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SENSITIVITY ANALYSIS OF REATA PASS HYDROLOGY SCOTTSDALE, ARIZONA

Prepared for:

CITY OF SCOTTSDALE
3939 Civic Center Plaza
Scottsdale, AZ 85251

Prepared by:

WATER RESOURCES ASSOCIATES, INC.
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Registered Professional Engineer (Civil)
CERTIFICATE NO. 13505
ROBERT L. WARD
Date Signed 11/8/91
ARIZONA, U.S.A.

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1.0 PURPOSE OF STUDY

Since 1988, several master drainage studies have been completed for areas of both Phoenix and Scottsdale that lie north of the Central Arizona Project (CAP) aqueduct and east of Cave Creek. These studies have produced very large rainfall/runoff models (HEC-1) that have been used to identify existing flooding potential and to develop an integrated, regional system of flood control channels and detention basins.

The HEC-1 models were also used by Cellar Barr Associates (1988-1989) for the initial delineation of Federal Emergency Management Agency (FEMA) floodplain zones for this watershed.

Due to the rapid urbanization of this area, there has been a continual debate over the accuracy and the methodology used to delineate the floodplain zones, and to some extent, over the hydrology that was applied to the methodology. Although an extensive verification analysis has previously been performed to validate the results of the HEC-1 model (Water Resources Associates, Inc., 1988, 1990, 1990), continued debate over the watershed hydrology has prompted the City of Scottsdale, City of Phoenix, and the Flood Control District of Maricopa County to re-examine the HEC-1 models to determine whether modeling refinements could cause any significant changes in the peak 100-year discharges produced by the models.

The large size of the watershed (approximately 150 square miles) precludes a re-examination of all the HEC-1 models that have been developed for this area. Due to the controversy that is associated with the Reata Pass watershed (FEMA Fan 2), Reata Pass was selected as the focal point of this study. Plate 1 illustrates the watershed boundaries and HEC-1 routing schematic used for this analysis.

The contents of this report present an extensive analysis and comparison of modeling refinements for the Reata Pass watershed. Each modeling revision is compared to the original baseline hydrology and to several independent peak discharge methodologies. Such a comparison identifies the sensitivity of each modeling revision and provides insight into the realism of the model results when compared with peak discharge calculations from independent methods (i.e., methods other than HEC-1).

The results of this study will form the basis for possible modeling revisions that will be recommended to FEMA for use in defining flood frequency series at each alluvial fan apex in the study area.

The following sections of this report present a detailed discussion of the modeling revisions, model verification analysis, and comparison of model results.

2.0 DISCUSSION OF MODELING REVISIONS

The original HEC-1 models that were created for the North Scottsdale and Phoenix Master Drainage Studies employed the kinematic wave methodology for the development of sub-basin runoff hydrographs and channel routing operations. All models used a 24-hour, Soil Conservation Service (SCS) Type IIA rainfall distribution and the SCS curve number methodology for simulating rainfall losses.

It is known that the kinematic wave channel routing methodology is sensitive to the channel geometry that is used in the procedure. The limitation of a four-point trapezoidal cross-section creates a significant obstacle to an accurate computer simulation of a natural desert wash that exhibits both main channel and overbank flow during large events, such as a 100-year flood.

Although the channel geometry in the original HEC-1 model was based on measured cross-sections at numerous locations throughout the watershed, the computer input limitation of a four-point section often required the application of engineering judgment to select a cross-section that would be hydraulically suitable for a wide range of flows. This limitation invites an obvious opportunity for additional analysis through the application of alternate channel routing methodologies available in HEC-1.

In addition to revisions in channel routing procedures, the sensitivity of the rainfall/runoff model was also examined as a function of: 1) decreasing the size of sub-basin boundaries; 2) changing the rainfall loss routine from curve numbers to the Green-Ampt infiltration algorithm; 3) revising the Manning's "n" value used in the channel routing operations; and 4) applying different rainfall distributions and storm durations to the models. These categories of input revisions resulted in 15 different scenarios of modeling assumptions for the Reata Pass watershed. These scenarios are defined as follows.

Scenario 1 - This is the original HEC-1 model that was developed for this watershed as part of the General Drainage Plan for North Scottsdale, Arizona, Water Resources Associates, Inc., 1988-1989. This model represents existing land-use conditions that were in-place as depicted on a 1987 aerial photograph of the watershed. This scenario is based on HEC-1 input file SC40.24I that was taken

from the 1988-1989 General Plan study (Volume I) for North Scottsdale. As used in this current study, input file SC40.24I was modified on March 12, 1990 to relocate Sub-basin 49 downstream of CP51.

Scenario 1 is used to establish a base-line condition for comparison with all subsequent modeling revisions.

Scenario 2 - This is the same as Scenario 1 except that all singular channel routing operations have been changed from kinematic wave to normal depth (with eight-point cross-section), modified Puls routing. The number of routing steps used for each operation (HEC-1 variable NSTPS) is based on the measured channel reach length, an average flow velocity of 6 fps, and a time interval of five minutes.

This change was only made to pure channel routing operations between sub-basins, i.e., the collector and main channel routing elements within each sub-basin (UK/RK/RK records) were left as kinematic wave.

Scenario 3 - This option explores the sensitivity of dividing the watershed into smaller sub-basins than those used in the original model. Smaller sub-basins will promote an increase in the homogeneity of the input parameters used to describe the physical characteristics of each sub-basin. All channel routing operations were left as kinematic wave in this scenario.

Scenario 4 - This model replaces all singular channel routing operations in Scenario 3 with normal depth, modified Puls routing. All internal sub-basin channel routing operations (UK/RK/RK) combinations) remain as kinematic wave. The number of routing steps used for each operation (HEC-1 variable NSTPS) is based on the measured channel reach length, an average flow velocity of 6 fps, and a time interval of five minutes.

This modification results in a total length of 38,950 feet of modified Puls routing compared to 23,900 feet of modified Puls routing in the original sub-basin boundary model evaluated under Scenario 2.

Scenario 5 - This revision replaces all SCS curve numbers in the Scenario 4 model with the Green-Ampt infiltration routine. Accordingly, this scenario reflects the enhanced sub-basin delineations,

the conversion of pure kinematic wave channel routing operations to modified Puls, and the simulation of rainfall losses by the Green-Ampt procedure.

Scenario 6 - This option evaluates the sensitivity of using the 12-hour, HEC-1 hypothetical rainfall distribution in place of the 24-hour, SCS Type IIA distribution. This new distribution was applied to the modeling parameters in Scenario 4, which consist of the enhanced sub-basin delineations, SCS curve numbers, and modified Puls routing for pure channel routing operations between sub-basins.

Scenario 7 - This model is the same as Scenario 6, except the 12-hour hypothetical distribution is replaced by the six-hour hypothetical distribution.

Scenario 8 - This scenario examines the new Muskingum-Cunge channel routing methodology included in the September 1990 HEC-1 program (Version 4.0). Scenario 8 changes all kinematic wave and modified Puls channel routing operations, in Scenario 4, to Muskingum-Cunge routing. This was done by simply replacing all RK and RS records by RD records. The RD records retained the identical channel routing data that was used with the RK records for kinematic wave operations and with the RC/RX/RY records used for modified Puls operations. This scenario retains the 24-hour, SCS Type IIA rainfall distribution and the SCS curve number methodology.

Scenario 9 - Based on the September 1990 HEC-1 User's Manual, Muskingum-Cunge channel routing (with an eight-point cross-section) can be combined with kinematic wave runoff procedures (UK record) to receive and route kinematic wave runoff within a sub-basin. The eight-point cross-section geometry is restricted to only the main channel and cannot be used with collector channels.

Scenario 9 attempted to modify Scenario 8 by replacing all four-point trapezoidal, main channel routing elements (within reach sub-basin) with an eight-point cross-section. Unfortunately, the HEC-1 program returned a fatal error message when the eight-point cross-section was combined with the Muskingum-Cunge routing methodology for a main channel element that was not precended by a collector channel. Inquiries will be made to the Corps of Engineers to determine the cause of this problem. Until this apparent program error is corrected, the results of Scenario 9 will be unavailable.

Scenario 10 - This scenario examines the sensitivity of the number of routing steps (HEC-1 variable NSTPS) in the modified Puls routing procedure on peak discharge. All previous scenarios used

NSTPS values that were a function of channel length, average flow velocity, and time interval. Discussions with representatives of the Hydrologic Engineering Center at Davis, California resulted in a recommendation to set the routing step at one for all modified Puls operations. Accordingly Scenario 10 resets variable NSTPS to one for all modified Puls routing operations used in Scenario 4.

Scenario 11 - This scenario was created to evaluate the effects of changing the channel "n" value that was used in the kinematic wave channel routing operations in the base-line model discussed under Scenario 1. The "n" values were changed from 0.045 to 0.055 on all RK records.

Scenario 12 - In a similar fashion as discussed for Scenario 10, Scenario 12 resets the modified Puls routing step variable (NSTPS) to one for all modified Puls routing operations in Scenario 2. Scenario 12 differs from Scenario 10 only in the fact that Scenario 12 uses the original sub-basin boundaries, while Scenario 10 uses the enhanced sub-basin boundaries installed as part of Scenario 4.

Scenario 13 - This option simply applies a six-hour hypothetical rainfall distribution to Scenario 12. The original sub-basin boundaries remain intact and the modified Puls routing step is set to one for all normal depth channel routing operations.

Scenario 14 - This scenario was created by applying a six-hour hypothetical rainfall distribution to Scenario 2. Accordingly, Scenario 14 is based on the original sub-basin delineations with singular channel routing operations converted to eight-point, modified Puls. The routing step variable (NSTPS) is set as a function of channel length, velocity, and time interval.

Scenario 15 - This option replaces the 24-hour Type IIA storm distribution in Scenario 10 with a six-hour hypothetical distribution. This scenario uses the revised sub-basin delineations with singular channel routing operations converted to eight-point, modified Puls. The routing step variable (NSTPS) is set at one.

For convenient reference, Table 2.1 presents a matrix which illustrates the combinations of modeling parameters used for each of the 15 scenarios.

It should be noted that work on the General Drainage Plan for North Scottsdale began in late 1987 and early 1988. During this time frame, the 1985 version of HEC-1 was the only available program code. Accordingly, the General Plan hydrology models were based on the 1985 HEC-1 program.

As the study limits expanded during the following years, a decision was made to stay with the 1985 version in order to maintain consistency in the model logic and results. It was well-known that the 1988 version of HEC-1 had changed the kinematic wave routing algorithms, and that such changes were creating significant differences in the model output when compared to the 1985 model results.

A 1990 version of HEC-1 is now available which has corrected some kinematic wave routing problems that were found to exist in the 1988 version. In order to identify the sensitivity of these changes to the Reata Pass HEC-1 model, the 1985, 1988, and 1990 versions of HEC-1 were applied to most of the 15 scenarios discussed in this report. Table 2.2 summarizes the modeling results at CP 51 (watershed outlet) for the 15 scenarios and different program versions of HEC-1.

The following subsections present more detailed discussions of the modeling revisions.

2.1 SUB-BASIN DELINEATIONS

The watershed boundaries for Reata Pass were taken from Plate 2 to the 1988-1989 General Drainage Plan for North Scottsdale, Arizona (WRA). However, the runoff from Sub-basin 49 was excluded from the total drainage area contributing to the Reata Pass apex at CP 51.

For the enhanced sub-basin delineation effort, United States Geological Survey (USGS) 7.5-minute quadrangle maps were used to sub-divide the basins in the Reata Pass watershed into smaller, more homogeneous areas. In order to retain continuity with the original, total sub-basin drainage areas, the total area of the sub-divided basins was compared to the original total area of the parent basin and any differences were prorated among all the newly subdivided basins.

For example, the original Sub-basin 40 was subdivided into Sub-basins 40A, 40B, and 40C. The combined planimetered area of these three basins was 0.5968 square miles, compared to 0.5844 square miles for the original parent basin. Accordingly, the 0.0124 square mile difference between

these two values was distributed (prorated by area) among the three sub-basins so that the total area of the three basins matched that of the original parent sub-basin (SUB 40).

All new sub-basins use the same numeric designation as their original parent sub-basins. However, an alpha character was attached to each number to distinguish between new sub-basins, i.e., 39A, 39B, 39C, etc.

The original parent sub-basin boundaries are identified in Figure 1.1 by a wider black line.

2.2 OVERLAND FLOW PARAMETERS

All kinematic wave overland flow lengths in the sub-divided basins were maintained at the same values used in the original parent sub-basins. However, some adjustments were made to overland flow slopes and overland flow roughness to reflect the increased homogeneity of the smaller sub-basins.

Slope adjustments were based on measurements taken from the USGS quadrangle maps. Adjustments to roughness values were based on the application of engineering judgement and the physical location of each sub-basin.

2.3 CHANNEL ROUTING OPERATIONS

When converting from four-point trapezoidal, kinematic wave channel geometries to eight-point modified Puls channel geometries, the four-point section bottom width was maintained as the bottom width of the main channel in the eight-point section. These bottom widths ranged from 20 to 75 feet.

The overbank geometries selected for the eight-point sections were a matter of engineering judgement, since no overbank areas are included in the kinematic wave channel geometries. Typically the overbank lengths were set from 100 to 140 feet on each side of the main channel. The overbanks were set with a cross-slope of 5H:1V to 7H:1V towards the main channel.

The depths of the main channels were set at a constant value of three feet for all eight-point routing operations. Main channel side-slopes were set at 2H:IV.

The lengths and slopes of the eight-point channel routing operations were set to match the length and slopes for the same routing paths used in the original base-line model. This was achieved by a cross-reference to the channel lengths and slopes on the original RK cards used for the base-line model.

2.4 CHANNEL ROUGHNESS

The original kinematic wave models used a Manning's "n" value of 0.045 for all routing operations. When converting to the eight-point cross-sections, the channel "n" value was set at 0.040, while each overbank "n" value was set at 0.055.

For Scenario 11, the kinematic wave "n" values were increased from 0.045 to 0.055.

2.5 RAINFALL LOSSES

Except for Scenario 5, all HEC-1 models used in this study employed the SCS curve number methodology to simulate rainfall losses due to interception, ponding, and infiltration. Scenario 5 used the Green-Ampt procedure.

The original sub-basin curve numbers were developed under the assumption of a "desert brush" vegetation community with a 15 percent cover density. Area-weighted curve numbers were developed to reflect the different percentages of hydrologic soil groups (HSG) that were found to comprise each sub-basin. HSG information was taken from the Soil Survey of Aguila-Carefree area, Parts of Maricopa and Pinal Counties, Arizona, U.S. Department of Agriculture, SCS, April 1986.

In order to provide some acknowledgement of channel transmission losses, the selected curve numbers were two to three points lower than those published for the same vegetation cover and densities in TR-55 (SCS, 1986). A detailed discussion of the original curve number calculations is presented in the Upper Indian Bend Wash Regional Drainage and Flood Control Plan, Preliminary Draft, September 5, 1990, Water Resources Associates, Inc.

For the purpose of this current study, new area-weighted curve numbers were computed for each of the new sub-basins that were used as the basis for Scenarios 3 through 10. These new curve number calculations used the same base soils map as was used for the original calculations. Table 2.3 summarizes the soils information that was used to compute weighted curve numbers for each of the new sub-basins.

Due to the relatively sparse amount of development in the Reata Pass watershed, no attempt was made to recompute the percents of impervious cover for each of the re-delineated, smaller sub-basins. Instead, each new sub-basin was assigned the same percent of impervious cover as used for the parent sub-basin. This approach maintained continuity in the amount of impervious cover used for both the original sub-basin delineations and the new sub-basin delineation.

Area-weighted Green-Ampt parameters were also computed for the new sub-basins for use in Scenario 5. Table 2.4 summarizes the Green-Ampt parameters as a function of the SCS soil mapping units that comprise each sub-basin.

The Green-Ampt parameters for each SCS soil unit were taken from the Aguila-Carefree Loss Rate Parameters table published by the Flood Control District of Maricopa County. Initial rainfall losses (HEC-1 variable IA) varied from 0.15 to 0.25 inches for sub-basins in the Reata Pass watershed.

2.6 RAINFALL DISTRIBUTION AND DURATION

The original HEC-1 models developed for the Upper Indian Bend Wash Regional Drainage and Flood Control Plan utilized a 24-hour, SCS Type IIA rainfall distribution. Because the regional drainage plan includes interconnected channels and detention basins, a rainfall distribution was sought to provide the large runoff volume desired for major detention basin design, as well as the high intensity rainfall needed to maximize peak discharge for the channel design. The 24-hour, Type IIA distribution satisfies both of these requirements.

For comparison purposes, the Type IIA distribution will usually provide a slightly higher peak discharge than the Type II distribution, and a slightly lower discharge than the HEC-1 hypothetical distribution. As stated previously, all scenarios used the 24-hour, Type IIA distribution except Scenarios 6, 7, and 13 which utilized the 12-hour and six-hour hypothetical distributions, respectively.

An areal reduction factor of 0.985 was applied to the 24-hour point precipitation value used for the Type IIA distribution. Both the 12-hour and six-hour hypothetical distributions used the TP-40 areal reduction algorithms in HEC-1 to convert point rainfall values to areally reduced values. A ten square mile area was used as the basis for the areal reduction in all models.

As a matter of technical interest, it should be noted that no increases in SCS curve numbers were made in transitioning from the 24-hour storm to the 12- and six-hour storms.

3.0 MODEL VERIFICATION

The only true test of a computer model's accuracy is by calibration to a recorded storm event. Unfortunately, there is no recorded rainfall/runoff data available for the Reata Pass watershed. As an alternative to such data, several independent peak discharge procedures have been employed to identify a probable envelope within which a reasonable 100-year discharge would lie.

This same approach was previously employed by WRA for the Phoenix, Scottsdale, and Upper Indian Bend Wash drainage studies that have been published since 1988. Some additional methods are included in this current study to provide an even broader picture of the probable hydrologic response of the Reata Pass watershed.

These procedures are listed as follows:

1. Peak discharge regression equations presented in Floodplain Delineation Criteria and Procedures, Report No. 4, Arizona Water Commission, (AWC) October 1973.

2. Peak discharge regression equations presented in Methods for Estimating the Magnitude and Frequency of Floods in Arizona, USGS Report: ADOT-RS-15 121, R.H. Roeske, September, 1978.

3. Peak discharge regression equations presented in Estimation of Magnitude and Frequency of Floods in Pima County, Arizona with Comparisons of Alternate Methods, USGS Water Resources Investigations Report 84-4142, Table 1, J.H. Eychaner, August 1984.

4. A Magnitude-Frequency-Area Relation for Floods in Arizona: A Study to Advance the Methodology of Assessing the Vulnerability of Bridges to Floods, Malvick, A.J., Laursen, E.M., Duffy, D. (University of Arizona), 1980.

5. Peak discharge methodology presented in Hydrology Manual for Engineering Design and Floodplain Management within Pima County, Arizona, Pima County Department of Transportation and Flood Control District, September 1979.

6. Graphical peak discharge method presented in Chapter 4 of Technical Release 55, Urban Hydrology for Small Watersheds, SCS, June 1986.

7. Graphical plots of drainage area versus unit discharge (cfs/sq. mi.) developed from log-Pearson III analyses of stream gage data in Maricopa County, Arizona.

8. Regression equations presented in Flood Frequency Estimates in Southeastern Arizona, ASCE Journal of Irrigation and Drainage Engineering, Boughton, Renard, and Stone, November 1987.

9. Peak discharge envelope curve developed by Creager, Justin, and Hinds, Engineering for Dams, Volume 1, Chapter 5, page 99, 1945).

10. Peak discharge envelope curve developed by Costa and presented in Hydraulics and Basin Morphology of the Largest Flash Floods in the Conterminous United States, Journal of Hydrology, Volume 93, page 313-338, 1987.

11. Peak discharge envelope curve developed by the Corps of Engineers and presented in the Feasibility Study for the Clark County Regional Flood Control District, Clark County, Nevada, 1988.

The following paragraphs provide a brief discussion of each of these procedures. It should be noted that some of these procedures predict the magnitude of the 100-year event, while others are envelope curves of either 100-year events or of all recorded events for certain areas.

Method 1 employs regional regression equations for eight different geographical regions in Arizona. Accordingly, these equations should reflect differences in topography and rainfall characteristics that might exist among these different regions. These equations were developed from a modification of the regional method described in USGS Water Supply Paper 1683. The 100-year peak discharge equation for the Reata Pass area is:

$$Q_{100} = 2{,}900A^{0.52} \tag{3.1}$$

where: Q_{100} = 100-year peak discharge (cfs)

A = drainage area (square miles)

Method 2 utilizes regional regression equations (developed from stream gage data) for five different regions of Arizona. Although the Reata Pass watershed is physically located in Region 3 (Central Mountain Area), its physical characteristics are probably more typical of Region 2 (Southwest Desert Area). Accordingly, the regression equations for both regions will be used for Reata Pass. The peak discharge regression equation for Region 2 is:

$$Q_{100} = 1{,}100\ A^{0.499} \tag{3.2}$$

where: Q_{100} = 100-year peak discharge (cfs)

A = drainage area (square miles)

The regression equation for Region 3 is:

$$Q_{100} = .553\ A^{0.610}\ E^{-1.30}\ p^{0.915} \tag{3.3}$$

where: Q_{100} = 100-year peak discharge (cfs)

A = drainage area (square miles)

E = mean basin elevation (thousands of feet)

p = mean annual precipitation (inches)

One standard error of estimate for Equation 3.2 is 83 percent, while one standard error for Equation 3.3 is 66 percent. One standard error indicates that approximately 68 percent of the actual values (sample size) used in the regression are within the listed percentage of the value predicted by the regression equation. Accordingly, standard errors of 83 and 66 percent would indicate widely spaced sets of data points, thus degrading the accuracy that could be expected from either Equation 3.2 or 3.3.

Method 3 is based on a regression analysis of steam gage data in and near Pima County, Arizona. In the regression analysis, 101 streamflow gaging stations, with more than 2,000 station-years of systematic flood data, were used.

Although Pima County is located in southern Arizona, it is a semiarid desert environment that is very similar to the desert watershed that comprises Reata Pass. The topographic characteristics of this watershed (i.e., drainage area size, mean basin elevation, main channel length, main channel slope, and shape factor) were found to be within the minimum and maximum envelopes of the data-base used for the regression analysis. The average 100-year, 24-hour rainfall for Pima County is about 4.7 inches, which compares favorably with the 4.4 inches for Reata Pass.

The 100-year peak discharge regression equation used with Method 3 is listed as follows.

$$\text{Log } Q_{100} = 3.044 + 0.646 \text{ Log } A - 0.049(\text{Log } A)^2 + 0.729 \text{Log } S \qquad (3.4)$$

$$- 0.367(\text{Log } S)^2 - 0.614(\text{Log } S)(\text{Log } Sh)$$

where: $\text{Log } Q_{100}$ = base 10 logarithm of the 100-year peak discharge (cfs)

A = drainage area (square miles)

S = main channel slope (10 to 85 percent of channel length) in percent

Sh = watershed shape factor (L^2/A), dimensionless

Peak discharge is computed from Equation 3.4 by raising 10 to the power of the computed logarithm.

Method 4 was developed by A.J. Malvick using data from 143 stream gage locations in Arizona. The gage records varied in length from 6 to 69 years.

The regression analysis conducted by Malvick resulted in both a maximum envelope curve, which contained all the data points, and a regression equation specifically developed for the 100-year event. The 100-year regression equation is:

$$Q_{100} = 826A^{(0.789-0.067\text{Log } A)} \qquad (3.5)$$

where: Q_{100} = 100-year peak discharge (cfs)

A = drainage area (square miles)

Method 5 is a semi-empirical, peak discharge equation that acknowledges such watershed characteristics as watercourse length, mean slope, basin roughness, length to center of gravity, drainage area size, and infiltration loss (SCS curve number). Although this procedure was developed in Pima County, it is based on physical watershed characteristics that allow it to be used in any semi-arid environment. It should be noted, however, that the procedure is limited to homogeneous sub-basins whose times of concentration are less than three hours.

A very sensitive parameter in Method 5 is the watershed basin factor, which represents the mean value of the roughness coefficient for all the principle watercourses in the contributing drainage basin. A basin factor of 0.045 was used with this procedure in order to be consistent with the Manning's "n" value used for the kinematic wave channel routing operations in the HEC-1 model.

It should be noted that this procedure is based on short duration curve numbers that are published in the procedural manual. Accordingly, Table 3.1 lists both 24-hour and one-hour curve numbers for each test basin. The one-hour curve numbers reflect data from the Pima County procedure.

Method 6 is based on a graphical peak discharge procedure described in the June 1986 edition of TR-55. Peak discharge is computed with the following equation.

$$q_p = q_u A_m Q F_p \tag{3.6}$$

where:

q_p = peak discharge (cfs)

q_u = unit peak discharge (cfs/sq. mi.)

A_m = drainage area (sq. mi.)

Q = runoff (inches)

F_p = pond and swamp adjustment factor

Using curves published in TR-55, the unit peak discharge (q_u) is computed as a function of time of concentration, initial abstractions, and 24-hour rainfall. For the Reata Pass watershed, times of concentration (Tc) were computed using procedures from TR-55. Flow lengths, slopes, curve numbers, etc. used for the Tc calculations were consistent with those used in the HEC-1 model. An SCS Type II rainfall distribution was used for selecting q_u. TR-55 does not publish curves for a Type IIA distribution.

Method 7 is based on the results of a log-Pearson III analysis conducted by the Flood Control District of Maricopa County (FCDMC) for stream gage locations within Maricopa County. Using a log-Pearson III distribution, a 100-year peak discharge was computed for 29 gage locations. The 100-year peak discharge values were then reduced to unit peak discharge (cfs/sq. mi.) and plotted as a function of drainage area size. Two lines were visually fit to those plots in order to establish both

a maximum envelope and an average response line. These two curves were applied to the Reata Pass watershed to estimate the maximum and average 100-year peak discharge values.

Method 8 is a 100-year envelope curve developed by Boughton, et. al. This is a statistically based curve that encompasses the 100-year peak discharges from 24 stream gage stations in southeastern Arizona. As such, it is not a maximum discharge envelope curve, but instead, establishes a statistical upper limit for the 100-year event. The equation defining this curve is (note the use of metric units):

$$Q_{100ENV}=31.6A^{(0.77-0.0664\text{Log } A)} \tag{3.7}$$

where: Q_{100ENV} = 100-year peak discharge envelope value (cubic meters/sec)

A = drainage area (square kilometers)

Equation 3.7 is published as a conservative estimate for 100-year peak discharge values within the study boundaries used for the data-base. Application of this envelope curve to areas in central Arizona is not considered to be a serious over-extension of the curve, due to the similar semiarid desert conditions that exist in both locations. As a result, it should provide a conservative upper boundary to use in evaluating the peak discharges developed for Reata Pass.

Method 9 is the envelope curve developed by Creager. This curve is defined by an equation that envelopes a set of unusual flood discharges that were recorded in both the United States and abroad. The following equation was developed in the early 1940's.

$$q = 46CA^{(0.894A^{-0.048})-1} \tag{3.8}$$

where: q = peak discharge per unit drainage area (cfs/square mile)

A = drainage area (square miles)

C = coefficient

With C = 100, Equation 3.8 was found to envelope all but three of the floods that occurred in Texas, North Carolina, and the Philippines.

Method 10 is the Costa envelope curve which was based on an analysis of the 12 largest flash floods ever measured in the semiarid areas of the United States. Using this data, J.E. Costa developed an envelope curve relating drainage area to maximum peak discharge.

Method 11 is based on a new envelope curve developed by the Corps of Engineers as part of a hydrology study completed in 1988 for Clark County, Nevada. This curve is based on recorded peak discharge data for Arizona, Nevada, and New Mexico.

Table 3.1 is a summary of all the peak discharge verification calculations performed for the Reata Pass watershed. Eleven concentration points within the watershed were selected for verification. Table 3.1 lists the physical characteristics associated with each of these locations, as well as the peak discharge estimates. The HEC-1 results for Scenario 13 are also listed in Table 3.1 for comparison with the other methods.

It should be noted that some of the peak discharge estimating curves do not extend below one square mile of drainage area, i.e., FCDMC, Costa, COE. As a result, no discharges are shown at the intersection of these methods and locations.

Concentration Points 27 and 1480 are also listed in Table 3.1. These locations are not part of the Reata Pass watershed. They are part of two adjacent, northerly watersheds which will be discussed in Section 4.0 of this report.

Figures 3.1 through 3.11 graphically illustrate the data listed in Table 3.1.

When reviewing the data presented in this section, it is important to be aware of the differences between the envelope curves and the methods that relate to a specific return interval. The envelope curves are not associated with a specific return interval, such as a 100-year event; i.e., these curves are simply drawn on the basis of enveloping all the floods included in the data base for each method. The one exception is the Boughton envelope curve, which was drawn on the basis of enveloping the discharges of all 100-year floods included in its data base. As such, the Boughton curve is not a true predictor of the 100-year event, but rather a statistical upper envelope of 100-year events.

4.0 IMPACT ON ADJACENT WATERSHED MODELS

As stated previously, a comprehensive, integrated drainage plan has been developed on the basis of the HEC-1 models presented in the report for the Upper Indian Bend Wash Regional Drainage and Flood Control Plan (1990). The Reata Pass watershed is an integral part of this proposed Regional Plan. Should the data presented in this current study result in permanent revisions to any channel routing methodologies, or other input parameters to the Reata Pass HEC-1 model, it would seem that consideration should be given to applying any such revisions to the remaining Regional Plan HEC-1 models. Such an approach would promote consistency in the modeling techniques used throughout the watershed that will be served by the Regional Plan.

However, a potential problem may be created if area-wide application of the Reata Pass modeling technique revisions cause peak discharges in adjacent watersheds to fall below acceptable design levels.

Although the Scope of Work for this study does not include an extension of the Reata Pass sensitivity analysis to adjacent watersheds, an abbreviated analysis was performed for a major concentration point on Rawhide Wash and at an apex location on FEMA Fan 5. The hydrologic results of this analysis are summarized at the bottom of Table 3.1.

Concentration Point 27 is the most southerly of three FEMA apex locations on Rawhide Wash. CP27, which is located about one-half mile north of Happy Valley Road near the intersection with Hayden Road, drains an area of approximately 14.02 square miles. Based on HEC-1 Model PH30.24I (see Volume 2 of the Paradise Valley Fan Terrace, Concept Drainage Study, February 1990), the predicted 100-year peak discharge for this site is 9,710 cfs. This discharge is based on existing land-use conditions and a 24-hour, Type IIA rainfall distribution. Figure 4.1 presents a comparison of this discharge to results of the same verification procedures discussed in Section 3.0 of this report.

When the channel routing operations for this model were converted to eight-point, modified Puls, the 100-year peak discharge at CP27 decreased to 8,514 cfs with the 24-hour rainfall distribution and to 7,077 cfs with the six-hour hypothetical distribution. Although these reduced values would appear

to still be reasonable design discharges, a review of the model output revealed a 68 percent decrease in the peak discharge associated with a single, major routing operation (24-hour storm). This is not considered a reasonable level of hydrograph attenuation. Therefore, it can only be concluded that any area-wide revision of channel routing techniques would have to be done very carefully and with a detailed review of the model output.

The upper apex on FEMA Fan 5 was also examined to determine the impacts of channel routing operations. This site, which is located near the intersection of Lone Mountain Road and Hayden Road, drains approximately 2.99 square miles. Based on HEC-1 Model PH20.24I (Volume 2, Paradise Valley Fan Terrace Study), the 100-year peak discharge for this location (CP 1480) is 4,124 cfs. This discharge is also based on existing land-use conditions and a 24-hour, Type IIA rainfall distribution. Figure 4.2 compares this site to the results of the verification procedures discussed in Section 3.0.

When the channel routing procedures for FAN 5 were converted from kinematic wave to eight-point modified Puls, the 100-year peak discharge at CP 1480 decreased to 3,698 cfs with 24-hour rainfall distribution and to 2,360 cfs with the six-hour hypothetical distribution. Again, large percentage reductions in discharge were noted in some channel routing operations. However, the end result still seems to yield a reasonable design discharges at CP 1480, when compared to the verification calculations.

As a matter of technical interest, it should be noted that the results from the modified Puls routing operations referenced for CP 27 and CP 1480 were based on a routing step (variable NSTPS) equal to one. When NSTPS was reset as a function of channel length, velocity, and time interval, the 68 percent attenuation for the routing operation through Rawhide Wash was reduced to 41 percent. Although this still seems somewhat high, it may have physical justification because of the potentially wide, shallow flow that might occur through portions of this routing reach.

In summary, the data presented in this Section would indicate that extreme care should be used in transferring any Reata Pass modeling techniques to other portions of the Upper Indian Bend Wash watershed.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The results of this study indicate that several changes in modeling techniques could be made to lower the peak 100-year discharge at the apex of the Reata Pass fan. The lower discharges created by these techniques are supported by the results of numerous independent peak discharge calculations. However, in arriving at a decision as to which techniques should be adopted (and which 100-year peak discharge is the most appropriate), two key issues should be given careful consideration.

The first issue is the fact that the hydrologic model for the Upper Indian Bend Wash watershed was created for the design of a major flood control system. The model is based on the occurrence of a very severe rainfall distribution superimposed on a severe antecendent moisture condition (AMC 2). Both of these conditions are prudent design assumptions for the development of large regional flood control systems. Both conditions are routinely used by the Soil Conservation Service in the design of flood control structures. Safety considerations would seem to support the use of such severe assumptions for flood hazard studies as well.

Attempts to use regional regression equations and stream gage data to assess the realism of the peak discharges produced by such modeling assumptions should be done with caution. Very little stream gage data in the southwestern United States has a sufficient record length to produce reliable estimates of 100-year floods. As a rule-of-thumb, gage records should not be extrapolated beyond twice the record length.

It is very doubtful that any of the gaged watersheds (including those used for regression equations) referenced in this study have ever experienced a true 24-hour, Type IIA storm on a nearly saturated soil. It would be interesting to replicate such an event on a gaged watershed and record the results. Perhaps a more common event for this region would be a much shorter duration storm on a nearly dry soil.

In the absence of a recorded rainfall/runoff event, which could be used for model calibration, engineering judgement must be used to select an appropriate set of modeling assumptions that will produce an acceptable design discharge. Safety considerations dictate that the design discharge

contain a certain degree of conservatism, while flood control system cost considerations warrant that such conservatism not be excessive.

The second issue to be considered is the impact that any Reata Pass modeling revisions would have on the design discharges generated within adjacent watersheds. Ideally, the hydrologic model used for the Upper Indian Bend Wash Regional Plan should incorporate the same modeling assumptions and logic throughout the watershed being served by the Regional Plan.

Based on a review of the 15 modeling scenarios discussed in Section 2.0, and the peak discharge comparisons presented in Section 3.0, Scenario 13 would appear to be a reasonable and easily implemented modeling revision for Reata Pass. This scenario would only require that the singular, kinematic wave routing operations be converted to eight-point, modified Puls routing and that the SCS Type IIA, 24-hour rainfall distribution be replaced by the six-hour hypothetical distribution generated by the PH record in HEC-1. This option will <u>not</u> require any new sub-basin delineations and the associated adjustments to curve numbers, overland flow components and channel routing parameters that revised basin delineations would necessitate.

Scenario 13 could easily (although maybe somewhat time consuming) be expanded to the adjacent HEC-1 models that comprise the remainder of the Upper Indian Bend Wash watershed.

However, the adoption of Scenario 13 to an area-wide basis should be done with caution. As noted in Section 4.0 of this report, normal depth, modified Puls channel routing was found to produce unrealistic levels of hydrograph attenuation at some locations near FEMA Fans 5 and 6. The output from any revised HEC-1 models should be carefully monitored to detect such problems.

Two suggested remedies for this attenuation problem would be to: 1) increase the number of routing steps (variable NSTPS) to a number greater than one (using procedures outlined in the HEC-1 Users Manual); or 2) divide the routing operation into shorter lengths and leave NSTPS = 1. The second of these two remedies would be the preferred approach since it would not violate a basic assumption of Scenario 13 (i.e., Scenario 13 assumes NSTPS = 1).

Based on discussions with the Dam Safety Division of the Arizona Department of Water Resources (ADWR), it does not appear that a 24-hour storm <u>volume</u> would be a mandatory design criteria for

the design of several large detention basins that are presently an integral part of the proposed Regional Plan. Should these detention basins fall under ADWR jurisdiction, ADWR would not presently impose a minimum volume requirement on the required flood control storage. This storage could be any desired volume. ADWR would only regulate the spillway design to ensure that the structural integrity of the detention basins would not be jeopardized once the allotted flood control storage was consumed. Accordingly, the use of a six-hour storm duration does not appear to be in conflict with any current regulatory criteria for dam design.

In conclusion, the adoption of the Scenario 13 modeling revisions would appear to be a fair compromise between the conservatism required for flood control system design and the economic concerns related to construction costs.

FIGURES

Figure 3.1
HEC-1 Peak Discharge Verification
CP 41 & 40C2 - Reata Pass

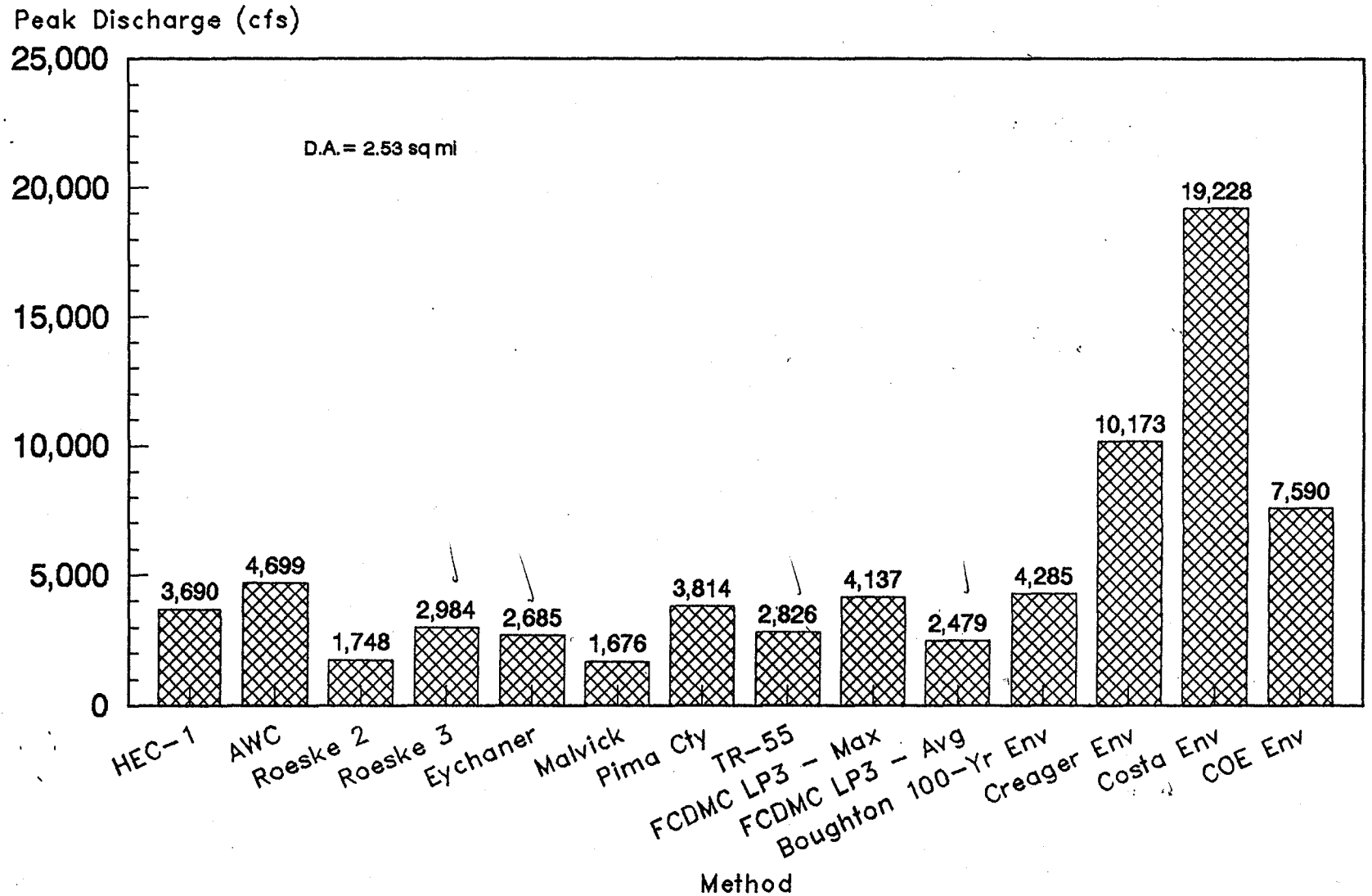


Figure 3.2
HEC-1 Peak Discharge Verification
CP 45 & 44C2 - Reata Pass

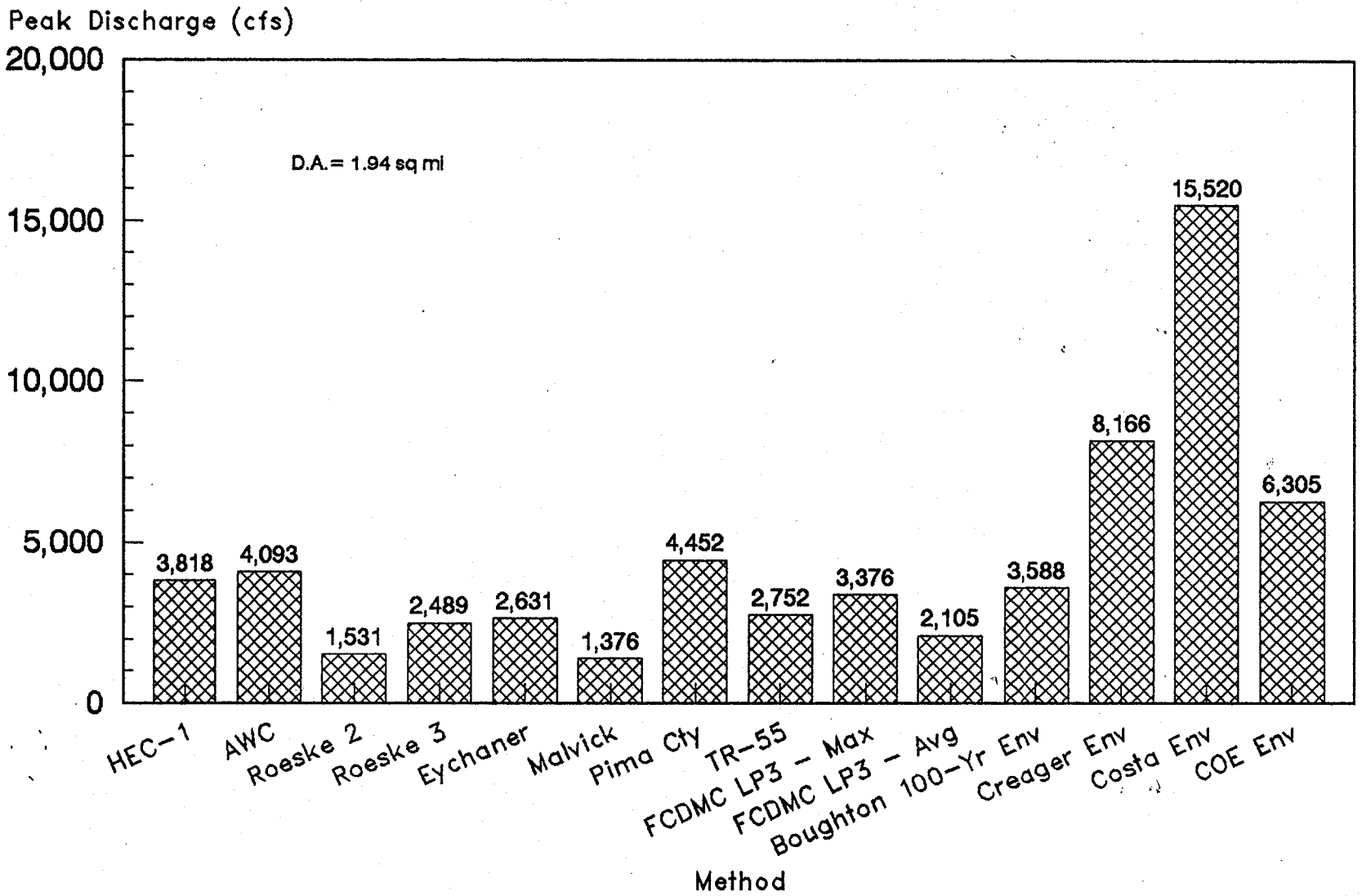


Figure 3.3
HEC-1 Peak Discharge Verification
CP 47 - Reata Pass

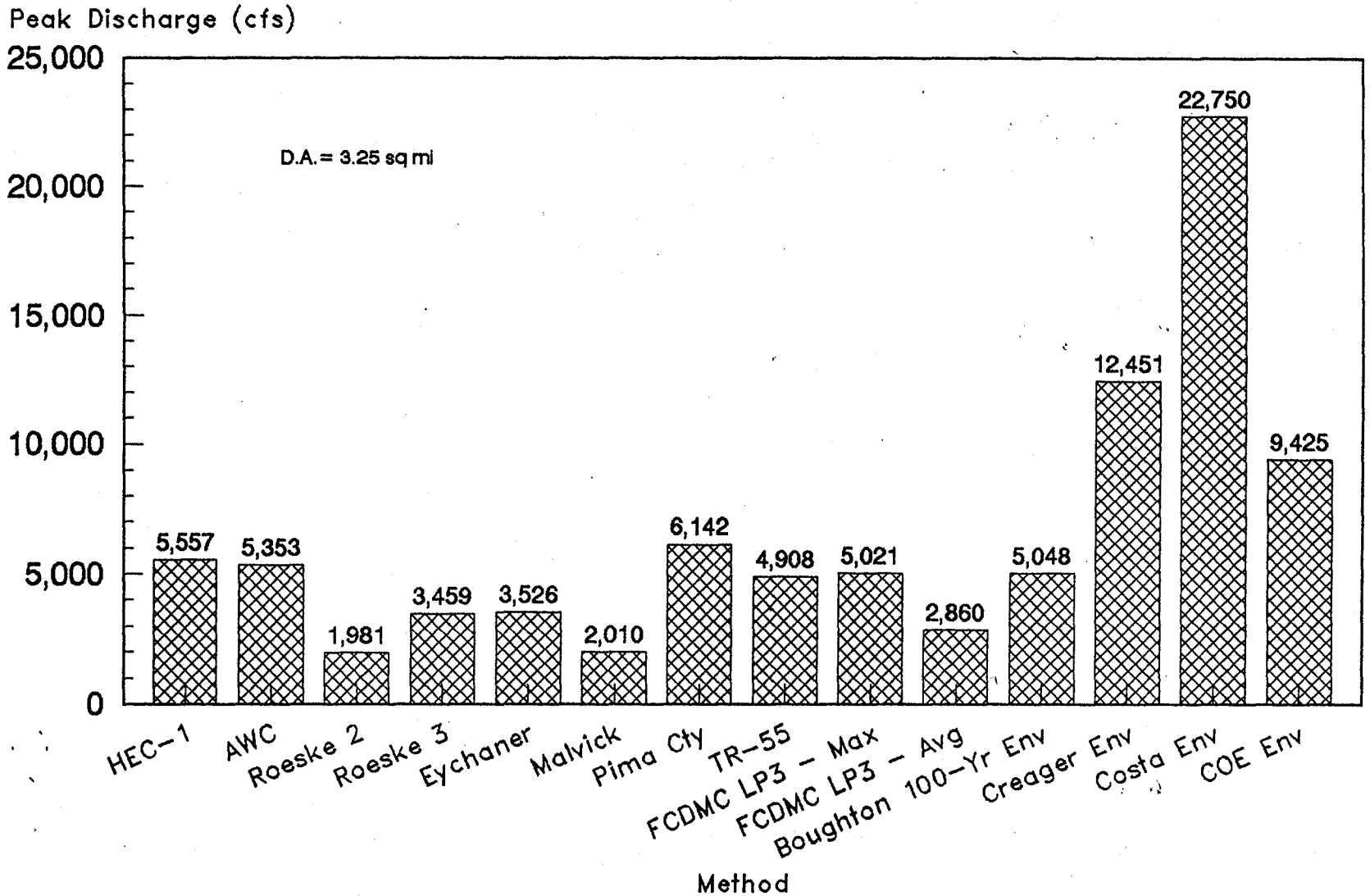


Figure 3.4
HEC-1 Peak Discharge Verification
CP 51 - Reata Pass

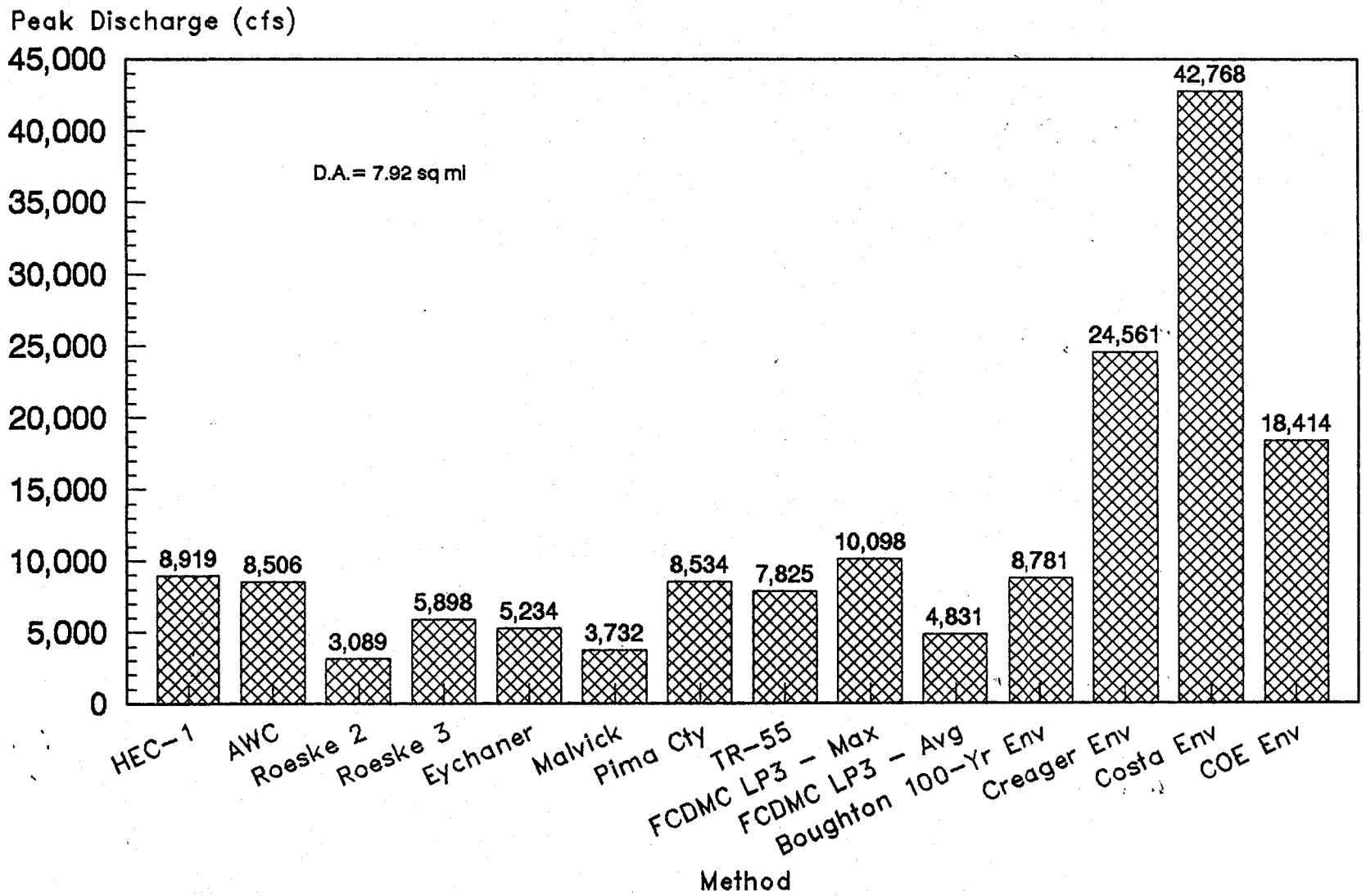


Figure 3.5
HEC-1 Peak Discharge Verification
SUB 39F - Reata Pass

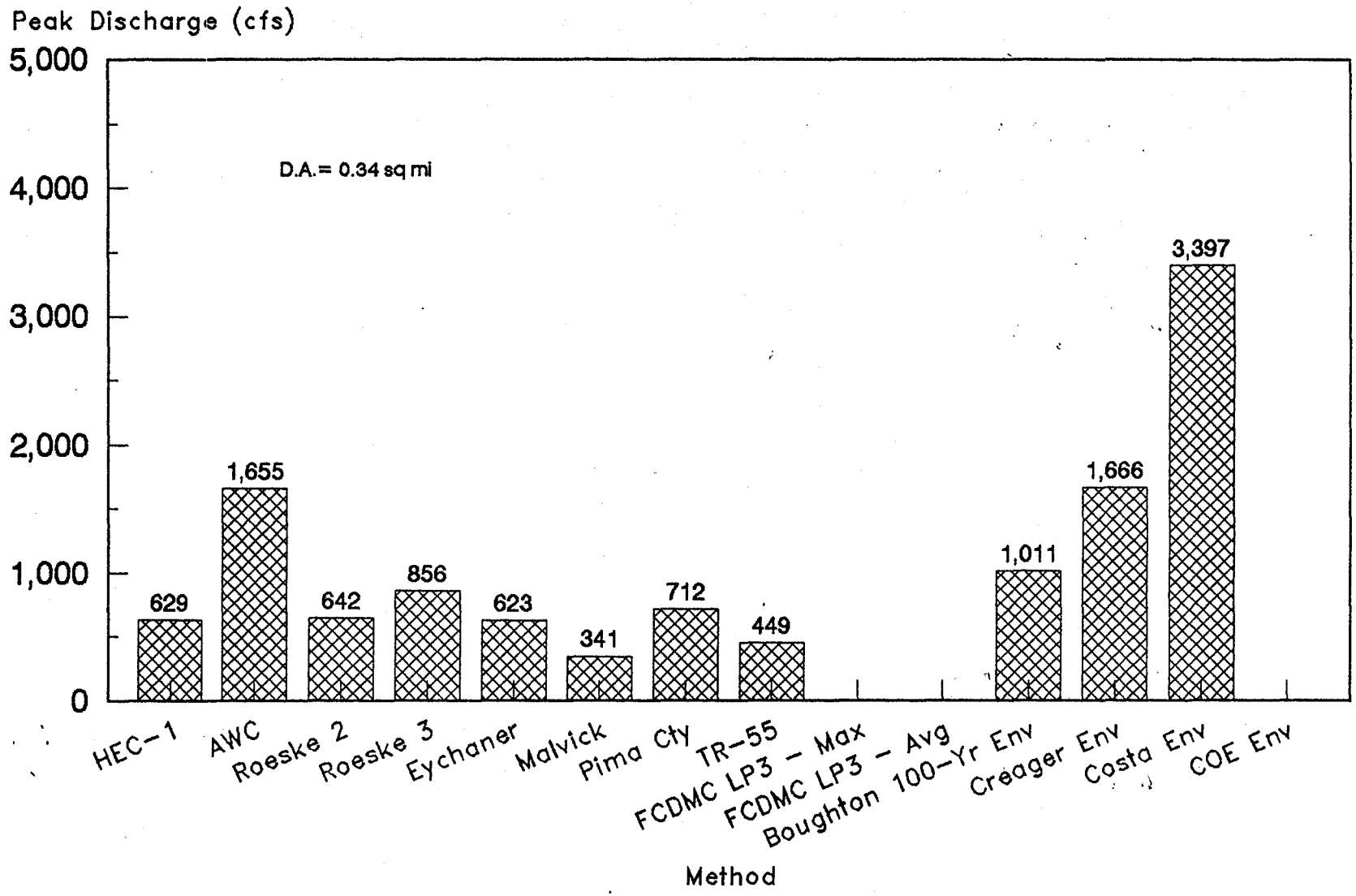


Figure 3.6
HEC-1 Peak Discharge Verification
SUB 40A - Reata Pass

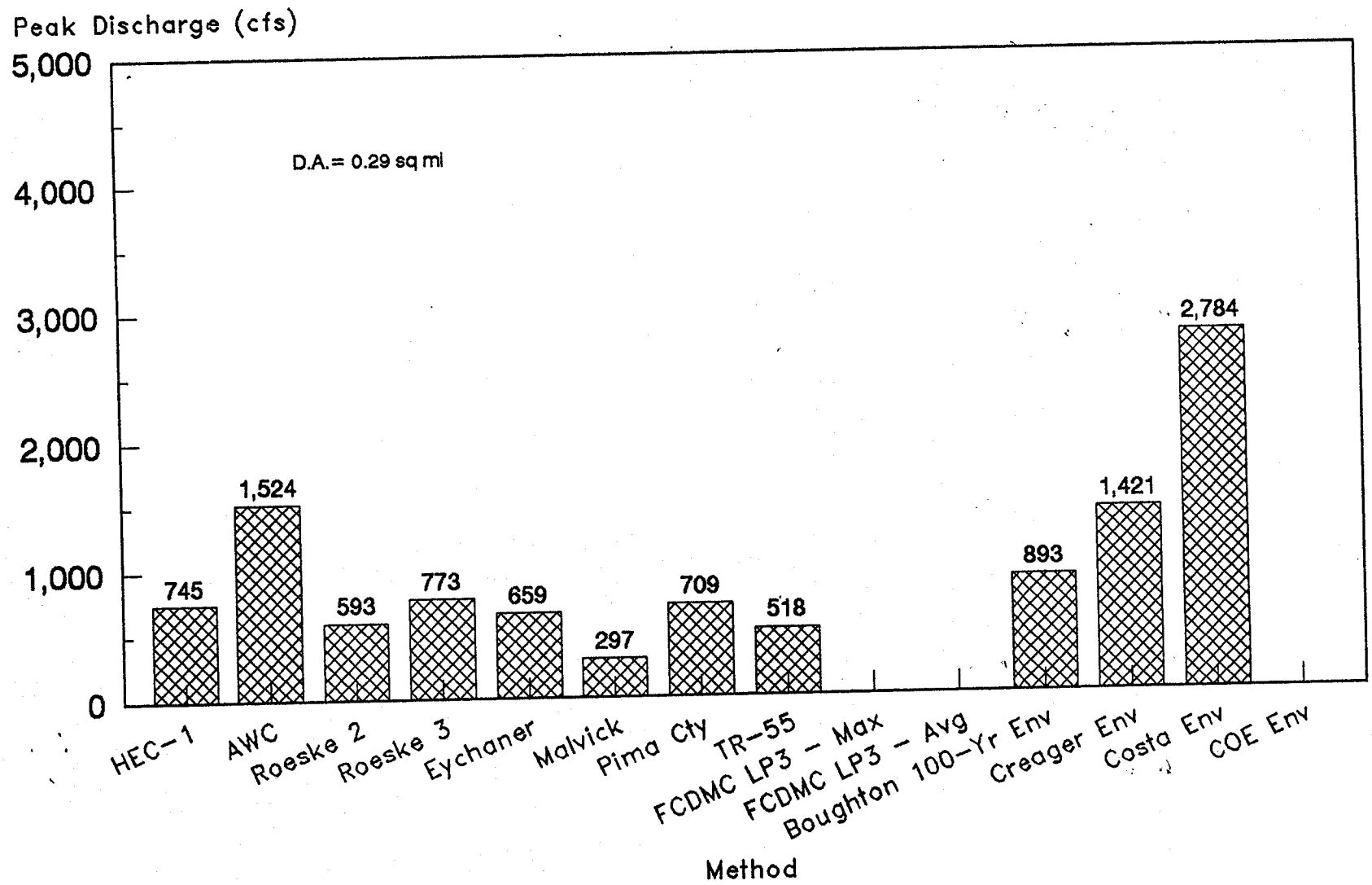


Figure 3.7
HEC-1 Peak Discharge Verification
SUB 41E - Reata Pass

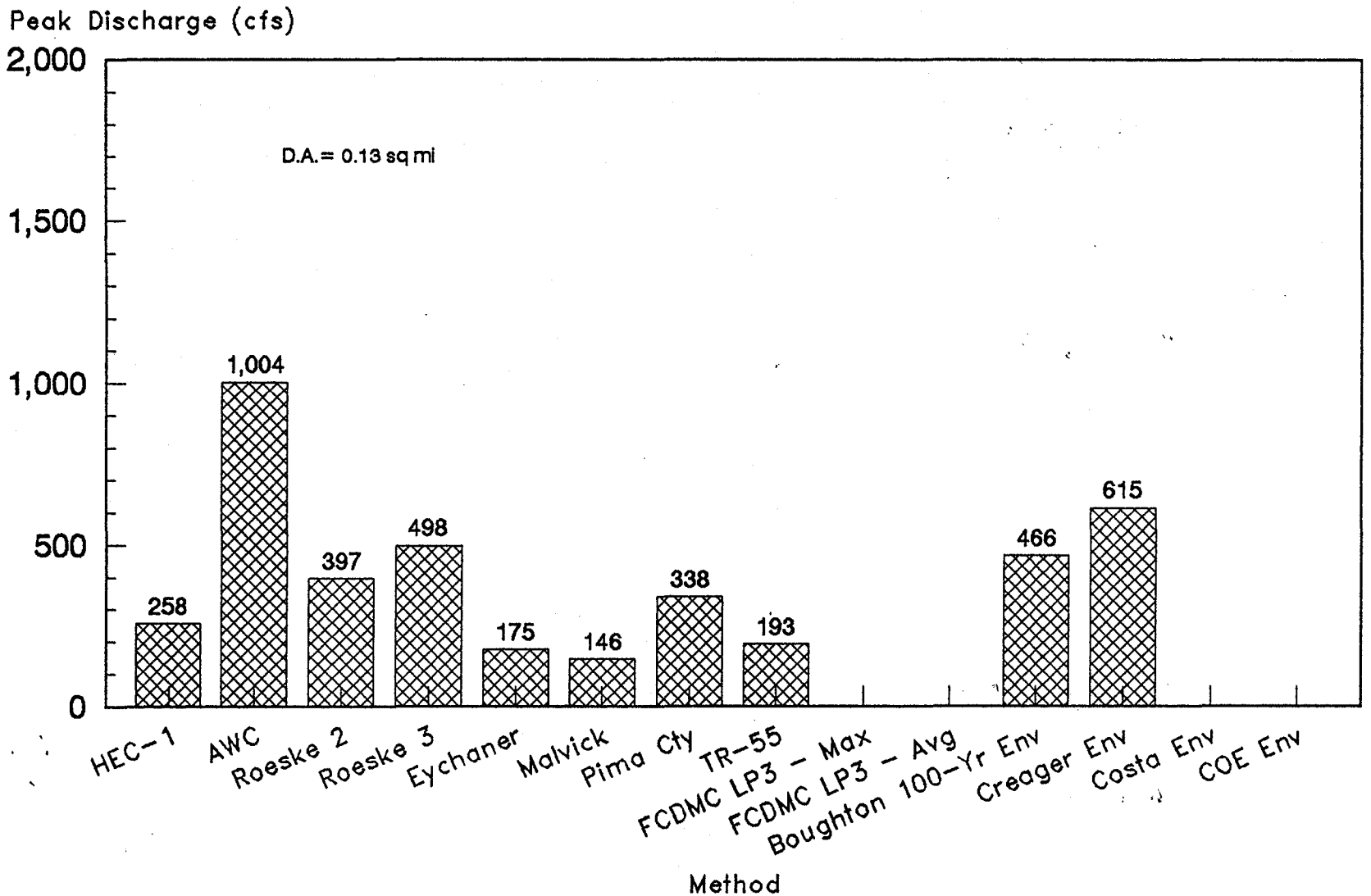


Figure 3.8
HEC-1 Peak Discharge Verification
SUB 42 - Reata Pass

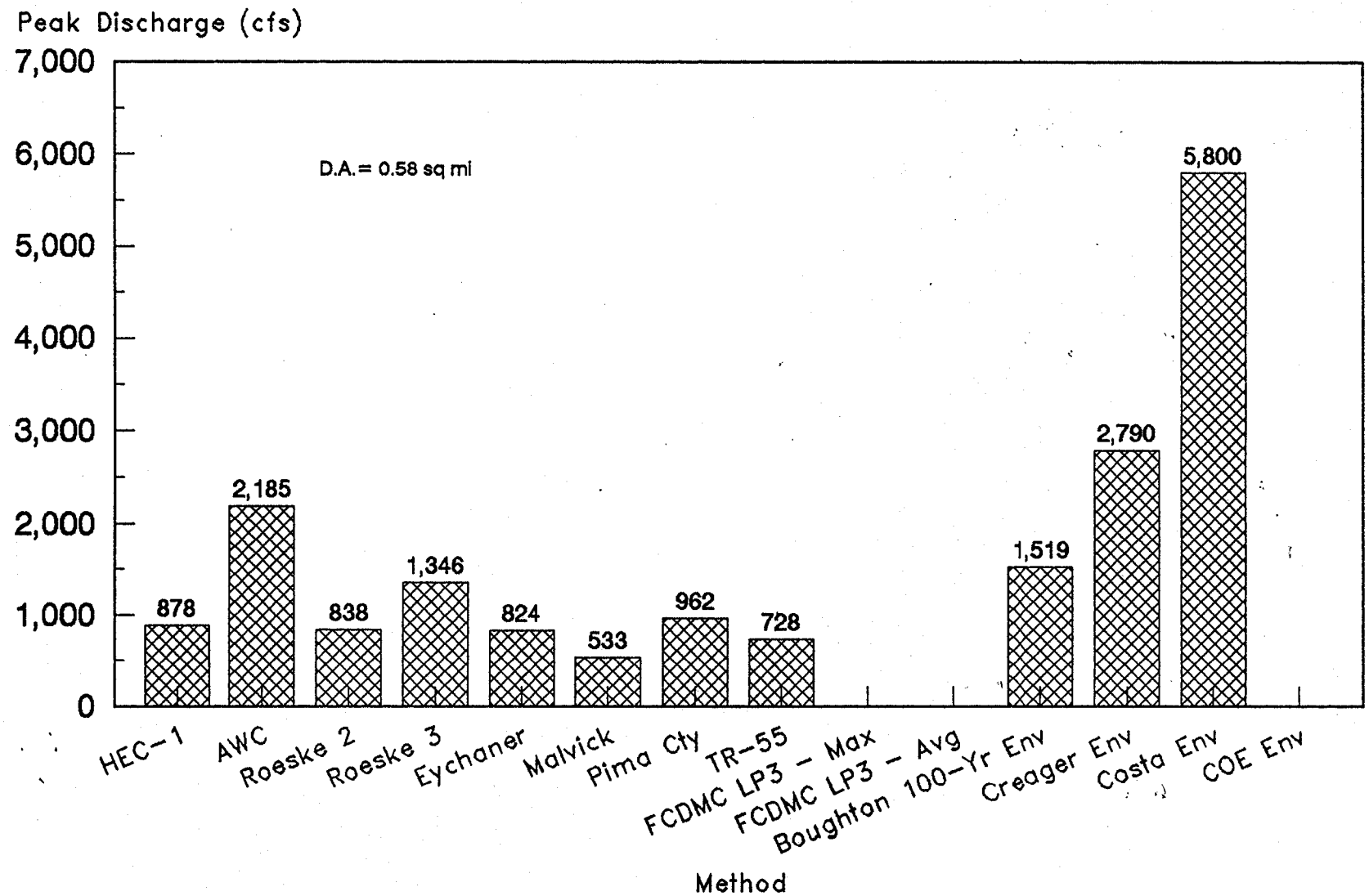


Figure 3.9
HEC-1 Peak Discharge Verification
SUB 43B - Reata Pass

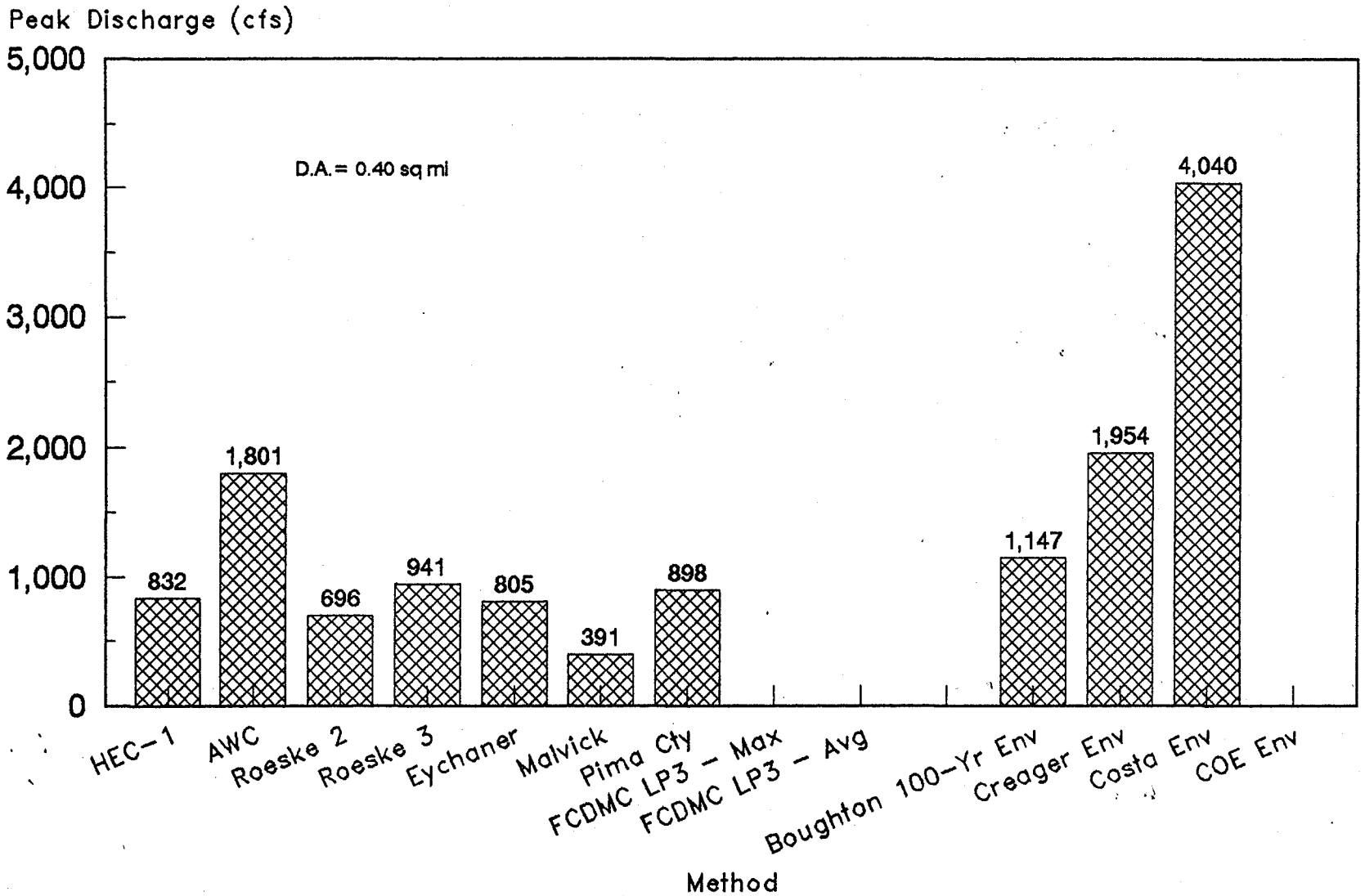


Figure 3.10
HEC-1 Peak Discharge Verification
SUB 44B - Reata Pass

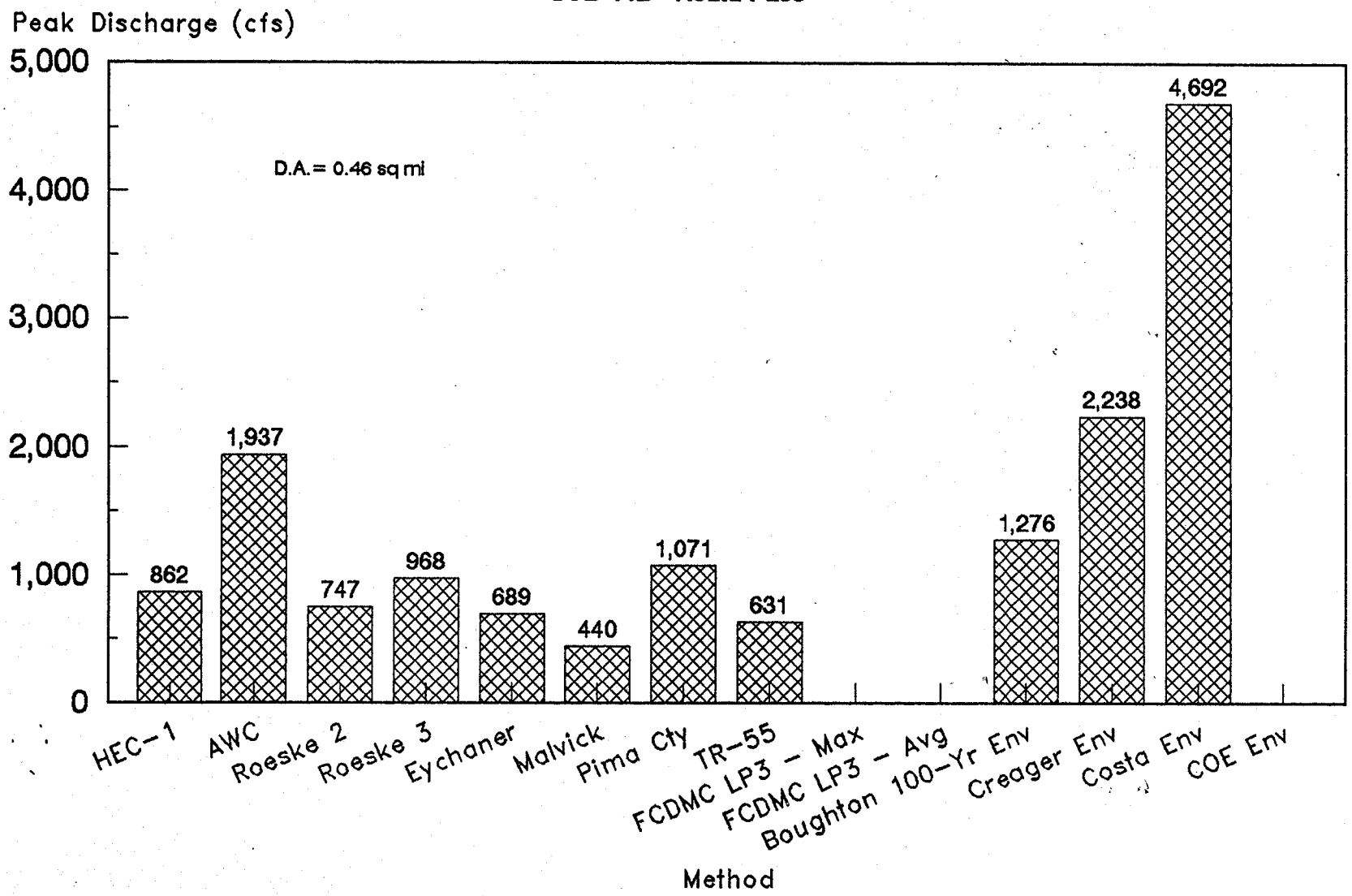


Figure 3.11
HEC-1 Peak Discharge Verification
SUB 46 - Reata Pass

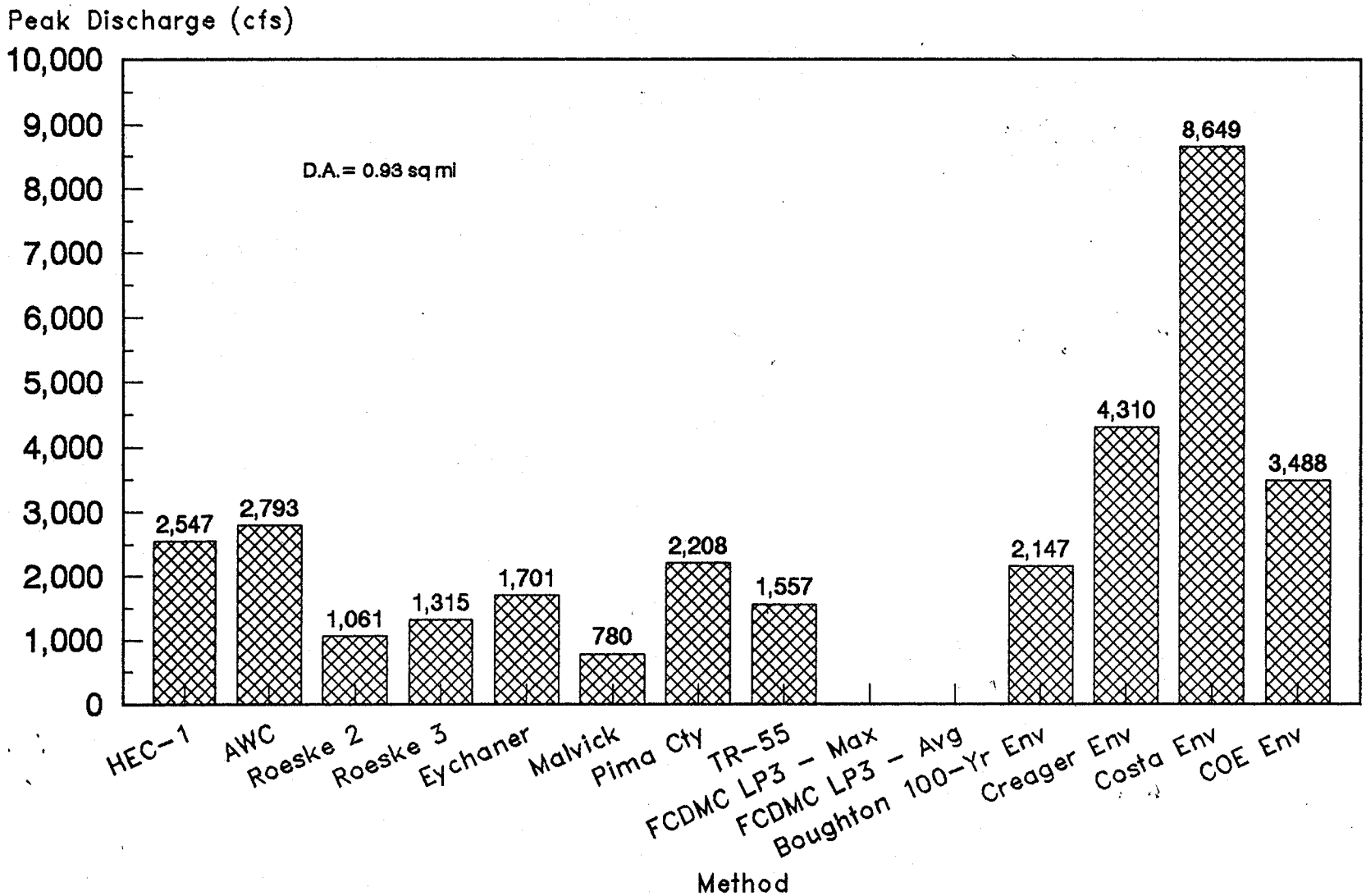


Figure 4.1
HEC-1 Peak Discharge Verification
CP 27 - Rawhide Wash

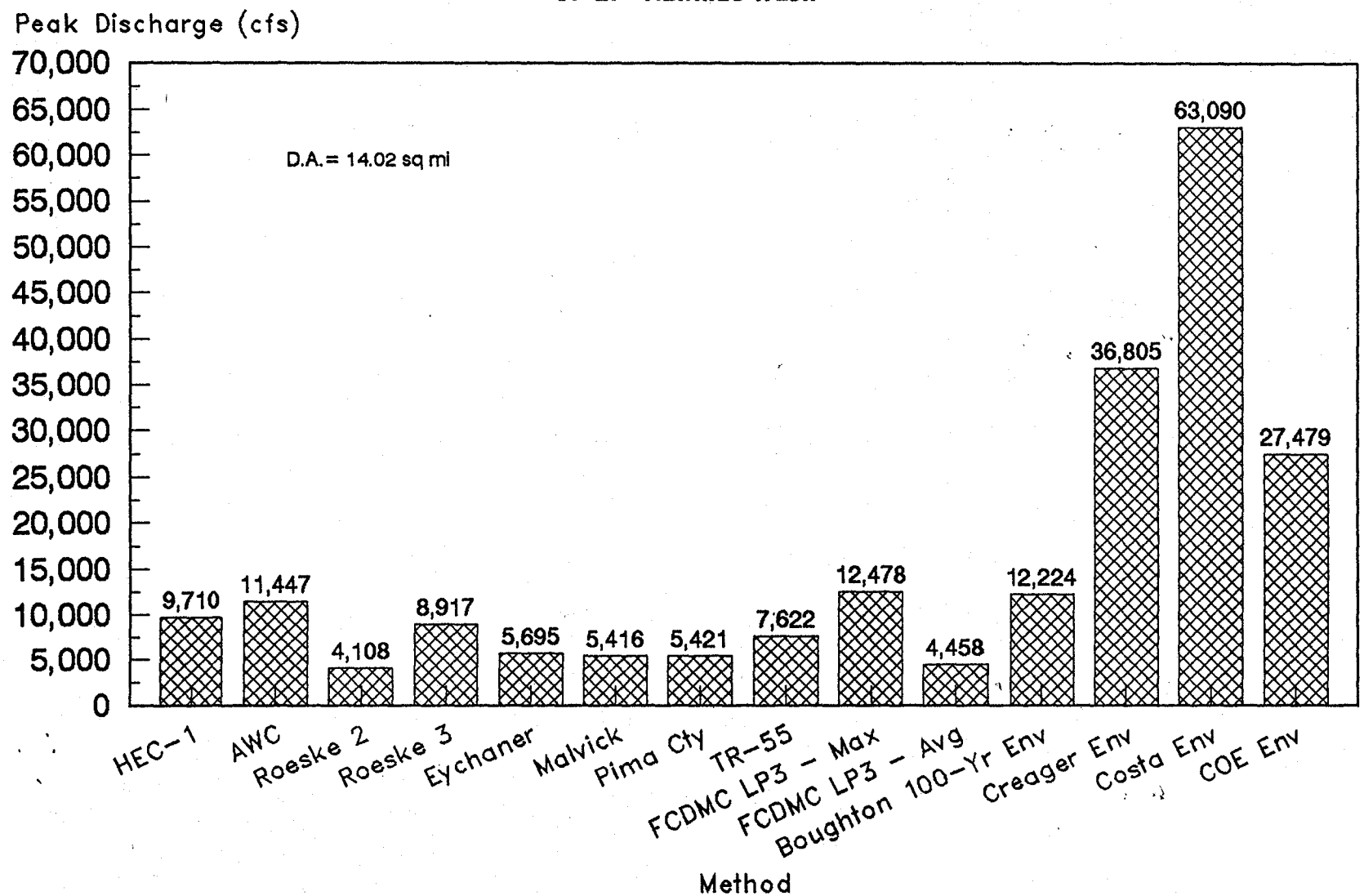
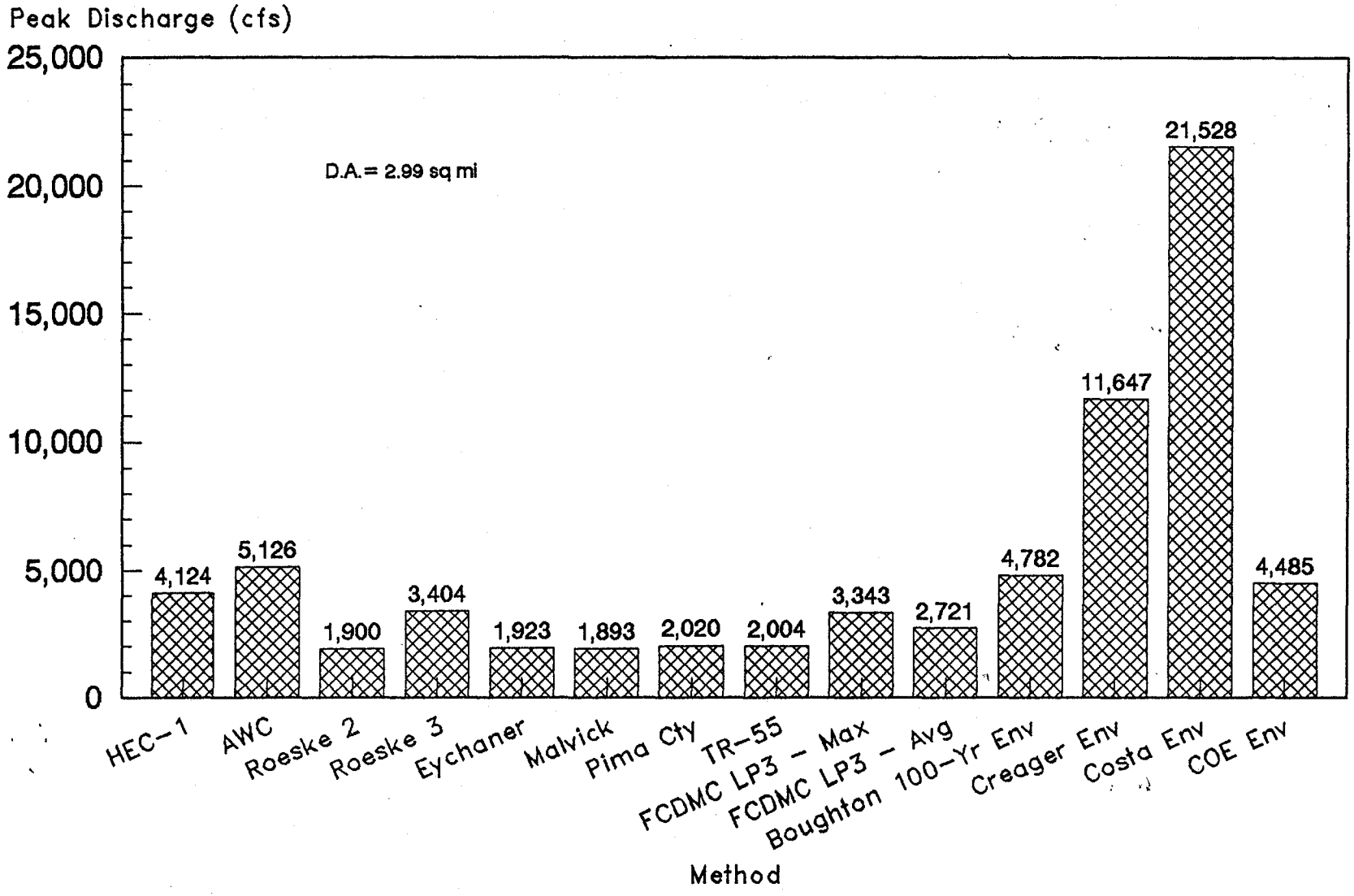


Figure 4.2
HEC-1 Peak Discharge Verification
CP 1480 - FEMA Fan 5



TABLES

Table 2.1
Matrix Of Modeling Revisions
Reata Pass Sensitivity Analysis

Scenario	Rainfall			Infiltration		Drainage Area		Channel Routing				Routing Steps		Manning's Roughness For Kinematic Channel Routing	
	24-Hr Type IIA	12-Hr Hypo.	6-Hr Hypo.	CN	Green-Ampt	Original Sub-Basins	Smaller Sub-Basins	All Kinematic Wave	Kinematic Plus Modified Puls	Muskingum-Cunge, 4-Point & 8-Point	Muskingum-Cunge, 8-Point	NSTPS > 1	NSTPS = 1	n=.045	n=.055
1	x			x		x		x				n/a	n/a	x	
2	x			x		x			x			x		x	
3	x			x			x	x				n/a	n/a	x	
4	x			x			x		x			x		x	
5	x				x		x		x			x		x	
6		x		x			x		x			x		x	
7			x	x			x		x			x		x	
8	x			x			x			x		n/a	n/a	n/a	
9	x			x			x				x	n/a	n/a	n/a	
10	x			x			x		x				x	x	
11	x			x		x		x				n/a	n/a		x
12	x			x		x			x				x	x	
13			x	x		x			x				x	x	
14			x	x		x			x			x		x	
15			x	x			x		x				x	x	

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Table 2.2
Summary Of Modeling Revision Results At CP 51
Reata Pass Sensitivity Analysis
Existing Land-use Conditions

Model Scenario	Q100 @ CP 51 (cfs)		
	1985 Version	1988 Version	1990 Version
1	13,810	15,186	15,144
2	12,668	12,862	12,878
3	12,680	* 2,181	13,523
4	11,786	12,099	12,186
5	–	14,597	14,646
6	–	–	11,564
7	–	–	10,330
8	–	–	12,715
9	–	–	–
10	10,745	10,667	10,732
11	12,962	14,103	13,983
12	11,583	11,543	11,592
13	9,110	9,264	9,324
14	–	–	11,099
15	–	–	8,919

File: TAB22.WK1 * HEC-1 channel routing error

Table 2.3
Weighted Curve Number Calculations, AMC II
Reata Pass Alluvial Fan Study, North Scottsdale, Arizona
Percentage of Soil Group by Sub-Basin

Sub-Basin Number	Soil Survey HSG & Curve Number						Weighted CN	Sum of Percents
	A	A/B	B	B/C	C	D		
	60	67	74	78	82	86		
39A	0	0	0	0	0	100	86.0	100
39B	0	0	50	0	0	50	80.0	100
39C	0	0	0	0	0	100	86.0	100
39D	0	0	0	0	0	100	86.0	100
39E	0	0	1	0	0	99	85.9	100
39F	0	0	0	0	0	100	86.0	100
39G	0	0	0	0	0	100	86.0	100
39H	0	0	0	0	0	100	86.0	100
39I	0	19	18	0	13	50	79.7	100
40A	0	0	0	0	0	100	86.0	100
40B	0	2	0	0	0	98	85.6	100
40C	0	35	19	0	0	46	77.1	100
41A	0	9	72	0	0	19	75.7	100
41B	0	0	40	0	0	60	81.2	100
41C	0	50	22	0	0	28	73.9	100
41D	0	15	77	0	0	8	73.9	100
41E	0	1	44	0	0	55	80.5	100
41G	1	22	77	0	0	0	72.3	100
43A	0	0	0	0	0	100	86.0	100
43B	0	0	0	0	0	100	86.0	100
43C	0	0	0	0	0	100	86.0	100
43D	0	15	0	0	0	85	83.2	100
43E	0	1	0	0	0	99	85.8	100
44A	0	0	0	0	0	100	86.0	100
44B	0	0	18	0	0	82	83.8	100
44C	0	18	0	0	0	82	82.6	100

File: RPCN.WK1

Table 2.4
Weighted Green–Ampt Parameters
Reata Pass Alluvial Fan Study, North Scottsdale, Arizona
Percentage of Soil Units by Sub–Basin

Sub–Basin Number	SCS Soil Mapping Unit No. 6	8	31	33	40	41	61	63	96	121	Volumetric Soil Moisture Deficit DTHETA (Dry)	Wetting Front Capillary Suction PSIF (inches)	Hydraulic Conductivity XKSAT (in/hr)	Sum of Percents
39A	0	0	0	0	0	0	100	0	0	0	0.30	5.97	0.24	100
39B	0	0	0	0	0	0	50	0	50	0	0.29	6.91	0.18	100
39C	0	0	0	0	0	0	100	0	0	0	0.30	5.97	0.24	100
39D	0	0	0	0	0	0	100	0	0	0	0.30	5.97	0.24	100
39E	0	0	0	0	0	0	98	0	2	0	0.30	6.01	0.24	100
39F	0	0	0	0	0	0	97	3	0	0	0.30	5.97	0.24	100
39G	1	0	0	0	0	0	92	7	0	0	0.30	5.94	0.25	100
39H	0	0	0	0	0	0	70	30	0	0	0.30	6.00	0.24	100
39I	25	0	0	15	0	0	30	15	15	0	0.32	5.01	0.48	100
40A	0	0	0	0	0	0	96	4	0	0	0.30	5.97	0.24	100
40B	8	0	0	0	0	0	57	35	0	0	0.30	5.72	0.31	100
40C	36	0	0	0	0	0	2	37	0	25	0.32	4.71	0.64	100
41A	12	0	0	0	0	0	0	23	0	65	0.31	5.53	0.52	100
41B	0	0	0	0	0	0	0	60	0	40	0.30	6.02	0.33	100
41C	50	0	0	0	0	0	30	0	20	0	0.32	4.56	0.69	100
41D	15	0	0	0	0	0	0	5	0	80	0.31	5.40	0.58	100
41E	2	0	0	0	0	0	0	50	0	48	0.30	5.93	0.37	100
41G	25	0	0	0	0	0	0	0	75	0	0.29	6.49	0.38	100
42	1	1	0	0	4	0	22	22	50	0	0.29	6.86	0.19	100
43A	0	0	0	0	0	0	77	23	0	0	0.30	6.00	0.24	100
43B	0	0	0	0	0	0	100	0	0	0	0.30	5.97	0.24	100
43C	1	0	0	0	0	0	99	0	0	0	0.30	5.93	0.25	100
43D	17	0	0	0	0	0	83	0	0	0	0.31	5.36	0.40	100
43E	0	0	0	0	0	0	76	24	0	0	0.30	6.00	0.24	100
44A	0	0	0	0	0	0	100	0	0	0	0.30	5.97	0.24	100
44B	0	0	15	0	0	0	47	20	0	18	0.31	5.61	0.31	100
44C	14	0	0	0	0	0	43	43	0	0	0.31	5.52	0.37	100
45	15	0	0	0	0	0	35	35	0	15	0.31	5.47	0.42	100
45A	0	0	80	0	0	0	10	10	0	0	0.34	4.01	0.37	100
46	0	0	92	0	0	0	4	4	0	0	0.35	3.70	0.39	100
47	0	25	20	0	24	3	25	3	0	0	0.33	4.78	0.32	100
48	0	33	0	0	67	0	0	0	0	0	0.32	4.95	0.32	100
50	0	0	80	0	20	0	0	0	0	0	0.34	3.93	0.38	100

File: REATAGA.WK1

Table 3.1

Summary Of Independent Peak Discharge Calculations Used For HEC-1 Model Verification

100-Year Event

Rosita Pass Watershed, Scottsdale, Arizona

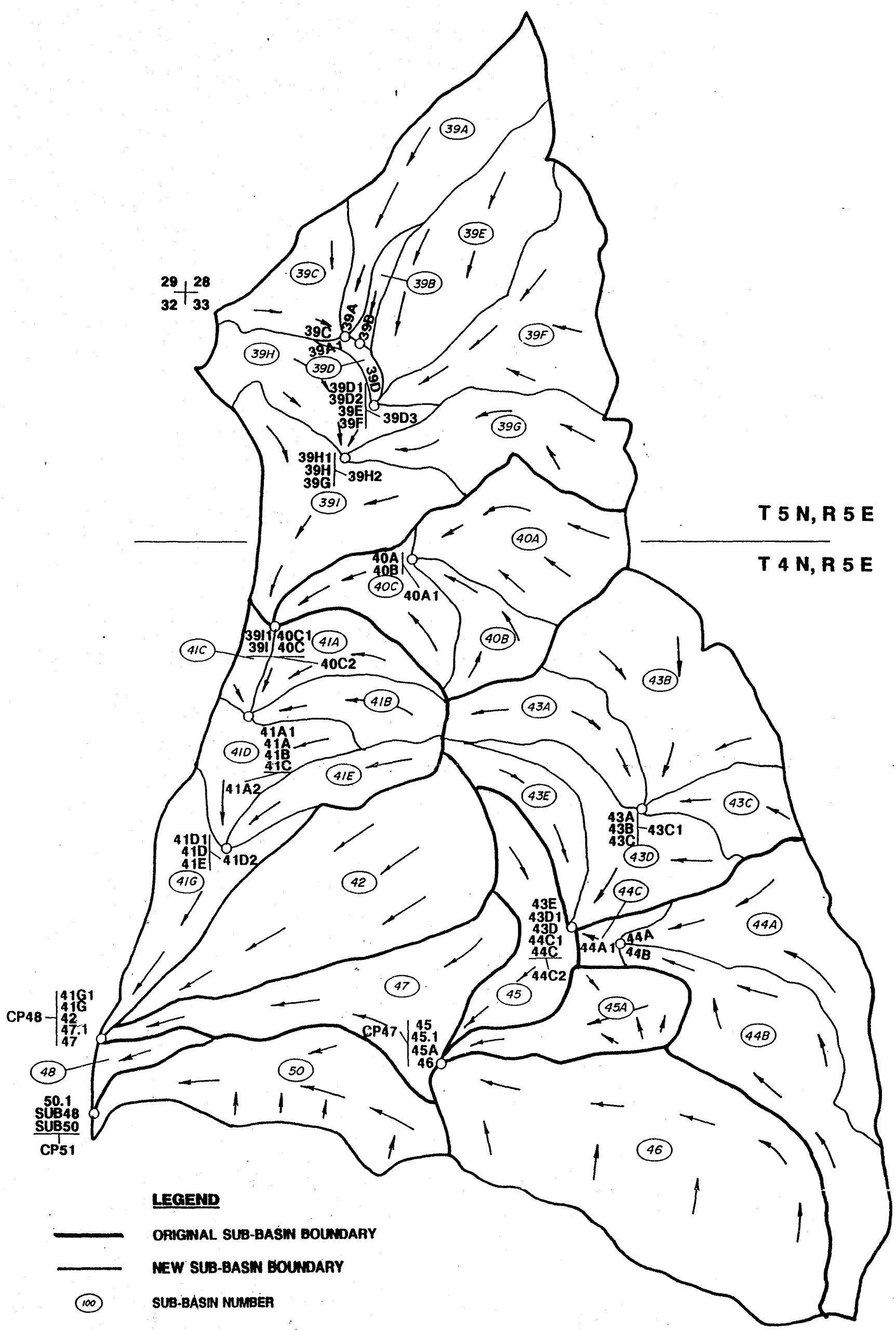
Concentration Point	Drainage Area (sq mi)	Channel Slope (ft/ft)	Channel Length (ft)	Length To Centroid (ft)	Mean Basin Elevation (ft, MSL)	SCS Curve Number (24-hr/1-hr)	Mean Annual Precipitation (in.)	Scenario 15 HEC-1 Model 2P90.6I	AWC Regression	Roesko Region 2	Roesko Region 3	Eychaner/USGS Regression	Malvick Regression	Pima County	TR-55	FCDMC LP3 Curves Maximum	FCDMC LP3 Curves Average	Boughton 100-Year Envelope	Creager Envelope	Costa Envelope	Corps Of Engineers Envelope
CP 41 & 40C2	2.53	0.0249	13,400	5,600	2,640	84.4/90.8	13.5	3,690	4,699	1,748	2,984	2,685	1,676	3,814	2,826	4,137	2,479	4,285	10,173	19,228	7,590
CP 45 & 44C2	1.94	0.0684	10,500	4,200	2,680	85.1/91.3	13.5	3,818	4,093	1,531	2,489	2,631	1,376	4,452	2,752	3,376	2,105	3,588	8,166	15,520	6,305
CP 47	3.25	0.0583	14,000	6,200	2,650	85/91.3	13.5	5,557	5,353	1,981	3,459	3,526	2,010	6,142	4,908	5,021	2,860	5,048	12,451	22,750	9,425
CP 51	7.92	0.0264	24,050	9,200	2,670	83/89.8	13.5	8,919	8,506	3,089	5,898	5,234	3,732	8,534	7,825	10,098	4,831	8,781	24,561	42,768	18,414
SUB 39F	0.34	0.0265	6,800	3,400	2,690	86/92	13.5	629	1,655	642	856	623	341	712	449	-	-	1,011	1,666	3,397	-
SUB 40A	0.29	0.0298	4,700	3,000	2,700	86/92	13.5	745	1,524	593	773	659	297	709	518	-	-	893	1,421	2,784	-
SUB 41E	0.13	0.1113	5,100	3,000	2,600	80.5/87.9	13.5	258	1,004	397	498	175	146	338	193	-	-	466	615	-	-
SUB 42	0.58	0.0573	8,750	5,800	2,440	78/86.1	13.5	878	2,185	838	1,346	824	533	962	728	-	-	1,519	2,790	5,800	-
SUB 43B	0.40	0.0198	4,800	2,400	2,700	86/92	13.5	832	1,801	696	941	805	391	898	590	-	-	1,147	1,954	4,040	-
SUB 44B	0.46	0.0684	7,600	4,400	2,820	83.8/90.4	13.5	862	1,937	747	968	689	440	1,071	631	-	-	1,276	2,238	4,692	-
SUB 46	0.93	0.0535	7,100	5,000	3,100	86/92	13.5	2,547	2,793	1,061	1,315	1,701	780	2,208	1,557	-	-	2,147	4,310	8,649	3,488
Rawhide Wash & FAN 5																					
CP 27	14.02	0.0194	50,600	26,000	2,540	81.9/88.9	13.5	1) 9,710	11,447	4,108	8,917	5,695	5,416	5,421	7,622	12,478	4,458	12,224	36,805	63,090	27,479
CP 1480	2.99	0.0208	40,100	14,000	2,580	83.7/90.3	13.5	2) 4,124	5,126	1,900	3,404	1,923	1,893	2,020	2,004	3,343	2,721	4,782	11,647	21,528	4,485

File: QSUMREAT.WK1

1) Original HEC-1 Model PH30.24I

2) Original HEC-1 Model PH20.24I

PLATES



LEGEND

▬▬▬	ORIGINAL SUB-BASIN BOUNDARY
▬▬▬	NEW SUB-BASIN BOUNDARY
(100)	SUB-BASIN NUMBER
C1000	CONCENTRATION POINT
DIV1000	FLOW DIVERSIONS
DT1000	FLOW DIVERSIONS
DR1000	DIVERSION RETRIEVES
←	DIRECTION OF FLOW

Water Resources Associates, Inc.	REATA PASS WATERSHED BOUNDARIES AND HEC-1 ROUTING SCHEMATIC	PLATE 1
PROJECT NO. AC02670 DRAWN BY K. Andrews		
DATE 11/7/91 CHECKED BY R. Ward		