0791 | 0.20 |
| | 41 | 0.79 | 78 | 322 | 0.0845 | 0.20 |
| | 42 | 0.58 | 80 | 150 | 0.1472 | 0.20 |
| | 43 | 1.07 | 86 | 272 | 0.0313 | 0.20 |
| | 44 | 0.85 | 85 | 295 | 0.0573 | 0.20 |
| | 45R | 0.12 | 85 | 1700 | 0.2059 | 0.20 |
| | 45L | 0.26 | 85 | 1150 | 0.3304 | 0.20 |
| | 46R | 0.18 | 86 | 800 | 0.3000 | 0.20 |
| | 46L | 0.75 | 86 | 2800 | 0.3982 | 0.20 |
| | 47 | 0.38 | 80 | 300 | 0.0933 | 0.20 |
| | 48 | 0.04 | 81 | 313 | 0.0455 | 0.20 |
| | 49 | 0.35 | 81 | 79 | 0.0149 | 0.20 |
| | 50R | 0.14 | 85 | 800 | 0.0875 | 0.20 |
| | 50L | 0.22 | 85 | 950 | 0.4842 | 0.30 |
| | 51 | 1.01 | 76 | 94 | 0.0213 | 0.10 |
| | 52 | 1.36 | 75 | 76 | 0.0213 | 0.10 |
| | 53 | 1.75 | 75 | 67 | 0.0213 | 0.10 |

This is in sharp contrast to the fan portion of the watershed which is characterized by a dense network of narrow, shallow, sinuous channels which intermittently mingle and then separate from each other. As a result, there is no dominant channel on the fan portion of the watershed which can be used as a main channel for the concentration of upstream flows.

Because of this discontinuity in drainage pattern, the fan portion of the watershed was modeled as wide strips (with runoff distributed uniformly across a portion of each strip), while the more defined channels in the upper reaches of the watershed were modeled in the more conventional riverine format where flows are routed via an existing channel to a concentration point at the sub-basin outlet.

## 4.2 Interception/Infiltration

Precipitation losses due to interception and infiltration were modeled using the SCS curve number option in HEC-1. Selection of curve numbers was based on information gathered on type of soil cover, vegetation density, land use, and soil moisture conditions. Average curve numbers were developed for each of the 53 sub-basins to account for the combined effect of these drainage basin characteristics.

Due to the potentially high infiltration rates that were suspected of existing on the alluvial fan portion of the watershed, a decision was made to conduct field tests to determine actual infiltration rates for the different hydrologic soil groups existing within the watershed. Double-cylinder ring infiltrometers were used to generate this data. The two infiltrometer rings were constructed of 1/4-inch steel plates which were fabricated into one-foot deep cylinders with individual diameters of 12-inches and 24-inches. The purpose of the 24-inch diameter outer ring is to provide a saturated barrier around the inner ring so that lateral infiltration will not occur within the inner ring.

The two cylinders were driven six-inches into the ground at each test location. The procedures followed during these tests are those recommended by Johnson "Suggested Method Of Test For Infiltration Rate In Field Using Double-Ring Infiltrometer" for ASTM (Special technical publication 479, 1970). Table 4.2 gives a summary of the infiltration test results. Appendix A presents

TABLE 4.2

RESULTS OF RING INFILTROMETER TESTS

| Site Location | Infiltration Rate in/hr | | SCS Hydrologic Soil Group |
|---|---|---|---|
| | Initial | Final | |
| 1. Near power substation north of Pinnacle Peak Road | 11.1 | 6.5 | B |
| 2. Sand-Bed wash near Site 1. Infiltration too high to measure. | N/A | N/A | A |
| 3. 1/2-mile south of Pinnacle Peak Road on Hayden Road | 10.1 | 4.2 | B-C |
| 4. Near dirt road 1.8 miles east of Pinnacle Peak restaurant | 8.1 | 6.6 | D |
| 5. NE of Hayden & Dixileta near western boundary of watershed | 1.9 | 1.5 | C |
| 6. Under powerline near eastern boundary of watershed | 1.5 | 1.0 | D |
| 7. 0.8 miles west of power substation, north side of Pinnacle Peak Road | 2.9 | 2.3 | B |
| 8. 0.8 miles south of Pinnacle Peak Road on Hayden Road | 5.0 | 5.4 | B |

photographs of the testing apparatus.

A review of Table 4.2 shows that there is considerable variation in steady-state infiltration rates within a given soil group, as well as considerable overlap in steady-state infiltration rates among the listed soil groups. Due to the large variability observed in infiltration rates, it was concluded that this data provided insufficient resolution of infiltration characteristics to be used as the basis for a hydrologic model of the watershed. Also, from a practical perspective, application of this data to the infiltration subroutines in HEC-1 was found to be very difficult, and would make the resulting model very complicated to use for possible analyses of future land use changes. However, the field data does serve as an important indicator that the soils in the watershed exhibit the potential for higher than normal permeability for the listed soil types.

Due to the limitations associated with the results of the infiltration tests, a decision was made to model the interception/infiltration characteristics of the watershed with SCS curve numbers. The primary guide used in selecting curve numbers was the Arizona Highway Department Manual entitled Hydrologic Design For Highway Drainage In Arizona. Figure 2-3 of that report presents curve numbers as a function of: 1) hydrologic soil group; 2) type of vegetative cover; and 3) vegetative cover density. The curve numbers in this figure are most accurate for storms of 1-hour duration or less (Runoff Curve Numbers For Semiarid Range and Forest Conditions, Woodward, 1973). Therefore, the curve numbers selected from this figure were adjusted for storm duration in accordance with recommendations provided by Woodward. Table 4.3 summarizes the curve number correlation for 1-, 3-, and 24-hour storm durations. The 3- and 24-hour curve numbers were used for the HEC-1 models developed for this study.

Because of the sparse development that presently exists in the watershed, all curve number selections were based on "desert brush" with 15% cover density. Soil moisture conditions were assumed to correspond to an average mositure content prior to the time the rainfall event occurs. This condition is designated as AMC-II by the Soil Conservation Service.

As previously discussed in Section 2.2, the City of Scottsdale requires that urban development be implemented in a manner that will not increase the peak rate of runoff beyond that existing under present-day conditions. This

TABLE 4.3

SUMMARY OF CURVE NUMBER ADJUSTMENTS AS A FUNCTION OF STORM DURATION[1]

| Storm Duration (hours) | Curve Numbers | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 83 | 84 | 85 | 86 | 87 | 88 | 89 | 90 | 91 | 92 |
| 3 | 80 | 81 | 83 | 84 | 84 | 86 | 87 | 88 | 89 | 90 |
| 24 | 75 | 76 | 78 | 79 | 80 | 81 | 82 | 83 | 85 | 86 |

[1] Curve number reductions made in accordance with Figure 4, Runoff Curve Numbers For Semiarid Range And Forest Conditions, Woodward, 1973

concept was approved by the City Council with the adoption of Ordinance No. 1644 on June 4, 1984. Informal discussions with City of Scottsdale staff indicate a committment to enforce the provisions of Ordinance No. 1644. Accordingly, for the purpose of this hydrologic analysis, it was assumed that areas that are presently undeveloped, but forecast by Scottsdale to develop in the future, will provide detention/retention storage so that the peak rate of runoff will be the same for both pre- and post-development conditions. The City staff has informally concurred in this approach.

## 4.3 Configuration of the Hydrologic Model

The HEC-1 model developed by the Corps of Engineers Hydrologic Engineering Center, was used to simulate runoff conditions in the study area. The kinematic wave option was used to determine the hydrologic response of the sub-basin areas and for routing the resulting hydrographs through the tributary channels of the basin. This option was selected because runoff processes can be simulated using measurable geographic features such as overland flow elements and the shape, boundary roughness, length, and slope of channel elements. Unlike unit hydrograph techniques, the kinematic wave approach also provides for a non-linear response of runoff characteristics, i.e., peak discharge does not necessarily increase linearly with direct runoff when using the kinematic wave methodology.

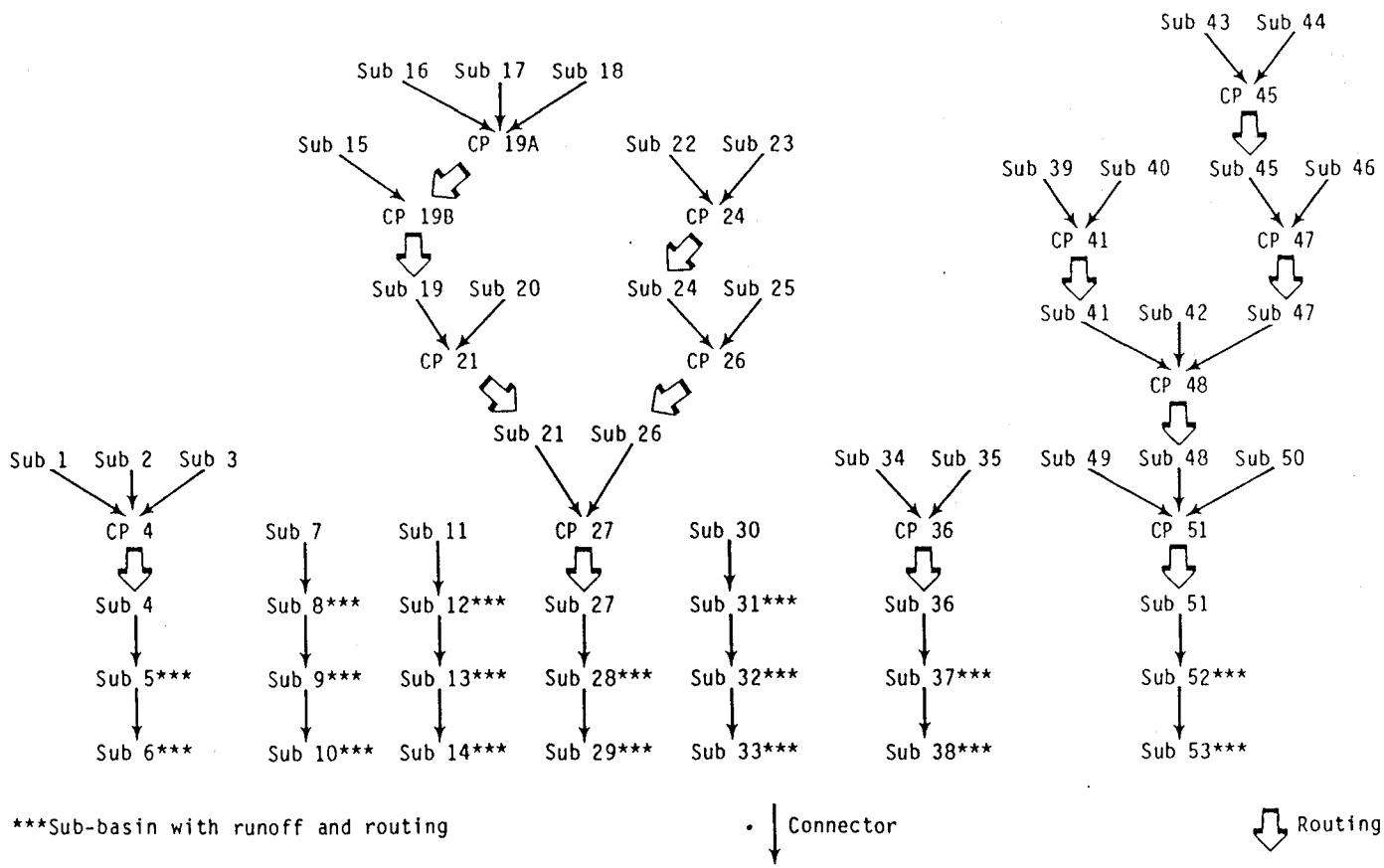
A network of sub-basins and connecting channels was configured that simulates the natural drainage pattern in the basin. Plate 3 presents an illustration of the drainage patterns, sub-basin boundaries, and concentration points used in the model, while Figure 4.1 is a schematic diagram of the model.

The following subsections describe the parameters that were used to model the physical characteristics of the watershed.

### 4.3.1 Overland Flow Length

Overland flow length is an important parameter in determining the runoff response of a watershed. Decreasing the overland flow length will result in an increase in peak discharge while lengthening the overland flow path will have the opposite effect, i.e., decrease the peak discharge. The higher peak

## SCHEMATIC DIAGRAM OF STREAM NETWORK

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**ROUTING SCHEMATIC FOR HEC–1 MODEL**

FIGURE 4.1

discharge associated with short overland flow lengths results from the shorter travel time it takes overland flow to reach a channel, where it combines with runoff from other overland flow elements and begins traveling more efficiently as open channel flow.

The selection of overland flow lengths for this watershed was made through an analysis of the rill or channel density of each of the 53 sub-basins comprising the seven independent sub-drainage areas. This analysis was based on a 1"=1000', 1984 aerial photograph of the watershed. Each sub-basin was delineated on this photo and lines were drawn perpendicular to the average flow pattern within each sub-basin. The number of rills or channels intercepted by each line was then made through a visual inspection of the photo. An average width between rills was then obtained by dividing the total length of the line by the number of rills intercepted by the line. The average overland flow length was then computed as one-half the average distance between rills, based on the assumption that one-half this distance will drain to one rill while the other half will drain to the adjacent rill. As many as two or three lines were drawn on some sub-basins in order to establish an average overland flow length for the entire sub-basin.

In order to illustrate the dramatic difference in the geomorphic characteristics of the alluvial fan terrace, versus the upper (northern) portion of the drainage basins, Table 4.4 presents the computed overland flow lengths as a function of these two different regions of the watershed. As would be expected, the fan area exhibits much shorter overland flow lengths as a result of the high rill density through the southern half of the basin.

### 4.3.2 Overland Flow Slope

An approximation of the slope of the overland flow planes was made with the use of U.S.G.S. 7.5-minute quadrangle maps (upper basin area) and surveyed cross-sections (lower basin area) of the watershed.

In the upper part of the basin, average overland flow slopes were measured along a line starting at a perpendicular angle to a prominent watercourse centerline (shown on the quadrangle map) and projected to the sub-basin boundary (without crossing another watercourse line). The length and elevation difference along this line was then used to compute the overland flow

**TABLE 4.4**

COMPARISON OF OVERLAND FLOW LENGTHS AS A

FUNCTION OF RILL/CHANNEL DENSITY AND WATERSHED LOCATION

| Fan Area | | | Upper Basin Area | | |
|---|---|---|---|---|---|
| Sub-Basin | Overland Flow Length (ft) | Average Length For Sub-Drainage Area (ft) | Sub-Basin | Overland Flow Length (ft) | Average Length For Sub-Drainage Area (ft) |
| 4 | 84 | | 1 | 257 | |
| 5 | 81 | 104 | 2 | 217 | 226 |
| 6 | 146 | | 3 | 204 | |
| 8 | 116 | | 7 | 101 | 101 |
| 9 | 95 | 106 | | | |
| 10 | 107 | | 11 | 104 | 104 |
| 12 | 95 | | 15 | 168 | |
| 13 | 86 | 94 | 16 | 242 | |
| 14 | 101 | | 17 | 275 | |
| | | | 18 | 322 | |
| 27 | 159 | | 19 | 222 | |
| 28 | 96 | 114 | 20 | 501 | |
| 29 | 88 | | 21 | 201 | 247 |
| | | | 22 | 138 | |
| 31 | 102 | | 23 | 186 | |
| 32 | 67 | 86 | 24 | 125 | |
| 33 | 89 | | 25 | 429 | |
| | | | 26 | 149 | |
| 36 | 116 | | | | |
| 37 | 61 | 87 | 30 | 165 | 165 |
| 38 | 84 | | | | |
| | | | 34 | 178 | 213 |
| 51 | 94 | | 35 | 247 | |
| 52 | 76 | 79 | | | |
| 53 | 67 | | 39 | 399 | |
| | | | 40 | 567 | |
| | | | 41 | 322 | |
| | | Average Length For Entire Lower Basin Area = 96 | 42 | 150 | |
| | | | 43 | 272 | 300 |
| | | | 44 | 295 | |
| | | | 45 | N/A | |
| | | | 46 | N/A | |
| | | | 47 | 300 | |
| | | | 48 | 313 | |
| | | | 49 | 79 | |
| | | | 50 | N/A | |
| | | | | | Average Length For Entire Upper Basin Area = 194 |
| | | | | | Average Length, Excluding Sub-Basins 7 & 11 = 230 |

slope. As many as five lines were used in some sub-basins in order to determine an average overland flow slope that would be typical of the entire sub-basin. Using this method, different overland flow slopes were computed for all the sub-basins comprising the upper half of the watershed.

Due to the large contour interval (20-feet) on the quadrangle maps and the relatively flat topographic relief across the lower, fan portion of the watershed, a different technique had to be employed for computing overland flow slopes. Accordingly, seven wide (200' to 400') cross-sections were field surveyed on the fan portion of the watershed. Once these cross-sections were plotted, typical cross-slopes to individual rills could be easily computed. This was done for several cross-sections and an average cross-slope was found to be 0.0213 ft/ft. This value was then used as the average overland flow slope for all sub-basins on the fan area.

### 4.3.3 Overland Flow Roughness

Depths of overland flow may be on the order of 1/4-inch or less. Under such conditions, the texture or surface composition of the ground has a significant impact on the travel time required for overland flow to reach a channel element. Field inspections of the watershed revealed distinct differences in surface soil composition and vegetation density from the lower to upper portions of the watershed. On the lower fan portion of the drainage area, the ground surface is relatively smooth and flat and is composed of a much finer (smaller grain-size) material than exists in the upper basin. The upper portion of the basin exhibits gravel size surface materials, along with scattered rocks and boulders, and a much more rugged surface topography. Vegetation also appears to be slightly more dense in the upper part of the basin than in the lower part of the basin.

Based on these observations, an overland flow roughness value of 0.1 was selected for the lower (fan) portion of the watershed and a value of 0.2 was selected for the upper portions of the basin. For the steep, rugged mountain slopes on the south side of Sub-Basin 50, an overland flow roughness value of 0.3 was used. Selection of these values was based on engineering judgment and guidelines presented in the HEC-1 User's Manual.

4.3.4 Channel Configuration

Runoff from overland flow planes is concentrated in the numerous dry washes that drain the watershed. Once the water enters these washes, it is routed downstream as open-channel flow. For the kinematic wave option, this routing procedure is a function of: 1) channel length; 2) channel slope; 3) channel shape; and 4) channel roughness. Because of the distinct difference in the geomorphology of the lower (fan) portion of the watershed versus the upper portion, two approaches were used to simulate the channel characteristics that water would encounter as it travels down the watershed.

Cross-sections were surveyed for six different washes within the upper region of the watershed. Due to the large size of the basin, as well as access problems, it was not possible to survey a wash in each sub-basin. Accordingly, based on a review of an aerial photograph, survey sites were selected that were thought to be typical of all channel geometries existing within the upper portion of the watershed. The cross-section data generated from this field work was used to develop generalized area (A) versus perimeter (P) relationships for channels in different regions of the watershed. Through a trial and error procedure, prismatic cross-sections were then investigated until one was found which exhibited a similar A vs. P relationship to that developed from the surveyed cross-sections. Refinements to this curve fitting process were made by introducing Mannings "n" value to the calculations in order that hydraulic conveyance could be matched (as closely as possible) between the natural and prismatic sections. The selection of "n" values for the surveyed cross-sections was made on the basis of calibrated "n" values presented in a photo report entitled "Roughness Coefficients for Stream Channels in Arizona" by Aldridge and Garrett, U.S.G.S., February 1973.

The use of prismatic sections to model the conveyance properties of the well defined, natural sections in the upper reaches of the watershed was easily accomplished. However, on the fan portion of the watershed, there is no single, well defined collector channel that transports flow across the fan. Accordingly, the collector channel geometry had to be dimensioned in a manner that would simulate the wide, shallow, sheetflow characteristics of the fan.

This process became somewhat complex in that the channel geometry on the fan will be dependent on the rate of flow across the fan, e.g., for small flows, the water may be contained within the banks of the numerous, small

channels that traverse the fan, while during larger floods, the bankfull capacity of these washes will be exceeded and the water will tend to spread out in a wide, shallow sheet.

This problem was resolved by first estimating the average rate of flow that could be carried within the bankfull capacity of the washes that cross the fan. The average bankfull capacity of a typical wash was computed through application of Manning's Equation to the cross-sectional geometry surveyed for six washes on the fan. This analysis revealed that a typical wash would carry approximately 80 cfs before overtopping its banks. This information was then combined with the number of rills/channels counted for each sub-basin on the fan in order to obtain the average peak rate of flow that could be contained within the channels flowing through each sub-basin.

Once this information (total channel capacity per sub-basin) was known, the HEC-1 model was executed to determine how much water would be delivered to the upper end of each sub-drainage area (A, B, C, D, E, F, G) strip crossing the fan. If this rate of flow was found to be in excess of the total rill/channel capacity of the downstream sub-basin, then the water was assumed to be spread across the sub-basin as wide, shallow sheet-flow. Under these conditions, a wide (500' to 4,000') trapezoidal channel section was then selected to route the water across the fan.

The dimensions of these wide channels were selected so as to provide realistic depths and velocities of flow across the fan. For these wide areas of inundation, realistic depths of flow were considered to be on the order of 1.5-feet or less while average velocities ranged from about 3-6 feet per second, with the higher velocities being encountered in the steeper, upper portions of the fan where the water was still concentrated in relatively narrow strips. As the water moved down the fan, it was assumed to spread out, which in most cases, resulted in a slight decrease in both average depth and velocity (in the downstream direction).

Once the initial channel geometry for each of the fan sub-basins was selected and coded into HEC-1, the model was re-run to determine what peak discharge would be routed through each sub-basin. This step was necessary to determine the attenuation effect that the selected channel geometry would have on the peak discharge being routed through each sub-basin. These peak discharge values were then used with Manning's Equation, and the assigned sub-

basin channel geometry, to compute the average depth and velocity of flow through each sub-basin. If the depths and velocities appeared realistic (based on previously defined envelopes for these parameters), the channel geometry was accepted as being representative of the flow conditions within the sub-basin being investigated. If flow conditions were found to be unrealistic, the channel geometry was adjusted and the model re-run. This procedure was continued until realistic depth and velocity conditions were produced for each sub-basin on the fan.

For those sub-basins on the fan that were found to have total rill/channel capacities approximately equal to the incoming flow, a slightly different approach was taken for selecting channel routing dimensions. For these cases, a trapezoidal section with a 50-foot bottom width was selected. Side-slopes for this channel section were varied from 50:1 to 200:1, as the water moved downstream. This flattening of the side-slope was made in order to keep the depth of flow in the channel to less than 2-feet and the average velocities in the 3 to 5 feet per second range. Due to the dense braiding pattern on the fan, and the fact that additional runoff was being intercepted as one moves downstream, it was assumed that as the water moved down-slope, it would feed into more and more small channels, thus causing an increase in the total channel perimeter and inundated width of flow. The flattening of channel side-slopes in adjacent downstream sub-basins provides a degree of simulation of this phenomenon since such channel geometry also produces an increase in perimeter and topwidth.

As with the previously discussed wide, shallow sections, an iterative procedure was also used to refine the dimensions of the narrower sections used to model those sub-basins with sufficient rill capacity to convey the predicted incoming flow. These refinements were in the form of side-slope adjustments until a side-slope was found that, when combined with the 50-foot bottom-width and predicted peak discharge, produced realistic depths and velocities of flow.

A Manning's "n" value of 0.045 was used to model all channel flow conditions on the fan. This was considered to be representative of both the wide, shallow sections as well as the narrower sections that simulate rill flow. The majority of the small channels on the fan were observed to have sufficient vegetation (brush) along the banks (and sometimes in the channel

bed) to support this "n" value.

## 4.4 Hydrologic Modeling Results

### 4.4.1 Verification of HEC-1 Model

Table 4.5 presents a summary of the peak discharge values predicted along the proposed freeway alignment from each of the seven sub-drainage areas that comprise the 60 square-mile watershed.

This table includes values for both the 50- and 100-year events, as a function of three different rainfall distributions.

In order to verify the HEC-1 model and to obtain guidance as to which rainfall distribution produces the most realistic results, independent peak discharge calculations were performed using regional regression equations developed from stream gage data in Arizona (Methods for Estimating the Magnitude and Frequency of Floods in Arizona, R. H. Roeske, U.S.G.S., 1978.) It should be noted that these equations were developed from gage data taken from incised, well-defined river cross-sections. The alluvial fan terrace being investigated in this study does not have a single well-defined channel to collect all upstream flows. As water spreads out across a fan, there should be significant attenuation in the peak discharge of the runoff hydrograph. Accordingly, the river-based regression equations will probably have a tendency to over-predict the peaks that will occur on the fan. This conclusion is supported by observations made by Mr. James H. Eychaner in a report entitled Estimation of Magnitude and Frequency of Floods in Pima County, Arizona, With Comparisons of Alternative Methods, U.S.G.S., August 1984. On pages 39 and 40 of that report, Mr. Eychaner discusses the severe peak discharge attenuation that can occur as water spreads laterally across floodplains. He also infers that peak discharge estimates using regression equations may be reduced by a factor of 0.5 under these conditions.

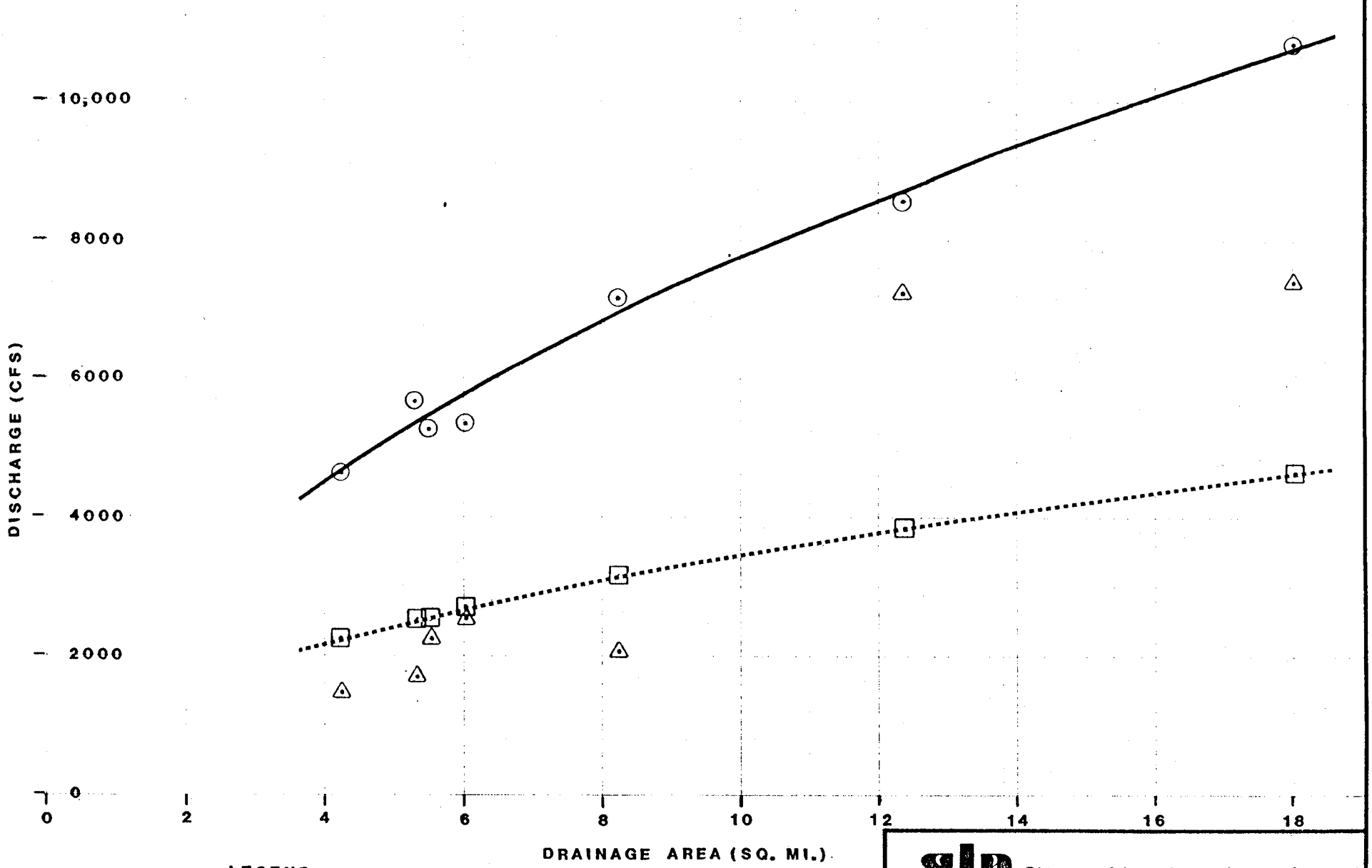
Figure 4.2 presents a graphical plot of the predicted peak discharge values obtained using the regression equations. Data from both Regions 2 and 3 are shown on this figure. Although the watershed under investigation is

TABLE 4.5

PEAK DISCHARGE SUMMARY

Outer Loop Freeway - North of CAP Aqueduct

| Sub-Drainage Area (sub-basins) | D.A. (sq. mi.) | Rainfall Distribution | Peak Discharge $Q_{100}$ (cfs) | Peak Discharge $Q_{50}$ (cfs) |
|---|---|---|---|---|
| A (1-6) | 8.25 | 24-Hour Hypothetical | 2038 | 1558 |
| B (7-10) | 5.33 | | 1678 | 1259 |
| C (11-14) | 4.26 | | 1445 | 1082 |
| D (15-29) | 18.09 | | 7339 | 5602 |
| E (30-33) | 5.53 | | 2228 | 1675 |
| F (34-38) | 6.03 | | 2570 | 1861 |
| G (39-53) | 12.37 | | 7202 | 5347 |
| A (1-6) | 8.25 | 24-Hour SCS Type 2A | 1749 | 1329 |
| B (7-10) | 5.33 | | 1398 | 1034 |
| C (11-14) | 4.26 | | 1208 | 897 |
| D (15-29) | 18.09 | | 6348 | 4772 |
| E (30-33) | 5.53 | | 1897 | 1405 |
| F (34-38) | 6.03 | | 2087 | 1473 |
| G (39-53) | 12.37 | | 6018 | 4441 |
| A (1-6) | 8.25 | 3-Hour Hypothetical | 1696 | 1323 |
| B (7-10) | 5.33 | | 1409 | 1075 |
| C (11-14) | 4.26 | | 1210 | 931 |
| D (15-29) | 18.09 | | 5902 | 4533 |
| E (30-33) | 5.53 | | 1841 | 1419 |
| F (34-38) | 6.03 | | 2057 | 1509 |
| G (39-53) | 12.37 | | 5510 | 4198 |




LEGEND

Q100, REGRESSION EQUATION, REGION 2

Q100, REGRESSION EQUATION, REGION 3

Q100, 24-HR HYPOTHETICAL

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GRAPHICAL COMPARISON OF PEAK DISCHARGE ESTIMATES

FIGURE 4.2

actually located within the Region 3 boundaries (as defined in the Roeske report), it is just on the edge of Region 2 and actually exhibits basin characteristics more closely aligned with those in Region 2.

For comparative purposes, the peak discharge values predicted by the 24-hour HEC-1 hypothetical distribution are also plotted on Figure 4.2. These discharge values are those that would be expected to occur along the freeway alignment at the downstream terminus of each of the seven sub-drainage areas.

A review of Figure 4.2 indicates the peak discharges predicted by HEC-1 are lower than those predicted by either the Region 2 or Region 3 regression equations for Sub-Drainage Areas A, B, C, E, and F and lie between the values predicted by the regression equations for Sub-Drainage Areas D and G.

Several factors exist which explain the seemingly low (when compared to the regression equation results) peak discharges predicted by HEC-1 for Areas A, B, C, E, and F. These factors are discussed as follows:

1. The majority of all these small drainage areas lie on the lower fan portion of the watershed, which consists predominantly of Hydrologic Soil Group B. This is a very permeable soil and will <u>not</u> produce large amounts of runoff.

2. The flow across these basins will be spread out among many small washes which will tend to widen the total width of flow in the downstream direction. This spreading effect will tend to attenuate the runoff hydrographs.

3. These small basins are all long and narrow with very high length to width ratios. As a result, the runoff hydrographs from adjacent sub-basins will not be combining at their peaks. This will significantly reduce the peak discharge at the outlet of each sub-drainage area.

4. As discussed previously, the regression equations are based on gage data obtained from watersheds with well-defined, incised channels. The small sub-drainage areas under investigation in this study do not have a single, well-defined, incised channel to efficiently transport floodwaters. Accordingly, application of the regression equations to

these small fan areas should be made with this limitation in mind, and, with the expectation that the regression equations will probably over-predict peaks for such conditions.

In contrast to the smaller sub-drainage areas, it is interesting to note that Sub-Drainage Areas D and G produce HEC-1 peaks that lie well above the Region 2 regression equation prediction. This is attributed to the fact that the upper half of each of these two basins consist predominantly of Hydrologic Soil Groups C and D, both of which produce higher rates of runoff than the B-soils found on the lower half of the fan portion of the basin.

The relatively small difference between peak discharge (as a function of drainage area, i.e., 12.37 mi.$^2$ vs. 18.09 mi.$^2$) is explained by the shape of the two basins. The upper half of Sub-Drainage Area D is much more elongated than that for Sub-Drainage Area G. As a result, the runoff from the sub-basins in the upper half of Area G tends to arrive at the top of the fan area at nearly the same time, thus producing a relatively high peak discharge. Since Area D is more elongated, the hydrographs from its upper sub-basins have a much more staggered arrival time at the top of the fan, thus resulting in a lower peak discharge compared to the same location for Area G.

Runoff hydrographs from both of the large sub-drainage areas (D and G) exhibit significant attenuation as the flow moves down the lower fan portion of the two basins. This attenuation is a result of the reduced velocities and depths of flow that accompany the lateral spreading of water as it enters the flat fan area. This hydrograph attenuation would probably be even more pronounced if the kinematic wave routing procedure had the capability to model storage characteristics of the sub-drainage area channels.

### 4.4.2 Recommendations

Based on a comparison of the peak discharge values predicted by HEC-1 to those predicted by regional regression equations, it is recommended that the design hydrology for this reach of the Outer Loop be based on the 24-hour, HEC-1 hypothetical rainfall distribution. In consideration of the unique alluvial fan features existing on this watershed, it is believed that the recommended design storm will provide a conservative approach which is in concert with that previously requested by DeLeuw, Cather & Company (Outer Loop

Management Consultant) and the Arizona Department of Transportation for that reach of the Outer Loop extending from the CAP Aqueduct to Camelback Walk.

Conservative assumptions used in developing the HEC-1 model for this watershed are summarized as follows:

1. Areal rainfall reduction was based on the area of each sub-drainage area rather than the entire 60 square mile watershed.

2. No channel transmission losses were modeled due to limitations within HEC-1.

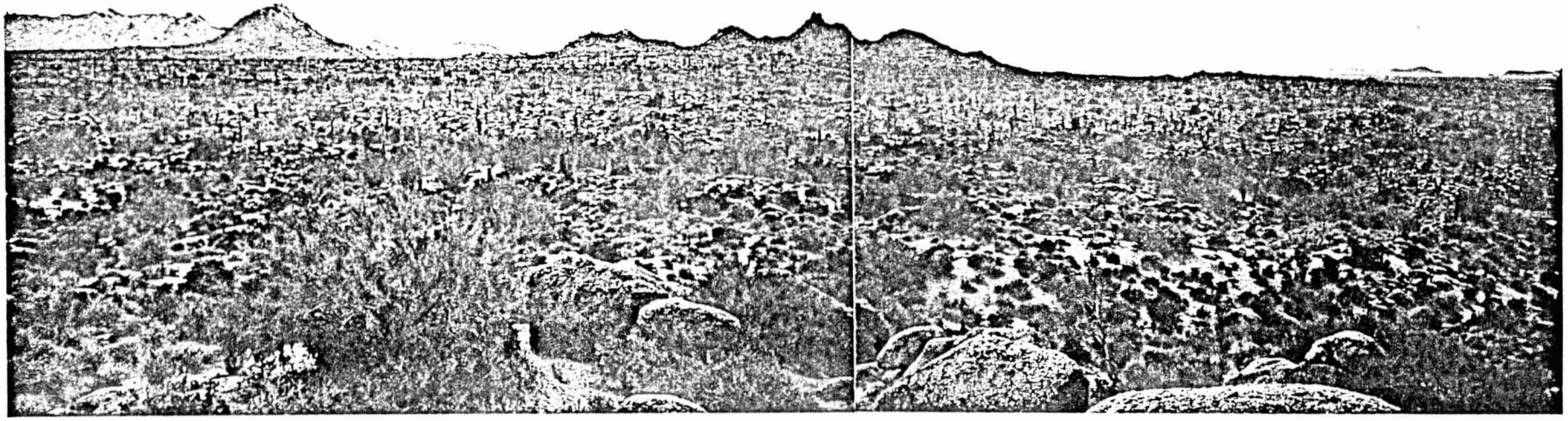
3. No channel storage effects were modeled in the hydrograph routing option provided by kinematic wave procedures.

4. Interception/infiltration characteristics were simulated in accordance with SCS curve numbers that were based on the different hydrologic soil groups within the watershed. Ring-infiltrometer tests conducted during this study indicate some Group D soils exhibit higher steady state infiltration rates than those in Group B. Accordingly, the use of SCS curve numbers (based on hydrologic soil group classifications) may over-predict the amount of direct runoff.

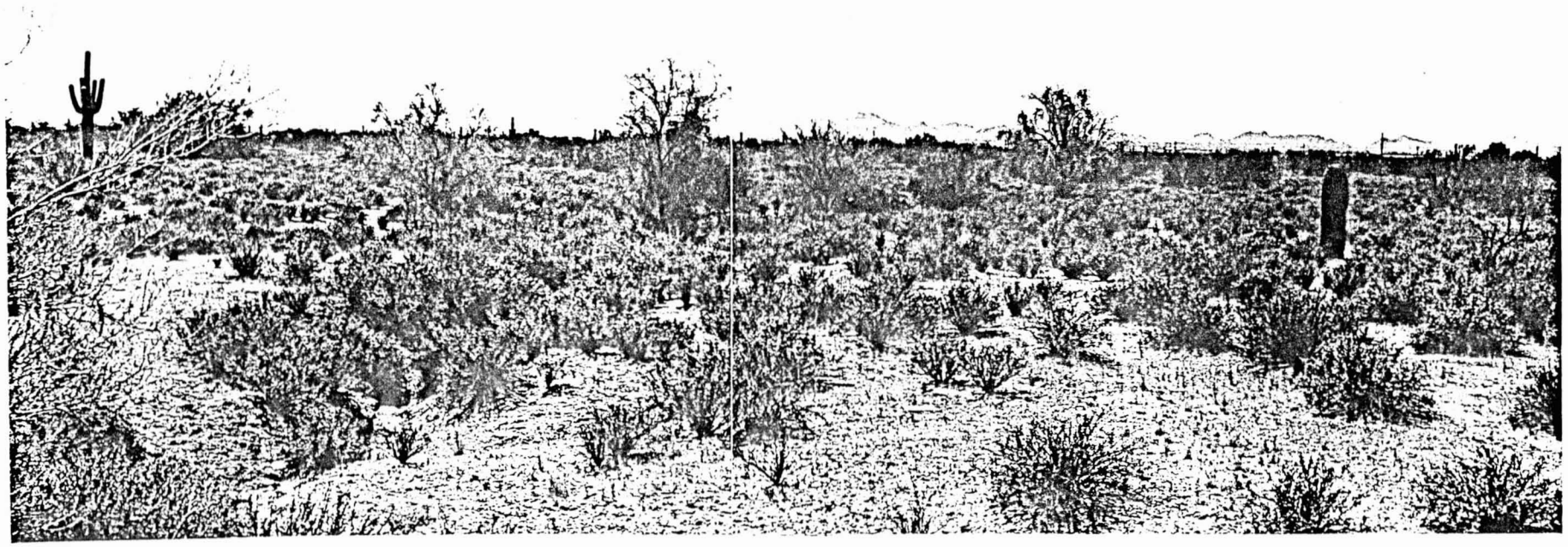
5. The combination of the large 24-hour rainfall depth, with the intense burst of rainfall provided at the mid-point in the HEC-1 hypothetical rainfall distribution, is a severe, unlikely combination of meteorological conditions for this region of Arizona. Accordingly, the runoff response produced by this combination should be conservative.

Appendix B provides a listing of the HEC-1 input and output data (both 100- and 50-year events) for the recommended 24-hour, hypothetical storm, while Appendix C presents hydrograph plots for the same events.

# APPENDIX A

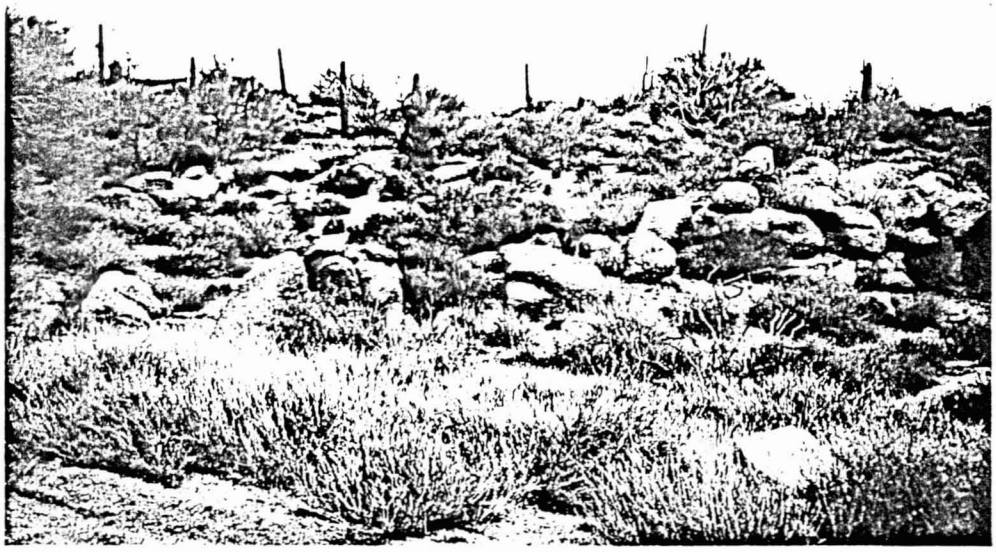


Looking southwesterly towards Pinnacle Peak
from Sub-Basin 39.
(9/20/86)

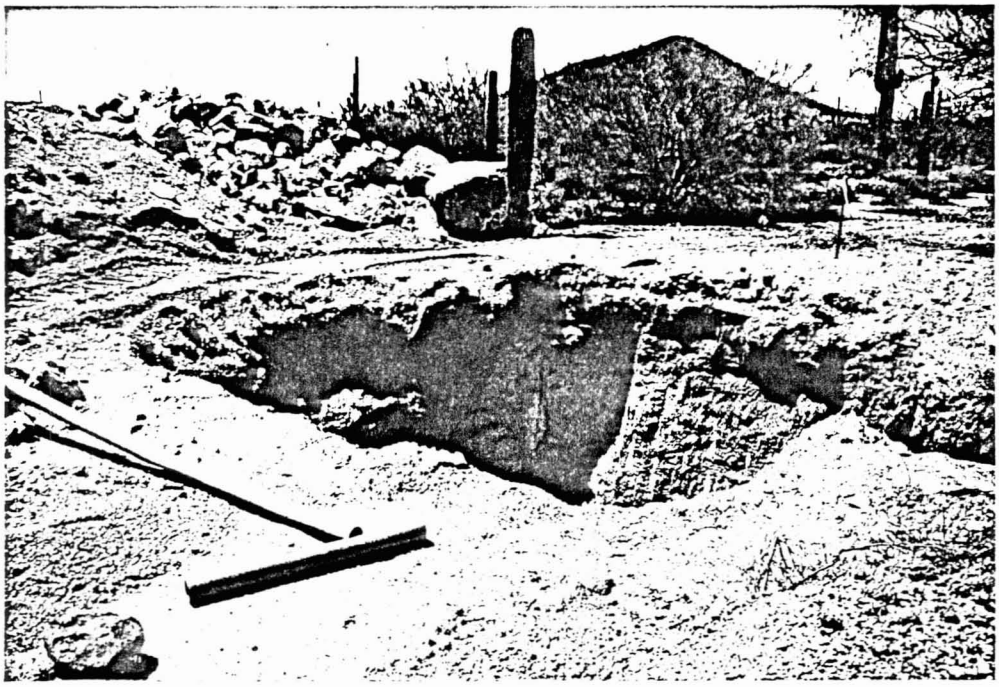


Looking downstream (southerly) at a typical section
of the alluvial fan terrace in Sub-Basin 9.
(6/18/86)

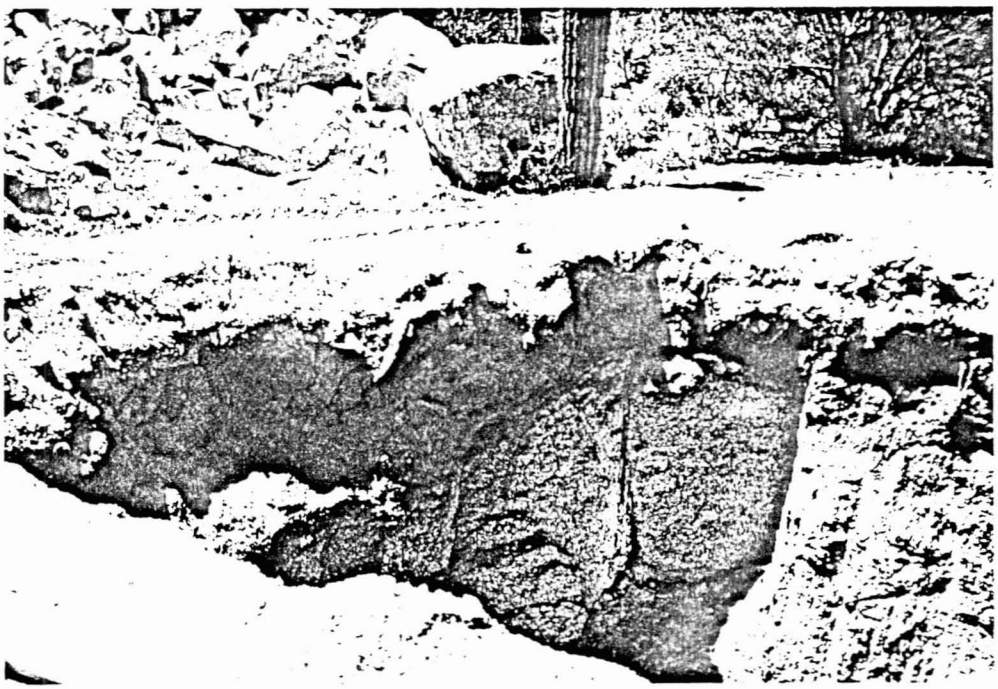
Fractured rock on desert surface near Dynamite Road east of Pima Road. (9/20/86)



Typical rock and boulder formations which frequently occur in the northern portions of the watershed. This photo was taken approximately 1.8 miles east of the Pinnacle Peak Restaurant. (9/20/86)



Excavation showing shallow depth of bedrock on Dixileta Road just east of Pima Road. (7/26/86)



Same location as top photo. Bedrock is 1-to 2-feet below the surface. (7/26/86)

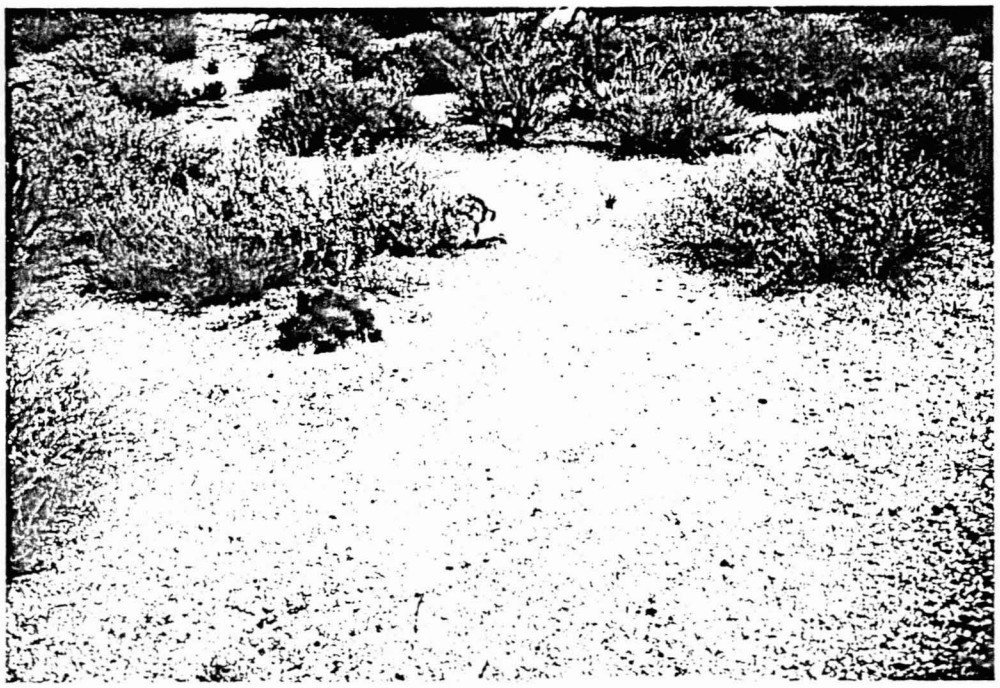
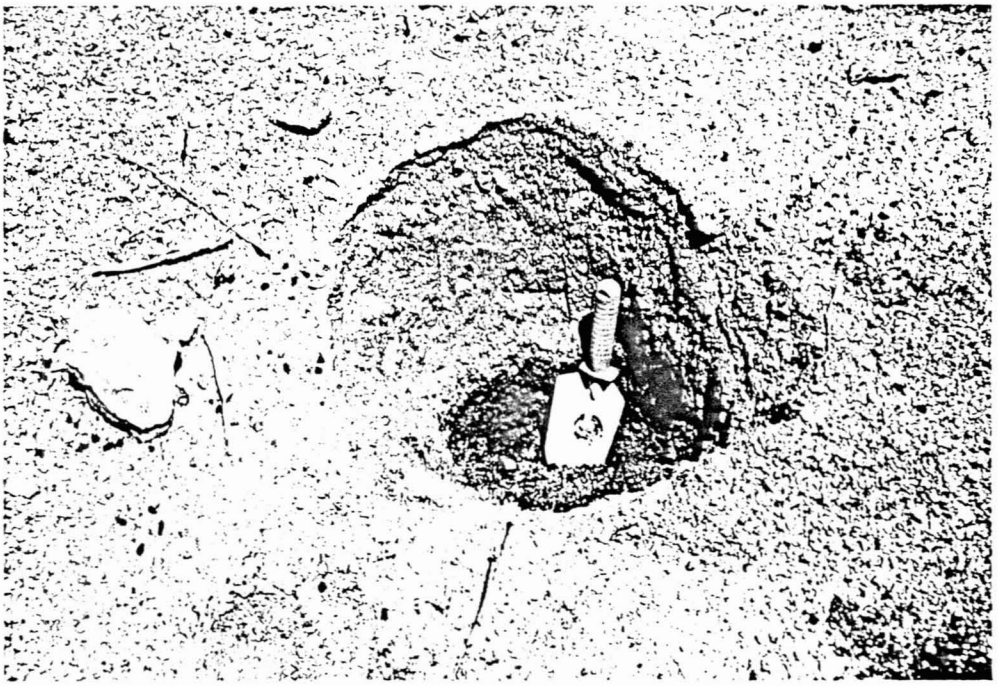


Photo of desert floor just east of Pima Road near the northern boundary of the watershed. (9/20/86)

Close-up photo of desert floor showing gravel-size particles. Same location as top photo. (9/20/86)

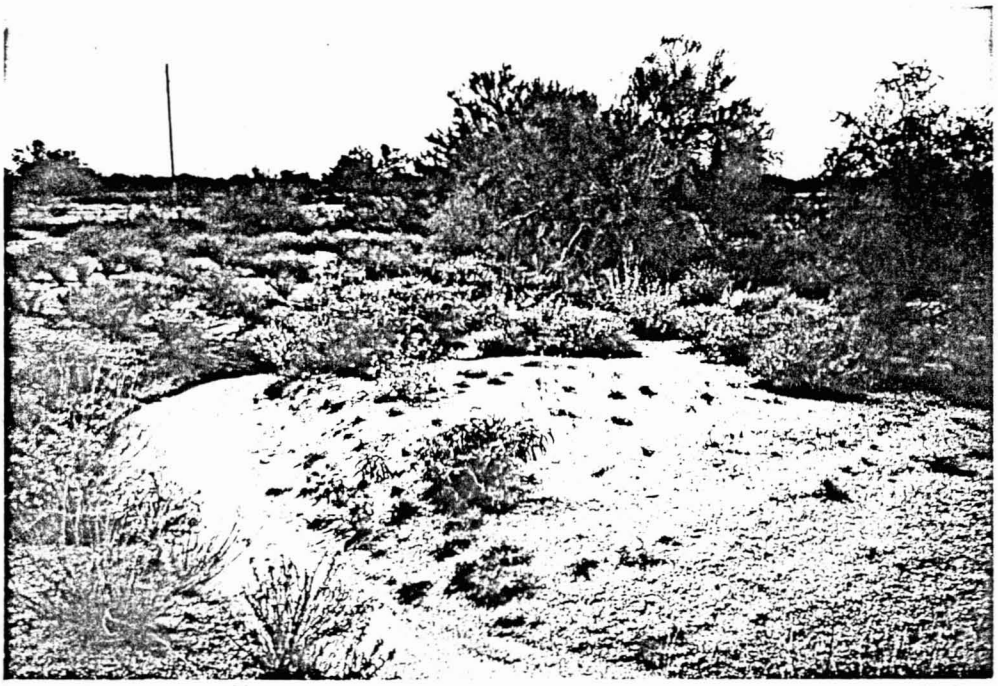
Typical sand-bed wash just west of infiltration test Site #5. (7/26/86)



Same location as top photo. A small hole has been excavated to determine the depth of sand. Dark colored material at the bottom of the hole represents the clay material that was encountered at a depth of about 9-inches. (7/26/86)

Looking upstream along a small wash just south of Pinnacle Peak Road and east of the power substation. Note fairly dense channel vegetation. (9/20/86)

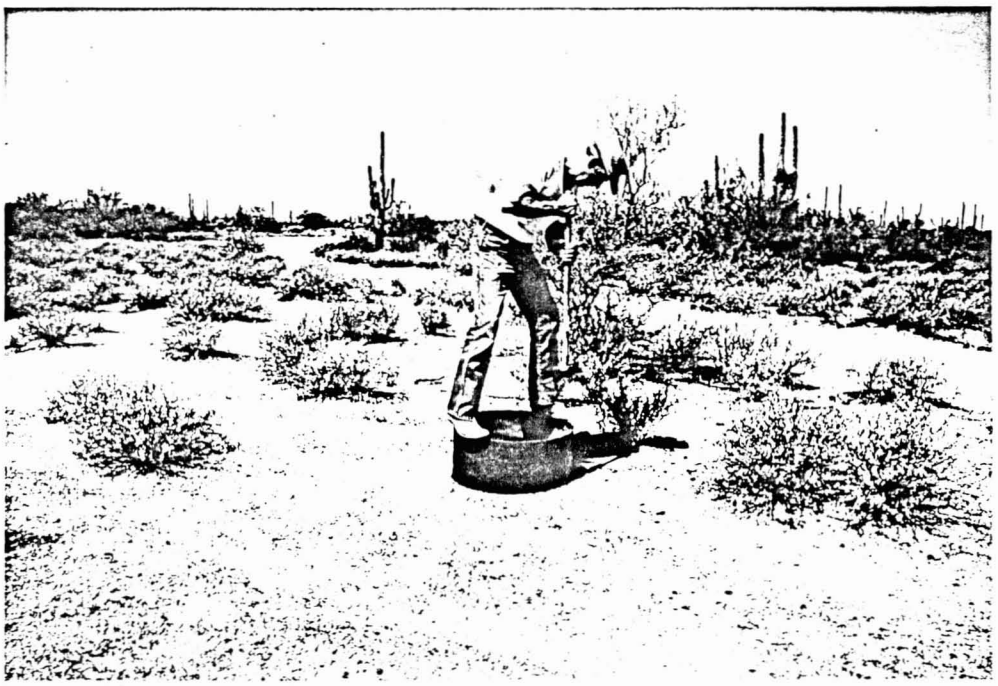
Looking downstream from same location shown in the top photo. (9/20/86)



Looking downstream at a typical sandbed rill on the alluvial fan terrace. This site is just north of Pinnacle Peak Road, approximately 1-mile west of the power substation.

Equipment used for ring infiltrometer testing. (7/26/86)

Steel rings used for infiltration tests. (7/26/86)



Using a sledgehammer to drive the outer,
24-inch diameter ring into the ground. (7/26/86)

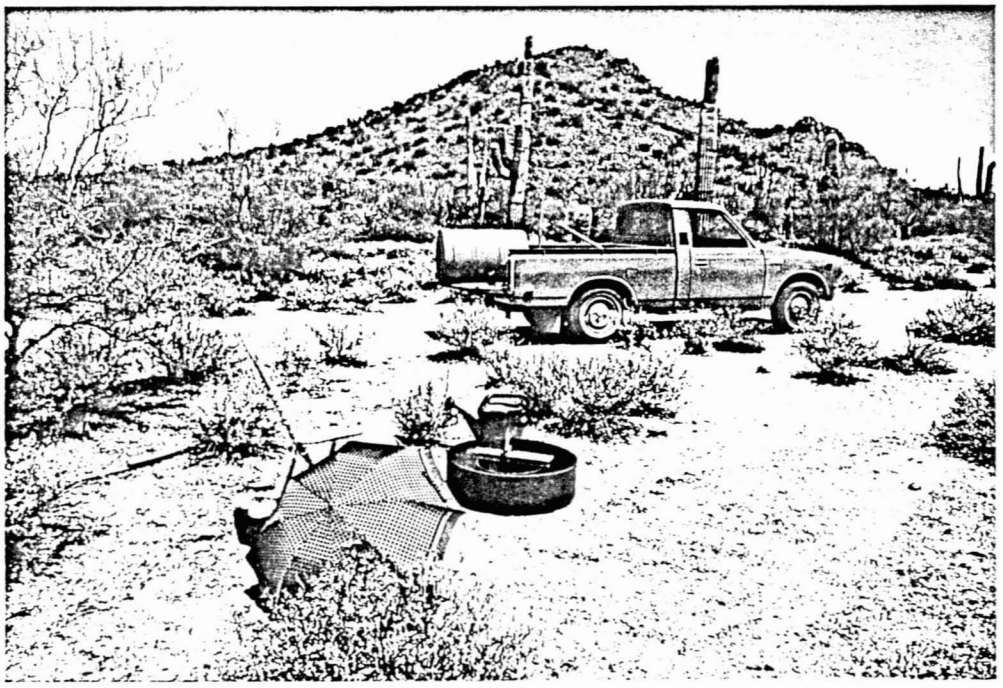
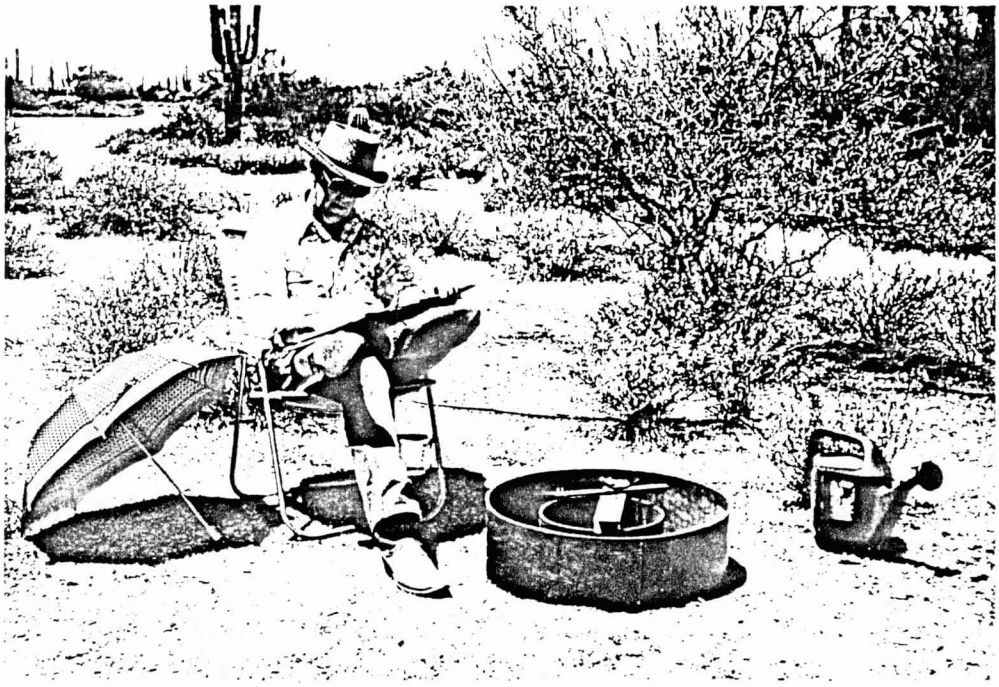
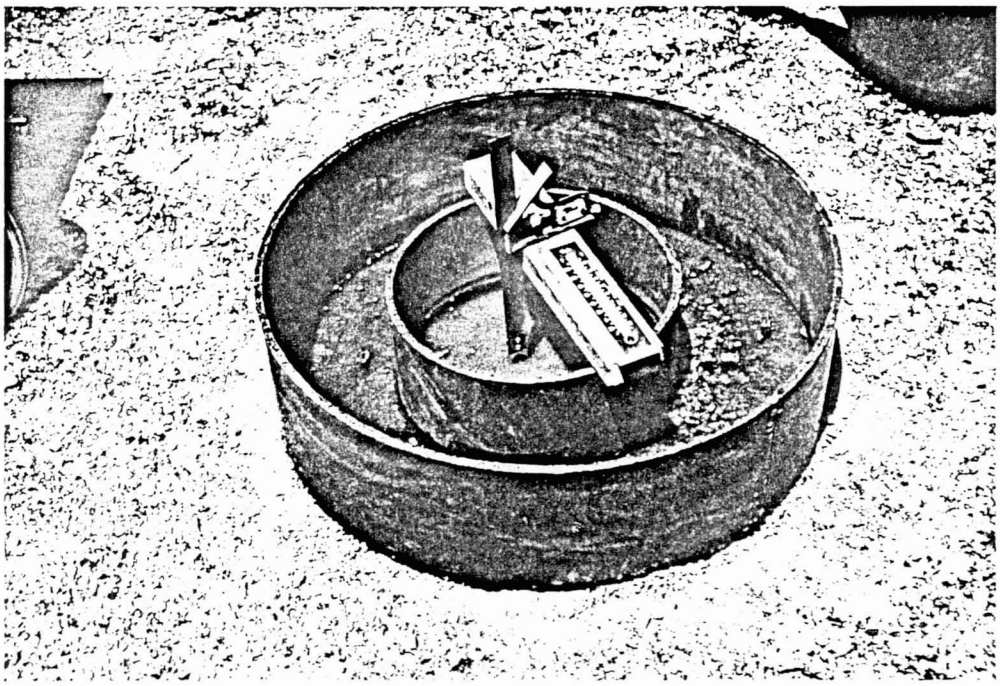


Photo showing both inner and outer rings in
place. (7/26/86)



Preparing data forms prior to initiating infiltration tests. (7/26/86)



Ring infiltration test under way at Site #5. Water has been added to both the inner and outer cylinders. (7/26/86)

# APPENDIX B

1. Sub-Drainage Area "A"

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID          ------------ SIMONS, LI & ASSOCIATES, INC.  PAZ=DC-01 ------------
  2           ID        DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
  3           ID           ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
  4           ID             100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
  5           IT     5. 12SEP86      0.    289.
  6           IO     5.      0.

  7           KK    SU81
  8           KM        RUNOFF FROM SUB-BASIN 1
  9           BA  0.3371
              *       HYPOTHETICAL STORM DISTRIBUTION
 10           PH            8.25    0.72    1.41    2.47    2.77    2.97    3.35    3.81    4.28
 11           LS     0.     79.
 12           UK    257.  0.0149    0.20    100.
 13           RK   7400.  0.0247   0.045            TRAP     50.     50.

 14           KK    SUB2
 15           KM        RUNOFF FROM SUB-BASIN 2
 16           BA  1.2170
 17           LS     0.     83.
 18           UK    217.  0.0148    0.20    100.
 19           RK  25600.  0.0195   0.060            TRAP      0.     15.

 20           KK    SUB3
 21           KM        RUNOFF FROM SUB-BASIN 3
 22           BA  0.5448
 23           LS     0.     81.
 24           UK    204.  0.0189    0.20    100.
 25           RK  17300.  0.0199   0.045            TRAP     50.     50.

 26           KK     CP4
 27           KM        COMBINE HYDROGRAPHS SUB1, SUB2, & SUB3
 28           HC     3.

 29           KK    SUB4
 30           KM        RUNOFF FROM SUB-BASIN 4
 31           BA  1.5102
 32           LS     0.     75.
 33           UK     84.  0.0213    0.10    100.
 34           RK  10633.  0.0191   0.045            TRAP     50.    100.     YES

 35           KK    SUB5
 36           KM        RUNOFF FROM SUB-BASIN 5
 37           BA  1.8713
 38           LS     0.     75.
 39           UK     81.  0.0213    0.10    100.
 40           RK  11433.  0.0175   0.045            TRAP     50.    130.     YES

 41           KK    SUB6
 42           KM        RUNOFF FROM SUB-BASIN 6
 43           BA  2.7719
 44           LS     0.     75.
 45           UK    146.  0.0213    0.10    100.
 46           RK  16100.  0.0124   0.045            TRAP     50.    200.     YES
 47           ZZ
```

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616

****

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

```
6 IO        OUTPUT CONTROL VARIABLES
                 IPRNT          5  PRINT CONTROL
                 IPLOT          0  PLOT CONTROL
                 QSCAL         0.  HYDROGRAPH PLOT SCALE

  IT        HYDROGRAPH TIME DATA
                  NMIN          5  MINUTES IN COMPUTATION INTERVAL
                 IDATE     12SEP86  STARTING DATE
                 ITIME        0000  STARTING TIME
                    NQ         289  NUMBER OF HYDROGRAPH ORDINATES
                NDDATE     13SEP86  ENDING DATE
                NDTIME        0000  ENDING TIME

            COMPUTATION INTERVAL     .08 HOURS
                 TOTAL TIME BASE   24.00 HOURS

      ENGLISH UNITS

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG       24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG       24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB1 | 323. | 12.50 | 64. | 18. | 18. | .34 | | |
| HYDROGRAPH AT | SUB2 | 766. | 12.92 | 261. | 75. | 75. | 1.22 | | |
| HYDROGRAPH AT | SUB3 | 333. | 12.92 | 109. | 31. | 31. | .54 | | |
| 3 COMBINED AT | CP4 | 1319. | 12.75 | 433. | 124. | 124. | 2.10 | | |
| HYDROGRAPH AT | SUB4 | 1589. | 12.75 | 669. | 194. | 194. | 3.61 | | |
| HYDROGRAPH AT | SUB5 | 1908. | 12.83 | 948. | 278. | 278. | 5.48 | | |
| HYDROGRAPH AT | SUB6 | 2038. | 13.67 | 1295. | 396. | 396. | 8.25 | | |

** NORMAL END OF HEC-1 ***

****
FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
****

THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
50-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB1 | 247. | 12.58 | 52. | 15. | 15. | .34 | | |
| HYDROGRAPH AT | SUB2 | 626. | 13.00 | 221. | 63. | 63. | 1.22 | | |
| HYDROGRAPH AT | SUB3 | 259. | 12.92 | 90. | 26. | 26. | .54 | | |
| 3 COMBINED AT | CP4 | 1063. | 12.83 | 364. | 104. | 104. | 2.10 | | |
| HYDROGRAPH AT | SUB4 | 1232. | 13.00 | 552. | 161. | 161. | 3.61 | | |
| HYDROGRAPH AT | SUB5 | 1440. | 13.00 | 775. | 229. | 229. | 5.48 | | |
| HYDROGRAPH AT | SUB6 | 1558. | 13.92 | 1043. | 323. | 323. | 8.25 | | |

** NORMAL END OF HEC-1 ***

2. Sub-Drainage Area "B"

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID          ------------ SIMONS, LI & ASSOCIATES, INC.  PAZ-DC-01 ------------
  2           ID        DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
  3           ID           ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
  4           ID             100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
  5           IT      5. 12SEP86      0.    289.
  6           IO      5.      0.

  7           KK    SUB7
  8           KM        RUNOFF FROM SUB-BASIN 7
  9           BA  0.6690
              *       HYPOTHETICAL STORM DISTRIBUTION
 10           PH            5.33    0.72    1.41    2.47    2.77    2.97    3.35    3.81    4.28
 11           LS      0.     79.
 12           UK    101.  0.0135    0.20  100.00
 13           RK  13200.  0.0198   0.045            TRAP     50.     50.

 14           KK    SUB8
 15           KM        RUNOFF FROM SUB-BASIN 8
 16           BA  1.0885
 17           LS      0.     75.
 18           UK    116.  0.0213    0.10    100.
 19           RK  10550.  0.0196   0.045            TRAP     50.    100.     YES

 20           KK    SUB9
 21           KM        RUNOFF FROM SUB-BASIN 9
 22           BA  1.4559
 23           LS      0.     75.
 24           UK     95.  0.0213    0.10    100.
 25           RK  10700.  0.0187   0.045            TRAP     50.    130.     YES

 26           KK   SUB10
 27           KM        RUNOFF FROM SUB-BASIN 10
 28           BA  2.1144
 29           LS      0.     75.
 30           UK    107.  0.0213    0.10    100.
 31           RK  12400.  0.0137   0.045            TRAP     50.    175.     YES
 32           ZZ
```

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616

****

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
100-YR, 24 HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

```
6 IO          OUTPUT CONTROL VARIABLES
                   IPRNT          5  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

 IT           HYDROGRAPH TIME DATA
                    NMIN          5  MINUTES IN COMPUTATION INTERVAL
                   IDATE     12SEP86  STARTING DATE
                   ITIME        0000  STARTING TIME
                      NQ         289  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE     13SEP86  ENDING DATE
                  NDTIME        0000  ENDING TIME

             COMPUTATION INTERVAL     .08 HOURS
                  TOTAL TIME BASE   24.00 HOURS

       ENGLISH UNITS

LUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG      24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG       24.00004        .01667       24.00000

LUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB7 | 529. | 12.58 | 128. | 36. | 36. | .67 | | |
| HYDROGRAPH AT | SUB8 | 941. | 12.58 | 304. | 88. | 88. | 1.76 | | |
| HYDROGRAPH AT | SUB9 | 1375. | 12.67 | 534. | 155. | 155. | 3.21 | | |
| HYDROGRAPH AT | SUB10 | 1678. | 13.00 | 846. | 251. | 251. | 5.33 | | |

*** NORMAL END OF HEC-1 ***

T10H50.DAT

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
50-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB7 | 399. | 12.67 | 105. | 30. | 30. | .67 | | |
| HYDROGRAPH AT | SUB8 | 706. | 12.67 | 247. | 71. | 71. | 1.76 | | |
| HYDROGRAPH AT | SUB9 | 1028. | 12.67 | 431. | 126. | 126. | 3.21 | | |
| HYDROGRAPH AT | SUB10 | 1259. | 13.08 | 680. | 203. | 203. | 5.33 | | |

*** NORMAL END OF HEC-1 ***

3. Sub-Drainage Area "C"

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID          ------------ SIMONS, LI & ASSOCIATES, INC.  PAZ-DC-01 ------------
  2           ID        DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
  3           ID           ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
  4           ID             100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
  5           IT      5. 12SEP86      0.    289.
  6           IO      5.      0.

  7           KK   SUB11
  8           KM        RUNOFF FROM SUB-BASIN 11
  9           BA  0.5865
              *       HYPOTHETICAL STORM DISTRIBUTION
 10           PH            4.26    0.72    1.41    2.47    2.77    2.97    3.35    3.81    4.28
 11           LS      0.     75.
 12           UK    104.  0.0234    0.20    100.
 13           RK   7200.  0.0219   0.045            TRAP     50.     50.

 14           KK   SUB12
 15           KM        RUNOFF FROM SUB-BASIN 12
 16           BA  1.3317
 17           LS      0.     75.
 18           UK     95.  0.0213    0.10    100.
 19           RK   9850.  0.0214   0.045            TRAP     50.    100.     YES

 20           KK   SUB13
 21           KM        RUNOFF FROM SUB-BASIN 13
 22           BA  1.2211
 23           LS      0.     75.
 24           UK     86.  0.0213    0.10    100.
 25           RK  10533.  0.0192   0.045            TRAP     50.    130.     YES

 26           KK   SUB14
 27           KM        RUNOFF FROM SUB-BASIN 14
 28           BA  1.1209
 29           LS      0.     75.
 30           UK    101.  0.0213    0.10    100.
 31           RK  11200.  0.0142   0.045            TRAP     50.    175.     YES
 32           ZZ
```

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
****

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

```
6 IO        OUTPUT CONTROL VARIABLES
                 IPRNT          5  PRINT CONTROL
                 IPLOT          0  PLOT CONTROL
                 QSCAL         0.  HYDROGRAPH PLOT SCALE

  IT        HYDROGRAPH TIME DATA
                  NMIN          5  MINUTES IN COMPUTATION INTERVAL
                 IDATE     12SEP86  STARTING DATE
                 ITIME        0000  STARTING TIME
                    NQ         289  NUMBER OF HYDROGRAPH ORDINATES
                NDDATE     13SEP86  ENDING DATE
                NDTIME        0000  ENDING TIME

            COMPUTATION INTERVAL     .08 HOURS
                 TOTAL TIME BASE   24.00 HOURS

      ENGLISH UNITS

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB11 | 631. | 12.42 | 97. | 27. | 27. | .59 | | |
| HYDROGRAPH AT | SUB12 | 1338. | 12.50 | 318. | 91. | 91. | 1.92 | | |
| HYDROGRAPH AT | SUB13 | 1527. | 12.75 | 510. | 147. | 147. | 3.14 | | |
| HYDROGRAPH AT | SUB14 | 1445. | 13.33 | 665. | 196. | 196. | 4.26 | | |

*** NORMAL END OF HEC-1 ***

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
50-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB11 | 479. | 12.42 | 79. | 22. | 22. | .59 | | |
| HYDROGRAPH AT | SUB12 | 1014. | 12.58 | 258. | 74. | 74. | 1.92 | | |
| HYDROGRAPH AT | SUB13 | 1135. | 12.83 | 412. | 120. | 120. | 3.14 | | |
| HYDROGRAPH AT | SUB14 | 1082. | 13.42 | 535. | 159. | 159. | 4.26 | | |

*** NORMAL END OF HEC-1 ***

4. Sub-Drainage Area "D"

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID          ------------ SIMONS, LI & ASSOCIATES, INC.  PAZ-DC-01 ------------
  2           ID        DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
  3           ID           ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
  4           ID             100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
  5           ID                            TOTAL AREA = 18.09 SQ MI
  6           IT      5. 12SEP86      0.    289.
  7           IO      5.      0.

  8           KK   SUB16
  9           KM        RUNOFF FROM SUB-BASIN 16
 10           BA  1.4809
              *       HYPOTHETICAL STORM DISTRIBUTION (PRECIP VALUES WITH AREAL REDUCTION)
 11           PH           18.09    0.72    1.41    2.47    2.77    2.97    3.35    3.81    4.28
 12           LS      0.     83.
 13           UK    242.  0.0147    0.20    100.
 14           RK  16500.  0.0214   0.045            TRAP     15.      2.

 15           KK   SUB17
 16           KM        RUNOFF FROM SUB-BASIN 17
 17           BA  1.1459
 18           LS      0.     83.
 19           UK    275.  0.0519    0.20    100.
 20           RK  10400.  0.0283   0.045            TRAP     15.      2.

 21           KK   SUB18
 22           KM        RUNOFF FROM SUB-BASIN 18
 23           BA  1.2670
 24           LS      0.     83.
 25           UK    322.  0.0962    0.20    100.
 26           RK  10400.  0.0175   0.045            TRAP     15.      2.

 27           KK   CP19A
 28           KM        COMBINE HYDROGRAPHS SUB16, SUB17, & SUB18
 29           HC      3.

 30           KK   CP19B
 31           KM        ROUTE CP19A TO CP19B
 32           RK   1100.  0.0155   0.045            TRAP     15.      2.

 33           KK   SUB15
 34           KM        RUNOFF FROM SUB-BASIN 15
 35           BA  1.3327
 36           LS      0.     83.
 37           UK    168.  0.0279    0.20    100.
 38           RK  23100.  0.0232   0.045            TRAP     15.      2.

 39           KK   CP19C
 40           KM        COMBINE HYDROGRAPHS SUB15 & CP19B
 41           HC      2.
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  42          KK   SUB19
  43          KM        RUNOFF FROM SUB-BASIN 19
  44          BA  0.1774
  45          LS      0.     85.
  46          UK    222.  0.0110    0.20    100.
  47          RK   1600.  0.0188   0.030           TRAP     50.     30.     YES

  48          KK   SUB20
  49          KM        RUNOFF FROM SUB-BASIN 20
  50          BA  0.5698
  51          LS      0.     85.
  52          UK    501.  0.0630    0.20    100.
  53          RK  10800.  0.0191   0.045           TRAP     15.      2.

  54          KK    CP21
  55          KM        COMBINE HYDROGRAPHS SUB19 & SUB20
  56          HC      2.

  57          KK   SUB21
  58          KM        RUNOFF FROM SUB-BASIN 21
  59          BA  3.6841
  60          LS      0.     81.
  61          UK    201.  0.0235    0.20    100.
  62          RK  29400.  0.0190   0.030           TRAP     50.     30.     YES

  63          KK   SUB22
  64          KM        RUNOFF FROM SUB-BASIN 22
  65          BA  1.3860
  66          LS      0.     85.
  67          UK    138.  0.0221    0.20    100.
  68          RK  14300.  0.0241    0.06           TRAP     6.5      8.

  69          KK   SUB23
  70          KM        RUNOFF FROM SUB-BASIN 23
  71          BA  0.3945
  72          LS      0.     85.
  73          UK    186.  0.0188    0.20    100.
  74          RK  10800.  0.0319    0.06           TRAP     6.5      8.

  75          KK    CP24
  76          KM        COMBINE HYDROGRAPHS SUB22 & SUB23
  77          HC      2.

  78          KK   SUB24
  79          KM        RUNOFF FROM SUB-BASIN 24
  80          BA  0.4550
  81          LS      0.     80.
  82          UK    125.  0.0127    0.20    100.
  83          RK   7000.  0.0246   0.045           TRAP     15.      2.     YES
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  84          KK   SUB25
  85          KM        RUNOFF FROM SUB-BASIN 25
  86          BA  1.5216
  87          LS      0.     85.
  88          UK    429.  0.0197    0.20    100.
  89          RK  17800.  0.0307    0.06            TRAP     6.5      8.

  90          KK    CP26
  91          KM        COMBINE HYDROGRAPHS SUB24 & SUB25
  92          HC      2.

  93          KK   SUB26
  94          KM        RUNOFF FROM SUB-BASIN 26
  95          BA  0.6095
  96          LS      0.     76.
  97          UK    149.  0.0129    0.20    100.
  98          RK  11700.  0.0232   0.050            TRAP      0.     65.     YES

  99          KK    CP27
 100          KM        COMBINE HYDROGRAPHS SUB21 & SUB26
 101          HC      2.

 102          KK   SUB27
 103          KM        RUNOFF FROM SUB-BASIN 27
 104          BA  0.7639
 105          LS      0.     75.
 106          UK    159.  0.0213    0.10    100.
 107          RK   3200.  0.0310   0.045            TRAP   1500.      2.     YES

 108          KK   SUB28
 109          KM        RUNOFF FROM SUB-BASIN 28
 110          BA  1.4235
 111          LS      0.     75.
 112          UK     96.  0.0213    0.10    100.
 113          RK   9333.  0.0303   0.045            TRAP   2500.      2.     YES

 114          KK   SUB29
 115          KM        RUNOFF FROM SUB-BASIN 29
 116          BA  1.8786
 117          LS      0.     75.
 118          UK     88.  0.0213    0.10    100.
 119          RK  10900.  0.0147   0.045            TRAP   4000.      2.     YES
 120          ZZ
```

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
****

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
TOTAL AREA = 18.09 SQ MI

```
7 IO          OUTPUT CONTROL VARIABLES
                   IPRNT          5  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

  IT          HYDROGRAPH TIME DATA
                    NMIN          5  MINUTES IN COMPUTATION INTERVAL
                   IDATE     12SEP86  STARTING DATE
                   ITIME        0000  STARTING TIME
                      NQ         289  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE     13SEP86  ENDING DATE
                  NDTIME        0000  ENDING TIME

             COMPUTATION INTERVAL     .08 HOURS
                 TOTAL TIME BASE   24.00 HOURS

        ENGLISH UNITS

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG         24.00004        .01667       24.00000
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB16 | 1929. | 12.42 | 315. | 90. | 90. | 1.48 | | |
| HYDROGRAPH AT | SUB17 | 2350. | 12.25 | 246. | 70. | 70. | 1.15 | | |
| HYDROGRAPH AT | SUB18 | 2662. | 12.25 | 272. | 77. | 77. | 1.27 | | |
| 3 COMBINED AT | CP19A | 6397. | 12.25 | 832. | 237. | 237. | 3.89 | | |
| ROUTED TO | CP19B | 6311. | 12.33 | 833. | 237. | 237. | 3.89 | | |
| HYDROGRAPH AT | SUB15 | 1477. | 12.33 | 281. | 80. | 80. | 1.33 | | |
| 2 COMBINED AT | CP19C | 7788. | 12.33 | 1114. | 317. | 317. | 5.23 | | |
| HYDROGRAPH AT | SUB19 | 7863. | 12.33 | 1155. | 329. | 329. | 5.40 | | |
| HYDROGRAPH AT | SUB20 | 901. | 12.33 | 130. | 37. | 37. | .57 | | |
| 2 COMBINED AT | CP21 | 8764. | 12.33 | 1285. | 366. | 366. | 5.97 | | |
| HYDROGRAPH AT | SUB21 | 7225. | 12.75 | 1979. | 568. | 568. | 9.66 | | |
| HYDROGRAPH AT | SUB22 | 1813. | 12.33 | 318. | 91. | 91. | 1.39 | | |
| HYDROGRAPH AT | SUB23 | 530. | 12.42 | 91. | 26. | 26. | .39 | | |
| 2 COMBINED AT | CP24 | 2321. | 12.33 | 409. | 117. | 117. | 1.78 | | |
| HYDROGRAPH AT | SUB24 | 2651. | 12.42 | 501. | 142. | 142. | 2.24 | | |
| HYDROGRAPH AT | SUB25 | 1633. | 12.58 | 350. | 100. | 100. | 1.52 | | |
| 2 COMBINED AT | CP26 | 4193. | 12.50 | 851. | 242. | 242. | 3.76 | | |
| HYDROGRAPH AT | SUB26 | 3569. | 12.83 | 937. | 267. | 267. | 4.37 | | |
| 2 COMBINED AT | CP27 | 10681. | 12.75 | 2915. | 835. | 835. | 14.02 | | |
| HYDROGRAPH AT | SUB27 | 10712. | 12.83 | 3026. | 867. | 867. | 14.79 | | |
| HYDROGRAPH AT | SUB28 | 9978. | 13.17 | 3114. | 907. | 907. | 16.21 | | |
| HYDROGRAPH AT | SUB29 | 7339. | 13.92 | 3125. | 940. | 940. | 18.09 | | |

*** NORMAL END OF HEC-1 ***

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
50-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
TOTAL AREA = 18.09 SQ MI

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB16 | 1509. | 12.42 | 265. | 75. | 75. | 1.48 | | |
| HYDROGRAPH AT | SUB17 | 1776. | 12.33 | 207. | 59. | 59. | 1.15 | | |
| HYDROGRAPH AT | SUB18 | 2076. | 12.25 | 230. | 65. | 65. | 1.27 | | |
| 3 COMBINED AT | CP19A | 5033. | 12.33 | 701. | 199. | 199. | 3.89 | | |
| ROUTED TO | CP19B | 4989. | 12.33 | 702. | 199. | 199. | 3.89 | | |
| HYDROGRAPH AT | SUB15 | 1172. | 12.42 | 237. | 68. | 68. | 1.33 | | |
| 2 COMBINED AT | CP19B | 6125. | 12.33 | 939. | 267. | 267. | 5.23 | | |
| HYDROGRAPH AT | SUB19 | 6041. | 12.33 | 974. | 276. | 276. | 5.40 | | |
| HYDROGRAPH AT | SUB20 | 687. | 12.42 | 111. | 31. | 31. | .57 | | |
| 2 COMBINED AT | CP21 | 6720. | 12.33 | 1084. | 308. | 308. | 5.97 | | |
| HYDROGRAPH AT | SUB21 | 5628. | 12.83 | 1652. | 475. | 475. | 9.66 | | |
| HYDROGRAPH AT | SUB22 | 1539. | 12.42 | 271. | 77. | 77. | 1.39 | | |
| HYDROGRAPH AT | SUB23 | 425. | 12.42 | 77. | 22. | 22. | .39 | | |
| 2 COMBINED AT | CP24 | 1964. | 12.42 | 348. | 99. | 99. | 1.78 | | |
| HYDROGRAPH AT | SUB24 | 2223. | 12.50 | 422. | 120. | 120. | 2.24 | | |
| HYDROGRAPH AT | SUB25 | 1297. | 12.58 | 296. | 84. | 84. | 1.52 | | |
| 2 COMBINED AT | | | | | | | | | |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| + | CP26 | 3438. | 12.50 | 718. | 204. | 204. | 3.76 |
| HYDROGRAPH AT | | | | | | | |
| + | SUB26 | 2879. | 12.92 | 784. | 224. | 224. | 4.37 |
| 2 COMBINED AT | | | | | | | |
| + | CP27 | 8453. | 12.83 | 2435. | 699. | 699. | 14.02 |
| HYDROGRAPH AT | | | | | | | |
| | SUB27 | 8476. | 12.92 | 2519. | 725. | 725. | 14.79 |
| HYDROGRAPH AT | | | | | | | |
| | SUB28 | 7719. | 13.33 | 2575. | 754. | 754. | 16.21 |
| HYDROGRAPH AT | | | | | | | |
| | SUB29 | 5602. | 14.08 | 2556. | 774. | 774. | 18.09 |

*** NORMAL END OF HEC-1 ***

5. Sub-Drainage Area "E"

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID          ------------ SIMONS, LI & ASSOCIATES, INC.  PAZ-DC-01 ------------
  2           ID        DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
  3           ID           ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
  4           ID             100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
  5           IT      5. 12SEP86      0.    289.
  6           IO      5.      0.

  7           KK   SUB30
  8           KM        RUNOFF FROM SUB-BASIN 30
  9           BA  1.3296
              *       HYPOTHETICAL STORM DISTRIBUTION
 10           PH            5.53    0.72    1.41    2.47    2.77    2.97    3.35    3.81    4.28
 11           LS      0.     81.
 12           UK    165.  0.1211    0.20    100.
 13           RK  11800.  0.0355   0.045            TRAP     50.    130.

 14           KK   SUB31
 15           KM        RUNOFF FROM SUB-BASIN 31
 16           BA  1.2325
 17           LS      0.     75.
 18           UK    102.  0.0213    0.10    100.
 19           RK   9450.  0.0228   0.045            TRAP     50.    130.     YES

 20           KK   SUB32
 21           KM        RUNOFF FROM SUB-BASIN 32
 22           BA  1.3985
 23           LS      0.     75.
 24           UK     67.  0.0213    0.10    100.
 25           RK   9300.  0.0214   0.045            TRAP     50.    130.     YES

 26           KK   SUB33
 27           KM        RUNOFF FROM SUB-BASIN 33
 28           BA  1.5717
 29           LS      0.     75.
 30           UK     89.  0.0213    0.10    100.
 31           RK   9500.  0.0161   0.045            TRAP     50.    175.     YES
 32           ZZ
```

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616

****

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

```
6 IO          OUTPUT CONTROL VARIABLES
                   IPRNT          5  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

  IT          HYDROGRAPH TIME DATA
                    NMIN          5  MINUTES IN COMPUTATION INTERVAL
                   IDATE     12SEP86  STARTING DATE
                   ITIME        0000  STARTING TIME
                      NQ         289  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE     13SEP86  ENDING DATE
                  NDTIME        0000  ENDING TIME

             COMPUTATION INTERVAL     .08 HOURS
                  TOTAL TIME BASE   24.00 HOURS

        ENGLISH UNITS

.LUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG       24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

.LUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB30 | 1570. | 12.42 | 279. | 79. | 79. | 1.33 | | |
| HYDROGRAPH AT | SUB31 | 1904. | 12.67 | 480. | 136. | 136. | 2.56 | | |
| HYDROGRAPH AT | SUB32 | 2115. | 12.83 | 702. | 202. | 202. | 3.96 | | |
| HYDROGRAPH AT | SUB33 | 2228. | 13.08 | 939. | 273. | 273. | 5.53 | | |

*** NORMAL END OF HEC-1 ***

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
50-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB30 | 1244. | 12.42 | 233. | 66. | 66. | 1.33 | | |
| HYDROGRAPH AT | SUB31 | 1473. | 12.67 | 395. | 112. | 112. | 2.56 | | |
| HYDROGRAPH AT | SUB32 | 1593. | 12.92 | 573. | 165. | 165. | 3.96 | | |
| HYDROGRAPH AT | SUB33 | 1675. | 13.25 | 762. | 223. | 223. | 5.53 | | |

*** NORMAL END OF HEC-1 ***

6. Sub-Drainage Area "F"

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID          ------------ SIMONS, LI & ASSOCIATES, INC.  PAZ-DC-01 ------------
  2           ID        DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
  3           ID           ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
  4           ID             100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
  5           IT      5. 12SEP86      0.    289.
  6           IO      5.      0.

  7           KK   SUB34
  8           KM        RUNOFF FROM SUB-BASIN 34
  9           BA  0.8433
              *       HYPOTHETICAL STORM DISTRIBUTION
 10           PH            6.03    0.72    1.41    2.47    2.77    2.97    3.35    3.81    4.28
 11           LS      0.     81.
 12           UK    178.  0.0984    0.20    100.
 13           RK   7400.  0.0587   0.045            TRAP     50.    100.

 14           KK   SUB35
 15           KM        RUNOFF FROM SUB-BASIN 35
 16           BA  1.2879
 17           LS      0.     80.
 18           UK    247.  0.0149    0.20  100.00
 19           RK  14700.  0.0328   0.045            TRAP     50.     50.

 20           KK    CP36
 21           KM        COMBINE HYDROGRAPHS SUB34 & SUB35
 22           HC      2.

 23           KK   SUB36
 24           KM        RUNOFF FROM SUB-BASIN 36
 25           BA  1.4538
 26           LS      0.     75.
 27           UK    116.  0.0213    0.10    100.
 28           RK   7767.  0.0281   0.045            TRAP    500.      2.     YES

 29           KK   SUB37
 30           KM        RUNOFF FROM SUB-BASIN 37
 31           BA  1.1896
 32           LS      0.     75.
 33           UK     61.  0.0213    0.10    100.
 34           RK   8433.  0.0237   0.045            TRAP   1000.      2.     YES

 35           KK   SUB38
 36           KM        RUNOFF FROM SUB-BASIN 38
 37           BA  1.2597
 38           LS      0.     75.
 39           UK     84.  0.0213    0.10    100.
 40           RK   8300.  0.0176   0.045            TRAP   1500.      2.     YES
 41           ZZ
```

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616

****

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

```
6 IO          OUTPUT CONTROL VARIABLES
                    IPRNT          5  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE

  IT          HYDROGRAPH TIME DATA
                     NMIN          5  MINUTES IN COMPUTATION INTERVAL
                    IDATE    12SEP86  STARTING DATE
                    ITIME       0000  STARTING TIME
                       NQ        289  NUMBER OF HYDROGRAPH ORDINATES
                   NDDATE    13SEP86  ENDING DATE
                   NDTIME       0000  ENDING TIME

              COMPUTATION INTERVAL     .08 HOURS
                  TOTAL TIME BASE   24.00 HOURS

        ENGLISH UNITS

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG       24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG       24.00004        .01667       24.00000

ALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG       24.00004        .01667       24.00000
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB34 | 1507. | 12.25 | 176. | 50. | 50. | .84 | | |
| HYDROGRAPH AT | SUB35 | 1128. | 12.58 | 253. | 72. | 72. | 1.29 | | |
| 2 COMBINED AT | CP36 | 2219. | 12.33 | 428. | 122. | 122. | 2.13 | | |
| HYDROGRAPH AT | SUB36 | 3224. | 12.42 | 660. | 188. | 188. | 3.58 | | |
| HYDROGRAPH AT | SUB37 | 2981. | 12.75 | 821. | 239. | 239. | 4.77 | | |
| HYDROGRAPH AT | SUB38 | 2570. | 13.17 | 965. | 288. | 288. | 6.03 | | |

*** NORMAL END OF HEC-1 ***

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
50-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB34 | 1178. | 12.25 | 147. | 41. | 41. | .84 | | |
| HYDROGRAPH AT | SUB35 | 923. | 12.67 | 211. | 60. | 60. | 1.29 | | |
| 2 COMBINED AT | CP36 | 1703. | 12.33 | 357. | 101. | 101. | 2.13 | | |
| HYDROGRAPH AT | SUB36 | 2376. | 12.50 | 543. | 155. | 155. | 3.58 | | |
| HYDROGRAPH AT | SUB37 | 2154. | 12.83 | 669. | 196. | 196. | 4.77 | | |
| HYDROGRAPH AT | SUB38 | 1861. | 13.33 | 776. | 234. | 234. | 6.03 | | |

*** NORMAL END OF HEC-1 ***

7. Sub-Drainage Area "G"

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID          ------------ SIMONS, LI & ASSOCIATES, INC.  PAZ-DC-01 ------------
  2           ID        DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
  3           ID           ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
  4           ID             100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
  5           ID                              TOTAL AREA = 12.37 SQ MI
  6           IT      5. 12SEP86      0.    289.
  7           IO      5.      0.

  8           KK   SUB39
  9           KM        RUNOFF FROM SUB-BASIN 39
 10           BA  1.9422
              *        HYPOTHETICAL STORM DISTRIBUTION
 11           PH           12.37    0.72    1.41    2.47    2.77    2.97    3.35    3.81    4.28
 12           LS      0.     85.
 13           UK    399.  0.0315    0.20    100.
 14           RK  13700.  0.0251    0.06            TRAP     6.5      8.

 15           KK   SUB40
 16           KM        RUNOFF FROM SUB-BASIN 40
 17           BA  0.5844
 18           LS      0.     83.
 19           UK    567.  0.0791    0.20    100.
 20           RK   8200.  0.0335    0.06            TRAP     6.5      8.

 21           KK    CP41
 22           KM        COMBINE HYDROGRAPHS SUB39 & SUB40
 23           HC      2.

 24           KK   SUB41
 25           KM        RUNOFF FROM SUB-BASIN 41
 26           BA  0.7911
 27           LS      0.     78.
 28           UK    322.  0.0845    0.20    100.
 29           RK   9700.  0.0263   0.026            TRAP     40.     55.     YES

 30           KK   SUB42
 31           KM        RUNOFF FROM SUB-BASIN 42
 32           BA  0.5844
 33           LS      0.     80.
 34           UK    150.  0.1472    0.20    100.
 35           RK   9300.  0.0571    0.06            TRAP     6.5      8.

 36           KK   SUB43
 37           KM        RUNOFF FROM SUB-BASIN 43
 38           BA  1.0708
 39           LS      0.     86.
 40           UK    272.  0.0313    0.20    100.
 41           RK   7500.  0.0257    0.06            TRAP     6.5      8.

 42           KK   SUB44
 43           KM        RUNOFF FROM SUB-BASIN 44
 44           BA  0.8537
 45           LS      0.     85.
 46           UK    295.  0.0573    0.20    100.
 47           RK  10600.  0.0674    0.06            TRAP     6.5      8.
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  48          KK    CP45
  49          KM        COMBINE HYDROGRAPHS SUB43 & SUB44
  50          HC      2.

  51          KK   SUB45
  52          KM        RUNOFF FROM SUB-BASIN 45
  53          BA  0.3757
  54          LS      0.     85.              0.     85.
  55          UK   1700.  0.2059    0.20   31.94
  56          UK   1150.  0.3304    0.20   68.06
  57          RK   3600.  0.0369   0.045           TRAP     15.      2.     YES

  58          KK   SUB46
  59          KM        RUNOFF FROM SUB-BASIN 46
  60          BA  0.9268
  61          LS      0.     86.              0.     86.
  62          UK    800.  0.3000    0.20   19.38
  63          UK   2800.  0.3982    0.20   80.62
  64          RK  11400.  0.0813   0.045           TRAP     15.      2.

  65          KK    CP47
  66          KM        COMBINE HYDROGRAPHS SUB45 & SUB46
  67          HC      2.

  68          KK   SUB47
  69          KM        RUNOFF FROM SUB-BASIN 47
  70          BA  0.3757
  71          LS      0.     80.
  72          UK    300.  0.0933    0.20    100.
  73          RK   7600.  0.0267   0.066           TRAP      0.     20.     YES

  74          KK    CP48
  75          KM        COMBINE HYDROGRAPHS SUB41, SUB42, & SUB47
  76          HC      3.

  77          KK   SUB48
  78          KM        RUNOFF FROM SUB-BASIN 48
  79          BA  0.0438
  80          LS      0.     81.
  81          UK    313.  0.0455    0.20    100.
  82          RK   1200.  0.0358   0.066           TRAP      0.     20.     YES

  83          KK   SUB49
  84          KM        RUNOFF FROM SUB-BASIN 49
  85          BA  0.3528
  86          LS      0.     81.
  87          UK     79.  0.0149    0.20    100.
  88          RK   6400.  0.0370   0.045           TRAP     50.     50.
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 89           KK   SUB50
 90           KM        RUNOFF FROM SUB-BASIN 50
 91           BA  0.3580
 92           LS      0.     85.              0.     85.
 93           UK    800.  0.0875    0.20   38.77
 94           UK    950.  0.4842    0.30   61.23
 95           RK   8600.  0.0473   0.045           TRAP     15.     2.

 96           KK    CP51
 97           KM        COMBINE HYDROGRAPHS SUB48, SUB49, & SUB50
 98           HC      3.

 99           KK   SUB51
100           KM        RUNOFF FROM SUB-BASIN 51
101           BA  1.0090
102           LS      0.     76.
103           UK     94.  0.0213    0.10    100.
104           RK   6400.  0.0329   0.045           TRAP   1500.     2.     YES

105           KK   SUB52
106           KM        RUNOFF FROM SUB-BASIN 52
107           BA  1.3557
108           LS      0.     75.
109           UK     76.  0.0213    0.10    100.
110           RK   6633.  0.0300   0.045           TRAP   2500.     2.     YES

111           KK   SUB53
112           KM        RUNOFF FROM SUB-BASIN 53
113           BA  1.7471
114           LS      0.     75.
115           UK     67.  0.0213    0.10    100.
116           RK   7767.  0.0196   0.045           TRAP   4000.     2.     YES
117           ZZ
```

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
****

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
100-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
TOTAL AREA = 12.37 SQ MI

```
 7 IO          OUTPUT CONTROL VARIABLES
                    IPRNT          5  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT          HYDROGRAPH TIME DATA
                     NMIN          5  MINUTES IN COMPUTATION INTERVAL
                    IDATE     12SEP86  STARTING DATE
                    ITIME       0000  STARTING TIME
                       NQ        289  NUMBER OF HYDROGRAPH ORDINATES
                   NDDATE     13SEP86  ENDING DATE
                   NDTIME       0000  ENDING TIME

                COMPUTATION INTERVAL     .08 HOURS
                     TOTAL TIME BASE   24.00 HOURS

         ENGLISH UNITS

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

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VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000

VALUE EXCEEDS TABLE IN LOGLOG        24.00004        .01667       24.00000
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT | SUB39 | 2467. | 12.42 | 449. | 128. | 128. | 1.94 | | |
| HYDROGRAPH AT | SUB40 | 856. | 12.33 | 127. | 36. | 36. | .58 | | |
| 2 COMBINED AT | CP41 | 3292. | 12.42 | 576. | 164. | 164. | 2.53 | | |
| HYDROGRAPH AT | SUB41 | 3851. | 12.50 | 720. | 204. | 204. | 3.32 | | |
| HYDROGRAPH AT | SUB42 | 1054. | 12.25 | 116. | 33. | 33. | .58 | | |
| HYDROGRAPH AT | SUB43 | 1979. | 12.33 | 256. | 73. | 73. | 1.07 | | |
| HYDROGRAPH AT | SUB44 | 1692. | 12.25 | 199. | 57. | 57. | .85 | | |
| 2 COMBINED AT | CP45 | 3671. | 12.25 | 455. | 129. | 129. | 1.92 | | |
| HYDROGRAPH AT | SUB45 | 4027. | 12.33 | 545. | 155. | 155. | 2.30 | | |
| HYDROGRAPH AT | SUB46 | 1516. | 12.25 | 226. | 64. | 64. | .93 | | |
| 2 COMBINED AT | CP47 | 5510. | 12.33 | 771. | 219. | 219. | 3.23 | | |
| HYDROGRAPH AT | SUB47 | 5360. | 12.42 | 843. | 239. | 239. | 3.60 | | |
| 3 COMBINED AT | CP48 | 9681. | 12.42 | 1678. | 475. | 475. | 7.50 | | |
| HYDROGRAPH AT | SUB48 | 9650. | 12.50 | 1687. | 477. | 477. | 7.55 | | |
| HYDROGRAPH AT | SUB49 | 539. | 12.33 | 72. | 20. | 20. | .35 | | |
| HYDROGRAPH AT | SUB50 | 642. | 12.25 | 84. | 24. | 24. | .36 | | |
| 3 COMBINED AT | CP51 | 10368. | 12.50 | 1841. | 522. | 522. | 8.26 | | |
| HYDROGRAPH AT | SUB51 | 10866. | 12.58 | 1982. | 562. | 562. | 9.27 | | |
| HYDROGRAPH AT | SUB52 | 9961. | 12.75 | 2130. | 613. | 613. | 10.62 | | |
| HYDROGRAPH AT | SUB53 | 7202. | 13.17 | 2267. | 671. | 671. | 12.37 | | |

*** NORMAL END OF HEC-1 ***

---------- SIMONS, LI & ASSOCIATES, INC. PAZ-DC-01 ----------
DELEUW, CATHER & CO. - OUTER LOOP HIGHWAY PROJECT CONCEPT DRAINAGE PLAN
ALL SUBBASINS TO OUTER LOOP HWY, EXISTING RUNOFF CHARACTERISTICS
50-YR, 24-HR RAINFALL USING HYPOTHETICAL STORM DISTRIBUTION
TOTAL AREA = 12.37 SQ MI

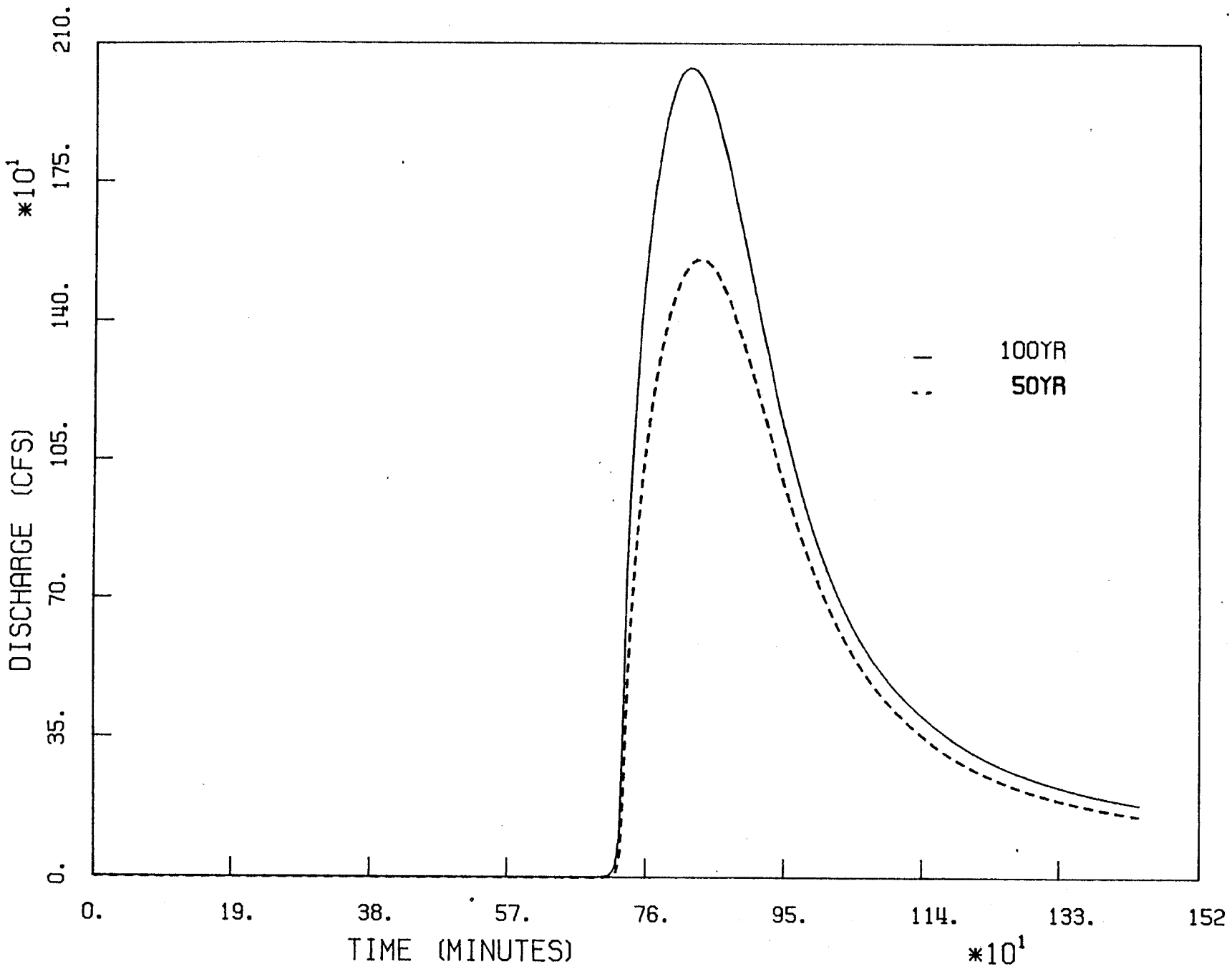
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

| OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR | 24-HOUR | 72-HOUR | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---|---|---|---|---|---|---|---|---|---|
| HYDROGRAPH AT + | SUB39 | 1946. | 12.50 | 381. | 108. | 108. | 1.94 | | |
| HYDROGRAPH AT + | SUB40 | 665. | 12.42 | 107. | 30. | 30. | .58 | | |
| 2 COMBINED AT | CP41 | 2593. | 12.42 | 488. | 138. | 138. | 2.53 | | |
| HYDROGRAPH AT | SUB41 | 2996. | 12.58 | 605. | 171. | 171. | 3.32 | | |
| HYDROGRAPH AT | SUB42 | 828. | 12.25 | 97. | 27. | 27. | .58 | | |
| HYDROGRAPH AT | SUB43 | 1626. | 12.33 | 218. | 62. | 62. | 1.07 | | |
| HYDROGRAPH AT | SUB44 | 1280. | 12.33 | 168. | 48. | 48. | .85 | | |
| 2 COMBINED AT | CP45 | 2906. | 12.33 | 387. | 109. | 109. | 1.92 | | |
| HYDROGRAPH AT | SUB45 | 3205. | 12.33 | 463. | 131. | 131. | 2.30 | | |
| HYDROGRAPH AT + | SUB46 | 1207. | 12.33 | 192. | 54. | 54. | .93 | | |
| 2 COMBINED AT + | CP47 | 4412. | 12.33 | 655. | 185. | 185. | 3.23 | | |
| HYDROGRAPH AT + | SUB47 | 4272. | 12.50 | 714. | 201. | 201. | 3.60 | | |
| 3 COMBINED AT | CP48 | 7711. | 12.50 | 1415. | 400. | 400. | 7.50 | | |
| HYDROGRAPH AT | SUB48 | 7511. | 12.50 | 1421. | 402. | 402. | 7.55 | | |
| HYDROGRAPH AT | SUB49 | 431. | 12.33 | 60. | 17. | 17. | .35 | | |
| HYDROGRAPH AT | SUB50 | 508. | 12.25 | 71. | 20. | 20. | .36 | | |
| 3 COMBINED AT | | | | | | | | | |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| + | CP51 | 8147. | 12.50 | 1551. | 439. | 439. | 8.26 |
| HYDROGRAPH AT | | | | | | | |
| + | SUB51 | 8300. | 12.67 | 1655. | 471. | 471. | 9.27 |
| HYDROGRAPH AT | | | | | | | |
| + | SUB52 | 7317. | 12.83 | 1765. | 511. | 511. | 10.62 |
| HYDROGRAPH AT | | | | | | | |
| + | SUB53 | 5347. | 13.42 | 1859. | 554. | 554. | 12.37 |

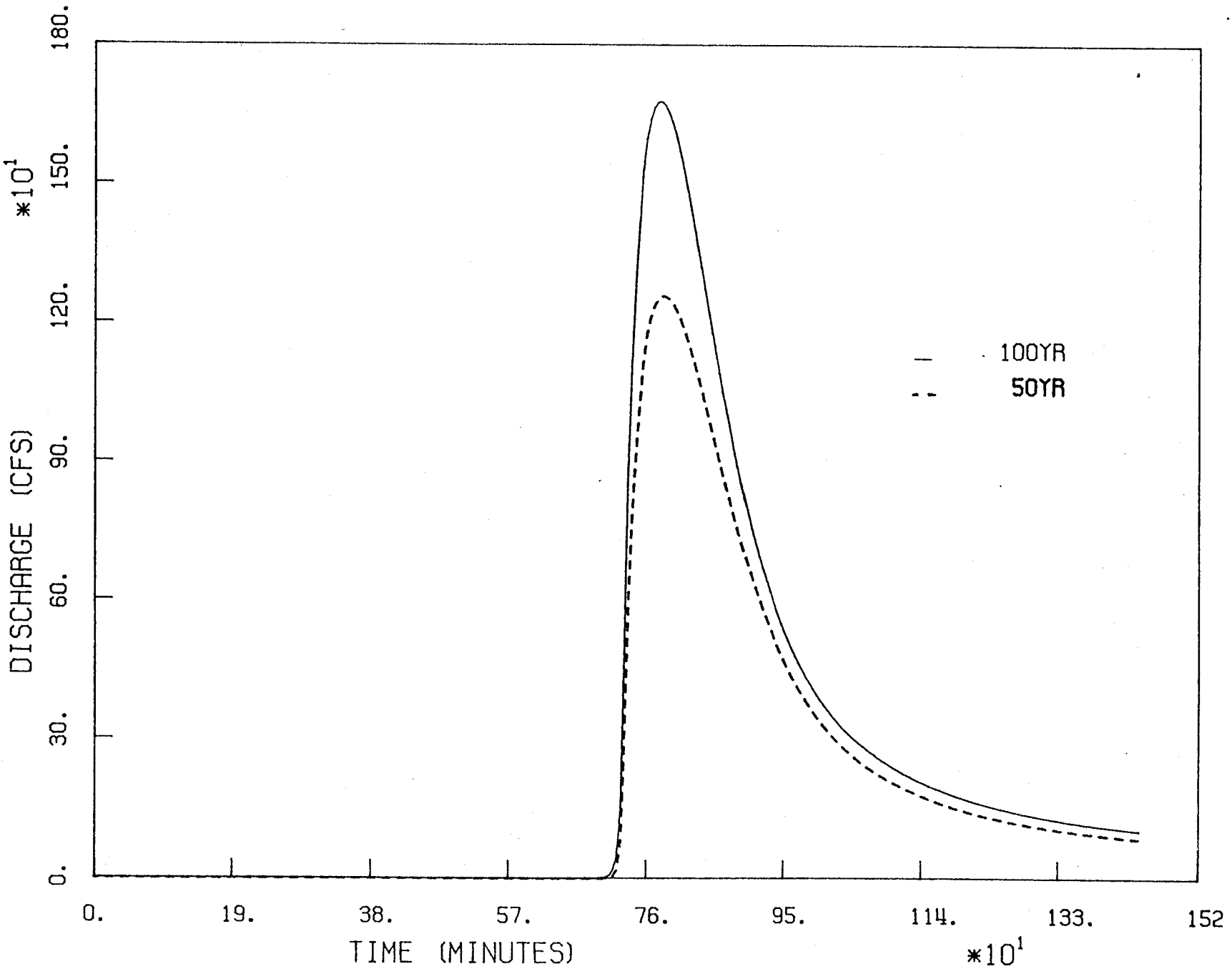
*** NORMAL END OF HEC-1 ***

# APPENDIX C

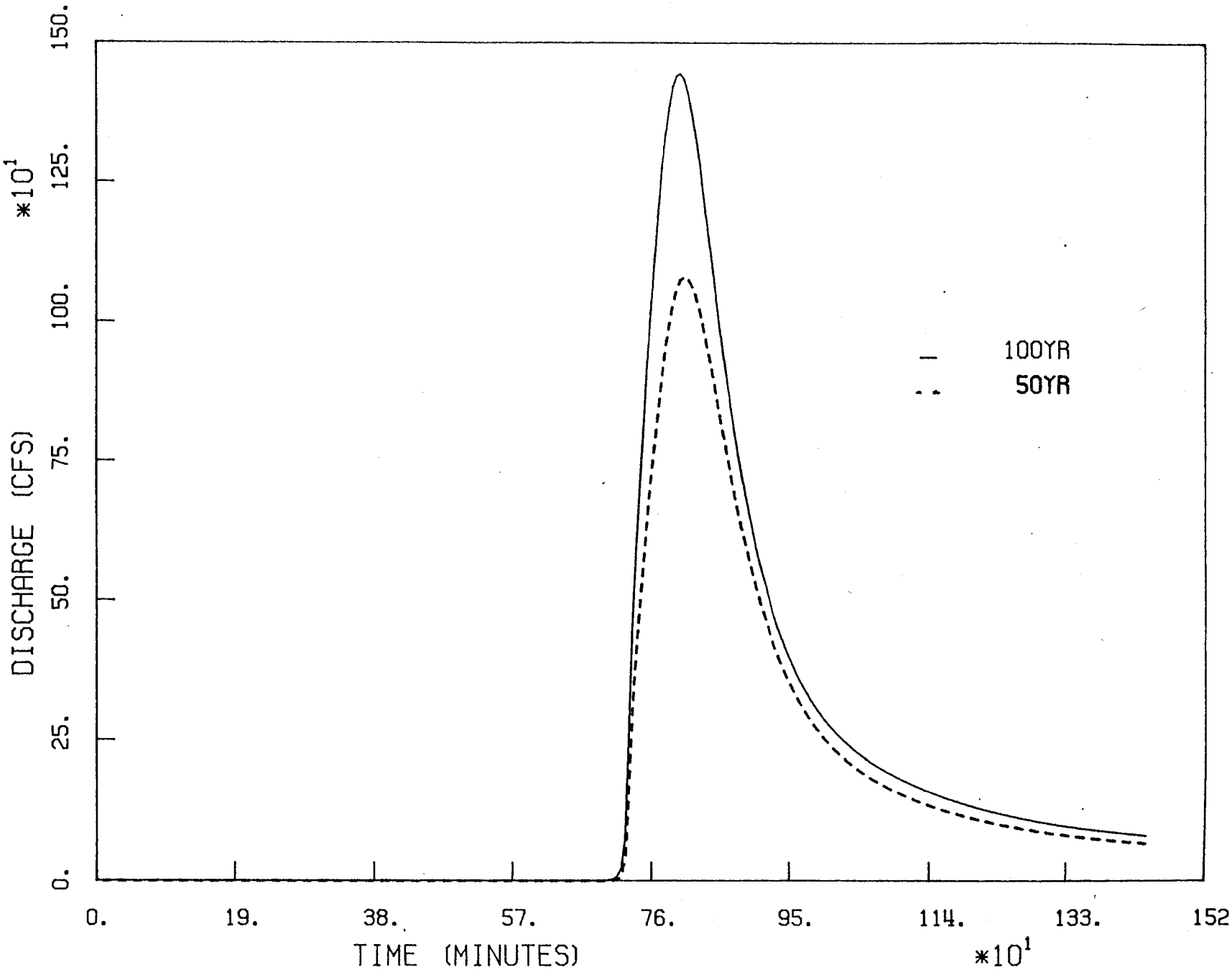



Runoff Hydrographs
Sub-Drainage Area "A"

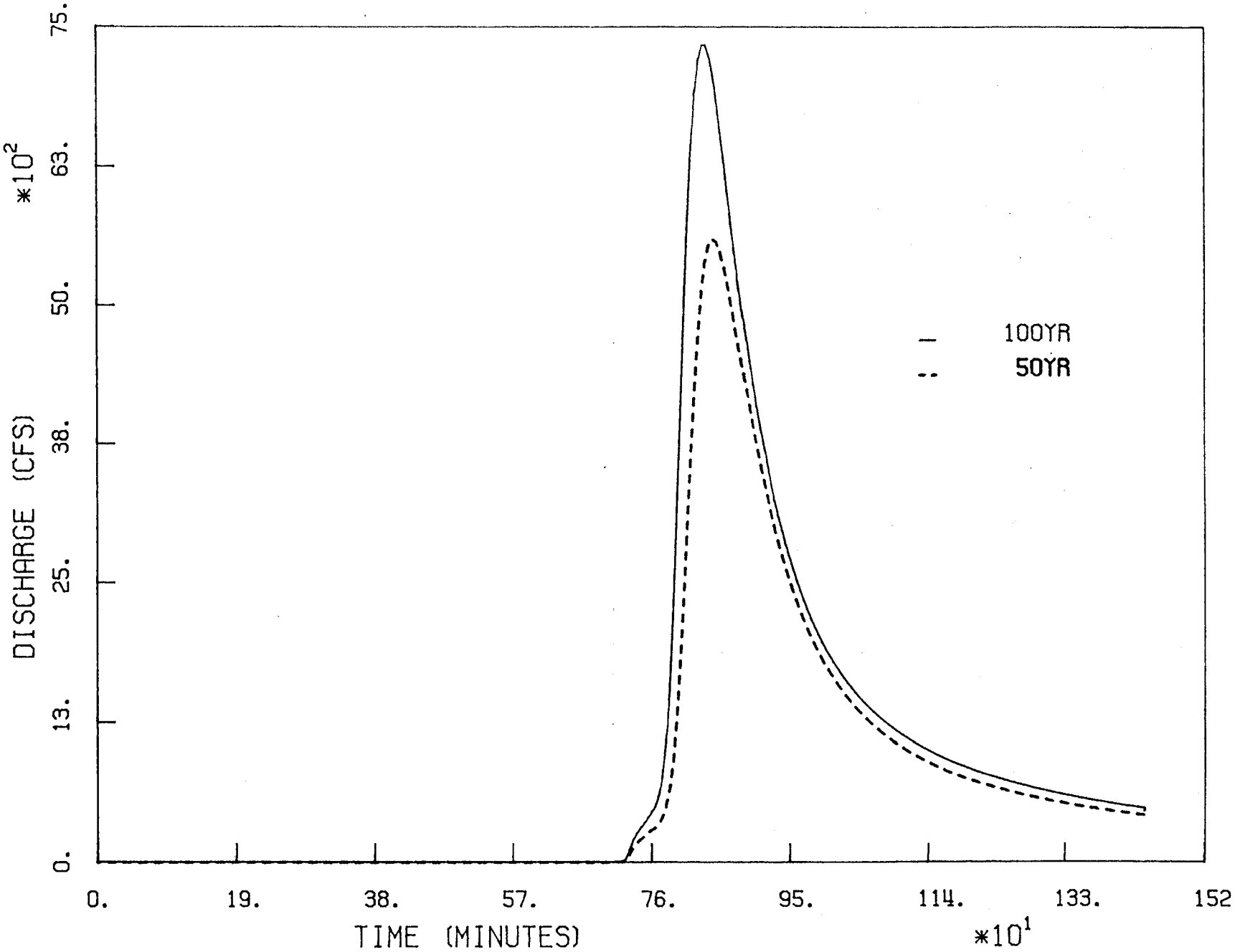
Runoff Hydrographs
Sub-Drainage Area "B"



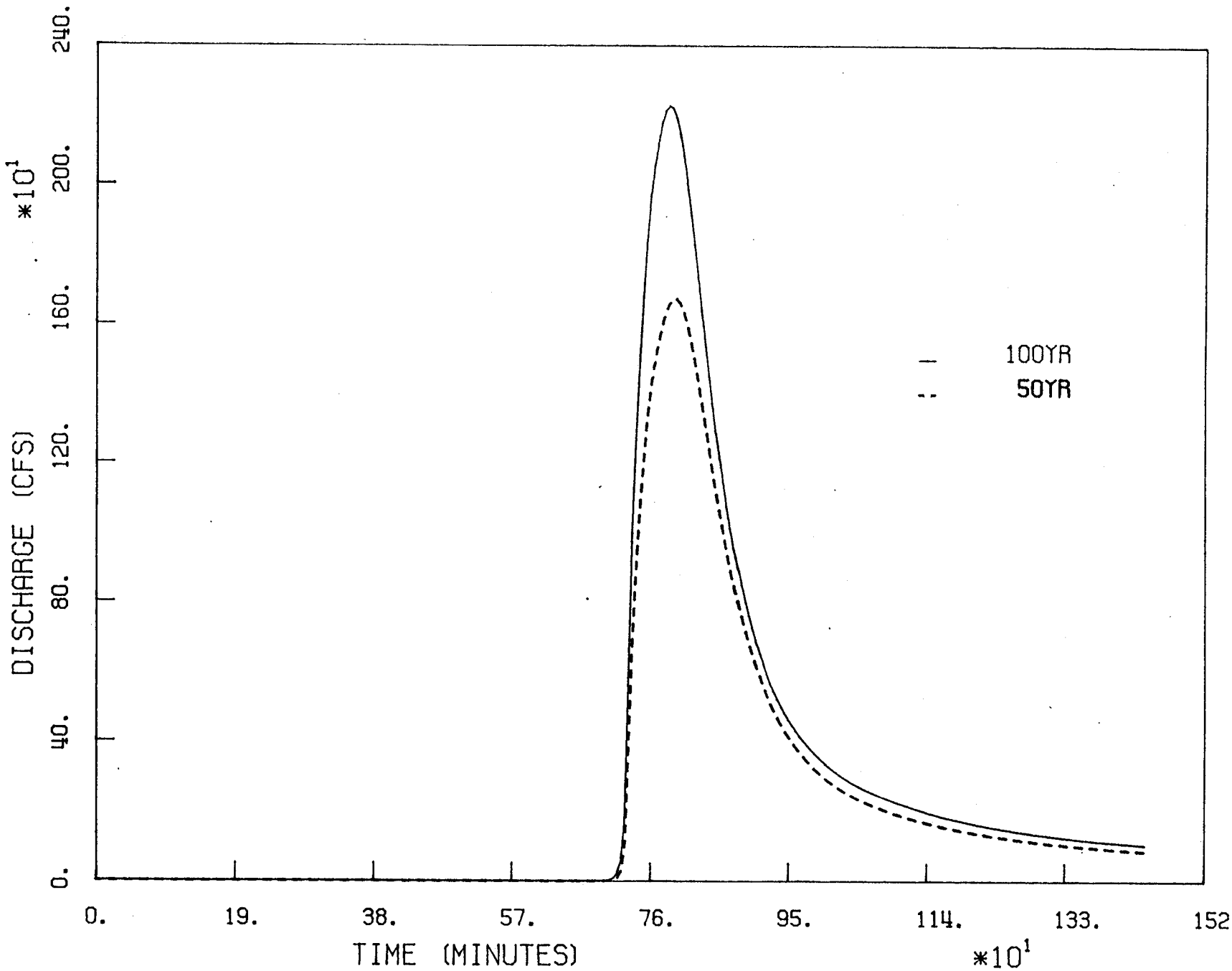
Runoff Hydrographs
Sub-Drainage Area "C"



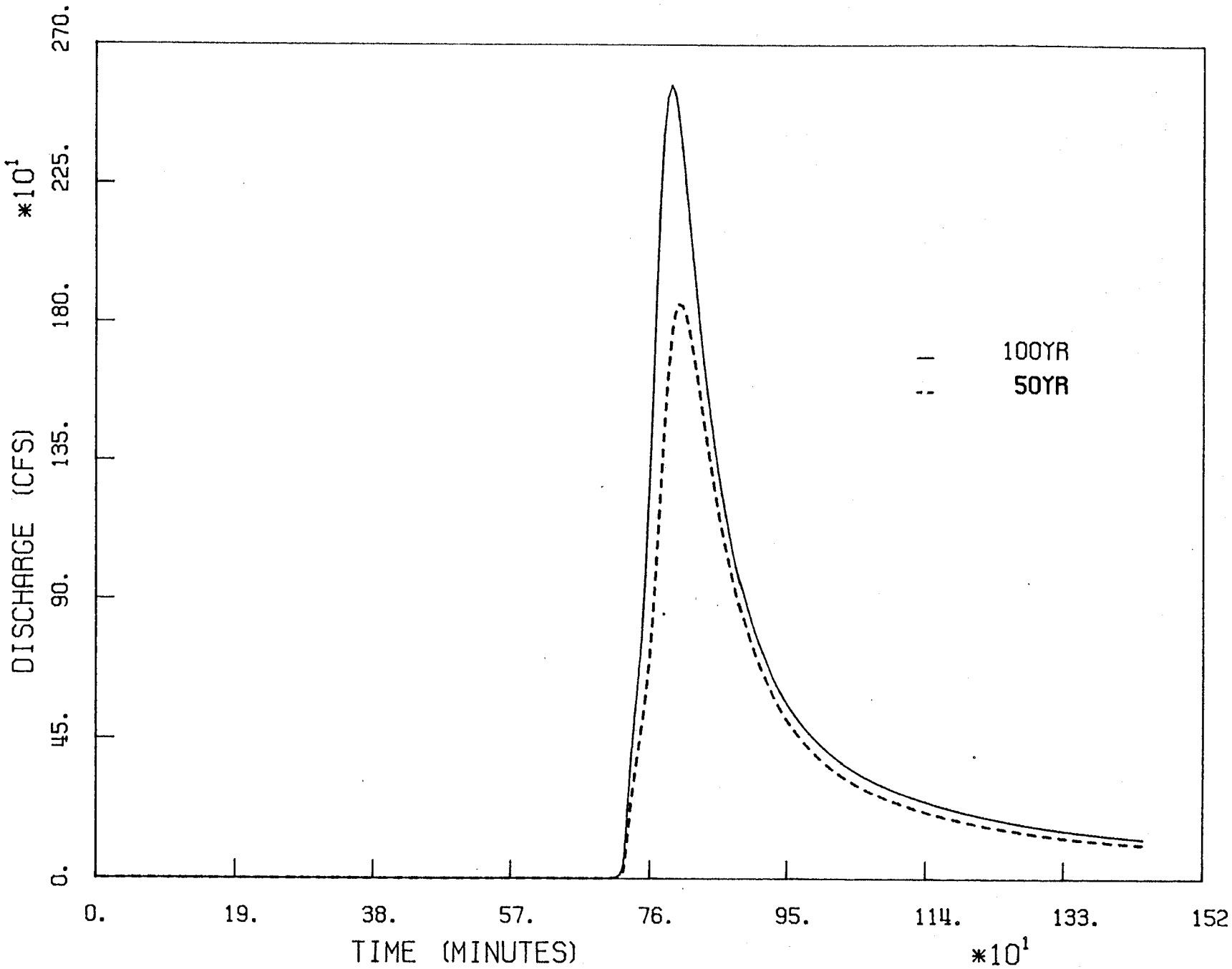
Runoff Hydrographs
Sub-Drainage Area "D"



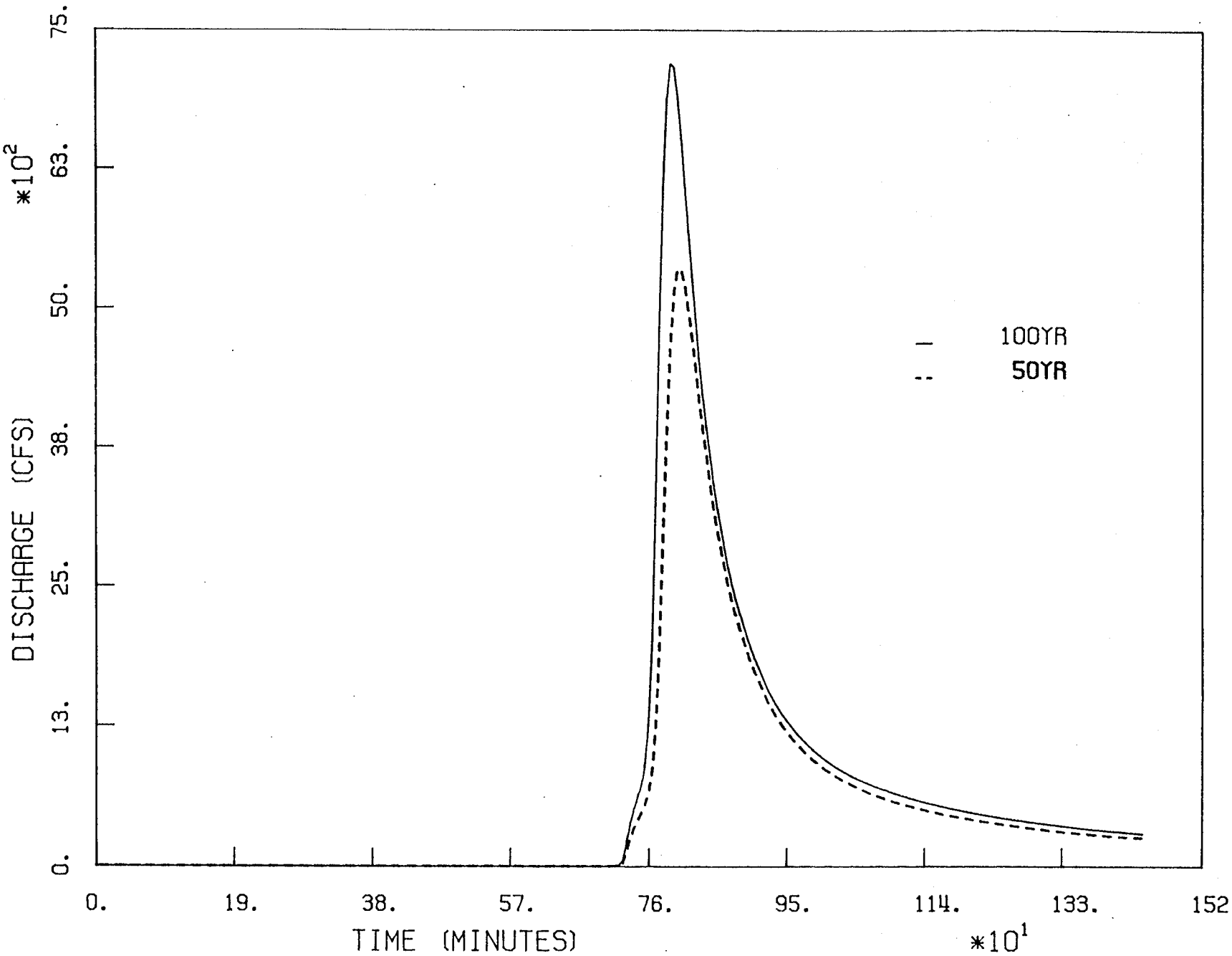
Runoff Hydrographs
Sub-Drainage Area "E"

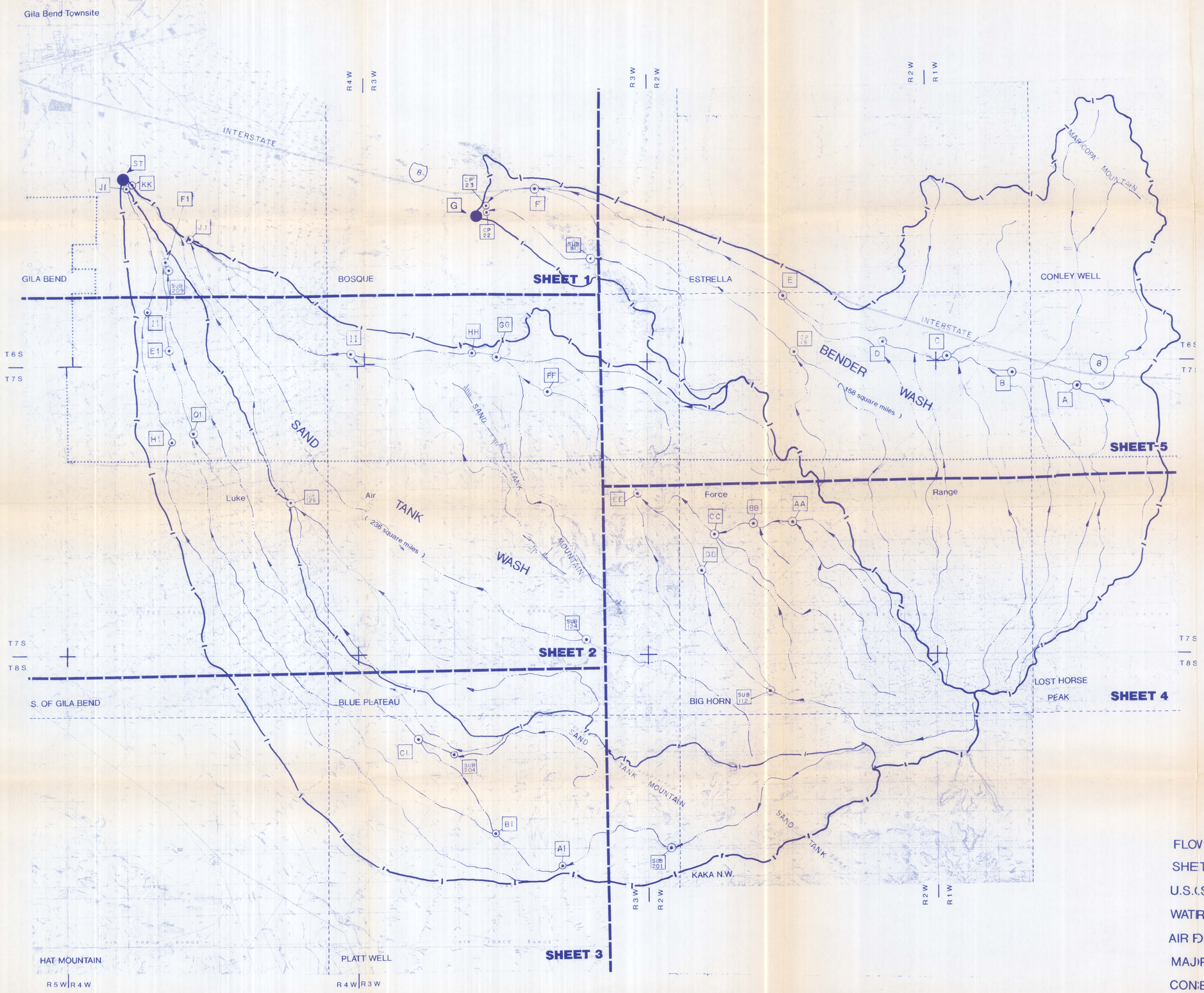


Runoff Hydrographs
Sub-Drainage Area "F"



Runoff Hydrographs
Sub-Drainage Area "G"






N

SCALE 1" = 5,000'

LEGEND

- FLOW PATHS
- SHEET MATCH LINES
- U.S.G.S BOUNDARIES
- WATERSHED BOUNDARIES
- AIR FORCE RANGE BNDY
- MAJOR CONCENTRATION PT.
- CONCENTRATION POINT
- CONC. PT. IDENTIFIER (AA)

| NO. | REVISIONS | DATE | BY | CHK. |
|---|---|---|---|---|
| | | | | |

# EXHIBIT "A"

# DRAINAGE WATERSHED MAP AND SHEET INDEX

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

SAND TANK & BENDER WASH
GILA BEND, AZ.

DESIGN
DESIGN CHK.
PLANS
PLANS CHK.
SUBMITTED BY

Recommended by
Approved by
Date
Date

SHEET
of

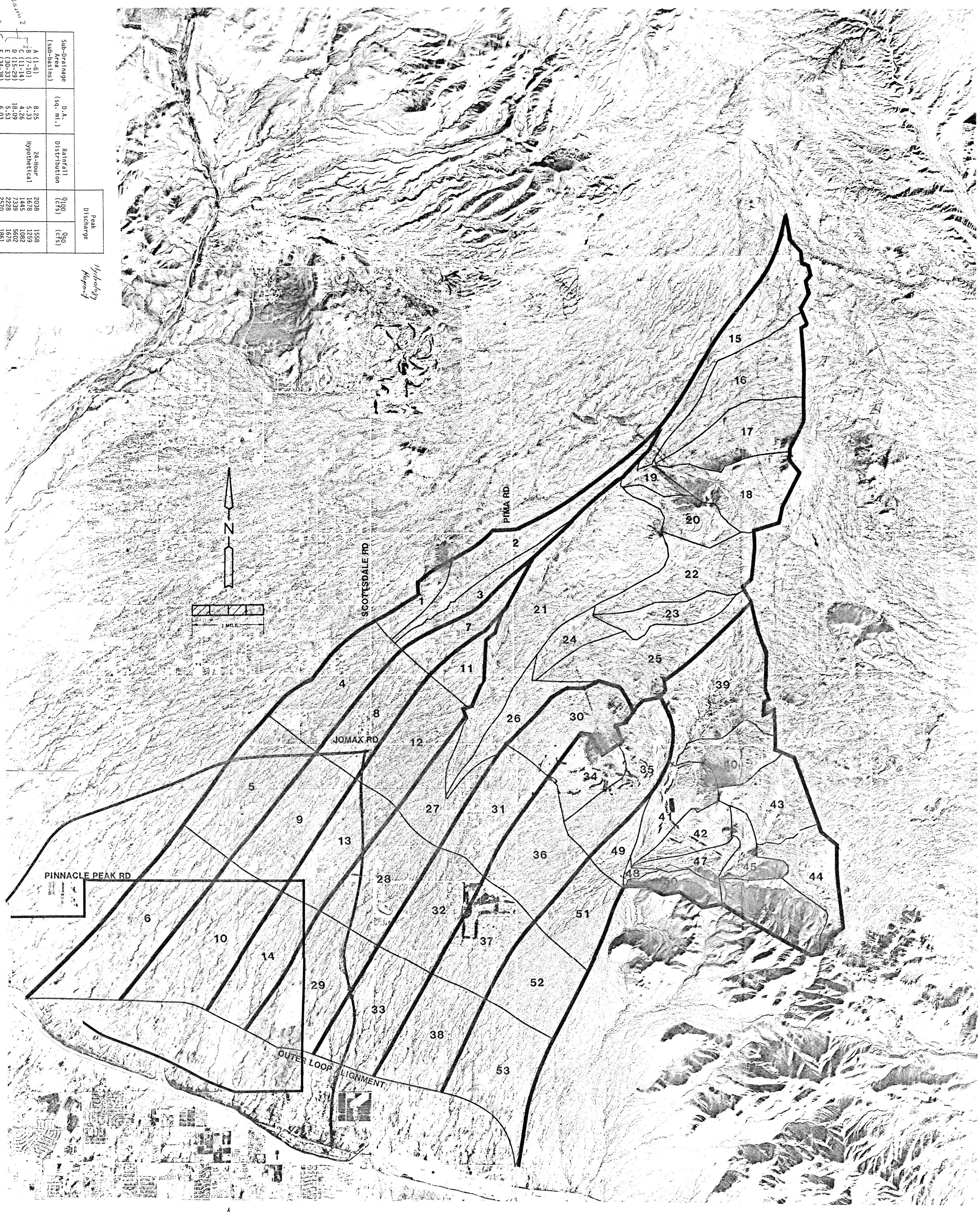
SHEET NO.
OF

Hydrology Report

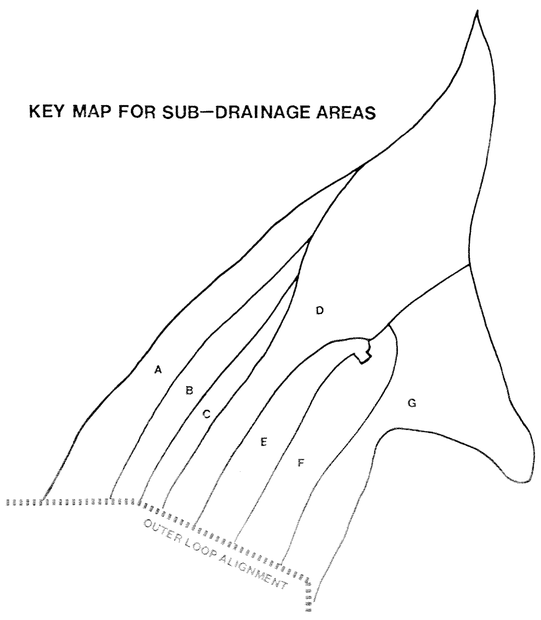
| Sub-Drainage Area (sub-basins) | D.A. (sq. mi.) | Rainfall Distribution | Peak Discharge $Q_{100}$ (cfs) | Peak Discharge $Q_{50}$ (cfs) |
|---|---|---|---|---|
| A (1-6) | 8.25 | 24-Hour Hypothetical | 2038 | 1558 |
| B (7-10) | 5.33 | | 1678 | 1259 |
| C (11-14) | 4.26 | | 1445 | 1082 |
| D (15-29) | 18.09 | | 7339 | 5602 |
| E (30-33) | 5.53 | | 2228 | 1675 |
| F (34-38) | 6.03 | | 2570 | 1861 |
| G (39-53) | 12.37 | | 7202 | 5347 |

Basin 2

Basin 3




KEY MAP FOR SUB–DRAINAGE AREAS




LEGEND

SUB–DRAINAGE BOUNDARY

SUB–BASIN BOUNDARY

36 SUB–BASIN NUMBER

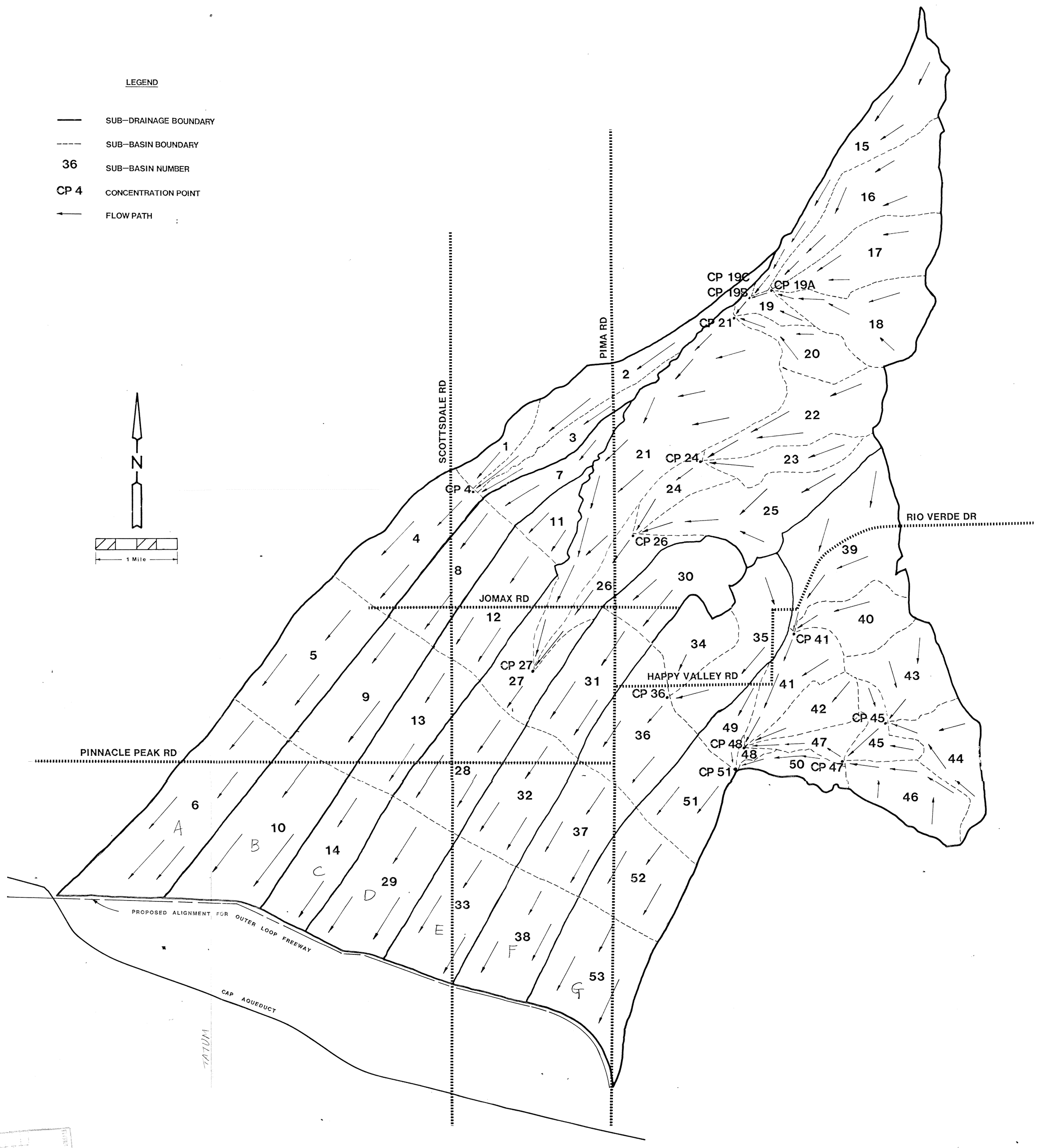
**sla** Simons, Li & Associates, Inc.

Cheyenne, WY. • Colorado Springs, CO. • Denver, CO. • Fort Collins, CO.
Newport Beach, CA. • Phoenix, AZ. • Tucson, AZ.

AERIAL PHOTOGRAPH OF WATERSHED BOUNDARIES

FLOOD CONTROL DISTRICT
RECEIVED
AUG 29 '89
PLATE 1

TELEDYNE POST N57474




AUG 26 '89

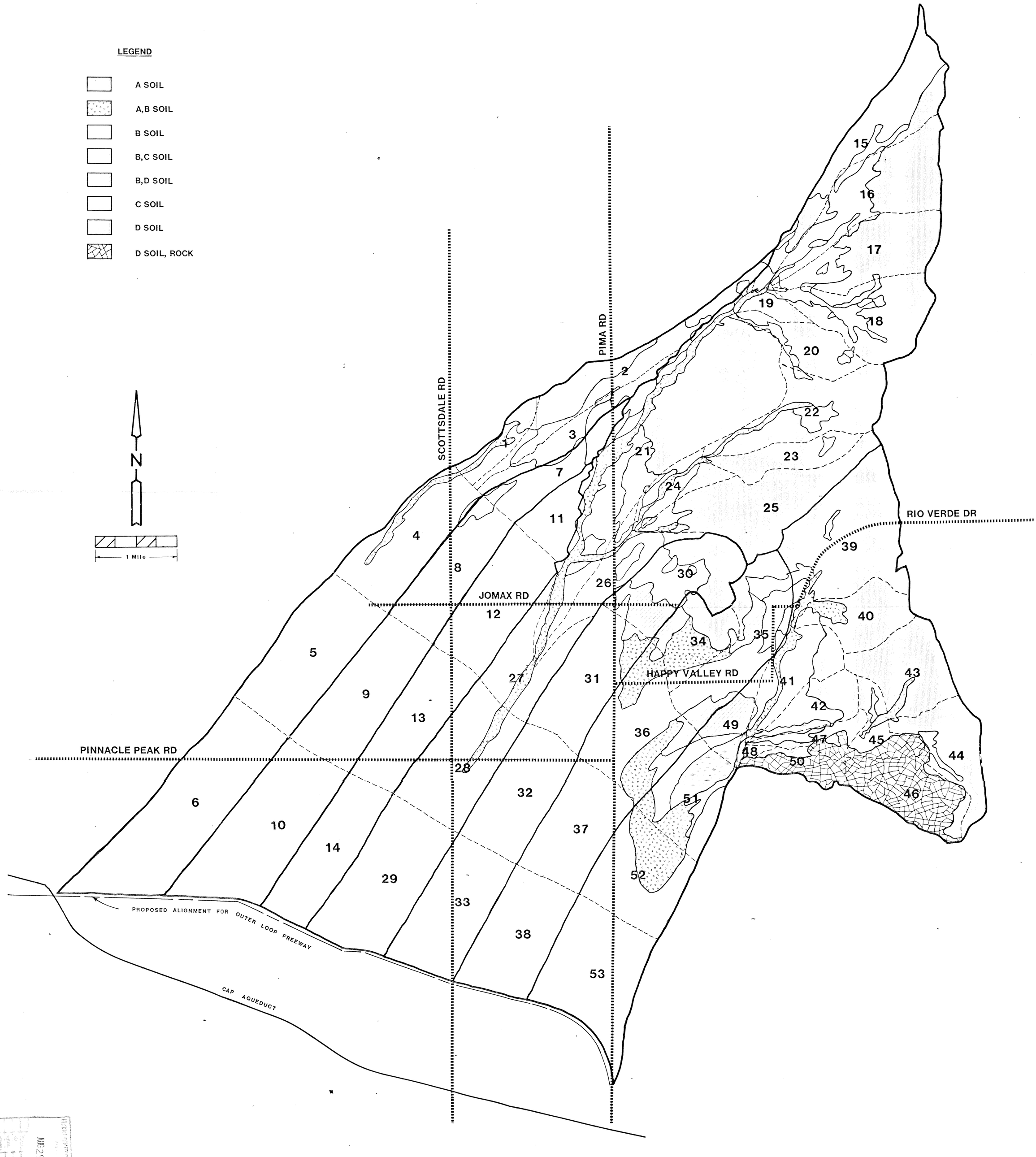
Project No.
Date:
Design:
Drawn:
Check:
Revisions:

PLATE 3

DRAINAGE PATTERNS, SUB–BASIN BOUNDARIES,
AND CONCENTRATION POINTS

Simons, Li & Associates, Inc.
Cheyenne, WY. • Colorado Springs, CO. • Denver, CO. • Fort Collins, CO.
Newport Beach, CA. • Phoenix, AZ. • Tucson, AZ.

TELEDYNE POST N57474




AUG 29 '89

Project No.
Date:
Design:
Drawn:
Check:
Revisions:

PLATE 2

HYDROLOGIC SOIL GROUP MAP

Simons, Li & Associates, Inc.
Cheyenne, WY. • Colorado Springs, CO. • Denver, CO. • Fort Collins, CO.
Newport Beach, CA. • Phoenix, AZ. • Tucson, AZ.