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GENERAL DRAINAGE PLAN
for
NORTH SCOTTSDALE, ARIZONA

Prepared for:

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EXECUTIVE SUMMARY

The City of Scottsdale is on the verge of experiencing major urbanization through a large portion of the Sonoran Desert located north of the Central Arizona Project (CAP) aqueduct and west of the McDowell Mountain drainage divide. This area, which extends north to the boundaries of the Tonto National Forest, encompasses portions of both the Indian Bend Wash and Cave Creek watersheds.

In concert with the City's commitment to manage development in a manner that will create a high standard of living, a study was requested to formulate a "General Drainage Plan" that would address the watershed hydrology, existing flooding problems, future flooding potential due to urbanization, concept drainage solutions to eliminate or reduce flooding problems to an acceptable level, and estimated costs to install a "concept" flood control system. The development of this "General Drainage Plan" was to be pursued in a manner that would also consider the potential for ground-water recharge and the water requirements for both riparian and xeric vegetation. Special emphasis was also given to developing a drainage plan that would minimize both the physical impacts on the natural fluvial processes at work in the watershed and the aesthetic impacts on the natural beauty that is characteristic of this desert environment. The development of the "General Drainage Plan" is the focus of this report.

A computerized rainfall/runoff model (HEC-1) was developed to define the hydrologic response of the approximate 135 square mile watershed. By varying the percent of impervious land

cover, to reflect both existing and forecast land-use classifications, this model was used to predict the potential for existing and future flooding problems. Existing land-use densities were obtained from a 1987 aerial photograph of the watershed, while future conditions were based on land-use projections in the Scottsdale General Plan and Tonto Foothills Plan.

The results of the rainfall/runoff model were combined with extensive field investigations and reviews of topographic and land-use maps in order to develop and examine the effectiveness of several different flood control concepts. These concepts were analyzed under the following four cases:

Case 1 - Limited channelization with on-site retention equivalent to fifty percent (50%) of the 100-year, 2-hour rainfall.

Case 2 - Regional detention basins sized to capture runoff from the 100-year, 24-hour storm. Basin release rates were based on a 36-hour drawdown period.

Case 3 - Limited channelization with no upstream detention or retention basins.

Case 4 - Limited channelization with regional detention basins (Case 2 plus Case 3).

A very approximate installation cost that would be associated with each of these scenarios is listed as follows:

Case 1 - $68,953,621
Case 2 - $98,879,548
Case 3 - $64,708,734
Case 4 - $90,410,801

These four cases present preliminary concepts that identify the general size and effectiveness of alternative drainage solutions. They should be viewed as a first step toward the development or evolution of an integrated "Master Plan" for the entire watershed. It is extremely important to emphasize the concept of an "integrated" drainage plan. Such a plan recognizes the influence or impact that the construction of specific flood control measures may have on adjacent or downstream properties. Failure to identify such impacts may merely serve to shift a flooding or erosion problem from one location to another. "Band-Aid" solutions to drainage problems in such a complex environment will only lead to long-term problems, including an increased potential for litigation.

The four cases were also used as preliminary scenarios for artificial ground-water recharge. Each case is evaluated with regard to the ease of recharge monitoring, volume calculations available for recharge and aesthetic qualities. Each recharge scenario is contingent on the drainage plan adopted by the City. Recharge credits will vary with recharge scenarios due to varying degrees of certainty with regard to actual volumes of water recharged.

The drainage system concepts presented in this report are not the final solution to the problems generated by the pending urbanization of this watershed. However, based on available data, the concepts provide a basic framework which can be used to guide future development in a direction that recognizes the potential drainage problems and offers integrated solutions that could be adopted as presented herein or modified to meet specific development scenarios that might occur in the future.

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1.0 INTRODUCTION

This report documents the engineering analysis that was performed to develop a General Drainage Plan for an approximate 135 square mile watershed in north Scottsdale, Arizona. The project study area is located north of the CAP aqueduct and includes that portion of the City of Scottsdale that lies south and west of the McDowell Mountains drainage divide. The northern limit of the study area joins the boundary of the Tonto National Forest. The project boundaries are graphically illustrated on Figure 1.1

The main thrust of this study revolves around an extensive hydrology analysis that was conducted to identify both existing and future runoff conditions in the watershed. The hydrologic models that were developed for this purpose formed the foundation for the development and analysis of several alternative flood control systems. Accordingly, the data presented in this report are based on an extensive amount of computer modeling. Seventy-nine (79) HEC-1 models were developed to provide the data presented in this report. An additional 13 HEC-1 models were constructed as part of a sensitivity analysis to select rainfall data and to verify the model results.

The output from the computer models is presented in separate publications entitled "Volume I, HEC-1 Models for Existing, Future and Maximum Density Buildout Land-Use Conditions With Existing Watershed Drainage Pattern" and " Volume II, HEC-1 Models For Concept Design of Four Flood Control System Alternatives".

The following sections of this report present detailed discussions related to the development of the HEC-1 models, the results of the modeling process, the type and cost of the evaluated flood control improvements, ground-water recharge potential, and water requirements for native vegetation.

2.0 DRAINAGE BASIN CHARACTERISTICS

2.1 DRAINAGE AREA

The project study limits and watershed boundaries are illustrated in Figure 2.1. As stated previously, the contributing watershed encompasses approximately 135 square miles.

The drainage area exhibits considerable variation in topographic features. The east and southeastern part of the basin include the McDowell Mountains, which are characterized by very rocky, steep-sloped terrain that serve as the source area for the creation of several alluvial fans. The steep slopes and highly impervious soils of these mountains are conducive to generating rapid and large rates of runoff when subjected to excessive rates of rainfall. Alluvial fans exist along the toe of the mountain slopes and coalesce westward forming the piedmont.

The central portion of the watershed is developed, in part, on of an active alluvial fan surface (Soil Conservation Service, 1986). This type of landform is difficult to analyze from both a hydrologic and hydraulic perspective because of the absence of well-defined channels and the transitory flow patterns across the terrace. The alluvial fan surface is characterized by literally hundreds of small, braided washes which have bankfull channel capacities ranging from approximately 25 cubic feet per second (cfs) to 250 cfs. During major floods, such as the 100-year event, the flow characteristics across this surface exhibit a wide, shallow sheetflow pattern. The channel patterns on an alluvial fan, or fan terrace, are very unpredictable and

unstable because of the limited channel capacity and alterations to channel geometry that often accompany the rapid erosion and sediment deposition processes that occur during periods of flooding.

North from Jomax Road to about Dixileta Road, the small washes on the alluvial fan terrace begin to exhibit more definition and corresponding channel capacity typical of inactive alluvial fan surfaces. This phenomenon tends to decrease the potential for sheetflow in the northern portion of the watershed. Substantial channel incisement is very prominent north and east of the Carefree area. North from Cave Creek Road, the watershed begins a rapid transition back to a mountain environment with very deep and well-defined channels.

2.2 LAND USE

The majority of the watershed is presently undeveloped. However, there are several pockets of scattered residential development, most notably in the Carefree area, south of Carefree along Scottsdale Road, and within an approximate two-mile radius of the Pinnacle Peak/Pima Road intersection. New development is also underway along Shea Boulevard, east of the CAP.

This existing development was simulated in the HEC-1 model (for existing floodplain conditions) by inputting a percent of impervious cover that was based on an actual count of existing structures observed on a 1987 aerial photograph of the watershed. As cited previously, future land-use conditions were also modeled by using an increase in the percent of impervious cover that was correlated with City of Scottsdale planning projections for the watershed.

These projections included residential (with densities ranging from 1/5 to 22 dwelling units per acre), tourist accommodations, low intensity resort, commercial, cultural/ institutional, office, light industrial, and open space. Data on existing and future land-use densities for each sub-basin in the project watershed was provided by Drake & Associates.

It is important to consider both existing and future land-use plans in the hydrologic modeling process because of the impact that such uses have on the runoff response of the watershed. An increase in the amount of impervious land cover will cause an increase in peak discharge rates and flooding potential.

2.3 SOIL TYPE AND VEGETATION

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

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

The composition of the project watershed in terms of hydrologic soil groups is presented in Plate 1. The information in this figure is based on the Aguila-Carefree soil survey (SCS, 1986). As can be noted in Plate 1, the watershed is composed of all four hydrologic soil groups(A,B,C, and D).

A review of the SCS soil survey maps indicated that several of the soil map units consisted of two or more major soils. Such combinations, which are referred to as either a soil complex, or soil association, often consist of multiple hydrologic soil groups. When multiple soil groups were encountered, the percentage of each soil (and its corresponding hydrologic soil group) within the soil complex/association was identified from the SCS mapping unit descriptions. These percentage figures were then combined with engineering judgement to select a single hydrologic soil group that was considered to be most representative of a specific soil complex/association. In two cases, the percentage of different hydrologic soil groups was so evenly balanced that the soil complex/association was used in the analysis as a function of two hydrologic soil groups.

Soil complex/associations that were evaluated and lists the hydrologic soil group(s) that were ultimately selected to represent a specific mapping symbol (number) are summarized in Table 2.1. The application of these data to the selection of SCS curve numbers will be discussed in Section 3.2 of this report.

Due to its predominantly undeveloped nature, the vegetation community in the watershed is typical of the Sonoran Desert and includes such species as mesquite, catclaw, creosote bush, palo verde, ironwood, cacti, etc. For those portions

of the watershed that have been developed, there has been an attempt to preserve, as much as possible, the natural desert landscape. Preservation of the natural character of the land is in concert with the Tonto Foothills Plan adopted by the City of Scottsdale.

2.4 EXISTING DRAINAGE IMPROVEMENTS

The low density of development that presently exists in the watershed has not been accompanied by any major flood control or drainage improvements. The majority of existing improvements consist of small channels and/or raised levees (berms) alongside some of the roadways or through portions of residential developments. The drainage ditches that exist along the side of some of the major roads (Scottsdale Road, Pima Road, Pinnacle Peak Road, etc.) may create some minor diversion of runoff during the more frequent floods, such as a 2-year event. However, severe floods, such as a 50- or 100-year event, will greatly exceed the capacity of these small channels and will continue to flow along their natural drainage path. During field inspections of the watershed, it was also observed most of the roads utilize "dip" sections at their intersection with the natural desert washes. This practice promotes the tendency for floodwater runoff to continue along its natural course rather than being diverted by the roadways.

Large flood control dikes are located along the north side of the CAP, but these are at the downstream limits of the study area and offer no protection to upstream areas.

Remnants of the Old Verde Canal are also located through portions of the watershed north of the CAP alignment. Due to frequent breaks in the canal embankment, no attempt was made to model any floodwater diversions that this man-made feature

might create. Impact on existing flooding potential would probable be negligible due to its location near the southern boundary of the study limits. For future watershed conditions, it was assumed the canal remnants would be destroyed as part of any development scenario.

3.0 HYDROLOGIC MODEL (HEC-1)

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

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

A network of sub-basins and connecting channels was configured that simulates the natural drainage pattern in the basin. Plate 2 presents an illustration of the drainage patterns, sub-basin boundaries, and concentration points used to model both existing and future runoff conditions. Plate 3 presents the same type of information for several of the concept drainage improvements that will be discussed in Section 6 of this report. Plates 4 and 5 present a summary of

land-use densities that were provided by the City of Scottsdale for use in this study. These data were taken from land use maps dated February 19, 1986 and July 1, 1987 for the Tonto Foothills Plan and the Scottsdale General Plan, respectively. The information on Plate 4 was used to develop percent of impervious cover to simulate runoff characteristics for future land-use conditions.

This section of the report presents a detailed discussion of specific components of the computer model that were created to simulate the rainfall/runoff response of the watershed. Complete listings of the input and output data associated with the HEC-1 models developed for this project are presented in "Volume I" and "Volume II", as previously referenced in Section 2.0 of this report.

3.1 DELINEATION OF DRAINAGE SUB-BASINS

As stated previously, the project watershed consists of approximately 135 square miles. Such a large area encompasses a wide range of topographic features, soil groups, and drainage patterns. In order to increase the accuracy of the hydrologic modeling process, it is necessary to sub-divide the watershed into much smaller sub-basins of relatively homogeneous hydrologic characteristics. The number and size of sub-basins is also dictated by the number of locations at which hydrologic output data is desired, i.e., detention basin outlets, channel locations,, etc.

In order to meet this criteria, the overall watershed was divided into 183 sub-basins (this number varied slightly to reflect differences in concentration points that were required when evaluating some of the concept drainage alternatives versus existing conditions). Plate 2

illustrates the sub-basin delineations that were used to model both existing and future flooding conditions for the watershed.

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

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

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

3.2 INTERCEPTION/INFILTRATION

Precipitation losses due to interception and infiltration were modeled using the SCS curve number option in HEC-1. Selection of curve numbers was based on information gathered on type of soil cover, vegetation density, land use, and soil

moisture conditions. An average curve number was developed for each sub-basin to account for the combined effect of these drainage basin characteristics.

A base curve number was developed for each of the four hydrologic soil groups (A,B,C, and D) under the assumption of 15% cover density and a "desert brush" vegetation community. Figure 2-15 from the City of Scottsdale Drainage Report Preparation, Section 2, Design Procedures and Criteria was used for the base curve number selection of hydrologic soil groups B, C, and D. Since this reference does not include hydrologic soil group A, a second technical reference was required. Accordingly, Table 2-2d, Urban Hydrology for Small Watersheds, SCS Technical Release 55, June, 1986 was used to develop a curve number for soil group A.

An important distinction between these two curve number references should be noted. Curve numbers in the City of Scottsdale reference are based on short duration storms (approximately 1-hour), while those in Technical Release 55 (TR-55) are based on long duration storms (approximately 24-hours). Accordingly, the soil group A curve number from TR-55 must be converted to a short duration value in order to be compatible with the B, C, and D soil group values taken from the Scottsdale reference.

The variation in curve number, as a function of storm duration, has been documented by Woodward (Runoff Curve numbers for Semiarid Range and Forest Conditions, ASAE, 1973). Based on an analysis of actual rainfall/runoff data, Woodward developed a set of curves relating changes in curve number to storm duration. These curves were used to convert the 24-hour, group A soil, curve number in TR-55 to a 1-hour curve number that would be consistent with the Scottsdale data.

As will be discussed in Section 3.5 of this report, a 24-hour storm duration was ultimately selected for use in this study. Accordingly, the 1-hour curve numbers discussed in the preceding paragraphs were converted, using Woodward's curves, to 24-hour duration values. A summary of curve number variation (by storm duration) is presented in Table 3.1.

The 24-hour curve numbers in Table 3.1 were used to develop a weighted curve number for each sub-basin in the watershed. Weighted sub-basin curve numbers were based on a visual estimate of the percentage of each sub-basin area occupied by each of the six following hydrologic soil group (HSG) categories:

1. 100% HSG A (CN = 63)
2. 50% HSG A + 50% HSG B (CN = 70)
3. 100% HSG B (CN = 77)
4. 50% HSG B + 50% HSG C (CN = 81)
5. 100% HSG C (CN = 85)
6. 100% HSG D (CN = 88)

These six categories reflect adjustments made for the multiple HSG soil complex/associations discussed previously in this section.

The "area-weighted" curve numbers that were obtained from this procedure were rounded to the nearest whole number for each sub-basin and were considered to be a final baseline curve number representative of natural desert conditions (i.e. no development). These final baseline curve numbers were used in all the HEC-1 models created for this study.

Modeling adjustments for increased runoff due to urbanization (both existing and future) were made by increasing the percent of impervious cover input to the LS card for each sub-basin; no changes were made to the sub-basin curve numbers. The relationship between percent of impervious cover and land-use classification was primarily based on "average percent of impervious area" taken from Table 2-2a, TR-55. The 85% and 72% impervious area values for commercial/business and industrial districts, respectively, were used without any adjustments. However, the percents of impervious area for residential districts were revised slightly upwards. The revision was made through a visual adjustment to a graphical plot of the residential lot sizes versus the percent of impervious area for each lot size. A smooth, visually fitted curve was then superimposed onto the TR-55 data in order to extend the data to span the entire range of zero to 100% impervious area. This graphical plot is illustrated in Figure 3.1. Combining the information from Figure 4.1, Table 2-2a (TR-55), and land-use classifications from the Tonto Foothills Plan and Scottsdale General Plan, a percent of impervious cover was established for each land-use category used in this study. These final data are summarized in Table 3.2.

In order to determine the percent of impervious cover for each sub-basin in the watershed, it was necessary to compute the area of each sub-basin being occupied by <u>each</u> of the land-use classifications listed in Table 3.2. An "area-weighted", average percent impervious was then computed for each sub-basin. Tables 3.3, 3.4, and 3.5 summarize these calculations for all sub-basins within the study area. These tables are based on <u>future</u> land-use conditions as projected by the Tonto Foothills Plan and Scottsdale General Plan.

Percent of impervious cover for existing land-use conditions was based on a visual count of every structure that was visible on a 1987 aerial photograph of the watershed. A transparency of the watershed sub-basin delineations (Plate 2) was superimposed over the photographs in order that the number of existing structures could be identified for each sub-basin. The total number of structures in each sub-basin was then divided by the total sub-basin area in order to arrive at an average number of units per acre. This average value was then used in conjunction with Figure 3.1 to determine the percent of impervious cover for each sub-basin. Tables 3.6, 3.7, and 3.8 list these "existing condition" values for all sub-basins in the watershed.

Additional adjustments to percent of impervious cover were also required to evaluate a maximum density buildout (MDB). The MDB was used to evaluate a worst-case condition in which the land-use densities in the Tonto Foothills plan and Scottsdale General Plan might be exceeded. Based on discussions with the City of Scottsdale, it was decided to simulate this case by increasing projected residential densities by 50% and non-residential percent impervious values by 10%. Accordingly the average number of dwelling units per acre for each residential land-use classification in Table 3.2 was multiplied by 1.5 and the resulting value was used to enter Figure 3.1 to obtain a new percent of impervious area to simulate the MDB.

For non-residential land-use, the percent of impervious area listed in Table 3.2 was simply multiplied by a factor of 1.1 to obtain the percent of impervious area for the MDB.

Table 3.9 summarizes the "percent of impervious area" calculations for the MDB. The values in this table were applied to each sub-basin in the watershed in order to compute an "area-weighted" percent of impervious cover for all sub-basins under the MDB condition. This procedure is identical to that previously described for future conditions. Tables 3.10, 3.11, and 3.12 list MDB impervious values for each sub-basin in the watershed.

3.3 OVERLAND FLOW PARAMETERS

Overland flow represents the shallow, sheetflow conditions that occur while runoff is moving from the point of raindrop impact to a channel. HEC-1 simulates this component of flow with input data describing the overland flow length, slope and roughness.

Except for the lower portions of the alluvial fan terrace, overland flow lengths were measured from a USGS 7.5-minute quadrangle map. These measurements, which utilized considerable engineering judgement, were based on a close examination of the topographic contour lines in order to determine the approximate distance that water would have to travel before reaching an indent in a contour line that could be considered representative of a channel. As many as four measurements were made in some sub-basins to determine an average length that could be considered typical of the entire sub-basin. In a few instances two overland flow planes were input to the HEC-1 to describe a sub-basin.

On the lower portions of the alluvial fan terrace, a different approach was taken to measure overland flow lengths. This approach was based on a 1"=1000', 1984 aerial

photograph of the watershed. Each sub-basin was delineated on this photo and lines were drawn perpendicular to the average flow pattern within each sub-basin. The number of rills or channels intercepted by each line was then made through a visual inspection of the photo. An average width between rills was then obtained by dividing the total length of the line by the number of rills intercepted by the line. The average overland flow length was then computed as one-half the distance between rills, based on the assumption that one-half this distance will drain to one rill while the other half will drain to the adjacent rill. As many as two or three lines were drawn on sub-basins in order to establish an average overland flow length for the entire sub-basin.

This fan terrace analysis was originally performed by Mr. Robert L. Ward, P.E., in 1986 and published as part of a report entitled Final Hydrology Report, Outer Loop freeway, North of the CAP Aqueduct, Simons, Li and Associates, Inc. (SLA), April, 1987. This overland flow length data has been adopted from the SLA report,, with no changes, for use in this study.

With the exception of the lower portion of the alluvial fan terrace, overland flow slopes were computed from the length and elevation measurements taken from the 1"=2000' USGS quadrangle maps previously referenced for measuring overland flow lengths.

Due to the large contour interval (20-feet) on the quadrangle maps and the relatively flat topographic relief across the lower fan portion of the watershed, a different technique had to be employed for computing overland flow slopes. Accordingly, seven wide (200' to 400') cross-sections were field surveyed on the fan portion of the watershed. Once these

cross-sections were plotted, typical cross-slopes to individual rills could be easily computed.

This was done for several cross-sections and an average cross-slope was found to be 0.0213 ft/ft. This value was then used as the average overland flow slope for all sub-basins on the fan terrace area. These cross-sections and fan slope measurements were also based on data from the 1987 SLA report prepared by Mr. Ward.

As with the length measurements, overland flow roughness values require considerable judgement. No values have been published specifically for desert land surfaces. Depths of overland flow may be on the order of 1/4-inch or less. Under such conditions, the texture or surface composition of the ground has a significant impact on the travel time required for overland flow to reach a channel element. Field inspections of the watershed revealed distinct differences in surface soil composition and vegetation density from the lower to upper portions of the watershed. On the lower fan portion of the drainage area, the ground surface is relatively smooth and flat and is composed of a much finer (smaller grain-size) material than exists in the upper basin. The upper portion of the basin exhibits gravel size surface materials, along with scattered rocks and boulders, and a much more rugged surface topography. Vegetation also appears to be slightly more dense in the upper part of the basin than in the lower part of the basin.

Based on these observations, five categories of surface topography and overland flow roughness were selected as being representative of the watershed. This data is summarized in Table 3.13.

3.4 CHANNEL ROUTING PARAMETERS

Runoff from overland flow planes is concentrated in the numerous dry washes that drain the watershed. Once the water enters these washes, it is routed downstream as open-channel flow. For the kinematic wave option, this routing procedure is a function of: 1) channel length; 2) channel slope; 3) channel shape; and 4) channel roughness. HEC-1 is capable of using as many as three different channel routing segments within a given sub-basin in order to simulate different channel geometries that occur as small collector channels drain to larger collector channels and, ultimately, to a main trunk channel.

Channel lengths and slopes were measured directly from the 1"=2000' USGS quadrangle maps. A trapezoid was used to model channel geometry throughout the watershed. The bottom width and side slopes of the trapezoid were based on extensive field measurements, aerial photographs, and engineering judgement (due to the large watershed size, it was not possible to measure every channel).

After the initial HEC-1 runs, the peak discharge values at numerous channel concentration points were used, with Manning's Equation, to compute the channel depth, velocity, and Froude Number. If these computed hydraulic parameters did not appear reasonable, the channel bottom widths and/or side-slopes were adjusted in the proper direction.

Due to the wide, sheetflow characteristics anticipated on the lower portions of the alluvial fan terrace, special consideration was given to the selection of the channel geometry. As

part of the 1987 SLA study, Mr. Ward field-surveyed six typical washes on the fan terrace. Using Manning's Equation, the average bankfull capacity of a typical wash was found to be approximately 80 cfs. This information was combined with the peak discharge data from HEC-1 to develop the following procedure for the selection of channel geometry across the lower fan terrace.

1. Using the number of washes/rills counted for each fan terrace sub-basin, and the HEC-1 peak discharge through each sub-basin, determine the average discharge per wash and the depth and velocity associated with the discharge.

2. Compare the results from Step 1 to the previously computed average, bankfull capacity of 80 cfs per wash. If this 80 cfs capacity is exceeded, sheetflow can be expected.

3. Concurrently with Step 2, compute the velocity, Froude Number, and depth of flow resulting from the HEC-1 channel geometry carrying the total HEC-1 peak discharge through each sub-basin. If the computed wash capacity from Step 1 is approximately equal to, or less than 80 cfs (Step 2), flow can be expected to be contained within the small washes and the HEC-1 channel velocity should be approximately the same as the wash velocity computed from Step 1. If these two velocities are not approximately the same, the HEC-1 channel geometry is revised, the model re-run, and this procedure is repeated. The Froude Number for the HEC-1 channel is also examined as part of this step. These Froude Numbers should be close to critical flow since that is the predominant flow regime across alluvial fans.

4. If the wash-capacity comparison made in Step 2 indicates substantial overflow would occur, then the computed wash-velocity will be erroneously high, since the wash would not actually carry all the excess water, i.e., it would spread across the desert as sheetflow. Under this condition, the computed HEC-1 channel velocities should be lower than those accompanying the excessive concentration of flow in a small wash. The lower HEC-1 channel velocities would be more representative of the shallow sheetflow that would accompany overflow of the washes.

   The acceptance of a HEC-1 channel velocity for this case is a matter of engineering judgement. Such judgement should be based on a review of the flow depth, velocity, and Froude Number accompanying the HEC-1 channel geometry. This data should be compared to what would be considered as realistic flow conditions across a wide swath of desert. In order to maintain proper timing of the flood-wave movement across the terrace, velocity might be given more weight than the other two parameters. Sheetflow velocities in the 4-7 fps range were considered realistic for this study, with the higher velocities being used where the sheetflow unit discharge was higher.

   Where necessary, the HEC-1 channel geometry was adjusted to provide a realistic range of flow velocities.

The adjustment of channel geometry dimensions across the fan terrace was found to be extremely influential in the attenuation of peak discharge as the floodwave moved down the terrace. For example, the 100-year peak discharge (existing

conditions) at SUB 27 is 9831 cfs, while approximately 4.5 miles downstream at SUB 29, it is only 6577 cfs. This attenuation was created by increasing the channel bottom width from 1500 feet in sub-basin 27 to 2500 feet in sub-basin 28, and to 3500 feet in sub-basin 29. This sensitivity justifies a careful examination of the channel hydraulics across the fan terrace.

Nearly all the channels in the watershed were modeled with a Manning's roughness value of 0.045. In some isolated cases, values of 0.050 and 0.055 were used. These roughness values were based on extensive field observations compared to calibrated "n" values presented in a photo report entitled "Roughness Coefficients for Stream Channels in Arizona" by Aldridge and Garrett, USGS, February, 1973. A summary of overland flow roughness values is presented in Table 3.13.

3.5 RAINFALL PARAMETERS

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

Rainfall depths for the study area were developed using isopluvial maps and regression equations presented in the Precipitation - Frequency Atlas of the Western United States, Volume VIII - Arizona, 1973. Due to the large drainage area size, sufficient variations in rainfall depths were noted on

the isopluvial maps to warrant using different rainfall values for different areas of the watershed. Table 3.14 summarizes the aerially reduced rainfall depths that were used in the HEC-1 models for different portions of the watershed. The rainfall depths in Table 3.14 are based on a 24-hour storm duration and a 10-square mile areal reduction factor.

The rainfall values in Table 3.14 were distributed over a 24-hour duration using the SCS Type IIA rainfall distribution. This distribution is graphically depicted in Figure 3.2, along with three other distributions that were considered for possible use in this project.

The selection of the rainfall parameters presented in this section was based on an extensive sensitivity analysis which is discussed in detail in Section 3.6 of this report.

3.6 SENSITIVITY ANALYSIS AND MODEL VERIFICATION

In order to generate a level of confidence in the HEC-1 modeling results, and to examine the sensitivity of certain rainfall parameters on the runoff response of the watershed, additional modeling and independent hydrologic calculations were performed. Both the rainfall sensitivity and model verification calculations were conducted concurrently in order to identify which rainfall parameters were providing the best agreement with the results of the independent hydrologic calculations. The results of these analyses are discussed in the following two subsections of this report. Due to the large size of the watershed, these analyses were confined to an approximate 7 square mile block of drainage area containing 13 sub-basins (SUB 2240 through SUB 2340).

3.6.1 Rainfall Sensitivity Analysis

Using the previously described HEC-1 model with the kinematic wave option for both overland flow and channel routing, the following rainfall parameters were investigated:

1. Rainfall Amount - Two sources of rainfall data were used: 1) NOAA Atlas 2, Volume VIII, Arizona; and 2) a combination of NOAA Atlas data and City of Phoenix rainfall data (Storm Drain Design Manual, September, 1985), whichever gave the higher value for a given frequency-duration.

2. Spatial Distribution - The rainfall was assumed to be evenly distributed over the entire contributing watershed. However, the sensitivity of peak discharge was examined as a function of changing the "areal reduction factor" from 10 square miles to 50 square miles.

3. Temporal Distribution - The following rainfall distributions were evaluated: 1) HEC-1 hypothetical, 24-hour; 2) HEC-1 hypothetical, 1-hour; 3) SCS Type II, 24-hour; 4) SCS Type IIA, 24-hour; 5) City of Phoenix, 24-hour.

Where possible, several combinations of these parameters were examined in order to establish the maximum and minimum values that might occur in peak discharge. However, some combinations were not possible. For examples, the SCS Type II, IIA, and City of Phoenix distributions only utilize the 24-hour rainfall value. Since the NOAA values were higher than the Phoenix rainfall values for this duration, only the NOAA data was used for these storms.

It should be noted that there is also some incompatibility in comparing runoff response due to changes in areal reduction factors. The HEC-1 program incorporates an internal subroutine that automatically adjusts all rainfall values (on the PH card) for a given storm duration when using the hypothetical rainfall distribution. This adjustment is initiated by the user inputting a desired drainage area size to be used by the program in computing the areal reduction factor - (ARF). In contrast, the two SCS distributions and the City of Phoenix distribution are adjusted as a result of the user modifying only the 24-hour rainfall for the desired areal reduction.

Another possible incompatibility in computing the ARF arises from the fact that the HEC-1 algorithm (see Equation 3.12 in the HEC-1 User's Manual, September, 1981) produces different results from that obtained from Figure 14 in the NOAA Atlas. Both methods produce nearly identical results for the 24-hour duration, but slightly different values for the shorter durations. For example, HEC-1 will generate an ARF of 0.815 for a 1-hour duration at 50 square miles, while NOAA produces an ARF of 0.800 for the same conditions. It should be noted, however, that these small discrepancies may be due to errors in visually reading such values from graphs.

The results of the rainfall sensitivity analysis are presented in tabular form in Table 3.15. Figure 3.3 graphically depicts the variation in peak discharge for a 1-hour, HEC-1 hypothetical storm when using NOAA rainfall versus the Osborn/Renard data. A plot of the various rainfall distributions was previously presented in Figure 3.2.

During a review of the sensitivity analysis results, some additional investigation was performed relative to recent research on areal reduction factors and accuracy of the rainfall data published in the NOAA Atlas. Using gage data from Walnut Gulch (AZ) and other portions of Arizona and New Mexico, Zehr and Myers (1984) concluded that "reductions of point rainfalls for area-size in the semi-arid Southwest are greater than previously published nationwide average depth-area curves". This research indicates that ARFs from NOAA do <u>not</u> reduce point rainfall values as much as new data indicate they could be reduced. In other words, ARFs from NOAA will tend to produce <u>higher</u> areal rainfall (and corresponding peak discharge) than the new data in NWS HYDRO-40 suggests is realistic for areas of the southwest.

To test the sensitivity of these factors, the Type IIA, 24-hour storm was run with ARFs for 10 square miles and 50 square miles, using the data from both NOAA and NWS HYDRO-40. For each of the two drainage area sizes, the average Q_{100} ratios (using ARFs from the two referenced sources) for 15 concentration points in the test basins were compared in order to quantify the change in runoff response. The resulting ratios were found to be as follows:

$$\frac{Q_{100},\ \text{10 sq. mi. ARF, NOAA}}{Q_{100},\ \text{10 sq. mi. ARF, HYDRO-40}} = 1.10$$

$$\frac{Q_{100},\ \text{50 sq. mi. ARF, NOAA}}{Q_{100},\ \text{50 sq. mi. ARF, HYDRO-40}} = 1.17$$

As expected, the NOAA Atlas values produce 10% to 17% higher runoff values than the new HYDRO-40 data.

A second research article deals with the accuracy of short-duration rainfall in the NOAA Atlas. Again, using gage data from Walnut Gulch, AZ, Osborn and Renard (1988) demonstrate that the Walnut Gulch data produces substantially higher rainfall values from short-duration rainfall (1-hour and less) for infrequent events such as the 50- and 100-year storms. For example, the rainfall depths for the 100-year event were approximately 27% greater than the data provided by the NOAA Atlas for the same watershed. Using a kinematic cascade rainfall-runoff model (KINEROS), Osborn and Renard demonstrate that such changes in rainfall create changes in peak discharge and runoff volume on the order of 31% to 200% and 37% to 200%, respectively.

Although the Osborn/Renard report only addresses the rainfall data for Walnut Gulch, one might assume (for the purpose of a sensitivity analysis) that the same relative increases in short-duration rainfall might apply to other regions in Arizona that are exposed to short-duration, convective thunderstorms. Accordingly, the 1-hour, 15-minute, and 5-minute rainfall values for the HEC-1 hypothetical 100-year , 1-hour storm were increased by factors of 1.29, 1.25, and 1.20, respectively to match the increases calculated by Osborn and Renard for these duration-frequency combinations.

When input to the HEC-1 model, these new rainfall values caused an average increase of 59% in Q_{100} at the 15 concentration points used in the sensitivity analysis. This data was based on a 100-year, 1-hour storm. The analysis was not extended to longer duration events because the Osborn/Renard data did not go beyond a 1-hour storm.

Based on the results of this analysis, the following recommendations were made relative to rainfall parameters for use in the General Drainage Plan for the North Scottsdale Area:

1. Rainfall Amount - Use NOAA Atlas 2, Volume VIII, Arizona, with separate rainfall calculations for the northern, central, and southern portions of the watershed.

The higher City of Phoenix rainfall data is not recommended for combination with the NOAA data for the following reasons:

a) The City's Storm Drain Design Manual indicates the rainfall is based on WTBM-44, which is believed to be the same data as the NOAA Atlas. Most of the differences with the published Phoenix rainfall data and the NOAA data generated for this study area, occur for durations of 1-hour or less. All rainfall values for less than a 1-hour duration are based on a percentage of the 1-hour value. Accordingly, if the 1-hour value is in error, all lesser duration values will be in error. An analysis of the published Phoenix 1-hour rainfall values indicate that they are slightly in error when compared to the 1-hour rainfall values that result from using the statistical equations presented on page 15 of the NOAA Atlas. This, of course, causes all lesser duration values to also be in error.

b) If the Phoenix numbers are not based on the NOAA Atlas, there would arise a problem of technically justifying a valid basis for combining data from two different sources. If a combination of data were to be made that

would cause an increase in runoff (beyond the NOAA data), a valid technical basis would be warranted for such a combination in order to defend the data against potential future critics.

The Osborn/Renard research at Walnut Gulch is considered too limited to be applied on a generalized statewide basis at this time. As discussed previously, their research only addressed storm durations up to 1-hour. Accordingly, there is no guidance provided on possible errors associated with longer duration events, such as the 24-hour storm.

2. Spatial Distribution - Recommend that rainfall be evenly distributed over the entire contributing watershed. Point rainfall values should be reduced to simulate a storm size of 10 square miles. The areal reduction factor (ARF) should be taken from the NOAA Atlas, Figure 14. When compared to data in the Zehr/Myer article, the use of Figure 14 (NOAA) will provide a degree of conservatism for runoff calculations.

The lower ARFs suggested in the Zehr/Myer report are not recommended because of their admission of considerable variation in the uncertainty of the results (on a statewide basis) due to sparse data in certain areas of the state.

Adoption of the ARFs from the NOAA Atlas will also, to some extent, counteract the effects of the suggestion by Osborn and Renard that the NOAA rainfall data may be lower than that which actually occurs in various regions of Arizona.

3. Temporal Distribution - Recommend adoption of the 24-hour, SCS Type IIA storm. The 24-hour duration will provide suitable runoff volumes for detention/retention basin design while the Type IIA distribution incorporates a short burst of high intensity rainfall to simulate convective thunderstorm characteristics that are common to the project area. This intensity will provide the high peak discharges that are recommended for the design of any channel improvements. This distribution was developed on the basis of thunderstorm data collected from the Arizona - New Mexico area.

The HEC-1 hypothetical distribution was rejected because of a tendency to generate peak discharge values that are significantly higher than discharges obtained with other distributions and independent peak discharge calculations. This distribution makes no attempt to acknowledge regional rainfall characteristics. The hypothetical distribution in HEC-1 also exhibits large variations in peak discharge when adjustments are made to reduce point rainfall as a function of the assumed areal extent of a given storm. For example, when adjusting the ARF from 10 square miles to 50 square miles, the HEC-1 hypothetical storm revealed an average decrease of 25% in peak discharge values, while under the same circumstances, the Type IIA storm only caused an average reduction of about 7% in peak discharge. As discussed previously, these differences are due to the way in which the ARF is applied to the rainfall values for the two storms.

It is believed that use of the HEC-1 ARF algorithm might lead future users of the HEC-1 models created for this study to unwittingly make adjustments for areal distribution of rainfall and end up with large variations in the peak discharges used for channel improvements in different parts

of the watershed. As noted previously, changes in the NOAA ARF, with subsequent application to the Type IIA storm, has much less influence on peak discharge.

The City of Phoenix rainfall distribution was rejected on the basis that it provides substantially lower rainfall intensities, during the 1-hour burst of rainfall in the middle of the storm, than the Type II, IIA, and HEC-1 hypothetical distributions. For example, maximum rainfall intensities for these distributions (based on a 24-hour rainfall depth of 4.23" and a 5-minute computation interval) were found to be 2.76, 4.68, 4.68, and 8.04 in/hr for the City of Phoenix, Type II, Type IIA, and HEC-1 hypothetical distributions, respectively. The low intensity associated with the Phoenix distribution translates into a significant reduction in peak discharge (see Table 3.15).

3.6.2 Verification of HEC-1 Model

In order to establish confidence in the results of computerized hydrology analyses, it is important to develop some procedure to calibrate and/or verify the computer results with measured data. Normally, the preferred approach is a two-step process, i.e., calibration followed by verification.

Calibration is the process of changing model coefficients, or other judgmental input parameters, until the model matches (with reasonable accuracy), the results from a measured event. Verification is the process of checking a calibrated model against a data set not used in the calibration process.

As might be expected, the scarcity of measured data makes the calibration/verification process a difficult achievement. However, the absence of measured data can be overcome, to some extent, by employing several independent methodologies to calculate peak discharge values at the same concentration points used in the HEC-1 model. These independent estimates can be compared to the HEC-1 results to see if sufficient differences result that would warrant adjustments to the model input parameters.

Four independent calculation procedures were selected to verify the results of the HEC-1 modeling used for this project. These procedures are listed as follows:

1. Peak discharge regression equations presented in Estimation of Magnitude and Frequency of Floods in Pima County, Arizona, With Comparisons of Alternative Methods, USGS Water Resources Investigations report 84-4142, Table 2, J.H. Eychaner, August 1974.

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. Graphical peak discharge method presented in Urban Hydrology for Small Watersheds, Technical Release 55, Soil Conservation Service, USDA, June 1986.

4. 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.

Although the regression equations developed under Procedure 1 were based primarily on stream gage data in and around Pima County, their use in the north Scottsdale area is justified on the basis of similar watershed characteristics in both areas.

Procedure 2 utilizes different regression equations for five geographical regions of Arizona. Although the north Scottsdale drainage area physically lies within the delineated boundaries of Region 3, its watershed characteristics are more representative of the southwest desert area defined as Region 2. Accordingly, the Region 2 regression equations were used for this study.

Procedure 3 (TR-55) is based on an SCS Type II rainfall distribution and uses a time of concentration that evaluates sheet flow, shallow concentrated flow, and open channel flow. Where applicable, the same overland flow and channel routing parameters that were used in the HEC-1 model were used in this procedure. The same SCS curve numbers were also used in the TR 55 procedure as were used in the HEC-1 model.

Procedure 4 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 rate (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 individual sub-basins whose times of concentration are less than three hours. Since this procedure is based on short duration storms, all SCS curve numbers taken from the HEC-1 model were converted to 1-hour values using Woodward's curves.

As stated previously, the large size of the project watershed dictated that only a small "test" section be selected for the model calibration/verification process. An approximate 7 square mile drainage area southwest of Thompson Peak (McDowell Mountains) was chosen for this analysis. Six sub-basins within this 7 square mile area were selected as being typical of three different geographical areas that compose the majority of the study area. These areas, and test sub-basins, are listed as follows:

1. Mountains - Sub-basin 2300
2. Mountain Foothills - Sub-basins 2255, 2270
3. Alluvial Fan/Fan Terrace - Sub-basins 2240, 2260, and 2290.

Table 3.16 presents a summary of the independent peak discharge calculations that were performed for each of these six sub-basins. For comparison purposes, the peak discharge values from the HEC-1 model (using the 24-hour, SCS Type IIA rainfall distribution) are also listed in this table.

Figures 3.4 through 3.9 graphically illustrate the data presented in Table 3.16. These figures also plot peak discharge data from the four rainfall distributions that were previously discussed in Section 3.6.1 and tabulated in Table 3.15. Figures 3.4 through 3.9 present a concise illustration of the sensitivity of the HEC-1 model to different rainfall distributions and provide a basis for comparing the results of these distributions to independent peak discharge calculations.

A review of the data in Table 3.16 and Figures 3.4 through 3.9 led to the conclusion that the 24-hour, SCS Type IIA rainfall distribution produced the most consistent agreement

with the independent peak discharge calculations. This distribution produces results which lie between the maximum and minimum extremes associated with the HEC-1 hypothetical distribution and the City of Phoenix distribution, respectively.

Results from the Type IIA distribution appear reasonable when compared to the results from the independent methodologies. It is most consistently supported by the results from the Pima County peak discharge procedure which, of the four independent methodologies, was considered most reliable for this environment. Since the USGS regression equations are essentially reflecting average watershed conditions, they might well be expected to under-predict peak discharge when applied to very steep, impervious mountain areas such as Sub-Basins 2300 and 2270; the results of the analysis confirm this supposition.

It is interesting to note that application of the TR-55 procedure to the mountainous terrain did not provide better agreement with the HEC-1 model results, since essentially identical input data was used in both methodologies. The resulting differences in peak discharge can only be attributed to different data processing algorithms in HEC-1 versus TR-55.

Overall, the results of this analysis lend confidence in the performance of the HEC-1 model when using the input data logic that was used to describe the physical drainage basin characteristics of the six test sub-basins. Since this same logic was used in generating the input data for the remaining sub-basins in the 135 square mile watershed, it can be concluded that all the watershed models are providing output data that is reasonably representative of the true rainfall/runoff response of the project study area.

4.0 ANALYSIS OF EXISTING AND FUTURE FLOODING POTENTIAL

A prerequisite to developing a drainage plan for a study area is to conduct an analysis of existing drainage patterns and identify, on a large-scale basis, those regions of the watershed that are most prone to flooding problems. Such an analysis was conducted using the previously described HEC-1 model and extensive field observations of watershed features.

As discussed in Section 3.2 of this report, three basic conditions were evaluated using the existing drainage pattern in the watershed:

1. Existing land-use conditions
2. Future land-use conditions based on projections in the Tonto Foothills Plan and Scottsdale General Plan.
3. Future maximum density buildout conditions that would greatly exceed present planning projections.

All three of these land-use conditions used the same HEC-1 sub-basin delineations and channel routing scenario that is depicted on Plate 2. The only modeling differences between the three conditions are the variations in percent of impervious cover that were previously discussed in Section 4.2. Since all three land-use conditions utilize an identical HEC-1 modeling structure,, changes in runoff response can easily be determined by simply reading the peak discharge at the same concentration point for each land-use condition.

The entire project watershed was modeled for the 2-, 10-, and 100-year events under land-use conditions 1 (existing) and 2 (future, based on published planning projections). Land-use condition 3 (maximum density buildout) was modeled for these same three events, but only for Sub-basins 1 through 53, and 2540 through 2680; these limited areas represent regions that are experiencing rapid development.

Tables 4.1, 4.2, and 4.3 present summaries of peak discharge data for selected concentration points under all three land-use conditions for the 100-, 10-, and 2-year events, respectively. Similar data for other concentration points in the study area may be obtained by reference to Plate 2 and the comprehensive HEC-1 output data contained in Volumes I and II of this report.

Due to the large size of the project area and the conceptual nature of this study, it was not practical to make detailed assessments of floodplain limits along every wash. Such an effort would require detailed topographic mapping and a substantial manhour effort. However, based on the results of the hydrologic modeling process, extensive field observations, reviews of USGS quadrangle maps and aerial photographs, it is possible to present a general appraisal of those regions of the watershed that are most prone to potential flooding problems.

As discussed in Section 2.1 (Drainage Area), the watershed can be described as consisting of three general geomorphic regions that each exhibit different flooding potentials to existing and future development. These three regions are discussed in the following paragraphs.

Region 1 (Sub-Basin Series 2000)

This region consists of the abrupt, mountainous terrain of the McDowell Mountains and the alluvial fans and fan terraces that lie along the south and western perimeter of the mountain front.

Floodplain limits could be readily identified along the well-defined canyons that extend into the mountain range. The abrupt rise in elevation along these canyons provides ample opportunity to locate any proposed development well above the flooding that would occur across the canyon floor.

Along the south side of this mountain range, the alluvial fans exhibit considerable evidence of fanhead entrenchment. Fan incision, due to local runoff emanating from the fan surface, is also apparent. Fanhead entrenchment is usually associated with a lack of tectonic activity (uplift) in the mountain source area. This entrenchment starts at the fan apex and gradually moves downstream across the fan surface. As this entrenchment progresses, more and more of the fan surface is cut off from the sediment being supplied by the mountain source area. As a result, rain falling directly on the fan surface will begin to incise localized channels that are not connected to the fan apex.

Fanhead entrenchment and localized incision are well developed along the south side of the McDowell mountains. As a result, these channels have substantial capacity to carry floodwater within their banks. Accordingly, as long as development is set back a reasonable distance from the channel banks, this area

should be free of any significant flooding problems. However, a major exception to this general assessment exists along the downstream end of Lost Dog Wash, at the end of Sub-basin 2450. An incised fanhead trench begins to lose definition at this point and exposes downstream areas to a very unpredictable sheetflow pattern. Development through this flood prone area should not be attempted without provisions for some type of flood control measure, preferably an armored channel.

The pattern of fairly well-defined washes continues as one proceeds from the south to the western side of the McDowell Mountains. For the most part, these washes appear to exhibit sufficient hydraulic capacity that development could probably be located on the high ground between washes and be relatively safe from flooding due to any overflow from the washes. However, due to variable nature of the cross-sectional geometry of these washes (throughout the watershed), site-specific floodplain mapping should be performed as part of any proposed development in order to identify any breakout points and maximum expected water surface profiles.

In addition to the previously cited Lost Dog Wash, a second major flooding potential occurs in Region 1 at an alluvial fan apex located at the east end of Pinnacle Peak Road. This location is on the border of Region 1 and Region 2 and is situated near the outlet of Sub-basin 48. The sand-bed channel which drains this area is approximately 60 feet wide and 2 feet deep at a point approximately 1000 feet south of Pinnacle Peak Road. The estimated bankfull capacity of this channel is less than 1000 cfs, while the estimated 100-year peak discharge is 14,380 cfs at this apex location. Obviously, such a large disparity between channel capacity and channel inflow will result in wide-spread flooding.

There is no way of predicting the actual flow pattern, or distribution of water, that would accompany a major flood at this site. However, due to the limited channel capacity, and knowledge of alluvial fan flooding characteristics, the water would almost certainly inundate a wide area. With such a large flood, it is also possible that an avulsion might be created that would channel more of the floodwater to one portion of the fan than another; however, there is no way of predicting the occurrence of this phenomenum or the redistribution of water across the fan surface that would be created by an avulsion.

Sub-basin 2005 would be the first area of Region 1 to receive waters from this alluvial fan apex. Due to the uncertainties of predicting flow patterns in such an environment, the HEC-1 model was constructed with a divert operation at the fan apex (Concentration Point 51) which directs 50 percent of the apex flow into Sub-basin 51 and 50 percent of the flow into Sub-basin 2005. This divert ratio takes a "middle-of-road" approach and was deemed adequate for general analysis purposes; however, in reality, it should be emphasized that this divert ratio could justifiably be based on an infinite number of different flow distributions.

For the purpose of developing specific tracts of land downstream of this fan apex, it would be recommended that worst-case scenarios be explored that maximize the divert ratio into the area under study. This would include Sub-Basins 2100 and 2120, since they could potentially receive some flows from the fan apex as the water spreads easterly across Sub-Basing 51, 52 and 53. The lower portions of Sub-Basins 2100 and 2120 might

also be exposed to a westerly migration of the alluvial fan outflows that are being routed through Sub-Basins 2090 and 2110. For the purpose of establishing a HEC-1 channel routing path, the outflows into Region 1 from the Pinnacle Peak Road alluvial fan apex were routed through Sub-Basins 2005, 2010, 2015, 2030, 2090 and 2110. Based on a review of aerial photographs and topographic mass, this appears to be the primary path that the majority of these flows would follow through Region 1 under present day conditions. However, there is perhaps an equal probability that major portions of the apex outflows could also travel through Sub-Basins 51, 52 and 53, which are located in Region 2.

This specific fan apex (east end of Pinnacle Peak Road) was considered to be one of the most critical flood prone areas in the watershed. Any development downstream of this apex should proceed with extreme caution and a complete understanding of alluvial fan characteristics. Failure to do so could lead to the potential for substantial flood damage.

Region 2 (Sub-Basin Series 1 to 53)

Region 2 is primarily composed of a large alluvial fan terrace. The northern part of this terrace (approximately north of Dynamite Road) exhibits a slightly "rolling" topography which provides the natural washes with sufficient cross-sectional geometry to convey floodwater within fairly well-defined limits. In most cases, the high ground that exists on the "ridges" between washes, should provide sufficient elevation to prevent major flooding problems to any proposed development.

However, south of Dynamite Road, the fan terrace is characterized by a smoother surface that is composed of hundreds of small, shallow, braided washes with average bankfull capacities of approximately 80 cfs. During large floods, such as the 100-year event, this portion of the fan terrace will be subjected to widespread floodwater inundation in response to the sheetflow conditions that will result from these small washes exceeding their bankfull capacity. Such areas of sheetflow might exhibit flow depths on the order of 6 to 18 inches and velocities ranging from 3 to 7 feet per second.

Sheetflow should be anticipated throughout the lower portions of this fan terrace. However, the areas with the highest probability of experiencing major damage from this phenomenum would be through Sub-basins 51, 52, and 53, which are located downstream of the active alluvial fan apex discussed for Region 1, and Sub-basins 27, 28, and 29, which are expected to receive large concentrations of runoff from the upper reaches of the fan terrace, north of Dynamite Road. However, the identification of these two high-risk areas should not be construed as exempting other locations on the lower portion of the fan terrace from serious flooding potential. The development that has previously occurred in this area shows little evidence of acknowledging the flood damage potential that is characteristic of this alluvial fan environment. Field inspections of this area revealed numerous residential structures with inadequate elevation of finished floors. Other structures were located immediately adjacent to the banks of washes, thus exposing such structures to both

floodwater inundation and structural damage that would occur in response to bank erosion undermining the foundations of such structures.

In the eastern part of Region 1, Sub-basins 39 to 50 comprise the drainage area that contributes runoff to the alluvial fan apex at the east end of Pinnacle Peak Road. This upstream area is characterized by a very hilly or rolling type of topography (even mountainous at some locations) that contains fairly well-defined washes that are sufficiently incised to preclude the potential for sheetflow. The high ground separating these washes could be developed with a minimal risk of flooding problems.

Region 3 (Sub-Basin Series 1000)

Region 3 also contains part of the alluvial fan terrace described for Region 2. However, the majority of this terrace that is located in Region 3 is of the rolling topography category that exhibits a larger wash capacity than that occurring on the lower portion of the terrace.

From a geographical perspective, Scottsdale Road might be considered as a general division line between the sheetflow portions of the terrace and the more incised areas of the terrace, with the sheetflow potential being located west of Scottsdale Road. Using this delineation, the area east of Scottsdale Road would be more easily developed because of the tendency for the natural washes to contain the majority of floodwater. Conversely, the higher probability of sheetflow west of Scottsdale Road would subject this area to more flood damage potential.

As one enters the northern part of Region 3 (approxi- mately north of an easterly extension of Carefree Highway), the topography becomes much more hilly and actually begins to transition into a mountain/foothills environment that is characterized by extremely well-defined and incised washes. The floodplain limits that are associated with these washes can be easily identified and used to define areas of developable property. The vertical topographic relief in this area should provide ample high-ground for relatively flood-free urbanization.

The preceding discussion should only be considered as very general, or qualitative, assessments of the flooding potential throughout the study area. Tables 4.1, 4.2, and 4.3 provide some insight into the increased flooding potential that might be expected to accompany future development of the watershed. A mandatory prerequisite for development to occur at any location in the watershed should include a site-specific floodplain analysis that recognizes the unique flooding hazards that exist in alluvial fan environments.

As a concluding data-set related to existing flooding potential, Table 4.4 summarizes the depth of flow, velocity of flow, topwidth, and minimum bank elevation for 39 cross-sections that were measured in the field. This data is listed for the 2-, 10-, and 100-year peak discharge for existing conditions. The dimensions of these cross-sections should be considered very approximate since they were based on both visual estimates and rough tape measurements. All hydraulic calculations were performed with HEC-2, using an average channel slope taken from the USGS quadrangle maps. When reviewing the data in Table 4.4, bankfull capacity is exceeded when the depth of flow is greater than the minimum bank elevation.

The increased volume of runoff that will be associated with future development conditions was also examined in order to provide an approximation of the additional inflows that might be delivered to the CAP detention basins. This information is presented in Table 4.5 in the form of 24-hour runoff volumes for selected sub-basins that outlet near the CAP. The volumes of water listed for each sub-basin reflect all contributions from any sub-basins upstream of the listed sub-basin.

Table 4.5 also provides sub-totals of runoff volume for groups of sub-basins that contribute to specific detention basins along the CAP. All calculations are based on a 100-year, 24-hour storm with a 10 square mile areal reduction factor.

5.0 ANALYSIS OF DRAINAGE IMPROVEMENT CONCEPTS

This section of the report describes the technical analysis that was completed to identify the performance characteristics of four different flood control system alternatives. These alternatives, which include various combinations of detention basins, retention basins, and channels, were evaluated at a concept level in order to identify maximum/minimum sizing envelopes, general performance characteristics, and approximate installation costs. As such, these concepts are not final design oriented; they are merely a first step in the process of finding effective solutions to the drainage problems that are being created by urbanization of the study area.

The drainage/flood control system concepts presented in the following sections of this report present ideas which may be considered for application to specific locations in the study area other than those that are discussed herein. Without knowing exactly where and how urbanization will actually occur, there is no way a final flood control system plan can be recommended as part of this report. Accordingly, the approach taken in this study was to develop hypothetical plans for the majority of the study area where high probabilities of flooding problems are anticipated. Such an approach provides a mechanism for evaluating the effectiveness and size of the proposed concepts under simulated "real world" conditions. However, for reasons previously stated, there is no way, at the present time, to forecast where every drainage problem will occur. Therefore, the proposed concepts may eventually have application to other watershed locations than those discussed for each of the four case evaluations.

Regardless of the manner in which the proposed drainage concepts are ultimately revised to accommodate specific development scenarios, it is very important that the City of Scottsdale not lose sight of the need for an integrated drainage plan that addresses the entire watershed. Such an approach anticipates and plans for the impacts that a specific improvement may create to adjacent properties. Some impacts, such as stream-bed degradation, may be propagated several miles downstream from the location of a detention or retention basin.

The location of specific drainage improvements and the HEC-1 routing schematic are shown on Plate 3.

5.1 CASE 1 - CHANNELIZATION AND ON-SITE RETENTION

This drainage system concept simulates the effectiveness of on-site retention basins and limited channelization downstream of retention areas. For the purpose of this study, Case 1 is defined as follows:

1. Requires on-site retention for fifty percent (50%) of the 100-year, 2-hour rainfall (areally adjusted for a 10 square mile storm size).

2. Provides manmade channels along the east side of certain reaches of Pima Road and Scottsdale Road. These channels will discharge to the CAP dikes.

3. All channels will incorporate periodic low-flow outlets to maintain a limited flow of water to the natural washes that are intercepted by the channel alignment. This will help maintain the natural vegetation along these washes.

Case 1 simulates the required on-site retention for all drainage areas upstream of basin sites labeled on Plate 3 as DB 9/10, DB 11, DB 12, DB 13, DB 22, DB 24, DB 25, and DB 26.

The selection of on-site retention requirements was based on discussions with the City of Scottsdale and an engineering analysis which computed the average percentage of direct runoff (DRO) that was expected to accompany the future land-use projections for the watershed. This analysis utilized the total sub-basin area, percent of projected impervious area, and the SCS curve number (for pervious areas) to compute the total DRO for each sub-basin. Impervious areas were assumed to generate 100% DRO. The total volume of DRO was then compared to the volume of rainfall (using the 100-year, 2-hour precipitation value of 2.65") to determine the percent of total rainfall that is converted to DRO. Based on a sampling of 15 sub-basins, with different land use classifications, the average DRO was found to be 50% of the total 100-year, 2-hour rainfall.

It is important to note that this percentage if DRO could justifiably be increased if the 24-hour rainfall were to be used instead of the 2-hour rainfall. The reader will recall that the 24-hour storm was used for all HEC-1 modeling presented in this report.

This percentage would also be increased of a 2-hour curve number were used to compute the interception/infiltration losses associated with the 100-year, 2-hour rainfall. Such an adjustment would create a higher curve number with resultant increases in runoff. A 24-hour curve number was used with the analysis since it was assumed the 2-hour rainfall was imbedded within the 24-hour rainfall that was distributed over a 24-hour time period.

Sample calculations using the 24-hour rainfall or the 2-hour curve number (with the 2-hour rainfall) indicated that the average amount of DRO might be as high as 65% of the total rainfall. Either of these two alternatives might be optional criteria that could be considered by the City should on-site retention be pursued as part of a General Drainage Plan for the watershed.

The effectiveness of on-site retention was evaluated with the reservoir storage-routing option in HEC-1. Retention basin performance was simulated by placing a theoretical 10-foot deep, cubical basin at the outlet of each drainage area for which on-site retention was being modeled. The volume of this basin was made equivalent to 50% of the 100-year, 2-hour rainfall. Basin surface area was then computed as volume divided by depth.

Having defined the geometry of the basin, a state/storage relationship was developed for input to the HEC-1 model. No outflows were allowed from this theoretical basin until a ponding depth of 10 feet had been reached. At this depth, the 50% DRO storage requirements were satisfied and any additional basin inflows could be released. These basin outflows were simulated by coding a top-of-dam and spillway elevation of 10 feet into the model. A weir-crest length of 501 feet as used to simulate basin overflow. As a matter of interest, it should be noted that the HEC-1 model would not perform routing operations through these artificial retention basins as long as the outflow from the basins was zero flow. In order to remedy this problem, a 1 cfs rate of discharge was added to each basin to force a finite amount of discharge during the time the basin was being filled to the prescribed depth of 10 feet.

The effectiveness of the proposed on-site retention can best be examined by comparing peak discharges, with and without basins in place, and at identical downstream concentration points. Such data for future land-use conditions is presented in Table 5.1, along with other pertinent basin parameters. Table 6.2 presents the same data for DB 22 and DB 24 under the maximum density buildout (MDB) scenario. The MDB scenario uses the same retention volume as for future land-use conditions; however, the volume of runoff to the basins is increased as a result of the higher degree of impervious cover.

Similar data for any desired location in the watershed can be obtained from the HEC-1 output summaries published in Volumes I and II of this report.

As can be seen from reviewing Table 5.1, on-site retention, in the prescribed volumes, is very effective in reducing peak discharge. Based on future land-use conditions, on-site retention was found to cause a reduction in peak discharge of 66% to 92%. For the eight basin locations that were modeled in this study, the average reduction in peak discharge was 82%.

The future condition retention basins were found to produce peak outflow rates that ranged between the 2- and 10-year peak discharge values at the same locations for existing watershed conditions with no upstream basins in place. This observation is based on a review of modeling results at DB 11, DB 13, and DB 22.

The impact that on-site retention has on downstream channel dimensions will be discussed in Section 5.3 (Case 3-Channelization).

5.2 CASE 2 - DETENTION BASINS

Case 2 evaluates the construction of regional detention basins on some of the major washes in the study area. All detention basins would be designed to drain within 36 hours after first receiving water. No manmade channels would be constructed as part of this alternative. The detention basins would be sized to reduce downstream peak discharges to a level that would not create widespread flooding. Any future development would be required to avoid the reduced floodplain limits that would remain downstream of those basins.

Pursuit of detention basin construction on an alluvial fan, or fan terrace environment, is complicated by two factors:

1. The topography of a fan terrace exhibits very shallow washes and tendencies towards sheetflow conditions. Under such conditions, long collector dikes would have to be built across the fan in order to intercept the sheetflow and funnel it into a detention basin. The shallow depth of the washes provides no natural storage capacity to hold or detain the flow once it has been collected. This is in contrast to the natural storage that would be provided by building a dam across a deep canyon.

   This lack of natural storage then requires that either pits be constructed below ground or high embankments be constructed above ground in order to provide artificial storage. Such large structures for above ground storage usually fall under the classification of a dam and must be constructed under State Dam Safety criteria.

2. Unless special precautions are taken, the construction of detention basins will trap sediment. This can occur in both retention and detention basins, regardless of whether the basins are constructed with the storage below natural ground or above ground.

   Retention of this sediment will cause long-term degradation in the washes downstream of such basins. This degradation occurs as the stream bed flattens its slope in order to reduce the flow velocity so that the sediment transport capacity will be reduced to match the reduction in sediment supply leaving the upstream retention/detention basin. The end product of this process is a deepening of the natural washes which promotes bank sloughing and lateral erosion.

The first of these two limitations is reflected in this study by the fact that no detention basins were evaluated south and west of an area bounded by Dynamite Road and Pima Road. This region comprises the flattest portion of the alluvial fan terrace where natural detention basin storage is non-existent.

Natural storage is even questionable in several of the basins that were evaluated upstream of these boundaries. However, as indicated previously in this report, as one moves north and east through the study area, the topography becomes more rugged (hilly), and thus more suitable for detention basin construction.

The second limitation on detention basin construction is applicable throughout the watershed, i.e., no matter where one locates a basin, the potential exists for trapping sediment and creating downstream degradation problems. However, this problem can be largely eliminated if the basins are constructed in a manner that would allow the sediment to pass through the basins. This could be accomplished by constructing a basin outlet that is flush with, and centered over, the existing streambed where the basin is located.

Such outlets would be designed to limit the discharge to a pre-determined peak that would satisfy downstream flood control requirements, yet still providing very little resistance to the free passage of the bed-material load, which is the critical sediment transport component in long-term aggradation/degradation.

During the course of this study, proportional weirs were investigated as a promising concept to achieve this dual goal of reducing peak discharge without significantly retarding the passage of bed-material load. Proportional weirs, which operate on a linear head-discharge relationship, would be constructed with their crest level with the existing stream bed. Figure 5.1 presents a sketch of the proportional weir concept. These weirs have been previously used for detention basin outlets in Colorado, but not at the scale or size that would be required for this project. Accordingly, if this concept were ever pursued, physical model testing would be recommended to verify the performance characteristics of the outlets. Multiple concrete box culverts might also be considered as an alternative to a proportional weir outlet.

All detention basins that were evaluated in this study were based on above ground storage being achieved through natural means or by the construction of artificial embankments. Below ground storage is possible, but would tend to promote sedimentation and clogging in the outlet conduits that would have to be constructed (below ground) to drain the basins. Headcutting, upstream of the inlets to below ground basins, would also be a problem that would require some type of cutoff wall design.

Due to the concept level approach pursued in this study, it was not economically feasible for the City of Scottsdale to acquire detailed topographic mapping of the study area.

Accordingly, USGS quadrangle maps were used to obtain all topographic data required for the detention basin analyses. These maps were enlarged to a scale of 1" = 1000' and superimposed onto a 1987 aerial photograph of the watershed. The 20-foot vertical contour interval on these maps creates a limitation on the accuracy of the stage/storage relationship that was developed for each basin.

Each basin was located on the topographic photo overlay in order that the surface area associated with each vertical contour line could be planimetered. These areas were then plotted versus elevation in order to develop a graph that could be used to convert maximum basin water surface elevations to a maximum area of surface inundation. This elevation/area relationship was also input to the HEC-1 model for use in the conic method of computing a state/storage relationship for each basin. As with Case 1, the reservoir storage routing option in HEC-1 was used for the detention basin analyses.

As stated previously, a 36-hour drawdown period was specified for each basin. This requirement was simulated by computing the average, constant discharge that would evacuate the basin inflow volume in a 36-hour period. A constant discharge of 0.3361 cfs will evacuate a volume of 1 acre-feet in 36 hours. Accordingly, the average discharge for the basins evaluated in this study was computed as 0.3361 times the 24-hour inflow volume to each basin. These computed outflow rates were input to the reservoir routing data used to model each basin.

It is recognized that the actual discharge from the basins will vary as a function of the hydraulic head in the reservoir. However, it is emphasized that the primary purpose of the detention basin analysis was to determine maximum basin dimensions, storage requirements, and inundated surface areas. In all cases, the basin outflows are so small that variations in discharge due to fluctuating head would have a negligible downstream impact.

As a matter of practical interest, it should be noted that the proportional weir concept would most likely use much higher release rates than those listed herein for a 36-hour drawdown time. These higher release rates would be required in order to pass a sufficient rate of sediment to prevent downstream degradation and/or general scour. Higher release rates would mean smaller reservoir requirements but larger downstream floodplain limits. Actual basin dimensions would have to be based on site-specific basin characteristics and consideration of downstream development scenarios.

Table 5.3 presents detention basin performance characteristics for 21 sites that were evaluated as part of this study. This data is based on a 36-hour drawdown period and

future land-use conditions. Table 5.4 presents the same data for four of these basins subjected to the maximum density buildout (MDB) scenario.

Figure 5.2 is a routing schematic to be used when reviewing the HEC-1 output for Detention Basins 24, 25, and 26. Plate 3 can be used for all other sites.

As with Case 1, the MDB scenario uses the same basin geometry and outflow rates as the future land-use condition case (Table 5.3). Table 5.4 merely shows the increase in inflow, reservoir water surface elevation, maximum storage, and surface area that would result from the MDB land-use assumptions.

The data in Table 5.3 should be used to establish an estimate of the upper envelope of detention basin sizes. The outflows resulting from the 36-hour drawdown assumptions are so small that downstream flooding would be essentially eliminated. However, as previously stated, the assumptions used to generate these small outflows do not make any provisions for dealing with the issue of reduced downstream sediment supplies due to sediment trapping within the basins. In order to pass sufficient sediment to mitigate this problem, the basin outflows would undoubtedly have to be much larger than those shown in Table 5.3. The degree of increase could only be determined through a detailed, site-specific sediment transport study. The actual design of a detention basin would have to evaluate the magnitude of downstream degradation and assess the consequences of accepting the degradation (along with the associated reduction in flood peaks) versus the impacts of reducing degradation by allowing larger water and sediment flows to leave the basins.

As a point of interest to those who may question the magnitude of reservoir induced stream bed degradation, it has been reported (Chow, Handbook of Applied Hydrology, 1964, page 17-6) that 14 feet of degradation occurred between 1935 and 1949 at a site on the Colorado River located 12 miles downstream of Hoover Dam.

It should be noted that neither the detention basins nor the retention basins evaluated in the report include allowances for sediment storage volume. Should any of these basin concept ever be pursued for actual design and construction, requirements for sediment storage should be considered. The Corps of Engineers (Phoenix, Arizona and Vicinity (including New River), Design Memorandum No. 2, Hydrology Part 2, 1982) has already assembled an excellent data base that could be used for such an analysis. Based on a 47-year record (1923-1970) of sediment deposition behind Cave Creek Dam (which is adjacent to the west side of the North Scottsdale drainage area boundary), the Corps computed an average sediment production rate of 0.24 acre-feet per square mile per year. Since this record did not include the September 1970 flood, the Corps increased this rate to 0.30 acre-feet per square mile per year for use in the design of several major flood control dams in the metropolitan Phoenix area. The Corps also notes that this value compares favorably with a 0.36 acre-feet per square mile per year measurement taken at Waddell Dam.

Nearly all the detention basin sites listed in Table 5.3 were field-inspected as part of this study. Only two of these sites were considered to have sufficient topographic relief to provide natural detention storage without the need for manmade embankments. Accordingly, if regional detention basins are pursued for this watershed, there is a high probability that raised embankments will be required along

the perimeter of the reservoir areas. However, such embankments could be easily contoured and landscaped to merge with the natural aesthetics of the area.

5.3 CASE 3 - CHANNELIZATION

As indicated in the preceding section, the lower portions of the alluvial fan terrace are not well-suited to detention basin construction because of relatively flat topography. For this area, an alternative solution might be the construction of large channels to intercept floodwater and carry them to a safe outlet, which for this project would be the large basins created by the CAP dikes. The large interceptor channels would also serve as outlets for local drainage systems that might accompany urbanization along either side of the channel alignments. For those channels that would intercept and divert flow away from a natural downstream path (Scottsdale and Pima Road channels), low-flow outlets would be installed in the interceptor channels to allow some portion of these flows to continue in their natural, southwesterly flowing watercourse. This will help maintain the natural vegetation complex along these washes.

Case 3 evaluates this concept in the form of major channels running north-south along the east side of Scottsdale Road and Pima Road. An approximation of the channel alignments is illustrated on Plate 3, except in the vicinity of Pima and Pinnacle Peak Roads, which is depicted on Figure 5.3. A third major trunk channel was also considered along a natural drainage corridor located south of Pinnacle Peak Road and approximately 1.5 miles east of Pima Road. On Plate 3, this channel alignment is shown passing through Sub-basins 2010, 2015, 2030, 2090, and 2110. This third major channel would discharge to the CAP basin located near "Horse World".

As can be seen from reviewing Plate 3, the Scottsdale Road channel actually consists of two channels ("A" and "B") which have a separation gap at Jomax Road. This break in the Scottsdale Road channel was made in order to avoid diverting un-natural drainage into the multiple CAP flood control basins that exist through this area. All channel alignments evaluated under Case 3 were designed to discharge their intercepted floodwater back into the same CAP basin that the water would reach under natural conditions. One notable exception to this criteria is the northern part of the Pima Road channel, which will divert runoff from Sub-basin 30, and portions of 34 and 36A, to the CAP basin east of Scottsdale Road, rather than to the natural outlet on the west side of Scottsdale Road.

It should be emphasized that the channel alignments shown along Scottsdale and Pima Roads are only conceptual, i.e., they are shown as straight lines on the Plate 3 routing schematic, when in fact they may be constructed on some type of curvalinear alignment that would be more aesthetically appealing. Such alignment revisions would not have any significant impact on the hydrology data presented in this report. Outlet locations for all channels should also be considered conceptual at this preliminary level of analysis.

The simulation of low-flow outlets, for cross-drainage that is intercepted by the channels, is also provided at a concept level by the periodic inclusion of divert routines in the HEC-1 routing schematic along Scottsdale and Pima Roads.

The lower half of channel "A" is shown on a curvalinear alignment. This lower segment of channel "A" is actually beyond the study limits and is only shown on Plate 3 for

continuity purposes to indicate the general direction that channel "A" will outlet. The HEC-1 model provides peak discharge data for this channel as far south as Pinnacle Peak Road. This channel would be proposed as a major trunk conduit that would accept outflows from smaller collector systems draining urban areas on either side of this alignment.

The channel along Pima Road is one continuous alignment which would tentatively outlet to the proposed detention basin (DB 28) along the north side of the Outer Loop Highway, in Sub-basin 53. (Note: The reader is cautioned that Plate 3 must be supplemented with Figure 5.3 when reviewing the Pima Road channel alignment; otherwise a misperception may occur that a break exists in this alignment north of Pinnacle Peak Road). Should this detention basin not be constructed, the Pima Road channel would simply be passed under the Outer Loop and discharged directly to the CAP detention basin located between Scottsdale Road and Pima Road. As shown on Figure 5.3, the Pima Road channel would also accept flows from a small interceptor channel proposed along a portion of the north side of Pinnacle Peak Road.

The third major channel alignment ("F") is located approximately 1.5 miles east of Pima Road and about 1 mile below the alluvial fan apex at the east end of Pinnacle Peak Road. This alignment follows a natural drainage swale with well-defined channel geometry through the upper portions of its length. However, the geometry through the lower 2 miles of the alignment is poorly defined and would most likely promote a sheetflow pattern. Even the incised channel geometry through the upper reaches does not have sufficient capacity to convey the 100-year flood without causing substantial overbank flooding (see hydraulic calculations in Table 4.4

for XSEC 10, 11, 12 and 13). As part of the General Drainage Plan, it would be recommended that this alignment ("F") be reserved as a natural drainage corridor and that detailed floodplain mapping be pursued along the entire alignment. This would identify actual floodplain limits, initiation of sheetflow conditions, etc.. Such information could be used to identify those reaches that might be left as natural floodplains (with no urban encroachments) versus those reaches where channel or levee improvements might be constructed to reduce the natural floodplain size to accommodate more development.

A detailed hydraulic analysis of channel "F" will certainly be complicated by the existence of the active alluvial fan apex at the east end of Pinnacle Peak Road. This complication arises from the fact that there is no way to predict the amount of flow that will exit this apex in the direction of channel "F". Unless some type of structure is constructed at the apex to control the direction and magnitude of this flow, the conservative approach would be to assume that 100% of the discharge at the fan apex would be directed to channel "F".

A control structure at the fan apex (i.e., detention basin or channelization) would be recommended as part of any drainage plan for this region. In the absence of such a control, <u>any</u> drainage or flood control system improvements downstream of the apex would have to assume a worst-case scenario of accepting 100% of the apex discharge. This could lead to excessive design costs that could be eliminated if a coordinated effort were made by downstream developers to address the problem at the fan apex.

The technical analysis of the channel alignments presented under Case 3 primarily focused on developing a HEC-1 routing schematic that would properly reflect the floodwater discharges that would be intercepted by each channel. These discharges were combined with one set of channel geometries to investigate approximate flow depths and velocities. Natural ground slopes from the USGS quadrangle maps were computed for each channel alignment and used in the hydraulic calculations. A Manning's roughness value of 0.025 and channel side-slopes of 2H:1V were also used in the calculations. Depending on the discharge at a specific location, the channel bottomwidths were varied from 30 to 100 feet.

The results of these preliminary hydraulic calculations are presented for future conditions in Tables 5.5, 5.6, and 5.7, while Tables 5.8, 5.9, and 5.10 present the same data for the maximum buildout scenario. For comparative purposes, these tables contain peak discharge and hydraulic data for not only Case 3, but also for the channel options that were proposed as part of Case 1 (on-site retention plus channelization) and Case 4 (regional detention basins plus channelization). The hydraulic data is summarized at identical channel locations for all three cases. No changes were made in the channel geometry from one case to the next. Accordingly, direct comparisons can be made on the changes in depth and velocity that accompany the variation in peak discharge generated by the three cases.

A review of Tables 5.5 through 5.10 indicates extremely high velocities through the majority of the channel sections. Such high velocities will mandate armored banks to prevent lateral erosion. Drop structures might also be considered as a means of decreasing the channel slopes in an effort to lower the flow velocity. However, it is very unlikely that

totally unlined banks would be a practical alternative for this environment. If bank-lining were not pursued, large set-backs would be required along each side of the channels in order to provide a buffer zone against bank erosion. Such an alternative would probably require considerable maintenance. The actual design of these channels would require an evaluation of different geometries that could carry the required discharges within an acceptable velocity envelope and available right-of-way width.

Soil-cement would be recommended as a viable alternative for bank stabilization. This product is manufactured on-site from local soils and creates a channel bank that has a very natural "earthy" color. This product has a proven history of outstanding performance under extremely adverse conditions.

Loose rock riprap is considered to be unstable for this high velocity environment. The rock sizes that would be required to provide theoretical stability would probably require D_{50} diameters in excess of 5 feet. Based on field data obtained from an inspection of known riprap failures, a D_{50} of 7.4 feet is recommended for an average channel velocity of 15 fps. (Source: USGS Water Resources Investigations Report 86-4128, Rock Riprap Design For Protection of Stream Channels Near Highway Structures, Volume 2 - Evaluation of Riprap Design Procedures, page 89, 1986).

Grass-lined channels would also be an unfavorable alternative for a natural desert environment. In addition to requiring extensive watering and some type of irrigation system, these channels would only be stable to maximum velocities in the 6 to 8 fps range. From an aesthetics standpoint, grass-lined channels would also present a very un-natural appearance in a desert landscape.

Regardless of the type of bank protection that might be selected (excluding grass) for the proposed channels, the product will have to be extended below the channel invert to prevent the banks from being undermined by various scour processes. These "toe-down" dimensions for the selected bank protection should consider vertical bed-movement due to the following phenomena:

1. Long-term aggradation/degradation due to changes in the watershed sediment supply to the channels.

2. Short-term general scour/deposition that will occur in response to changes in channel geometry.

3. Local scour due to any channel obstructions, such as bridge piers.

4. Bend scour due to secondary currents induced by channel curvature.

5. Low-flow incisement.

6. Troughs created in the channel bottom due to the formation of bed-forms such as antidunes.

In the absence of any bridge piers or sharp channel curvature, antidune troughs would probably be one of the larger scour components to consider in the channel design. For a typical velocity of 15 fps, these troughs can extend as much as 3 feet below the channel invert; for a velocity of 20 fps, these troughs could extend to a depth of approximately 5.4 feet. Based on experience with similar projects, total scour depths (excluding pier scour and long-term degradation) of 3 to 6 feet could probably be anticipated through these channels.

The concept of channelization could realistically be pursued throughout the watershed. However, because of aesthetic concerns, an attempt has been made to limit its application to those areas where other solutions are not well suited. Channels will provide very efficient mechanisms for removing floodwater from the study area. Their aesthetic qualities could be enhanced by using products such as soil-cement and the use of natural vegetation to screen the channels. Vegetated, benched cross-sections might also be considered as an alternative to a pure trapezoidal section. However, such variations will undoubtedly increase the construction cost and right-of-way requirements associated with the channels.

5.4 CASE 4 - CHANNELIZATION AND DETENTION BASINS

Case 4 is a combination of Case 2 detention basins and the Case 3 channelization scheme. This is a logical combination of alternatives to evaluate since the detention basin concept was not well-suited to the lower portion of the alluvial fan terrace where the channelization scheme was employed.

The lower portions of the fan terrace are sufficiently removed from the detention basin sites so that the uncontrolled, intervening drainage areas begin to become large enough to generate peak discharge values that are potentially damaging. Installing regional detention basins in the upper portions of the watershed and channelization in the lower areas of the watershed creates a balanced flood control system that is capable of protecting the entire watershed. Savings in channelization costs are also realized since the upstream detention basins are capable of significantly reducing the peak discharges that have to be carried by the channels.

Technical details of the two concepts comprising Case 4 have been previously described in Section 5.2 (Detention Basins) and Section 5.3 (Channelization). The HEC-1 routing schematics for Case 4 are identical to Case 1 (see Plate 3), the only difference being that on-site retention basins have been replaced by regional detention basins.

The detention basin performance characteristics for Case 4 are identical to those previously presented in Tables 5.3 and 5.4. The impact of these basins on downstream channel hydraulics is presented in Tables 5.5 through 5.10.

Again, it should be emphasized that the 36-hour drawdown assumptions associated with these basins are probably creating outlet discharges that are lower than what would be required to pass sufficient bed-material loads to prevent downstream degradation. Accordingly, the channel discharge values presented for Case 4 in Tables 5.5 through 5.10 may be unrealistically low.

5.5 STATE TRUST LANDS SOUTH OF OUTER LOOP

A unique segment of the study area is a block of State Trust Land bounded by Scottsdale Road, Pima Road, the proposed Outer Loop, and the CAP. This area has already undergone extensive land planning and incorporates an elaborate system of interior drainage channels.

This interior drainage system is dependent upon the construction of two large detention basins (DB 27 and DB 28, see Plate 3) along the north side of the proposed alignment for the Outer Loop Highway. Details of this proposed drainage system, and detention basin design, are presented in

a report entitled Drainage Analysis For State Trust Lands, Outer Loop Highway, Scottsdale Road To Pima Road, Simons, Li & Associates, Inc., June, 1987.

The hydrology analysis for DB 27 and DB 28 was based on existing watershed conditions with no upstream flood control improvements. Accordingly, should any of the drainage improvements evaluated for the General Drainage Plan for the north Scottsdale area ever be constructed, there is a high probability that the size of DB 27 and DB 28 could be reduced.

The HEC-1 modeling structure for the Scottsdale General Drainage Plan is completely compatible with that developed for the State Trust Lands. As a result, those models could easily be linked together to examine the impact of any upstream flood control improvements. However, it should be noted that the Scottsdale General Drainage Plan incorporates refinements to the HEC-1 model used for the drainage area above DB 27 and DB 28. Different rainfall distributions are also used in the two models. Any attempts to link these models together should be coordinated with Mr. Robert L. Ward, P.E., who developed the HEC-1 models for both the State Trust Lands and the Scottsdale General Drainage Plan.

6.0 GROUND-WATER RECHARGE

6.1 BACKGROUND

In June, 1987, the City of Scottsdale received a report from Boyle Engineering entitled "Utilization of Groundwater Reservoir under City of Scottsdale". In that report they addressed the topic of artificial ground-water recharge using excess water from the CAP aqueduct. As part of the study, conceptual locations were presented for initial recharge drywells.

Water Resources Associates has completed its study on the drainage system currently developed on the alluvial fans in northern Scottsdale. A purpose of this study is to better define the drainage basins and calculate the volumes of water that could be generated as storm water runoff. Knowledge of these values can aid the City of Scottsdale in planning future community development through directing flood control measured thereby providing significant quantities of water than can be used for artificial recharge.

Water Resources Associates has used the cases presented in Section 5.0 of this report to develop several preliminary scenarios for ground-water recharge. Four scenarios are presented here which appear to be the most reasonable for the hydrogeologic environment and future land use. Scenarios considered in this report are:

1. Channelization and on-site retention;

2. Detention;

3. Channelization and

4. Channelization and Detention Basins.

6.2 CHANNELIZATION AND ON-SITE RETENTION

In this scenario, individual development areas would be required to design and install retention basins within the development for 50% of the volume of storm water runoff generated from a 2-hour, 100-year event. Each basin would require one or more monitoring wells to document the amount of recharge. Recovery of the recharged water would be by production wells down-gradient from the basin. Runoff not captured by the retention basins would be carried away either by concrete, soil cemented, or earth channels and directed toward the detention basins already existing north of the CAP aqueduct.

This scenario can provide good documentation for ground-water credits since the volumes of water can be well documented. Anticipated volumes of runoff are presented in Table 6.1 and early recoverable through existing and future municipal water wells. Problems with this scenario are centered around several issues. Development land is very expensive in North Scottsdale. Some developments may be required to have more than one large basin, or the area of runoff capture may be such that large dikes will need to be constructed. Water not retained in basins would be carried off by a system of canals to the detention basins north of the CAP aqueduct.

The construction of the channels can be of concrete, soil cemented or earth. Concrete and soil-cemented channels would deliver all storm water runoff to the CAP detention basins.

Earthen channels could be designed with sieve bottoms and sides to permit recharge along the entire length of each channel. Based on WRA's drainage investigation, the amount of basins needed for this scenario would create considerable aesthetic problems, and remove much land from development. Earthen channels would probably be more aesthetically pleasing than concrete or soil-cemented and therefore, more attractive to the community. Each component of this scenario will require detailed, site-specific geologic and geotechnical investigations to assure that subsurface conditions are such that recharge water can migrate, relatively unimpeded, to the ground-water table.

6.3 ALL DETENTION

The second scenario for storm water recharge is for all runoff to be contained and slowly discharged through detention basins. The drainage analysis compiled by WRA indicates that there are at lease 7 sites available for detention basin construction. Volumes of runoff into the detention basins are presented in Table 6.1. The water would be released from the detention basins and into natural drainageways. Documentation of actual amounts of ground-water recharge from detention basins would be difficult to obtain since water would be stored in the basin for only 36 hours.

6.4 ALL CHANNELIZATION

A third option for conveyance and recharge of storm water runoff is to direct the water into large channels which follow one or more of the major north-south roads in north Scottsdale. The channels would follow the gradient of the roads and deliver the runoff to the detention basins along the CAP aqueduct. The construction of the channels can be of

concrete, soil-cemented or earthen. The concrete-lined and soil-cemented channels would deliver nearly all runoff to the detention basins. The earthen channels could be sealed or designed as sieves. The sieve concept would permit recharge along the entire length of the channel. If the sieve design is selected, certain maintenance and geotechnical considerations will be required. Channel bottom maintenance may be needed to assure maximum percolation of the recharge water. Careful attention will be needed with regard to the subsurface conditions. Shallow changes in the geologic conditions, such as caliche horizons, or clay lenses can cause local water logging of the soils and should be avoided. Monitor wells will be required along the course of the recharge or sieve channels.

6.5 CHANNELIZATION AND DETENTION

The fourth scenario for recharge of storm water runoff is through the combination of the use of detention basins and channelization. In this instance, storm water can be captured in developments in the detention basins and slowly released into washes which drain into major north-south channels. The basins can act as sources of ground-water recharge during the periods when the detention basins contain and release runoff. The channels can be constructed as described above. Runoff that reaches the CAP detention basins can also be used for recharge credits.

6.6 DISCUSSION

BAsed on our mapping and calculations, it appears that storm water runoff is a viable source for artificial recharge. Regardless of the scenario or combination of scenarios the City of Scottsdale should be able to determine the amount of

recharge and receive credit for it. Furthermore, existing ground-water production wells should be adequate for recharge water recovery.

Due to economic considerations with regard to the cost of land and aesthetic reasons, the combination of detention basins and channelization seems to be the most reasonable recharge scenario. This system would create recharge basins in the detention areas located on the coarser-grained portion of the alluvial fan. Storm water runoff released from the detention basins at a constant rate would reduce scour in the natural washes and preserve existing vegetation patterns. The channels would be constructed in roadway easements thereby minimizing the cost of purchasing more land. If the channels were constructed of native materials they could be used as sieve-type recharge systems and also no significantly deter from the natural aesthetics in northern Scottsdale.

Regardless of the type of recharge technique selected for this area, detailed subsurface investigations will be required to confirm the presence or absence of geological materials that could reduce or restrict vertical movement of recharge water. Of primary concern will be the development of caliche at or near the surface, or if a buried caliche horizon is present at depth. Also, since clay content increases on alluvial fans with time, clay content and alluvial fan surface age must be considered when designing the recharge structure. Similarly, clay lenses or strata may be present in the subsurface. These aquacludes and/or aquatards can greatly affect the benefit of recharge water either in terms of water logging of the soils through lateral migration or loss by evapotranspiration, or both.

7.0 COST ESTIMATES FOR DRAINAGE IMPROVEMENT CONCEPTS

7.1 COST ESTIMATES

Construction cost estimates were developed for each of the design alternatives presented in this report. Since this is a concept report, based on preliminary information, a contingency factor of 25% was added to the computed construction costs. This factor is included for unknown problems and engineering costs that may be identified in the advanced stages of the drainage system design.

Unit costs used for developing the cost estimates were the following:

1) Rights-of-Way costs were estimated on the basis of $120,000 per acre.

2) Bridge costs were estimated on the basis of $45 per square foot of deck area.

3) Channel lining costs were estimated on the basis of $20 per square yard.

4) Excavation costs were estimated on the basis of $5 per cubic yard.

5) Rip Rap costs were estimated on the basis of $20 per cubic yard.

Table 7.1 summarizes the costs for each alternative presented in this report. Tables 7.2 through 7.5 present a detailed

itemization of the construction quantities and costs that were used to develop the cost estimates. Landscaping costs are not included in the estimates.

Bridge construction costs were estimated for each design alternative based on the required width and length of each bridge. Bridges were located at major cross-roads along Scottsdale Road and Pima Road. Bridges would also be required at the Outer Loop Highway crossings.

For the cost estimate, channels were assumed to be in "cut" throughout their length. velocities were limited to 20 feet per second during the 10-year event. Drop structures were located to prevent velocities from exceeding 20 feet per second.

Retention/Detention facilities were designed according to the typical section shown in Figure ..... Retention/ detention structures were designed with a minimum of three feet of freeboard.

Cost estimates are preliminary since, at this stage of development, the location of retention/detention facilities and channel alignments are subject to changes in the concept design.

8.0 CONCLUSIONS AND RECOMMENDATIONS

It is not within the scope of work of this project to recommend a specific flood control solution for the north Scottsdale area. Such solutions will be dependent upon: 1) the way in which development actually occurs; 2) public discussion and acceptance of specific solutions; 3) some type of funding plan; and 4) continued review of drainage requirements and aesthetic factors by the City of Scottsdale.

This report presents considerable technical data on four flood control concepts which could be pursued, enhanced, or otherwise modified to provide effective drainage solutions for the north Scottsdale area. Accordingly, this report should be used as a comprehensive data base to assist the City in arriving at a final solution to the drainage issues in this area. This process will undoubtedly take several years and considerable debate en route to the acceptance and construction of a final master drainage plan.

In addition to the detailed technical data that accompanies this report, several general conclusions are summarized as follows:

1. The study area presently contains regions of severe flood damage potential, primarily on the alluvial fan terrace south of Dynamite Road and downstream of the alluvial fan apex located at the east end of Pinnacle Peak Road. The outlet of Lost Dog Wash, in the vicinity of 120th and 124th Street (south of Cactus Road) is also a dangerous environment that is subject to severe flooding as water leaves an

entrenched alluvial fan channel and transitions into a braided flow pattern.

2. Future development in the project watershed must recognize the potential for widespread sheetflow across the alluvial fan terraces and aprons. Development in these areas should consider substantial elevation of building pads, armoring of raised building pads, and lateral setbacks from natural washes.

3. The forecast land-uses for this area will intensify the flooding potential if preventive action is not taken. The development of flood control solutions for the watershed should use an approach that considers impacts to adjacent and downstream properties. Failure to do so may simply transfer a flooding problem from one location to another.

4. Detention basins, retention basins, channels, and combinations thereof, are all effective solutions to the flooding problems that exist in the watershed. Since the construction costs of such measures do not appear to be significantly different, the selection of a specific structural solution may be largely influenced by right-of-way availability, aesthetic factors, or site-specific circumstances.

5. The HEC-1 models developed for this study are very comprehensive and consideration should be given to using the results from these models as a guide for the approval of hydrology data submitted for the development of specific parcels of land within the watershed.

6. The HEC-1 models used for the concept design analysis (Cases 1, 2, 3 and 4) provide valuable insight into the performance characteristics of several different types of flood control solutions. It is recommended that these models be used as the foundation for the development of a comprehensive Master Drainage Plan for the study area.

7. Cases presented in this report for flood control can also be considered as possible scenarios for ground-water recharge. Recharge will require extra cost for monitoring wells and geotechnical investigations. Recharge considerations revolve around the fact that ground-water recharge can be monitored and the City can receive credit for it, and recharge of the ground water can be implemented without extensive land purchases.

9.0 REFERENCES

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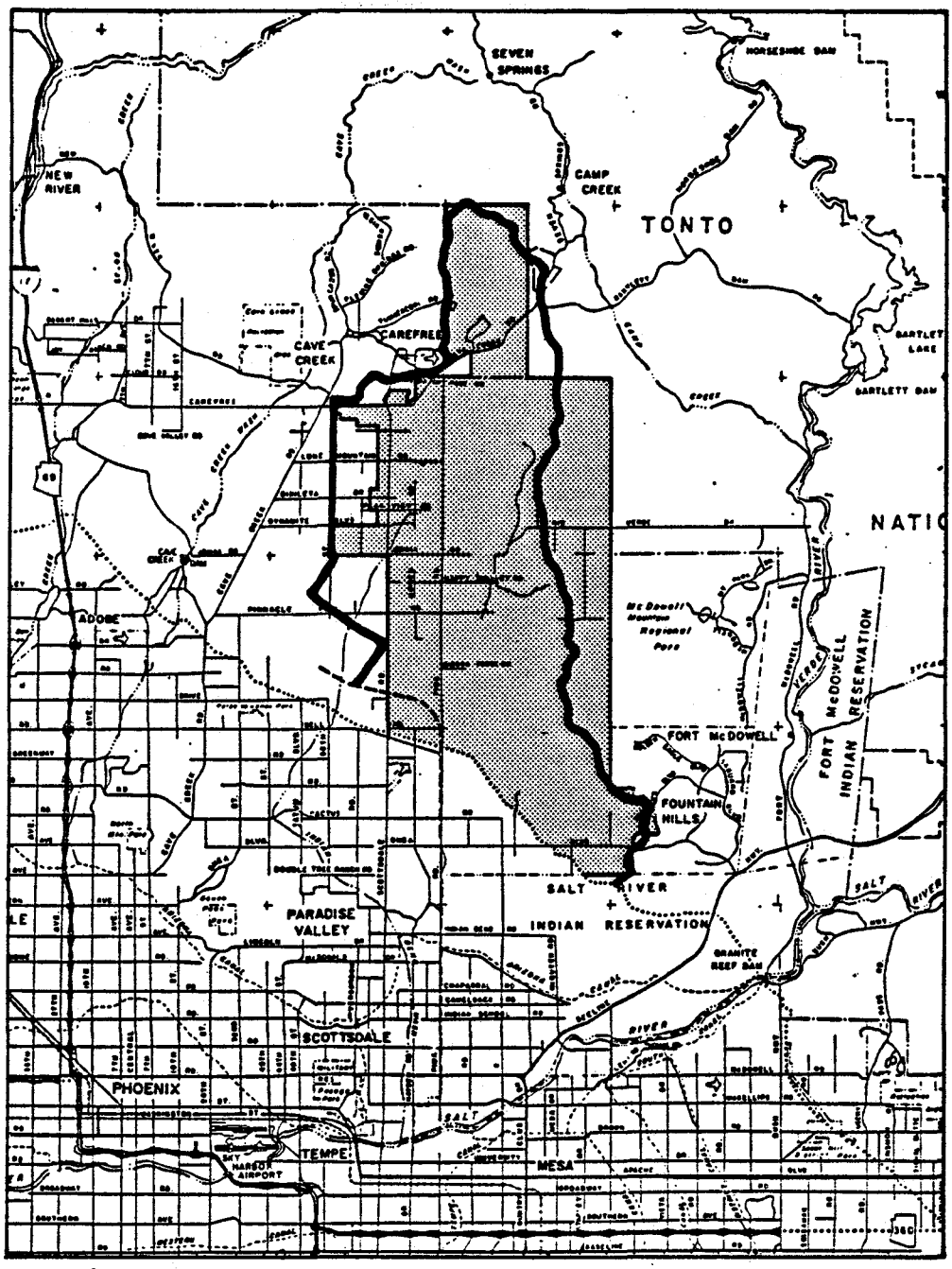
FIGURES



N

6 0 6 MILES

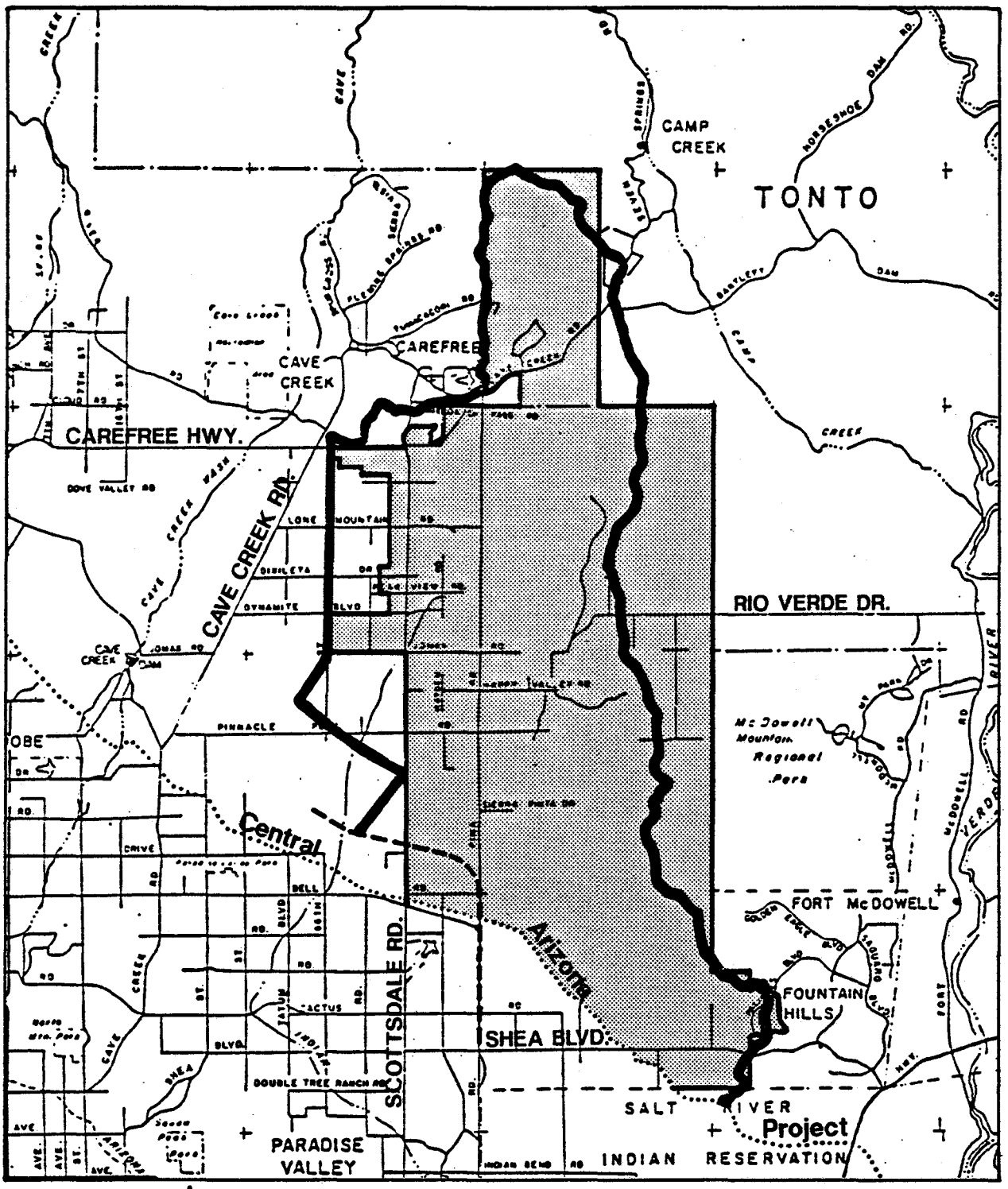
LEGEND

City Of Scottsdale (North Of C.A.P.)

Watershed Boundary

BASE MAP : Maricopa Co. Dept. Of Planning And Development

Water Resources Associates, Inc.	PROJECT LOCATION MAP	FIGURE 1.1
PROJECT NO 08701960 / DRAWN BY KMA		
DATE 10/3/88 / CHECKED BY JJH		



LEGEND

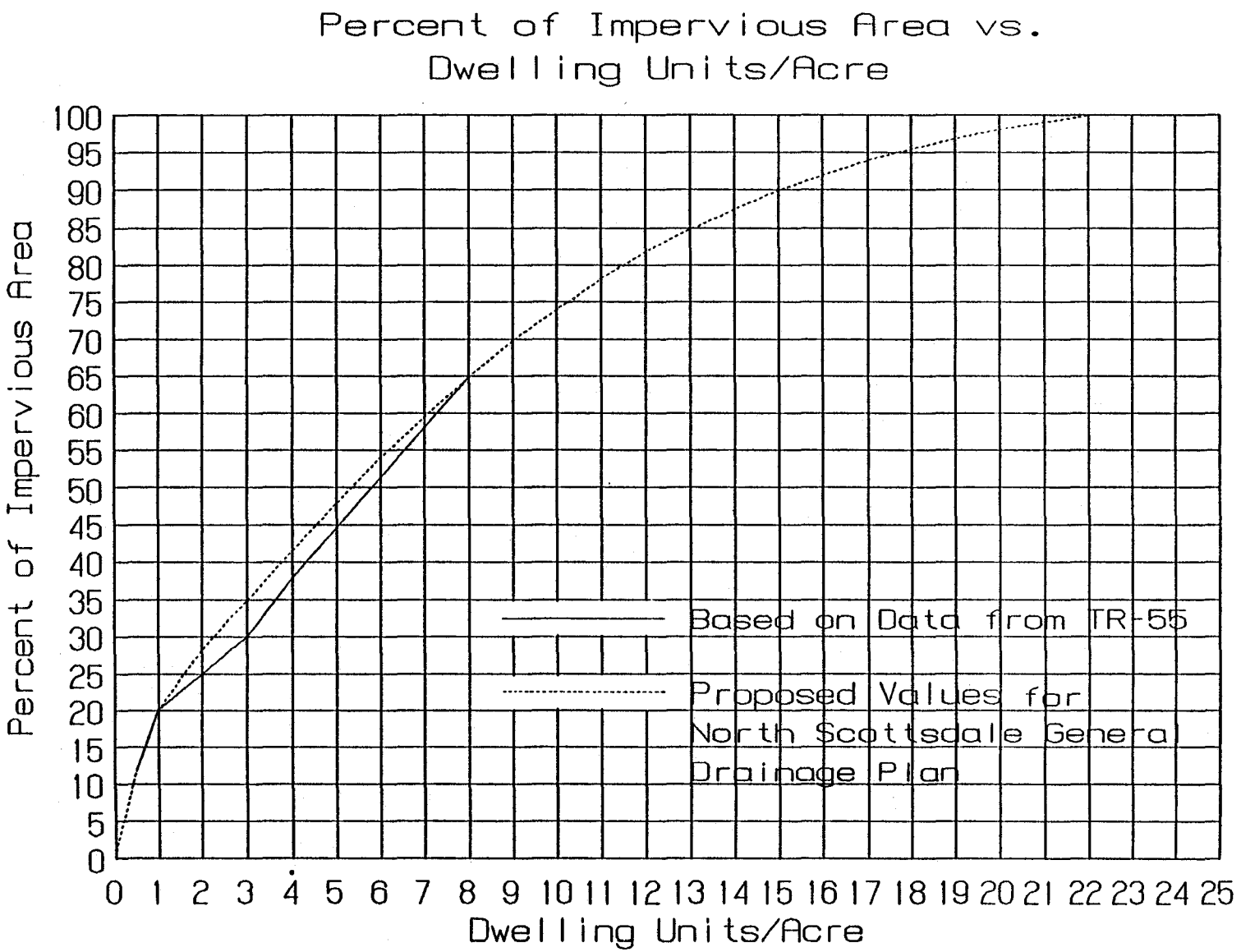
City Of Scottsdale (North Of C.A.P.)

Watershed Boundary

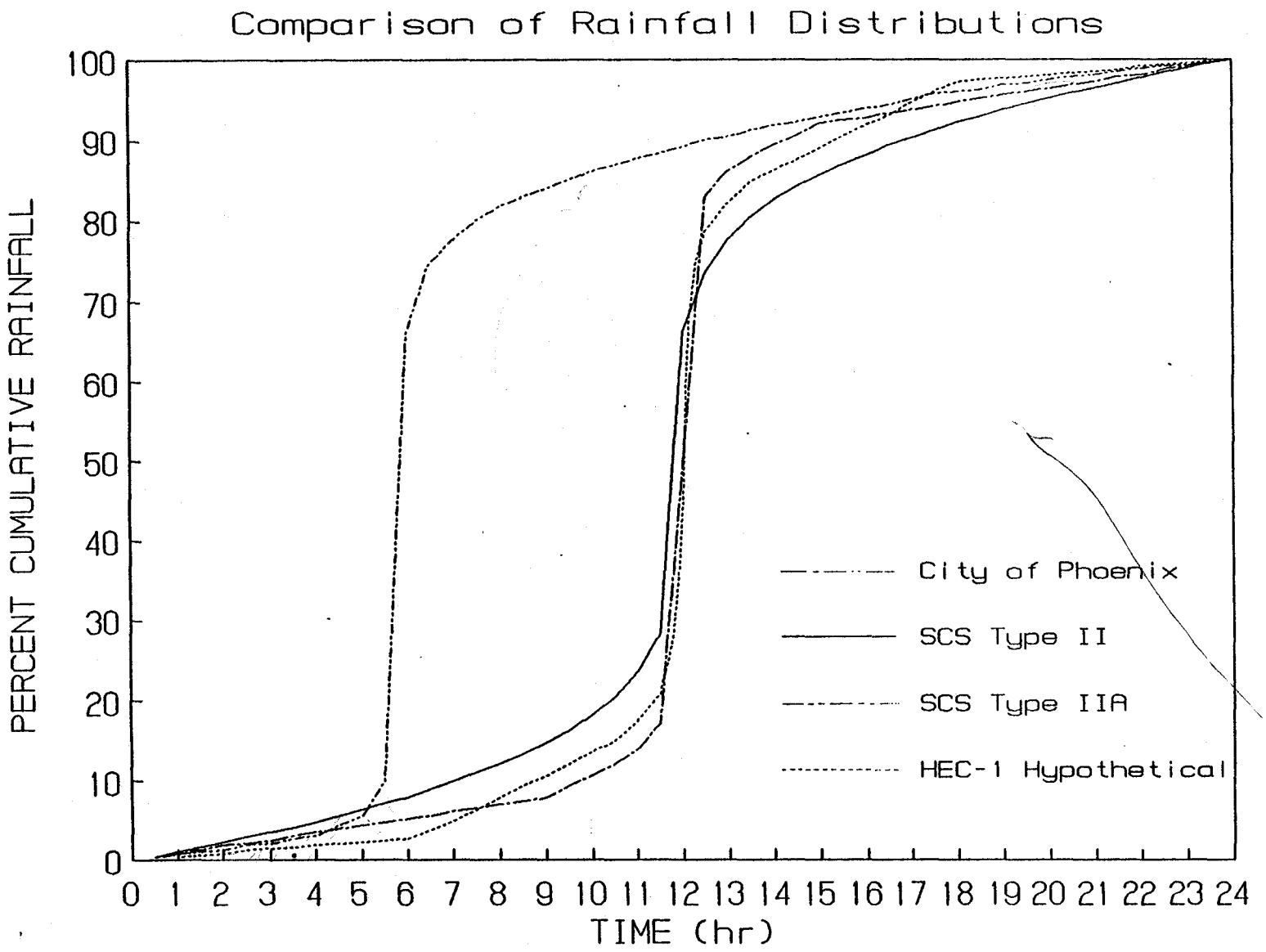
4 0 4 MILES

BASE MAP : Maricopa Co. Dept. Of Planning And Development

Water Resources Associates, Inc.	STUDY LIMITS AND WATERSHED BOUNDARIES	FIGURE 2.1
PROJECT NO. 08701960 DRAWN BY KMA		
DATE 10/3/88 CHECKED BY JJH		

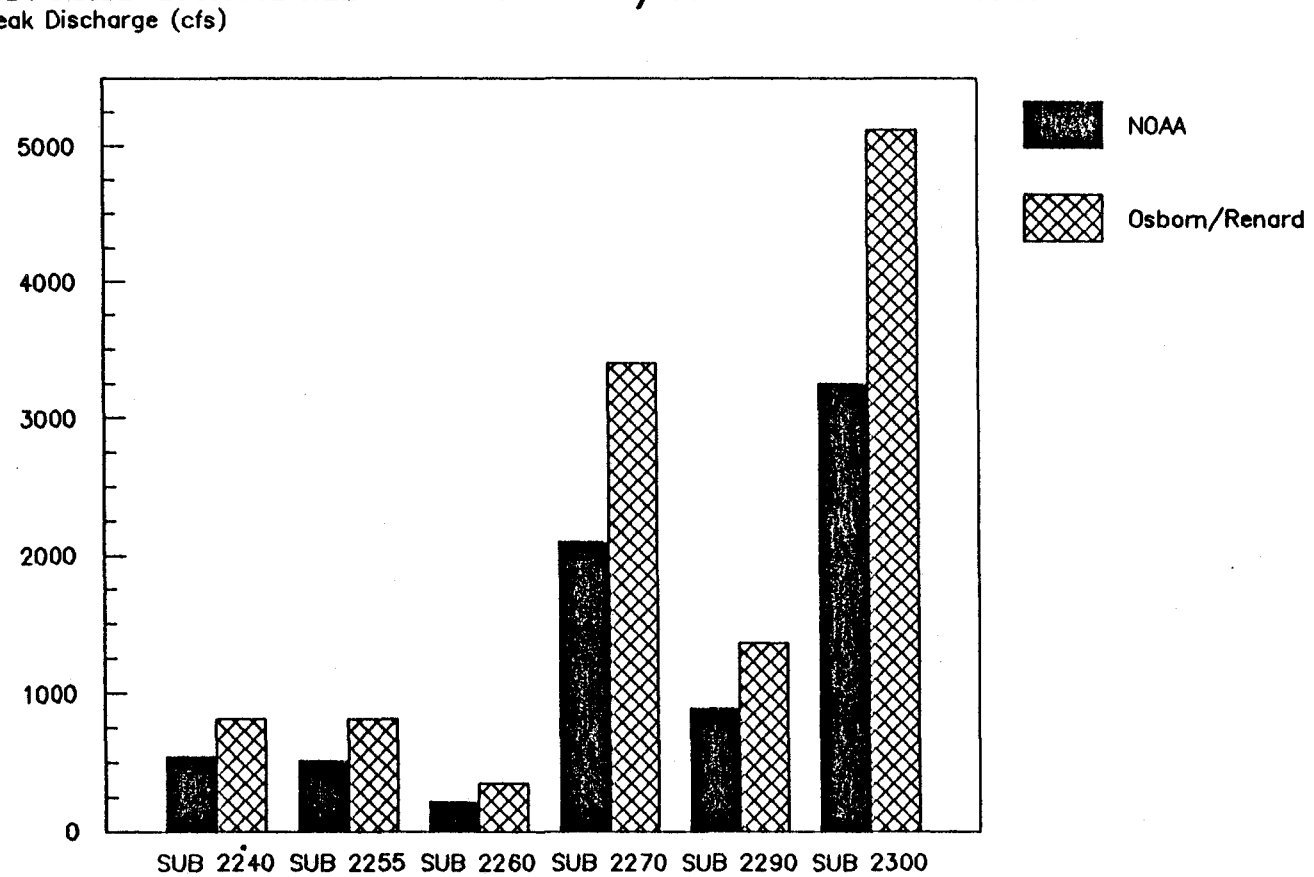


Water Resources Associates, Inc.	PERCENT OF IMPERVIOUS AREA VS. DWELLING UNITS/ACRE
PROJECT NO 08701960 DRAWN BY RLW	
DATE 9/2/88 CHECKED BY JJH	



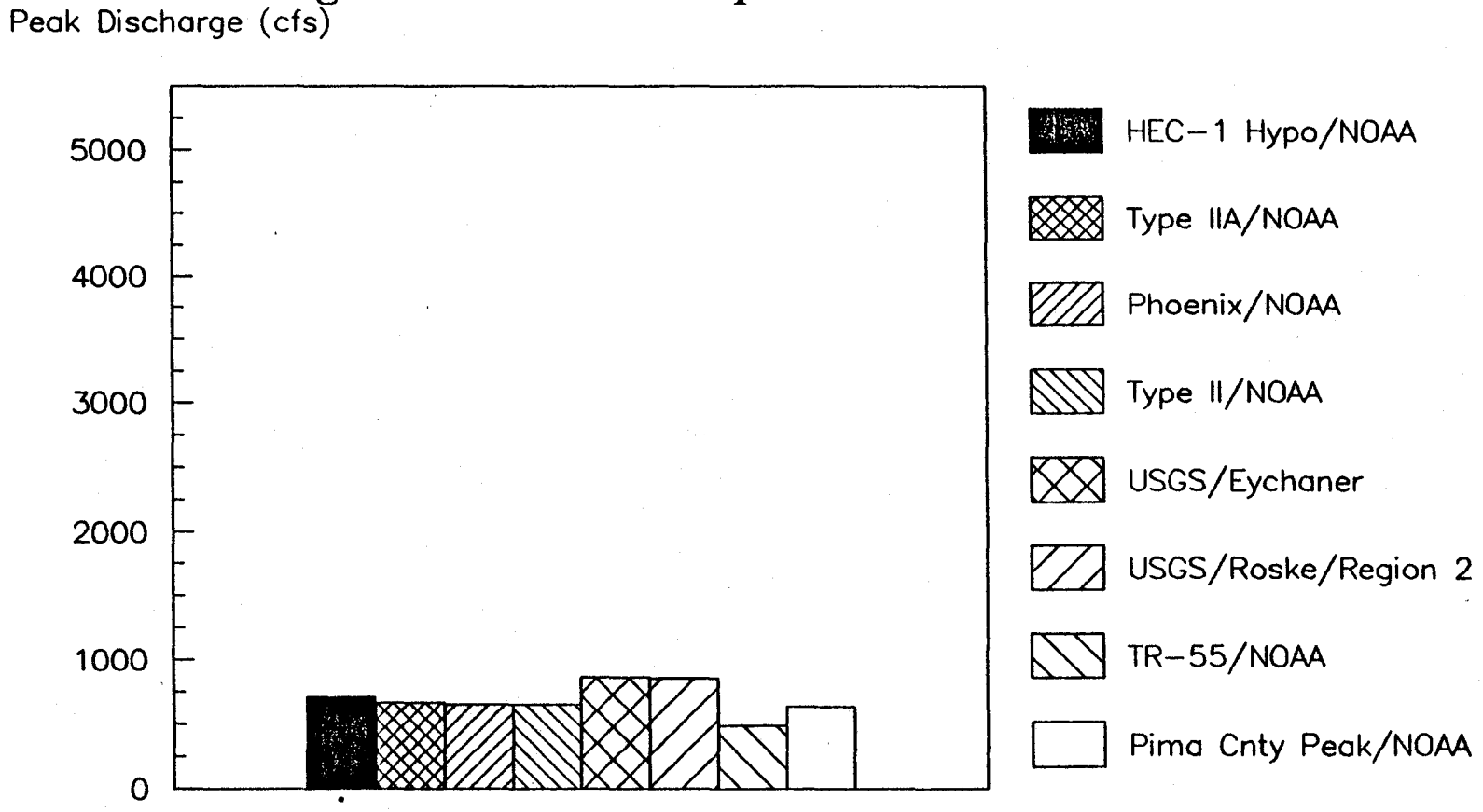
Water Resources Associates, Inc.		COMPARISON OF RAINFALL DISTRIBUTIONS
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	

Qp (cfs), 1-Hour HEC-1 Hypothetical Storm
NOAA Rainfall vs Osborn/Renard Rainfall



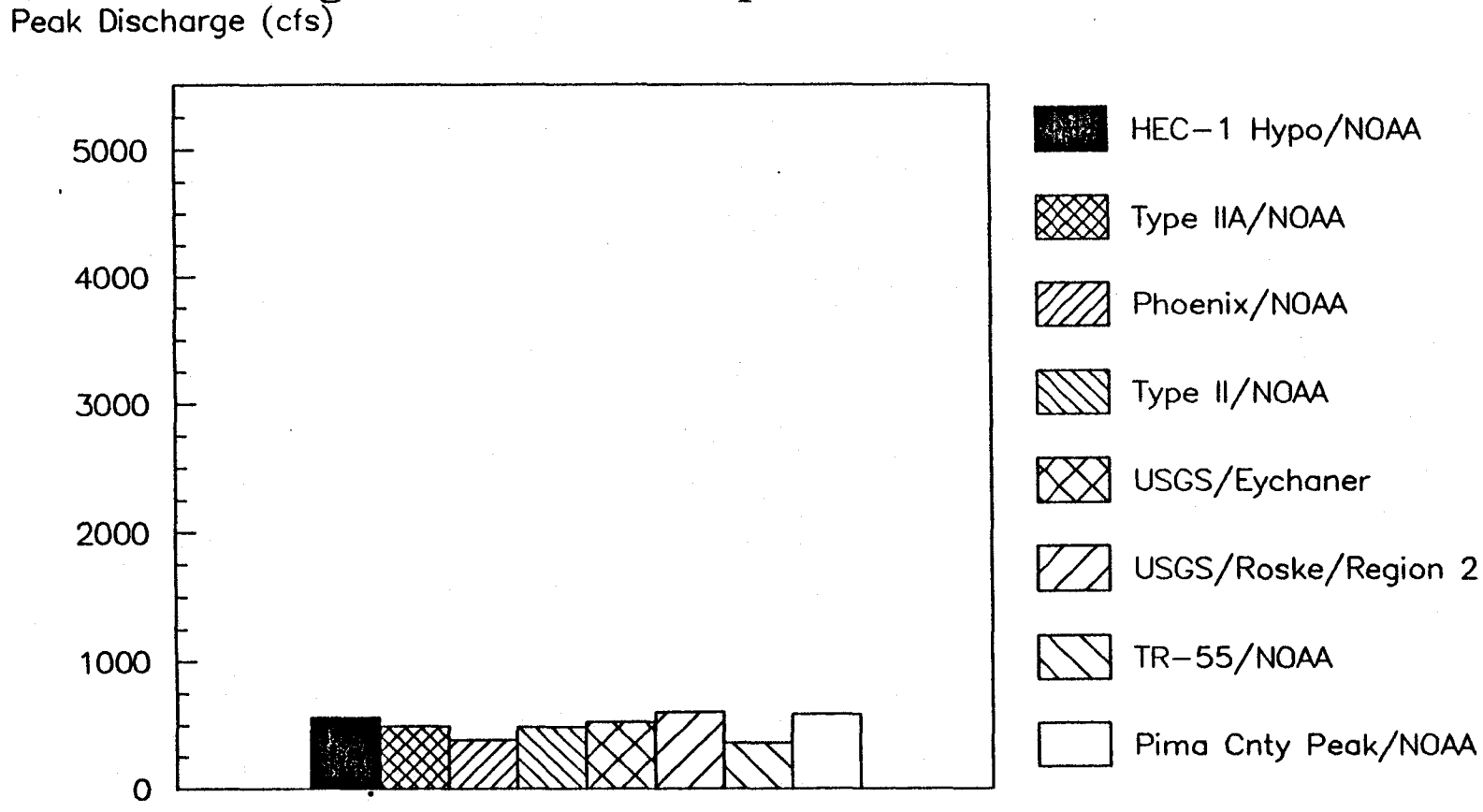
Water Resources Associates, Inc.		COMPARISON OF PEAK DISCHARGE FROM NOAA VS. OSBORN/RENARD RAINFALL
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	

Sub-Basin 2240, Comparison of Qp Calculations
Drainage Area = 0.60 Square Miles



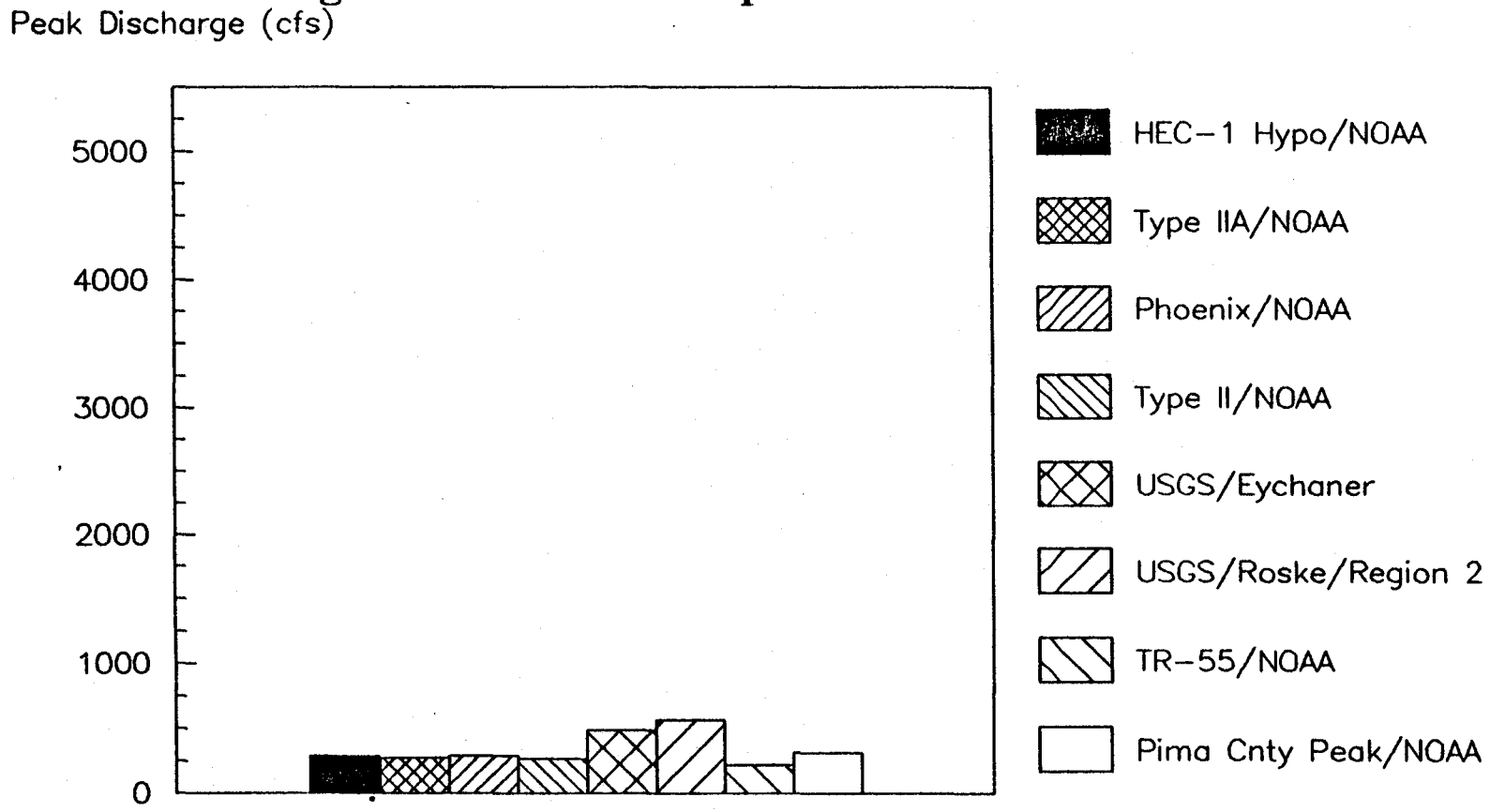
Water Resources Associates, Inc.		COMPARISON OF PEAK DISCHARGE CALCULATIONS SUB-BASIN 2240
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	

Sub-Basin 2255, Comparison of Qp Calculations
Drainage Area = 0.30 Square Miles



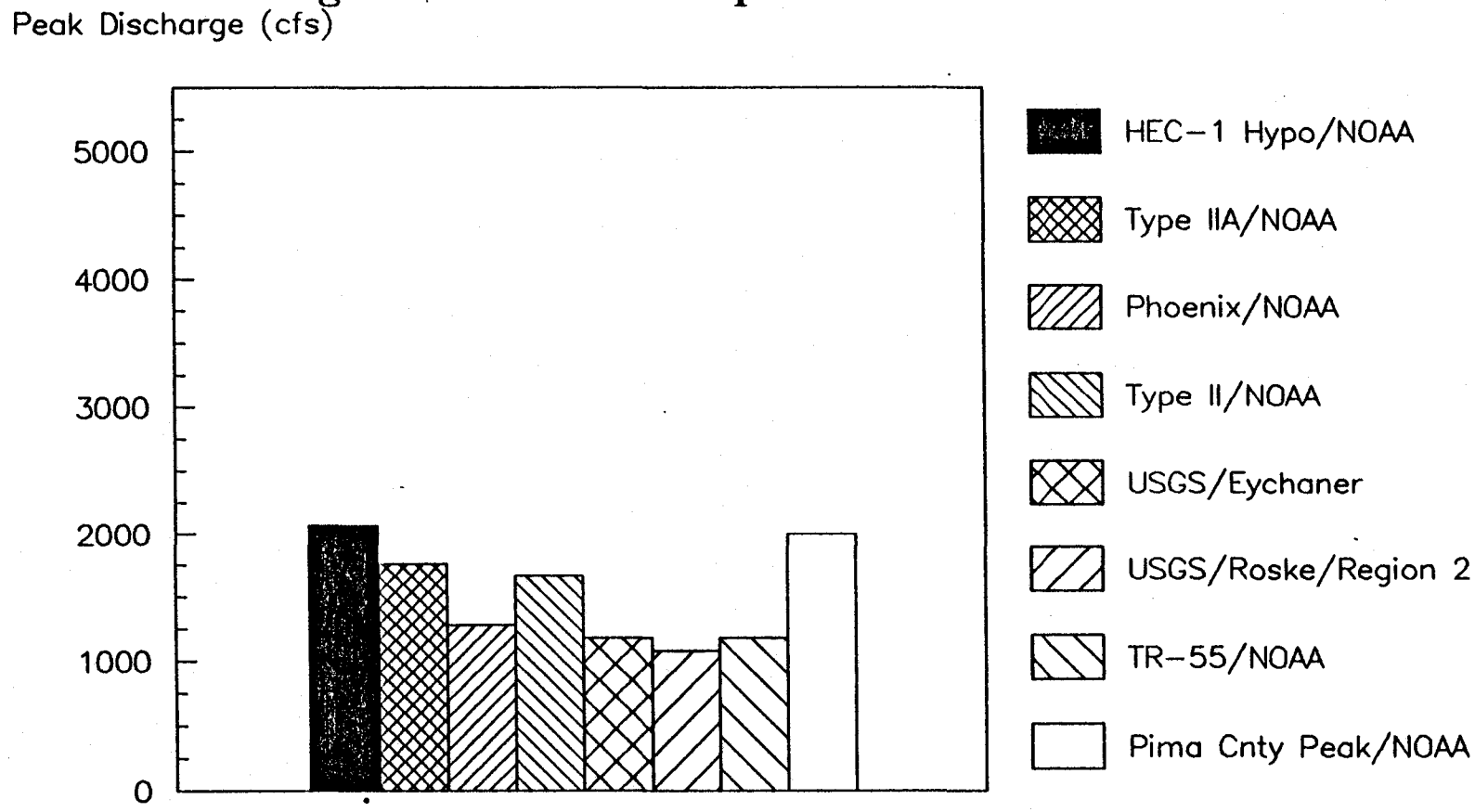
Water Resources Associates, Inc.		COMPARISON OF PEAK DISCHARGE CALCULATIONS SUB-BASIN 2255
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	

Sub-Basin 2260, Comparison of Qp Calculations
Drainage Area = 0.26 Square Miles



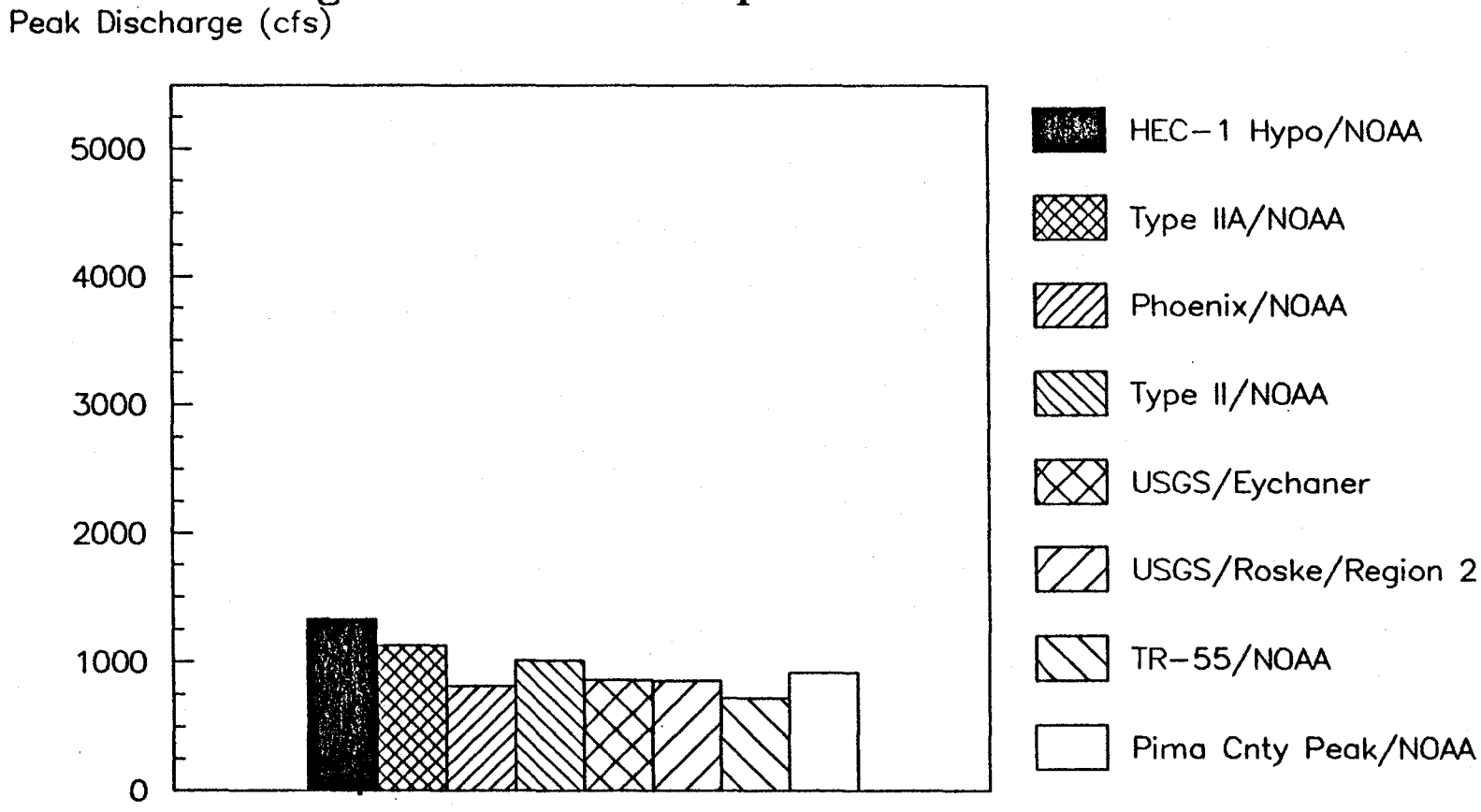
Water Resources Associates, Inc.		COMPARISON OF PEAK DISCHARGE CALCULATIONS SUB-BASIN 2260
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	

Sub-Basin 2270, Comparison of Qp Calculations
Drainage Area = 0.97 Square Miles



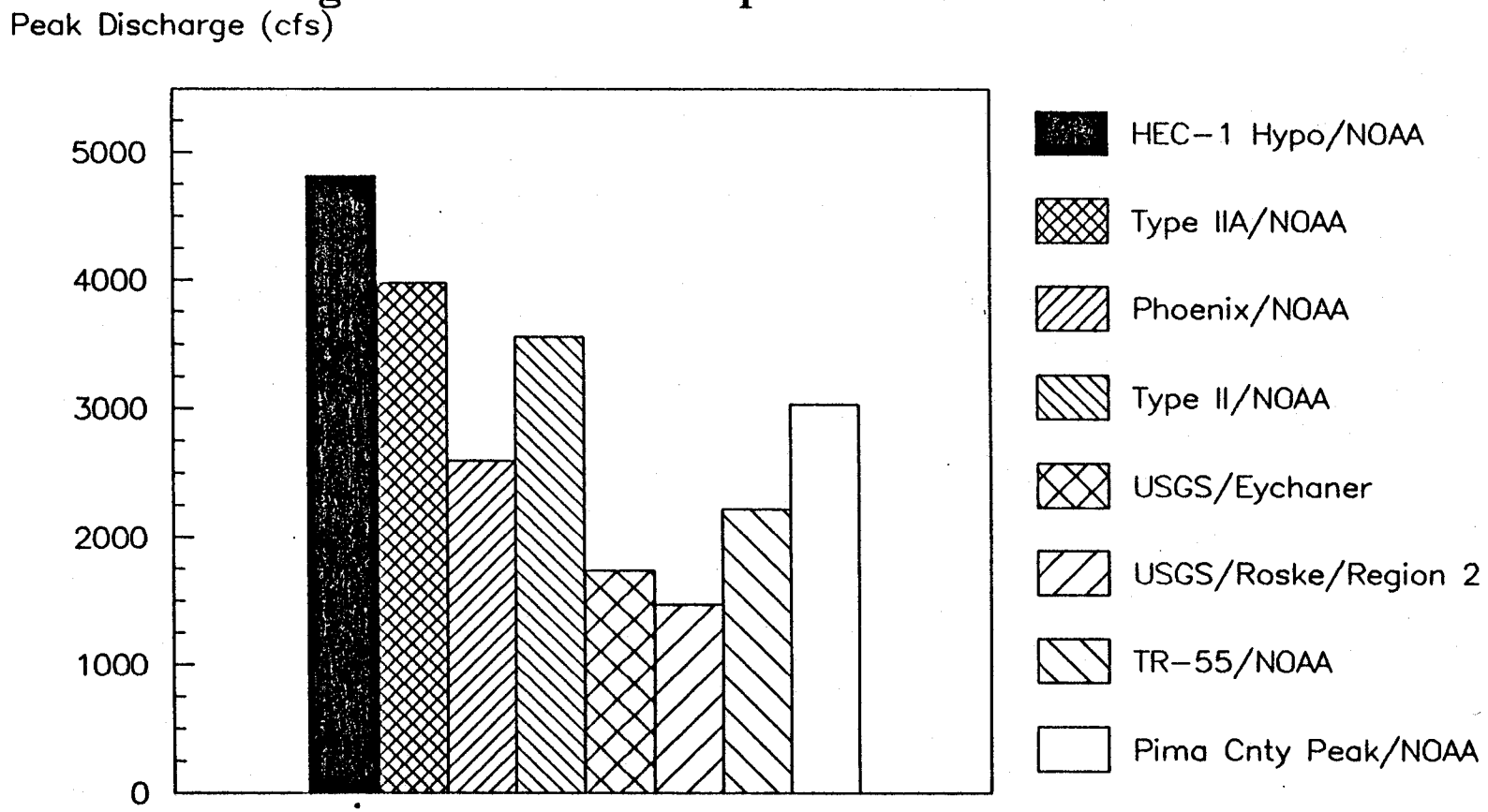
Water Resources Associates, Inc.		COMPARISON OF PEAK DISCHARGE CALCULATIONS SUB-BASIN 2270
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	

Sub-Basin 2290, Comparison of Qp Calculations
Drainage Area = 0.60 Square Miles



Water Resources Associates, Inc.		COMPARISON OF PEAK DISCHARGE CALCULATIONS SUB-BASIN 2290
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	

Sub-Basin 2300, Comparison of Qp Calculations
Drainage Area = 1.80 Square Miles



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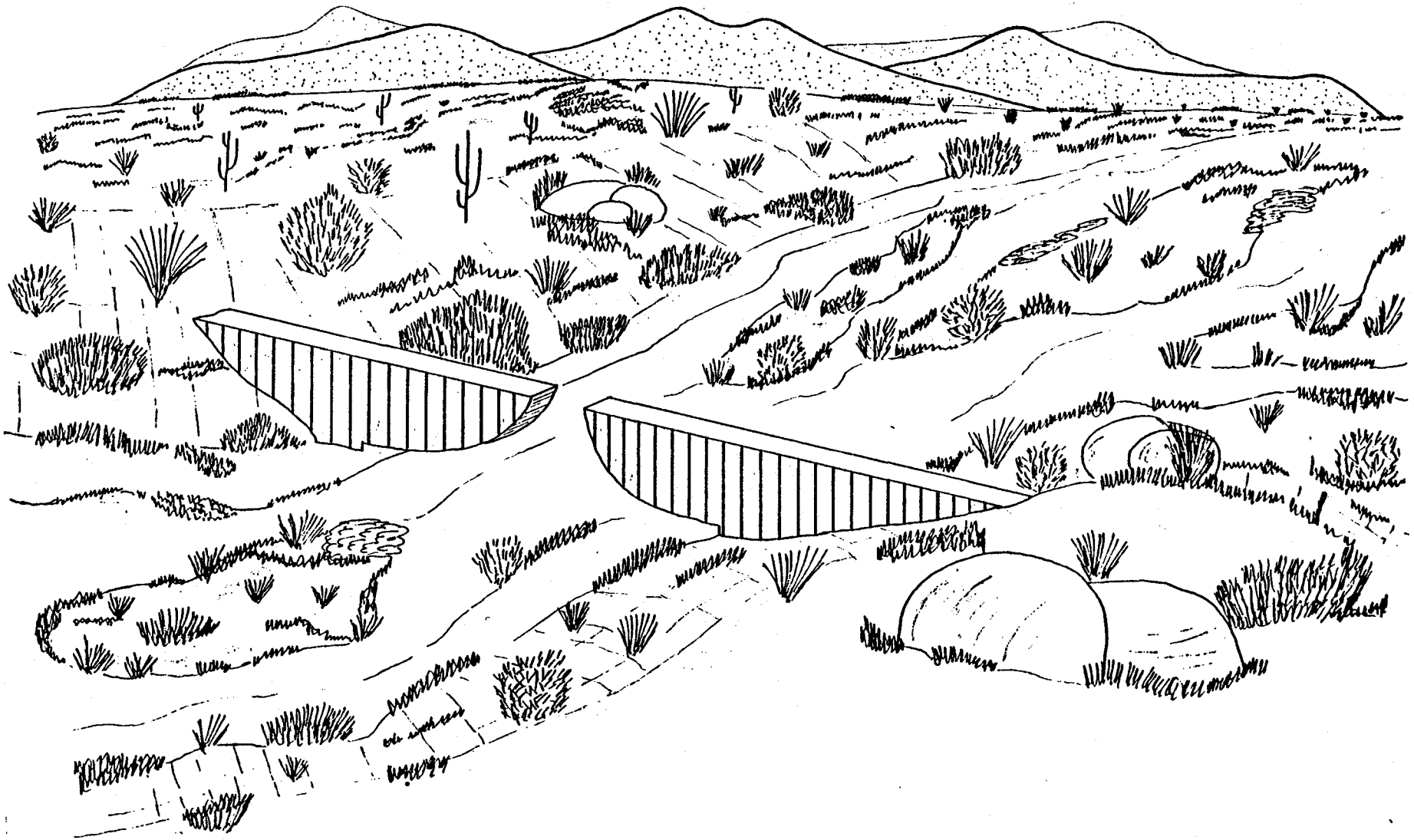
PROJECT NO 08701960	DRAWN BY RLW	
DATE 9/2/88	CHECKED BY JJH	



Water Resources Associates, Inc.		PROPORTIONAL WEIR DETENTION BASIN OUTLET
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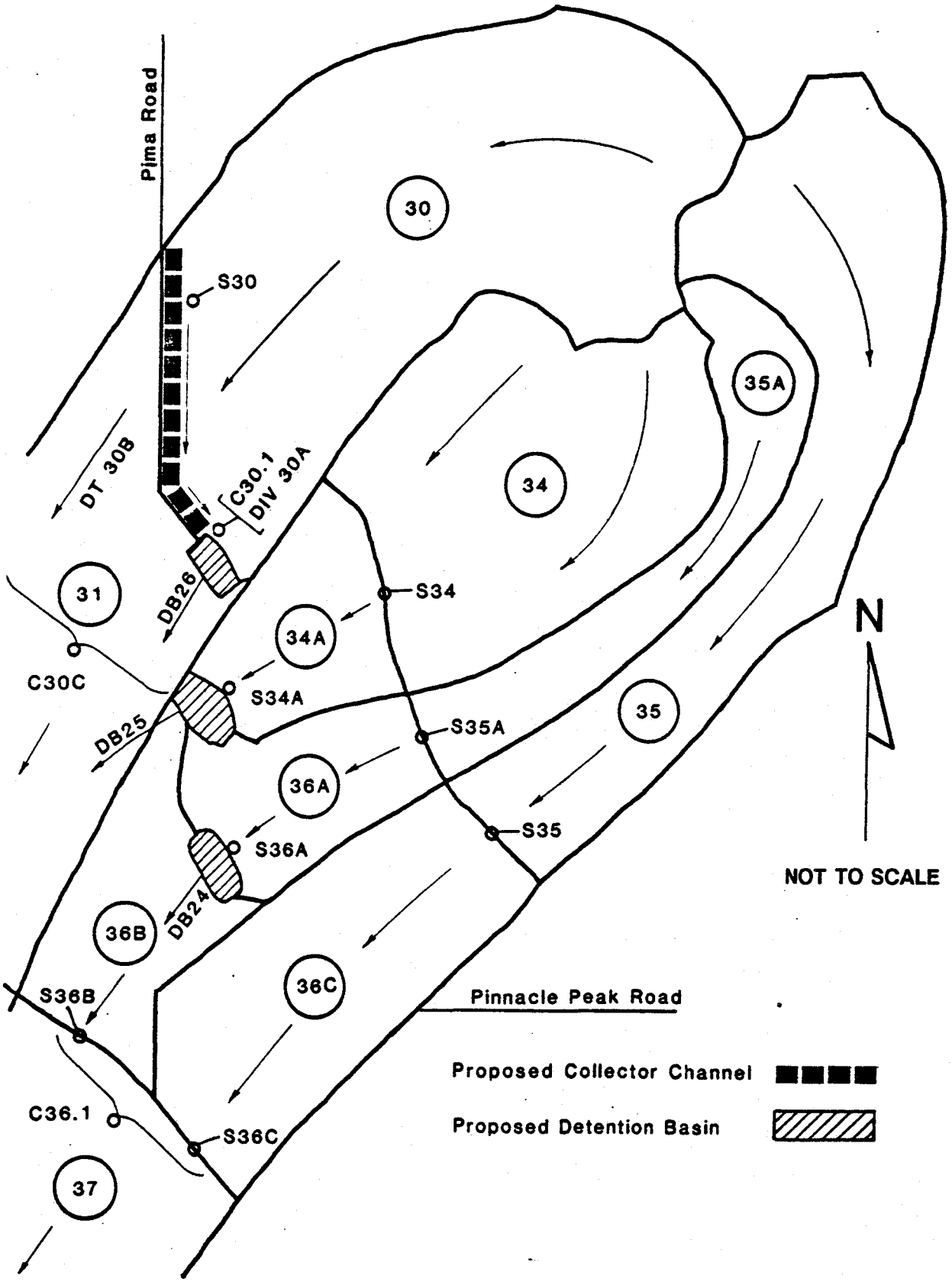
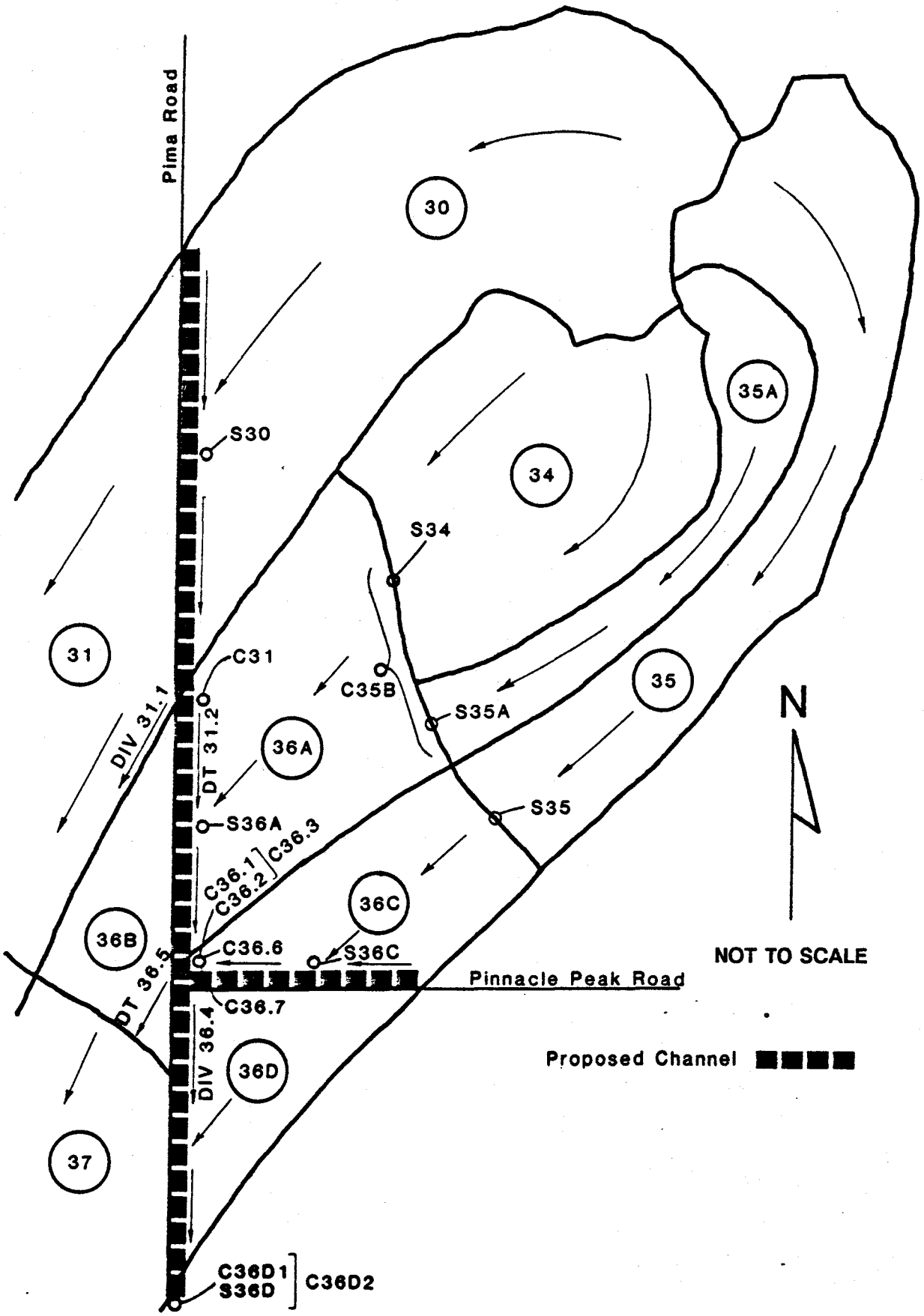


Water Resources Associates, Inc.

PROJECT NO. 08701960	DRAWN BY RLW
DATE 9/2/88	CHECKED BY JJH

CASE 2, HEC-1 ROUTING SCHEMATIC NEAR PIMA AND PINNACLE PEAK ROADS



Water Resources Associates, Inc.

PROJECT NO. 08701960 | DRAWN BY RLW
DATE 9/2/88 | CHECKED BY JJH

CASE 3, HEC-1 ROUTING SCHEMATIC
NEAR PIMA AND
PINNACLE PEAK ROADS

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TABLES

TABLE 2.1

SUMMARY OF HYDROLOGIC SOIL GROUP ASSIGNED TO SOIL COMPLEX/ASSOCIATIONS

Map Symbol	Soil Complex/ Association	Hydrologic Soil Group	Comments
3 & 4	Antho- Carrizo- Maripo Complex	B A B	35% Antho 30% Carrizo 20% Maripo 15% Brios (A), Gilman (B), Vint (B), Denure (B), Momoli (B), Carrizo (A) Use HSG B for Map Symbols 3 & 4.
6 & 7	Anthony- Arizo Complex	B A	40% Anthony 40% Arizo 20% of the group is sandy soils similar to Arizo Use 50% HSG B & 50% HSG A for Map Symbols 6 & 7.
39	Eba- Nickel- Cave Association	C B D	30% Eba 25% Nickel 25% Cave 20% Rock outcrop Use HSG C for Map Symbol 39.
40,41, 42,43	Eba- Pinaleno Complex	C B	45% Eba 35% Pinaleno 20% Arizo (A), Anthony (B), Contenental (C), Ohaco (C), Greyeagle (D), Nickel (B), Vado (B), Tres Hermanos (B). Use 50% HSG B & 50% HSG C for Map Symbols 40,41,42,43.

TABLE 2.1 (CONTINUED)

SUMMARY OF HYDROLOGIC SOIL GROUP ASSIGNED TO SOIL COMPLEX/ASSOCIATIONS

Map Symbol	Soil Complex/ Association	Hydrologic Soil Group	Comments
91 & 92	Momoli-Carrizo Complex	B A	45% Momoli 35% Carrizo 20% Mohall (B), Tremant (B), Gunsight (B), Chuckawalla (B), Denure (B), Gilman (B), Maripo (B), Carrizo (A) Use HSG B for Map Symbols 91 & 92.
93 & 94	Nickel-Cave Complex	B D	50% Nickel 35% Cave 15% Arizo (A), Anthony (B), Pinaleno (B), Greyeagle (D) Use HSG C for Map Symbols 93 & 94.

TABLE 3.1

SUMARY OF SCS CURVE NUMBERS AS A FUNCTION OF STORM DURATION AND HYDROLOGIC SOIL GROUP

Hydrologic Soil Group	Curve Number By Storm Duration[1]					
	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
A	74	71	69	66	63	60
B	83	81	80	78	76	74
C	89	87	86	85	83	82
D	92	91	90	88	87	86

Notes:

1 Adjustments made in accordance with Runoff Curve Numbers for Semiarid Range and Forest Conditions, Woodward, 1973. Values are rounded to the nearest integer.

All curve numbers are based on "desert brush" with 15% cover density. Antecedent Moisture Condition 2 is assumed.

TABLE 3.2

PERCENT OF IMPERVIOUS AREA FOR LAND-USE CLASSIFICATIONS

Land-Use Classification	Percent Impervious	Comments
1/5 - 1/3 Du/Ac	6.9	Avg = 4/15 Du/Ac = 0.267 Du/Ac
1/3 - 1 Du/Ac	14.7	Avg = 2/3 Du/Ac = 0.667 Du/Ac
1 - 2 Du/Ac	24.1	Avg = 1.5 Du/Ac
2 - 4 Du/Ac	34.9	Avg = 3 Du/Ac
4 - 8 Du/Ac	54.0	Avg = 6 Du/Ac
8 - 12 Du/Ac	74.0	Avg = 10 Du/Ac
12 - 22 Du/Ac	94.0	Avg = 17 Du/Ac
Tourist Accomodations	85.0	From TR-55, Table 2-2a
Low-Intensity Resort	85.0	From TR-55, Table 2-2a
Support Commercial	85.0	From TR-55, Table 2-2a
General Commercial	85.0	From TR-55, Table 2-2a
Cultural/ Institutional	85.0	From TR-55, Table 2-2a
Minor Office	85.0	From TR-55, Table 2-2a
Major Office	85.0	From TR-55, Table 2-2a
Light Industrial	72.0	From TR-55, Table 2-2a
Open Space	1.0	Engineering Judgement

TABLE 3.3

IMPERVIOUS AREA CALCULATION FOR FUTURE CONDITION, URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 1 to 53

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	1/5 to 1/3 6.90%	1/3 to 1 14.70%	1 to 2 24.10%	2 to 4 34.90%	4 to 8 54.00%	8 to 12 74.00%	12 to 22 94.00%	Tourist Accomodations 85.00%	Low Intensity Resort 85.00%	Support Commercial 85.00%	General Commercial 85.00%	Cultural/ Institutional 85.00%	Minor Office 85.00%	Major Office 85.00%	Light Industrial 72.00%	Weighted % Impervious
1	215.75	41.89	0	173.86	0	0	0	0	0	0	0	0	0	0	0	0	0	12.04
2	778.88	134.41	0	508.28	64.27	0	7.09	0	0	0	47.52	0	17.31	0	0	0	0	19.32
3	348.67	11.57	40	252.40	29.28	0	0	0	0	0	15.42	0	0	0	0	0	0	17.25
4	966.53	0	0	966.53	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
6	1774.02	1774.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
7	428.16	4.30	178.96	236.70	8.20	0	0	0	0	0	0	0	0	0	0	0	0	11.48
8	696.64	69.66	0	578.86	0	0	21.24	0	27.08	0	0	0	0	0	0	0	0	17.61
10	1353.22	1353.22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
11	375.36	31.39	214.68	129.29	0	0	0	0	0	0	0	0	0	0	0	0	0	9.09
12	852.29	0	0	724.39	51.85	0	0	0	0	0	0	0	0	76.05	0	0	0	21.54
14	717.38	717.38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
15	852.93	0	0	852.93	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
16	947.78	161.28	0	786.50	0	0	0	0	0	0	0	0	0	0	0	0	0	12.37
17	733.38	210.24	0	523.14	0	0	0	0	0	0	0	0	0	0	0	0	0	10.77
18	810.88	710.35	25.88	74.65	0	0	0	0	0	0	0	0	0	0	0	0	0	2.45
19	113.54	28.26	0	85.28	0	0	0	0	0	0	0	0	0	0	0	0	0	11.29
20	364.67	274.34	0	90.33	0	0	0	0	0	0	0	0	0	0	0	0	0	4.39
21	2357.82	974.97	549.70	803.11	30.04	0	0	0	0	0	0	0	0	0	0	0	0	7.34
22	887.04	305.91	581.13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4.87
23	252.48	61.56	131.41	59.51	0	0	0	0	0	0	0	0	0	0	0	0	0	7.30
24	291.20	69.97	130.13	91.10	0	0	0	0	0	0	0	0	0	0	0	0	0	7.92
25	973.82	252.56	198.91	522.35	0	0	0	0	0	0	0	0	0	0	0	0	0	9.55
26	390.08	14.01	174.24	201.83	0	0	0	0	0	0	0	0	0	0	0	0	0	10.72
27	488.90	0	0	431.43	57.47	0	0	0	0	0	0	0	0	0	0	0	0	15.80
28	911.04	345.82	0	88.58	73.08	71.55	70.28	0	0	65.89	0	0	27.82	140.51	1.89	0	25.62	34.70
30	850.94	132	624.41	94.53	0	0	0	0	0	0	0	0	0	0	0	0	0	6.85
31	788.80	8.10	2.87	777.83	0	0	0	0	0	0	0	0	0	0	0	0	0	14.53
32	895.04	292.77	0	384.28	33.32	43.63	0	0	0	25.31	0	26.60	6.46	41.87	24.75	2.92	13.13	22.44
33	1005.89	667.30	0	0	0	318.98	9.10	0	0	0	0	0	0	0	10.51	0	0	13.11
34	539.71	256.78	3.96	239.41	6.90	0	32.66	0	0	0	0	0	0	0	0	0	0	10.62
35	824.26	499.69	0	198.50	36.36	0	0	0	0	0	0	0	89.71	0	0	0	0	14.46
36	930.43	0	559.14	327.72	0	14.34	0	0	0	0	0	0	18.56	0	10.67	0	0	12.53
37	761.34	199.28	0	335.71	91.05	114.71	14.91	0	0	0	0	0	5.68	0	0	0	0	16.58
38	806.21	171.97	0	0	178.89	165.94	151.28	0	138.13	0	0	0	0	0	0	0	0	38.98
39	1243.01	170.65	694.06	117.42	0	65.71	0	0	0	0	59.55	0	120.43	0	15.19	0	0	20.57
40	374.02	94.45	106.49	112.11	60.97	0	0	0	0	0	0	0	0	0	0	0	0	10.55
41	506.30	254.48	78.43	40.89	88.58	0	43.92	0	0	0	0	0	0	0	0	0	0	11.66
42	374.02	154.06	20.21	30.54	75.65	44.40	0	0	0	0	49.36	0	0	0	0	0	0	22.21
43	685.31	89.14	0	468.52	127.65	0	0	0	0	0	0	0	0	0	0	0	0	14.67
44	546.37	161.68	0	384.69	0	0	0	0	0	0	0	0	0	0	0	0	0	10.65
45	130.37	38.98	0	31.37	31.68	28.34	0	0	0	0	0	0	0	0	0	0	0	17.28
45A	110.08	70.57	0	16.38	0	23.13	0	0	0	0	0	0	0	0	0	0	0	10.16
46	593.15	491.66	0	101.49	0	0	0	0	0	0	0	0	0	0	0	0	0	3.34
47	240.45	94.18	31.37	53.52	61.38	0	0	0	0	0	0	0	0	0	0	0	0	10.72
48	28.03	2.96	25.07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.28
49	225.79	4.05	221.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.79

TABLE 3.3 (CONTINUED)

IMPERVIOUS AREA CALCULATION FOR FUTURE CONDITION, URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 1 to 53

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	1/5 to 1/3 6.90%	1/3 to 1 14.70%	1 to 2 24.10%	2 to 4 34.90%	4 to 8 54.00%	8 to 12 74.00%	12 to 22 94.00%	Tourist Accomodations 85.00%	Low Intensity Resort 85.00%	Support Commercial 85.00%	General Commercial 85.00%	Cultural/ Institutional 85.00%	Minor Office 85.00%	Major Office 85.00%	Light Industrial 72.00%	Weighted % Impervious
50	229.12	159.39	43.33	26.40	0	0	0	0	0	0	0	0	0	0	0	0	0	3.69
51	683.20	.17	267.16	372.55	0	43.32	0	0	0	0	0	0	0	0	0	0	0	12.93
52	867.65	0	0	748.43	119.22	0	0	0	0	0	0	0	0	0	0	0	0	15.99
53	1118.14	685.81	0	157.57	258.86	0	0	7.83	8.07	0	0	0	0	0	0	0	0	9.46

TABLE 3.4

IMPERVIOUS AREA CALCULATION FOR FUTURE CONDITION, URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 1000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	1/5 to 1/3 6.90%	1/3 to 1 14.70%	1 to 2 24.10%	2 to 4 34.90%	4 to 8 54.00%	8 to 12 74.00%	12 to 22 94.00%	Tourist Accomodations 85.00%	Low Intensity Resort 85.00%	Support Commercial 85.00%	General Commercial 85.00%	Cultural/ Institutional 85.00%	Minor Office 85.00%	Major Office 85.00%	Light Industrial 72.00%	Weighted % Impervious
1000	209.86	197.22	12.64	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.36
1010	601.02	485.54	115.48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2.13
1020	713.98	691.04	22.94	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.19
1030	234.18	223.51	10.67	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.27
1040	709.82	461.13	248.69	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3.07
1050	327.81	71.50	256.31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5.61
1060	883.39	361.92	114.17	363.48	0	43.82	0	0	0	0	0	0	0	0	0	0	0	9.08
1070	869.12	555.33	0	0	0	288.81	0	0	0	0	24.98	0	0	0	0	0	0	14.68
1080	724.99	344.54	0	0	0	94.22	106.74	0	0	0	179.49	0	0	0	0	0	0	34.01
1090	429.76	152.31	0	0	0	277.45	0	0	0	0	0	0	0	0	0	0	0	22.89
1100	323.71	137.94	109.59	0	0	76.18	0	0	0	0	0	0	0	0	0	0	0	10.98
1110	84.48	0	84.48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.90
1120	29.38	0	29.38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.90
1130	115.26	0	115.26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.90
1140	133.63	0	133.63	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.90
1150	530.30	123.76	151.47	72.42	0	128.25	0	0	0	0	54.40	0	0	0	0	0	0	21.37
1160	884.29	318.13	0	95.57	309.86	101.77	0	0	0	0	58.96	0	0	0	0	0	0	20.08
1170	270.40	42.28	0	94.33	99.68	0	0	0	0	0	0	0	34.11	0	0	0	0	24.89
1180	206.59	73.49	0	122.85	0	0	0	0	0	0	0	0	0	10.25	0	0	0	13.31
1190	669.44	112.63	0	476.50	0	0	0	0	0	0	0	0	80.31	0	0	0	0	20.83
1200	64.26	0	0	64.26	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
1210	687.30	147.71	0	538.12	1.47	0	0	0	0	0	0	0	0	0	0	0	0	11.78
1220	433.86	37.93	0	276.61	119.32	0	0	0	0	0	0	0	0	0	0	0	0	16.09
1230	390.72	71.74	0	192.42	77.58	30.10	0	0	0	18.88	0	0	0	0	0	0	0	19.00
1240	302.08	100.93	0	60.98	0	28.08	20.39	0	0	0	41.09	0	50.61	0	0	0	0	35.99
1250	335.17	42.24	0	270.18	0	0	0	0	0	0	22.75	0	0	0	0	0	0	17.75
1260	314.50	261.84	0	52.66	0	0	0	0	0	0	0	0	0	0	0	0	0	3.29
1270	405.89	49.30	0	342	0	14.59	0	0	0	0	0	0	0	0	0	0	0	13.76
1280	265.41	78.77	0	170.09	12.57	3.98	0	0	0	0	0	0	0	0	0	0	0	11.38
1290	276.42	44.63	0	152.30	39.38	1.58	26.20	0	12.33	0	0	0	0	0	0	0	0	21.20
1300	165.31	0	0	165.31	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
1310	305.34	0	0	305.34	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
1320	120.32	98.12	0	22.20	0	0	0	0	0	0	0	0	0	0	0	0	0	3.53
1330	163.46	75.25	0	88.21	0	0	0	0	0	0	0	0	0	0	0	0	0	8.39
1340	994.50	82.14	0	639.88	210.81	8.63	34.36	0	0	0	0	0	18.68	0	0	0	0	18.41
1350	601.02	42.25	0	421.47	1.34	0	33.47	44.60	0	0	29.64	0	0	0	28.25	0	0	27.12
1360	419.65	56.33	0	84.94	69.13	0	0	0	0	49.13	45.44	0	0	114.68	0	0	0	49.46
1370	532.61	25.12	0	424.49	69.73	0	10.98	2.29	0	0	0	0	0	0	0	0	0	16.35
1380	595.52	0	0	595.52	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
1390	934.34	111.31	0	823.03	0	0	0	0	0	0	0	0	0	0	0	0	0	13.07
1400	845.25	14.47	0	815.52	0	0	0	0	0	0	0	0	0	15.26	0	0	0	15.73
1410	105.15	0	0	105.15	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
1420	435.71	3.16	0	432.55	0	0	0	0	0	0	0	0	0	0	0	0	0	14.60
1430	916.42	0	0	889.28	0	0	0	17	0	0	0	0	10.14	0	0	0	0	16.58
1440	1208.45	45.61	4.46	1064.90	31.65	0	0	36.35	0	25.48	0	0	0	0	0	0	0	17.67
1450	490.37	89.05	0	386.94	0	0	14.38	0	0	0	0	0	0	0	0	0	0	13.36

TABLE 3.4 (CONTINUED)

IMPERVIOUS AREA CALCULATION FOR FUTURE CONDITION, URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 1000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	1/5 to 1/3 6.90%	1/3 to 1 14.70%	1 to 2 24.10%	2 to 4 34.90%	4 to 8 54.00%	8 to 12 74.00%	12 to 22 94.00%	Tourist Accomodations 85.00%	Low Intensity Resort 85.00%	Support Commercial 85.00%	General Commercial 85.00%	Cultural/ Institutional 85.00%	Minor Office 85.00%	Major Office 85.00%	Light Industrial 72.00%	Weighted % Impervious
1460	606.08	0	0	589.32	0	0	0	0	0	0	0	0	16.76	0	0	0	0	16.64
1470	190.53	0	0	190.53	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
1475	1081.28	78.51	0	467.51	230.92	165.53	108.50	0	0	14.82	0	0	15.49	0	0	0	0	24.72
1480	640.96	36.17	0	592.68	12.11	0	0	0	0	0	0	0	0	0	0	0	0	14.10
1490	841.15	155.11	0	670.54	0	0	15.50	0	0	0	0	0	0	0	0	0	0	12.90
1500	1148.74	50.75	0	1027.26	0	0	1.77	0	0	0	68.96	0	0	0	0	0	0	18.38

TABLE 3.5

IMPERVIOUS AREA CALCULATION FOR FUTURE CONDITION, URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 2000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	1/5 to 1/3 6.90%	1/3 to 1 14.70%	1 to 2 24.10%	2 to 4 34.90%	4 to 8 54.00%	8 to 12 74.00%	12 to 22 94.00%	Tourist Accomodations 85.00%	Low Intensity Resort 85.00%	Support Commercial 85.00%	General Commercial 85.00%	Cultural/ Institutional 85.00%	Minor Office 85.00%	Major Office 85.00%	Light Industrial 72.00%	Weighted % Impervious
2000	511.94	405.11	106.83	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2.23
2005	87.23	3	84.23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.70
2010	59.71	28.87	30.84	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4.05
2015	259.39	184.43	0	30.76	0	44.20	0	0	0	0	0	0	0	0	0	0	0	8.40
2020	69.76	67.87	0	0	0	1.89	0	0	0	0	0	0	0	0	0	0	0	1.92
2030	150.14	0	0	100.22	0	49.92	0	0	0	0	0	0	0	0	0	0	0	21.42
2040	594.56	496.55	0	0	0	98.01	0	0	0	0	0	0	0	0	0	0	0	6.59
2050	549.12	443.19	0	0	0	105.93	0	0	0	0	0	0	0	0	0	0	0	7.54
2060	98.69	49.75	0	0	0	48.94	0	0	0	0	0	0	0	0	0	0	0	17.81
2065	106.50	15.03	0	0	0	91.47	0	0	0	0	0	0	0	0	0	0	0	30.12
2070	937.54	907.71	0	0	0	29.83	0	0	0	0	0	0	0	0	0	0	0	2.08
2080	87.23	0	0	0	0	87.23	0	0	0	0	0	0	0	0	0	0	0	34.90
2090	234.18	0	0	115.44	0	118.74	0	0	0	0	0	0	0	0	0	0	0	24.94
2100	242.88	0	0	124.14	78.86	39.88	0	0	0	0	0	0	0	0	0	0	0	21.07
2110	331.52	105.44	0	0	3.82	147.74	0	0	0	0	0	0	0	74.52	0	0	0	35.26
2120	912.32	218.29	0	245.39	356.09	0	0	0	0	0	0	0	0	0	0	0	92.55	20.90
2130	191.49	115.18	0	0	0	76.31	0	0	0	0	0	0	0	0	0	0	0	14.51
2135	74.82	25.96	0	0	0	48.86	0	0	0	0	0	0	0	0	0	0	0	23.14
2140	123.07	0	0	0	0	123.07	0	0	0	0	0	0	0	0	0	0	0	34.90
2150	439.87	247.28	0	0	0	192.59	0	0	0	0	0	0	0	0	0	0	0	15.84
2160	631.74	57.54	0	0	115.50	458.70	0	0	0	0	0	0	0	0	0	0	0	29.84
2170	296.58	51.56	0	27.04	217.98	0	0	0	0	0	0	0	0	0	0	0	0	19.23
2180	642.82	629.99	0	6.10	6.73	0	0	0	0	0	0	0	0	0	0	0	0	1.37
2190	78.53	13.54	0	17.43	47.56	0	0	0	0	0	0	0	0	0	0	0	0	18.03
2200	162.05	99.42	0	62.63	0	0	0	0	0	0	0	0	0	0	0	0	0	6.29
2210	1136.83	958	0	160.26	18.57	0	0	0	0	0	0	0	0	0	0	0	0	3.31
2220	362.24	52.42	0	116.75	193.07	0	0	0	0	0	0	0	0	0	0	0	0	17.73
2230	210.75	52.97	0	157.78	0	0	0	0	0	0	0	0	0	0	0	0	0	11.26
2240	386.11	61.52	0	19.48	255.94	0	0	0	0	0	0	0	0	49.17	0	0	0	27.70
2250	143.30	13.31	0	57.87	72.12	0	0	0	0	0	0	0	0	0	0	0	0	18.16
2255	189.12	66.78	0	0	122.34	0	0	0	0	0	0	0	0	0	0	0	0	15.94
2260	169.41	18.07	0	0	151.34	0	0	0	0	0	0	0	0	0	0	0	0	21.64
2270	621.70	342.43	0	279.27	0	0	0	0	0	0	0	0	0	0	0	0	0	7.15
2280	115.71	50.55	0	8.99	40.28	0	0	0	0	0	0	0	0	15.89	0	0	0	21.64
2290	384.32	39.23	0	336.92	8.17	0	0	0	0	0	0	0	0	0	0	0	0	13.50
2300	1152.45	1065.27	0	87.18	0	0	0	0	0	0	0	0	0	0	0	0	0	2.04
2310	194.24	32.55	0	161.69	0	0	0	0	0	0	0	0	0	0	0	0	0	12.40
2320	238.27	43.13	0	189.56	0	0	0	0	0	0	0	0	0	5.58	0	0	0	13.87
2330	307.65	220.45	0	87.20	0	0	0	0	0	0	0	0	0	0	0	0	0	4.88
2335	224.96	31.23	0	134.58	0	0	0	0	0	0	0	0	0	59.15	0	0	0	31.28
2340	316.80	119.14	0	33.43	0	0	0	0	0	0	0	0	0	164.23	0	0	0	45.99
2350	275.46	153.88	0	2.08	0	0	0	0	0	0	0	0	0	119.50	0	0	0	37.54
2360	148.29	130.56	0	0	17.73	0	0	0	0	0	0	0	0	0	0	0	0	3.76
2370	116.16	61.63	0	0	54.53	0	0	0	0	0	0	0	0	0	0	0	0	11.84
2380	29.38	23.15	0	0	6.23	0	0	0	0	0	0	0	0	0	0	0	0	5.90
2390	133.12	72.18	0	0	34.83	0	0	0	0	0	26.11	0	0	0	0	0	0	23.52

TABLE 3.5 (CONTINUED)

IMPERVIOUS AREA CALCULATION FOR FUTURE CONDITION, URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 2000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	1/5 to 1/3 6.90%	1/3 to 1 14.70%	1 to 2 24.10%	2 to 4 34.90%	4 to 8 54.00%	8 to 12 74.00%	12 to 22 94.00%	Tourist Accomodations 85.00%	Low Intensity Resort 85.00%	Support Commercial 85.00%	General Commercial 85.00%	Cultural/ Institutional 85.00%	Minor Office 85.00%	Major Office 85.00%	Light Industrial 72.00%	Weighted % Impervious
2400	234.62	117.26	0	0	72.27	0	0	0	0	0	45.09	0	0	0	0	0	0	24.26
2410	159.81	50.49	0	0	40.57	0	8.12	0	0	0	60.63	0	0	0	0	0	0	41.43
2420	638.72	611.22	0	0	0	0	0	0	0	0	27.50	0	0	0	0	0	0	4.62
2430	528	482.34	0	0	0	0	0	0	0	0	45.66	0	0	0	0	0	0	8.26
2440	506.43	375.09	0	70.77	0	19.48	0	0	0	0	41.09	0	0	0	0	0	0	11.03
2450	155.65	0	0	80.85	0	5.60	0	0	0	0	69.20	0	0	0	0	0	0	46.68
2460	224.51	46.71	0	90.14	25.61	0	30.44	0	0	0	31.61	0	0	0	0	0	0	28.15
2470	163.01	1.80	0	132.98	0	0	0	0	0	0	0	0	0	28.23	0	0	0	26.72
2480	535.36	160.24	0	252.75	77.65	44.72	0	0	0	0	0	0	0	0	0	0	0	13.65
2490	142.34	25.85	0	104.43	12.06	0	0	0	0	0	0	0	0	0	0	0	0	13.01
2500	292.03	90.14	0	11.45	43.12	64.73	23.86	0	0	27.57	0	0	5.62	25.54	0	0	0	33.69
2510	79.42	0	0	79.42	0	0	0	0	0	0	0	0	0	0	0	0	0	14.70
2520	60.16	7.96	0	30.69	4.98	14.71	0	0	0	1.82	0	0	0	0	0	0	0	20.73
2525	107.01	0	0	97.65	0	0	0	0	0	0	0	0	0	9.36	0	0	0	20.85
2530	110.66	29.55	0	81.11	0	0	0	0	0	0	0	0	0	0	0	0	0	11.04
2540	77.57	11.07	0	7.90	0	8.64	.72	0	0	28.13	0	0	0	21.11	0	0	0	59.98
2545	132.22	5.50	0	126.72	0	0	0	0	0	0	0	0	0	0	0	0	0	14.13
2550	208.90	23.25	0	171.19	0	0	0	0	0	1.69	0	0	0	12.77	0	0	0	18.04
2560	333.76	217.05	0	34.64	6.97	4.89	66.89	0	0	0	0	0	3.32	0	0	0	0	14.86
2570	271.81	152.28	0	29.24	0	60.45	14.86	0	0	0	0	0	0	14.98	0	0	0	17.54
2580	237.82	13.15	0	0	28.23	0	21.98	0	0	9.15	0	0	26.35	130.64	0	8.32	0	70.26
2590	453.18	103.35	0	146.35	55.20	125.30	0	0	0	0	0	0	0	22.98	0	0	0	21.87
2600	412.29	407.53	0	2.43	0	0	2.33	0	0	0	0	0	0	0	0	0	0	1.38
2610	303.04	158.71	0	62.29	31.65	14.68	6.74	0	0	14.49	0	0	0	14.48	0	0	0	17.08
2620	167.62	106.79	0	0	24.84	35.99	0	0	0	0	0	0	0	0	0	0	0	11.70
2625	174.02	110.59	0	0	0	47.35	16.08	0	0	0	0	0	0	0	0	0	0	15.12
2630	172.61	104.71	0	0	0	0	45.47	0	0	22.43	0	0	0	0	0	0	0	25.88
2640	11.01	8.48	0	0	0	0	0	0	0	2.53	0	0	0	0	0	0	0	20.30
2650	216.26	123.88	0	0	20.34	0	45.39	0	0	5.64	21.01	0	0	0	0	0	0	24.65
2660	108.80	86.37	0	0	0	0	0	0	0	0	22.43	0	0	0	0	0	0	18.32
2670	98.24	84.31	0	0	0	0	13.93	0	0	0	0	0	0	0	0	0	0	8.52
2680	114.30	64.77	0	0	0	36.04	13.49	0	0	0	0	0	0	0	0	0	0	17.94

TABLE 3.6

Impervious Area Calculations For Existing Drainage Conditions
North Scottsdale General Drainage Plan
Sub-Basin Series 1 to 53

Sub-Basin No.	Total Basin Area (ac)	Number of Existing Structural Units in Basin	Existing Dwelling Units/Acre	% Impervious Cover
1	215.75	1	.0046	1.10
2	778.88	10	.0128	1.28
3	348.67	7	.0201	1.44
4	966.53	90	.0931	3.05
5	1197.63	16	.0134	1.29
6	1774.02	0	.0000	0
7	428.16	22	.0514	2.13
8	696.64	54	.0775	2.71
10	1353.22	0	.0000	0
11	375.36	9	.0240	1.53
12	852.29	15	.0176	1.39
14	717.38	0	.0000	0
15	852.93	0	.0000	0
16	947.78	0	.0000	0
17	733.38	0	.0000	0
18	810.88	0	.0000	0
19	113.54	0	.0000	0
20	364.67	0	.0000	0
21	2357.82	9	.0038	1.08
22	887.04	0	.0000	0
23	252.48	0	.0000	0
24	291.20	1	.0034	1.07
25	973.82	1	.0010	1.02
26	390.08	9	.0231	1.51
27	488.90	13	.0266	1.59
28	911.04	11	.0121	1.27
30	850.94	20	.0235	1.52
31	788.80	79	.1002	3.20
32	895.04	81	.0905	2.99
33	1005.89	0	.0000	0
34	539.71	35	.0648	2.43
35	824.26	16	.0194	1.43
36	930.43	47	.0505	2.11
37	761.34	297	.3901	9.58
38	806.21	0	.0000	0
39	1243.01	34	.0274	1.60
40	374.02	1	.0027	1.06
41	506.30	24	.0474	2.04
42	374.02	4	.0107	1.24
43	685.31	1	.0015	1.03
44	546.37	0	.0000	0
45	130.37	0	.0000	0
45A	110.08	0	.0000	0
46	593.15	0	.0000	0
47	240.45	3	.0125	1.28

TABLE 3.6 (CONTINUED)

Impervious Area Calculations For Existing Drainage Conditions
North Scottsdale General Drainage Plan
Sub-Basin Series 1 to 53

Sub-Basin No.	Total Basin Area (ac)	Number of Existing Structural Units in Basin	Existing Dwelling Units/Acre	% Impervious Cover
48	28.03	4	.1427	4.14
49	225.79	9	.0399	1.88
50	229.12	4	.0175	1.39
51	683.20	95	.1391	4.06
52	867.65	6	.0069	1.15
53	1118.14	0	.0000	0

TABLE 3.7

Impervious Area Calculations For Existing Drainage Conditions
North Scottsdale General Drainage Plan
Sub-Basin Series 1000

Sub-Basin No.	Total Basin Area (ac)	Number of Existing Structural Units in Basin	Existing Dwelling Units/Acre	% Impervious Cover
1000	209.86	0	.0000	0
1010	601.02	1	.0017	1.04
1020	713.98	0	.0000	0
1030	234.18	0	.0000	0
1040	709.82	0	.0000	0
1050	327.81	0	.0000	0
1060	883.39	0	.0000	0
1070	869.12	2	.0023	1.05
1080	724.99	12	.0166	1.37
1090	429.76	0	.0000	0
1100	323.71	0	.0000	0
1110	84.48	2	.0237	1.52
1120	29.38	0	.0000	0
1130	115.26	1	.0087	1.19
1140	133.63	5	.0374	1.82
1150	530.30	3	.0057	1.13
1160	884.29	1	.0011	1.02
1170	270.40	4	.0148	1.33
1180	206.59	2	.0097	1.21
1190	669.44	7	.0105	1.23
1200	64.26	2	.0311	1.68
1210	687.30	67	.0975	3.15
1220	433.86	5	.0115	1.25
1230	390.72	48	.1229	3.70
1240	302.08	11	.0364	1.80
1250	335.17	0	.0000	0
1260	314.50	47	.1494	4.29
1270	405.89	118	.2907	7.40
1280	265.41	13	.0490	2.08
1290	276.42	68	.2460	6.41
1300	165.31	30	.1815	4.99
1310	305.34	22	.0721	2.59
1320	120.32	0	.0000	0
1330	163.46	0	.0000	0
1340	994.50	5	.0050	1.11
1350	601.02	5	.0083	1.18
1360	419.65	4	.0095	1.21
1370	532.61	13	.0244	1.54
1380	595.52	7	.0118	1.26
1390	934.34	8	.0086	1.19
1400	845.25	67	.0793	2.74
1410	105.15	0	.0000	0
1420	435.71	72	.1652	4.63
1430	916.42	8	.0087	1.19
1440	1208.45	0	.0000	0

TABLE 3.7 (CONTINUED)

Impervious Area Calculations For Existing Drainage Conditions
North Scottsdale General Drainage Plan
Sub-Basin Series 1000

Sub-Basin No.	Total Basin Area (ac)	Number of Existing Structural Units in Basin	Existing Dwelling Units/Acre	% Impervious Cover
1450	490.37	1	.0020	1.04
1460	606.08	82	.1353	3.98
1470	190.53	0	.0000	0
1475	1081.28	0	.0000	0
1480	640.96	2	.0031	1.07
1490	841.15	97	.1153	3.54
1500	1148.74	142	.1236	3.72

TABLE 3.8

Impervious Area Calculations For Existing Drainage Conditions
North Scottsdale General Drainage Plan
Sub-Basin Series 2000

Sub-Basin No.	Total Basin Area (ac)	Number of Existing Structural Units in Basin	Existing Dwelling Units/Acre	% Impervious Cover
2000	511.94	0	.0000	0
2005	87.23	3	.0344	1.76
2010	59.71	1	.0167	1.37
2015	259.39	0	.0000	0
2020	69.76	0	.0000	0
2030	150.14	0	.0000	0
2040	594.56	0	.0000	0
2050	549.12	0	.0000	0
2060	98.69	0	.0000	0
2065	106.50	0	.0000	0
2070	937.54	0	.0000	0
2080	87.23	0	.0000	0
2090	234.18	0	.0000	0
2100	242.88	0	.0000	0
2110	331.52	0	.0000	0
2120	912.32	0	.0000	0
2130	191.49	0	.0000	0
2135	74.82	0	.0000	0
2140	123.07	0	.0000	0
2150	439.87	0	.0000	0
2160	631.74	0	.0000	0
2170	296.58	0	.0000	0
2180	642.82	0	.0000	0
2190	78.53	0	.0000	0
2200	162.05	0	.0000	0
2210	1136.83	0	.0000	0
2220	362.24	0	.0000	0
2230	210.75	0	.0000	0
2240	386.11	0	.0000	0
2250	143.30	0	.0000	0
2255	189.12	0	.0000	0
2260	169.41	0	.0000	0
2270	621.70	0	.0000	0
2280	115.71	0	.0000	0
2290	384.32	0	.0000	0
2300	1152.45	2	.0017	1.04
2310	194.24	0	.0000	0
2320	238.27	0	.0000	0
2330	307.65	0	.0000	0
2335	224.96	0	.0000	0
2340	316.80	1	.0032	1
2350	275.46	0	.0000	0
2360	148.29	0	.0000	0
2370	116.16	0	.0000	0
2380	29.38	0	.0000	0

TABLE 3.8 (CONTINUED)

Impervious Area Calculations For Existing Drainage Conditions
North Scottsdale General Drainage Plan
Sub-Basin Series 2000

Sub-Basin No.	Total Basin Area (ac)	Number of Existing Structural Units in Basin	Existing Dwelling Units/Acre	% Impervious Cover
2390	133.12	0	.0000	0
2400	234.62	0	.0000	0
2410	159.81	0	.0000	0
2420	638.72	0	.0000	0
2430	528	0	.0000	0
2440	506.43	0	.0000	0
2450	155.65	0	.0000	0
2460	224.51	0	.0000	0
2470	163.01	0	.0000	0
2480	535.36	0	.0000	0
2490	142.34	0	.0000	0
2500	292.03	0	.0000	0
2510	79.42	2	.0252	1.55
2520	60.16	0	.0000	0
2525	107.01	3	.0280	1.62
2530	110.66	9	.0813	2.79
2540	77.57	7	.0902	2.98
2545	132.22	7	.0529	2.16
2550	208.90	5	.0239	1.53
2560	333.76	0	.0000	0
2570	271.81	1	.0037	1.08
2580	237.82	1	.0042	1.09
2590	453.18	3	.0066	1.15
2600	412.29	0	.0000	0
2610	303.04	0	.0000	0
2620	167.62	0	.0000	0
2625	174.02	0	.0000	0
2630	172.61	1	.0058	1.13
2640	11.01	0	.0000	0
2650	216.26	1	.0046	1.10
2660	108.80	0	.0000	0
2670	98.24	0	.0000	0
2680	114.30	0	.0000	0

TABLE 3.9

PERCENT IMPERVIOUS AREA FOR MAXIMUM DENSITY BUILDOUT LAND-USE CLASSIFICATIONS

Land-Use[1]	Average[1] Du/Ac	1.5 x Avg.[2] Du/Ac	% Impervious[4] @ 1.5 Avg Du/Ac
1/5 - 1/3 Du/Ac	0.267	0.4	10.0
1/3 - 1 Du/Ac	0.667	1.0	20.0
1 - 2 Du/Ac	1.5	2.25	29.9
2 - 4 Du/Ac	3	4.5	45.0
4 - 8 Du/Ac	6	9.0	69.8
8 - 12 Du/Ac	10	15.0	90.0
12 - 22 Du/Ac	17	25.5	100.0

	% Impervious[1]	1.1 x % Impervious[3]
Tourist Accomodations	85	93.5
Low-Intensity Resort	85	93.5
Support Commercial	85	93.5
General Commercial	85	93.5
Cultural/ Institutional	85	93.5
Minor Office	85	93.5
Major Office	85	93.5
Light Industrial	72	79.2
Open Space	1	1(no change)

Notes:

1 Based on General Plan & Tonto Foothills Plan
2 50% increase to reflect maximum density buildout
3 10% increase to reflect maximum density buildout
4 Values read from Figure 4.1

TABLE 3.10

MAXIMUM DENSITY BUILDOUT

IMPERVIOUS AREA CALCULATION FOR URBANIZED DRAINAGE SUB-BASINS

NORTH SCOTTSDALE GENERAL DRAINAGE PLAN

SUB-BASIN SERIES 1 to 53

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	.40 10.00%	1.00 20.00%	2.25 29.90%	4.50 45.00%	9.00 69.80%	15.00 90.00%	25.50 100.00%	Tourist Accomodations 93.50%	Low Intensity Resort 93.50%	Support Commercial 93.50%	General Commercial 93.50%	Cultural/ Institutional 93.50%	Minor Office 93.50%	Major Office 93.50%	Light Industrial 79.20%	Weighted % Impervious
1	215.75	41.89	0	173.86	0	0	0	0	0	0	0	0	0	0	0	0	0	16.31
2	778.88	134.41	0	508.28	64.27	0	7.09	0	0	0	47.52	0	17.31	0	0	0	0	24.11
3	348.67	11.57	40	252.40	29.28	0	0	0	0	0	15.42	0	0	0	0	0	0	22.30
4	966.53	0	0	966.53	0	0	0	0	0	0	0	0	0	0	0	0	0	20
6	1774.02	1774.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
7	428.16	4.30	178.96	236.70	8.20	0	0	0	0	0	0	0	0	0	0	0	0	15.82
8	696.64	69.46	0	578.86	0	0	21.24	0	27.08	0	0	0	0	0	0	0	0	22.73
10	1353.22	1353.22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
11	375.36	31.39	214.68	129.29	0	0	0	0	0	0	0	0	0	0	0	0	0	12.69
12	852.29	0	0	724.39	51.85	0	0	0	0	0	0	0	0	76.05	0	0	0	27.16
14	717.38	717.38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
15	852.93	0	0	852.93	0	0	0	0	0	0	0	0	0	0	0	0	0	20
16	947.78	161.28	0	786.50	0	0	0	0	0	0	0	0	0	0	0	0	0	16.77
17	733.38	210.24	0	523.14	0	0	0	0	0	0	0	0	0	0	0	0	0	14.55
18	810.88	710.35	25.88	74.65	0	0	0	0	0	0	0	0	0	0	0	0	0	3.04
19	113.54	28.26	0	85.28	0	0	0	0	0	0	0	0	0	0	0	0	0	15.27
20	364.67	274.34	0	90.33	0	0	0	0	0	0	0	0	0	0	0	0	0	5.71
21	2357.82	974.97	549.70	803.11	30.04	0	0	0	0	0	0	0	0	0	0	0	0	9.94
22	887.04	305.91	581.13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.90
23	252.48	61.56	131.41	59.51	0	0	0	0	0	0	0	0	0	0	0	0	0	10.16
24	291.20	69.97	130.13	91.10	0	0	0	0	0	0	0	0	0	0	0	0	0	10.97
25	973.82	252.56	198.91	522.35	0	0	0	0	0	0	0	0	0	0	0	0	0	13.03
26	390.08	14.01	174.24	201.83	0	0	0	0	0	0	0	0	0	0	0	0	0	14.85
27	488.90	0	0	431.43	57.47	0	0	0	0	0	0	0	0	0	0	0	0	21.16
28	911.04	345.82	0	88.58	73.08	71.55	70.28	0	0	65.89	0	0	27.82	140.51	1.89	0	25.62	40.10
30	850.94	132	624.41	94.53	0	0	0	0	0	0	0	0	0	0	0	0	0	9.71
31	788.80	8.10	2.87	777.83	0	0	0	0	0	0	0	0	0	0	0	0	0	19.77
32	895.04	292.77	0	384.28	33.32	43.63	0	0	0	25.31	0	26.60	6.46	41.87	24.75	2.92	13.13	26.74
33	1005.89	667.30	0	0	0	318.98	9.10	0	0	0	0	0	0	0	10.51	0	0	16.54
34	539.71	256.78	3.96	239.41	6.90	0	32.66	0	0	0	0	0	0	0	0	0	0	14.03
35	824.26	499.69	0	198.50	36.36	0	0	0	0	0	0	0	89.71	0	0	0	0	16.92
36	930.43	0	559.14	327.72	0	14.34	0	0	0	0	0	0	18.56	0	10.67	0	0	16.68
37	761.34	199.28	0	335.71	91.05	114.71	14.91	0	0	0	0	0	5.68	0	0	0	0	21.50
38	806.21	171.97	0	0	178.89	165.94	151.28	0	138.13	0	0	0	0	0	0	0	0	46.34
39	1243.01	170.65	694.06	117.42	0	65.71	0	0	0	0	59.55	0	120.43	0	15.19	0	0	24.67
40	374.02	94.45	106.49	112.11	60.97	0	0	0	0	0	0	0	0	0	0	0	0	13.97
41	506.30	254.48	78.43	40.89	88.58	0	43.92	0	0	0	0	0	0	0	0	0	0	14.95
42	374.02	154.06	20.21	39.54	75.45	44.40	0	0	0	0	49.36	0	0	0	0	0	0	26.30
43	685.31	89.14	0	468.52	127.65	0	0	0	0	0	0	0	0	0	0	0	0	19.37
44	546.37	161.68	0	384.69	0	0	0	0	0	0	0	0	0	0	0	0	0	14.38
45	130.37	38.98	0	31.37	31.68	28.34	0	0	0	0	0	0	0	0	0	0	0	22.16
45A	110.08	70.57	0	16.38	0	23.13	0	0	0	0	0	0	0	0	0	0	0	13.07
46	593.15	491.66	0	101.49	0	0	0	0	0	0	0	0	0	0	0	0	0	4.25
47	240.45	94.18	31.37	53.52	61.38	0	0	0	0	0	0	0	0	0	0	0	0	13.78
48	28.03	2.96	25.07	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9.05

TABLE 3.10 (CONTINUED)

MAXIMUM DENSITY BUILDOUT
IMPERVIOUS AREA CALCULATION FOR URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 1 to 53

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	.40 10.00%	1.00 20.00%	2.25 29.90%	4.50 45.00%	9.00 69.80%	15.00 90.00%	25.50 100.00%	Tourist Accomodations 93.50%	Low Intensity Resort 93.50%	Support Commercial 93.50%	General Commercial 93.50%	Cultural/ Institutional 93.50%	Minor Office 93.50%	Major Office 93.50%	Light Industrial 79.20%	Weighted % Impervious
49	225.79	4.05	221.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9.84
50	229.12	159.39	43.33	26.40	0	0	0	0	0	0	0	0	0	0	0	0	0	4.89
51	683.20	. .17	267.16	372.55	0	43.32	0	0	0	0	0	0	0	0	0	0	0	17.67
52	867.65	0	0	748.43	119.22	0	0	0	0	0	0	0	0	0	0	0	0	21.36
53	1118.14	685.81	0	157.57	258.86	0	0	7.83	8.07	0	0	0	0	0	0	0	0	11.71

TABLE 3.11

MAXIMUM DENSITY BUILDOUT
IMPERVIOUS AREA CALCULATION FOR URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 1000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	.40 10.00%	1.00 20.00%	2.25 29.90%	4.50 45.00%	9.00 69.80%	15.00 90.00%	25.50 100.00%	Tourist Accomodations 93.50%	Low Intensity Resort 93.50%	Support Commercial 93.50%	General Commercial 93.50%	Cultural/ Institutional 93.50%	Minor Office 93.50%	Major Office 93.50%	Light Industrial 79.20%	Weighted % Impervious
1000	209.86	197.22	12.64	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.54
1010	601.02	485.54	115.48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2.73
1020	713.98	691.04	22.94	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.29
1030	234.18	223.51	10.67	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.41
1040	709.82	461.13	248.69	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4.15
1050	327.81	71.50	256.31	0	0	0	0	0	0	0	0	0	0	0	0	0	0	8.04
1060	883.39	361.92	114.17	363.48	0	43.82	0	0	0	0	0	0	0	0	0	0	0	12.16
1070	869.12	555.33	0	0	0	288.81	0	0	0	0	24.98	0	0	0	0	0	0	18.28
1080	724.99	344.54	0	0	0	94.22	106.74	0	0	0	179.49	0	0	0	0	0	0	39.75
1090	429.76	152.31	0	0	0	277.45	0	0	0	0	0	0	0	0	0	0	0	29.41
1100	323.71	137.94	109.59	0	0	76.18	0	0	0	0	0	0	0	0	0	0	0	14.40
1110	84.48	0	84.48	0	0	0	0	0	0	0	0	0	0	0	0	0	0	10
1120	29.38	0	29.38	0	0	0	0	0	0	0	0	0	0	0	0	0	0	10
1130	115.26	0	115.26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	10
1140	133.63	0	133.63	0	0	0	0	0	0	0	0	0	0	0	0	0	0	10
1150	530.30	123.76	151.47	72.42	0	128.25	0	0	0	0	54.40	0	0	0	0	0	0	26.30
1160	884.29	318.13	0	95.57	309.86	101.77	0	0	0	0	58.96	0	0	0	0	0	0	24.41
1170	270.40	42.28	0	94.33	99.68	0	0	0	0	0	0	0	34.11	0	0	0	0	29.95
1180	206.59	73.49	0	122.85	0	0	0	0	0	0	0	0	0	10.25	0	0	0	16.89
1190	669.44	112.63	0	476.50	0	0	0	0	0	0	0	0	80.31	0	0	0	0	25.62
1200	64.26	0	0	64.26	0	0	0	0	0	0	0	0	0	0	0	0	0	20
1210	687.30	147.71	0	538.12	1.47	0	0	0	0	0	0	0	0	0	0	0	0	15.94
1220	433.86	37.93	0	276.61	119.32	0	0	0	0	0	0	0	0	0	0	0	0	21.06
1230	390.72	71.74	0	192.42	77.58	30.10	0	0	0	18.88	0	0	0	0	0	0	0	23.95
1240	302.08	100.93	0	60.98	0	28.08	20.39	0	0	0	41.09	0	50.61	0	0	0	0	41.65
1250	335.17	42.24	0	270.18	0	0	0	0	0	0	22.75	0	0	0	0	0	0	22.59
1260	314.50	261.84	0	52.66	0	0	0	0	0	0	0	0	0	0	0	0	0	4.18
1270	405.89	49.30	0	342	0	14.59	0	0	0	0	0	0	0	0	0	0	0	18.59
1280	265.41	78.77	0	170.09	12.57	3.98	0	0	0	0	0	0	0	0	0	0	0	15.20
1290	276.42	44.63	0	152.30	39.38	1.58	26.20	0	12.33	0	0	0	0	0	0	0	0	26.77
1300	165.31	0	0	165.31	0	0	0	0	0	0	0	0	0	0	0	0	0	20
1310	305.34	0	0	305.34	0	0	0	0	0	0	0	0	0	0	0	0	0	20
1320	120.32	98.12	0	22.20	0	0	0	0	0	0	0	0	0	0	0	0	0	4.51
1330	163.46	75.25	0	88.21	0	0	0	0	0	0	0	0	0	0	0	0	0	11.25
1340	994.50	82.14	0	639.88	210.81	8.63	34.36	0	0	0	0	0	18.68	0	0	0	0	23.85
1350	601.02	42.25	0	421.47	1.34	0	33.47	44.60	0	0	29.64	0	0	0	28.25	0	0	33.73
1360	419.65	56.33	0	84.94	69.13	0	0	0	0	49.13	45.44	0	0	114.68	0	0	0	55.73
1370	532.61	25.12	0	424.49	69.73	0	10.98	2.29	0	0	0	0	0	0	0	0	0	21.73
1380	595.52	0	0	595.52	0	0	0	0	0	0	0	0	0	0	0	0	0	20
1390	934.34	111.31	0	823.03	0	0	0	0	0	0	0	0	0	0	0	0	0	17.74
1400	845.25	14.47	0	815.52	0	0	0	0	0	0	0	0	0	15.26	0	0	0	21.00
1410	105.15	0	0	105.15	0	0	0	0	0	0	0	0	0	0	0	0	0	20
1420	435.71	3.16	0	432.55	0	0	0	0	0	0	0	0	0	0	0	0	0	19.86
1430	916.42	0	0	889.28	0	0	0	17	0	0	0	0	10.14	0	0	0	0	22.11

TABLE 3.11 (CONTINUED)

MAXIMUM DENSITY BUILDOUT
IMPERVIOUS AREA CALCULATION FOR URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 1000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	.40 10.00%	1.00 20.00%	2.25 29.90%	4.50 45.00%	9.00 69.80%	15.00 90.00%	25.50 100.00%	Tourist Accomodations 93.50%	Low Intensity Resort 93.50%	Support Commercial 93.50%	General Commercial 93.50%	Cultural/ Institutional 93.50%	Minor Office 93.50%	Major Office 93.50%	Light Industrial 79.20%	Weighted % Impervious
1440	1208.45	45.61	4.46	1064.90	31.65	0	0	36.35	0	25.48	0	0	0	0	0	0	0	23.16
1450	490.37	89.05	0	386.94	0	0	14.38	0	0	0	0	0	0	0	0	0	0	18.01
1460	606.08	0	0	589.32	0	0	0	0	0	0	0	0	16.76	0	0	0	0	22.03
1470	190.53	0	0	190.53	0	0	0	0	0	0	0	0	0	0	0	0	0	20
1475	1081.28	78.51	0	467.51	230.92	165.53	108.50	0	0	14.82	0	0	15.49	0	0	0	0	31.62
1480	640.96	36.17	0	592.68	12.11	0	0	0	0	0	0	0	0	0	0	0	0	19.11
1490	841.15	155.11	0	670.54	0	0	15.50	0	0	0	0	0	0	0	0	0	0	17.41
1500	1148.74	50.75	0	1027.26	0	0	1.77	0	0	0	68.96	0	0	0	0	0	0	23.65

TABLE 3.12

MAXIMUM DENSITY BUILDOUT

IMPERVIOUS AREA CALCULATION FOR URBANIZED DRAINAGE SUB-BASINS

NORTH SCOTTSDALE GENERAL DRAINAGE PLAN

SUB-BASIN SERIES 2000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0 1.00%	.40 10.00%	1.00 20.00%	2.25 29.90%	4.50 45.00%	9.00 69.80%	15.00 90.00%	25.50 100.00%	Tourist Accomodations 93.50%	Low Intensity Resort 93.50%	Support Commercial 93.50%	General Commercial 93.50%	Cultural/ Institutional 93.50%	Minor Office 93.50%	Major Office 93.50%	Light Industrial 79.20%	Weighted % Impervious
2000	511.94	405.11	106.83	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2.88
2005	87.23	3	84.23	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9.69
2010	59.71	28.87	30.84	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5.65
2015	259.39	184.43	0	30.76	0	44.20	0	0	0	0	0	0	0	0	0	0	0	10.75
2020	69.76	67.87	0	0	0	1.89	0	0	0	0	0	0	0	0	0	0	0	2.19
2030	150.14	0	0	100.22	0	49.92	0	0	0	0	0	0	0	0	0	0	0	28.31
2040	594.56	496.55	0	0	0	98.01	0	0	0	0	0	0	0	0	0	0	0	8.25
2050	549.12	443.19	0	0	0	105.93	0	0	0	0	0	0	0	0	0	0	0	9.49
2060	98.69	49.75	0	0	0	48.94	0	0	0	0	0	0	0	0	0	0	0	22.82
2065	106.50	15.03	0	0	0	91.47	0	0	0	0	0	0	0	0	0	0	0	38.79
2070	937.54	907.71	0	0	0	29.83	0	0	0	0	0	0	0	0	0	0	0	2.40
2080	87.23	0	0	0	0	87.23	0	0	0	0	0	0	0	0	0	0	0	45
2090	234.18	0	0	115.44	0	118.74	0	0	0	0	0	0	0	0	0	0	0	32.68
2100	242.88	0	0	124.14	78.86	39.88	0	0	0	0	0	0	0	0	0	0	0	27.32
2110	331.52	105.44	0	0	3.82	147.74	0	0	0	0	0	0	0	74.52	0	0	0	41.73
2120	912.32	218.29	0	245.39	356.09	0	0	0	0	0	0	0	0	0	0	0	92.55	25.32
2130	191.49	115.18	0	0	0	76.31	0	0	0	0	0	0	0	0	0	0	0	18.53
2135	74.82	25.96	0	0	0	48.86	0	0	0	0	0	0	0	0	0	0	0	29.73
2140	123.07	0	0	0	0	123.07	0	0	0	0	0	0	0	0	0	0	0	45
2150	439.87	247.28	0	0	0	192.59	0	0	0	0	0	0	0	0	0	0	0	20.26
2160	631.74	57.54	0	0	115.50	458.70	0	0	0	0	0	0	0	0	0	0	0	38.23
2170	296.58	51.56	0	27.04	217.98	0	0	0	0	0	0	0	0	0	0	0	0	23.97
2180	642.82	629.99	0	6.10	6.73	0	0	0	0	0	0	0	0	0	0	0	0	1.48
2190	78.53	13.54	0	17.43	47.56	0	0	0	0	0	0	0	0	0	0	0	0	22.72
2200	162.05	99.42	0	62.63	0	0	0	0	0	0	0	0	0	0	0	0	0	8.34
2210	1136.83	958	0	160.26	18.57	0	0	0	0	0	0	0	0	0	0	0	0	4.15
2220	362.24	52.42	0	116.75	193.07	0	0	0	0	0	0	0	0	0	0	0	0	22.53
2230	210.75	52.97	0	157.78	0	0	0	0	0	0	0	0	0	0	0	0	0	15.22
2240	386.11	61.52	0	19.48	255.94	0	0	0	0	0	0	0	0	49.17	0	0	0	32.90
2250	143.30	13.31	0	57.87	72.12	0	0	0	0	0	0	0	0	0	0	0	0	23.22
2255	189.12	66.78	0	0	122.34	0	0	0	0	0	0	0	0	0	0	0	0	19.70
2260	169.41	18.07	0	0	151.34	0	0	0	0	0	0	0	0	0	0	0	0	26.82
2270	621.70	342.43	0	279.27	0	0	0	0	0	0	0	0	0	0	0	0	0	9.53
2280	115.71	50.55	0	8.99	40.28	0	0	0	0	0	0	0	0	15.89	0	0	0	25.24
2290	384.32	39.23	0	336.92	8.17	0	0	0	0	0	0	0	0	0	0	0	0	18.27
2300	1152.45	1065.27	0	87.18	0	0	0	0	0	0	0	0	0	0	0	0	0	2.44
2310	194.24	32.55	0	161.69	0	0	0	0	0	0	0	0	0	0	0	0	0	16.82
2320	238.27	43.13	0	189.56	0	0	0	0	0	0	0	0	0	5.58	0	0	0	18.28
2330	307.65	220.45	0	87.20	0	0	0	0	0	0	0	0	0	0	0	0	0	6.39
2335	224.96	31.23	0	134.58	0	0	0	0	0	0	0	0	0	59.15	0	0	0	36.69
2340	316.80	119.14	0	33.43	0	0	0	0	0	0	0	0	0	164.23	0	0	0	50.96
2350	275.46	153.88	0	2.08	0	0	0	0	0	0	0	0	0	119.50	0	0	0	41.27
2360	148.29	130.56	0	0	17.73	0	0	0	0	0	0	0	0	0	0	0	0	4.46
2370	116.16	61.63	0	0	54.53	0	0	0	0	0	0	0	0	0	0	0	0	14.57

TABLE 3.12 (CONTINUED)

MAXIMUM DENSITY BUILDOUT
IMPERVIOUS AREA CALCULATION FOR URBANIZED DRAINAGE SUB-BASINS
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
SUB-BASIN SERIES 2000

		Dwelling Units/Acre & Percent Impervious Cover								Non-Residential Land-Use & Percent Impervious Cover								
SUB-BASIN NO.	Total Basin Area (ac)	0	.40	1.00	2.25	4.50	9.00	15.00	25.50	Tourist Accomodations	Low Intensity Resort	Support Commercial	General Commercial	Cultural/ Institutional	Minor Office	Major Office	Light Industrial	Weighted % Impervious
		1.00%	10.00%	20.00%	29.90%	45.00%	69.80%	90.00%	100.00%	93.50%	93.50%	93.50%	93.50%	93.50%	93.50%	93.50%	79.20%	
2380	29.38	23.15	0	0	6.23	0	0	0	0	0	0	0	0	0	0	0	0	7.13
2390	133.12	72.18	0	0	34.83	0	0	0	0	0	26.11	0	0	0	0	0	0	26.70
2400	234.62	117.26	0	0	72.27	0	0	0	0	0	45.09	0	0	0	0	0	0	27.68
2410	159.81	50.49	0	0	40.57	0	8.12	0	0	0	60.63	0	0	0	0	0	0	46.93
2420	638.72	611.22	0	0	0	0	0	0	0	0	27.50	0	0	0	0	0	0	4.98
2430	528	482.34	0	0	0	0	0	0	0	0	45.66	0	0	0	0	0	0	9.00
2440	506.43	375.09	0	70.77	0	19.48	0	0	0	0	41.09	0	0	0	0	0	0	12.85
2450	155.65	0	0	80.85	0	5.60	0	0	0	0	69.20	0	0	0	0	0	0	53.58
2460	224.51	46.71	0	90.14	25.61	0	30.44	0	0	0	31.61	0	0	0	0	0	0	34.28
2470	163.01	1.80	0	132.98	0	0	0	0	0	0	0	0	0	28.23	0	0	0	32.52
2480	535.36	160.24	0	252.75	77.65	44.72	0	0	0	0	0	0	0	0	0	0	0	17.84
2490	142.34	25.85	0	104.43	12.06	0	0	0	0	0	0	0	0	0	0	0	0	17.39
2500	292.03	90.14	0	11.45	43.12	64.73	23.86	0	0	27.57	0	0	5.62	25.54	0	0	0	39.99
2510	79.42	0	0	79.42	0	0	0	0	0	0	0	0	0	0	0	0	0	20
2520	60.16	7.96	0	30.69	4.98	14.71	0	0	0	1.82	0	0	0	0	0	0	0	26.64
2525	107.01	0	0	97.65	0	0	0	0	0	0	0	0	0	9.36	0	0	0	26.43
2530	110.66	29.55	0	81.11	0	0	0	0	0	0	0	0	0	0	0	0	0	14.93

TABLE 3.13

SUMMARY OF OVERLAND FLOW ROUGHNESS VALUES
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN

	Topographic Area	Overland Flow Roughness Value
1.	Alluvial fan terrace	0.10
2.	Rolling fan terrace	0.15
3.	Northern portions of watershed with well-defined channel geometry and coarse-grained surface conditions	0.20
4.	Transition areas from steep mountain slopes to alluvial fans and fan terraces	0.25
5.	Steep mountain slopes with rugged rock outcrops	0.30

TABLE 3.14

SUMMARY OF RAINFALL VALUES USED IN HEC-1 MODELS

Return Interval	Sub-Basin Series 1000	Sub-Basin Series 1-53	Sub-Basin Series 2000
100-yr	4.71"	4.32"	4.23"
10-yr	3.19"	2.92"	2.85"
2-yr	2.13"	1.92"	1.87"

Notes:

All rainfall values on this page are based on a 24-hour storm duration and a 10 square mile areal reduction factor.

TABLE 3.15

SUMMARY OF SENSITIVITY ANALYSES
FOR RAINFALL PARAMETERS
SUB-BASINS 2240 - 2340
(all discharge values in cfs)

Concentration Point or Sub-Basin No.	Total Basin Area * (sq. mi.)	24-Hour Hypothetical, NOAA Rainfall 10 Sq. Mi. *NOAA ARF	24-Hour Hypothetical, NOAA Rainfall 50 Sq. Mi. *NOAA ARF	24-Hour Hypothetical, NOAA/PHX Rainfall 10 Sq. Mi. *NOAA ARF	24-Hour Type IIA, NOAA Rainfall 10 Sq. Mi. NOAA ARF	24-Hour Type IIA, NOAA Rainfall 10 Sq. Mi. HYDRO-40 ARF	24-Hour Type IIA, NOAA Rainfall 50 Sq. Mi. NOAA ARF	24-Hour Type IIA, NOAA Rainfall 50 Sq. Mi. HYDRO-40 ARF	24-Hour PHX Distribution, NOAA Rainfall 10 Sq. Mi. NOAA ARF	24-Hour Type II, NOAA Rainfall 10 Sq. Mi. NOAA ARF	1-Hour Hypothetical NOAA Rainfall 10 Sq. Mi. *NOAA ARF	1-Hour Hypothetical Osborn/Renard Rainfall 10 Sq. Mi. *NOAA ARF
2240	.60	712	505	771	664	535	546	431	645	649	543	811
2260	.26	290	219	313	273	241	246	193	285	262	219	352
2255	.30	565	439	607	491	456	462	400	381	480	512	812
2270	.97	2064	1564	2238	1763	1633	1655	1430	1282	1664	2096	3404
2271	1.27	2619	1969	2844	2249	2075	2104	1830	1663	2144	2573	4142
2250	1.49	2850	2160	3089	2610	2410	2443	2072	1942	2369	2893	4940
2251	1.76	3104	2351	3361	2832	2600	2638	2223	2227	2581	3024	5150
2280	1.94	3111	2447	3359	2859	2644	2682	2265	2392	2648	3345	5433
2290	.60	1332	987	1448	1130	1015	1034	841	812	1009	889	1363
2300	1.80	4814	3427	5179	3975	3722	3764	3123	2592	3560	3252	5122
2302	DIVERT	2407	1713	2589	1987	1861	1882	1562	1296	1780	1626	2561
2310	DIVERT	2536	1928	2701	2229	2046	2066	2040	1671	2001	1630	2592
2320	.37	857	665	943	706	655	664	539	500	700	563	876
2330	.48	1211	937	1299	1015	947	959	843	692	993	1054	1576
2340	.50	960	748	1039	847	777	789	670	630	779	756	1233

* These ARFs were actually computed with the HEC-1 algorithm and applied to each of the rainfall values on the PH card.

TABLE 3.10

SUMMARY OF INDEPENDENT PEAK DISCHARGE CALCULATIONS
FOR VERIFICATION OF HEC-1 MODEL
NORTH SCOTTSDALE GENERAL DRAINAGE PLAN
100-YEAR STORM

Concentration Point or Sub-Basin No.	Total Basin Area (sq. mi.)	USGS Regression Equations by Eychaner	USGS Regression Equations by Roske, Region 2	TR-55 NOAA Rainfall 10 Sq. Mi. ARF	Pima County Peak, NOAA Rainfall 10 Sq. Mi. ARF	HEC-1 NOAA Rainfall 24-Hour, Type IIA Storm 10 Sq. Mi. ARF
2240	.60	862	852	485	629	664
2260	.26	486	562	216	314	273
2255	.30	526	603	356	583	491
2270	.97	1180	1083	1179	1997	1763
2290	.60	860	852	721	918	1130
2300	1.80	1738	1475	2216	3032	3975

TABLE 4.1

COMPARISON OF EXISTING AND FUTURE RUNOFF RESPONSE 100-YEAR EVENT

Outlet Location	HEC-1 Concentration Point	100-Year Peak Discharge		
		Existing Conditions	Future Conditions	Maximum Density Buildout
Sub-Basin 1040	1040	9017	9077	
Sub-Basin 1070	1072	6478	6934	
Sub-Basin 1190	1190	1319	1703	
Sub-Basin 1370	1372	2714	3695	
Sub-Basin 1450	1450	3614	4644	
Sub-Basin 8	Sub 8	753	1001	1105
Sub-Basin 16	Sub 16	2314	2699	2821
Sub-Basin 27	Sub 27	9831	11,794	12,293
Sub-Basin 35	Sub 35	995	1239	1283
Sub-Basins 41, 42,47	CP 48	13,502	14,830	15,398
Sub-Basin 2120	2120	523	830	
Sub-Basin 2210	2210	3466	3541	
Sub-Basin 2340	2340	851	1165	
Sub-Basin 2450	2450	5553	5890	
Sub-Basin 2480	2480	1576	1725	
Sub-Basin 2580	2580	2306	2695	2803
Sub-Basin 2610	2610	2022	2128	2144

TABLE 4.2

COMPARISON OF EXISTING AND FUTURE RUNOFF RESPONSE 10-YEAR EVENT

Outlet Location	HEC-1 Concentration Point	10-Year Peak Discharge		
		Existing Conditions	Future Conditions	Maximum Density Buildout
Sub-Basin 1040	1040	5030	5073	
Sub-Basin 1070	1072	3475	3991	
Sub-Basin 1190	1190	429	693	
Sub-Basin 1370	1372	961	1608	
Sub-Basin 1450	1450	1306	1823	
Sub-Basin 8	Sub 8	256	395	462
Sub-Basin 16	Sub 16	822	1106	1201
Sub-Basin 27	Sub 27	3601	4485	4765
Sub-Basin 35	Sub 35	445	637	677
Sub-Basins 41, 42,47	CP 48	6182	8125	7655
Sub-Basin 2120	2120	173	378	
Sub-Basin 2210	2210	1469	1560	
Sub-Basin 2340	2340	347	671	
Sub-Basin 2450	2450	2461	2733	
Sub-Basin 2480	2480	588	774	
Sub-Basin 2580	2580	1047	1374	1475
Sub-Basin 2610	2610	694	801	818

TABLE 4.3

COMPARISON OF EXISTING AND FUTURE RUNOFF RESPONSE 2-YEAR EVENT

Outlet Location	HEC-1 Concentration Point	2-Year Peak Discharge		
		Existing Conditions	Future Conditions	Maximum Density Buildout
Sub-Basin 1040	1040	2550	2635	
Sub-Basin 1070	1072	1253	1952	
Sub-Basin 1190	1190	141	300	
Sub-Basin 1370	1372	264	717	
Sub-Basin 1450	1450	417	750	
Sub-Basin 8	Sub 8	69	144	179
Sub-Basin 16	Sub 16	257	380	436
Sub-Basin 27	Sub 27	1092	1565	1741
Sub-Basin 35	Sub 35	129	239	264
Sub-Basins 41, 42,47	CP 48	1534	2603	2920
Sub-Basin 2120	2120	39	154	
Sub-Basin 2210	2210	381	453	
Sub-Basin 2340	2340	81	358	
Sub-Basin 2450	2450	759	987	
Sub-Basin 2480	2480	171	283	
Sub-Basin 2580	2580	220	647	749
Sub-Basin 2610	2610	171	220	230

why are some [illegible] twice w/ diff Q's?

TABLE 4.4

SUMMARY OF CHANNEL HYDRAULICS FOR EXISTING WASHES

X-SEC	Location	Return Interval (years)	Discharge (cfs)	Flow Depth (ft)	Channel Velocity (fps)	Topwidth @ Flow Depth (ft)	Minimum Bank Elevation (ft)
1	Sub 2450	2	759	2.09	72.5	50.37	16.00
		10	2461	4.35	11.24	50.77	16.00
		100	5553	7.33	14.96	51.30	16.00
2	Sub 2450	2	759	1.54	7.01	71.02	3.00
		10	2461	3.34	10.35	72.00	3.00
		100	5553	5.73	13.56	72.00	3.00
3	Sub 2370	2	44	0.92	4.53	12.07	6.50
		10	166	1.96	6.93	15.52	6.50
		100	365	3.41	6.76	54.16	6.50
4	Sub 2380	2	124	1.09	5.33	22.57	3.50
		10	416	2.35	7.56	45.15	3.50
		100	889	3.64	8.53	74.29	3.50
5	Sub 2390	2	26	0.64	3.32	13.55	6.00
		10	124	1.56	5.61	17.25	6.00
		100	351	2.68	6.06	74.81	6.00
6	Sub 2340	2	40	0.46	2.95	30.32	8.00
		10	173	1.11	5.07	33.54	8.00
		100	426	1.87	6.97	37.35	8.00
7	Sub 2190	2	402	1.87	4.99	86.08	4.00
		10	1072	2.71	6.35	124.62	4.00
		100	2297	3.61	7.66	166.05	4.00
8	Sub 2200	2	51	1.00	3.15	27.46	8.00
		10	193	1.75	4.45	44.48	8.00
		100	501	2.60	5.65	63.39	8.00
9	Sub 2210	2	381	2.34	7.88	25.35	3.00
		10	1469	5.03	11.96	28.00	3.00
		100	3466	8.42	15.92	28.00	3.00
10	Sub 2090	2	1133	1.50	5.90	129.00	1.00
		10	4109	3.27	9.78	129.00	1.00
		100	9226	5.42	13.24	129.00	1.00

TABLE 4.4 (CONTINUED)

SUMMARY OF CHANNEL HYDRAULICS FOR EXISTING WASHES

X-SEC	Location	Return Interval (years)	Discharge (cfs)	Flow Depth (ft)	Channel Velocity (fps)	Topwidth @ Flow Depth (ft)	Minimum Bank Elevation (ft)
11	Sub 2010	2	796	2.05	7.99	64.66	7.00
		10	3148	4.87	12.07	91.63	7.00
		100	6873	7.79	14.87	112.00	7.00
12	Sub 2010	2	942	2.21	7.91	61.72	4.50
		10	3515	4.88	11.39	78.00	4.50
		100	7958	7.78	14.88	78.00	4.50
13	Sub 2030	2	977	2.38	7.54	74.85	2.00
		10	3578	4.79	10.73	94.88	2.00
		100	7935	7.27	13.48	105.00	2.00
14	Sub 2140	2	430	2.22	7.48	33.40	4.50
		10	983	3.51	8.10	83.63	4.50
		100	2103	4.68	10.41	88.18	4.50
15	Sub 2150	2	128	2.13	5.16	23.40	5.00
		10	467	3.44	8.08	37.82	5.00
		100	1295	5.40	10.30	55.40	5.00
16	Sub 2160	2	548	3.84	7.37	48.89	4.00
		10	1268	5.32	9.27	52.32	4.00
		100	2969	7.76	11.85	54.00	4.00
17	Sub 2130	2	403	0.56	3.60	201.12	8.00
		10	1177	1.06	5.51	202.12	8.00
		100	2646	1.75	7.48	203.51	8.00
18	Sub 2135	2	410	2.20	7.84	27.54	7.00
		10	972	3.72	9.89	32.77	7.00
		100	2069	5.82	11.87	39.94	7.00
19	Sub 2160	2	398	2.48	6.69	28.93	3.00
		10	1251	4.69	9.49	36.08	3.00
		100	3284	7.92	13.08	37.00	3.00
20	Sub 48	2	1508	2.29	8.51	79.58	4.00
		10	6069	5.68	13.33	83.00	4.00
		100	13,137	9.35	17.28	83.00	4.00

TABLE 4.4 (CONTINUED)

SUMMARY OF CHANNEL HYDRAULICS FOR EXISTING WASHES

X-SEC	Location	Return Interval (years)	Discharge (cfs)	Flow Depth (ft)	Channel Velocity (fps)	Topwidth @ Flow Depth (ft)	Minimum Bank Elevation (ft)
21	Sub 51	2	1615	2.86	9.48	61.00	2.00
		10	6439	7.06	15.09	61.00	2.00
		100	14,380	12.02	19.72	61.00	2.00
22	Sub 2470	2	61	0.90	3.85	21.20	7.50
		10	231	1.85	5.82	18.83	7.50
		100	511	2.80	7.25	36.37	7.50
23	Sub 2480	2	171	1.72	4.87	28.83	8.50
		10	588	3.16	6.77	42.92	8.50
		100	1576	5.11	6.85	158.46	8.50
24	Sub 2490	2	95	1.10	4.38	21.31	10.00
		10	266	2.02	6.26	24.06	10.00
		100	498	2.92	7.62	26.76	10.00
25	Sub 2500	2	73	0.69	4.49	24.78	6.50
		10	320	1.77	7.07	29.08	6.50
		100	878	3.29	9.33	35.17	6.50
26	Sub 2570	2	55	0.75	4.72	17.01	12.00
		10	189	1.64	6.69	20.54	12.00
		100	428	2.68	8.27	24.70	12.00
27	Sub 2560	2	90	0.55	3.75	44.10	12.00
		10	469	1.53	6.90	46.05	12.00
		100	1064	2.61	8.95	48.21	12.00
28	Sub 2580	2	220	1.55	6.12	24.47	7.50
		10	1047	3.57	7.96	128.15	7.50
		100	2306	4.54	10.44	130.08	7.50
29	Sub 2610	2	171	1.21	4.14	155.42	16.00
		10	694	1.84	6.77	156.68	16.00
		100	2022	2.74	9.71	158.48	16.00
30	Sub 2335	2	282	2.16	8.12	17.00	2.00
		10	1149	5.32	13.00	17.00	2.00
		100	3216	10.45	18.31	17.00	2.00

TABLE 4.4 (CONTINUED)

SUMMARY OF CHANNEL HYDRAULICS FOR EXISTING WASHES

X-SEC	Location	Return Interval (years)	Discharge (cfs)	Flow Depth (ft)	Channel Velocity (fps)	Topwidth @ Flow Depth (ft)	Minimum Bank Elevation (ft)
31	Sub 2340	2	81	1.23	4.79	17.41	8.00
		10	347	2.65	7.29	25.91	8.00
		100	851	4.11	9.28	34.65	8.00
32	Sub 2260	2	19	0.59	2.32	19.80	6.00
		10	79	1.17	3.41	31.48	6.00
		100	273	2.04	4.70	48.88	6.00
33	Sub 2250	2	334	1.26	3.98	80.61	7.00
		10	1144	2.46	5.81	107.13	7.00
		100	2610	3.77	7.32	136.03	7.00
34	Sub 1500	2	299	2.26	3.86	66.73	5.50
		10	841	3.37	4.95	99.98	5.50
		100	2062	4.72	6.20	140.56	5.50
35	Sub 1320	2	255	2.29	7.28	17.58	2.50
		10	679	4.11	10.03	18.00	2.50
		100	1526	7.02	12.70	18.00	2.50
36	Sub 1310	2	510	2.25	7.75	31.50	2.50
		10	1359	4.06	10.98	32.00	2.50
		100	3052	6.82	14.39	32.00	2.50
37	Sub 1220	2	184	0.77	4.70	51.54	4.50
		10	521	1.48	6.86	52.95	4.50
		100	1519	2.99	9.59	55.98	4.50
38	Sub 1480	2	374	2.22	6.32	29.33	2.50
		10	1135	4.30	9.33	30.00	2.50
		100	2931	7.92	12.73	30.00	2.50
39	Sub 2230	2	6	0.05	0.50	250.02	5.50
		10	29	0.12	0.94	250.04	5.50
		100	134	0.31	1.74	250.11	5.50

TABLE 4.5

Approximate Inflow Volumes (ac. ft.) to CAP Detention Basins
100-Year, 24-Hour Storm
North Scottsdale General Drainage Plan

	Approximate CAP Reach & Contributing Sub-Basins																			
	32nd Street to 48TH Street		48th Street to Scottsdale Road			Scottsdale Road to Pima Road		East of Pima Road to Sub-Basin 2340												
Land Use	SUB 6	SUB 10	SUB 14	SUB 29	SUB 33	SUB 38	SUB 53	SUB 2110	SUB 2120	SUB 2160	SUB 2170	SUB 2220	SUB 2240	SUB 2280	SUB 2290	SUB 2310	SUB 2320	SUB 2335	1/2 of SUB 2340
Existing Conditions	801	502	392	1986	563	599	871	1183	124	346	40	468	53	210	71	160	40	214	26
Total:	1303 AF		2941 AF			1470 AF		2935 AF											
Future Conditions	892	566	469	2255	650	721	982	1317	164	406	52	489	74	238	79	164	46	231	39
Total:	1458 AF		3374 AF			1703 AF		3299 AF											
Maximum Density Buildout	926	587	491	2297	677	755	1020	N/A											
Total:	1513 AF		3465 AF			1775 AF													

TABLE 5.1

SUMMARY OF CASE 1 - ON-SITE RETENTION BASIN PERFORMANCE
(FUTURE CONDITIONS, 100-YEAR EVENT)

Basin Designation	Basin Location	Drainage Area (sq mi)	Basin Volume (ac ft)	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Discharge Comparison (cfs) @ Downstream Location, Existing Conditions, No Retention vs. MDB Conditions with Retention
DB 9/10	Outlet of Sub 21A & Sub 21B	8.19	578.8	10,100	1291	@ Sub 21, 5492 vs. 1510
DB 11	Sub 1480	2.99	211.3	5101	1736	@ Sub 1480, 4124 vs. 1736
DB 12	Sub 2A	1.05	74.2	1256	371	@ Sub 2, 913 vs. 367
DB 13	Outlet of Sub 24 & 25	3.76	265.7	6378	949	@ Sub 26, 6097 vs. 970
DB 22	Outlet of Sub 41, 42, 47	7.50	530.0	15,041	2860	@ Sub 48, 13,137 vs. 2825
DB 24	Sub 36A	0.65	45.9	665	82	N/A, routing schematic is significantly different from existing conditions
DB 25	Sub 34A	1.13	79.9	1574	133	N/A, routing schematic is significantly different from existing conditions
DB 26	Sub 30	1.50	106.0	2214	183	N/A, routing schematic is significantly different from existing conditions

TABLE 5.2

SUMMARY OF CASE 1 - ON-SITE RETENTION BASIN PERFORMANCE (MAXIMUM DENSITY BUILDOUT SCENARIO, 100-YEAR EVENT)

Basin Designation	Basin Location	Drainage Area (sq mi)	Basin Volume (ac-ft)	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Discharge Comparison (cfs) @ Downstream Location, Existing Conditions, No Retention vs. MDB Conditions with Retention
DB 22	Outlet of Sub 41,42, 47	7.50	530.0	15,398	3253	@ Sub 48, 13,137 vs. 3250
DB 24	Sub 36A	0.65	45.9	706	106	N/A, routing schematic is significantly different from existing conditions.

TABLE 5.3

SUMMARY OF CASE 2 - DETENTION BASIN PERFORMANCE
(FUTURE CONDITIONS, 100-YEAR EVENT)

Basin Designation	Basin Location	Drainage Area (sq mi)	Maximum Height of Reservoir Water Surface (ft)	Maximum Storage During Routing (ac-ft)	Maximum Area of Surface Inundation (ac)	24-Hour Inflow Volume (ac-ft)	Q_{in} (cfs)	Q_{out} (cfs)
DB 4	Sub 1180	0.32	9.22	31	9.2	42	777	14
DB 5	Sub 1190	1.05	21.43	106	13.8	137	1703	46
DB 6	Outlet of Sub 1430 & 1440	3.32	30.97	333	31.2	444	4596	149
DB 19	Sub 1310	2.71	29.77	170	20.2	242	3175	81
DB 29	Sub 1250	N/A, upstream diversions	16.59	192	44.0	255	2192	86
DB 18	Sub 1230	2.36	26.51	231	20.9	315	3481	106
DB 7	Sub 1475	1.99	25.73	231	30.7	299	3633	100
DB 16	Sub 1390	2.39	25.48	242	27.2	337	3342	113
DB 17	Sub 1340	1.55	29.73	163	13.4	219	2618	74
DB 11	Sub 1480	2.99	26.95	339	44.0	449	5101	151
DB 15	Sub 1450	4.09	37.13	397	32.1	538	4644	181
DB 8	Outlet of Sub 19 & 20	5.97	36.94	523	40.9	803	9487	270
DB 12	Sub 2A	1.05	20.03	98	14.0	150	1256	50
DB 13	Outlet of Sub 24 & 25	3.76	29.24	334	29.9	511	6378	172
DB 9/10	Outlet of Sub 21A & 21B	8.19	26.39[1]	689[1]	65.7[1]	1071[1]	10,000[1]	360

TABLE 5.3 (CONTINUED)

SUMMARY OF CASE 2 - DETENTION BASIN PERFORMANCE
(FUTURE CONDITIONS, 100-YEAR EVENT)

Basin Designation	Basin Location	Drainage Area (sq mi)	Maximum Height of Reservoir Water Surface (ft)	Maximum Storage During Routing (ac-ft)	Maximum Area of Surface Inundation (ac)	24-Hour Inflow Volume (ac-ft)	Q_{in} (cfs)	Q_{out} (cfs)
DB 20	Outlet of Sub 39 & 40	2.53	26.21	253	23.3	378	5939	127
DB 21	Outlet of Sub 45, 45A, 46	3.23	50.09	322	14.3	478	7618	161
DB 22	Outlet of Sub 41, 42, 47	7.50	52.49[2]	729[2]	38.3[2]	1092[2]	15,041[2]	367
DB 24	Sub 36A	0.65	11.91	47	8.8	75	665	25
DB 25	Sub 34A	1.13	12.16	83	15.1	129	1574	43
DB 26	Sub 30	1.50	15.70	109	16.3	149	2214	50

Notes:

1) Detention basin DB 8 is not in-place.
2) Detention basins DB 20 and DB 21 are not in-place.

TABLE 5.4

SUMMARY OF CASE 2 - DETENTION BASIN PERFORMANCE
(MAXIMUM DENSITY BUILDOUT SCENARIO, 100-YEAR EVENT)

Basin Designation	Basin Location	Drainage Area (sq mi)	Maximum Height of Reservoir Water Surface (ft)	Maximum Storage During Routing (ac-ft)	Maximum Area of Surface Inundation (ac)	24-Hour Inflow Volume (ac-ft)	Q_{in} (cfs)	Q_{out} (cfs)
DB 8	Outlet of Sub 19 & 20	5.97	37.32	539	43.8	821	9807	270
DB 12	Sub 2A	1.05	20.35	102	14.5	155	1321	50
DB 13	Outlet of Sub 24 & 25	3.76	29.52	342	30.2	520	6527	172
DB 22[1]	Outlet of Sub 41, 42, 47	7.50	53.17	754	39.3	1121	15,398	367

Notes:

1) Detention basins DB 20 and DB 21 are not in-place.

TABLE 5.5

SUMMARY OF CASES 1, 3, AND 4 CHANNEL HYDRAULIC CALCULATIONS
(FUTURE CONDITIONS, 100-YEAR EVENT)

CHANNEL "A" - SCOTTSDALE ROAD

Approximate Channel Reach	Case	HEC-1 Concentration Point	Q_{100} (cfs)	Channel Slope (ft/ft)	Channel Bottom-Width (ft)	Flow Depth (ft)	Velocity (fps)	Froude Number
Scottsdale Road, Sub-Basin 1490A	1	1490A	936	.0144	50	1.77	9.89	1.35
	3		2668			3.28	14.39	1.48
	4		519			1.24	7.94	1.28
Scottsdale Road, Sub-Basin 1500A	1	1500A3	1716	.0144	50	2.53	12.32	1.43
	3		4979			4.71	17.79	1.55
	4		1246			2.09	10.98	1.39
Scottsdale Road, Sub-Basin 4A	1	4A3	2059	.0144	50	2.82	13.15	1.45
	3		6947			5.70	19.84	1.59
	4		2329			3.03	13.73	1.46
Scottsdale Road, Sub-Basin 8A	1	8A3	2700	.0144	50	3.30	14.45	1.48
	3		7373			5.90	20.22	1.60
	4		2949			3.48	14.90	1.49
West of Scottsdale Road, Jomax to Happy Valley Road	1	8.8	3173	.0194	75	2.64	15.00	1.68
	3		7665			4.44	20.60	1.81
	4		3484			2.79	15.52	1.69
West of Scottsdale Road, Happy Valley Road to Pinnacle Peak Road	1	9.6	9878	.0194	75	5.15	22.50	1.85
	3		9890			5.15	22.50	1.85
	4		9894			5.15	22.50	1.85

TABLE 5.6

SUMMARY OF CASES 1, 3, AND 4 CHANNEL HYDRAULIC CALCULATIONS
(FUTURE CONDITIONS, 100-YEAR EVENT)

CHANNEL "B" - SCOTTSDALE ROAD

Approximate Channel Reach	Case	HEC-1 Concentration Point	Q_{100} (cfs)	Channel Slope (ft/ft)	Channel Bottom-Width (ft)	Flow Depth (ft)	Velocity (fps)	Froude Number
Scottsdale Road, Sub-Basin 12	1	Sub 12	1643	.0165	75	1.87	11.16	1.47
	3		1643			1.87	11.16	1.47
	4		1643			1.87	11.16	1.47
Scottsdale Road, Sub-Basin 28A	1	28A3	4165	.0165	75	3.25	15.73	1.60
	3		12,255			6.12	22.96	1.75
	4		4534			3.42	16.22	1.61
Scottsdale Road, Sub-Basin 33A	1	33A3	6452	.0165	75	4.21	18.39	1.66
	3		14,067			6.63	24.05	1.77
	4		6889			4.37	18.82	1.67

TABLE 5.7

SUMMARY OF CASES 1, 3, AND 4 CHANNEL HYDRAULIC CALCULATIONS (FUTURE CONDITIONS, 100-YEAR EVENT)

CHANNELS "C", "D", AND "E" - PIMA ROAD

Approximate Channel Reach	Case	HEC-1 Concentration Point	Q_{100} (cfs)	Channel Slope (ft/ft)	Channel Bottom-Width (ft)	Flow Depth (ft)	Velocity (fps)	Froude Number
Pima Road, Sub-Basin 30	1	30.1	2304	.0215	30	3.54	17.57	1.80
	3		N/A			-	-	-
	4		2304			3.54	17.57	1.80
Pima Road, Sub-Basin 36D	1	36D2	1508	.0214	50	2.08	13.36	1.69
	3		5176		100	2.89	16.91	1.80
	4		1639		50	2.19	13.76	1.70
Pima Road, Sub-Basin 52A	1	52A2	5042	.0214	50	4.23	20.38	1.87
	3		10,917		100	4.51	22.23	1.92
	4		5329		50	4.37	20.77	1.88
Pima Road, Sub-Basin 53A	1	53A3	5037	.0214	50	4.23	20.38	1.87
	3		10,802		100	4.48	22.14	1.92
	4		5384		50	4.40	20.84	1.88
Under Pima Road and into Outer Loop Detention Basin 28	1	53.4	4374	.0089	50	5.02	14.51	1.23
	3		10,454		100	5.69	16.50	1.28
	4		4724		50	5.25	14.88	1.27
Pima Road, Sub-Basin 30	1	31	N/A			-	-	-
	3		2202	.0215	30	3.45	17.32	1.79
	4		N/A			-	-	-
Pima Road, Sub-Basin 36A	1	36.3	N/A			-	-	-
	3		4270	.0215	50	3.84	19.30	1.85
	4		N/A			-	-	-
Pinnacle Peak Road, Sub-Basin 36C	1	Sub 36C	1175	.0278	30	2.23	15.31	1.92
	3		1175			2.23	15.31	1.92
	4		1175			2.23	15.31	1.92

TABLE 5.8

SUMMARY OF CASES 1, 3, AND 4 CHANNEL HYDRAULIC CALCULATIONS
(MAXIMUM DENSITY BUILDOUT, 100-YEAR EVENT)

CHANNEL "A" - SCOTTSDALE ROAD

Approximate Channel Reach	Case	HEC-1 Concentration Point	Q_{100} (cfs)	Channel Slope (ft/ft)	Channel Bottom-Width (ft)	Flow Depth (ft)	Velocity (fps)	Froude Number
Scottsdale Road, Sub-Basin 1490A	1	1490A	1079	.0144	50	1.92	10.42	1.37
	3		2808			3.38	14.65	1.49
	4		546			1.28	8.10	1.29
Scottsdale Road, Sub-Basin 1500A	1	1500A3	2006	.0144	50	2.77	13.03	1.45
	3		5298			4.88	18.16	1.56
	4		1341			2.19	11.28	1.40
Scottsdale Road, Sub-Basin 4A	1	4A3	2385	.0144	50	3.07	13.84	1.47
	3		7393			5.91	20.24	1.60
	4		2514			3.17	14.10	1.47
Scottsdale Road, Sub-Basin 8A	1	8A3	2922	.0144	50	3.46	14.85	1.49
	3		7846			6.11	20.63	1.61
	4		3166			3.62	15.27	1.50
West of Scottsdale Road, Jomax to Happy Valley Road	1	8.8	3573	.0194	75	2.83	15.66	1.70
	3		8224			4.62	21.11	1.82
	4		3747			2.91	15.94	1.71
West of Scottsdale Road, Happy Valley to Pinnacle Peak Road	1	9.6	11,053	.0194	75	5.50	23.38	1.87
	3		11,086			5.51	23.41	1.87
	4		11,111			5.51	23.43	1.87

TABLE 5.9

SUMMARY OF CASES 1, 3, AND 4 CHANNEL HYDRAULIC CALCULATIONS
(MAXIMUM DENSITY BUILDOUT, 100-YEAR EVENT)

CHANNEL "B" - SCOTTSDALE ROAD

Approximate Channel Reach	Case	HEC-1 Concentration Point	Q_{100} (cfs)	Channel Slope (ft/ft)	Channel Bottom-Width (ft)	Flow Depth (ft)	Velocity (fps)	Froude Number
Scottsdale Road, Sub-Basin 12	1	Sub 12	1808	.0165	75	1.98	11.57	1.48
	3		1808			1.98	11.57	1.48
	4		1808			1.98	11.57	1.48
Scottsdale Road, Sub-Basin 28A	1	28A3	4511	.0165	75	3.41	16.19	1.61
	3		12,751			6.26	23.27	1.75
	4		4934			3.59	16.72	1.62
Scottsdale Road, Sub-Basin 33A	1	33A3	7052	.0165	75	4.43	18.98	1.67
	3		14,558			6.76	24.33	1.77
	4		7453			4.58	19.35	1.68

TABLE 5.10

SUMMARY OF CASES 1, 3, AND 4 CHANNEL HYDRAULIC CALCULATIONS
(MAXIMUM DENSITY BUILDOUT, 100-YEAR EVENT)

CHANNELS "C", "D", AND "E" - PIMA ROAD

Approximate Channel Reach	Case	HEC-1 Concentration Point	Q_{100} (cfs)	Channel Slope (ft/ft)	Channel Bottom-Width (ft)	Flow Depth (ft)	Velocity (fps)	Froude Number
Pima Road, Sub-Basin 30	1	30.1	2396	.0215	30	3.62	17.79	1.80
	3		N/A			-	-	-
	4		2396			3.62	17.79	1.80
Pima Road, Sub-Basin 36D	1	36D2	1216	.0214	50	1.84	12.35	1.66
	3		5294		100	2.93	17.05	1.80
	4		1216		50	1.84	12.35	1.66
Pima Road, Sub-Basin 52A	1	52A2	5086	.0214	50	4.25	20.44	1.87
	3		11,668		100	4.69	22.77	1.93
	4		5207		50	4.31	20.60	1.87
Pima Road, Sub-Basin 53A	1	53A3	5097	.0214	50	4.26	20.46	1.87
	3		11,540		100	4.66	22.68	1.93
	4		5194		50	4.30	20.59	1.87
Under Pima Road and into Outer Loop Detention Basin 28	1	53.4	4865	.0089	50	5.34	15.02	1.24
	3		11,247		100	5.94	16.93	1.29
	4		5011		50	5.43	15.17	1.25
Pima Road, Sub-Basin 30	1	31	N/A			-	-	-
	3		2299	.0215	30	3.53	17.56	1.80
	4		N/A			-	-	-
Pima Road, Sub-Basin 36A	1	36.3	N/A			-	-	-
	3		4447	.0215	50	3.93	19.57	1.85
	4		N/A			-	-	-
Pinnacle Peak Road, Sub-Basin 36C	1	Sub 36C	1049	.0278	30	2.08	14.73	1.90
	3		1049			2.08	14.73	1.90
	4		1049			2.08	14.73	1.90

TABLE 6.1

VOLUMES OF STORM WATER RUNOFF FOR NORTHERN SCOTTSDALE

BASIN IDENTIFIER	STORAGE (ACRE FEET)	24 HR. INFLOW (ACRE FEET)	Q IN (CFS)	Q OUT (CFS)
DETENTION				
~~DB-8~~	539	821	9807	270
DB-9/10	689	1071	10100	360
DB-11	339	449	5101	151
DB-12	98	150	1256	50
DB-13	334	511	6378	172
DB-22	729	1029	15041	367
DB-24	47	75	1574	43
DB-26	109	149	2214	50
TOTALS	2884			
RETENTION				
DB-9/10	578.8	n/a	10100	1291
DB-11	211.3	n/a	5101	1736
DB-12	74.2	n/a	1256	371
DB-13	265.7	n/a	6378	949
DB-22	530.0	n/a	15041	2860
DB-24	45.9	n/a	665	82
DB-25	79.9	n/a	1574	148
DB-26	106.0	n/a	2214	188
TOTALS	1891.8			

CHANNELS

	EXISTING CONDITIONS	FUTURE CONDITIONS	MAXIMUM DENSITY BUILDOUT
48TH STREET TO SCOTTSDALE ROAD	2941	1458	3465
SCOTTSDALE ROAD TO PIMA ROAD	1470	3574	1775
EAST OF PIMA ROAD	2935	1703	n/a
TOTALS (AF)	7346	6735	5240

TABLE 7.1

COST ESTIMATE SUMMARY

Case	Drainage Improvement Alternative	Cost
1	Channelization and On-Site Retention	$68,953,621
2	Detention Basins Only	$98,879,548
3	Channelization Only	$64,708,734
4	Channelization and Detention Basins	$90,410,801

TABLE 7.2

NORTH SCOTTSDALE DRAINAGE STUDY
COST ESTIMATES

CASE 1

(1) Basin	(2) Fill, cy	(3) Fill Cost	(4) Rip Rap, cy	(5) Rip Rap Cost	(6) Basin Ease.	(7) Chann.	(8) Channel Cost	(9) Channe Ease.
910	53,763	$268,815	6,193	$123,860	28	A	$6,331,251	64
11	13,055	65,275	1,468	29,360	13	B	5,713,386	58
12	14,245	71,225	1,973	39,460	12	C	4,322,246	43
13	14,889	74,445	2,029	40,580	12	D & E	1,379,217	16
25	39,169	195,845	4,887	97,740	19			
26	33,359	166,795	4,223	84,460	18			
22	14,674	73,370	1,690	33,800	6			
24	30,014	150,070	3,782	75,640	12			
		$1,065,840		$524,900	120		$17,746,100	181

Total Easement = 301 acres

TABLE 7.3

NORTH SCOTTSDALE DRAINAGE STUDY
COST ESTIMATES

CASE 2

(1) Basin	(2) Fill, cy	(3) Fill Cost	(4) Rip Rap, cy	(5) Rip Rap Cost	(6) Basin Ease.	(7) Chann	(8) Channel Cost	(9) Channe Ease.
910	210,601	$1,053,005	14,749	$294,980	68			
11	95,708	478,540	7,093	141,860	60			
12	52,089	260,445	4,884	97,680	19			
13	115,921	579,605	9,282	185,640	39			
25	52,206	261,030	5,918	118,360	22			
26	69,285	346,425	7,160	143,200	19			
22	317,692	1,588,460	16,586	331,720	13			
24	30,014	150,070	3,782	75,640	12			
4	28,150	140,750	3,407	68,140	24			
5	43,960	219,800	4,165	83,300	13			
18	134,898	674,490	10,317	206,340	29			
20	109,707	548,535	8,539	170,780	31			
21	102,904	514,520	4,655	93,100	10			
19	93,159	465,795	7,262	145,240	20			
29	164,539	822,695	14,845	296,900	64			
6	80,149	400,745	6,612	132,240	39			
7	43,794	218,970	3,660	73,200	33			
16	118,442	592,210	9,245	184,900	33			
17	125,416	627,080	9,520	190,400	23			
15	247,838	1,239,190	15,567	311,340	45			
8	147,462	737,310	10,328	206,560	48			
		$11,919,670		$3,551,520	664		$0	0

Total Easement = 664 acres

Easement Cost per Sq Ft = $2.75

Engineering & Contingencies 25%

Basins Cost	$15,471,190
Channels Cost	0
Structures Cost	0
Total	$15,471,190
Engineering & Contingencies	3,867,798
Easement Cost	79,540,560
	$98,879,548

TABLE 7.4

NORTH SCOTTSDALE DRAINAGE STUDY
COST ESTIMATES

CASE 3

(1) Basin	(2) Fill, cy	(3) Fill Cost	(4) Rip Rap, cy	(5) Rip Rap Cost	(6) Basin Ease.	(7) Chann.	(8) Channel Cost	(9) Channe Ease.
						A	$6,331,251	64
						B	5,713,386	58
						D	10,112,390	90
						E	601,502	7
		$0		$0	0		$22,758,529	219

Total Easement = 219 acres

Easement Cost per Sq Ft =	$2.75	Basins Cost	$ 0
		Channels Cost	22,758,529
		Structures Cost	8,021,250
Engineering & Contingencies	25%	Total	$ 30,779,779
		Engineering & Contingencies	7,694,945
		Easement Cost	26,234,010
			$ 64,708,734

TABLE 7.5

NORTH SCOTTSDALE DRAINAGE STUDY
COST ESTIMATES

CASE 4

(1) Basin	(2) Fill, cy	(3) Fill Cost	(4) Rip Rap, cy	(5) Rip Rap Cost	(6) Basin Ease.	(7) Chann	(8) Channel Cost	(9) Channe Ease.
910	210,601	$1,053,005	14,749	$294,980	68	A	$6,331,251	64
11	95,708	478,540	7,093	141,860	60	B	5,713,386	58
12	52,089	260,445	4,884	97,680	19	C	4,322,246	43
13	115,921	579,605	9,282	185,640	39	D & E	1,379,217	16
25	52,206	261,030	5,918	118,360	22			
26	69,285	346,425	7,160	143,200	19			
22	317,692	1,588,460	16,586	331,720	13			
24	30,014	150,070	3,782	75,640	12			
		$4,717,580		$1,389,080	252		$17,746,100	181

Total Easement = 433 acres

Easement Cost per Sq Ft = $2.75

Engineering & Contingencies 25%

Basins Cost	$ 6,106,660
Channels Cost	17,746,100
Structures Cost	6,980,625
Total	$ 30,833,385
Engineering & Contingencies	7,708,346
Easement Cost	51,869,070
	$ 90,410,801

PLATES