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Albuquerque
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Control
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2600 PROSPECT N.E. - ALBUQUERQUE, N.M. 87107
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April 25, 1989

RECEIVED
APR 2 6 1989
HYDROLOGY SECTION

Lt. Col. Kent Gonser
US Army Corps of Engineers
P.O. Box 1580
Albuquerque, New Mexico 87103-1580

Attention: Ben Alanis

RE: Hydrologic Analysis of the South Pino Drainage Area

Enclosed for your information is subject study done for AMAFCA by Bohannan-Huston, Inc. Its purpose was to determine how the Standard Project Storm (SPS) might affect the South Pino drainage area, given the substantial changes which have occurred since the North Diversion Channel (NDC) was built.

The study also was to determine if project lands acquired for the NDC are still needed. They are. However, at some future time, we may want to explore measures along the north portion of the South Pino west of I-25 to reduce, control or otherwise accommodate SPS flows.

Although the report is considered final, any comments you might have will be duly recorded and considered.

Sincerely,

Larry A. Blair,
Executive Engineer

Copy:

Dwayne Sheppard, City of Albuquerque, Hydrology, PWD
Fred Aguirre, City of Albuquerque, Hydrology, PWD

HYDROLOGIC ANALYSIS
OF THE
SOUTH PINO DRAINAGE AREA

APRIL, 1989

RECEIVED
APR 26 1989
HYDROLOGY SECTION

PREPARED FOR:

AMAFCA
2600 PROSPECT, N.E.
ALBUQUERQUE, NM 87125

PREPARED BY:

BOHANNAN-HUSTON, INC.
COURTYARD I
7500 JEFFERSON ST., N.E.
ALBUQUERQUE, NM 87109

JOB NO. 88232.05

INTRODUCTION

Authority

In October of 1988, the Albuquerque Metropolitan Flood Control Authority (AMAFCA) authorized Bohannan-Huston Inc. (BHI) to proceed with a study of the Standard Project Storm (SPS) for the South Pino Drainage Basin. This report presents the findings and recommendations for the study area.

Scope

Hydrologic studies of the South Pino Arroyo have been performed in the past for the 100-year return storm event. The North Diversion inlet for the South Pino, however, is designed to carry the SPS discharge to meet Army Corps of Engineers (COE) criteria. Since AMAFCA is considering various projects in the vicinity of the inlet including concrete channelization of the South Pino all the way to the North Diversion, it was judged prudent to analyze the arroyo performance during the SPS.

The scope includes development of a HEC-1 (developed at the Hydrologic Engineering Center, Davis, California) computer model to simulate basin rainfall and runoff as well as routing of the resultant flows to trace their path to the North Diversion Channel. Channels and important crossing structures will be analyzed for possible flow breakouts. Finally, structural improvements are recommended where needed.

Description of the South Pino Watershed

The South Pino watershed is an 8.3 square mile drainage basin beginning at the crest of the Sandia Mountains and discharging into the North Diversion Channel (see Figure 1). The watershed can be divided by geographic features and land use as follows:

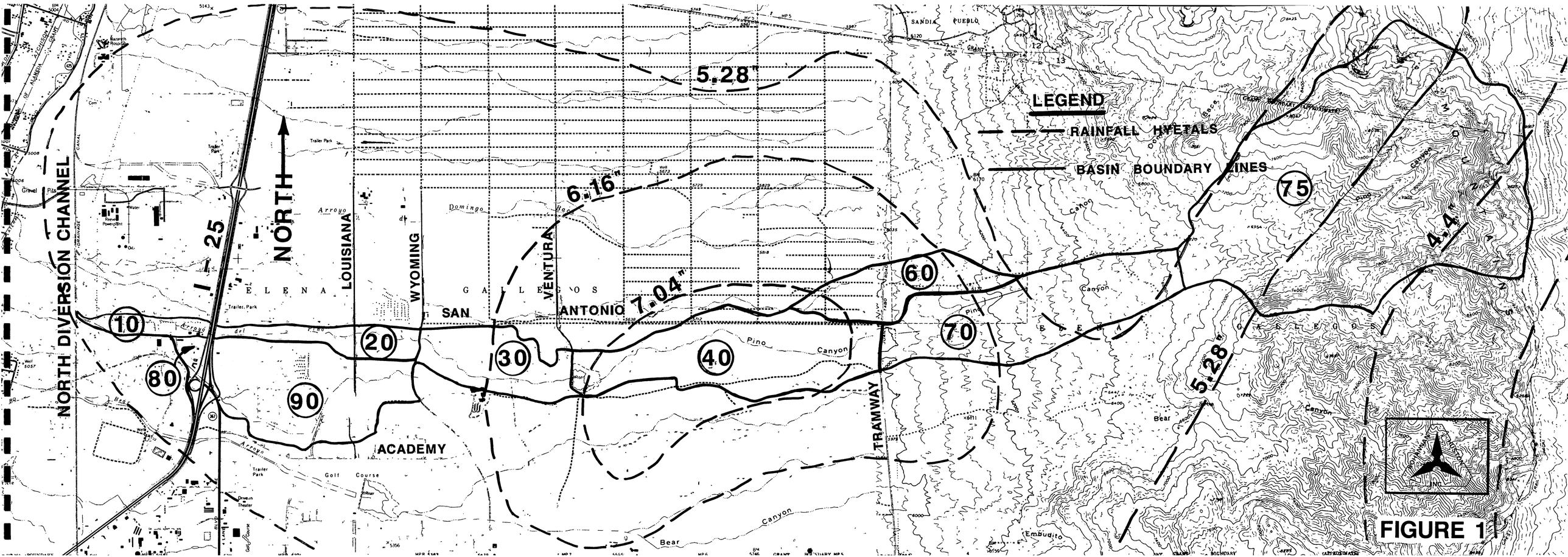

LEGEND
RAINFALL HYETALS
BASIN BOUNDARY LINES
5.28"
6.16"
7.04"
5.28"
4.4"
NORTH
NORTH DIVERSION CHANNEL
I-25
LOUISIANA
WYOMING
VENTURA
SAN ANTONIO
ACADEMY
TRAMWAY
ELENA GALLEGOS
BEAR CANYON
10
20
30
40
60
70
75
80
90

FIGURE 1

National Forest

The upper 3.8 square miles (Basin 75) are characterized by the extremely steep western slopes of the Sandia Mountains and are located within the Cibola National Forest. No development is expected in this area, and native forest vegetation will characterize the land use in the future.

Sandia Heights Low Density Development

From the forest service boundary west to Tramway Boulevard, the Sandia Heights Subdivision is characterized by low density residential development and an emphasis on natural landscaping and minimal disturbance of the desert environment (see Basins 60 and 70). Slopes are steep (5%), but impervious areas due to development are limited. Low density development throughout the basins is expected in the future.

Tanoan Subdivision

Between Tramway Boulevard and Ventura Street the watershed consists of the Tanoan Golf Course and the surrounding single and multi-residential units. The Pino arroyo flows through the golf course, and residences are constructed outside of the 100-year flood plain. The majority of the area is characterized by irrigated fairway, with a smaller portion dedicated to medium and high density housing. This area is shown as Basin 40 on Figure 1.

The Albuquerque Academy

From Ventura Street downstream to Wyoming Boulevard, the Pino arroyo flows through the grounds of the Albuquerque Academy in a natural earthen channel. The Academy basin (Basin 30) consists of a fairly impervious campus grounds surrounded by the natural east mesa

desert environment. A large portion of the Academy land was slated for development, but, with the advent of a new head administrator, the Academy has changed its philosophy, and now intends to keep the remaining portion in a pristine condition. This report assumes no future development in this basin beyond that needed for Academy facilities.

Wyoming to Interstate Highway 25

This area (Basins 20 and 90) is rapidly being developed, and total development is assumed in the future. The land use is single and multiple residences with some commercial development. Total concrete channelization of the Pino and Borealis arroyos running through the area is expected in the near future.

I-25 to the North Diversion Channel

This downstream portion of the watershed (Basins 10 and 80) is a rapidly developing commercial area consisting of light industries and offices. The Borealis arroyo runs through Basin 80 and joins the Pino in Basin 10, where the combined flow continues westward to the dike wingwalls of the North Diversion Channel inlet of the South Pino. The area is expected to see complete development in the future except for parks and flood easements.

Hydraulic History of the South Pino

Human intervention in the historic flow paths of the east side arroyos has been common since development began on the east mesa. Major impacts to the South Pino arroyo that affect this report are listed chronologically below. Some dates are estimated.

1965-1967	Construction of I-25
1965-1967	Diversion of the Beta arroyo (Borealis) to the South Pino
1967	Construction of the North Diversion Channel
1978	Construction of new Pino Dam (Tramway Flood Retention Facility)
1978	Diversion of North Pino to South Pino below Tramway Blvd.
1981-1982	Removal of old Pino Dam

These actions affect the modeling of the South Pino for 1967, 1988 and ultimate development scenarios and were taken into account while building the computer models for the watershed. The construction of the new Pino Dam has a large impact on arroyo flowrates.

Modeling Objectives

The hydrologic and hydraulic analyses performed for the design of the North Diversion Channel inlet of the South Pino is no longer available. The COE Design Memo #5 states that the maximum discharge expected at the inlet for the SPS event is 6,900 cubic feet per second (cfs). The SPS used in the original study was the transposed Las Cruces storm of 1935.

In order to simulate the original study, AMAFCA directed BHI to construct a computer model using HEC-1 and calibrate it to agree approximately with the Design Memo #5 flow rate of 6,900 cfs at the North Diversion Channel for watershed conditions at the time of the North Diversion construction. Once calibrated, the model would then be changed to reflect 1988 and ultimate development land use and to incorporate any hydraulic changes such as the construction of the new Pino Dam. These objectives have been followed in the analysis, and the results are presented in the following sections.

HYDROLOGY

The construction and calibration of the computer model is presented in this section along with discharge results.

Rainfall.

As stated in the scope, the event to be analyzed in this study is the Standard Project Storm (SPS). The rainfall for this storm is usually obtained from a survey of severe historic storms in the general vicinity of the drainage basin. Alternately, severe storms experienced in meteorologically similar areas can be transposed over the study area. For areas where no appropriate historic storm can be found, the COE suggests its version of the SPS, which is a percentage of the COE synthesized Probable Maximum Precipitation (PMP).

The North Diversion study capsulized in Design Note #5 used the August 29-30, 1935 Las Cruces, New Mexico storm as the SPS, reducing the isohyets 12 percent to account for altitude gain. (Typically, rainfall is reduced 6-10% for each 1000' gain in altitude.) In keeping with this precedent, the Las Cruces storm was selected as appropriate for the present study, including the 12% reduction. The rotation of the isohyets and the reduced precipitation for the Pino study are shown on Figure 1. The distribution is shown graphically on Figure 2 and numerically (for a 7.84" storm) in the appendix. With the concurrence of the COE, a six-hour duration storm was chosen as appropriate.

On the request of the COE, we also analyzed the watershed runoff using the COE synthetic Standard Project Storm. The distribution is shown graphically on Figure 2 and numerically in the appendix. The total precipitation (4.8 inches) represents 40% of the PMP presented in the COE Hydrometeorological Report #55A. This precipitation was applied uniformly across the watershed.

RAINFALL DISTRIBUTION

INCREMENTAL RAINFALL VARIES FROM BASIN TO BASIN FOR LAS CRUCES STORM

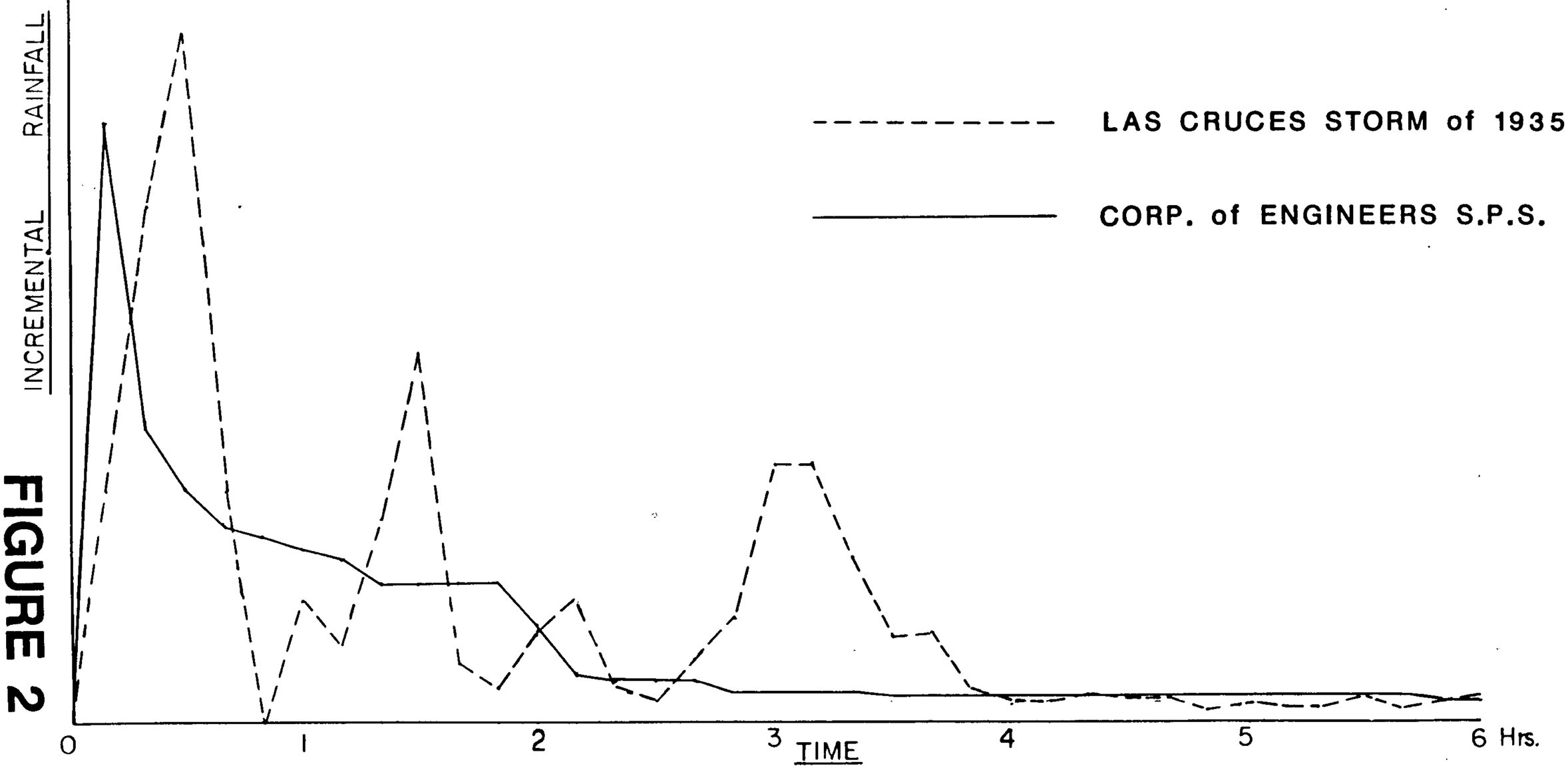


FIGURE 2

Snyder Synthetic Unit Hydrograph

The design study for the North Diversion Channel used the Snyder Synthetic Unit Hydrograph approach to predict runoff. In order to replicate the original study as closely as possible, the Snyder method was chosen for this study. This method relies upon correlation of the dependent variables of lag time and peak discharge with various physiographic watershed characteristics. The lag-time relationship derived by Snyder is:

$$tp = t1 + .25(tR-tr) \qquad \text{where}$$

$$tR = .083$$
$$tr = t1/5.5$$
$$t1 = Ct(LLca)^{.3}$$

tp = the adjusted lag time (hr)
t1 = the lag time (hr)
Ct = the coefficient representing variations of watershed slopes and storage
L = length of the main stream channel (mi) from the outlet to the divide
Lca = length along the main channel to a point nearest the watershed centroid (mi)

The coefficient Ct was obtained from the COE study of basins in Arizona and New Mexico (see Plate B-3 in the appendix). Snyder's peaking coefficient Cp was taken from the same study (see Plate B-3).

HEC-1 Computer model

The COE's Flood Hydrograph Package "HEC-1" was chosen for modeling the watershed. Since the Snyder method does not produce a complete hydrograph, HEC-1 uses the Clark method to affect a Snyder

unit graph. The initial Clark parameters are estimated from the Snyder's parameters, Tp and Cp. Details of the Clark method can be found in the HEC-1 user's manual.

Initial and Constant Abstractions

Initial and continued abstractions must be subtracted from the precipitation to determine excess rainfall. In this study we use a uniform rate loss dependent on the Soil Conservation Service's hydrological soil groups. Table 1 shows the initial and constant abstractions used in the report. Soil groups were obtained from the Bernalillo County Soil Survey.

Table 1

Initial Abstractions

Undeveloped basins	0.50 in.
Developed basins	0.40 in.

Constant Abstractions

Hydrologic Soil Group	A	0.40 in/hr
Hydrologic Soil Group	B	0.25 in/hr
Hydrologic Soil Group	C	0.10 in/hr
Hydrologic Soil Group	D	0.02 in/hr

Basin Development

Development within the watershed is reflected in several ways in the HEC-1 model. The Snyder's coefficient Ct is dependent on development (see Plate B-3 in the appendix). The initial rainfall abstractions shown in Table 1 vary with development. The model also requires input of the percentage of impervious area with the basin, as precipitation losses are not computed for these areas. Imperviousness, defined as surfaces that do not allow infiltration of rainfall (roofs, driveways, streets), increases with development. Development, then, is very important in setting up an accurate model.

In the previous section entitled "Description of the South Pino Watershed," some general 1988 and ultimate development assumptions are stated. Table 2 presents the specific scenarios used in the models for urbanization and imperviousness.

Table 2

	% Urbanization			% Imperviousness		
Basin	**1967**	**1988**	**Ult. Develop.**	**1967**	**1988**	**Ult. Develop.**
10	0	20	90	0	8	36
20	0	50	80	0	15	24
30	0	40	50	0	12	15
40	0	50	60	0	15	18
60	0	30	60	0	6	12
70	0	10	30	0	1	3
75	0	0	0	0	0	0
80	30	50	70	12	20	28
90	40	95	95	12	29	29

Routing of Flows

Routing flows is an important aspect of watershed modeling, as it allows addition of hydrographs in correct time sequence and reflects peak attenuation caused by storage. The South Pino model simulates routing through open channels and through the new Pino Dam. A Modified Puls channel routing algorithm is used for the channel and a storage vs discharge routine for reservoir routing. Stage/storage/discharge information for the Pino Dam was obtained from the design plans of the Tramway Floodwater Retaining Structure.

Historic Watershed Changes

In the previous section entitled "Hydraulic History of the South Pino," hydraulic changes in the South Pino arroyo were chronologized. The two impacts reflected in modeling from the changes are the addi-

tion of the Pino Dam and the diversion of the North Pino into the South Pino in 1978. The model depicting conditions at construction of the North Diversion does not include the dam and diversion. Subsequent scenarios (1988 and ultimate development) do model the diversion (addition of Basin 60) and the Pino Dam.

Calibration of the Model

The watershed was modeled using existing conditions in 1967, the time of construction of the North Diversion channel. The Las Cruces storm rainfall was applied by placing the center of the storm near the center of the watershed. The model was then calibrated by moving the center of the storm and rotating the hyetals until the peak discharge at the North Diversion channel was approximately 6,900 cfs, the flowrate found in the original design study and presented in Design Note #5. This 1967 version was then used as the base model on which the Las Cruces storm 1988 and ultimate development scenarios and the COE SPS 1967, 1988 and ultimate development scenarios were built.

Summary of Discharges

Using the information presented in Tables 1 and 2, the 1967 model was adjusted to reflect 1988 and ultimate development conditions. These models were run to determine discharges. Additionally, the 1967, 1988 and ultimate development models were rerun using the rainfall from the COE SPS event. The discharges from these scenarios are summarized in Table 3.

Table 3

South Pino Standard Project Storm Discharges (cfs)

Location	Las Cruces Storm 1967	1988	Ultimate	COE SPS 1967	1988	Ultimate
S. Pino @ Tramway	4814	4845	4926	6355	6374	6429
Pino Dam Outflow	N/A	3091	3105	N/A	2599	2635
S. Pino @ I-25	5848	3641	3695	7462	2755	2862
Borealis @ I-25	1274	1831	1831	1251	1557	1557
S. Pino @ N. Div. Chan.	6910	4698	5414	8209	4533	4877

The model results presented in Table 3 assume that flows remain within the arroyo floodplains. The decrease in discharges at the North Diversion channel for the 1988 and ultimate development scenarios reflect the installation of the Pino Dam, which stores much of the flood waters and, thus, reduces the downstream peak flows.

COE SPS vs. Las Cruces SPS

A comparison of the COE SPS and the Las Cruces SPS flows at the North Diversion shows that for the 1976 scenario, the COE SPS produces approximately 20% higher flow than the Las Cruces SPS (8209 cfs vs 6910 cfs). However, for the 1988 and ultimate development scenarios, the COE SPS produces less runoff than the Las Cruces storm (4533 cfs vs 4698 cfs and 4877 cfs vs 5414 cfs). This seeming anomaly can be explained by comparing the rainfall distributions of the two storms shown in Figure 2. The synthetic COE rainfall has only one peak,

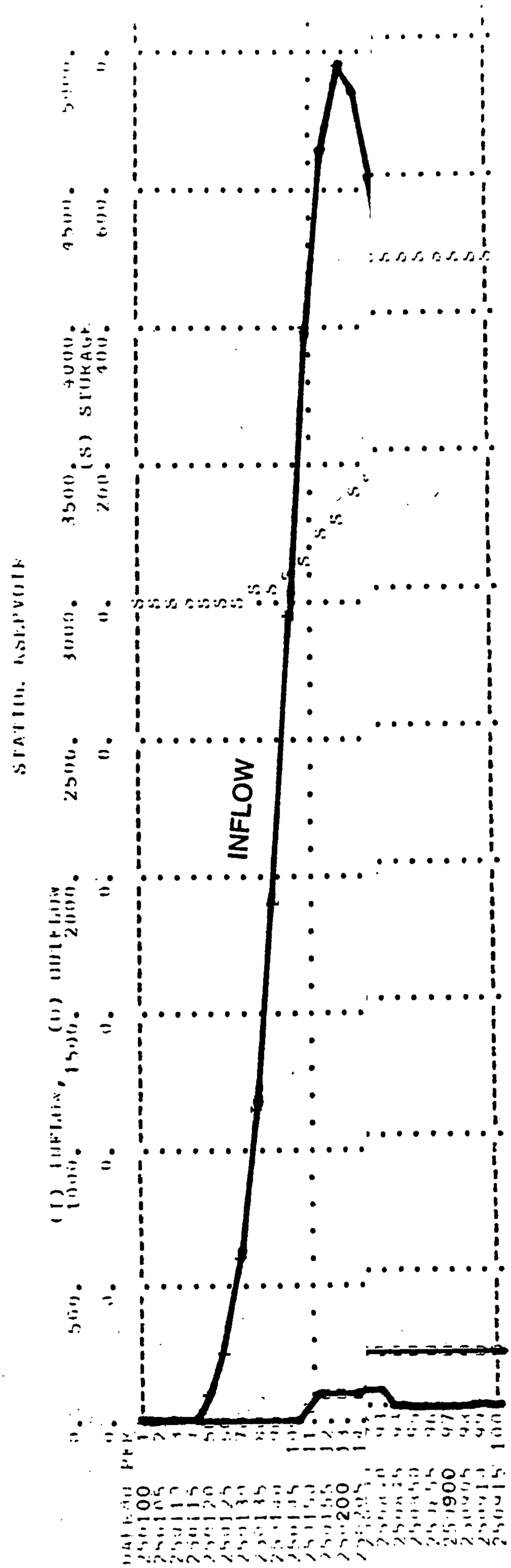
while the Las Cruces storm has three. The Pino Dam peak outflows for the 1988 and ultimate development scenarios of the Las Cruces storm are produced by the third peak of the Las Cruces rainfall.

This phenomenon is graphically illustrated by Figure 3, a plot of the Pino Dam inflow and outflow hydrographs for the ultimate development Las Cruces storm. The inflow hydrograph closely follows the rainfall distribution. The first two peaks fill the reservoir, and the emergency spillway starts to flow at 3 hours, 15 minutes on the downward side of the second inflow peak. Then, the third inflow peak hits the dam, and the emergency spillway discharge swells to an outflow peak of 3105 cfs at 4 hours, 45 minutes. The impact of the third inflow peak on a full reservoir produced the greatest outflow discharge of the storm.

Dissimilarly, the COE SPS outflow hydrograph at the Pino Dam (see Figure 4) peaks after the first (and only) inflow peak. The emergency spillway starts to flow at 2 hours, 40 minutes on the downward side of the inflow hydrograph and its discharge increases until hour 3:00. From this point in time on, the inflow is less than the outflow, so no additional outflow peak develops. So even though the COE SPS inflow peak is 20% greater than the Las Cruces storm inflow peak, the combination of the reservoir storage characteristics and the COE SPS runoff characteristics results in a smaller peak outflow.

HYDRAULIC ANALYSIS

The South Pino watershed is currently experiencing rapid development. This trend is expected to continue until ultimate development is reached, probably within the next 20 years. Only Basin 75, lying within the Cibola National Forest, is not expected to develop further. Because of this trend, it was deemed prudent to use the ultimate development scenario to analyze the hydraulic behavior of the watershed. Since the North Diversion Channel and all its inlets were

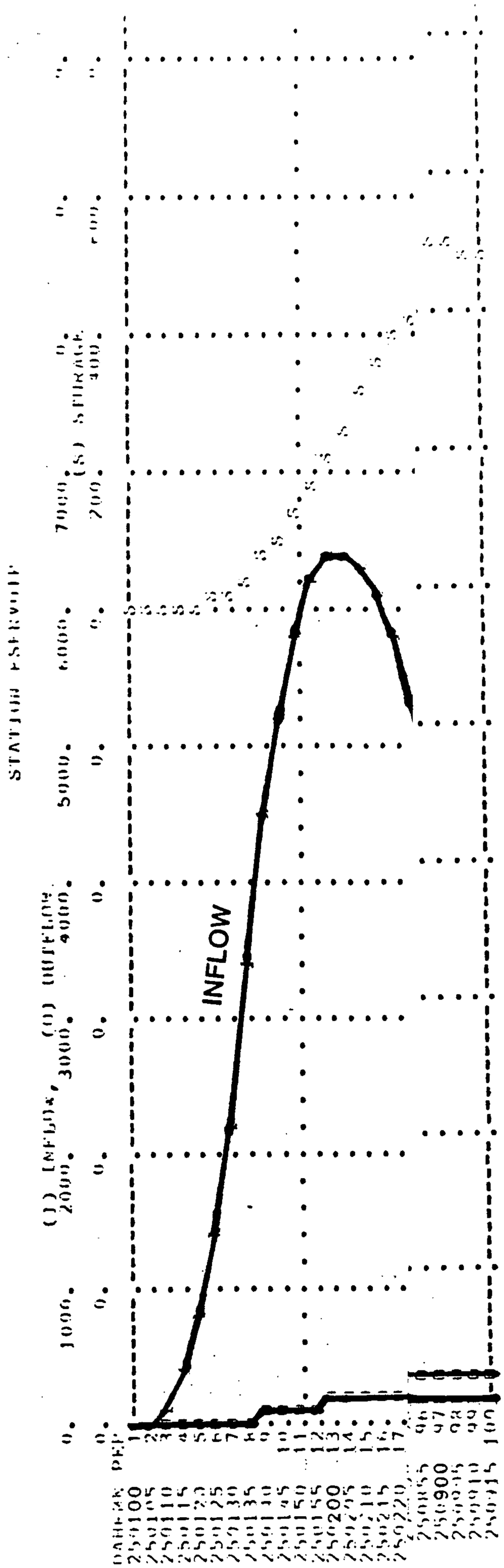


LAS CRUCES

-SPS-

ULTIMATE DEVELOPMENT

FIGURE 3



C.O.E.

-SPS-

ULTIMATE

DEVELOPMENT

FIGURE 4

designed using the Las Cruces SPS flows, it was further decided (with the concurrence of the COE) to use this rainfall for the current study. The Las Cruces ultimate development scenario, then, is used in all hydraulic analysis.

Of primary interest is the amount of flow that actually reaches the South Pino inlet to the North Diversion Channel. To determine this information, hydraulic locations where flow might possibly leave the watershed must be scrutinized. This study analyzed the capacity of channels and crossing structures to determine flow paths.

Channel Capacities

The main channels of the South Pino and Borealis, formally called the Beta, arroyos were analyzed for capacity. The majority of these reaches is currently concrete or grass lined. This study assumed that all portions of the main channels would be concrete or grass (Tanoan) lined to carry the 100-year storm flow except the Albuquerque Academy reach.

Channels were modeled using HEC-1, and the Las Cruces ultimate development flows were simulated. In all cases, the channel freeboard was adequate to contain the SPS flows. Tanoan and Academy flows were contained within the floodway. No breakout from the arroyo floodways is anticipated due to inadequate channel capacity.

Crossing Structure Capacities

All crossing structures analyzed in the study were found to be inadequate to carry the SPS. This is to be expected, as these structures are designed to carry the lesser 100-year discharge. SPS flow at the Ventura Street and Wyoming Boulevard crossings of the South Pino will overtop the streets, but will remain in the channel floodway. This is true for all other crossings of the South Pino and Borealis upstream of Interstate 25.

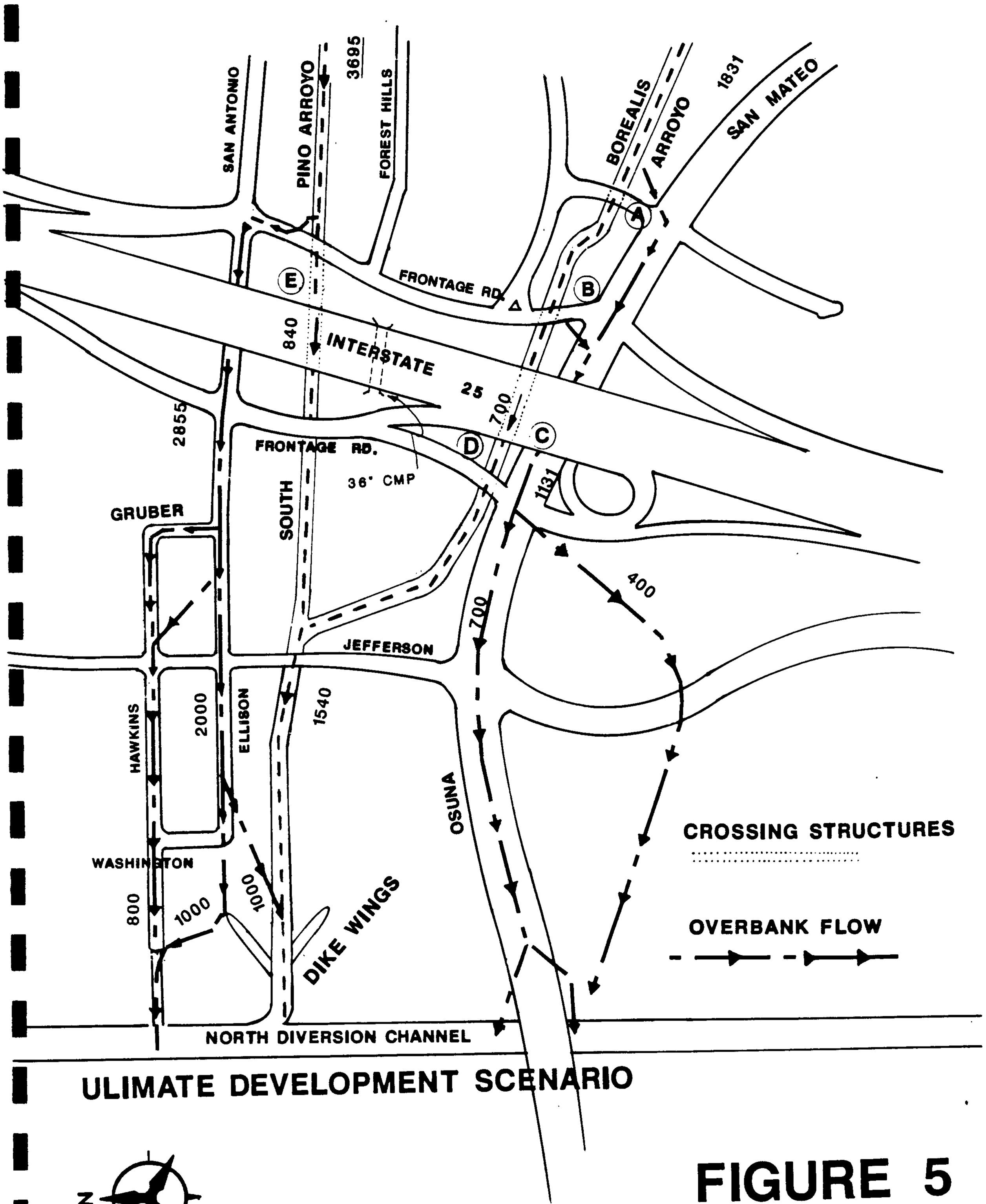
Five crossing structures were analyzed at I-25. They are identified as Structures A, B, C, D and E on Figure 5. Structures A, B, C and D carry flow from the Borealis Arroyo, while Structure E is located on the South Pino. The structures, all concrete box culverts, were analyzed for inlet control, as the inlets are unimproved and will choke the flow. The computed capacities are shown in Table 4 along with the SPS discharges upstream of the structures.

Table 4

Structure	Size	Headwater Ft.	Culvert	Capacity cfs	SPS Discharge cfs
A	2-4'x8'	4		720	1831
B	2-4'x8'	4		720	1831
C	8'x8'	3		700	1831
D	2-6'x6'	3		760	1831
E	8'x8'	4		840	3695

Obviously, the I-25 crossing structures do not have the capacity to carry the SPS flows. The Borealis flows (Structures A, B, C and D) will overflow southward to Osuna/San Mateo Boulevard where they will be directed west under I-25. Just west of I-25, the flow will split with the majority continuing westward down Osuna Road and the rest spilling south and west as shown in Figure 5. Both flows will be intercepted by the North Diversion Channel.

The South Pino overflow from Structure E will flow north to San Antonio/Ellison Road where it will turn west and cross under I-25. Figure 5 shows the likely flow paths of this flood water and the estimated peak discharges of each path. Some of the flow proceeding west on Ellison will cross northward to Hawkins via Gruber Avenue, Jefferson, Washington and interspersed driveways (approx. 800 cfs).



ULIMATE DEVELOPMENT SCENARIO

FIGURE 5

CROSSING STRUCTURES

Some of the remaining Ellison flow will be recaptured into the South Pino Watershed by the wing dike on the South Pino inlet to the North Diversion. The remainder of the flow will proceed to Hawkins and on to the North Diversion Channel.

In summary, the box culverts at I-25, while capable of carrying the 100-year flow, become constrictions for the SPS discharges. I-25, with its elevated profile, becomes a barrier to westward flow. The flow proceeds along the lower east frontage roads to the Osuna and San Antonio underpasses, where it can once more follow the natural slope westward. Osuna Road and Ellison Road then become the "channels" of the flood waters. Meanwhile, the remaining flow in the South Pino Channel and the Borealis Channel has been reduced to 1540 cfs (840+700) by the I-25 constrictions, This 1540 cfs is well below the 100-year design flow (2023 cfs) of the South Pino downstream of Jefferson.

RECOMMENDATIONS

Possible mitigating measures will be discussed in this section and summarized at the end. The I-25 crossing structures will be addressed first, followed by channels and street flow.

I-25 Crossing Structures

Structures C and E cross under I-25, and their upgrade by enlargement or by installing additional conduits would require either tunneling or a bore and jack technique underneath I-25. Both methods are extremely expensive, and run the risk of causing settlement of the I-25 surface above the penetrations. This option is considered unfeasible because of the cost and the unacceptable possible settlement. Structures A, B and D are located on frontage roads, and could be upgraded by removing the existing concrete box culverts (CBCs) and installing larger ones.

The crossing structures were analyzed for Table 4 assuming inlet control (choking at the inlet). The CBCs might be more efficient if supercritical (high velocity) flow could be maintained through the inlet of the boxes. This would involve improving the inlets to the boxes with hydraulically efficient transitions. A comparison is shown below between the existing inlets and the improved ones:

Structure	Inlet Control (cfs)	Manning's Eq. (Supercritical) (cfs)	SPS Discharge (cfs)
A	720	634	1831
B	720	634	1831
C	700	1206	1831
D	760	1431	1831
E	840	1490	3695

Structures C, D and E would carry considerably more flow with the inlet improvement, though not enough flow to carry the SPS discharge. Structures A and B achieve no improvement from inlet upgrade, however, as explained previously, these structures could be removed and replaced with culverts that could match the Structure C capacity of 1206 cfs. Such an upgrade would reduce the overflow to Osuna Road to 1831-1206 = 625 cfs for the SPS.

The inlet upgrade to Structure E would allow an additional 650 cfs through the structure, and is recommended. Even with this improvement, however, a large portion of the SPS South Pino flow will flood down Ellison (3695-1490 = 2205 cfs). This flow will be addressed in the "Streets" section of the recommendations.

Channels

All channels except the Tanoan Golf Course and Albuquerque Academy reaches are expected to be concrete lined with 100-year capacity plus freeboard in the ultimate development scenario. These channels can carry the SPS flow by using the freeboard. Occasional out-of-bank flow

can be expected from wave action and curve superelevation, but such transient flows are expected to produce minimal damage. No upgrade of channels, then, is necessary to improve SPS flow behavior. The in-progress South Pino Arroyo design from Jefferson to the North Diversion Channel inlet basin is adequate with its 100-year plus freeboard design.

As stated in the scope, AMAFCA is particularly interested in the effects of the SPS near the North Diversion inlet of the Pino. Our analysis shows no SPS out-of-bank flows on the south side of the future Jefferson to North Diversion Channel. Further retention of the south wing dike is not necessary to collect SPS flows once the channel is extended to the inlet concrete. On the north side of the channel, flows from Ellison Road can be expected to cross overland to the channel (see Figure 5). Retention of the north wing dike is essential to collect and guide these flows to the inlet.

Osuna Road Street Flow

With no improvement to the Borealis crossing structures, a peak flow of 1131 cfs for the SPS is expected to spill to San Mateo/Osuna Road from the Borealis Arroyo. WITH improvements to the crossing culverts, the spill discharge would be approximately 625 cfs. In either case, some of this flow is expected to break out of the street and continue in a southwesterly direction as shown on Figure 5. Prevention of this breakout could be accomplished with minor expense by berming a portion of the south side of Osuna in back of the curb. This would protect the existing businesses and industries located just south of Osuna. No other actions are recommended for the Osuna flow. It is our opinion that the street is the best place for SPS flows in this situation, and interception and disposal of this flow would be prohibitively expensive.

Ellison Road Street Flow

Some 2205 to 2855 cfs is expected down Ellison Road, depending on whether the inlet to Structure E is improved. The natural slope of the land along Ellison is to the northwest towards Hawkins Street, which appears to be constructed along the bottom of an old arroyo or swale. This natural slope carries flow away from the South Pino inlet, an undesirable phenomenon.

Flows could be encouraged to stay in Ellison by raising the elevation of the cross streets (Gruber, Jefferson and Washington) and driveways on the south side of Ellison, a comparatively minor effort. This would direct the flow to the intersection of Ellison and Washington.

Beyond this intersection, the flows would continue westward, with part of the flood flowing to the South Pino inlet and part heading northwest toward Hawkins. To capture all of the Ellison flow and direct it to the South Pino inlet would require major effort. A dike or a combination of dike and channel would be needed from the intersection to the existing dike wings. This would require obtaining a large amount of easement and probably outright purchase of valuable commercial property in the area. The benefits to property already protected from the 100-year storm may not be great enough to justify the expenditures.

Summary of Recommendations

1. Upgrade the inlets to Structures C and D to 1206 cfs by constructing hydraulically efficient approaches. Structure D must carry at least as much flow as Structure C.

2. Remove existing Structures A and B and replace with structures capable of carrying as much flow as Structure C.

3. Upgrade the inlet to Structure E to 1490 cfs by constructing a hydraulically efficient approach.

4. The in-progress design of the South Pino lined channel from Jefferson to the North Diversion inlet is adequate with its 100-year discharge capacity plus freeboard.

5. If the South Pino Channel were lined all the way to the inlet, the south bank would be outside the SPS flood plain. The south wing dike of the inlet to the North Diversion would then no longer be necessary, and could be removed.

6. Retention of the north wing dike is essential to capture and direct flows from Ellison into the North Diversion inlet of the South Pino.

7. Ellison street flows should be directed toward the South Pino inlet by raising the elevation of the cross streets (Gruber, Jefferson and Washington) and driveways on the south side of Ellison.

8. While all SPS flows at the intersection of Ellison and Washington could possibly be intercepted and directed toward the South Pino inlet of the North Diversion, it would require major land acquisition and dike work, and may not provide a favorable ratio of benefits to costs.

CONCLUSION

The computer model simulating the Standard Project Storm runoff for the South Pino Arroyo and the accompanying analysis of hydraulic structures showed that the SPS flows remain in the watershed from the headwaters in the Sandia Mountains to Interstate 25. At the Interstate, crossing structures designed to carry the 100-year flow were found to be inadequate to pass the greater SPS discharges. The resulting excess flows crossed under the Interstate via the San Mateo and San Antonio underpasses and continued west to the North Diversion in Osuna Road and Ellison Road. Some of the Ellison flows are recaptured by the wind dike of the South Pino inlet to the North Diversion channel, but the remaining Ellison flows and the Osuna Road flows enter the North Diversion on either side of the South Pino inlet (see Figure 5). The carrying capacity of the I-25 culverts can be increased, however, the proposed improvements are not adequate to carry all the SPS discharge for either the Borealis or South Pino Arroyo. Recommendations made to confine street flow to Osuna and Ellison.

The recommendations made in this report will alleviate some of the out-of-bank flows and return a portion of the escaping flows back into the South Pino watershed. However, it

is our opinion that the construction of the Interstate Highway 25 and its accompanying underpasses has irrevocably diverted significant amounts of the flows in excess of the 100-year storm into the watersheds neighboring the South Pino on the north and south. This unpleasant result is tempered by the long odds of an event greater than the 100-year storm occurring within the life of I-25 or the North Diversion Channel.

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Convert 5 MIN SPS Distribution to 10 MIN

COMPUTATION SHEET

ject Using 1935 Las Cruces STORM & distribution from El Paso Study, File Convert to 5 minute

Project Alamogordo, NMC Computed by [illegible] Date Mar 26 Checked By _____ Date _____ Book No _____

Time Period (10 min)	SPS Rainfall (inches)		Σ rf			Time Period (5 min)	SPS Rainfall (inches)		Time Period (5 min)	SPS Rainfall (inches)	
1	0.45		0.45			1	0.06		31	0.25	
2	0.99		1.44			2	0.39		32	0.23	
3	1.33		2.77			3	0.60		33	0.13	
4	0.45		3.22			4	0.49		34	0.19	
5	0		3.22			5	0.58		35	0.08	
6	0.24		3.46			6	0.75		36	0.08	
7	0.15		3.61			7	0.37		37	0.13	
8	0.39		4.00			8	0.06		38	0.04	
9	0.71		4.71			9	0		39	0.05	
10	0.14		4.82			10	0		40	0.01	
11	0.07		4.89			11	0.06		41	0.02	
12	0.18		5.07			12	0.18		42	0.02	
13	0.24		5.31			13	0.04		43	0.02	
14	0.06		5.37			14	0.11		44	0.02	
15	0.04		5.41			15	0.11		45	0.02	
16	0.13		5.54			16	0.28		46	0.03	
17	0.20		5.74			17	0.40		47	0.02	
18	0.48		6.22			18	0.31		48	0.02	
19	0.48		6.70			19	0.07		49	0.02	
20	0.32		7.02			20	0.07		50	0.02	
21	0.16		7.18			21	0.02		51	0.01	
22	0.17		7.35			22	0.05		52	0.01	
23	0.06		7.41			23	0.15		53	0.02	
24	0.04		7.45			24	0.03		54	0.01	
25	0.04		7.49			25	0.14		55	0.01	
26	0.05		7.54			26	0.10		56	0.01	
27	0.04		7.58			27	0.02		57	0.01	
28	0.04		7.62			28	0.04		58	0.01	
29	0.02		7.64			29	0.02		59	0.02	
30	0.03		7.67			30	0.02		60	0.02	
31	0.02		7.69			31	0.03		61	0.01	
32	0.02		7.71			32	0.10		62	0.01	
33	0.04		7.75			33	0.09		63	0.01	
34	0.02		7.77			34	0.11		64	0.02	
35	0.03		7.80			35	0.19		65	0.02	
36	0.04		7.84			36	0.29		66	0.02	

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PMP Distribution (6 Hour)

Plot mass Rainfall curve at 1 through 6 hr Rf to determine incremental rainfall

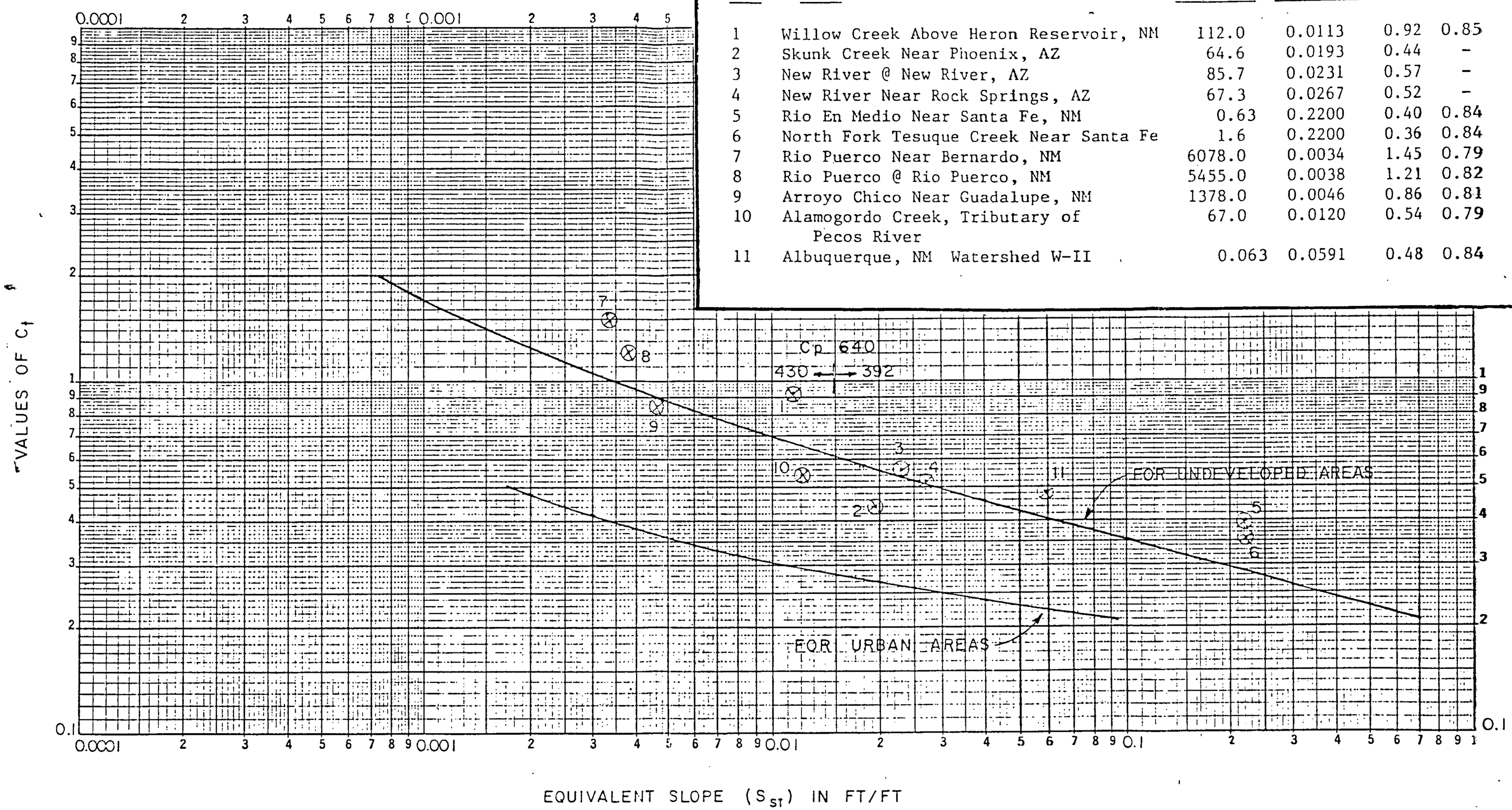
COMPUTATION SHEET

Subject PMP for area above Diversion Channel (27.14 sq. mi) File _____

Project Albuquerque, NM Computed by [illegible] Date 7/1/86 Checked By _____ Date _____ Book No _____

	Time Period (Hours)	Time Period (5 min)	Rainfall (inches)	ΔRainfall	Cumulative Arranged (inches)		Time Period (Hours)	Time Period (5 min)	Rainfall (inches)	ΔRainfall (inches)	Cumulative Arranged (inches)	
		1	2.50	2.50	0.06			37	13.89	0.09	2.50	
		2	3.75	1.25	0.06			38	13.98	0.09	1.01	
		3	4.76	1.01	0.06			39	14.07	0.09	0.80	
		4	5.60	0.84	0.06			40	14.16	0.09	0.71	
		5	6.40	0.80	0.06			41	14.24	0.08	0.60	
		6	7.14	0.74	0.06			42	14.32	0.08	0.60	
		7	7.85	0.71	0.07			43	14.40	0.08	0.20	
		8	8.50	0.65	0.07			44	14.48	0.08	0.16	
		9	9.10	0.60	0.07			45	14.56	0.08	0.16	
		10	9.70	0.60	0.07			46	14.64	0.08	0.14	
		11	10.30	0.60	0.07			47	14.72	0.08	0.13	
	1	12	10.71	0.41	0.07		4	48	14.80	0.09	0.12	
		13	10.91	0.20	0.08			49	14.87	0.07	0.11	
		14	11.09	0.18	0.08			50	14.94	0.07	0.11	
		15	11.25	0.16	0.08			51	15.01	0.07	0.11	
		16	11.41	0.16	0.08			52	15.08	0.07	0.11	
		17	11.57	0.16	0.09			53	15.15	0.07	0.11	
		18	11.72	0.15	0.09			54	15.22	0.07	0.10	
		19	11.86	0.14	0.10			55	15.29	0.07	0.09	
		20	12.00	0.14	0.11			56	15.36	0.07	0.09	
		21	12.13	0.13	0.11			57	15.43	0.07	0.08	
		22	12.26	0.13	0.11			58	15.50	0.07	0.08	
		23	12.38	0.12	0.11			59	15.57	0.07	0.08	
	2	24	12.50	0.12	0.11		5	60	15.64	0.07	0.08	
		25	12.61	0.11	0.12			61	15.70	0.06	0.07	
		26	12.72	0.11	0.13			62	15.76	0.06	0.07	
		27	12.83	0.11	0.14			63	15.82	0.06	0.07	
		28	12.94	0.11	0.15			64	15.88	0.06	[illegible]	
		29	13.05	0.11	0.16			65	15.94	0.06	0.07	
		30	13.16	0.11	0.18			66	16.00	0.06	0.07	
		31	13.27	0.11	0.41			67	16.06	0.06	0.06	
		32	13.38	0.11	0.60			68	16.12	0.06	0.06	
		33	13.49	0.11	0.65			69	16.18	0.06	0.06	
		34	13.60	0.11	0.74			70	16.24	0.06	0.06	
		35	13.70	0.10	0.84			71	16.30	0.06	0.06	
	3	36	13.80	0.10	1.25		6	72	16.36	0.06	0.06	

NO.	NAME	DRAINAGE AREA (SQ.MI.)	SLOPE (FT./FT.)	C_t	C_p	$C_{p\,640}$
1	Willow Creek Above Heron Reservoir, NM	112.0	0.0113	0.92	0.85	542
2	Skunk Creek Near Phoenix, AZ	64.6	0.0193	0.44	–	–
3	New River @ New River, AZ	85.7	0.0231	0.57	–	–
4	New River Near Rock Springs, AZ	67.3	0.0267	0.52	–	–
5	Rio En Medio Near Santa Fe, NM	0.63	0.2200	0.40	0.84	540
6	North Fork Tesuque Creek Near Santa Fe	1.6	0.2200	0.36	0.84	540
7	Rio Puerco Near Bernardo, NM	6078.0	0.0034	1.45	0.79	506
8	Rio Puerco @ Rio Puerco, NM	5455.0	0.0038	1.21	0.82	525
9	Arroyo Chico Near Guadalupe, NM	1378.0	0.0046	0.86	0.81	518
10	Alamogordo Creek, Tributary of Pecos River	67.0	0.0120	0.54	0.79	506
11	Albuquerque, NM Watershed W-II	0.063	0.0591	0.48	0.84	540



NOTE: FROM FEBRUARY 1978 FIS FOR NE & CENTRAL EL PASO, TX

U. S. ARMY ENGINEER DISTRICT, ALBUQUERQUE
CORPS OF ENGINEERS
ALBUQUERQUE, NEW MEXICO

DESIGNED BY: DES
DRAWN BY: JL
CHECKED BY: LFJ

RIO GRANDE AND TRIBUTARIES — EL PASO, TEXAS
EL PASO FLOOD CONTROL PROJECT
CENTRAL AREA

SNYDER'S C_t VERSUS EQUIVALENT SLOPE

TO ACCOMPANY GENERAL DESIGN MEMORANDUM NO. 14
DATE: MAY 1987
FILE NUMBER RG-ELP-DM-K-B2
PLATE B-3