SURFACE-WATER HYDROLOGY OF THE PROPOSED LOW-LEVEL RADIOACTIVE WASTE ISOLATION SITE, HUDSPETH COUNTY, TEXAS

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Final Contract Report

Prepared for

Texas Low-Level Radioactive Waste Disposal Authority
under Interagency Contract Number IAC(90-91)0268

by

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April 1990

CONTENTS

Figures

Tables

EXECUTIVE SUMMARY

This report describes the surface-water hydrology at the proposed low-level radioactive waste isolation site in Hudspeth County, Texas. The objective of these investigations was to evaluate the flooding potential at the site based on computer simulation of runoff from observed and hypothetical rain events. Analytic techniques and assumptions used in this study are based on recommendations of federal and state regulatory agencies regarding flood insurance and dam safety criteria. Published topographic maps, aerial photographs, and site surveys were used for delineating drainage basins and surface-water pathways on the study area. Surface-water runoff volumes were calculated for rain events monitored at the site during the study period. Hydrologic computer models were employed to determine correlation of rainfall to surface-water runoff. These computer models were calibrated using rainfall and stream-flow data measured at the site. Flood profiles were calculated for 100-yr and probable maximum rain events, which were estimated from historical data. The following conclusions regarding the flooding potential at the study area were drawn on the basis of these studies:

(1) Computer simulation indicates that floods resulting from hypothetical 100-yr and probable maximum precipitation events are contained within existing channels in the study area, leaving large interchannel areas unflooded. Some overland sheet flow is encountered over the flat area, but the velocities of flow are very small.

(2) Rainfall events recorded during the 1988–1989 period were short and localized. The response of runoff to rainfall is rapid and the duration of the peak water flow after rainfall is relatively short.

(3) Flow velocities range from 3 to 13 ft/sec (0.9 to 4 m/sec) in channels and are lower over flat areas. Maximum depth of flow due to a 100-yr flood in the better defined channel on the central part of the study area is about 5 ft (1.5 m).

INTRODUCTION

The purpose of surface-water hydrology studies at the proposed low-level radioactive waste isolation site in Hudspeth County, Texas, was to define the flooding potential as interpreted from the applicable regulatory requirements. Federal Emergency Management Agency Report 37 (FEMA, 1985), which details guidelines and specifications for flood insurance studies, and other published reports (Texas Department of Water Resources, 1979a, 1979b, 1979c) were used as primary sources to define the scope of field reconnaissance and hydrologic evaluation. These regulatory guidelines require evaluation of flooding potential from surface-water runoff caused by hypothetical 100-yr and probable maximum precipitation events.

The 100-yr flood elevation is defined as a flood elevation that has a 1-percent chance of being equaled or exceeded in any given year. Thus, 100 yr is the probable return frequency. Probable maximum precipitation (PMP) is defined as the analytically estimated greatest depth of precipitation for a given duration that is physically possible and reasonably characteristic within a particular geographical region at a certain time of year (Chow and others, 1988). The PMP concept is somewhat vague because it cannot be perfectly estimated and its probability of occurrence is unknown. Flood size estimated from PMP is used mainly to develop engineering design criteria.

The 100-yr floodplain map published by FEMA (1985) for the Hudspeth County area (fig. 1) delineates a flooding potential in only a narrow strip along the main arroyo channels and their upstream drainage areas. The FEMA map, based on a qualitative assessment of possible flow conditions, identifies a floodplain defined by approximate methods (not using computer simulation) and does not show base flood elevations, depths, or velocities. The FEMA floodplain in the southern and southwestern part of the study area (fig. 1) is confined to the Camp Rice Arroyo channel. In the central part of the study area, this floodplain approximately aligns with the channels that drain into the lower fork of the Alamo Arroyo. The FEMA map provided a reference starting point for this study, which reevaluated the flood potential in greater detail.

The scope of this project included:

(1) Delineation of drainage basins, drainage divides, and potential surface-water pathways on and near the study area.

(2) Collection of data on rainfall and surface-water runoff.

(3) Development and evaluation of a hydrologic model to simulate extent of flooding at the site due to actual and hypothetical storms.

(4) Definition of floodplains resulting from hypothetical 100-yr and probable-maximum rain storms.

The empirical approach adopted to meet these objectives consisted of estimating soil properties, monitoring rainfall and surface-water runoff rates, matching simulated flows to observed data on surface-water runoff, and predicting flow characteristics based on calibrated computer models. Flooding potential from 100-yr and probable-maximum floods were simulated using computer models HEC-1 (U.S. Army Corps of Engineers, 1981) and HEC-2 (U.S. Army Corps of Engineers, 1982). These models incorporate the influence of surface topography and channel characteristics and calculate depth, velocity, and profile of surface water flow. These elements were used to delineate floodplains on the study area.

SITE CHARACTERISTICS, DATA, AND METHODOLOGY

Surface Drainage Environment

The Hudspeth County study area is located approximately 41 mi (65 km) southeast of El Paso, on an alluvial plain characterized by gentle slope (1 to 1.5 percent) with dendritic drainage patterns. The study area is within the watershed of the lower fork of the Alamo Arroyo and the upper fork of the Camp Rice Arroyo (fig. 2). The inter-arroyo area is quite flat and contains subtle surface-water divides. Drainage channels within the study area are not well defined (fig. 3), except in the eastern and southeastern parts where channel depths are 1.5 to 2 ft (0.5 to 0.6 m) and widths are 3 to 5 ft (0.9 to 1.5 m). These ephemeral stream channels are generally dry except during rainfall events in the summer. Rainfall elicits a rapid runoff response in the basin under study: runoff closely follows the onset of rainfall, peaks, and then recedes rapidly. There is negligible interception of rainfall by vegetation and little depression storage, and numerous small channels rapidly carry away the runoff. Absence of well-incised channels in the central part of the study area indicates that overland sheet flow contributes significantly to surface-water runoff.

Soil Characteristics

Baumgardner (1989) described eight principal landforms on the surface of the site, characterized by their vegetation cover, shape, local relief, elevation and position relative to other landforms, and grain size of surficial sediments. These landforms are: dune, drainageway, floodplain, interdune, interfluve, colluvium, topographic high, and upland. The surface soils mostly consist of coarse gravel and sands (Baumgardner, 1989). Bed materials in the channels are sand and fine gravel. The area has sparse to moderate vegetation of drought-tolerant grasses, cacti, and spiny shrubs (fig. 4). The creosote plant is present on all landforms and dominates in topographically high areas. Mesquite is most common on dunes but is also found on drainageways and floodplains. Tarbush is also abundant on drainageways and floodplains. Plant distribution in this area is controlled by soil water-holding capacity. Water infiltration properties of the surface soils were assumed to be those of soil group "C" according to the USDA Soil Conservation Service (1975) classification.

Rainfall Data

Rain events in West Texas mostly are localized storms causing high-intensity rainfall in a small area, whereas adjacent subbasins may receive little precipitation. For this project, the Department of Meteorology, Texas A&M University, compiled historical rainfall data from stations located in Hudspeth and El Paso Counties for the 1859–1989 period. In El Paso, annual rainfall varying between 4.3 and 17.3 inches (11 and 44 cm) was recorded during this period, approximately 60 percent of the rainfall occurring between June 1 and September 30. In addition, site-specific rainfall data were gathered at rain gauges installed at four stations in the study area and at one rain gauge on the Diablo Plateau (fig. 2). Precipitation data for a 100-yr return frequency storm were calculated by the Department of Meteorology. The Texas Low-Level Radioactive Waste Disposal Authority and the Department of Meteorology, Texas A&M University, suggested the approximate value to be used for the probable maximum precipitation. Rainfall intensity distribution was estimated by using published techniques (Chow and others, 1988).

The summary of surface flow events in table 1 demonstrates the localized nature of rainfall at the study area, wherein only a few events yielded runoff at all the monitoring stations. In most other instances rainfall was concentrated over such a small area that only one or two stream gauges recorded the flow.

Surface-Water Runoff Data

Drainage basins and surface-water pathways on the study area were delineated on topographic maps using aerial photographs and site surveys. Water levels in ephemeral streams carrying the runoff were recorded at stream gauging stations 1, 2, and 3 (fig. 2). One stillwell also was placed in the swale that carries runoff downstream from the southwest corner of the site. Additionally, 28 crest-stage gauges were installed between channels A and C to measure depth of overland sheet flow (fig. 2). A summary of the depths of water recorded at gauging stations and the stillwell is contained in table 1.

Channel Cross Sections

Channel cross-sectional profiles were surveyed and channel areas were calculated. Three channel profiles were surveyed near gauging station 1, one profile was obtained at gauging station 3, and three channel profiles were surveyed at gauging station 2 (fig. 5). The total cross section AA″ at gauging station 2 included a nearby unpaved road, whereas the partial cross section AA′ did not include the road (fig. 6). The other two cross sections, BB′ and CC′, were 50 and 63 ft (15.2 and 19.2 m) upstream of gauging station 2 (figs. 7 and 8). The location of these cross sections is shown in figure 9.

Floodplain Simulation

The analytic technique used in this study consisted of computer modeling with HEC-1 (U.S. Army Corps of Engineers, 1981) and HEC-2 (U.S. Army Corps of Engineers, 1982) computer programs. The HEC-1 program simulates the precipitation-runoff process and computes discharge hydrographs and peak flow at locations of interest. The fraction of rainfall lost to the soil due to infiltration can also be estimated by HEC-1 simulation. Peak discharge of the flood wave is used by the HEC-2 program to calculate the profile of flood in channels as a function of channel geometry, length, roughness factor, and initial water elevation. HEC-2 is designed for modeling flow in well-defined channels. It makes several simplifying assumptions, including one-dimensional flow, rigid boundary conditions, steady or gradually varied flow, and constant fluid properties. However, HEC-2 is adequately flexible for the case of relatively flat topography such as exists in the study area and provides reliable results when applied with due consideration of geomorphology and ambient hydrologic conditions.

Evaluation of flooding at the study area focused on (1) the drainage basin and subbasins forming the watershed for the Alamo Arroyo and (2) a larger area including the Camp Rice and

Alamo Arroyo drainage basins and the interarroyo plain. Modeling of the precipitation-runoff process was organized along the following steps: (1) measured water levels were used to calculate flow in stream channels resulting from selected rainfall events; (2) these flow data and the measured rainfall data were used to calibrate the HEC-1 computer model and to estimate water loss and runoff-hydrograph parameters; and (3) the estimated hydraulic parameters were incorporated in HEC-2 computer model to calculate surface runoff from hypothetical 100-yr and probable maximum precipitation events. Technical details of these analyses are included in the following sections.

Calculation of Discharges

Surface water runoff at the study area was calculated from stream flow data. Water levels recorded on paper charts at stream gauging stations 1, 2, and 3 (fig. 2) were digitized. Then, water levels at 10-min intervals during stream flows were interpolated. These water depths in stream channels along with channel cross-section data were incorporated in a BASIC computer program for calculating discharges at specific locations along the channels. Manning's correlation for steady-state flow (Chow and others, 1988) was used in the BASIC program for calculating discharge:

$$Q = (1/n)\, A\, R^{2/3}\, S^{1/2} \qquad (1)$$

where,

Q = discharge [m³/sec], n = Manning's roughness coefficient [dimensionless],

A = channel cross-sectional area [m²], P = wetted perimeter [m] and is obtained from the channel cross section, $R = A/P$ = hydraulic radius [m],

and S = friction slope [m/m].

The water-level data recorded at the stillwell and at the crest-stage gauges were not used directly in the surface-water runoff calculation. However, these data provided a verification of the range of values calculated from the stream gauge data. Five surface-water runoff events that resulted from rainfall on 7-29-88, 8-2-88, 8-9-88, 8-21-88, and 9-2-88 were analyzed for gauging station 2. Figure 10 compares observed discharge hydrographs of the 7-29-88 event at gauging stations 1 and 2. Precipitation data from rain gauge 1, which is nearest to gauging station 1, and data from rain gauge 3, which is nearest to gauging station 2 (fig. 2), are also included in figure 10. The calculated discharges for cross sections AA″ and AA′ (with and without the inclusion of nearby road section) at gauging station 2 are shown in figures 11 through 15.

Gauging station 1 lies at the confluence of flow from basins II and III (fig. 2). Discharge at gauging station 1 was expected to be larger than that at gauging station 2, where runoff from only drainage basin II was measured, because all runoff from drainage basins II and III passes gauging station 1. However, the peak discharge of 4.8 ft^3/sec (0.14 m^3/sec) observed at gauging station 1 is smaller than either the observed peak discharge of 137.1 ft^3/sec (3.88 m^3/sec) for cross section AA″ (including the road) or the partial cross-section AA′ discharge of 33.8 ft^3/sec (0.96 m^3/sec) at gauging station 2 (fig. 10). The access road passing along the south side of drainage basin III appears to channel and divert water from the main stream at gauging station 1; therefore, data from gauging station 1 might not accurately reflect runoff processes within the study area.

There were no major rainfall or runoff events between September 15, 1988, and December 30, 1989, the period covered by the data record at gauging station 3.

Simulation with HEC-1

The HEC-1 computer program (Flood Hydrograph Package) was used to simulate surface-water runoff at gauging station 2 for the five recorded events. Weighted averages of rainfall data for the five rain gauges (fig. 2) were used. Data from rainfall gauging station 4 on the Diablo Plateau were not available for the events of 7-29-88, 8-2-88, and 8-9-88. Rainfall data for the five rainfall events are summarized in tables 2 through 6.

Part of precipitation infiltrates into the ground and reduces the amount of surface runoff. Evapotranspiration and capture of rainfall due to vegetation and ponding of water due to local depressions in the ground surface also contribute to loss of precipitation. These losses are calculated in HEC-1 simulations on the basis of specified values of CN and STRTL, where CN is the curve number related to the type of soil group and STRTL is the initial loss before ponding. The Soil Conservation Service (1975) has related infiltration loss characteristics of soil groups to curve numbers on the basis of empirical data. Values of CN and STRTL are dependent on antecedent moisture conditions (the moisture content of the soil prior to the rain event). The loss of rainfall due to infiltration is inversely related to the soil moisture content. High infiltration leaves smaller excess volume of water for runoff. The value of CN also incorporates the amount of vegetation cover and nature of land use (urban, range, or cultivated land).

Scanlon and others (1990) estimated recharge rates and moisture profiles in the unsaturated zone at the study area. However, their technique did not correlate moisture conditions in the top 1.6 ft (0.5 m) of the soil to instantaneous infiltration rates immediately following a rain event. Thus, no direct measure of infiltration losses was available for the study area.

The Clark Unit Hydrograph method was used to transform excess rainfall (precipitation minus infiltration) to basin outflow. Parameters T_c (time of concentration) and R_s (storage coefficient) were specified for the Clark unit hydrograph. T_c is the time at which the whole watershed begins to contribute to runoff (that is, the time for flow from the farthest point on the watershed to the outlet stream). R_s defines the time of storage in the system. Values of 0.6 hr for T_c and 0.52 hr for R_s were used, based on values in Chow and others (1988) and U.S. Army Corps of Engineers (1981).

Parameter Calibration

In the absence of previous data on curve numbers for the study area, calibration runs were made with the HEC-1 model for matching observed runoff discharge with calculated values based on iterations of curve numbers. Calibration was done by two methods: (1) several estimated values were input for CN, and the hydrograph computed by the program was compared to the observed hydrograph and (2) an option for optimization available in the HEC-1 program was used (without specifying any value for CN), and again the simulated and observed hydrographs were compared. Curve number values that resulted in the best match between observed and simulated hydrographs were then considered for computation of runoff from the hypothetical 100-yr and probable maximum rainfall events.

Figures 11 through 15 compare hydrographs calculated from the observed water depths with those obtained from HEC-1 with various SCS curve numbers. STRTL value was not specified in the input, allowing the HEC-1 program to compute a default value as a function of a given curve number. In figure 11, the simulated hydrograph based on a curve number of 75 has about the same total volume of water as the observed hydrograph for cross section AA″ but a different time and magnitude of peak discharge. The peak discharge simulated with a CN value of 73 better matches the observed peak discharge, although the total simulated volume is less than that observed. Because peak discharge is the critical parameter to match, 73 appears to be the appropriate estimate of curve number for the rainfall event of 7-29-88. Similarly, curve numbers of 90, 90.5, 87, and 87 were estimated to be appropriate for rainfall events of 8-2-88, 8-9-88, 8-21-88, and 9-2-88, respectively (figs. 12 to 15). For the rain event of 8-9-88, a CN value of 90.5 was accepted as a better estimate than the two values (90 and 91) used in the simulation.

The second calibration method to optimize curve numbers yielded CN values ranging from 67.7 to 90.0 and STRTL values from 4.0 to 21.2 mm (table 7) for the 5 rainfall events analyzed. The five optimization runs yielded curve numbers with an average of 82 and standard deviation of 6.5 (table 7). The variability in computed CN values is attributable to the dependence of infiltration on antecedent moisture conditions, which are different prior to each rainfall event. More than 98 percent of rainfall for the five events was lost to infiltration (table 8).

The occurrences of the observed and simulated peak discharges are not synchronous because of the underestimation of channel storage. Imperfect mathematical description of channel geometry also contributes to this lack of synchronization. For the purpose of delineating the floodplain, the volume of peak flow, not its exact time of occurrence, is the critical factor. Therefore, greater emphasis was placed on estimating hydrologic parameters that provided best match of peak surface-water runoff between observed and simulated events.

Modeling of 100-yr and Probable Maximum Floods

In addition to the average CN value of 82, a conservative maximum value of 90 (table 7) was also used in simulations of the 100-yr and probable maximum flood (PMF) events. The HEC-1 program computed default values of initial abstraction (STRTL) from the specified values of the curve number.

Estimated rainfall data for the hypothetical 100-yr and probable maximum floods were obtained from the Department of Meteorology, Texas A&M University. The intensity and temporal distributions of rainfall were calculated following methods used in studies of flooding in El Paso, Texas (Espey Huston & Associates, Inc., 1981; Frederick and others, 1977; Miller and others, 1984). Table 9 shows the 5-min-interval rainfall distributions used in this study for both the 100-yr return frequency and probable maximum precipitation.

The calibrated hydrologic parameters were used in conjunction with the hypothetical precipitation data in HEC-1 models to obtain estimates of peak surface-water runoff. These runoff values were then used in the HEC-2 models to determine flood profiles in the study area. The HEC-1 and HEC-2 programs were run sequentially for each channel configuration described in the following section.

Simulation with HEC-2

Floodplains for the hypothetical rain events in the study area were delineated in three stages: (1) drainage basins II and III (channels B and C in fig. 2), which are in the northern part of the study area and within the Alamo Arroyo watershed, were considered separate units; (2) drainage basins I, II, and III were considered a single unit; and (3) the Alamo Arroyo watershed near the study area comprising the interarroyo area between the lower fork of Alamo Arroyo and upper fork of Camp Rice Arroyo, and the watershed for the Camp Rice Arroyo were considered as two contiguous but separate basins.

HEC-1 simulation of runoff from a 100-yr precipitation event showed that drainage basin II (channel B) would have a peak discharge of 15,000 cfs (425 m^3/sec), and drainage basin III (channel C) a peak discharge of 6,500 cfs (184 m^3/sec). HEC-2 simulation was then performed for channel B using this peak discharge value. Nine channel cross sections were constructed (fig. 16) from a topographic map of the study area. HEC-2 results indicated that the upstream parts of channel B (cross sections 8 and 9) could not convey more than 4,000 cfs (113 m^3/sec). Note that the depth of flow on cross section 8 at the local divide would be less than 0.5 ft (0.15 m). The extra discharge (11,000 cfs) would overflow the surface-water divide between drainage basins II and III and diverge into nearby channel C. A discharge of 17,500 cfs (496 m^3/sec), therefore, was used in the HEC-2 simulation for channel C because of the extra discharge of 11,000 cfs (312 m^3/sec) diverging from channel B. Channel C was found to have adequate capacity to contain all the discharge. Total wetted areas in the channels as well as the floodplain are outlined in figure 16. Flow velocities in the channels ranged from 2.8 ft/sec (0.85 m/sec) to 7.5 ft/sec (2.7 m/sec). Maximum water depth of 3.5 ft (1.1 m) occurred in sections 4 and 5 (fig. 16). No HEC-2 runs were made for drainage basin I owing to lack of calibration data at gauging station 3.

The second HEC-2 simulation stage was with drainage basins I, II, and III (including north part of study area) taken as a single basin (fig. 17). HEC-1 simulation of runoff from the 100-yr precipitation event showed that this combined area would have a peak discharge of 30,500 cfs (864 m^3/sec). Water flow focused on the southern part of this area, leaving the northern part outside of the floodplain. The 100-yr floodplain outlined in figure 14 encloses flood elevations of 0.5 ft (0.15

m) and greater. This result is based on the assumption that the local surface-water divides between subbasins I, II, and III do not prevent the surface runoff from coalescing into a single channel.

In the third stage of HEC-2 simulation, runoff analyses were made separately for the watersheds of the lower fork of Alamo Arroyo and the upper fork of Camp Rice Arroyo. These two watersheds were treated as unconnected basins. The tributary option in HEC-2 was used for the Alamo Arroyo watershed. A single well-defined channel was identified for the Camp Rice Arroyo watershed. Figure 18 shows the channel configuration and cross-section lines across the channels incorporated in the HEC-2 model. Peak flows were calculated with HEC-1 program for the area downstream of each cross-section line. Figure 19 represents the simulated 100-yr floodplain for the Alamo Arroyo and Camp Rice Arroyo watersheds. Figures 20 and 21 show the channel configurations and simulated floodplain in the Alamo Arroyo and Camp Rice Arroyo watersheds for probable maximum precipitation (PMP) case.

The number of channel cross sections used in HEC-2 simulation was dictated by the ability of the HEC-2 model to solve the internal continuity and transport equations, given the range of topographic relief. Too few cross sections located far apart violate the boundary conditions imposed on the model, leading the solution of the flow equations to oscillate between subcritical and supercritical flow regime; the correct model solution requires the flow regime to be either subcritical or supercritical along the whole flowpath. The total potential and kinetic energy of flow in converging tributaries was matched by adjusting cross-section orientations. Numerous simulations with different basin configurations were performed to determine the sensitivity of the model and to produce the best estimates of 100-yr and PMP floodplains.

The tributary-channel configurations used to simulate the 100-yr and the PMP floods (figs. 18 and 20) differed to reflect the capacity of channels within subbasins I and II to contain the simulated flow within their boundaries and to maintain spatial continuity of flow in the subbasins in HEC-2 simulations. If the 100-yr flood was modeled with an abbreviated subbasin I as in the PMF simulation (fig. 20), flow would be concentrated in the main channel downstream of section 6 and flow in subbasins I would be depleted. If the PMF in subbasins I and II was modeled with the channel configuration used to simulate the 100-yr flood (fig. 18) (that is, channels converging downstream at section 2), then simulated flow in subbasin I downstream of section 17 would spill southward across the local drainage divide into subbasin II.

Table 10 contains the peak discharge, flow velocity, and floodplain cross-widths across the various section lines from the HEC-1 and HEC-2 simulations. Table 11 contains similar data for the simulation of floodplain for probable maximum precipitation. Flow velocities for the 100-yr flood ranged between 3 and 13 ft/sec (0.91 and 3.96 m/sec), and for the probable maximum flood they ranged between 3 and 17 ft/sec (0.91 and 5.18 m/sec).

A final set of HEC-1 and HEC-2 simulations was performed for the Alamo Arroyo and Camp Rice Arroyo watersheds, with peak surface-water discharges computed using a curve number of 90. Compared to the average curve number of 82 used in previously mentioned simulations, this higher value was considered in order to obtain a greater peak discharge and a delineation of the most extensive potential floodplains. The simulation summary in table 10 shows that although the higher curve number results in a nearly 26 percent greater peak discharge at several cross sections, the incremental increase in floodplain widths and channel flow velocities is less than 10 percent.

The distribution of hypothetical rainfall and resulting discharge hydrographs for 100-yr and probable maximum precipitation events calculated for the northern part of the study area are shown in figures 22 and 23, respectively. Runoff rapidly follows onset of precipitation, and the flood wave (defined as 50 percent of peak flow) duration is from 1.5 to 2 hr.

DISCUSSION

Surface-water runoff simulated with HEC-1 and HEC-2 models tends to concentrate in the relatively better defined channel in the southeastern part of the Alamo Arroyo watershed (figs. 18 and 20). The absence of well-incised channels results in shallow overland sheet-flow in the central and northern part of the study area. Owing to the flat topography at the site, the selection of drainage boundaries between subbasins influences the predicted flood profiles in the channels (compare figs. 16 and 17). Introduction of too many drainage divides forces surface-water runoff into narrow flowpaths and creates wetted areas where they might not actually occur.

The best representation of the floodplain and channel configuration has the Camp Rice Arroyo watershed draining into a single, well-defined channel and the Alamo Arroyo watershed draining through tributary channels into the lower fork of Alamo Arroyo (figs. 18 and 20). This model minimizes the sensitivity of flood profiles to too many boundaries of small drainage basins. During simulation with various model configurations, it was observed that due to the flat topography of the interarroyo plain, the north-central and southwestern sections of the study area in the Alamo Arroyo watershed experience sheet flow. Where flood elevations exceed a few inches, however, the bulk of the water is transferred across the discontinuous, subtle surface-water divides to the better defined channel draining into the lower fork of Alamo Arroyo. Runoff in the watershed of Camp Rice Arroyo is totally contained in the arroyo and does not overflow the drainage divide between the two watersheds. Floodplains simulated for the 100-yr rainfall event using HEC-1 and HEC-2 with average and maximum curve numbers (82 and 90) are not appreciably different.

CONCLUSIONS

The HEC-1 and HEC-2 flooding analyses for the Alamo and Camp Rice Arroyo watersheds show that most of the central and southwestern parts of the study area are not inundated by the 100-yr or probable maximum floods. The calculated runoff is contained in existing channels. There probably would be some very shallow overland sheet flow over much of the remaining area. The surface-water runoff resulting from a probable maximum precipitation covers a broader floodplain but still leaves a major part of the central and southwestern sections of the study area uninundated. The flow velocities range from 3 ft/sec (0.91 m/sec) to 13 ft/sec (3.96 m/sec) in the better defined sections of the channels and are expected to be much lower over the flat area experiencing sheet flow. Moreover, the runoff-to-rainfall response is quite rapid, and the peak flow of the flood wave is carried away in a short period (about 2 hr). Maximum depth of water flow due to 100-yr flood in the channel of the central part of the study area is about 5 ft (1.5 m).

ACKNOWLEDGMENTS

This research was funded by the Texas Low-Level Radioactive Waste Disposal Authority. The conclusions of the authors are not necessarily approved or endorsed by the Authority.

Technical editing was by Tucker F. Hentz, of the Bureau of Economic Geology, and Larry W. Mays, Department of Civil Engineering, Arizona State University, Tempe, Arizona. Amanda R. Masterson edited this publication. Word processing was by Melissa Snell and Susan Lloyd. Figures were drafted by Richard L. Dillon, and by Joel L. Lardon, Yves Oberlin, and Maria Saenz under Mr. Dillon's supervision.

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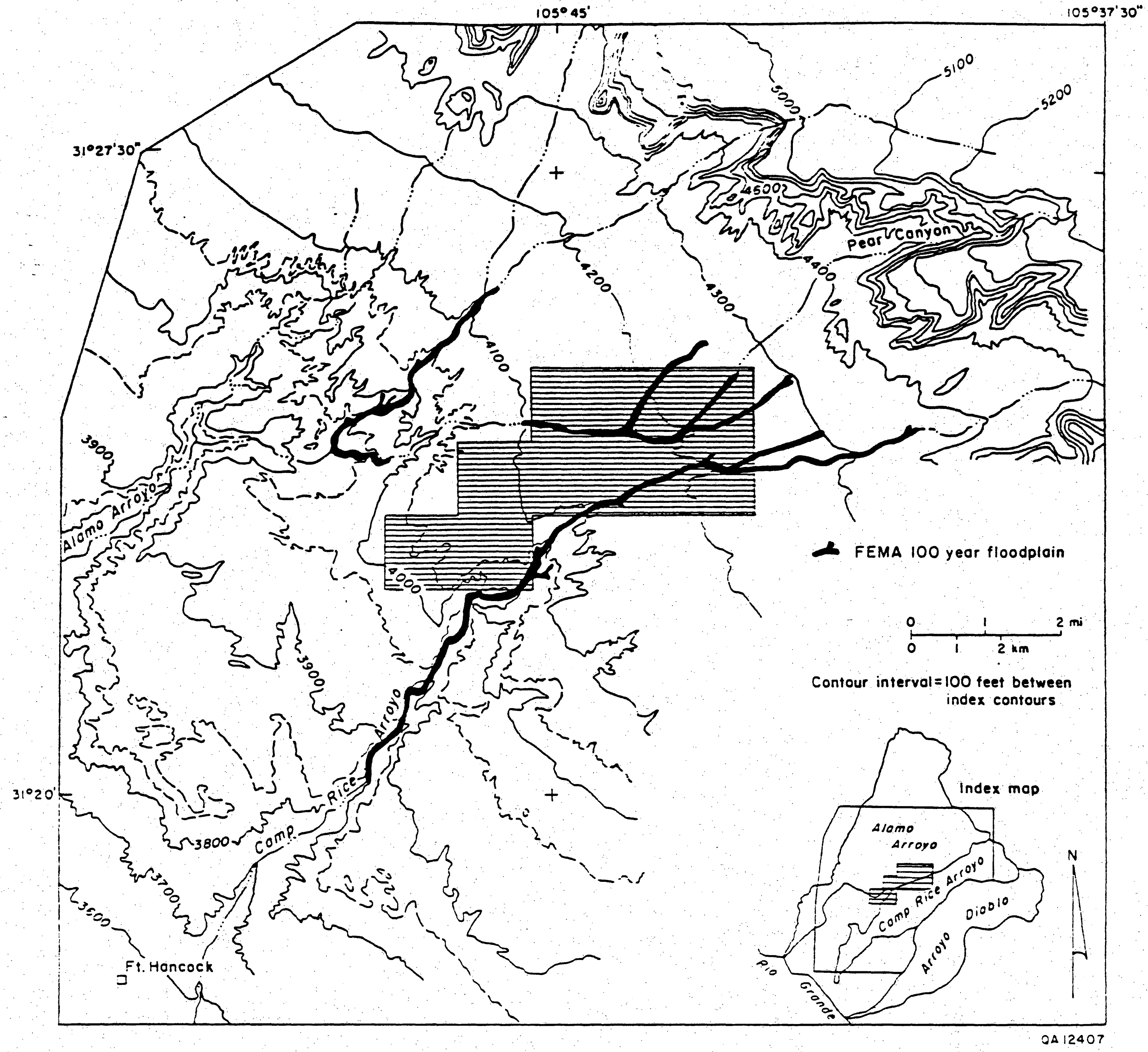


Figure 1. Map showing 100-yr floodplain on proposed Texas low-level radioactive waste isolation site (FEMA, 1985). Delineation of floodplain done by approximate methods. FEMA does not identify the boundary of the 100-yr floodplain in the lower reaches of the Camp Rice and Alamo Arroyos.

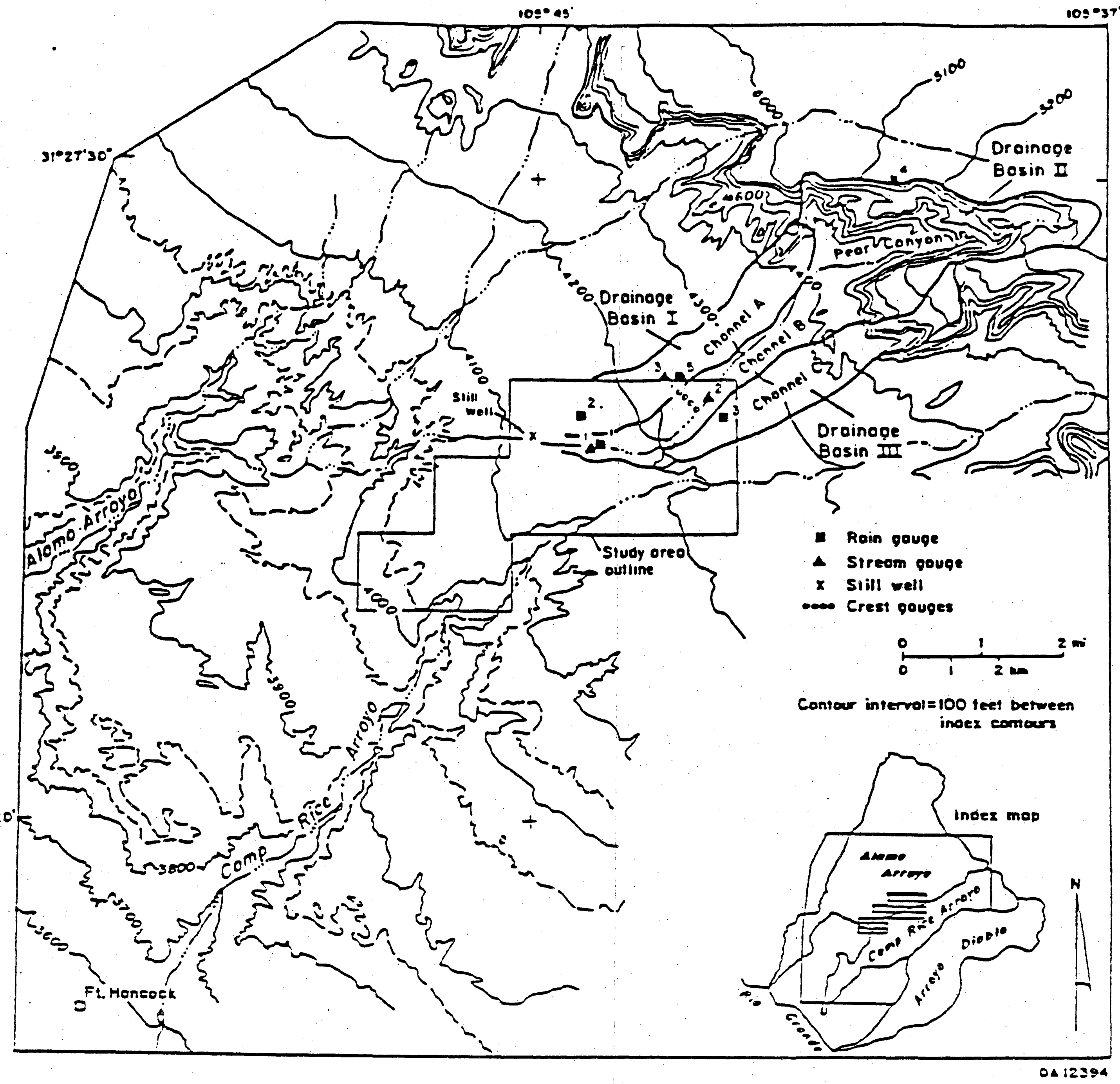


Figure 2. Surface-water hydrology study area and location of rainfall and surface water runoff monitoring devices. The drainage basins were delineated for determining flood profiles in the channels.

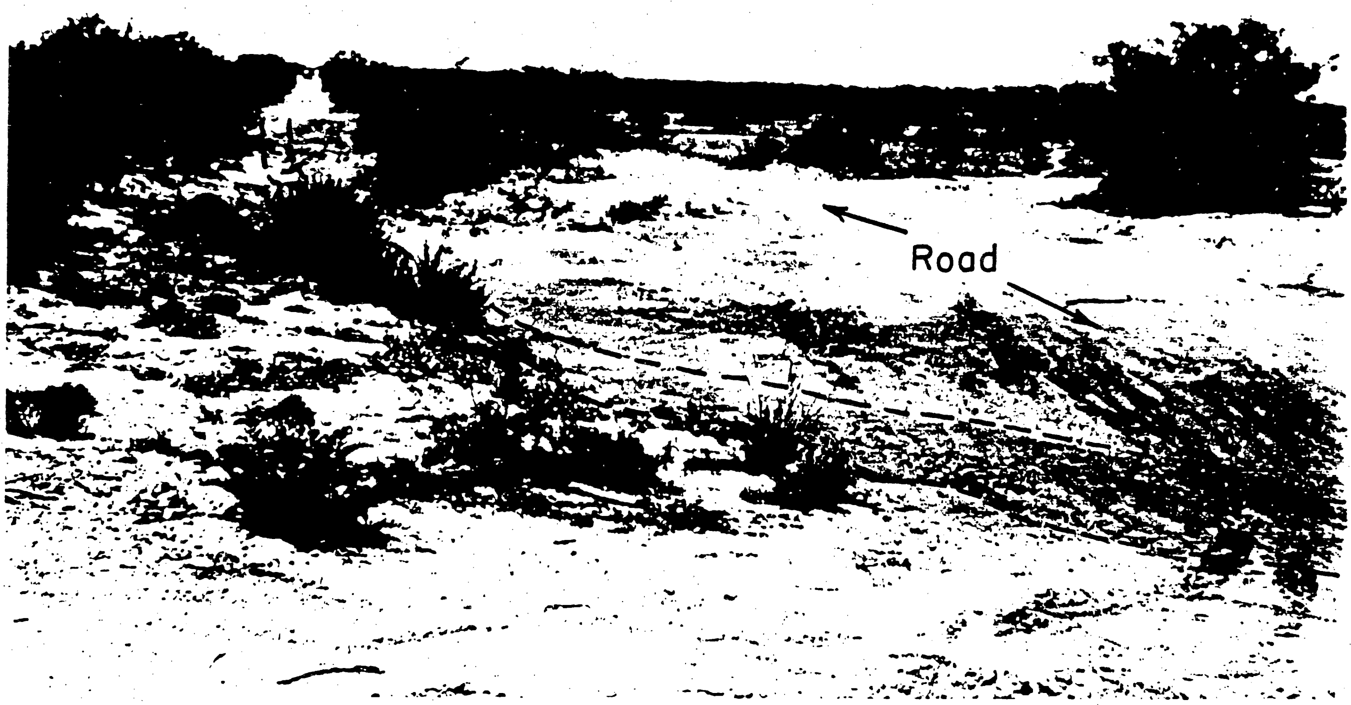


Figure 3. Surface-water drainage environment on the study area. Dashed lines outline the channel boundaries. View is looking south from stream-gauging station 2. Height of mesquite bush on the right is approximately 12 ft (3.7 m).

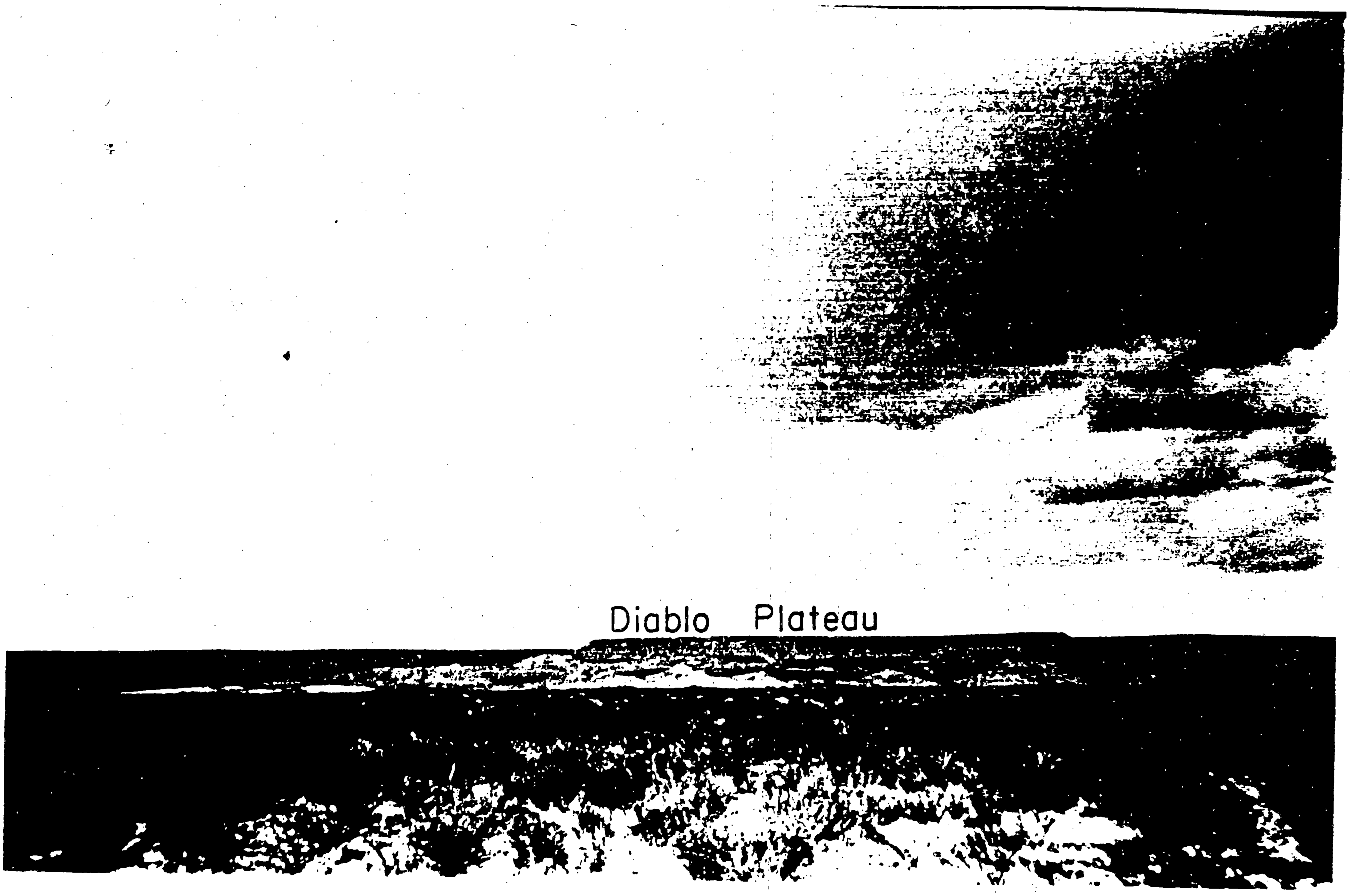


Figure 4. Surface vegetation and landforms on the study area. Relief of the Diablo Plateau Escarpment in the background is approximately 800 ft (244 m). View is toward northeast from study area.

Figure 5. Drainage channel at stream-gauging station 2. Inlet tubes shown by arrow carry the channel discharge into a barrel inside the brick structure, where water level is monitored by a continuous recording device. Note the sparse vegetation dominated by creosote plant in the foreground. View is in the southwest direction. Height of the station house is approximately 7 ft (2.1 m).

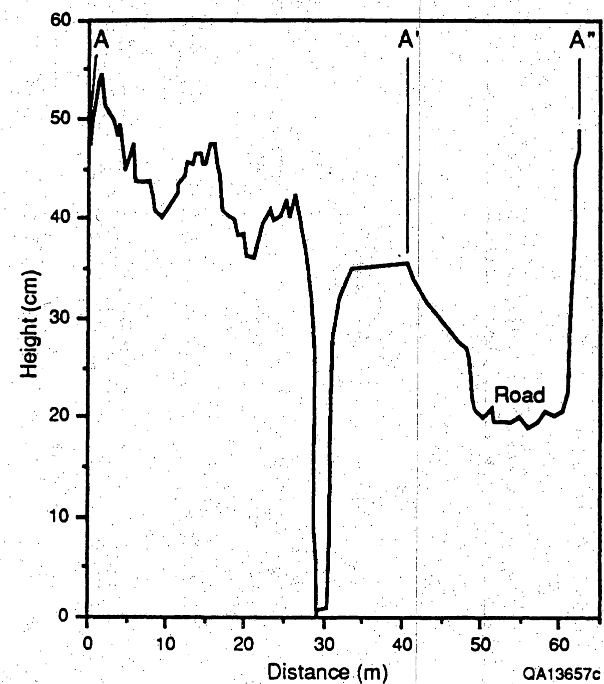


Figure 6. Cross-section profile AA'A" of channel at gauging station 2. Part of the nearby unpaved road is included in the section A'A". Runoff spills over from the stream channel into section A'A" following a major rainfall event. Partial section AA' and total section AA" were both used to determine runoff.

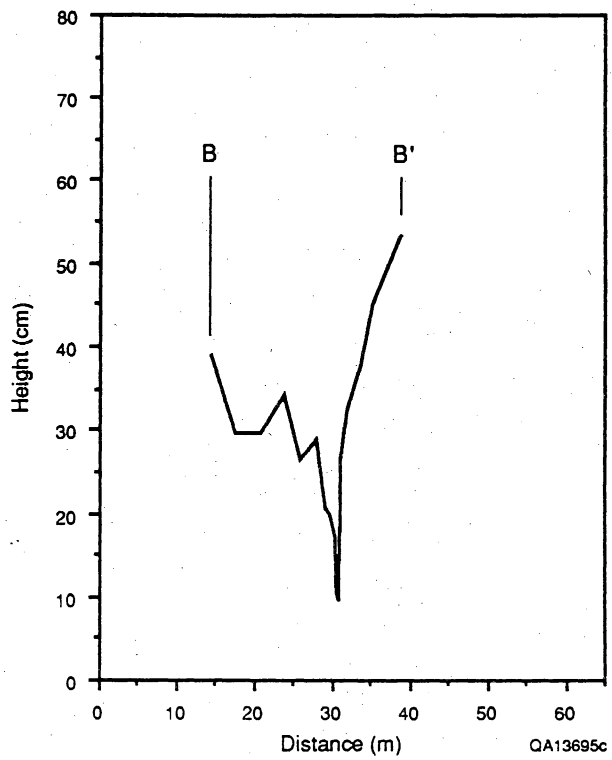


Figure 7. Channel cross-section profile BB' 50 ft (15.2 m)upstream of gauging station 2.

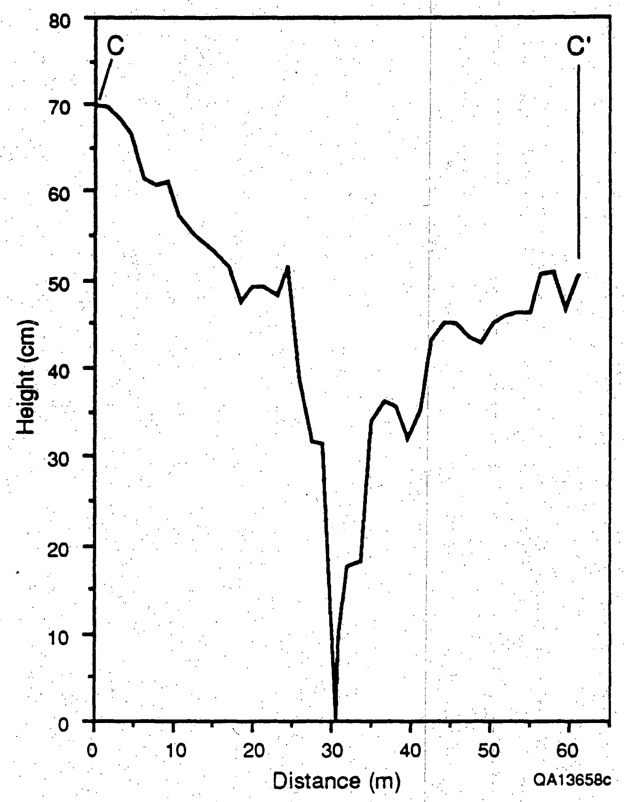


Figure 8. Channel cross-section profile CC' 63 ft (19.2 m) upstream of gauging station 2.

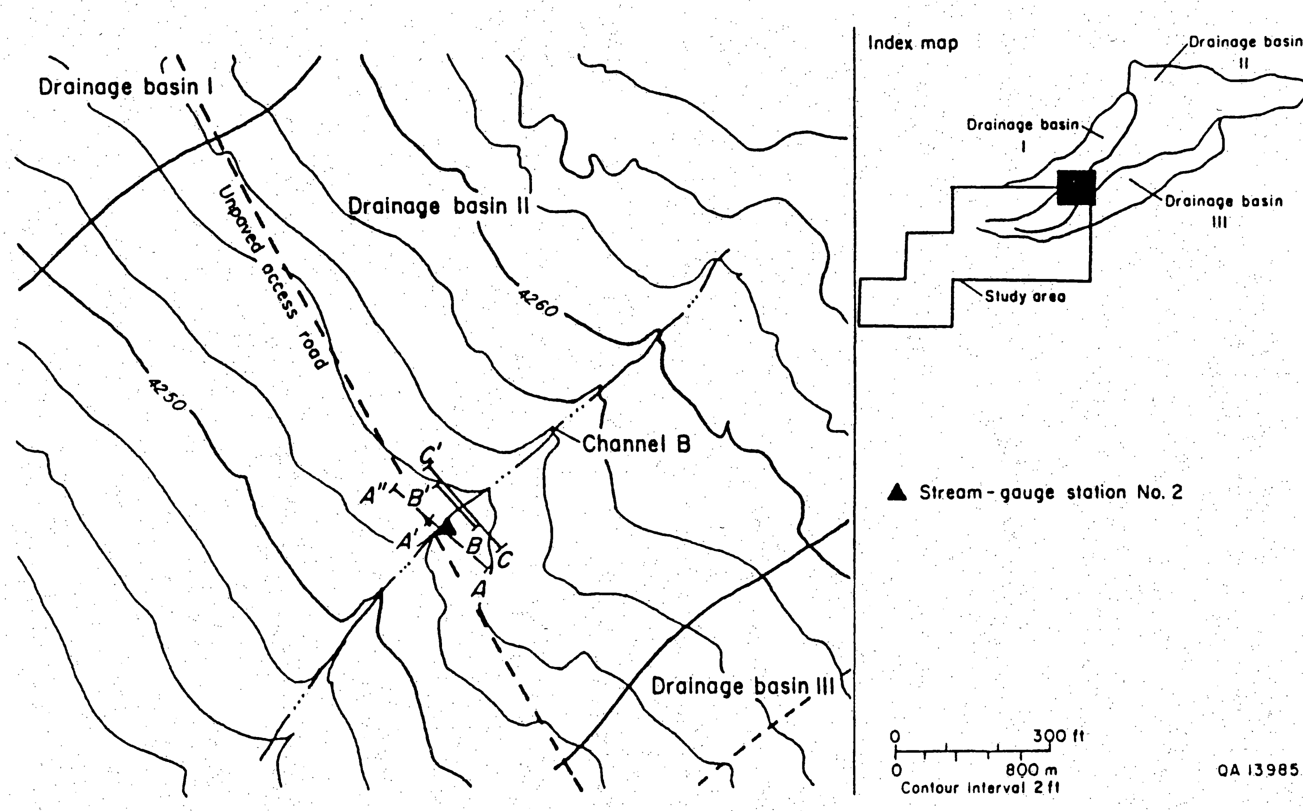


Figure 9. Location of channel cross-section profiles AA'A", BB', and CC' near gauging station 2.

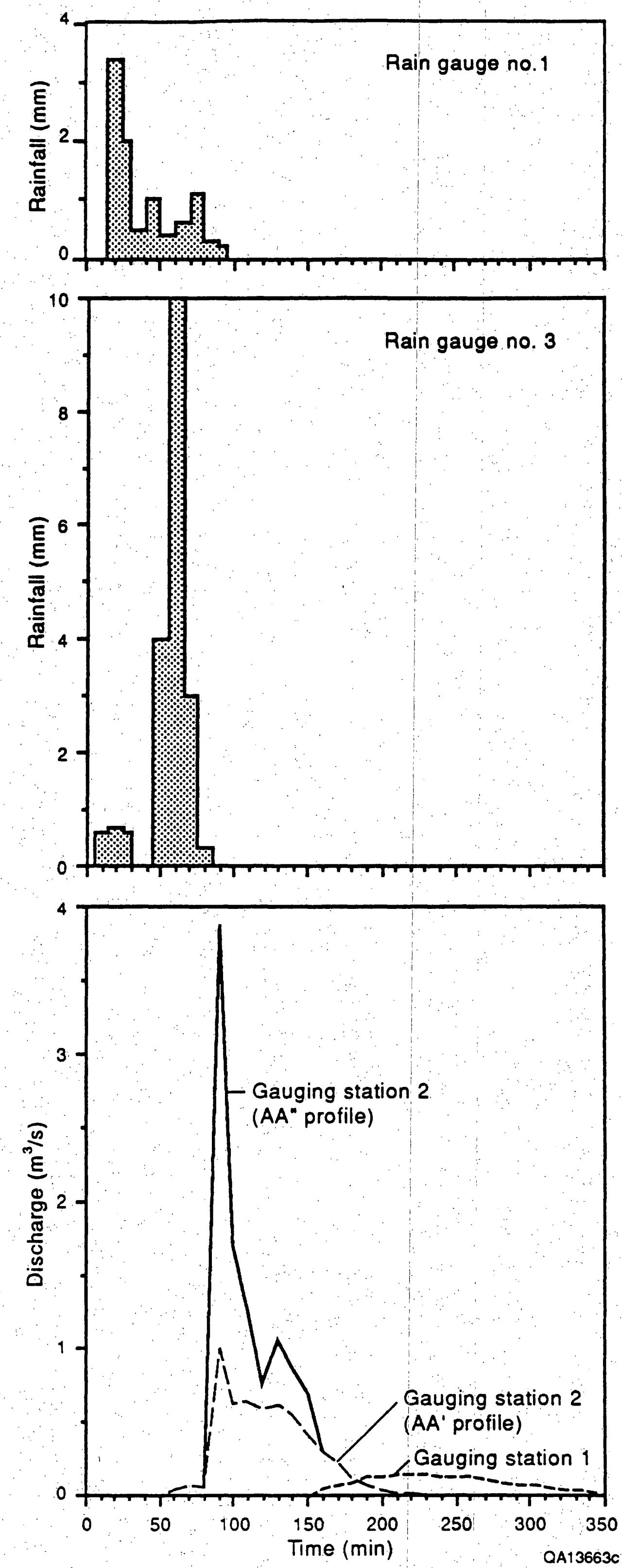


Figure 10. Comparison of observed hydrographs at gauging stations 1 and 2 and corresponding rainfall data for the 7-29-88 event.

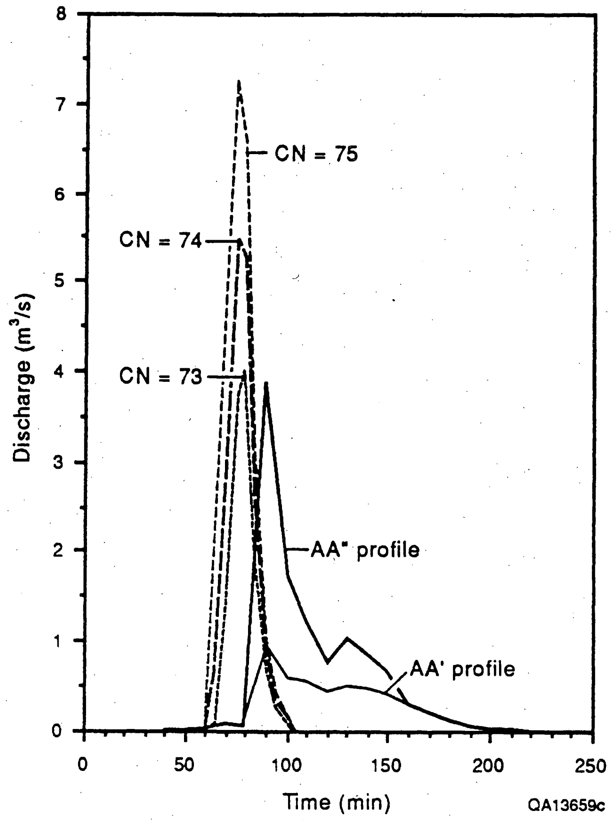


Figure 11. Hydrograph at gauging station 2 on 7-29-88 (whole basin model). AA' and AA" flow profiles are observed discharges. Other curves represent computer-simulated discharge.

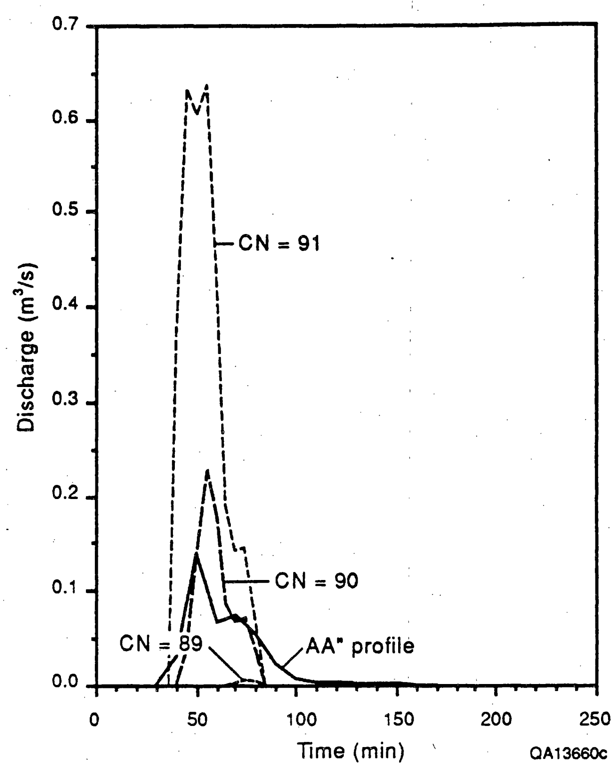


Figure 12. Hydrograph at gauging station 2 on 8-2-88 (whole basin model). AA" is observed discharge, with flow in the unpaved road section. Other curves represent computer-simulated discharge.

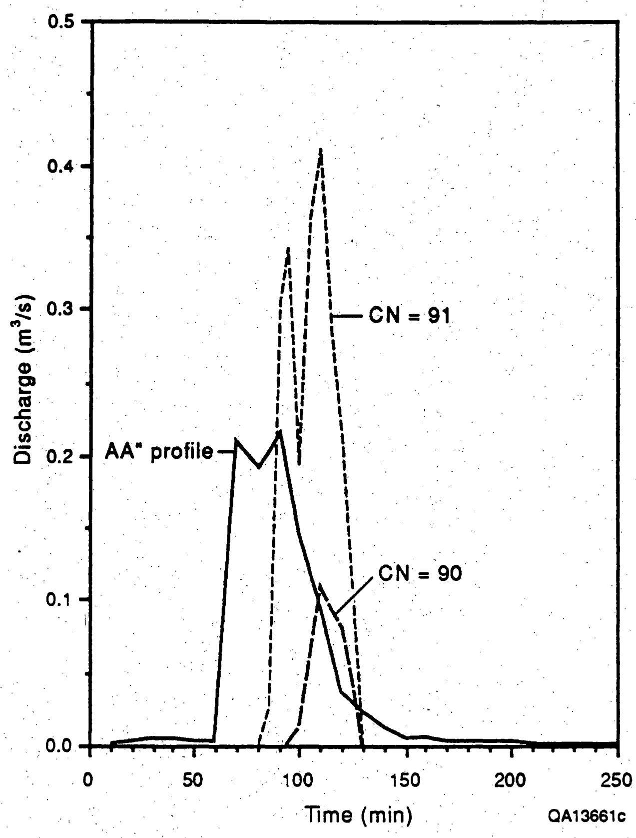


Figure 13. Hydrograph at gauging station 2 on 8-9-88 (whole basin model). AA" is observed discharge, with flow in the unpaved road section. Other curves represent computer-simulated discharge.

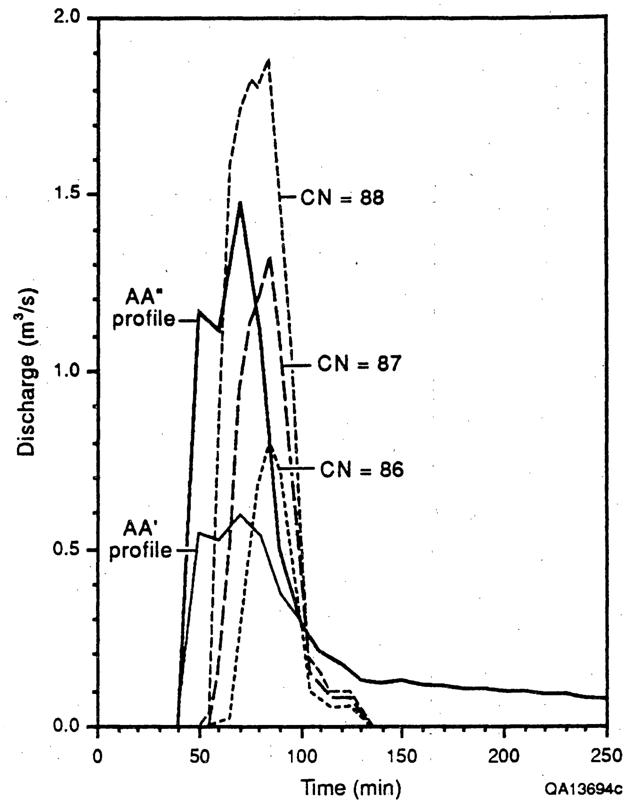


Figure 14. Hydrograph at gauging station 2 on 8-21-88 (whole basin model). AA" and AA' are observed discharges, with and without flow in the unpaved road section, respectively. Other curves represent computer-simulated discharge.

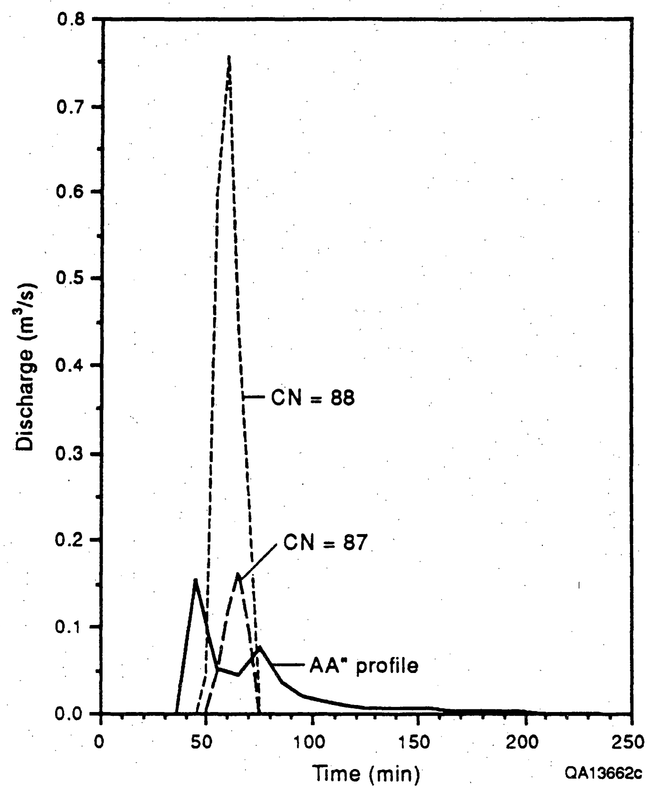


Figure 15. Hydrograph at gauging station 2 on 9-2-88 (whole basin model). AA" is observed discharge, with flow in the unpaved road section. Other curves represent computer-simulated discharge.

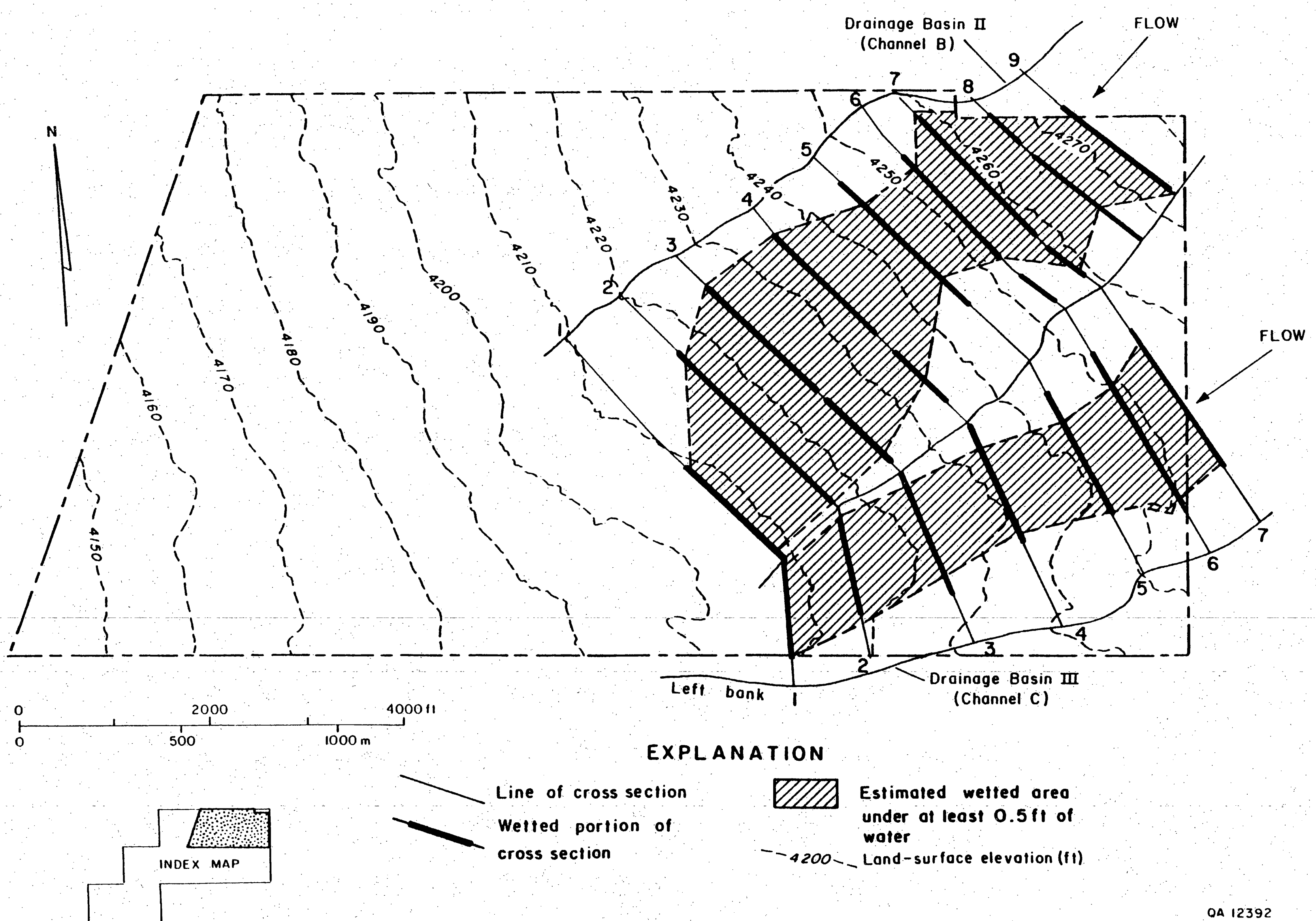


Figure 16. Map showing 100-yr flood profiles in two main channels in northern part of the study area (watershed for lower fork of Alamo Arroyo).

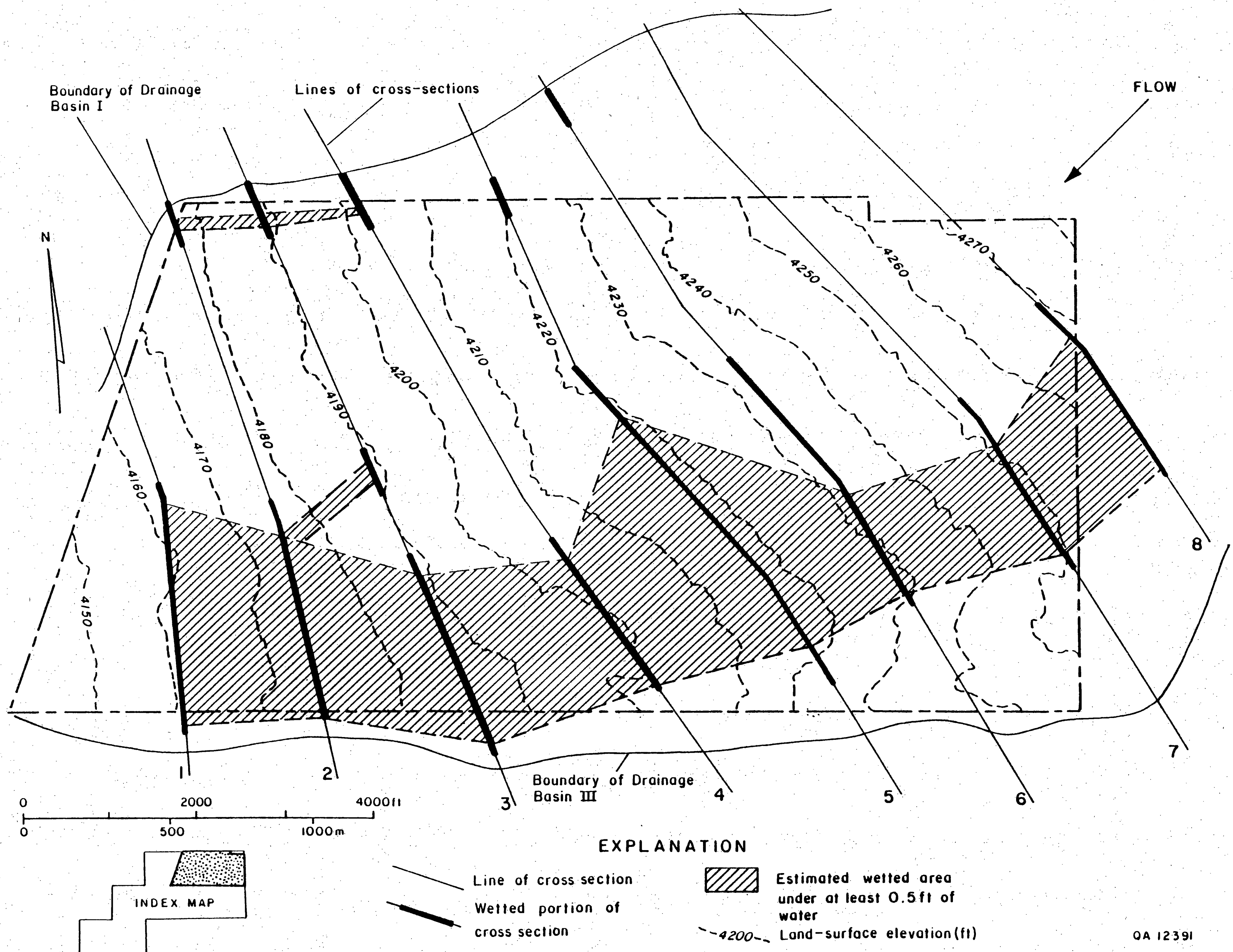


Figure 17. Map showing 100-yr flood profile in north part of study area. Channel C is considered as a single channel for the basin (watershed for lower fork of Alamo Arroyo).

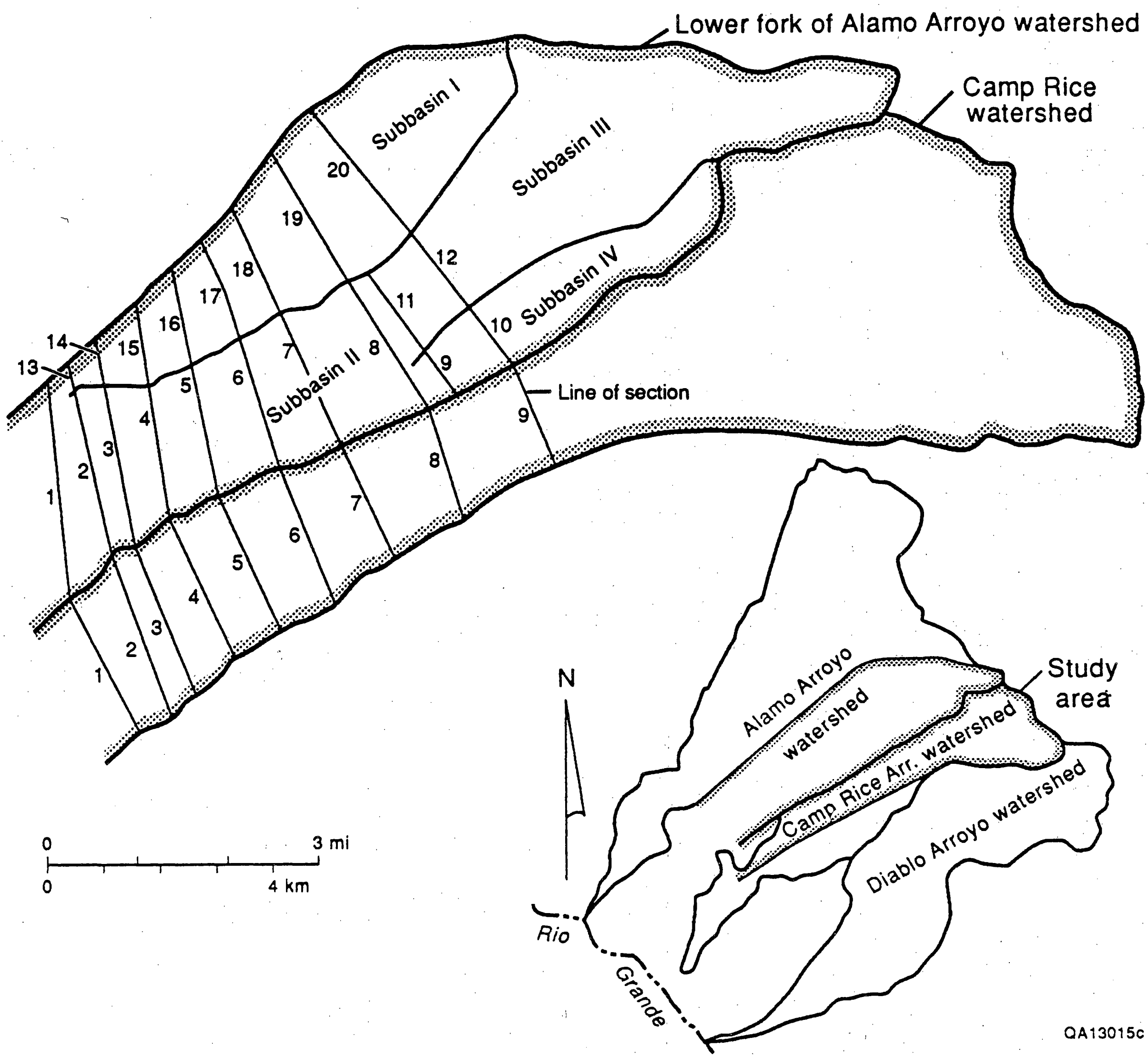


Figure 18. Channel configuration for HEC-2 tributary option simulation of a 100-yr flood between the lower fork of Alamo Arroyo and Camp Rice Arroyo.

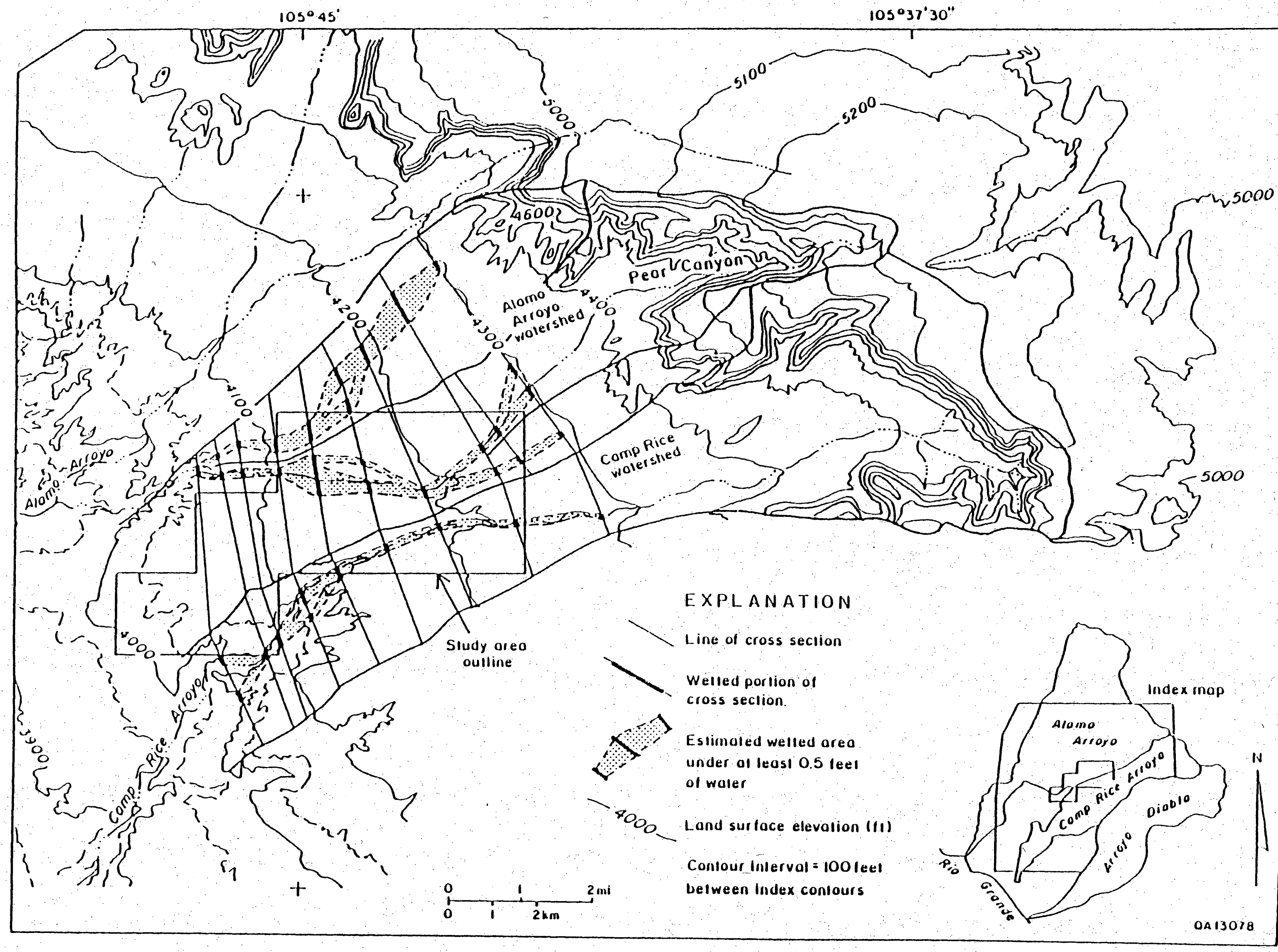


Figure 19. Map showing 100-yr flood profile for HEC-2 tributary option simulation.

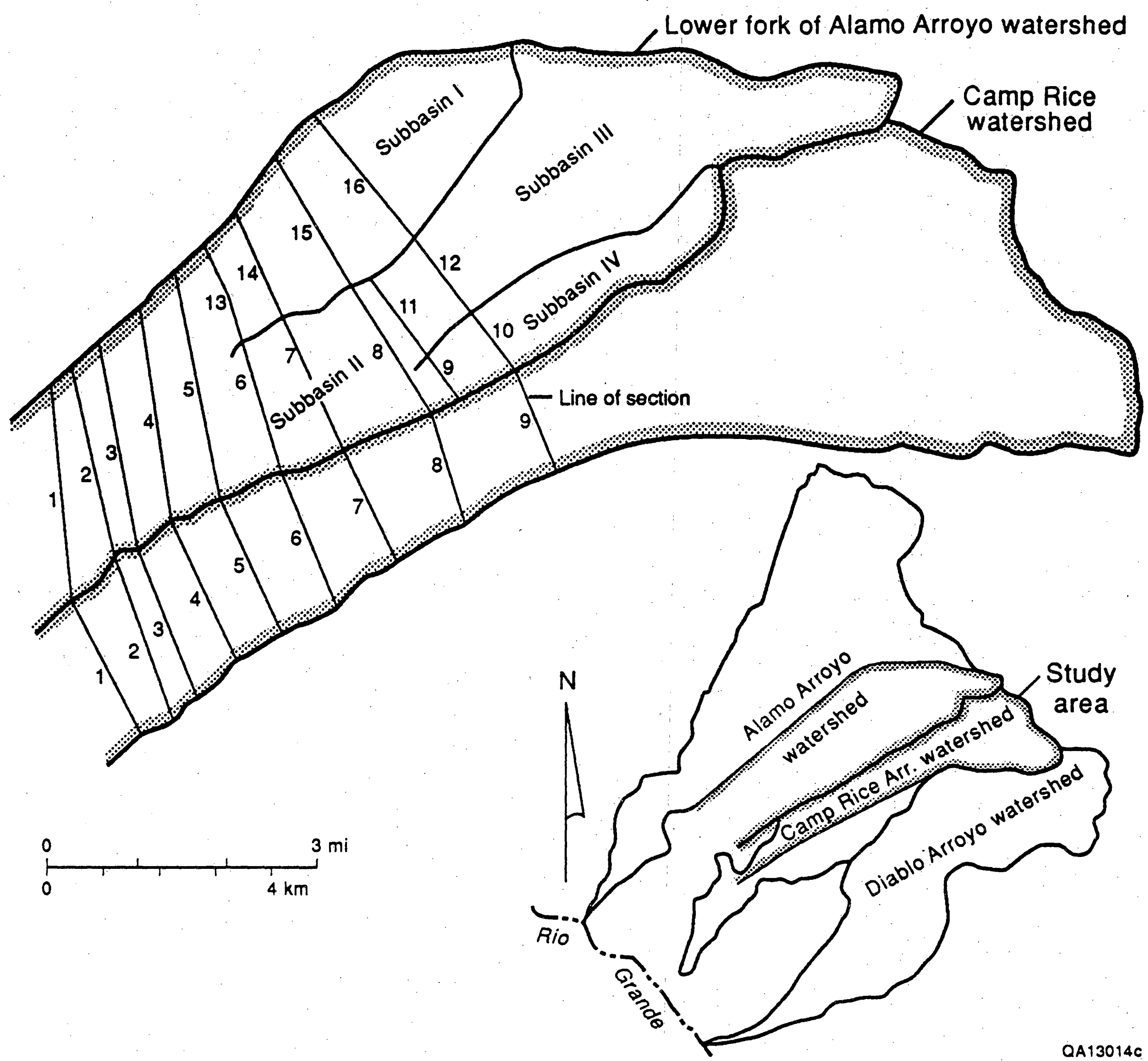


Figure 20. Channel configuration for HEC-2 tributary option simulation of probable maximum flood between the lower fork of Alamo Arroyo and Camp Rice Arroyo.

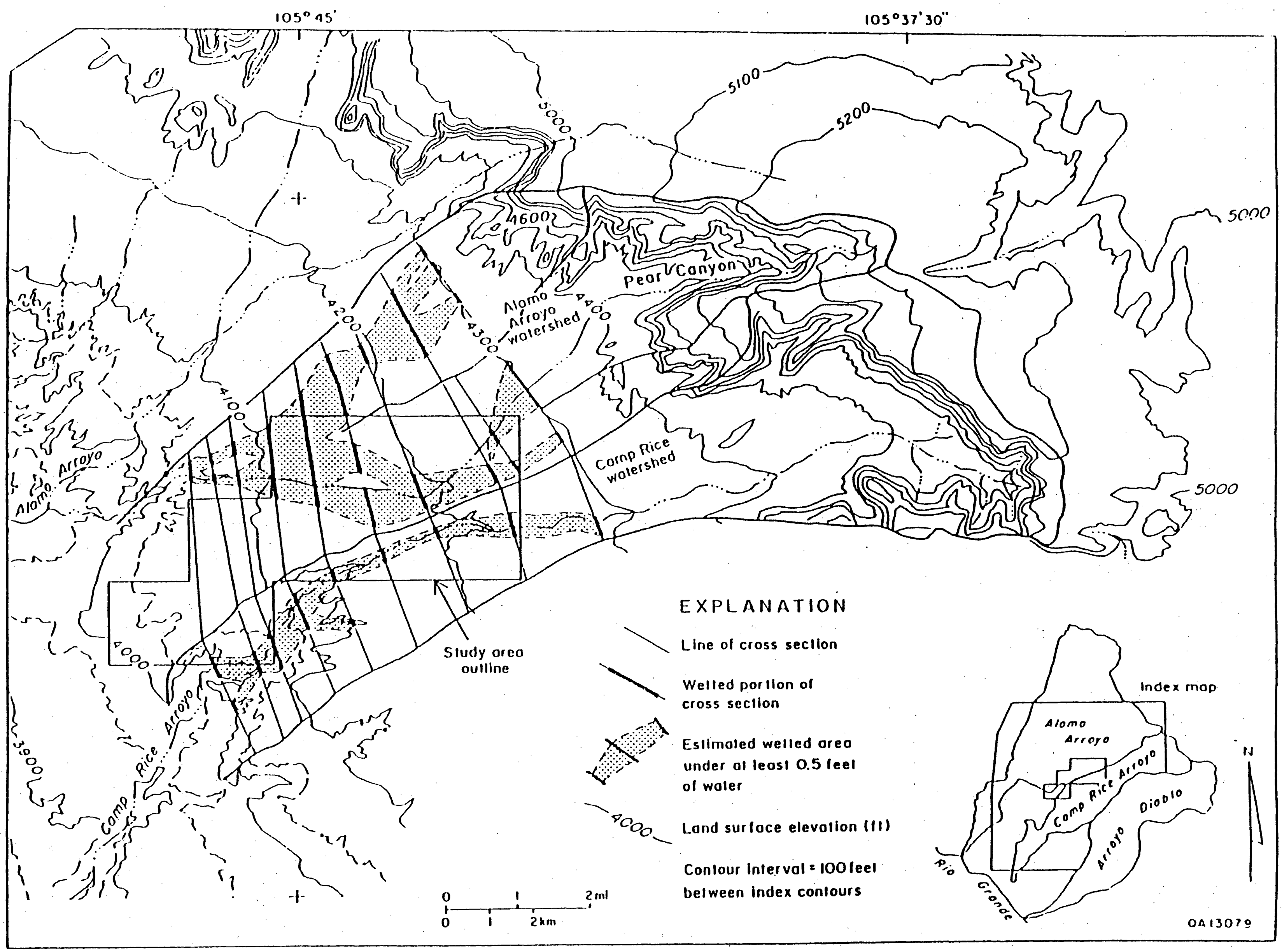
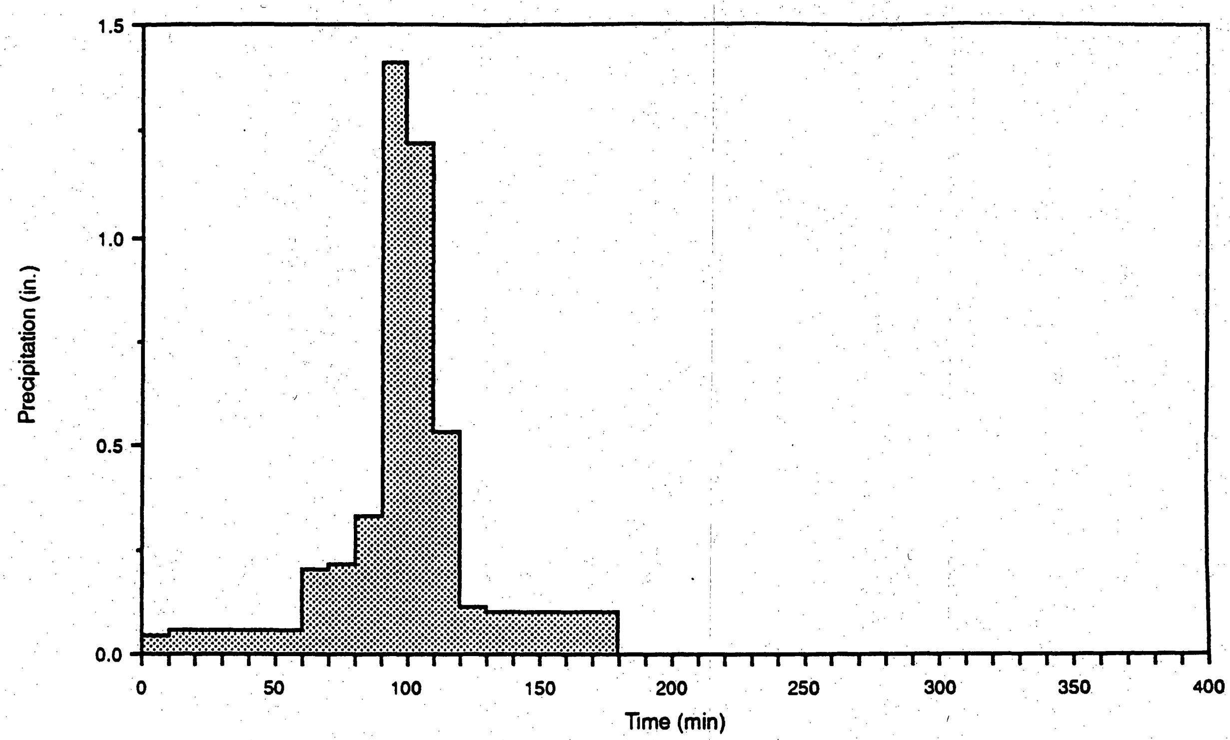


Figure 21. Probable maximum flood profile for HEC-2 tributary option simulation.



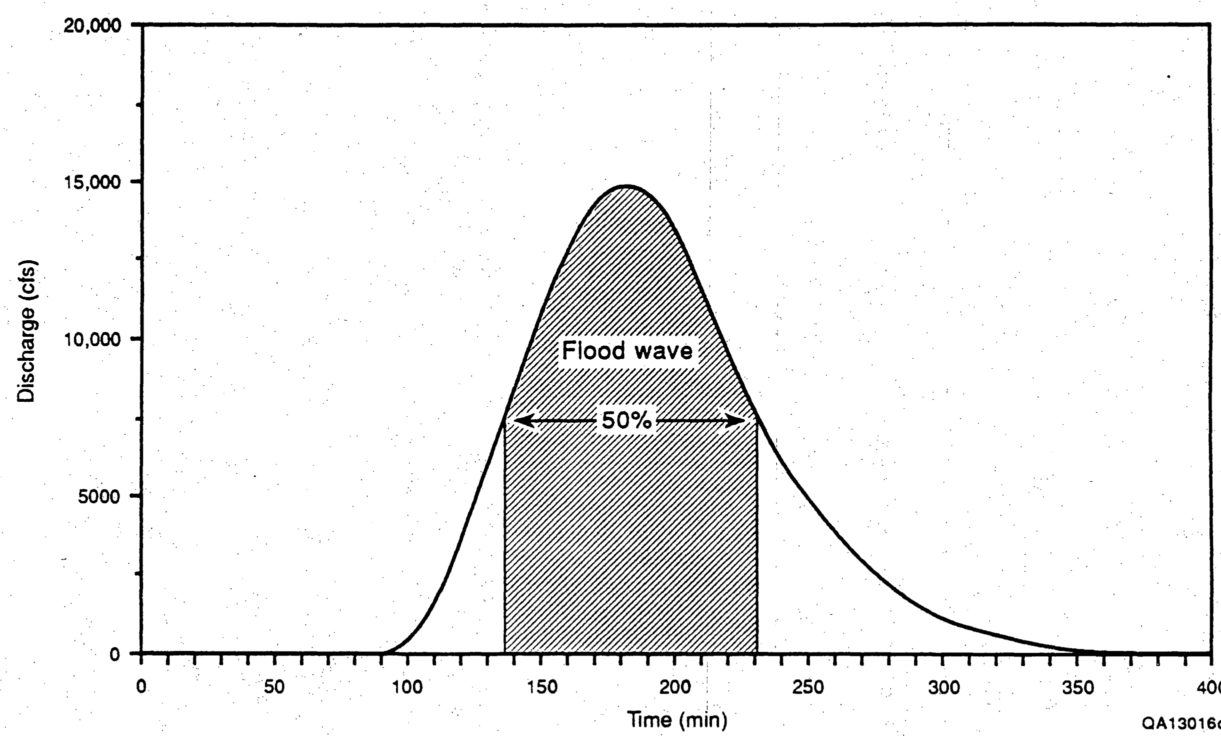


Figure 22. Rainfall distribution and discharge hydrograph for 100-yr precipitation.

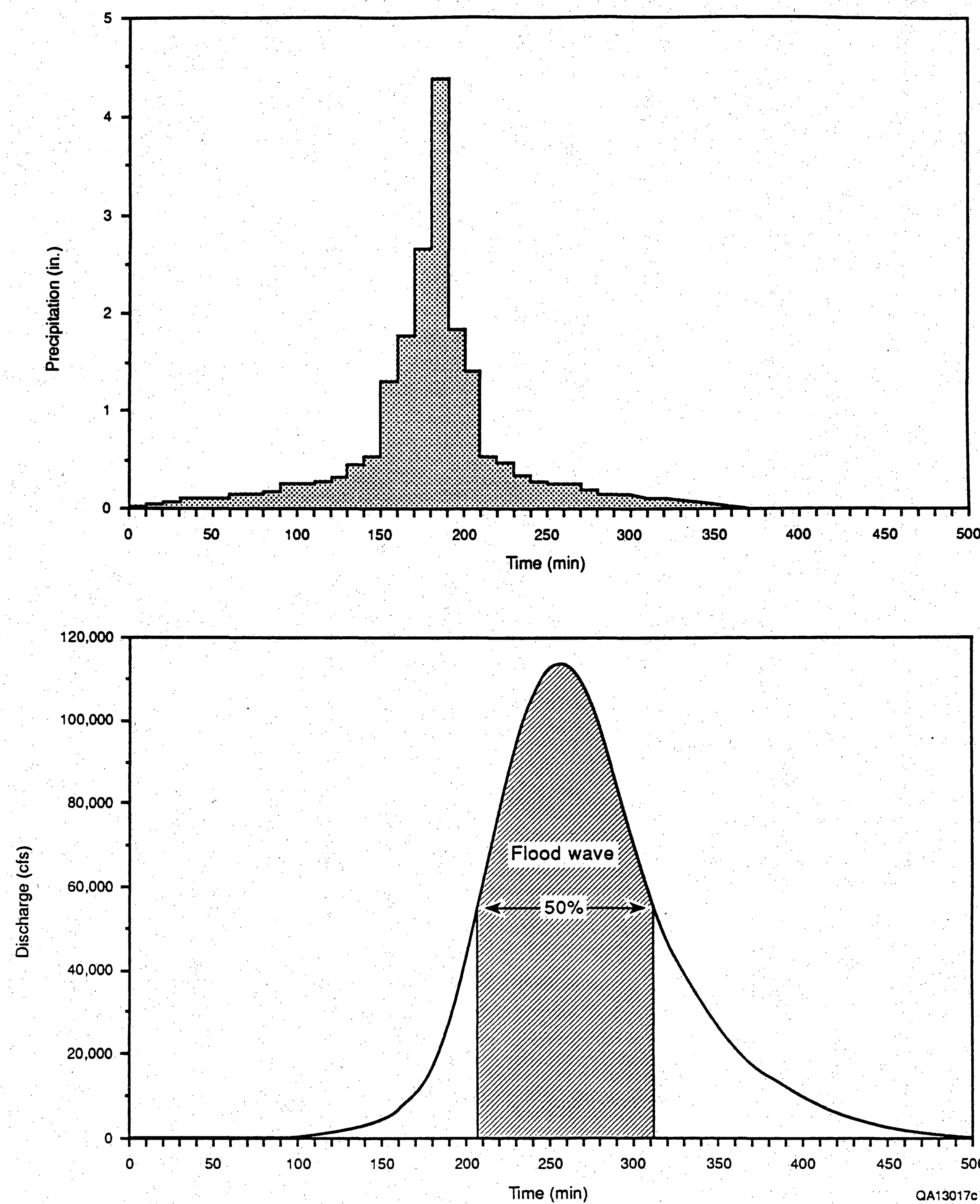


Figure 23. Rainfall distribution and discharge hydrograph for probable maximum precipitation.

Table 1. Summary of surface-water runoff in channels monitored at gauging stations and water level at stillwell.

Date	Gauging Station 1	Gauging Station 2	Gauging Station 3	Stillwell 1
6/27/88-6/28/88	no flow	not available	not available	not available
6/28-7/1	n.f.	n.a.	n.a.	n.a.
7/1-7/5	n.f.	n.a.	n.a.	32 in. @1510 hr, 7/1
7/5-7/8	n.f.	n.a.	n.a.	n.f.
7/8-7/11	18 in. @0620 hr, 7/10	plugged, not screened	n.a.	26.5 in. @0600 hr, 7/10
7/11-7/20	n.f.	not screened	n.a.	n.f.
7/20-7/25	n.f.	n.f.	n.a.	n.f.
7/25-7/28	n.f.	~14 in. @1415 hr, 7/27	n.a.	n.f.
7/28-7/31	~12 in. @1345 hr, 7/29	~17 in. @1340 hr, 7/29	n.a.	15.6 in. @1335 hr, 7/29
7/31-8/3	n.f.	~6 in. @1920 hr, 8/2	n.a.	n.f.
8/3-8/6	n.f.	~3.5 in. @1220 hr, 8/5	n.a.	n.f.
8/6-8/9	~3 in. @1920 hr, 8/7	~3.3 in. @1940 hr, 8/7	n.a.	4.2 in. @1905 hr, 8/7
8/9-8/12	n.f.	~7 in. @1240 hr, 8/9	n.a.	0.85 in. @1335 hr, 8/9
8/12-8/15	n.f.	n.f.	n.a.	n.f.
8/15-8/18	~1.5 in. @2200 hr, 8/15	n.f.	n.a.	3.3 in. @2145 hr, 8/15
8/18-8/21	n.f.	n.f.	n.a.	n.f.
8/21-8/24	n.f.	~14 in. @1430 hr, 8/21	n.a.	n.f.
8/24-8/27	n.f.	~7 in. @1825 hr, 8/26	n.a.	n.f.
8/27-8/30	n.f.	~2.5 in. @0605 hr, 8/30	n.a.	3 in. @0115 hr, 8/28
8/30-9/2	n.f.	n.f.	n.a.	n.f.
9/2-9/5	~5 in. @1305 hr, 9/2	~9.1 in. @1315 hr, 9/2	n.a.	n.f.
9/5-9/15	n.f.	n.f.	n.a.	n.f.
9/15-9/20	n.f.	n.f.	n.f.	n.f.
9/20-9/23	n.f.	~1.1 in. @2040 hr, 9/21	~1.9 in. @1750 hr, 9/20 ~2.2 in. @2020 hr, 9/21	~1.3 in. @1910 hr, 9/20 ~0.8 in. @2140 hr, 9/21
9/23-10/8	n.f.	n.f.	n.f.	n.f.
10/8-10/11	n.f.	~0.7 in. @1250 hr, 10/10	~1.7 in. @1240 hr, 10/10	~0.4 in. @1730 hr, 10/10
10/11/88-2/14/89	n.f.	n.f.	n.f.	n.f.
2/14-2/17	n.f.	n.f.	n.f.	~0.4 in. @1920 hr, 12/16
2/17-5/9	n.f.	n.f.	n.f.	n.f.
5/9-5/12	n.f.	~2.0 in. @2050 hr, 5/9	n.f	n.f.
5/12-5/27	n.f.	n.f.	n.f.	n.f.
5/27-5/30	n.f.	~3.7 in. @2100 hr, 5/27	~2.5 in. @2120 hr, 5/27	n.f.
5/30-6/11	n.f.	n.f.	n.f.	n.f.
6/11-6/14	n.f.	n.f.	n.f.	~0.72 in. @1925 hr, 6/13
6/14-6/20	n.f.	n.f.	n.f	n.f.
6/20-6/23	~8.04 in. @1915 hr, 6/20	n.f.	~2.7 in. @1920 hr, 6/20	n.f.
6/23-7/29	n.f.	n.f.	n.f.	n.f.
7/29-8/1	~0.84 in. @0315 hr, 7/30	~2.3 in. @0300 hr, 7/30	~2.4 in. @0300 hr, 7/30	~0.48 in. @0335 hr, 7/30
8/1-8/10	n.f.	n.f.	n.f	n.f.
8/10-8/13	n.f.	~5.1 in. @1635 hr, 8/11	~5.3 in. @1620 hr, 8/11	n.f.
8/13-8/25	n.f.	n.f.	n.f.	n.f.
8/25-8/28	~2 in. @1420 hr, 8/26 ~3.5 in. @0200 hr, 8/27	~7.2 in. @1425 hr, 8/26 ~2.4 in. @0130 hr, 8/27	~4 in. @1445 hr, 8/26 ~3 in @0145 hr, 8/27	~5.2 in. @1530 hr, 8/26 ~26.5 in. @0130 hr, 8/27
8/28-9/12	n.f.	n.f.	n.f.	n.f.
9/12-9/15	~0.8 in. @2100 hr, 9/12	~3.8 in. @2020 hr, 9/12	~2.3 in. @2100 hr, 9/12	~0.5 in. @0300 hr, 9/13
9/15/89-1/1/90	n.f.	n.f.	n.f.	n.f.

n.f. = no flow
n.a. = not available

Table 2. Rainfall data (mm) for the 7-29-88 event, recorded at the five rain gauges.

TIME(from)	TIME(to)	Rain gauge 1	Rain gauge 2	Rain gauge 3	Rain gauge 4	Rain gauge 5
13:05	13:10	0.00	0.60	0.00	n.a.	0.25
13:10	13:15	0.00	1.00	0.60	n.a.	0.25
13:15	13:20	0.00	2.00	0.60	n.a.	0.13
13:20	13:25	3.40	3.00	0.70	n.a.	0.13
13:25	13:30	3.40	3.00	0.70	n.a.	0.38
13:30	13:35	2.00	2.00	0.60	n.a.	0.38
13:35	13:40	0.50	2.00	0.00	n.a.	1.27
13:40	13:45	0.50	1.00	0.00	n.a.	1.27
13:45	13:50	1.00	1.00	0.00	n.a.	3.05
13:50	13:55	1.00	0.60	4.00	n.a.	3.05
13:55	14:00	0.40	0.60	4.00	n.a.	1.78
14:00	14:05	0.40	0.00	10.00	n.a.	1.78
14:05	14:10	0.60	0.00	10.00	n.a.	2.03
14:10	14:15	0.60	0.40	3.00	n.a.	2.03
14:15	14:20	1.10	0.40	3.00	n.a.	1.27
14:20	14:25	1.10	1.20	0.30	n.a.	1.27
14:25	14:30	0.30	1.20	0.30	n.a.	0.00
14:30	14:35	0.30	0.30	0.00	n.a.	0.00
14:35	14:40	2.00	0.30	0.00	n.a.	0.00
	total =	16.80	20.60	37.80	- -	20.32
average (of 4 gauges) =		23.88				

Table 3. Rainfall data (mm) for the 8-2-88 event, recorded at the five rain gauges.

TIME(from)	TIME(to)	Rain gauge 1	Rain gauge 2	Rain gauge 3	Rain gauge 4	Rain gauge 5
18:50	18:55	1.40			n.a.	
18:55	19:00	1.40	1.00		n.a.	
19:00	19:05	1.00	1.00		n.a.	
19:05	19:10	1.00	1.00		n.a.	
19:10	19:15	0.20	0.30	1.70	n.a.	0.63
19:15	19:20		0.30	1.70	n.a.	0.64
19:20	19:25		0.10	1.40	n.a.	3.18
19:25	19:30		0.10	1.40	n.a.	3.18
19:30	19:35			0.30	n.a.	0.51
19:35	19:40			0.30	n.a.	0.51
19:40	19:45			0.10	n.a.	0.50
19:45	19:50			0.10	n.a.	0.13
19:50	19:55				n.a.	0.13
19:55	20:00				n.a.	0.12
20:00	20:05				n.a.	0.12
	total =	5.00	3.80	7.00	- -	9.65
average (of 4 gauges) =		6.36				

Table 4. Rainfall data (mm) for the 8-9-88 event, recorded at the five rain gauges.

TIME(from)	TIME(to)	Rain gauge 1	Rain gauge 2	Rain gauge 3	Rain gauge 4	Rain gauge 5
12:30	12:35			0.60	n.a.	0.38
12:35	12:40			0.60	n.a.	0.38
12:40	12:45			0.50	n.a.	0.26
12:45	12:50			0.50	n.a.	0.25
12:50	12:55			0.00	n.a.	0.13
12:55	13:00			0.00	n.a.	0.12
13:00	13:05			0.00	n.a.	0.00
13:05	13:10			0.00	n.a.	0.00
13:10	13:15			0.00	n.a.	0.00
13:15	13:20			0.00	n.a.	0.00
13:20	13:25			0.00	n.a.	0.00
13:25	13:30			0.00	n.a.	0.00
13:30	13:35	0.70		2.80	n.a.	0.41
13:35	13:40	0.70	0.40	2.80	n.a.	0.42
13:40	13:45	0.60	0.70	0.60	n.a.	2.00
13:45	13:50	0.00	0.70	0.60	n.a.	2.00
13:50	13:55	0.00	0.10	0.00	n.a.	1.53
13:55	14:00	0.00	0.10	0.00	n.a.	1.52
14:00	14:05	0.00	0.00	0.10	n.a.	0.13
14:05	14:10	0.20	0.00	0.10	n.a.	0.13
14:10	14:15	0.20	0.10	0.20	n.a.	0.13
14:15	14:20		0.10	0.20	n.a.	0.13
14:20	14:25			0.10	n.a.	0.12
14:25	14:30			0.10	n.a.	0.12
	total =	2.40	2.20	9.80	- -	10.16
	average (of 4 gauges) =	6.14				

Table 5. Rainfall data (mm) for the 8-21-88 event, recorded at the five rain gauges.

TIME(from)	TIME(to)	Rain gauge 1	Rain gauge 2	Rain gauge 3	Rain gauge 4	Rain gauge 5
14:00	14:05			0.60		
14:05	14:10			4.00		
14:10	14:15			3.50		
14:15	14:20			3.50		
14:20	14:25			3.00		
14:25	14:30			2.00		
14:30	14:35			0.30		0.25
14:35	14:40			0.30	0.40	0.26
14:40	14:45	1.00		0.10	4.50	0.76
14:45	14:50	1.00	0.40	0.10	4.50	0.76
14:50	14:55	0.50	0.80	0.10	2.00	1.16
14:55	15:00	0.50	0.80	0.10	2.00	1.16
15:00	15:05	0.20	0.30		0.50	1.16
15:05	15:10	0.20	0.30		0.50	1.17
15:10	15:15		0.20		0.20	1.16
15:15	15:20		0.20		0.20	1.16
15:20	15:25		0.10		0.10	1.16
15:25	15:30		0.10		0.10	0.51
15:30	15:35					0.51
15:35	15:40					0.13
15:40	15:45					0.12
15:45	15:50					0.07
15:50	15:55					0.07
15:55	16:00					0.06
16:00	16:05					0.06
	total =	3.40	3.20	17.60	15.00	11.69
average (of 5 gauges) =		10.18				

Table 6. Rainfall data (mm) for the 9-2-88 event, recorded at the five rain gauges.

TIME(from)	TIME(to)	Rain gauge 1	Rain gauge 2	Rain gauge 3	Rain gauge 4	Rain gauge 5
12:55	13:00			0.80		
13:00	13:05			3.00		
13:05	13:10	3.30		3.00		
13:10	13:15	3.30		0.90		
13:15	13:20	3.50	2.40	0.90		
13:20	13:25	3.50	1.50	0.20		
13:25	13:30	0.70	1.50	0.20		
13:30	13:35	0.70	0.70			0.76
13:35	13:40	0.20	0.70			0.77
13:40	13:45					3.43
13:45	13:50					3.43
13:50	13:55					0.76
13:55	14:00					0.76
	total =	15.20	6.80	9.00	0.00	9.91
average (of 5 gauges) =		8.18				

Table 7. Optimized LS card (SCS curve number loss rate) parameters from HEC-1 for gauging station 2.

	Channel cross-sections								
	section AA"		section AA'		section BB'		section CC'		First estimate
DATE	STRTL	CN	STRTL	CN	STRTL	CN	STRTL	CN	CN*
7/29/88	19.7	88.4	21.2	89.0	19.0	80.3	20.4	81.5	73.0
8/02/88	5.3	81.9	5.3	81.9	5.5	81.8	6.0	86.3	90.0
8/09/88	4.4	79.4	4.4	79.4	4.0	67.7	4.8	73.0	90.5
8/21/88	5.0	77.9	6.4	81.9	6.7	84.2	6.1	68.2	87.0
9/02/88	7.5	90.0	7.5	90.0	7.5	89.5	7.7	87.0	87.0

* estimated CN values used for first calibration of the HEC-1 model.

STRTL (mm) : initial abstraction before ponding
average value = 8.7
standard deviation = 5.8

CN : SCS curve number
average value = 82.0
standard deviation = 6.5
conservative value = 90.0

Table 8. Percentage of rainfall lost to infiltration for whole basin (from HEC-1 simulation).

Date	Rainfall (mm)	Loss (mm)	Loss %	Cumulative Rainfall (mm) Since Previous Event
7-29-88	23.88	23.62	98.90	0.51
8-02-88	6.36	6.35	99.80	1.45
8-09-88	6.14	6.12	99.70	17.02
8-21-88	10.18	10.05	98.70	0.00
9-02-88	8.18	8.17	99.90	2.70
100-yr event	86.36	43.17	50.00	0.00

Table 9. Five-minute interval distributions (inches) of 100-yr and probable maximum flood precipitations.

TIME (HR:MIN)	100-YR	PMF	TIME (HR:MIN)	100-YR	PMF
0:05	0.00	0.01	3:05	0.07	3.19
0:10	0.00	0.02	3:10	0.13	1.18
0:15	0.00	0.02	3:15	0.13	0.94
0:20	0.00	0.03	3:20	0.09	0.89
0:25	0.00	0.03	3:25	0.16	0.71
0:30	0.00	0.04	3:30	0.17	0.71
0:35	0.00	0.05	3:35	0.52	0.27
0:40	0.00	0.05	3:40	0.89	0.26
0:45	0.00	0.05	3:45	0.88	0.25
0:50	0.00	0.05	3:50	0.35	0.22
0:55	0.00	0.05	3:55	0.33	0.19
1:00	0.00	0.05	4:00	0.20	0.15
1:05	0.00	0.07	4:05	0.06	0.14
1:10	0.00	0.08	4:10	0.06	0.14
1:15	0.00	0.08	4:15	0.06	0.13
1:20	0.00	0.08	4:20	0.04	0.13
1:25	0.00	0.08	4:25	0.06	0.13
1:30	0.00	0.10	4:30	0.06	0.13
1:35	0.00	0.12	4:35	0.04	0.10
1:40	0.00	0.13	4:40	0.06	0.10
1:45	0.00	0.13	4:45	0.06	0.08
1:50	0.00	0.13	4:50	0.04	0.08
1:55	0.00	0.14	4:55	0.06	0.08
2:00	0.00	0.14	5:00	0.06	0.07
2:05	0.01	0.15	5:05	0.00	0.06
2:10	0.03	0.18	5:10	0.00	0.05
2:15	0.03	0.21	5:15	0.00	0.05
2:20	0.03	0.24	5:20	0.00	0.05
2:25	0.03	0.26	5:25	0.00	0.05
2:30	0.03	0.27	5:30	0.00	0.04
2:35	0.03	0.59	5:35	0.00	0.04
2:40	0.03	0.71	5:40	0.00	0.03
2:45	0.03	0.89	5:45	0.00	0.03
2:50	0.03	0.89	5:50	0.00	0.02
2:55	0.03	1.12	5:55	0.00	0.02
3:00	0.03	1.54	6:00	0.00	0.01

Total rainfall (inches):	4.9	19.5
Duration (hr):	3.0	6.0

Table 10. Peak discharges (cfs) used in HEC-2 for 100-yr flood tributary-flow option and the resulting wetted lengths and flow velocities (CN=82 and CN=90).

Alamo Arroyo watershed

Cross-section	Discharge (cfs)		Flood Width (ft)		Flow Velocity (ft/sec)	
	CN = 82	CN = 90	CN = 82	CN = 90	CN = 82	CN = 90
1	23820	30060	524	552	10.1	10.9
2	14800	18670	244	268	12.6	13.2
3	14400	17940	481	497	7.4	7.9
4	14000	17460	566	598	9.3	9.9
5	13430	16770	3567	3677	3.5	3.8
6	12780	15990	1510	1693	4.8	5.0
7	12030	15080	1045	1139	5.6	5.9
8	10910	13730	829	902	6.2	6.6
9	2750	3480	315	349	6.6	6.8
10	2330	2930	401	433	3.8	4.1
11	8140	10250	641	700	5.8	6.1
12	7790	9820	1753	1892	4.7	5.0
13	9020	11390	194	211	8.6	9.1
14	8800	11070	248	271	10.6	11.1
15	8340	10540	1547	1694	4.2	4.4
16	7820	9870	2986	3011	3.5	3.8
17	7170	9040	2780	2876	2.9	3.1
18	6550	8270	1404	1447	5.2	5.6
19	5340	6720	2268	2356	2.9	3.2
20	3900	4900	552	605	6.1	6.4

Camp Rice Arroyo watershed

Cross-section	Discharge (cfs)		Flood Width (ft)		Flow Velocity (ft/sec)	
	CN = 82	CN = 90	CN =82	CN = 90	CN = 82	CN = 90
1	30270	38150	915	999	9.5	10.1
2	29690	37400	879	942	8.0	8.6
3	29320	36930	1741	1777	8.2	8.7
4	28740	36190	1287	1334	9.0	9.6
5	28090	35340	679	745	8.6	9.3
6	27340	34380	677	695	10.5	11.1
7	26580	33410	401	434	11.6	12.6
8	25740	32330	699	773	10.0	10.2
9	24880	31210	835	918	8.7	9.5

Table 11. Peak discharges (cfs) used in HEC-2 for probable maximum flood tributary-flow option and the resulting wetted lengths and flow velocities (CN=82).

	Alamo Arroyo watershed			Camp Rice Arroyo watershed		
Cross-section	Discharge (cfs)	Flood width (ft)	Flow velocity (ft/sec)	Discharge (cfs)	Flood width (ft)	Flow velocity (ft/sec)
1	113600	791	16.0	142900	1483	14.6
2	105880	763	16.6	139390	1372	12.6
3	104510	1428	11.5	137200	2144	12.0
4	103150	3450	8.7	133730	1756	13.6
5	101200	7350	6.0	129740	1232	13.9
6	65060	4836	5.8	125270	866	15.6
7	62270	2320	8.0	120700	712	17.8
8	58170	1535	9.3	115630	1870	11.1
9	14230	1261	6.5	110400	1580	13.3
10	11720	695	6.3			
11	43320	1048	9.5			
12	40500	3437	6.4			
13	36130	4643	2.8			
14	4183	1902	8.3			
15	4236	2895	5.1			
16	17510	1662	7.0			