WHITE TANKS/ AGUA FRIA

AREA DRAINAGE MASTER STUDY

FLOOD CONTROL DISTRICT of MARICOPA COUNTY

PART A: FLOOD STUDY TECHNICAL DATA NOTEBOOK

The WLB Group Inc.

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WHITE TANKS / AGUA FRIA AREA DRAINAGE MASTER STUDY

Part A:
Flood Study Technical Data Notebook

Prepared For:
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
2801 West Durango
Phoenix, Arizona 85009

October, 1992

REGISTERED PROFESSIONAL ENGINEER (CIVIL)
CERTIFICATE NO. 15594
MARK T. GAVAN
10-27-92
Date Signed
ARIZONA, U.S.A.

Prepared by:
THE WLB GROUP, INC.
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SECTION 1: GENERAL DOCUMENTATION AND CORRESPONDENCE

1.1 SPECIAL PROBLEM REPORTS

There were no special problem reports prepared for this study. However, a few unique situations were encountered in the White Tanks/Agua Fria Area Drainage Master Study (ADMS). Refer to **Section 3: Hydrologic Analysis** and **Section 4: Hydraulic Analysis** for specific modeling assumptions or problems associated with drainage areas or individual washes.

1.2 LIST OF CONTACTS

A list of people and organizations contacted in this study are as follows.

CONTACT LIST
White Tanks/Agua Fria ADMS

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Corridor Engineer
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Mr. Steve Cleveland
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Mr. Robert Musselwhite
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1.3 MEETING MINUTES AND REPORTS

Copies of meeting minutes, reports, and telephone conversations are located on the following pages.

Jan 18, 1990

Initial Meeting With Luke AFB To Explain White Tanks/Agua Fria ADMS

Have a concept on the Dysart Drain but no detail analysis on it yet.

Have survey on the Dysart Drain and a few drainage analyses have been done on the Base itself.

Future development is planned for the north side of the facility.

West Side Joint Land Use Study by MAG. 1988 – Harry Wolf with APOT

They will get us copies of their studies and drainage concepts.

Richard & Zane are our contacts. Mainly coordinate with Zane Hoit

East of Litchfield Park is a planned new development that is to be built in this square mile area Indian School & Dysart

PRC Torps did a study on Dysart Drain also.
P&D Tech.

Try to obtain funding for a future golf course south of Litchfield Park

High Spot - 1/2 mile east of Dysart Road and Ponds back to the west.

Attendees

Name	Position	Phone
Jeff Erickson	WLB	279-1016
GREG RODZENKO	Flood Control	262-1501
RICHARD MOUSEL	LUKE AFB AZ	856-3635
ZANE Hoit	LUKE AFB	856-7634
GORDON BUCHANAN	LUKE AFB	856-6462
Ron DiBenedetto	Luke AFB	856-3634
MARK GAVAN	THE WLB GROUP	279-1016

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Report of Meeting

Project title: WHITE TANKS ADMS Date: 1-22-90

Job no. 289036 Prepared by: MTG

Report to: FILE

Re: CATERPILLAR PROVING GROUNDS

Date of meeting: 1-22-90 Time: 1:30 To: 1:45

Place of meeting: TELEPHONE CONVERSATION

Attendees:

DON SWANSON - CATERPILLAR
MARK GAVAN - WLB

Comments:

- DON WILL BE OUR POINT OF CONTACT (853-9586).
- CATERPILLAR IS MOVING OPERATIONS TO SOUTHERN ARIZONA IN JUNE OF 1990.
- THEY INTEND TO SELL THEIR PROPERTY, BUT IT HAS NOT BEEN SOLD YET.
- CATERPILLAR HAS NO PLANS TO CHANGE ANY OF THE DRAINAGE FROM THE WAY IT NOW EXISTS.

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COMPUTATION DATA SHEET

Subject WHITE TANKS/AGUA FRIA ADMS Prepared by JSE Date 1-25-90

Telephone Conversation W/ Garth Weiser V.P. Continental Homes
483-0006

He said they have a subdivision within Sierra Vista and to contact Coe + Van Loo for any drainage plans that they may have preformed for the original piece of property.

Ask for Ash Patel.

Form 209

Jan 29, 1990

Meeting w Geir Sverdrup Maricopa Planning & Development

Spencer Developement Corp & Wigwam Creek

MAG to get Westside Joint Land Use Study
Transportation & Planning Office

G. William Larson - Primary Contact
837-8037

Feb 1 - 90

Initial Meeting With City of Glendale
on White Tanks / Agua Fria ADMS

In attendance

Grant Anderson - City of Glendale, City Engineer
Dan Sherwood - City of Glendale, Engineer
Greg Rodzenko - FCD, Project Manager
Jeff Erickson - WLB, Project Engineer

Made initial introductions and Greg Rodzenko started the meeting by explaining the ADMS process and what FCD's role is and the schedule of the study.

Mr Anderson explained they were happy to see this type of study and described the area of Glendale's strip annexation. He said to contact Dave Prescott in their Planning Division to obtain a recent study for the Western Area Planning by the City of Glendale

Mr Erickson then explained what types of work would be performed by WLB in the White Tanks/Agua Fria ADMS, such as hydrology, hydraulics, Area Drainage Master Plans etc.

Mr Sherwood indicated he would like a copy of the Scope of Work and a copy of the FCD Hydrology Manual when it becomes available. The City does not have any studies in the area since it is only strip annexed, but will give us any information they may have in the area. Coordination will be with Mr. Sherwood.

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COMPUTATION DATA SHEET

Subject _____ Prepared by _____ Date _____

Telephone Conversation w/ 1-2-90
Michelle Schuler
Morton Salt

She said they have no drainage plans or studies as such that she knows about. They have large berms around the perimeter of their site that diverts offsite runoff and a pond on site that retains all on site flow. I'm sure they have to submit something to FCD on drainage at one time when they started but she didn't seem to know.

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Report of Meeting

Project title: WHITE TANKS/AGUA FRIA AOMS Date: April 5th 1990

Job no. 284036 Prepared by: JSE

Report to: Greg Radzenko, Project Manager, Flood Control District of Maricopa County

Re: Meeting with Glenn Bush, Yost & Gardner, City Engineer for City of Goodyear

Date of meeting: April 3, 1990 Time: 2:00 To: 3:00

Place of meeting: Yost & Gardner Office

Attendees: Glenn Bush, - Yost & Gardner
Mark Gavan - WLB
Jeff Erickson - WLB

Comments: Mr Bush will be reporting to Mr Steve Cleveland the City Manager of Goodyear. Main problem is the flooding of Bullard Wash where it crosses the Southern Pacific railroad on the southend of Phoenix-Goodyear Airport.

There is the possibility of developing this and turning Bullard Wash into a linear park scheme.

They have a 36" storm drain in Van Buren to ½ mile west of Dysart Road. This is an ADOT storm drain in Dysart Road to empty the detention basins on the north side of I-10 and their pipe hooks into this. There is also a storm drain in Yuma Rd. and Litchfield Road that were a county projects that runs from south of I-10 to the Gila River. Glenn will get us plans for these systems if possible, seems they don't

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Report of Meeting

Project title: WHITE TANKS/AGUA FRIA ADMS Date: April 5, 1990

Job no. 289036 Prepared by: JSE

Report to: Greg Rodzenko, Project Manager, Flood Control District of Maricopa County

Re: Meeting with Scott Lind, City Manager of the City of El Mirage.

Date of meeting: April 4, 1990 Time: 2:00 pm To: 4:00 pm

Place of meeting: City Hall, El Mirage

Attendees: Scott Lind – El Mirage
Mark Gavan – WLB
Jeff Erickson – WLB

Comments: Mr Lind indicated they have a problem with flooding behind Grand Avenue and we should talk to ADOT & Kimberly Horne about this problem. They have a masterplan just north of Grand Avenue and some engineering has been completed by Hooke Engineering. Contact Don Pearson for their plans on this.

El Mirage has a 48" in storm Drain in Thunderbird Rd. that was done by Royden Eng. which is now Donahue. Mr Lind said he would have his City Engineer make copies of their plans for us and will get them to us when they are done. They are interested in seeing the full delineation of Lower El Mirage Wash.

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Report of Meeting

Project title: WHITE TANKS/AGUA FRIA ADMS Date: April 5, 1990

Job no. 289036 Prepared by: JSE

Report to: Greg Rodzenko, Project Manager, Flood Control District of Maricopa County

Re: Meeting with Mr. Craig Aper, City of Litchfield Park

Date of meeting: April 5, 1990 Time: 9:30 To: 11:30

Place of meeting: City Hall - Litchfield Park

Attendees: Craig Aper - Litchfield Park
Jeff Erickson - WLB

Comments: Mr. Aper indicated they have a definite problem with flooding along Camelback Road from Murphy Dam. This was due to some storms he characterized as "Micro-Bursts." Litchfield Park is very flat and does not have adequate drainage facilities. He knew about FCD's effort with County Highway to provide a drainage channel along Camelback Road, but was concerned that it would not be sized big enough for the above mentioned event.

He also indicated they have a storm drain in Indian School Road that outlets through an inverted siphon under the RID Canal into a drainage swale that carries flows south to the I-10 dentention basins. This channel has no slope and tailwater ponds causing backwater upstream in the storm sewer and back out the inlets. Also have significant ponding behind RID & Airline Canal. He will give us what plans and reports he has for the area.

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Report of Meeting

Project title: WHITETANKS/AGUA FRIA ADMS Date: 4-6-90

Job no. 289036 Prepared by: JSE

Report to: Greg Radzenko - Project Manager - Flood Control District of Maricopa County

Re: Meeting with Mr. Fred Carpenter - City Manager

Date of meeting: April 6, 1990 Time: 10:00 To: 11:00

Place of meeting: Buckeye Municipal Building

Attendees: Fred Carpenter, City of Buckeye
Jeff Erickson, WLB

Comments: Mr. Carpenter said they basically did not have any information for with the exception of the Buckeye General Developement Plan.

He indicated that when public meetings occur they can incorporate a computer message in their water bills. We may use this to inform general public of place & time of the meeting

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April 13, 1990

Mr. Greg Rodzenko
Project Manager
Flood Control District of
Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Re: Meeting with Mr. Carlos Palma
City Manager, City Of Avondale

Project: White Tanks/Agua Fria ADMS
WLB No. 289036

Subject: Meeting Minutes
April 10, 1990
Time: 11:00
Place: City of Avondale
Attendees: Carlos Palma - City Manager
Mike Springfield - Planner
Glenn Gibbons - Engineering
Bill Bedoya - Mayor, City of Avondale
Mark Gavan, Jeff Erickson - WLB

Minutes taken by: Jeff Erickson, Project Engineer

The following items were discussed:

Mr. Palma indicated he was not too happy with the FCD not letting him know what was happening in Litchfield Park on Camelback Road. He asked us what we would do if Avondale does not accept the White Tanks study. We don't think he understood what we were telling him about the ADMS. Mr.Palma indicated they have flooding problems along the north side of the railroad and they have no outlets downstream of where structures are located. Also, the dike along the Agua Fria has no provision for passing flows according to Mr. Palma.

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We are not familiar with the drainage process along the dike and will need additional information on this issue. Avondale has a storm drain located in Central running to Western and then continuing to Litchfield Road, then south to the Gila River. They said this was a County job and it outlets into a ditch. There is also a storm drain from Central Avenue to Dysart Road in Western Avenue and it dumps into a channel that continues to the Agua Fria. Mr. Gibbons will make us copies of plans of these storm drains along with a map of their annexation limits.

Avondale is planning to build a Community College that will cover approximately 105 acres in the area of Thomas Road and Dysart Road. They are also very concerned about diverting additional flows into the Agua Fria, and were making comments about another IGA to get more compensation if this happens.

It was a very interesting meeting. You probably should have been there to field some questions on behalf of the Flood Control District. I Suggest that the FCD set up a meeting with them to explain current projects taking place.

JSE/cp

A:WORKING/8936.mn

Meeting w/ City of Avondale – Floodplain Presentations
(4-24-92)

Mike or Jim will get us a copy of schematic plan of drainage scheme.

– Land Use Plans

– Presented floodplain delineations in their area. Not much affecting them except for ponding areas.

White Tanks/Agua Fria Floodplain Presentation w/City of Avondale
4-24-92

Attendees	Representing/Position	Phone No
Jeff Erickson	WLB Group - Project Engineer	—
Nicolas V. Volma	Avondale	932-2400
MIKE SPRINGFIELD	AVONDALE	932-1909
JIM BADOWICH	AVONDALE	932-1909
BILL BATES	"	" "
GREG RODZENKO	FLOOD CONTROL	506-1501

White Tanks/ Agua Fria Floodplain Presentations
Minutes of Meeting w/ City of Glendale 2:30 pm
April 7, 1992

List of Attendees - See attached sheet.

A.) Presented floodplain delineations to City of Glendale representatives.

B) Grant Anderson thought carrying all water collect on the Reems Road alignment at Northern Ave should be continued south around the western side of Luke AFB, thus downsizing the needed capacity of Dysart Drain.

C.) They also indicated that a study was ongoing as to whether Glendale should annex the western area west of Agua Fria River. Does not look like this would happen according to the planners, but Grant A. would like to see this happen.

D.) Also indicated a minor change in City limits of Litchfield and Glendale as we have shown. They would like to get copies of mapping as soon as possible.

WHITE TANKS / AGUA FRIA ADMS 4-7-92

INFORMATIONAL MEETING W/ CITY OF ~~[illegible]~~ GLENDALE

LIST OF ATTENDEES

NAME	REPRESENTING	NUMBER
Jeff S. Erickson	The WLB Group	279-1016
Dan Sherwood	City of Glendale	435 4152
Doug Kukino	" " "	435-4212
DAVID PRESCOTT	Glendale Planning	435-4206
Greg Rodzenko	Flood Control	506-1501
MARK GAUAN	WLB	279-1016
GRANT ANDERSON	Glendale	435-4152

White Tanks/Agua Fria Floodplain Presentations
Minutes of Meeting w/ Town of Buckeye 9:00am April 10, 1992

In Attendance

Fred Carpenter City Manager-Town of Buckeye
Kelli Sertich City Planner-Buckeye
Greg Schulke A-N West City Engineer
Greg Radzenko FCDMC
Tim Murphy FCDMC
Jeff Erickson WLB Group

A) Presented floodplain delineations to the group in areas that have affect in the Buckeye jurisdiction.

B) Explained also areas that affected El Mirage to Greg Schulke who is also El Mirage's city engineer.

C) Told them about public meetings upcoming and Fred indicated they could put a notice in their water bills.

D) Kelli indicated she would like mapping and aerial photos copies for planning purposes They were directed to get ahold of FCD after mapping is submitted
Tim Murphy also talked about Buckeye ADMS adjoining this study area

White Tanks / Agua Fria Floodplain Presentations

Minutes of Meeting w/ City of Litchfield Park 1:00 pm
April 7, 1992

In Attendance
Robert Musselwhite – City Manager - Litchfield Park 935-54.
Greg Rodzenko – FCDMC
Mark Gwan - WLB
Jeff Erickson – WLB.

A.) Presented floodplain delineations to Mr Musselwhite in the area of Litchfield Park.

B.) Mr Musselwhite indicated they had a ponding problem at S. Ancora Drive at the East turnaround. This is due to majority of drainage area flow associated here flows to this spot and there are only 2 C.B.s located here to drain this corner. Home pads are built up as is the south lots along with a wall with no outlet through it.

C. He said previously the C.B. pipe outlet into a standpipe south of Indian School. Indian School was rebuilt higher and south field was filled. The C.B. pipe was then connected into a new C.B on Indian School which ultimately connects into a pipe on Litchfield Road

which outlets south of the RID canal. This pipe does not have capacity required and Ponding behind RID canal causes head to be built up which in turn causes water to flow backward out the the C.B.'s at Arcora Drive.

D. Mr. Musselwhite indicated that Robert Murphy with Gannet Flemming had a survey done in the area to better nail down the problem.

E. He also indicated that local flooding occurs at Indian School and Castillo due to upstream inflow on Dysart Road.

F) Explained how Colter Channel will help out their situation.

White Tanks/Agua Fria Floodplain Delineations
Minutes of Meeting w/ Luke AFB 11:00am April 10, 1992
In Attendance - See attached sheet.

A) Explained to the group about floodplain delineations affecting Luke AFB

B) Showed them where Dysart Drain has insufficient capacity and where breakouts occur onto the A.F.B.

C) Explained the delineations of approximate A zones and their implication.

D) They indicated that the Corps of Engineers had done a damage assessment study on the base and Greg said he had a copy. (Based our Dysart Drain Hydro).

E) Tim Murphy talked also about the Gila Bend ADMS which affects Luke's Gunnery Range.

List of Attendees

Name	Position	Phone No
Anne-Marie Geiger	Civil Engineer	856-7634
Bill Melche	Utilities Mgr	856-6394
Gordon J. Buchanan, P.E.	Chf Engr & Tech Dsgn	856-6462
Dean E Johnson	Heavy Repair Superintendent	856-6075
Zane Hoif	58 CES/CEE	856-7634
Greg Rodzenko	Flood Control	506-1501
Tim Murphy	FCD MC	506-1501
Jeff Erickson	WLB Group Inc	279-1016

White Tanks/ Agua Fria Floodplain Delineations
Meeting Minutes w/ City of Goodyear 10:00am April 13, 1992

In Attendance

Steve Cleveland	City of Goodyear - City Manager
Glenn Bush	Yost + Gardner City Engineer
Lynn Kartchner	City of Glendale Public Works Dir.
Harvey Krauss	Asst City Manager
Greg Bodzanko	FCDMC
Jeff Erickson	WLB Group.

A) Presented floodplain delineations throughout Goodyear annexed and unannexed areas. They are very interested in Bullard Wash a Pebble Creek is a big developement going in now. They want copies of the mapping as soon as possible so as to regulate the developement.

B) Indicated Paul LeBrun from Corps of Engineers had talked to them about going in on channelizi of Bullard Wash at Phoenix-Goodyear Airport and to the south to the Gila River.

C) Paul Walker said Goodyear is a city not town as indicated by city limits on WLB's 1"=400' mapping.

D.) Also Litchfield Park has now annexed area to the east where Goodyear had previous strip annexation.

E.) Asked Greg about how they should regulate Bullard Wash. As to whether accepting our current delineations or waiting to model improvements proposed by Pebble Creek. Greg said since they were their own floodplain managers they could do what they wanted and we may hold off on submitting Bullard to FEMA. This should be pursued further to see what they want to do here.

F.) Asked us if we had studied the small gullies in southern part of the watershed. Said we had not identified floodplains on these areas but we can address them in our report as their are a few subdivisions within these gullies where houses have been flooded in the past.

6.) We will be in touch to gather information they have on drainage in Pebble Creek develop and incorporate this information into our master plan

Put in Goodyear File (ADMS)

Glenn Bush 264-6424

Phase I = 9 holes Phased itself little over a section 800 acres in pieces - Western Part 9 holes & subdivision West of Bullard South of Indian School. Approx. 300 Acres.

- Very Rough Grading Plans
- Mass Grade GC -
- Late Canal
- Storm Drains on GC.
- Irrigation Pump Station on GC.
- Street Improvements 207 214 Pebble Creek Parkway
- No Subdivision Plans Yet
- Showing Spoil Deposition

Estrella Parkway & Meadcanders West of present Reems Rd. about ½ mile.

City of Goodyear Public Works - Lynn Kartchner Director

Matches in at Camelback & Thomas Rd.

	FP	FW	24-hr WLB-Q	6-hr CUL-Q	6-hr BA R-Q	
Camelback	1600'	600'	4240	2941	2600	
Indian School	2400'	700'	4121		2519	? BW 25 191'
Thomas	4800'	1700'	4703	3585.	2560 ?	252

White Tanks/Agua Fria Floodplain Presentations
Minutes of Meeting w/ City of Surprise 10:00 April 14, 1992

List of Attendees — See attached sheet.

A) Presented an overview to Dick McLamb - Surprise City Manager since he arrived after our initial meetings before the study started.

B) Then began to present floodplain delineations within Suprise City limits explaining how they affect their jurisdiction. They would like to get copies of the mapping for their planning purposes.

C) Indicated Section 32 on Sheet 5 is Kingswood Park not Sun Valley.

D) Said a landowner complained about ponding behind Estrella Freeway south of Grand Avenue Connection. We drove out and looked at it and it didn't look that bad.

E) Contact New Sun to put in notice for public meetings

Meeting w/ Town of Surprise April 14, 1992

List of Attendees

Name	Position - Representing	Phone No.
Shirley Berg	Planning Director	#583-1090
Jerald O. Morrow	Community Dev. Director	583-1088
CATHY REGESTER	FLOOD CONTROL	506-1501
Jeff Erickson	WLB	279-1016
Grey Rodzenko	FCDMC	506-1501
Richard McCamb	City Manager	583-1080

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Report of Meeting

Project title: White Tanks / Agua Fria ADMS Date: 5-14-92

Job no. 289036 Prepared by: JSE

Report to: Greg Rodzenko, FCDMC

Re: Meeting with Robson Developement and SunCor to explain floodplain delineations.

Date of meeting: 5-14-92 Time: 8:00 To: 10:30

Place of meeting: WLB Group

Attendees: Duane Black, Tom Hill - SunCor; Jason Burgess, Dallas Paulson, Dave Sturgeon - Robson Communities; Jack Moody - Coe & Van Loo; Greg Rodzenko - FCDMC; Mark Gavan, Jeff Erickson - WLB

Comments: This meeting was held to present floodplain delineations to representatives from SunCor and Robson Developement. These two groups have major land holdings in the Study Area. Both companies indicate they have imminent construction on Bullard Wash and would like WLB to incorporate these changes into the FEMA submittal. Since WLB is just about ready to submit the entire package to FEMA, we will include a letter stating that Bullard Wash should be reviewed last and we will submitt revised Bullard Wash delineations before the FEMA TEC has completed it's review. Greg Rodzenko of the FCDMC agreed to this stipulating that these changes would be paid for by the developers. WLB will provide a proposal to the companies for this additional work and send a schedule to Greg Rodzenko for his approval.

Copies to: File.

Form 114

1.4 GENERAL CORRESPONDENCE

1.4.1 Community and Other Local Interests

Correspondence with the incorporated communities within the study area and other local interests is located on the following pages.

36

"COMMUNTIES OF THE WHITE TANKS AREA"

THIS LETTER FROM THE FCD OF MARICOPA CO. WAS SENT TO EACH INCORPORATED COMMUNITY AND LUKE AIR FORCE BASE IN DECEMBER OF 1989

Dear CITY MANAGER:

The Flood Control District of Maricopa County is about to start an Area Drainage Master Study (ADMS) for the West Valley. The study area extends from Grand Avenue on the North to the Gila River on the South, and the Agua Fria River on the East to the White Tank Mountains on the West (attached is a map of the study area). The WLB Group, an engineering firm with a full service office in Phoenix, will be the study contractor. The study will begin in December, 1989 and it is anticipated that the study will be finished in May, 1992.

The purpose of the study is to identify problems and develop solutions associated with drainage in the existing, developed portions of the watershed. The study is also intended to provide regional planning tools to solve anticipated drainage problems.

The initial task of the study is to collect and analyze existing background information for the study area and meet with local officials from the study communities. Please appoint a point of contact from your city (town staff) who is familiar with your drainage problems, if any, so that our study consultant and representatives from the District staff can schedule an intial meeting.

If you or elected officials from your city (town) would like a presentation outlining the study process and purpose, we will be happy to schedule it with you. A series of public meetings will be conducted to infrom local residents of the study purposes and progress. These meetings will also be coordinated with the local communities.

spelling

Greg Rodzenko, Water Resources Planner, will be the project manager for the study. Please feel free to contact him at 262-1501 with any questions.

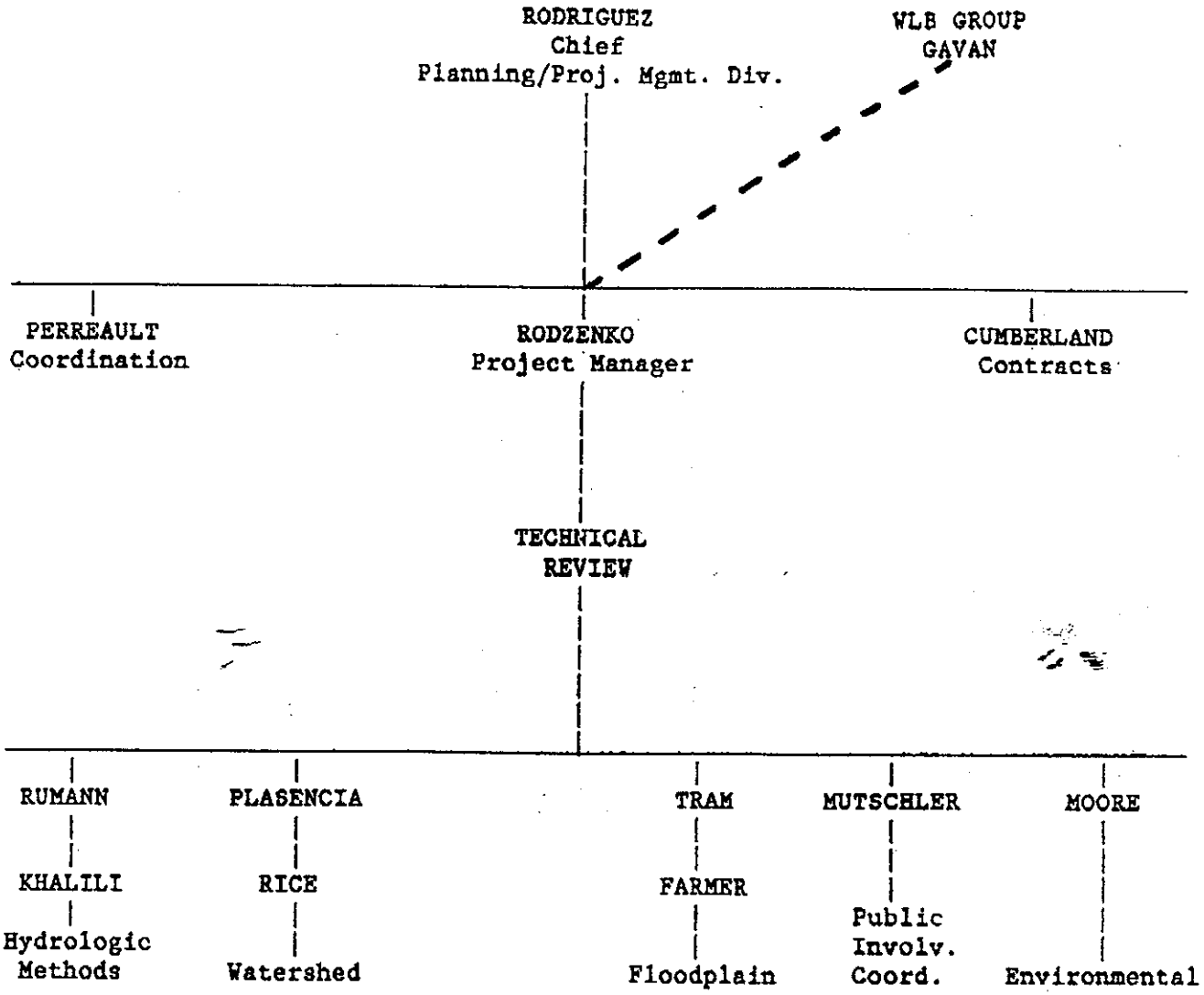
Sincerely,

Enclosures

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

WHITE TANKS/AGUA FRIA ADMS

CONTRACT FCD 89-50



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Landscape Architecture

January 16, 1990

Copy of this letter was sent to each contact identified is Section 1.2 of this report

name
company
address
city

Re: White Tanks/Agua Fria Area Drainage Master Study
FCD No. 89-50
WLB No. 289036

Dear last:

This letter is to inform you that The WLB Group has started an Area Drainage Master Study (ADMS) for the West Valley under contract to the Flood Control District of Maricopa County, Arizona. The study area extends from Grand Avenue on the north to the Gila River on the south, and the Agua Fria River on the east to the White Tanks Mountains on the west (attached is a map of the study area). The study began in December, 1989 and is anticipated to be completed in May, 1992.

The purpose of the study is to identify existing flooding problems and develop solutions for the existing, developed portions of the watershed. The study is also intended to provide regional drainage planning tools for future development.

The initial task of the study is to collect and analyze existing background information for the study area and meet with local public officials and interested private concerns located within the study area. Please appoint someone from within your organization who is familiar with your particular existing drainage problems, concerns, and future plans so that The WLB Group and representatives from the Flood Control District Staff can schedule an initial meeting.

We would also like, at this time, to start gathering any pertinent information that will help with the study. This would include any information you may have regarding existing grading and drainage plans and

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333 East Osborn Suite 380 · Phoenix, Arizona 85012 · (602) 279-1016

The WLB Group Inc.

January 16, 1990
last
Page Two

reports, future development plans, structures, land ownership maps, etc. Please be thinking about this type of information that may be useful to us. We would like to have someone from your organization call or write us to set up a meeting. The purpose of the meeting will be to discuss the study and for us to obtain any information you have that may be useful in identifying existing drainage problems or in planning for future drainage systems.

Please feel free to contact myself or Mark Gavan at 279-1016 if you have any questions regarding the White Tanks/Agua Fria ADMS.

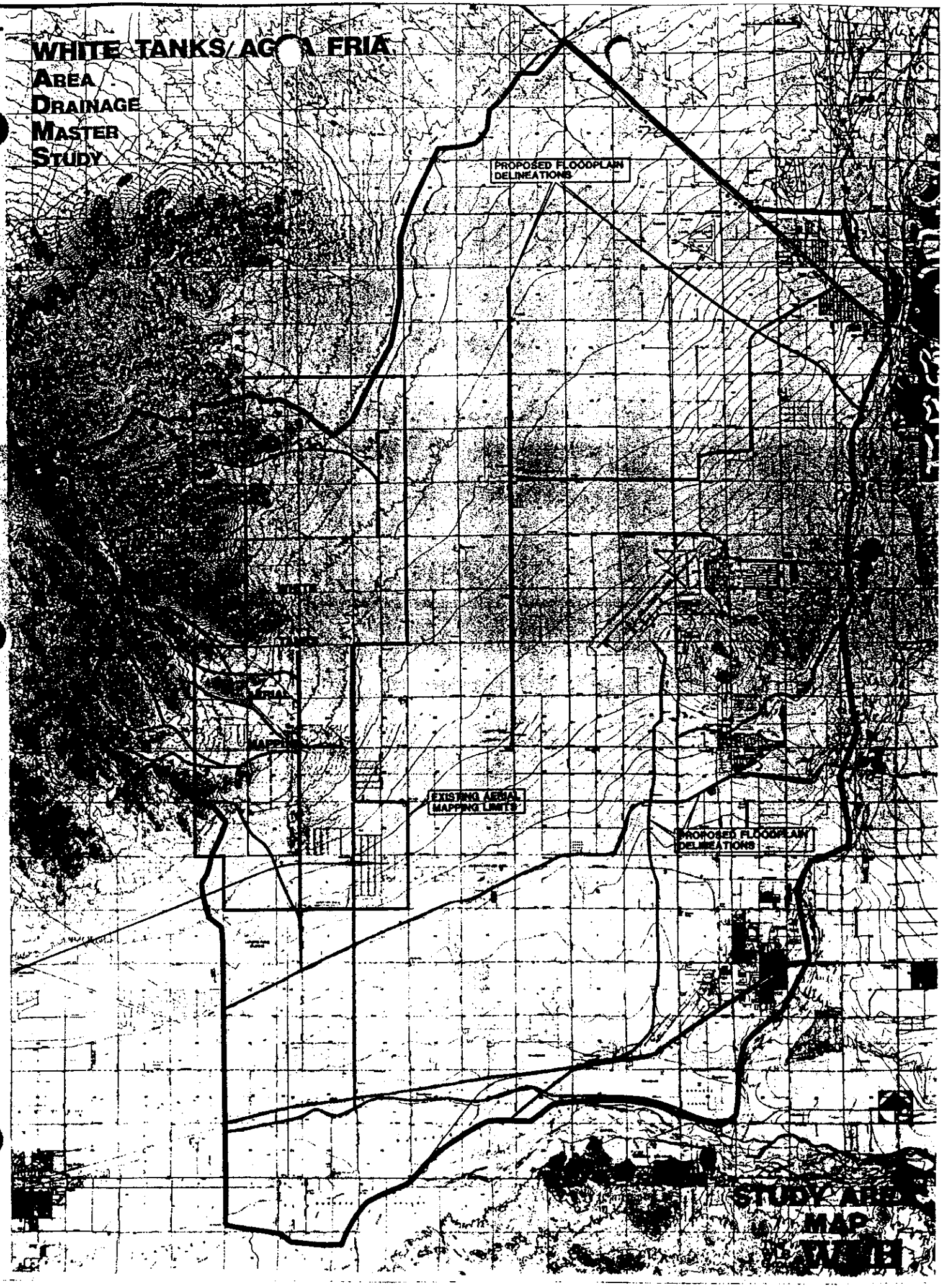
Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson
Project Engineer

JSE:srm

A:LETTERS/JSE-8936.LS


WHITE TANKS/AGUA FRIA
AREA
DRAINAGE
MASTER
STUDY
PROPOSED FLOODPLAIN DELINEATIONS
EXISTING AERIAL MAPPING LIMITS
PROPOSED FLOODPLAIN DELINEATIONS
STUDY AREA
MAP

United States
Department of
Agriculture

Soil
Conservation
Service

201 E. Indianola Ave.
Suite 200
Phoenix, AZ 85012

January 22, 1990

RECEIVED
JAN 1990
WLB-PHX

Jeff S. Erickson
Project Engineer
WLB Group
333 East Osborn, Suite 380
Phoenix, AZ 85012

Dear Mr. Erickson:

In response to your letter dated January 16, 1990 we have assigned Mr. Harry Millsaps to coordinate with you on the White Tanks/Agua Fria Drainage Master Study, which you are completing for the Maricopa County Flood Control District. Mr. Millsaps is an hydraulic engineer on our Program Planning Staff, and may be contacted directly by calling (602) 640-2547.

Any materials you might need from the Soil Conservation Service (SCS) may be obtained through Mr. Millsaps. He will also represent the SCS at any meetings which you might schedule.

We are looking forward to working with you on this project, and would like to receive a copy of the final results.

Sincerely,

ACTING FOR

DONALD W. GOHMERT
State Conservationist

CONTINENTAL HOMES, INC.

11000 NORTH SCOTTSDALE ROAD/SUITE 234
SCOTTSDALE, ARIZONA 85254

POST OFFICE BOX 60010
PHOENIX, ARIZONA 85082-0010

(602) 483-0006

RECEIVED
JAN 1990
WLB-PHX

January 22, 1990

Mr. Jeff Erickson
WLB Group
333 East Osborn, Suite 380
Phoenix, Arizona 85012

Dear Jeff:

We received your letter of January 16, 1990 regarding the White Tanks/Agua Fria Area Drainage Master Study. With regard to any property that we own within this area, we would be happy to meet with you.

Our time is flexible and our offices are open for a meeting. Please call me at 483-0006 to schedule a meeting.

Sincerely,

Garth R. Wieger
Vice President of Land Development

April 28, 1992

(A copy of this letter was sent to the list of contacts identified in Section 1.2 of this report)

Re: Announcement of Completion of White Tanks/Agua Fria
Area Drainage Master Study Floodplain Delineations
WLB NO. 289036

Dear Mr. Kienow:

The WLB Group, Inc., an engineering consultant for the Flood Control District of Maricopa County, is pleased to announce the completion of floodplain delineations for the White Tanks/Agua Fria Area Drainage Master Study.

This study encompasses an area of approximately 220 square miles, roughly bounded by the Gila River to the south, Agua Fria River to the east, White Tank Mountains and Dean Road to the west, and McMicken Dam and Grand Avenue to the north. Municipalities located within the study area include Avondale, Buckeye, El Mirage, Glendale, Goodyear, Litchfield Park, Surprise, Luke Air Force Base, and a small strip annexed area of Phoenix.

Detailed floodplain and floodway analyses and approximate delineations were performed on many washes and drainage swales within the study area using the U.S. Army Corps of Engineers' HEC-2 Water Surface Profiles computer program. The HEC-1 rainfall-runoff computer program was also utilized to compute peak discharges throughout the watershed and, in some instances, was used to define ponding water surface elevations behind structures such as canals, railroads, and roadway embankments.

Detailed studies were performed on the following washes using HEC-2:

- Beardsley Canal Wash
- Cholla Wash
- North Fork Cholla Wash
- Waterfall Wash

- White Tank #3 Wash
- Bedrock Wash
- North Fork Bedrock Wash
- Jackrabbit Trail Wash
- Tuthill Dike Wash
- Bulldozer Wash
- Caterpillar Wash
- Tractor Wash
- Caterpillar Dike Wash
- White Granite Wash
- North Fork White Granite Wash
- 191st Avenue Wash
- Perryville Road Wash
- Bullard Wash
- AT&SF Railroad Channel - Agua Fria River to Greenway Road
- Lower El Mirage Wash
- Lower El Mirage Wash Tributary
- Interstate 10 - Jackrabbit Trail to Tuthill Dike
- Litchfield Wash

Detailed studies of ponding areas utilizing the HEC-1 computer model were delineated for the following areas:

- Interstate 10
- Roosevelt Irrigation District Canal
- Southern Pacific Railroad
- Buckeye Canal
- Agua Fria River Dike
- White Tanks Flood Retarding Structures #3 and #4
- Litchfield Park Detention Facility

Approximate delineations were computed using the HEC-2 model for the following areas:

- Cotton Lane Wash - Indian School Road to Olive Avenue
- Cotton Lane Wash - Olive Avenue to Waddell Road
- Interstate 10 - RID Canal to Cotton Lane
- Atchison, Topeka and Santa Fe Railroad Spur - Northern Avenue north to Waddell Road
- Bullard Wash - From south end of Luke AFB to Reems Road

- Bullard Wash - From Gila River to south end of Phoenix-Goodyear Municipal Airport
- Dysart Drain - Agua Fria River to Reems Road
- Interstate 10 - Perryville Road to Jackrabbit Trail

Approximate delineations were also computed using normal depth calculations and approximation techniques for the following areas:

- Ponding behind Airline Canal
- Approximate delineations of conveyance corridors behind Interstate 10
- Approximate delineations behind Southern Pacific Railroad where appropriate
- Approximate delineation of Bullard Wash breakout west of Estrella Parkway and south of State Route 80
- Approximate delineations of breakouts along the Dysart Drain onto Luke AFB
- Reems Road approximate delineation from Northern Avenue to Beardsley Road

Refer to the attached 11" x 17" floodplain map for locations and extents of all the above mentioned delineations.

We are in the process of setting up public meetings at two locations within the study area to present the floodplain delineations to the general public. These meetings are tentatively set for the following dates:

- May 11, 1992 from 7:00 to 9:00 PM
  Dysart High School Cafeteria
  11405 North Dysart Road
  El Mirage, Arizona

- May 12, 1992 from 7:00 to 9:00 PM
  Avondale Jr. High
  Central and La Canada
  Goodyear, Arizona.

Actual times and locations will also be published in local papers and municipal water bills if available.

April 28, 1992

Page Four

If you have any questions about the floodplain delineations in this area please contact:

Greg Rodzenko Project Manager Flood Control District of Maricopa County (602) 506-1501	or	Mark Gavan Jeff Erickson The WLB Group, Inc. (602) 279-1016

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

JSE:srm

B:LETTERS\289036\4-17.L

THE CITY OF SURPRISE, ARIZONA

289036

CITY MANAGER'S OFFICE
12604 SANTA FE DRIVE
SURPRISE, AZ 85374
602-583-1080 - FAX 602-583-1084

May 4, 1992

MAY 1992
RECEIVED
WLB-PHX

Jeff S. Erickson, P.E.
The WLB Group
333 East Osborn
Suite 380
Phoenix, Az. 85012

Re: White Tanks/Agua Fria Area Drainage Master Study
Floodplain Delineations- WLB No. 289036

Dear Jeff:

This letter is written as a follow up to our meeting of a week ago. I wish to address the floodplain lines along Santa Fe Drive running from just west of Dysart to Factor Street.

We are seriously concerned over the north boundary of that floodplain area and would ask you to take another look at it to determine whether it is possible to keep this floodplain area south of Santa Fe Drive.

Your attention and consideration to this matter is greatly appreciated.

Happiness,

Richard W. McComb,
City Manager

c.c.: Herschell Morrow

The WLB Group Inc. WLB

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Memorandum

To: File

From: Jeff Erickson

Date: May 10, 1992

Subject: Telephone Conversation w/ Jan Farmer, FCD, about City of Surprise Floodplain Delineation.

Copies To:

Jan said she talked to Pedro and we are to terminate our detailed analysis on the AT+SF railroad channel at Greenway road leaving the existing Zone A delineation upstream of Greenway Rd. We will comply with this request and make the necessary changes.

1.4.2 Study Coordinator (Flood Control District of Maricopa County)

The Flood Control District of Maricopa County reviewed both the hydrology and hydraulics in the White Tanks/Agua Fria ADMS. The following correspondence took place during the review process.

The WLB Group Inc

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March 30, 1990

Mr. Greg Rodzenko
Flood Control District of
Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
Map Check Profiles
WLB No. 289036

Dear Greg:

This letter is written to request your concurrence on our proposed method of field surveying the map check profiles.

Our interpretation of the "FEMA Guidelines and Specifications for Study Contractors" dated September, 1985, would indicate that approximately 135 profiles of one mile in length are required for the 200 square miles of mapping. That would result in 135 miles of profiles which we feel is excessive.

FEMA requires one profile for every three stereo models. We propose to survey the profiles on the mile streets that separate the stereo models. Thereby checking 6 stereo models with each profile. The result would be approximately 70 profiles instead of 135.

We also propose a profile length of 1/4 mile mile instead of one mile as required by FEMA. Typically, the 1/4 mile length will result in profiles that will begin at a control point at the section corner and extend half way to the control point at the quarter section corner. That seems like adequate checking to us.

If you have any questions, please let me know.

Sincerely,

THE WLB GROUP, INC.

Mark T. Gavan

Mark T. Gavan, P.E., R.L.S.
Assistant Vice President

MTG:srm

A:LETTERS/MTG-8936.L

Offices located in Tucson, Phoenix, Las Vegas, and Rancho Cucamonga, California
333 East Osborn Suite 380 · Phoenix, Arizona 85012 · (602) 279-1016

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

BOARD OF DIRECTORS
Betsey Bayless
James D. Bruner
Carole Carpenter
Tom Freestone
Ed Pastor

APR 1990 RECEIVED WLB-PHX

289036

APR 1 0 1990

Mr. Mark T. Gavan, P.E., R.L.S.
Assistant Vice President
The WLB Group
333 E. Osborn Road, Suite 380
Phoenix, Arizona 85012

SUBJECT: FCD 89-70
White Tanks/Agua Fria ADMS
Map Check Profiles

Dear Mark:

This letter is to authorize the procedure for map check profiles on the White Tanks/Agua Fria ADMS.

The procedure outlined in your letter of March 30, 1990 is acceptable, i.e., using 70 profiles of 1/4 mile length each. Please check with FEMA that this procedure is acceptable with them.

Sincerely,

Gregory Rodzenko

Gregory Rodzenko

The WLB Group Inc.

Engineering · Planning
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August 10, 1990

Greg Rodzenko
Project Manager
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 84009

Re: White Tanks ADMS
WLB No. 289036

Dear Greg:

We are submitting for your review the following items:

1. HEC 1 computer runs on floppy disk for White Tanks Flood Retarding Structures # 3 and 4.

2. Related computations and backup material bound in 3-ring binder.

3. Work study map showing delineated watersheds and concentration points.

These are preliminary runs and we would like your comments concerning the setup of the models and the input data associated with them.

Please be aware that the summary table output of storage volumes and elevations are incorrect due to a error in the program. It prints data associated with the last transposition hydrograph area and not the values associated with the interpolated hydrograph for that particular contributing area. This is a problem that HEC is aware of but has not fixed.

We have modeled storage behind Interstate 10 in a simplified manner rather than delineating small subwatersheds that contribute to each culvert. Modeling each small watershed would compromise the integrity of the model by requiring very small time intervals to correctly model the peak flows. Please comment on our technique and as to whether it is appropriate or not.

The WLB Group Inc

If you have any questions, please call Mark Gavan or myself at 279-1016.

Sincerely,
THE WLB GROUP, INC.

Jeff S. Erickson
Project Engineer

JSE:tlg

8936.ltr

To: Greg Rodzenko, Planning

56

Subject: White Tanks - Agua Fria ADMS
Comments on the hydrology for the White Tank Structures

Date: 8/22/90

By: Valerie Rice, Watershed Management

I have reviewed the White Tank/Agua Fria ADMS - Subwatershed for white Tanks Flood Retarding Structures #3 and 4, preliminary submittal for comments, 8/10/90.

Comments for the whole project:

I. General:

1. I would like to see the subbasin boundaries on the larger scale maps that were made.

2. There needs to be some data sheets for the determination of the precipitation.

3. The version of the FCD MCUHP1 and MCUHP2 used to determine Tc should be referenced. The most recent version is dated 5/22/90 compiled on 6/28/90.

II. Soils

1. I'd like to see the subbasin boundaries on the SCS soils maps.

2. We need the weighted average values data sheet for soils that fall within the area that is within the SCS's "Soil Survey of Aguila-Carefree Area, Parts of Maricopa and Pinal Counties, AZ."

3. Soil Data Sheet (map units weighted average values)

   ** a. The PSIF should be interpolated from Table 4.2 in the FCD Hydrology Design Manual rather than averaged. The capillary suction (PSIF) is directly related to the hydraulic conductivity (XKSAT). When the PSIF is averaged from the given values in the table, the number does not always correlate to the averaged XKSAT.

   ** b. In a discussion I had with Steve Waters he suggested that only half of the percentage of rock outcrops be estimated as impervious area (RTIMP).

   c. Is there a reason that the Soils Data Sheets estimate IL?

AUG 27 '90 16:14 FROM FLOOD CONTROL PAGE.002

III. Green and Ampt Parameters:

** 1. For the determination of DTHETA use dry soil conditions for the desert area and normal soil conditions for the agricultural and residential areas.

** 2. As discussed in II.3.a, the PSIF should be interpolated from Table 4.2 rather than averaged.

3. How was vegetation cover estimated?

** 4. The definition between Mountainous, Hillslope, and Alluvium is not clearly stated in the FCD Hydrology Design Manual. The individual site must be looked at with respect to the potential for slowing the flow (roughness) and vegetation. In many of the areas "Mountains", with a vegetated surface, is used to determine the surface retention loss (IA). I believe that many of the areas should be placed in the "Hillslope, Sonoran Desert" category. There are a number of Alluvial Fan areas with relatively flat slopes (<50 feet/mile) that are classified as "Hillslopes". I believe that these subbasins should be classified as "Desert and rangeland, flat slope". Davar Khalili suggests that in some cases an <u>average</u> value can be selected, i.e., a value that falls between alluvial and hillslope. For an example if a watershed is 35% Hillslope and 65% alluvium, the IA can be interpolated between the value from Table 4.1 of the Hydrology Design Manual.

IV. Time of Concentration:

1. The criteria used to determine if the Tc for a subbasin should be determined by using the Clark method or the S-graph method should be based on the size of the subbasin. Most of the subbasins fall within the criteria of the Clark method, yet the majority of the Tc's for the subbasins are determined using the S-graph method.

2. I'd like a map indicating the flow paths.

** 3. As discussed above in III.4, the Kb should be adjusted to estimate the land classification. Kb is a very sensative parameter in the determination of the Tc, thus care must be given in determining Kb.

4. The reference for the slope averaging equation needs to be included in the text.

5. S-Graph Method

a. How was the Lca estimated?

AUG 27 '90 16:15 FROM FLOOD CONTROL PAGE.003

6. Clark Method

** a. The Hydrology Design Manual states in the application section 7.4 Notes on Calculation Parameters for use in the Clark Unit Hydrograph #5. When calculating Tc for natural watersheds with overall slopes greater than 200 feet/mile, use Figure 5.4 to adjust the slope. Therefore any slope less than 200 feet/mile should not be adjusted.

b. If the average slope equation is to be used on flow paths that have distinct grade breaks, the individual slopes that are greater than 200 should be adjusted then plugged back into the equation to get the average slope. We did a sensativity analysis to determine if the length or the height difference (Li and H, respectively) from WLB's average slope equation. Our analysis indicated that the Li should remain constant and the H adjusted.

V. Routing:

1. Was a field survey conducted for typical cross-sections for the normal depth routing?

2. How were the Manning's "n" value estimated? Are there some pictures to indicate typical cross-sections of the washes?

3. How were the velocities determined for the normal depth routing?

4. The HEC-1 Manual uses the equation "Reach length/average velocity/time interval" to estimate NSTPS on the RS card. WLB uses the equation on the RM card (Muskingum Routing) to determine the NSTPS. In some cases the NSTPS increase. I do not know if it is correct to use the Muskingum routing NSTPS for the RS card?

5. Transmission Losses are not indicated in the modeling. It was not specifically written into the scope that transmission losses would be required.

VI. Nomenclature:

1. To route a hydrograph from one concentration point (CP) to the next, use the designation RCPa or Ra (when the number of digits is greater than 6) where "a" indicates the name of the hydrograph being moved. ie: CP1 is being routed to CP2. Call the routed hydrograph RCP1 instead of CP2. This will eliminate the confusion of having two or more concentration points named CP2.

AUG 27 '90 16:15 FROM FLOOD CONTROL PAGE.004

2. Rename the intermediate concentration points. Where there are two tributaries converging and hydrographs need to be added for one of the tributaries use a numbering system that will indicate that the intermediate CP's are not the final combined CP.

ment on White Tank #3 Hydrology:

I. General Comments:

1. Check flows around the dike in Subbasin 16. If the discharge at elevation 1267 in the storage routing includes the weir flow over the dike then the ST card is not needed.

2. Are the dikes going to be modeled as existing conditions? The boundary between subbasin 16 and 17 is considered as a dike that acts as a boundary where the dikes in subbasin 17 are not.

3. Does the routed hydrograph from CP17 need to be routed? It is in the ponding area. Does it make any difference.

4. Rename the combined hydrograph at the structure to WT#3 instead of CP13.

5. In the HEC-1 run, the hydrograph development for subbasin 17 has an incorrect value for the LG card 5th field. The value is 1.000 it should read .40 according to the Green & Ampt loss rate parameters data sheet.

6. Subbasin 3 has a Tc longer than 1.5 hours. The Clark method is used to determine the Tc. The slope for this subbasin is only 27.3 ft/mi., which is relatively flat. The subbasin is classified as "Hillslope - rough and/or moderate vegetation" and corresponding m and b are used to determine Kb. It is my opinion that this subbasin should fall under the category of "alluvial fan, desert rangeland - bare or nearly bare ground". When the parameters for m and b are changed and Kb recalculated, Tc falls below 1.5 hours.

AUG 27 '90 16:16 FROM FLOOD CONTROL PAGE.006

Comments on White Tank #4 Hydrology

I. General Comments:

1. HEC-1 Program

   a. Subbasin 29 has the incorrect LG and UC values according to the Green and Ampt parameter data sheet.

   b. At CP41A2, the diverted flow should be at station 6261+65 not 6251+65.

   c. The hydrograph development block for following subbasins need to be changed:
      41 instead of CP41
      43 instead of CP43-3
      44 instead of CP44
      45 instead of CP45
      47 instead of CP47

   d. The routing of hydrograph from 28 should read R28 rather than 30.

   e. RS47 should be reference as WT4.

2. Does CP44 actually contribute to the storage behind White Tank #4?

3. Subbasins 29 and 31 indicate a Tc longer than 1.5 hours for the Clark method. When I ran the data on the latest version of MCUHP1 (5/22/99 compiled on 6/28/90) I got different answers. The values that the program determine for me were less than 1.5 hours.

II. Routing:

1. I haven't specifically looked at each of the routing parameters yet. All previous general comments regarding Normal Depth routing apply.

2. WLB includes a data section of the dimensions of the channel north of I-10. Something similar should be developed for the natural channels.

III. Culverts along I-10:

1. How was the storage determined for the culverts?

AUG 27 '90 16:17 FROM FLOOD CONTROL PAGE.007

2. I'm not sure exactly what is happening with the culverts under I-10? The stage-storage includes weir flow over a dike. I assume that the dike is I-10. What it looks like a hydrograph is being developed for the subbasins (41A and 41) then routing the entire hydrograph east along I-10 and diverting the flows as a culvert is reached. The hydrograph for subbasin 43 is developed and added 1/2 way into the subbasin. I can think of two other methods to analyze the flows in the culverts under I-10.

    a. The first would be to break down the subbasins so that each culvert had its own watershed. Then the stage-storage and diversion would be applicable for the hydrograph. WLB's letter states that they do not think this is an accepatable technique since modeling the small subbasins would compromise the integrity of the model by requiring very samll time intervals to correctly model the peak flows.

    b. The second method would be to combine all the culverts in the subbasin into a single stage-storage-discharge rating curve.

    c. A combination of the two above method might be the best alternative. This is done for routing a number of the flows through culverts in Subbasin 43, 45, and 46. The subbasin should be delineated for the combined culverts, then routed.

2. The stage-storage rating curve includes the weir flow over the dike (I-10?) but is not included in the diversion if the elevation is over the freeway.

3. Where did the elevation for the top of the dike come from? If the dike is I-10, did WLB use the I-10 as-builts?

4. WLB did not include stage-storage for the dike at Tuthill Rd. alignment and I-10 (referred to hereafter as the Tuthill Dike). The top of the Tuthill Dike is at 1092.1. Any flows above that elevation would flow to the east. In WLB's modeling the flows do not reach that elevation because of the large excavation pit on the Caterpillar Proving Grounds. But what if these pits were filled in the future. It might be handy to have the stage storage determined to indicate overtopping of the Tuthill Dike to the east.

5. The culvert for CP45-1 was not included in the HEC-1 model.

6. There is no analysis to determine if flows exceed the capacity of the channel along Jackrabbit Trail.

AUG 27 '90 16:18 FROM FLOOD CONTROL PAGE.008

I suggest having a meeting the Dave Creighton of the Arizona Department of Water Resources. Since ADWR is the leading agency in the state for FEMA Mr. Creighton's concerns and suggestions will need to considered. We would like to bring ADWR in on the review process as soon as possible to reduce the risk of delay caused by addressing any comments that Mr. Creighton might have.

RESPONSE TO FCD COMMENTS

I. General

1. O.K. Pencil Manuscript Form Submitted Later
2. We can provide these. Will send to them next Tuesday.
3. We need most recent version and should be sent any updates as they are made. Recieved latest version from Steve + Ghassan

II. Soils

1. Can provide.
2. Can provide.
3. a) No mention in manual about interpolating values as far as we are able to ascertain.

   FCD manual examples show these are weighted values. Suggest clarification.

   b) Why - Ask about residential impervious areas and what % they think is appropriate. TR-55, City of Phoenix

   Ask Dave Creighton about ag + desert areas.

   c) Yes, we may not always use Green-Ampt, and this is the same form as FCD soil tables.

III. Green and Ampt Parameters May take a couple subbasins to model the ag reservoirs.

1. We believe normal is appropriate for all areas. Need to discuss.
2. Look at FCD's own examples. Use wet-normal with full with partial full
3. Used field investigation, aerial photos and good engineering judgement. Define scale of aerial photos.
4. Need to discuss. We will look at this.

IV. Time of Concentration We will change these to Clark

1. It was our understanding that the Mountain S-graph was to be used in

mountainous areas we can accommodate and use Clark if so directed.

2. We can provide. Show on work Maps.
3. This will be completed correctly if land classification %is chosen correctly.
4. O.K.
5. S-graph method
   a) To the centroid of the watershed.
6. Clark Method
   a) Look at graph we copied from manual. We can discuss. Change this
   b) ~~Let's discuss~~. We will change this

V. Routing

1. No, taken from 400 scale 2 ft. topo maps.
2. We can provide pictures and documentation.
3. First was an estimate and then plus discharges into a similar trap channel to compute actual velocities, or taken directly from HEC-1 routing data after 1st run.
4. Basically same equation.
5. Discuss this. Calculate cfs RL card in/hr converts to wetted acre ft. Talk to Dave Creighton + set up meeting with AOWR.

VI.

1. O.K.
2. O.K.

I. General Comments

1. O.K., we'll take out.
2. Yes. Don't understand this comment.

3. Yes. We think it should be routed for low flows to the low point behind the dam.
4. OK.
5. We will change.
6. We believe we are still correct here. Look at subbasin on topo. Ask what happens with Tc 1.5 hrs. Break Down Subbasins

I. General Comments

1. a) Ours seem to be o.k. Give Val new sheet on.
   b) Typo - will change.
   c) Typo - will change.
   d) Typo - will change.
   e) O.K.
2. We will change this subbasin.
3. We need new update. Break Subbasins Down if Tc ≥ 1.5 hrs

II. Routing

1. O.K.
2. O.K.

III. Culvert Along I-10

1. From I-10 plans and Topo maps.
2. a)
   b) > discuss these
   c)
   model seperately and input hydrographs
2. Routed along freeway. Nothing goes over.
3. I-10 plans - No - Yes

We can add future case for future

4. Yes, we did and this is only existing conditions model.

5. This was judgement call for existing case.

6. Correct. We will perform HEC-2 and then refine HEC-1 model.

Talk HEC-2 and run to check breakout.

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Transmittal

To: Greg Rodzenko
Flood Control District of
Maricopa County

Date: 9-5-90

Job No.: Z89036

Drawing/Spec Reference:

Re: White Tanks Precipitation Data Sheets

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

Material Format		Requested Action	
☐ Letter	☐ Shop Drawing	☐ For Your Approval	☒ Your Review
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☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☐ Sample	☐ C.O./C.B.	☐ As Requested	☐
☐	☐	☐	☐

Remarks: Greg,
Here are the precipitation data sheets as requested by Valerie Rice. Please give these to her so she can review them along with Steve Waters & Dawn. If they have any questions, have them call me.

Copies To: File

By: Jeff S. Einhorn

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Transmittal

To: Greg Rodzenko
Flood Control District of Maricopa County

Date: 11/30/90
Job No.: 289036
Drawing/Spec Reference:

Re: White Tanks/Agua Fria ADMS

We Transmit [X] Herewith [ ] Under Separate Cover [ ] Via

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[ ] Sample	[ ] C.O./C.B.	[ ] As Requested _____	[ ] _____
[ ] _____	[ ] _____	[ ] _____	[ ] _____

Remarks: Greg,

We are submitting for your review cross section and centerline locations to be used for HEC-2 floodplain delineations as set forth in the scope of work. Please have the hydraulics division look these over and make any comments or corrections they deem necessary. We will need these prints back after you have reviewed them.

Copies To:

File

By: Jeff S. Erickson
Project Engineer

JF 289036

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January 15, 1991

Greg Rodzenko
Flood Control District of Maricopa County
3335 W. Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
Preliminary Hydrology for Phase 1
WLB No. 289036

Dear Greg:

We are submitting the preliminary hydrology for Phase 1 of the White Tanks/Agua Fria ADMS. Please review this submittal and make comments as necessary. The following items are included in this transmittal:

1. HEC-1 Hard Copy of Phase 1.
2. Summary of Peak Flows in Numerical Order.
3. MCUHP1 and MCUHP2 Input Data.
4. MCUHP1 and MCUHP2 Output Data.
5. Routing Parameters.
6. Diversion Tables.
7. Pictures for Backup Documentation of Mannings "n" Values.
8. Table of Weighted Soil Map Units for the Study Area.
9. 1" = 4000′ Drainage Area Map (3 copies).
10. 1" = 400′ Work Maps with Delineations and Flow Paths.
11. SCS Soil Maps with Delineations.
12. Notes Explaining Assumptions and Procedures Used.
13. Copy of HEC-1 Run Used to Calculate How the Areas Associated with the Diverts are Combined.

We will require the return of the 1" = 400′ work maps and SCS soil maps when you are finished with your review as these are the originals and we do not have copies.

The WLB Group Inc.

January 15, 1991
Greg Rodzenko
Page Two

We would like to call your attention to a recurring inconsistency in the HEC-1 model that we believe to be largely a result of the considerable difference in rainfall intensity between rainfall pattern #1 and pattern #2. The problem is best described with an example. Please refer to the preliminary hydrology for Phase 1. You will see that subbasin No. 104 generates a peak flow of 252 cfs which is routed across subbasin No. 112 and attenuates to a peak of 111 cfs. the peak from subbasin No. 112 is 1054 cfs. These two flows combined at CP112 for a peak of 1017 cfs. Therefore, the combined peak at CP112 is lower that the individual peak from subbasin No. 112. This inconsistency is one which is repeated throughout the model in the range of drainage areas from 0.5 to 3 square miles. We should have a meeting after your review of the Phase 1 hydrology and discuss this problem.

If you have any questions or need more backup documentation, please call Mark Gavan or myself.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson
Project Engineer

JSE:tlg

letters\289036\1-15.jse

The WLB Group Inc.

Notes on Preliminary Hydrology for
Phase 1 of the White Tanks/Agua Fria ADMS

The following assumptions and procedures were employed for the preliminary hydrology for Phase 1 of the White Tanks/Agua Fria ADMS.

1.) The area for each divert was calculated by taking the percentage of flow associated with the divert compared to the total discharge at that particular concentration point and applying those areas proportionally to each divert or remainder. A copy of the HEC-1 run used to calculate these areas is included in this submittal.

2.) Storage routines were ignored for the agricultural reservoirs as our assumption was on the conservative side with regards to the design of future structures. There is no guarantee that these will be in place for future development.

3.) Structures on the Dysart Drain, ATS&F railroad, etc. will be defined more accurately by the HEC-2 analysis that will be preformed at a later date and we will then incorporate these storage - discharge routines along with their appropriate diverts in the final hydrology run. Otherwise, there are very few structures within Phase 1.

4.) The 10/2/90 version of MCUHP1 & 2 was used for this run.

5.) Vegetative cover was estimated using aerial photos for the entire area and then average values for each subbasin were employed.

6.) The slope averaging equation used in the S-graph method was taken from the Hydrology Manual for Pima County.

7.) Velocities for normal depth routing were computed by first running a model with estimated velocities and then refining these velocities based on storage - outflow data computed within HEC-1. Average Area = Average Storage/Reach Length x 43560, then Velocity = Average Discharge/Average Area.

8.) Precipitation data was computed as an average over the whole watershed and the corresponding computations were submitted previously.

9.) Diversions were computed by taking cross sections along the roads and/or control sections next to the road and normal depth flows were estimated for each divert.

10.) Cross sections for normal depth routing were taken from the 400 scale topographic maps.

11.) The subbasins above Grand Avenue were not included in this model, but will be included in the final HEC-1 run.

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Transmittal

To: Tom Hieb

Flood Control District

Date: 1-29-91

Job No.: 289036

Drawing/Spec Reference:

Re: White Tanks / Agua Fria ADMS

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☐ Sample	☐ C.O./C.B.	☒ As Requested	☐
☐	☒ Floppy Disk	☐	☐

Remarks: Tom,

Here is the HEC-1 runs for PHASE I. Both input & output files are designated as WTPHS1 & WTPHS1.OUT. If you need anything else please feel free to call and we are looking forward to working with you!

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By: [signature]
Project Engineer

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Report of Meeting

Project title: WHITE TANKS/ AGUA FRIA ADMS Date: 2-13-91

Job no. 289036 Prepared by: JSE

Report to: Greg Rodzenko, Tom Heib, Tom Donaldson — FCD
Amir Motamedi

Re: PHASE 1 HYDROLOGY COMMENTS

Date of meeting: 2-13-91 Time: 1:30 To: 3:30

Place of meeting: Flood Control District

Attendees: FCD — Greg Rodzenko, Tom Heib, Tom Donaldson, Amir Motamedi
WLB — Mark Gavan, Jeff Erickson, Ghassan Aouad

Comments: This meeting was held to discuss comments on the preliminary submittal of the Phase 1 Hydrology for the White Tanks / Agua Fria ADMS. The following comments and recommendations were made:

1. Increase number of ordinates from 144 to 300 to capture peak flow.
2. Look to see for sure if we capture the total peak with 300.
3. Recommended that S-graphs be used for entire watershed as Clark Method would be very weildly for different storm events and sometimes the T_c is not being calculated correctly.
4. Diverts were taked about and it was decided to use entire drainage area above that particular concentration point.
5. Loss Rate Parameters — It was decided that we would look at the percent cover for typical ag areas and think about whether saturated or normal conditions would be applicable. We sent over a sensitivity run and are awaiting a decision.

(cont.)

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Form 114

(6) Tom wanted to know if our soils tables had interpolated PSIF values. We checked and found out that these values were not interpolated, however the values used for each subbasin are interpolated from the XKSAT value for bare ground. We will change the soil table with our final submittal so everything will match up.

(7) Mark Gavan mentioned that ADOT is using the same types of procedures in their new manual and suggested that Greg have someone in FCD contact them to talk about the problems and general comments that FCD has recieved on it's new method.

(8) Explained Diversion cross-sections

(9) Explained method for obtaining average velocities for the routing reaches using stage-storage-discharge information from a full printout of HEC-1 run.

(10) Tom suggested that we check velocities in routing reaches as some were less than 1ft/sec and seemed slow. He also mentioned there was no data sheet for R/O/A.

The meeting ended and we said we would get back to them on the issues that were not resolved.

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Transmittal

To: Tom Heib

FCD

Date: 2-18-91

Job No.: 289036

Drawing/Spec Reference:

Re: White Tanks/ Agua Fria ADMS – Sensitivity Analysis

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☐ ~~HECH + DATA~~	☐	☐	☐

Remarks: Tom,

This is another sensitivity analysis we computed and it seems to give more reasonable results than the previous one. A Basin factor of .12 was used for both fallow and vegetated conditions along with normal and saturated field situations. We believe this more accurately models the actual field condition than the previous model and therefore we are comfortable with the results.

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By: [illegible], P.E.

FLOOD CONTROL DISTRICT of Maricopa County

289036

77

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

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MAR 8 1991

Mr. Jeff Erickson, P.E.
The WLB Group
333 East Osborn, Suite 380
Phoenix, Arizona 85012

SUBJECT: White Tanks ADMS

Dear Jeff:

This letter is to confirm and clarify information agreed upon in our meeting of February 13th and our subsequent telephone conversations of February 14th and 15th.

From our discussions with the WLB staff and the sensitivity analyses submitted by Jeff Erickson, the following changes to the hydrologic methods and assumptions were agreed upon:

1. The S-graph method should be used to generate runoff hydrographs for all the subbasins in the model. A Kn value of 0.12 would be typical for agricultural subbasins and Kn = 0.03 would be typical for urban subbasins. These values may vary to reflect nontypical hydrologic or hydraulic conditions, or if the resulting peak flows are unrealistic.

2. The area assigned to each concentration point should correspond to the total area of all subbasins that drain, either partially or fully to that point.

3. Rainfall loss parameters for agricultural areas should be calculated based on a fully vegetated condition and a saturated soil profile (DTHETA = 0).

The following review comments regarding the material submitted on January 15, 1991, should also be addressed:

1. The number of time steps used in the model should be increased from 144 to 300. When a model for the entire study area is completed, the output should be checked to determine if 300 ordinates is adequate to estimate peak flows and ponding volumes at all critical concentration points in the study area.

2. Routing reaches with velocities of less than one ft/s need to be checked to ensure that the cross section used is representative of the entire routing reach.

3. Routing reach R102 needs to be checked. The peak flow in this reach increases after routing.

4. No data sheet was available for routing reach R100A.

5. A label for diversion 2D147 and an arrow for direction of flow between subbasins 197 and 198 should be included on the watershed map.

Please review the above-given information. If you have any questions pertaining to these comments, please call me at 262-1501.

Sincerely,

Tom Hieb
Hydrologist

The WLB Group Inc

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March 20, 1991

Greg Rodzenko
Flood Control District of Maricopa County
3335 W. Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
Preliminary Hydrology for Phase 2
WLB No. 289036

Dear Greg:

We are submitting the preliminary hydrology for Phase 2 of the White Tanks/Agua Fria ADMS. Please review this submittal and make comments as necessary. The following items are included in this transmittal:

1. Floppy Disks Containing Input and Output Data for Phase 2 HEC-1 Computer Runs.
2. HEC-1 Hard Copy of Phase 1.
3. Summary of Peak Flows in Numerical Order.
4. MCUHP1 and MCUHP2 Input Data.
5. MCUHP1 and MCUHP2 Output Data.
6. Routing Parameters.
7. Diversion Tables.
8. Pictures for Backup Documentation of Mannings "n" Values.
9. Table of Weighted Soil Map Units for the Study Area.
10. 1" = 4000′ Drainage Area Map (3 copies).
11. 1" = 400′ Work Maps with Delineations and Flow Paths.
12. SCS Soil Maps with Delineations.
13. Notes Explaining Assumptions and Procedures Used.
14. Copy of the Drainage Area Map used to Calculate Area for the Combination of Hydrographs.
15. Stage-Storage-Discharge tables.
16. Velocity Computations for each Routing Reach.

We will require the return of the 1" = 400′ work maps, SCS soil maps and drainage area map with area calculations when you are finished with your review as these are the originals and we do not have copies.

The WLB Group Inc.

March 20, 1991
Greg Rodzenko
Page Two

We will be submitting the entire area as one complete model once comments have been received for Phase 2 and this will include previous comments for WT#3, WT#4, and Phase 1.

If you have any questions or need more backup documentation, please call Mark Gavan or myself.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson
Project Engineer

JSE:tlg

jse.ltr

Notes on Preliminary Hydrology for
Phase 2 of the White Tanks/Agua Fria ADMS

The WLB Group Inc.

The following assumptions and procedures were employed for the preliminary hydrology for Phase 2 of the White Tanks/Agua Fria ADMS.

1.) The area assigned to each concentration point should correspond to the total area of all subbasins that drain, either partially or fully to that point. A copy of the Drainage Area Map run used to calculate these areas is included in this submittal.

2.) Storage routines were ignored for the agricultural reservoirs as our assumption was on the conservative side with regards to the design of future structures. There is no guarantee that these will be in place for future development. Other storage areas behind canals, roads, railroads, etc. have been taken into account.

3.) Structures and diversions along canals, railroads, and Interstate 10 will be defined more accurately by the HEC-2 analysis that will be preformed at a later date and we will incorporate these storage discharge routines along with their appropriate diverts in the final hydrology run. Diverts were still calculated approximately using the 400 scale mapping.

4.) The 10/2/90 version of MCUHP1 & 2 was used for this run.

5.) Agricultural areas were assumed to be under full crops with a saturated soil condition as agreed upon by the Flood Control District.

6.) The slope averaging equation used in the S-graph method was taken from the Hydrology Manual for Pima County.

7.) Velocities for normal depth routing were computed by first running a model with estimated velocities and then refining these velocities by using an average discharge computed by the HEC-1 run and then plugging in these values with an average trapezoidal cross-section to compute velocities thus refining the Routing Steps involved.

8.) Precipitation data was computed as an average over the whole watershed and the corresponding computations were submitted previously.

9.) Diversions were computed by taking cross sections along the roads and/or control sections next to the road and normal depth flows or weir flows were estimated for each divert.

10.) Cross sections for normal depth routing and weir flow calculations were taken from the 400 scale topographic maps.

11.) River routing was accomplished by assuming a 1000 foot wide channel. We were then able to keep the model continuous by routing flows around the edge of the drainage area in both the Agua Fria and Gila Rivers.

12.) S-graphs were employed for the entire area as directed by the Flood Control District.

The WLB Group Inc. WLB

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Transmittal

To: Tom Heib
Flood Control District of Maricopa County

Date: 4-8-91

Job No.: 289036

Drawing/Spec Reference:

Re: 24-Hour Storm on White Tanks ADMS Structure #4

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

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☐ Prints	☐ Modification Drawings	☐ Information	☐ Make Recommendation
☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☐ Sample	☐ C.O./C.B.	☐ As Requested	☐
HEC-1 Run	☒ Floppy Disk	☐	☐

Remarks: Tom,

Here is the 24-hour run with the SCS Type II distribution for White Tanks #4. This run produces higher volumes and peak discharges for areas above the Pattern #2 distribution associated with the 6-hr storm event. Please call us if you have any questions and you may want to show this to Rumann, Tram, Johnson and any other interested parties as this may affect the 100-yr floodplain delineations that we will be delineating throughout the watershed.

Copies To: File

By: [signature] P.E.

The WLB Group Inc

Engineering · Planning
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May 10, 1991

Mr. Greg Rodzenko
Flood Control District
of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Re: White Tanks ADMS
Meeting Minutes
Date: May 10, 1991
Time: 10:00 AM
Attendees: Greg Rodzenko, Joe Tram, Jan Opstein, Tom Heib - FCD
Mark Gavan, Jeff Erickson (Minutes), Ghassan Aouad - WLB
WLB No. 289036

Comments:

1. Tom suggested we recheck our areas for diverts to make sure they are added up correctly.

2. Joe Tram was leaning toward the use of the 24-hour storm to define floodplain delineations. We will set up a meeting later next week to discuss this along with a discussion of whether to delineate ponding areas behind canals and roadways using HEC-1 or HEC-2.

3. WLB will run a comparison of Mountain S-graphs vs. Valley S-graphs and get back to Tom with our results.

4. HEC-2 runs behind structures will start at the low point or outlet and then match into the 100-year WSEL behind the structure.

5. Tom would like us to document the impending improvements in the watershed that will take place within the next year.

6. We will also document our peak discharges versus previous studies in the watershed.

cc: Tom Heib
Joe Tram
Jan Opstein

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333 East Osborn Suite 380 • Phoenix, Arizona 85012 • (602) 279-1016

Transmittal

To: Mr. Greg Rodzenko
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Date: May 16, 1991

Job No.: 289036

Drawing/Spec Reference:

Re: White Tanks ADMS Mapping Format

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

Material Format		Requested Action	
☐ Letter	☐ Shop Drawing	☒ For Your Approval	☒ Your Review
☐ Memo	☐ Clarification Drawing	☐ For Your Signature	☐ Please Comment
☐ Prints	☐ Modification Drawings	☐ Information	☐ Make Recommendation
☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☒ Sample	☐ C.O./C.B.	☐ As Requested	☐
☐	☐	☐	☐

Remarks: Greg,

Please find enclosed in this submittal three prints of a typical White Tanks ADMS 400 scale contour map. This is how we envision the final product will look with the exception of the new certification. We have also enclosed copies of this. Please review this and have any people you feel may have some ideas about what they want to see on the final product look at it also. If you have any questions, call Mark Gavan or myself at 279-1016.

Copies To:

By: Jeff S. Erickson, P.E.

The WLB Group Inc.

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May 23, 1991

Mr. Greg Rodzenko
Project Manager
Flood Control District
of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Re: White Tanks ADMS
Meeting Minutes-Discussion of Floodplain Delineations
Date: May 22, 1991
Time: 2:00 PM
Place: Flood Control District of Maricopa County
Attendees: Greg Rodzenko, Joe Tram, Jan Opstein - FCD
Mark Gavan, Jeff Erickson (Minutes), Ghassan Aouad - WLB
WLB No. 289036

The reason this meeting was called was to discuss procedures for delineating floodplains along the canals, railroads, and highways.

Discussion:

Sections along I-10 were discussed first. Joe Tram indicated that ponding areas can be delineated by using the peak stage output from the 24-hour HEC-1 model. A phone call received from Jan Opstein on 5/23/91 indicated the need to designate these ponding areas throughout the study area as Zone AE. Joe also said that the conveyance, parallel to I-10, leading to the ponding areas could be delineated using simple normal depth calculations and should be designated with a zone A (with no water surface elevations).

The RID Canal and the Buckeye Canal floodplain delineations will be based on top of canal bank elevations to define water surface elevations for ponding areas. HEC-1 water surface elevations would also be utilized where they were computed. It was decided to round up the top of bank elevations to the nearest foot to account for accuracy limits in the mapping. The delineations which are based on the peak stage from the HEC-1 model will be designated zone AE. The delineations which are based on top of bank elevation will be designated zone A. Some judgement will be made in a few areas where a HEC-2 run may be justified, based on significant parallel conveyance.

The WLB Group Inc.

May 23, 1991
Meeting Minutes
Page Two

The delineations along the Southern Pacific Railroad will be handled in the same manner as described in the I-10 delineations.

Joe indicated that the Agua Fria River dike had ponding areas designated behind it and a study was done by Jerry Jones to ascertain the water surface elevations. He will get us a copy of the report. We will also delineate these areas using the peak stage from our HEC-1 model.

He told us not to worry about delineating the gravel pit area behind the RID canal east of Dysart Road unless ponding is caused by offsite flows. We will check into this and get back to the Flood Control District.

Jan Opstein said she will send us a copy of the ADWR format for floodplain delineations.

It was also decided to use the 100-year/24-hour storm to obtain peak discharges and peak water surface elevation. We will check the model of the 100-year/6-hour storm to see whether it generates higher peak Q's on the upstream end of the study reaches.

If you have any questions or would like to clarify the documentation in these notes, please call Jeff Erickson at 279-1016.

cc: Joe Tram
Jan Opstein

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Transmittal

To: Tom Heib
Flood Control District

Date: 6-3-91

Job No.: 289036

Drawing/Spec Reference:

Re: White Tank ADMS Hydrology

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

Material Format		Requested Action	
☐ Letter	☐ Shop Drawing	☒ For Your Approval	☒ Your Review
☐ Memo	☐ Clarification Drawing	☐ For Your Signature	☐ Please Comment
☒ Prints	☐ Modification Drawings	☐ Information	☐ Make Recommendation
☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☐ Sample	☐ C.O./C.B.	☐ As Requested	☐
☒ HEC-1 Runs	☐	☐	☐

Remarks: Tom,

Please find the following enclosed:

1. HEC-1 run & floppy disk for 6-hour event, entire watershed.
2. HEC-1 run + floppy disk for 24-hour event, entire watershed.
3. 2 sets of corresponding drainage area map.
4. Your red-lined map.
5. WLB map showing cumulative drainage areas.

I will need the return of the cumulative drainage area map when you are finished.

Copies To: File

By: [illegible] P.E.

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

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JULY 09 1991

Mark T. Gavan, P.E.
WLB Group
333 E. Osborn, Suite 333
Phoenix, Arizona 85012

Subject: White Tanks ADMS
Hydrology Review

Dear Mr. Gavan:

The Flood Control District (FCD) review of the White Tanks hydrology is complete. Following are review comments:

1. Routing reach SR271 is unstable. Peak flow increases from 1104 to 1314 cfs in this reach. The rating curve for this routing needs to be refined so that this does not occur.

2. Routing reach R374 should be checked and revised if necessary. The difference between peak inflow and outflow for this reach is approximately 8 hours.

3. Routing reach R290 caused the HEC I program to crash during the run for the 6 hour model. This routing is not necessary since the routing reach is only 120 feet long. It should be removed from the calculations and left in the code as a comment.

Otherwise, the 24 hour hydrology model is suitable for floodplain mapping.

An aside - FCD staff have been impressed with the quality of work delivered by the WLB Group. The hydrologic modeling has been well conceived, the quality control on deliverables has been outstanding. All of the staff at the WLB Group has been very responsive to questions and easy to work with. My compliments.

Please call with any questions.

Sincerely,

Greg Rodzenko

Gregory Rodzenko
Regional Drainage Planner

Enclosure

The WLB Group Inc.

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July 24, 1991

Greg Rodzenko
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85007

Re: White Tanks/Agua Fria ADMS
HEC-2 Floodway and Floodplain Delineations
WLB No. 289036

Dear Greg:

We are making our first partial submittal of the White Tanks/Agua Fria ADMS HEC-2 floodplain and floodway delineations. The following streams and ponding areas have been delineated.

1. Tuthill Dike Wash - 'Wash 5'
2. Wash 5A
3. Wash 5B
4. Wash 5C
5. Wash 5D
6. Wash 5E
7. Wash 5EI
8. White Tanks Structure No. 4
9. Ponding and conveyance along I-10 West of Tuthill Dike.

These streams and ponding areas are tributary to the Tuthill Dike Wash. The following items are being submitted for your review.

1. HEC-2 floodplain and floodway data (hardcopy and floppy disks).
2. Topographic mapping (scale 1"=400') with delineations (we will need these returned as they are originals).
3. Backup documentation for I-10 delineations.
4. Drainage area maps (2).
5. Notes on hydraulic modeling.

Please have Tom Heib give the 100-year, 24-hour hydrologic model and documentation pictures of the Manning's "n" values to Jan Opstein for her review of the HEC-2 models.

The WLB Group Inc.

Greg Rodzenko
July 24, 1991
Page Two

We will be submitting other delineations as we finish them and to help speed up the review process. If you have any questions, please call me at 279-1016.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

JSE:tlg

B:letters\289036\7-24.1

NOTES FOR FLOODPLAIN
AND FLOODWAY DELINEATIONS

The WLB Group Inc

Tuthill Dike Wash and Tributaries

1. The floodplain and floodway delineations just upstream of the I-10 box culverts on Tuthill Dike Wash were delineated by estimating the effective flow area through the ponding area. We are not sure how the ponding area should be identified (Floodplain or Floodway) since any encroachment into the ponding area will lead to increased peak discharge which will spill east over the dike. In addition, any raising of the dike will raise the ponding water surface elevations.

2. A floodway was also delineated on Tuthill Dike Wash through the detention basin on the southeast corner of Subbasin 42.

3. The dike confining Wash "5D" is not engineered and will breech during the 100 year storm event. We may want to designate a Zone A downstream of the dike to identify the flood risk.

4. Wash 5C has an interesting modeling problem between cross-section 1.196 to 1.416. Flow can cross this road in many places as it flows south along the west side of the road. During field reconnaissance, after the last big storm, it was discovered that the road had washed out where the floodplain is now designated. Therefore, our assumption was to take the entire flow across at this point. Since we can't be sure where the road will wash out in the future, we may want to delineate floodplains on several of the washes downstream of the road. Please advise. Also, please note that there are no culverts under the road.

5. We did not delineate a floodplain between the ponding area in Subbasin 23 and the outflow from the ponding area in Subbasin 21 as this is a big pit with an undefined channel connecting the two ponding areas. The peak discharge out of the ponding area in subbasin 21 is only 174 CFS.

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August 1, 1991

Greg Rodzenko
Flood Control District of Maricopa County
3335 W. Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
Response to HEC-1 Comments
WLB No. 289036

Dear Greg:

This letter is in response to the comments made by Tom Heib on July 9, 1991 on the White Tanks/Agua Fria ADMS HEC-1 model. We have responded to each comment as follows:

1. Routing reach SR271 was refined and now provides a stable rating curve.

2. We checked routing reach R374 and found that indeed the peak outflow occurs approximately 8 hours later. This is due to a large inflow hydrograph with a peak around 24 hours into the storm. We have included these hydrographs for your review. Therefore, we did not change this routing reach.

3. Routing reach R290 was taken out per your comments.

We have included a new copy of the HEC-1 model and floppy disk of the updated run and copies of the new Drainage Area Map. This includes revised diversions along Dysart Drain as a result of the HEC-2 work that has been completed along this stretch. Please discard the old HEC-1 data so that there will be no confusion regarding the most recent run. Thank you for your comments and if you have any other questions, please call me at 279-1016.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

JSE:tlg

cc: Tom Heib, FCD

MEMORANDUM

TO: The WLB Group, Jeff Erickson

FROM: FCDMC, Jan Opstein

Date: August 6, 1991

Subject: White Tanks ADMS, Floodplain Delineation, Summary of Comments for:
Tuthill Dike Wash - Wash 5
Wash 5A
Wash 5B
Wash 5c
Wash 5D
Wash 5E
Wash 5E1
White Tanks Structure No. 4

FROM: FCDMC, Jan Opstein

1. What is the difference between 5M and 5 Tuthill Dike.
2. Use 10.6 encroachment method, on all floodway analysis to optimize encroachments based on obtaining a target difference in energy grade line elevation between natural and encroached conditions.
3. X-Section in which "n" value permits, adjust channel stations to maximize conveyance area and reduce encroachment area.
4. Ineffective flow areas must be blocked out.
5. Extended cross sections should not occur in the floodplain analysis.
6. Identify areas of ineffective flow for cross sections, 1.152, 1.198, 1.260, 2.170 in Wash "5"-Tuthill Dike, 0.305 in Wash "5C"-Tuthill Dike, 1.188 in Wash "5B"-Tuthill Dike.
7. Revaluate cross sections in which the channel stations area outside the encroached area. If channel stations remain outside the encroached area then the encroachment should span the full width of the cross section.
8. Reorient cross sections: 0.324, 0.381, 0.408, 0.477, 0.585, 0.969 of Wash "5E" To Ponding Area; 0.240, 0.384, 1.553, 1.604, 1.676, 1.764, 1.843, 1.916 of Wash "5B"-Tuthill Dike; 0.456, 0.602, 0.705, 0.801 of Wash "5A" Caterpillar.
9. Need to review "N" value documentation, summary of the field inspections and photographs to document "N" values.

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Transmittal

To: Jan Opstein
Flood Control District of
Maricopa County

Date: 8-6-91

Job No.: 289036

Drawing/Spec Reference:

Re: Dysart Drain HEC-2 Floodplain Analysis

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

Material Format		Requested Action	
☐ Letter	☐ Shop Drawing	☐ For Your Approval	☒ Your Review
☐ Memo	☐ Clarification Drawing	☐ For Your Signature	☐ Please Comment
☒ Prints	☐ Modification Drawings	☐ Information	☐ Make Recommendation
☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☐ Sample	☐ C.O./C.B.	☐ As Requested	☐
☒ HEC-2 Run	☒ Hardcopy	☐	☐

Remarks: Jan,

Please find enclosed a preliminary submittal for Dysart Drain. Included are originals of the 400' scale topos with cross sections, HEC-2 hardcopy, and HEC-2 run on floppy disk. Tom Heib has the new hydrology for this area. Please review and give us your comments.

Copies To: Greg Rodzenko, FCD

By: [signature]
Project Engineer

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August 7, 1991

Mr. Greg Rodzenko
Flood Control District of Maricopa County
3335 W. Durango
Phoenix, Arizona 85007

Re: White Tanks/Agua Fria ADMS
WLB No. 289036

Dear Greg:

This is a submittal of the 1"=400' topographic mapping for the White Tanks/Agua Fria ADMS. We are submitting a set of prints at this time and will turn over the mylars once the entire project has been completed. Also included, is the original set of base mylars provided by the aerial mapping companies and a copy of the reference elevation marks.

If you require any additional prints of the mapping, please call and we will provide what is needed during the remainder of the study.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

JSE:tlg

B:letters\289036\8-7.1

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August 7, 1991

Mr. Greg Rodzenko
Maricopa County Flood Control District
3335 E. Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
WLB No. 289036

Dear Greg:

Please find enclosed the key index to the new numbering system for the White Tanks/Agua Fria ADMS mapping. We have also included the old numbering system for the original base map mylars. The new numbering system incorporates the existing White Tanks mapping. You should probably file the prints and base mylars submitted previously with the existing mapping so that the mapping is all together until this project is finished at which time you will receive the complete set of new mylars.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

JSE:tlg

B:letters\289036\8-7.12

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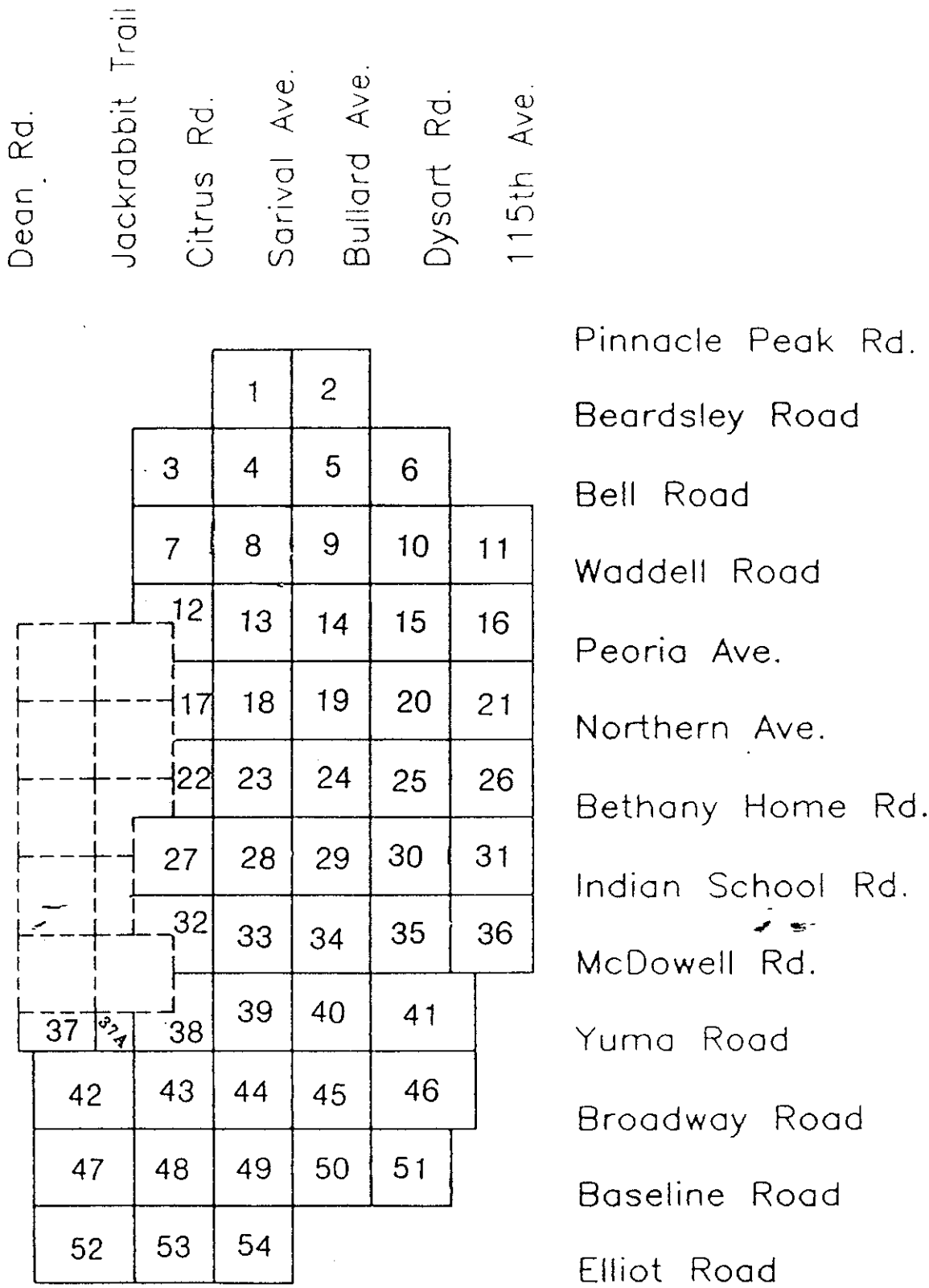
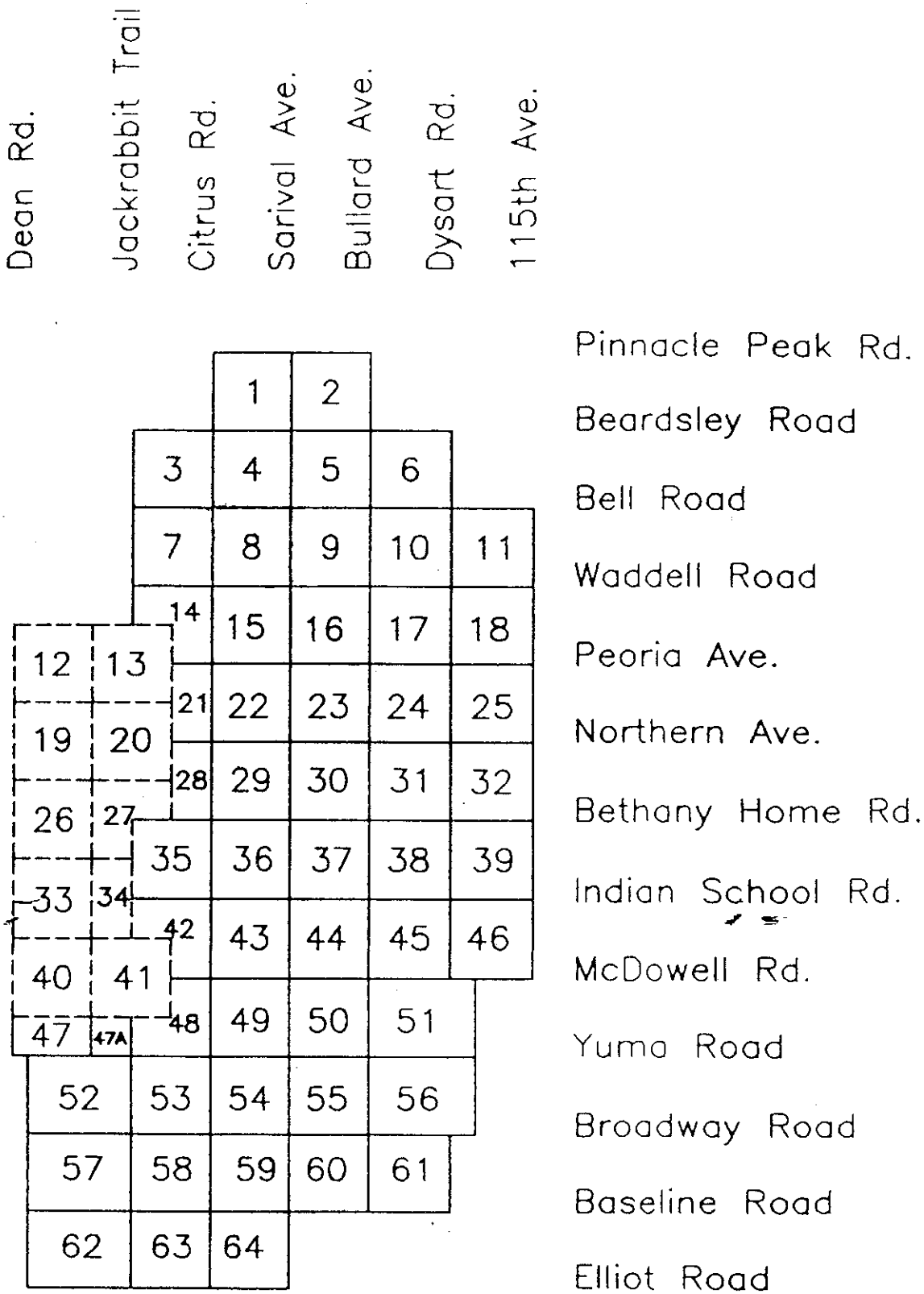
WHITE TANKS / AGUA FRIA ADMS



OLD NUMBERING SYSTEM

WHITE TANKS / AGUA FRIA ADMS



NEW NUMBERING SYSTEM

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AUG 1991 RECEIVED WLB-PHX

MEMORANDUM

TO: The WLB Group, Jeff Erickson

FROM: FCDMC, Jan Opstein

Date: August 13, 1991

Subject: White Tanks ADMS, Floodplain Delineation, Summary of Comments for: Dysart Drain

FROM: FCDMC, Jan Opstein

1. What were the determining factors used in selecting the starting water surface elevations in the HEC2 model for Dysart Drain? A review of the Agua Fria River floodplain study indicates the base flood elevation to be 1059.4 feet at Dysart Drain.

2. Identify areas of break out to the south, as a "Zone A", delineating the flood hazard greater than one foot. Document in the HEC2 input deck areas where the flow break out to the south.

3. Identify areas in which the channel capacity is exceeded and where ponding occurrs along the north side of the drain, both in the HEC2 input deck and on the flood hazard map.

4. Cross sections with the HEC2 warning comment, "extended cross sections" need clarification in the input deck.

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September 9, 1991

Greg Rodzenko
Flood Control District
of Maricopa County
3335 W. Durango St.
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
WLB No. 289036

Dear Greg:

We are submitting the following floodplain/floodway delineations for your review.

A. 1.H2I - Beardsley Canal Wash
B. 1A.H2I - Cholla Wash
C. 1A1.H2I - Cholla Wash Tributary
D. 1B.H2I - Waterfall Wash
E. 2.H2I - White Tank #3 Wash
F. 3.H2I - Bedrock Wash
G. 3A.H2I - Bedrock Wash Tributary
H. 10.H2I - Bullard Wash
I. 13.H2I - Lower El Mirage Wash
J. 13A.H2I - Lower El Mirage Wash Tributary

Included in this submittal are the following items:

1. Floppy disk with HEC-2 input data.
2. HEC-2 hardcopy output.
3. 400 scale original prints with floodplains and floodways.
4. 1"=4000' topographic map with wash names.

The WLB Group Inc.

We will require the return of the 400 scale originals after your review is completed. Please call if you have any questions.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

JSE:tlg

B:letters\289036\9-9.1

JF 289036

102

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SEP 1991
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SEPTEMBER 26 1991

Mark T. Gavan, P.E.
WLB Group
333 East Osborn, Suite 380
Phoenix, Arizona 85012

Subject: White Tanks Agua Fria ADMS
Floodplain Delineation Review Comments

Dear Mark:

Following are review comments on the floodplain delineations:

For all flood delineations:

1. Use comment cards verses note records for documentation of extended cross sections, divided flows (in special cases), and other site specific characteristics typical of the subject wash. This will provide added documentation to the output when obtaining information for the subject wash.

2. We suggest all bridge and culverts be verified by hand calculations or other computer models.

3. Extended cross sections will not be allowed unless supported by documentation.

4. Cross sections which identify divided flows should be adjusted when ever possible to avoid islands within the floodplain and floodway.

5. Verify all floodway widths, with mapped floodway and HEC2 Output.

6. For washes with a braided channel bottom it is our preference to set channel bank stations such that the defined "Floodway" would be at the outer boundary of the channel and would include all the braided stream paths within the channel.

7. The flow regime for washes 1, 1A, 1A1, 1B, 2, 3, and 3A are flowing at critical depth. It is our preference that all floodway analyses for these washes be based on a target difference in the energy grade line between the natural and encroached conditions, with the final floodway analysis being encroachment method 1.

Wash 1 - Beardsley Canal

1. Adjust cross sections 0.231, 2.741 and 3.391 to avoid an island within the floodplain.

2. Provide comment card for cross section 1.616 identifying confluence White Tanks #3.

3. Adjust cross section 1.844 to resolve the message "extended cross sections".

4. Provide a comment card for cross sections 2.392, 2.300, 2.267 identifying confluence with Wash 1B.

5. Continue floodway between cross sections 1.159 to 1.313.

6. Tie in the floodway of Wash 1 with Wash 1B.

Wash 1A - Cholla Wash

1. Mapped floodway widths do not match floodway widths as determined in the HEC-2 run. Please note that not all cross sections were checked but will need to be verified with the output. In addition, reassess the floodway analysis based on the target difference in the energy grade line elevation between the natural and encroached conditions.

X-Section		Mapped	HEC-2 Output
0.188 375	228		
0.278 270	254		
0.680		200	170
1.697 335	315		
2.426 130	99		

Wash 1A1 - Cholla Wash Tributary

1. The final floodway must be determined using encroachment method 1.

Wash 1B - Waterfall Wash

1. Provide a comment card regarding the message "extended cross section" for cross sections 0.00 and 0.055 which are located at the confluence with Wash 1.

2. The flow regime of Wash 1B from cross section 1.479 on, is flowing at critical depth in a well defined channel, recommend that floodway equal floodplain limits.

Wash 2 - White Tanks #3

1. Mapped floodway width does not match floodway width as determined in the HEC-2 Output.

X-Section	Mapped	HEC-2 Output
0.703 110	90	

Wash 3 - Bedrock Wash

See General Comments

Wash 3A - Bedrock Wash Tributary

1. Submitted HEC-2 input from the disk did not match hardcopy HEC-2 output.

2. Identify the island in cross sections 0.147 and 1.640 with the statement, "Community designated flood hazard area, Zone B".

3. Identify ineffective flow areas for cross sections 1.063, 1.351, and 1.451.

Wash 10 - Bullard Wash

1. Mapped floodway widths do not match floodway widths in the HEC-2 output.

X-Section	Mapped	HEC-2 Output	
4.858 320	300		
8.364 360	240		
12.775		250	280
12.742		510	494

2. Cross sections which are extended more than one foot must have full documentation as to flood hazard characteristics at this/these cross sections, such as break out, split flow and/or diversions. If the natural grade is such that the cross section does not have a reasonable interception point, set artificial barriers such that the flood hazard can be mapped to one (1) foot of flow, documentation must also accompany this approach.

Wash 13 - El Mirage Wash

1. Mapped floodway widths do not match floodway widths in HEC-2 output.

X-Section Mapped	HEC-2 Output	
0.386	375	75
1.174	170	220
1.259	100	80
1.725	250	235
2.488	70	165

2. Provide comment cards for extended cross sections documenting that the flood delineation was terminated at a flow depth of one foot.

3. The Manning "n" value of 0.03 through the golf course is underestimated for the right and left overbanks. Based on the depth of flow of less than one (1) foot to 1.5 foot and velocities of 1.5 to 3 fps and the varying topography, recommend a manning "n" value of 0.035 to 0.045 be used to describe the right and leftover banks through this reach of El Mirage Wash.

Wash 13A El Mirage Wash Tributary

1. Revise extended cross sections 2.399 and 2.302 which map the right overbank or document the limited grade in the right overbank to contain the flow.

Field "n" value photo documentation

The field photo documentation will need to be compiled and bound with the final submittal with the appropriate narrative for each photo. An example of the preferred format for this document will be provided to you for your review. Please continue to provide photo documentation of the "n" values for each submittal for our review. In addition, please include the stereo photographs for the submitted areas. In turn the photos will be returned to you for final compilation of the document.

Enclosed is the revised ADWR format for documentation for floodplain delineations which should be used as a guide for the format of this project.

Please call me with any questions.

Sincerely,

Greg Rodzenko

Gregory Rodzenko
Project Manager

The WLB Group inc

Engineering · Planning
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Landscape Architecture

October 7, 1991

Mr. Greg Rodzenko
Flood Control District of
Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
WLB No. 289036

Dear Greg:

This is the final submittal of the preliminary floodplain and floodway delineations for the White Tanks/Agua Fria ADMS. The following items are included in this submittal:

1. HEC-2 detailed floodplain and floodway delineations (hardcopy and floppy disk) for the following washes:

   A. Wash 4 - Jackrabbit Trail Wash (Including Split Flow Analysis Run)
   B. Wash 6 - 191st Avenue Wash (Including Split Flow Analysis Run)
   C. Wash 7 - Perryville Road Wash (Including Split Flow Analysis Run)
   D. Wash 12 - Grand Avenue, Agua Fria River to Bell Road
   E. Wash 14-2 - I-10, Tuthill Dike to Jackrabbit Trail
   F. Wash 14-3 - I-10, Jackrabbit Trail to Perryville Road
   G. Wash 12 - Litchfield Park Dam Wash

2. HEC-2 approximate delineations (hardcopy and floppy disks):

   A. Wash 8 - Cotton Lane, Indian School Road to Olive Avenue
   B. Wash 9 - Cotton Lane, Olive Avenue to Waddell Road
   C. Wash 14-6 - I-10, R.I.D. Canal Crossing West to Cotton Lane
   D. Wash 18 - Atchison Topeka & Santa Fe Railroad - Northern Avenue to 1/2 mile north of Olive Avenue.
   E. Wash 19 - Atchison Topeka & Santa Fe Railroad - 1/2 mile west to 1/2 mile east of Litchfield Road.
   F. Wash 20 - Atchison Topeka & Santa Fe Railroad - 1/2 mile west of Litchfield Road to 3/4 mile north of Cactus Road.

Offices located in Tucson, Phoenix, Las Vegas, and Rancho Cucamonga, California
333 East Osborn Suite 380 · Phoenix, Arizona 85012 · (602) 279-1016

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October 7, 1991
Mr. Greg Rodzenko
Page Two

3. Documentation for approximate delineations by normal depth methods and delineations of ponding areas. These include:

   A. Ponding behind Airline Canal
   B. Ponding and approximate floodplain delineations behind I-10
   C. Ponding behind R.I.D. Canal
   D. Ponding and approximate floodplain delineations behind Southern Pacific Railroad where appropriate.
   E. Ponding behind Buckeye Canal
   F. Ponding along west side of Agua Fria River dike

   Ponding water surface elevations were taken from the HEC-1 hydrology model.

4. New update of the HEC-1 hydrology model with minor revisions (hardcopy and floppy disk.) Please discard the old run and replace it with the up-dated run of October 5, 1991. A list of these revisions follows based on more detailed hydraulic analyses:

   A. Divert at CP261 now goes to CP277. Remainder flows to CP275.
   B. Divert 3D294A is added and storage-discharge table revised at CP279.
   C. Concentration points at CP330 and CP331 were combined.
   D. Divert DI212 was added at CP194 and DI211 taken out.

5. Print of updated Drainage Area Map with above revisions (scale: 1"=4000').

6. 8 1/2" x 11" xerox of wash locations and names.

7. Notes on floodplain, floodway and ponding area delineations. (Included along with this transmittal letter.)

The WLB Group Inc

October 7, 1991
Mr. Greg Rodzenko
Page Three

We are continuing to incorporate your comments from the first two submittals and should submit the final floodplain delineations on or near October 28, 1991.

Please call with any questions you may have or for any additional information you may require.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson

Jeff S. Erickson, P.E.
Project Engineer

JSE:srm

Attachments

B:LETTERS\289036\10-7.L

The WLB Group Inc

October 8, 1991
289036

Notes on Floodplain, Floodway and Ponding Area Delineations for White Tanks/Agua Fria ADMS.

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1. Ponding areas along the Agua Fria River were previously defined with the Agua Fria FIS. The ponding W.S.E.L.'s from the White Tanks/Agua Fria ADMS HEC-1 run tend to be higher than what was previously delineated. We have shown these ponding areas on the floodplain maps for your review.

2. Ponding areas along I-10, R.I.D. Canal, Southern Pacific Railroad, and Buckeye Canal were delineated using HEC-1 Stage-Storage-Discharge results.

3. Approximate floodplain delineations were computed along the AT&SF railroad on Cotton Lane and on the AT&SF railroad spur from Luke AFB north to approximately Waddell Road, using the HEC-2 model. The discharge calculated with HEC-1 was reduced in several places along the railroad to keep the W.S.E.L. no more than 1.0′ to 1.5′ over the top of the railroad.

4. Split flow runs were executed on Wash 4, Wash 6, and Wash 7 to determine the discharges which are contained in each wash. These discharges were then input back into the HEC-2 model, without the split flow option, to compute water surface elevations. Extended cross-sections in the HEC-2 output indicate areas where flow will divert out of the wash.

5. Approximate floodplain delineations along I-10 and the Southern Pacific railroad using normal depth calculations for certain reaches where discharges were deemed significant enough to justify a floodplain. See the documentation sheets for a description of these areas.

6. Ponding water surface elevations in the detention basins along I-10 west of the Agua Fria River are computed in the HEC-1 model, however, one controlling water surface elevation is used from the downstream basin. This is also the highest water surface elevation of all four basins, therefore, the assumption was made to make this the controlling W.S.E.L. in each basin since HEC-1 does not have the capability to balance water surface elevations in a series of detention basins. Using the highest elevation is a conservative assumption because it ignores the extra storage capacity (i.e., the capacity below the highest water surface elevation) in the other three basins.

The WLB Group

October 8, 1991
289036
Page 2 of 2

We should point out that the HEC-1 model stopped short (at 300 ordinates) of capturing the peak stage. We are in the process of obtaining the new 2000 ordinate program which will calculate the peak stage. However, we are confident that the water surface elevation will not raise much higher and will be contained in the basins.

7. An approximate floodplain was not delineated downstream of the breakout at Northern Avenue and Beardsley Canal as is defined in the scope of work because there is very little capacity along the north side of Northern Avenue and, downstream of Northern, flow continues as sheet flow to the southeast.

8. Ponding along the Airline Canal is defined as an approximate ponding area with a water surface elevation corresponding to the nearest whole foot elevation above the top of the canal. Please review this carefully through Litchfield Park and along the subdivision north of Camelback Road.

9. Floodplain and floodway delineations for Wash 12 along Grand Avenue were matched into existing 100-year elevations established in the Agua Fria FIS.

The WLB Group Inc

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Offices located in Tucson, Phoenix, Las Vegas, and Rancho Cucamonga, California
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Transmittal

To: Jan Opstein

Flood-Control District

3335 W. Durango
Phoenix, Arizona 85009

Date: 10/15/91

Job No.: 289036

Drawing/Spec Reference:

Re: White Tanks/Agua Fria Mannings "n" picture documentation

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

Material Format		Requested Action	
☐ Letter	☐ Shop Drawing	☐ For Your Approval	☐ Your Review
☐ Memo	☐ Clarification Drawing	☐ For Your Signature	☐ Please Comment
☐ Prints	☐ Modification Drawings	☐ Information	☐ Make Recommendation
☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☐ Sample	☐ C.O./C.B.	☒ As Requested	☐
☐	☒ Pictures	☐	☐

Remarks: Jan - Please find enclosed mannings "n" picture documentation in ordered format as requested.

Copies To: file

By: Jeff S. Erickson

JF 289036

FLOOD CONTROL DISTRICT
of
Maricopa County

2801 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 506-1501
Fax (602) 506-4601

D. E. Sagramoso, P.E., Chief Engineer and General Manager

BOARD OF DIRECTORS
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NOV 1991 RECEIVED WLB-PHX

NOVEMBER 15 1991

Mark T. Gavan, P.E.
The WLB Group
333 E. Osborn, Suite 380
Phoenix, Arizona 85012

Subject: Floodplain Delineation
10/7/91 submittal review comments

Dear Mark:

The referenced submittal has been reviewed. The following comments should be incorporated into the FIS report:

General Comments:

1. Address and document in the HEC-2 input deck with comment cards, and in the final report, changes in peak discharges due to break-out conditions, and what occurs to the flow after it leaves the system.

2. All extended cross sections must have documentation which supports the extended cross section message in the HEC-2 output.

3. Submit all assumptions and analysis on how peak discharges were determined in the "capacity flow analysis," [sic] used to determine the approximate flood hazard limits.

Jackrabbit Trail Wash (Wash 4)

1. Output data was cut off for cross sections 2.765 and 3.154.

2. Mapped floodplain topwidth for cross section 3.813 does not match HEC-2 topwidth; mapped topwidth equals 130 feet; HEC-2 topwidth equal 69.4 feet.

3. Mapped floodway topwidth for cross section 1.443 does not match HEC-2 topwidth; mapped topwidth equals 145 feet; HEC-2 topwidth equals 79.1 feet.

191st Avenue Wash (Wash 6)

1. Suggest cross section 0.266 and 0.426 be rearranged to eliminate the divided flow.

2. Verify effective flow area along the left overbank for cross sections 0.950 and 1.734

3. Address areas where peak discharges are modified due to "break out" conditions, both in the HEC-2 input deck and in the final report.

Perryville Road Wash (Wash 7)

1. Mapped floodplain topwidths do not match topwidths in the HEC-2 output for the following cross sections:

X-Section	Mapped	Output
0.523	345 ft.	324 ft.
0.606	355 ft.	263 ft.
1.545	200 ft.	169 ft.
2.018	160 ft.	201 ft.
2.066	275 ft.	307 ft.

2. Mapped floodway topwidths do not match topwidths in the HEC-2 output for the following cross sections:

X-Section	Mapped	Output
0.523	200 ft.	169 ft.
0.606	170 ft.	137 ft.
1.429	120 ft.	88 ft.
1.545	150 ft.	130.5 ft.

3. Verify effective flow area for cross section 3.461 along the left overbank.

4. Verify effective flow area for cross section 2.874 along the right overbank.

5. Address and document variations in the peak discharge where "break out" conditions occur, in the HEC-2 input deck and in the final text.

Grand Avenue, Agua Fria River to Bell Road (Wash 12)

1. Mapped floodplain topwidths do not match topwidths in the HEC-2 output for the following cross sections:

X-Section	Mapped	Output
1.085	950 ft.	678.93 ft.
1.189	1130 ft.	692 ft.
1.294	1130 ft.	810.6 ft.
1.394	1140 ft.	697 ft.
1.489	525 ft.	467.8 ft.
1.716	440 ft.	305.5 ft.
1.814	400 ft.	350 ft.
2.600	190 ft.	97.6 ft.

2. Mapped floodway topwidths do not match topwidths in the HEC-2 output for the following cross sections:

X-Section	Mapped	Output
0.992	335 ft.	319.7 ft.
1.189	720 ft.	681 ft.
1.394	320 ft.	292.6 ft.

I-10, Tuthill Dike to Jackrabbit Trail (Wash 14-2)

1. Mapped floodway topwidth for cross section 0.044 does not equal HEC-2 output. The mapped distance equals 140 ft., the HEC-2 output equals 104 ft.

Litchfield Park Dam Wash (Wash 12)

1. Document the capacity of the detention basin, in the final report. 50-year? 100-year? 500-year?

Cotton Lane Wash, Indian School Road to Olive Avenue (Wash 8)

1. See general comments 1 and 3.

Atchison Topeka and Santa Fe Railroad - Northern Avenue 1/2 mile north of Olive Avenue (Wash 18)

1. Verify that the 100-year peak discharge is conveyed through the culvert. Variation in the topwidth and channel area through the culvert indicate that flows should overtop the culvert, while the culvert output indicates that the flow is conveyed through the culverts.

2. See general comments 1 and 3.

North side of I-10 between Citrus Road and Perryville Road

1. Adjust discharge analysis at concentration point CP 275 for 200 feet east of Perryville Road, from CP275/4 to CP275/8.

Photo documentation

1. Photo for "typical desert area" is too dark to assess the roughness coefficient.

2. The photo documentation for Mannings "N" Values should be in a section distinct from the other general photographs. The general stream photographs should be placed in a specific section for that wash.

Please call me at 506-1501 with any questions.

Sincerely,

Greg Rodzenko
Project Manager

The WLB Group Inc.

November 18, 1991
Mr. Greg Rodzenko
Page Two

Included in this submittal are the following:

1. HEC-2 hardcopy output.
2. Floppy disk with HEC-2 input data.
3. 1" = 400' scale original worksheets with delineations.
4. Response to previous comments.
5. Updated HEC-2 model dated October 28, 1991 (Please discard old model.)

A meeting should be set up to go over the response to comments after you have reviewed them. If we can be of further assistance, please call.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

Attachments

B:LETTERS\289036\11-11.L

The WLB Group Inc

Response to Comments Dated August 6, 1991 from FCDMC
First Preliminary HEC-2 Submittal

1. WLB did not submit Wash 5M and therefore should be disregarded.

2. A 10.6 encroachment method was incorporated on these and all subsequent floodway analyses. A 9.1 encroachment field was then utilized, after the initial run, to smooth the floodway in some cases or to maximize the natural conveyance area while minimizing velocity increases.

3. Mannings "n" Values were set using aerial photos, topographic mapping, field reconnaissance and sound engineering judgement throughout washes on the watershed. A few adjustments may be made where encroachments were made inside the stated channel limits.

4. Ineffective flow areas were looked at closely on each HEC-2 model. If areas are ineffective, the cross-section will be truncated at that point or a X3 card will be used to define effective flow limits. Again, judgement is involved to ascertain where upstream cross-sections will provide inflow into an otherwise ineffective flow situation.

5. This is usually the case if the wash is confined, however, there are areas within WLB's HEC-2 analysis where breakouts will occur. A split flow analysis will then be performed and resulting discharges used to calculate actual water surface elevations.

   These breakouts will be documented in subsequent HEC-2 submittals. Another scenario is when we have purposely kept the cross-section extended, for instance along the railroad at Cotton Lane, to illustrate that the railroad will be overtopped at any given area dependent upon the inflow of major flows. These areas are hard to pinpoint, thus the assumption of trying to keep an equal elevation over the top of the railroad at any given cross-section. Of course, if any of these scenarios are not the case, the initial input will be carefully scrutinized to ascertain if there are any input errors.

6. The following documentation relates to comments made by FCDMC on ineffective flow areas:

   A. Wash 5 - Cross Sections 1.152, 1.198 and 1.260: The cross section is limited to effective flow areas because of ponding in this area. Effective flow limits are placed to estimate effective flow due to expansion at the upstream end and contraction at the downstream end. Actual limits of the floodplain will be based on ponding or floodplain water surface elevations.

The WLB Group Inc

B. Wash 5 - Cross Section 1.518: Flow is effective in the right overbank as a result of upstream inflow.

C. Wash 5 - Cross Section 2.170: This cross section was modified to reflect correct encroachment condition. First submittal was incorrect.

D. Wash 5B - Cross Section 1.188: Identified ponding area in right overbank as floodplain yet ineffective flow within this particular ponding area. Documented this in HEC-2 run.

E. Wash 5C - Cross Section 0.305: Ponding area to the right of Station 10100 is ineffective due to expansion limitations. This is now documented in HEC-2 run.

7. Cross sections in which channel stations are outside the encroachment area were re-evaluated and either bank stations have been relocated or the encroachment will span the width of the cross section to maintain maximum conveyance characteristics.

8. Reorientation of cross sections is addressed as follows:

A. Wash 5A - Cross Section 0.456: Bank stations were changed and ineffective flow area in left bank eliminated. Cross section did not need reorientation.

B. Wash 5A - Cross Section 0.602: Cross section was reoriented to show actual limits of floodplain. Flow is effective in left overbank due to upstream inflow.

C. Wash 5A - Cross Section 0.705: Cross section was not reorientated. Flow is effective in left overbank due to upstream inflow. Documented in HEC-2 run and see mapped floodplain.

D. Wash 5A - Cross Section 0.801: Cross section was not reoriented. Flow is effective in left overbank due to upstream inflow. Documented in HEC-2 run and see mapped floodplain.

E. Wash 5B - Cross Section 0.240 and 0.384: These cross sections were not reoriented due to the fact of wide shallow flooding and upstream flows do contribute to make effective flow areas in both the right and left overbank. Floodway is modified throughout Wash 5B from comments received on first preliminary submittal.

The WLB Group Inc.

F. Wash 5B - Cross Sections 1.553, 1.604, 1.676, 1.764, 1.843 and 1.916: These cross sections were also not reoriented due to the controlling decision of upstream inflow. Look closely at the floodplain mapping to verify this decision. Again this is wide shallow flooding and we think it is best to be conservative in this situation. The floodway was modified due to comments made on the first preliminary submittal.

G. Wash 5E - Cross Sections 0.324, 0.381, 0.408, 0.477 and 0.585: We looked at these cross sections and determined that upstream inflow contributes to overbank flow areas downstream. We have, however, modified cross section end stations to exclude left ineffective flow areas in cross sections 0.408 and 0.477. Cross section 0.381 was redefined to show low flow area in the roadway.

H. Wash 5E - Cross Section 0.969: This cross section was reoriented and floodplain redefined.

9. We have submitted documented pictures of manning "n" values for your review and will submit the aerial photography for further documentation.

The WLB Group Inc

Response to Comments Dated August 13, 1991
From Flood Control District of Maricopa County

Dysart Drain Submittal

1. This run was started at critical depth with an approximate WSEL in Field 9 on the TI card. Once the floodplain was ascertained, it can be matched into the existing 100-year Agua Fria Floodplain as directed by FEMA. We should not use the starting 100-year WSEL in the Agua Fria River as our starting WSEL.

2. Zone A will be delineated for a distance south of the two breakouts on Dysart Drain. Approximate methods will be incorporated to delineate this area. This data is included for your review. Documentation was already included in the HEC-2 run.

3. I believe that we have already shown this on the maps. Further documentation will be provided in the HEC-2 analysis.

4. Documentation will be provided to describe what is happening in these particular areas.

The WLB Group Inc

Response to Comments
Dated September 26, 1991

1. Further documentation will be provided where needed within the HEC-2 runs to document special situations within each delineation.

2. Bridges and culverts have already been verified within the HEC-1 model. Backup documentation was submitted with the HEC-1 submittal.

3. Extended cross sections in these HEC-2 models usually indicate an area where flows are not contained. Further documentation within the HEC-2 model will better explain these situations.

4. Cross sections with divided flows were looked at carefully previous to submitting them for review. We have gone back and looked at these specific areas and either changed the limits of effective flow or documented why the divided flow is effective due to upstream inflow. In some areas, it is just not possible to keep the islands out of the floodplain or floodway because of wide shallow flooding and attempts to confine the floodplain width result in extended cross sections indicating that, indeed the full cross section will become effective.

5. Floodplain and floodway widths will be reverified and changed if there is a discrepancy between mapped widths and HEC-2 output. It should be noted that there may be a percentage of error due to drafting and width of floodplain lines. We will try to be as precise as possible given the constraints posed by 1" = 400′ mapping.

6. Channel bank stations will be changed to reflect floodways that incorporate braided stream paths within the channel or encroachments will remain outside the bank stations to include braided stream paths.

7. A 10.6 method will initially be performed for floodways in this study. A final floodway analysis will be performed utilizing method 1 to set final floodway widths.

Wash 1 - Beardsley Canal

1. Cross Section 0.231 - This cross section shows divided flow as does cross section 0.307. This is in fact the case due to upstream inflow at cross section 0.390 and makes flows in the right overbank effective.

   Cross Section 2.741 - The floodplain was adjusted to show the island and the consequent breakout downstream to make the right overbank effective. If flows were confined to the high point on the bank, they would overtop in the restricted condition.

The WLB Group inc

Cross Section 3.391 - We do not have a cross section labeled 3.391 in Wash "1". Cross section 3.301 does not have divided flow. We are not sure which cross section is being referred to here.

2. Cross Section 1.616 - HEC-2 documentation of confluence will be provided.

3. Cross Section 1.844 - Cross section was modified and flow is now confined.

4. Comment card is provided at cross section 2.217 to identify confluence with Wash "1B" - Waterfall Wash.

5. Floodway and floodplain are coincident from cross section 1.159 to 1.313. This is due to ponding upstream of Northern Avenue and flows breakout to the east over Beardsley Canal. As per previous telephone conversations with FCDMC, we will leave this as is.

6. Same response as above.

Wash 1A - Cholla Wash

1. Cross Section 0.188 - Remapped floodway as shown on HEC-2 run.

   Cross Section 0.278 - Remapped floodway as shown on HEC-2 run.

   Cross Section 0.680 - Remapped floodway as shown on HEC-2 run.

   Cross Section 1.697 - Due to small errors of each side of floodway. This is difficult considering the scale and some consideration should be given for a small percentage of error in mapped floodplain widths. We will try as best we can to put on accurate widths.

   Cross Section 2.426 - Remapped floodway as shown on HEC-2 run.

   All cross section widths were checked again and now match HEC-2 output.

Wash 1A1 - Cholla Wash Tributary

1. This is not necessary since the floodway and floodplain are coincident even with a Method 6 run, however, we will provide the Method 1 run as requested.

The WLB Group Inc

Wash 1B - Waterfall Wash

1. Comment cards were provided to describe the situation of flows breaking out to the east over Beardsley Canal.

2. Floodway and floodplain limits were set equal upstream of cross section 1.479 as requested.

Wash 2 - White Tank #3 Wash

1. HEC-2 output now matches mapped floodway width.

Wash 3 - Bedrock Wash

Have incorporated more documentation and rechecked floodplain and floodway widths to match HEC-2 output.

Wash 3A - North Fork Bedrock Wash

1. HEC-2 hardcopy output and input from the floppy disk are now the same run.

2. The island in cross section 0.147 is very small and FEMA's policy is to not show islands anyway. Cross section 1.640 was modified in a subsequent run to limit effective flow and the floppy disk copy of the previously submitted run did not reflect this. New run should correct this problem.

3. Old floppy disk HEC-2 run was incorrect. New version shows the correct effective flow limits.

Wash 10 - Bullard Wash

1. Discharges have been changed due to some modifications in the HEC-1 model, floodplain and floodway stations have been checked with the new HEC-2 output.

2. All cross sections with extended WSEL have been documented in the HEC-2 input file.

The WLB Group inc.

Other Notes:

- Cross-sections numbers have been changed upstream of cross-section 9.034.
- Discharges have been reduced at some reaches to limit the WSEL extension to 1 foot. This has been documented in the HEC-2 input file.
- Diverted flows, where ever they occur, have been documented in the HEC-2 input file.
- Culverts at Indian School Road and Camelback Road have been modeled and documented in the HEC-2 run.

Wash 13 - El Mirage Wash

1. Discharges have been changed due to some modifications in the HEC-1 model. Floodplain and floodway stations have been checked with the new HEC-2 output.
2. A general review has been done. Subsequently, there are no more cross sections with extended WSEL, especially upstream of Section 2.113 where cross sections have been modified.
3. The Manning "n" Values have been changed into "0.035" for the overbanks in the golf course reach.

Other Notes:

- Divided flows have been documented in the HEC-2 input.
- The floodplain and floodway stations have been matched to those from Agua Fria River Floodplain maps.

Wash 13 A - El Mirage Wash Tributary

1. WSEL extension at Section 2.302 and 2.399 has been documented in the HEC-2 input file. Also documented is the divided flow at Section 2.302.

Other Notes:

- Discharges have been changed slightly due to some modifications in the HEC-1 model.

The WLB Group Inc

- A general review has been done to eliminate other WSEL extensions and divided flows.

- The floodplain and floodway stations have been checked with the new HEC-2 output.

Field "n" value photo documentation has been revised per previous comments and was submitted with last submittal. Also, the stereo photographs have been submitted recently for your review.

The WLB Group inc.

Engineering · Planning
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December 4, 1991

Mr. Greg Rodzenko
Flood Control District of
Maricopa County
2801 West Durango
Phoenix, Arizona 85009

Re: Response to Comments From FCD
Dated 11/15/91 - HEC-2
Floodplain and Floodway Delineations

Dear Greg:

The following letter addresses review comments made on a preliminary HEC-2 submittal dated 10/7/91. Review comments were dated 11/15/91. We have incorporated final revisions deemed necessary as per Flood Control District comments and are now resubmitting the following delineations for your final approval:

1. HEC-2 DETAILED FLOODPLAIN AND FLOODWAY DELINEATIONS
   A. Wash 4 - Jackrabbit Trail Wash
   B. Wash 4.SPF - Jackrabbit Trail Wash - Split Flow Analysis
   C. Wash 6 - 191st Avenue Wash
   D. Wash 6.SPF - 191st Avenue Wash - Split Flow Analysis
   E. Wash 7 - Perryville Wash
   F. Wash 7.SPF - Perryville Road Wash - Split Flow Analysis
   G. Wash 12 - Grand Avenue - Agua Fria River to Bell Road
   H. Wash 14-2 - I-10, Jackrabbit Trail to Tuthill Dike
   I. Wash 14-3 - I-10, Perryville Road to Jackrabbit Trail
   J. Wash 21 - Litchfield Wash

2. HEC-2 APPROXIMATE DELINEATIONS

   A. Wash 8 - Cotton Lane - Indian School Road to Olive Avenue
   B. Wash 9 - Cotton Lane - Olive Avenue to Waddell Road
   C. Wash 14-6 - I-10, RID Canal Crossing West to Cotton Lane
   D. Wash 18 - Atchison, Topeka & Santa Fe Railroad - Northern Avenue to 12 miles north of Olive Avenue
   E. Wash 19 - Atchison, Topeka & Santa Fe Railroad - 1/2 mile west to 1/2 mile east of Litchfield Road.
   F. Wash 20 - Atchison, Topeka & Santa Fe Railroad - 1/2 mile east of Litchfield Road to 3/4 mile north of Cactus Road.

The WLB Group Inc

Mr. Greg Rodzenko
December 4, 1991
Page Two

3. APPROXIMATE DELINEATIONS BY NORMAL DEPTH METHODS AND DELINEATION OF PONDING AREAS

   A. Ponding behind Airline Canal
   B. Ponding and approximate floodplain delineations behind I-10
   C. Ponding behind R.I.D. Canal
   D. Ponding and approximate delineations behind the Southern Pacific Railroad where appropriate
   E. Ponding along west side of Agua Fria River dike.

   Note: Ponding water surface elevations were taken from the HEC-1 hydrology model.

Included in this submittal are the following:

1. HEC-2 hardcopy output for appropriate runs.
2. Floppy disk with HEC-2 input data.
3. Documentation of approximate delineations where appropriate.
4. Response to comments dated November 15, 1991.
5. New picture documentation of Mannings "n" values.

We will provide the original prints of floodplain and floodway delineations when you are ready to review them as we are still digitizing cross sections into the computer. Also, I would like to exchange copies for the original documentation that we submitted previously. If you have any further questions, please call.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson,
Project Engineer

JSE:tlg

B:letters\289036\11-26.1

The WLB Group Inc.

RESPONSE TO COMMENTS DATED
NOVEMBER 15, 1991 FROM FCDMC

PRELIMINARY HEC-2 SUBMITTAL
DATED OCTOBER 7, 1991

General Comments

1. Documentation will be provided in the HEC-2 input deck to document discharges due to breakout conditions and what happens to those flows after they leave the system.

2. Extended cross sections resulting from breakout situations will be documented, otherwise we will rectify the error leading to an extended cross section message.

3. We have submitted split flow analyses and documentation was provided in the "Notes on Floodplain, Floodway, and Ponding Area Delineations for White Tanks/Agua Fria ADMS" dated October 7, 1991 stating our assumptions when utilized. We will document these assumptions in the HEC-2 input deck and later in a wash by wash description of assumptions, hydraulic modeling techniques and special situations in the final report.

JACKRABBIT TRAIL WASH (WASH 4)

1. A new printout of the HEC-2 run will be provided with a complete printout of the output.

2. You must look at the Start Stations and End Stations in a divided flow situation. The TOPWID in the HEC-2 output only maps actual WSEL distance. No changes were made.

3. Mapped Floodway now matches HEC-2 output.

191ST AVENUE WASH (WASH 6)

1. Divided flows at cross sections 0.266 and 0.426 are two of many cases due to the braided channel type. Rearrangement will not prevent divided flow.

2. Left overbank at section 0.950 and 1.734 have been assumed effective due to the braided channel type and upstream inflows. Excluding them may overestimate breakout flows in the split flow analysis and thus underestimate floodplain limits.

The WLB Group Inc.

3. Discharges as derived from the 24-hour storm HEC-1 model were used for a split flow analysis (see documentation in the split flow analysis run). Remaining discharges were then used for floodplain and floodway analysis (see documentation in floodplain/floodway run). Breakout flow conditions are now documented in the input file.

   Other notes:

   * Starting WSEL is taken from the HEC-1 Run.

   * Note records have been replaced by comment cards.

PERRYVILLE ROAD WASH (WASH 7)

1. X-section 0.523 and 0.606 - These floodplains are mapped correctly based on Start Station and End Station distances. The TOPWID in the HEC-2 output only maps the actual WSEL distance and neglects any distance above the WSEL when divided flow is occuring. No changes were made.

   X-section 1.545 - Mapped limits now match HEC-2 output.

   X-section 2.018 - Mapped limits now match HEC-2 output.

   X-section 2.066 - Mapped limits now match HEC-2 output.

2. X-section 0.523 and 0.606 - See comments above.

   X-section 1.429 - Mapped limits now match HEC-2 output.

   X-section 1.545 - Mapped limits now match HEC-2 output.

3. Cross section 3.461 was modified and now correctly models the area in question. Left overbank flow is effective due to upstream inflow.

4. Cross section 2.874 has been modified to limit effective flow in the right overbank, as suggested.

5. Breakout flows are now documented in the HEC-2 run.

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GRAND AVENUE, AGUA FRIA RIVER TO BELL ROAD (WASH 12)

1. X-section 1.085 through X-section 2.600
   These floodplains were mapped correctly based on Start Station and End Station distances. The TOPWID in the HEC-2 output only maps the actual water surface elevation distance and neglects any distance above the WSEL when divided flow is occuring. No changes were made.

2. X-section 0.992, 1.189 and 1.394
   See comment 1 above.

I-10, JACKRABBIT TRAIL TO TUTHILL DIKE (WASH 14-2)

1. Believe you are talking about floodplain width here instead of floodway width. Due to divided flow, the HEC-2 output only prints actual WSEL width and not Start Station and End Station width. No changes were made.

LITCHFIELD WASH (WASH 21)

1. A comment card has been added to document matching the floodplain to the WSEL due to storage capacity in the Litchfield Park Detention Basin.

   Other notes:

   * Note records have been replaced by comment cards.

COTTON LANE WASH, INDIAN SCHOOL ROAD TO OLIVE AVENUE (WASH #8)

1. T-cards and C-cards have been incorporated to document peak flows and breakout flow conditions.

   Other notes:

   * Note records have been replaced by comment cards.

   * Cross sections 0.059, 1.738, 2.055 and 2.135 have been modified to avoid divided flow conditions.

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ATCHISON, TOPEKA & SANTE FE RAILROAD, NORTHERN AVENUE TO 1/2 MILE NORTH OF OLIVE AVENUE (WASH 18)

1. The 100-year peak discharge conveyance through the culvert, cross sections 0.928 and 0.943 (now cross sections 0.928 and 0.948), has been checked. The effective flow area option has been incorporated at cross section 0.948 upstream of the culvert. An approximate 5.3 foot drop exists immediately upstream of the culvert entrance. The culvert will pass this flow without overtopping the road.

   Other notes:

   * Note records have been replaced by comment cards.

   * Peak discharges at the reach upstream of cross section 0.685 has been changed from 651 cfs to 634 cfs.

   * Extended WSELs have been documented by comment cards.

   * Divided flows have been documented by comment cards.

NORTH SIDE OF I-10 BETWEEN CITRUS ROAD AND PERRYVILLE ROAD

1. Instead of changing the flow from CP275/4 to CP275/8, we moved the concentration point to 400 feet east of Perryville Road and thus kept the discharge at 34 cfs. It would be difficult to delineate an approximate floodplain based on 17 cfs.

PHOTO DOCUMENTATION

1. The photo documentation will be separated into three sections as follows:
   A. General Photographs Depicting Study Area
   B. Hydrologic Picture Documentation of Mannings "n" Values
   C. Hydraulic Picture Documentation of Manning's "n" Values

FLOOD CONTROL DISTRICT
of
Maricopa County

2801 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 506-1501
Fax (602) 506-4601

D. E. Sagramoso, P.E., Chief Engineer and General Manager

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P. Ben Arredondo
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James D. Bruner
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Tom Freestone

JAN 1992 RECEIVED WLB-PHX

JAN 07 1992

Mark T. Gavan, P.E.
WLB Group
333 East Osborn, Suite 380
Phoenix, Arizona 85012

SUBJECT: White Tanks ADMS
Floodplain review

Dear Mark:

Following are comments on the submittals of 11/18/91 and 12/4/91:

1. Because of the complexity of the Dysart Drain system, the entire area should be delineated as a Zone A for the FEMA submittal. This would include all areas which break out to the south, and also those which pond up against the canal to the north. For Flood Control District (FCD) in-house use, all hydraulic runs should reference the determined water surface elevations for the area.

2. In reference to the delineations for the upper reach of Bullard Wash at Reems Road: the flood hazard area beginning at Reems Road and continuing around Luke Air Force Base, should be identified as a Zone A for the FEMA submittal. Bullard Wash will then tie-in at the training dike (approximately cross section 10.197), with a two-district floodway being delineated. For FCD in-house use, a two-district floodway analysis of Bullard Wash from Reems Road to the designated tie-in (cross section 10.197), with the appropriate back-up materials, is needed.

3. There is a ponding area east of Litchfield Road and just south of Dysart Drain, in which a ponded water-surface-elevation has been determined from the HEC-1 analysis. This area should be designated as a Zone A for the FEMA submittal. For FCD in-house use, we request all hydraulic runs to reference the water-surface-elevation generated by the HEC I analysis.

4. There is a ponding area along I-10 from Jackrabbit Road to Perryville Avenue. This entire area should be identified as a Zone A for the FEMA submittal.

5. There is a ponding area behind the Airline Canal and the RID Canal in which water surface elevations have been determined from the HEC I analysis. The area be should be designated as a Zone AO, which is defined as a "area which corresponds to the areas of 100-Year shallow flooding where average depths are between 1 and 3 feet. The depth should be averaged along the cross section and then along the direction of flow to determine the extent of the zone."

6. It had been agreed in a previous meeting that the flood hazard delineation for Cotton Lane, and the Atchison Topeka & Santa Fe Railroad, will be identified as a Zone A in the FEMA submittal. For FCD in-house use, all hydraulic analysis will be provided for the mapped flood hazard area, with water-surface-elevations and the appropriate back-up materials.

7. Per a phone discussion between Jan Opstein of FCD and Jeff Erickson of WLB Group: WLB will delineate a Zone A, based on the high berm elevation of Reems Road, from Northern Avenue, north to Grand Avenue, or to where the limits of conveyance cease.

Please call with any question.

Sincerely,

Greg

Greg Rodzenko
Project Manager

The WLB Group Inc.

Engineering · Planning
Surveying · Urban Design
Landscape Architecture

February 19, 1992

Mr. Greg Rodzenko
Flood Control District of
Maricopa County
2801 West Durango
Phoenix, Arizona 85009

Re: Response to Comments dated January 7, 1992
White Tanks ADMS - Floodplain Review
WLB No. 289036

Dear Greg:

Following are the response to comments dated January 7, 1992 for the submittals made on November 18, 1991 and December 4, 1991.

1. Dysart Drain and corresponding breakout areas to the south have been delineated as a Zone A per FCD instructions. WLB's final submittal will include the HEC-2 run for Dysart Drain with corresponding cross section locations and water surface elevations. The FEMA submittal will include only cross section locations with the HEC-2 run for documentation of the approximate floodplain delineation.

2. An approximate delineation on Bullard Wash will be shown as Zone A upstream of cross section 10.197 for the FEMA submittal. For FCD in-house use, a two-district floodway will be delineated above this cross section. We have also designated a Zone A approximate delineation downstream of cross section 2.371 due to the numerous breakouts occurring in this reach and due to the complexity of trying to model this area with HEC-2.

   A two-district floodway will also be performed in this reach for FCD in-house use. We have also shown an approximate floodplain delineation downstream of these breakouts. This includes areas west of Estrella Parkway, south of the dike located immediately south of S.R. 85, and breakouts along the south end of the Phoenix - Goodyear Airport. This has been carefully documented in the HEC-2 analysis for Bullard Wash (Wash 10).

3. The ponding area south of Dysart Drain has been designated a Zone A as requested. Documentation for the HEC-1 stage-storage discharge table is included in the backup documentation for the HEC-1 model.

The WLB Group Inc.

February 19, 1992
Mr. Greg Rodzenko
Page Two

4. The area between Jackrabbit Trail to Perryville Road has been designated a Zone A as requested. This approximate delineation is documented by a HEC-2 analysis and will be submitted for FEMA review.

5. Per telephone conversations with Michelle Monde - FEMA Technical Reviewer, Pedro Calza and Jan Opstein - FCDMC, we have decided to designate ponding areas behind the RID Canal, Buckeye Canal, Interstate 10, and Southern Pacific Railroad as Zone AH's with corresponding BFE's taken from the HEC-1 model where applicable. The Airline Canal will be designated Zone A with approximate floodplain delineations based on the nearest whole foot elevation above the top of the canal as taken from the 1" = 400' topographic mapping.

6. A Zone A approximate floodplain delineation is incorporated behind AT&SF Railroad spurs at Cotton Lane and north of Luke Air Force Base. HEC-2 runs will be submitted to FEMA for backup documentation of the approximate delineations performed on these areas. We will also submit complete HEC-2 runs with corresponding WSEL's on the maps for FCDMC in-house use.

7. An approximate delineation has been performed on Reems Road from Northern Avenue north to Beardsley Road as per instructions from the FCDMC. The area will be labeled Zone A and submitted to FEMA for acceptance. Delineations were based on the top of left bank controlling elevations and checked by normal depth equations to document that this assumption is valid. These will be included in the final submittal.

We are in the process of finalizing floodplains and floodways on the 1" = 400' mapping and we are preparing abstracts and final hydrology and hydraulic reports for the FEMA submittal. Computer drafting of the floodplains has taken longer than expected, but we are just about finished and we will be contacting you soon to set up public meetings. Please call with any questions about these comments or scheduling.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson

Jeff S. Erickson, P.E.
Project Engineer

JSE:srm

B:LETTERS\289036\2-19.L

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Offices located in Tucson, Phoenix, Las Vegas, and Rancho Cucamonga, California
333 East Osborn Suite 380 • Phoenix, Arizona 85012 • (602) 279-1016

289036

Memorandum

To: Jan Opstein, FCDMC

From: Jeff Erickson

Date: 2-26-92

Subject: Telephone Conversation about Delineating White Tanks Structures No 3 + 4.

Copies To: File.

Jan,

As per our telephone conversation on 2-26-92, regarding how to delineate the ponding areas behind White Tanks Structure #3 + #4. We will define the 100-year ponding WSEL's as a floodway and match the 100-year floodways from delineated washes into this elevation. We will also designate these areas as Zone AE with a corresponding WSEL as discussed.

Jeff J. Erickson
Project Engineer

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333 East Osborn Suite 380 • Phoenix, Arizona 85012 • (602) 279-1016

Transmittal

To: Jan Opstein

FCDMC

Date: 4-16-92

Job No.: 289036

Drawing/Spec Reference:

Re: White Tanks Floodplain Maps.

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

Material Format		Requested Action	
☐ Letter	☐ Shop Drawing	☐ For Your Approval	☐ Your Review
☐ Memo	☐ Clarification Drawing	☐ For Your Signature	☐ Please Comment
☒ Prints	☐ Modification Drawings	☐ Information	☐ Make Recommendation
☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☐ Sample	☐ C.O./C.B.	☒ As Requested	☐
☐	☐	☐	☐

Remarks: Jan,

Here are the preliminary floodplain delineation plots. Look them over and give to Dave Johnson for his perusal. We need to get these back as soon as possible to incorporate final changes into the computer. Greg Rodzenko also needs to make copies of sheets 20, 27, 34, & 41 for the SCS to look at White Tank FRS # 3 & #4 and possibly connecting them and strengthening the structures. Contact him when you recieve this. Give me a call when your done and I'll have someone pick them up

Copies To: File.

By: [illegible]

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

Subject: White Tanks - ADMS Final Maps - Editing Comments File: White Tanks

To: WLB Group Jeff Erickson From: JMOpstein, FCDMC Date: 4/21/92

1. Section Corners. Sections will be identified by the section number only, which will be darken. Also Rm's

2. Flood Zone designation behind White Tanks 3 and 4 will be designated as Zone A within the FCDMC ROW. FCDMC staff will provide the ROW limits for the structures.

3. Flows which break out of the conveyance system will be identified with an arrow and the break out discharge for all detailed reaches only.

4. A Zone D designation and a Zone X designation should be identified, where appropriate, for all panels which displays floodplain delineations. No need except for areas hard to deferentiate — Michelle M. — Baker Engineering

5. Per discussions with Hershell Morrow with the Town of Surprise, he requested that the delineated floodplain be removed from the FEMA submittal. The District would still want to maintain a file of the delineated area for our records.

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May 26, 1992

Mr. Greg Rodzenko
Flood Control District of
Maricopa County
2801 West Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
WLB No. 289036

Dear Greg:

The following changes were made to the final floodplain maps per verbal and hand-written comments received from Jan Opstein on April 21, 1992.

- Section numbers in the middle of each section and Elevation Reference Marks were darkened.
- Flood Zone designation AE behind White Tanks #3 and #4 was changed to Zone A. Also right of way limits for the structures were added to the maps.
- Flows which break out of conveyance systems were identified with an arrow and appropriate discharge for each detailed reach.
- Zone X's were shown where it is hard to differentiate between zones.
- Limit of detailed study on AT & SF Railroad Channel was stopped at Greenway Road.
- White Tank Regional Park boundaries were incorporated on the mapping.
- Bullard Wash floodway was redefined at Roosevelt Irrigation Canal.
- Zone AH designations behind Interstate 10 were changed to Zone A designations.
- All limits of study were identified on the floodplain mapping.
- Trails behind Beardsley Canal were screened.

The WLB Group Inc.

May 26, 1992
Mr. Greg Rodzenko
Page Two

- Delineations were cut off at exact limits of study as directed.
- Airline Canal delineation was more clearly distinguished from the existing Agua Fria River delineation.
- Delineation on Beardsley Canal Wash at confluence with Caterpillar Wash was revised to reflect correct discharges.
- The thalweg symbol was changed to a thinner line weight.

These changes represent the final modifications to the floodplain maps that we will send to FEMA.

Sincerely,

THE WLB GROUP, INC.

Jeff S. Erickson, P.E.
Project Engineer

JSE:srm

Attachments

B:LETTERS\289036\5-26.L

1.4.3 Other Agencies

The following correspondence took place with various other agencies.

The WLB Group Inc.

Engineering · Planning
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January 17, 1990

Mr. Jim Nelson
Vice President
Cella Barr and Associates
5062 North 19th Avenue
Phoenix, Arizona 85015

Re: White Tanks/Agua Fria ADMS
WLB No. 289036

Dear Jim:

As we discussed in our meeting of January 16, 1990, I am requesting the following items relating to the Estrella Freeway south of Grand Avenue.

- Estrella Freeway Hydrologic Investigation Report, July 14, 1987, plus addendum.
- Technical Memorandum - Estrella Freeway Drainage Alternatives; Cotton Lane Section, March 3, 1988.
- Preliminary Plans - Estrella Freeway, Cotton Lane Section.
- Interim Roadway Plans - Segment 1, Plans and Hydrology Report (Current Design Stage).
- Interim Roadway Plans - Segment 2 and 3 (when available).

We will use this information while preparing the White Tanks/Agua Fria Area Drainage Master Study (ADMS) for the Flood Control District of Maricopa County, and hope to incorporate your design concepts for the Estrella Freeway into the overall Area Drainage Master Plan for this area.

We would like to thank you for forwarding this request to ADOT on our behalf and if there are any questions regarding this ADMS, please contact Mark Gavan or myself at 279-1016.

Sincerely,

Jeff S. Erickson

Jeff S. Erickson
Project Engineer

JSE:srm

A:LETTERS/JSE-8936.L2

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ARIZONA DEPARTMENT OF TRANSPORTATION

RECEIVED JAN 1990 WLB-PHX

LETTER OF TRANSMITTAL

DATE Jan 26, 90

ATTENTION Jeff S. Erikson

RE:

CONTRACT NO.

PROJECT NO.

White Tanks/Agua Fria Area Drainage Master Study

TO The WLB Group
333 E. Osborn, Suite 380
Phoenix, AZ 85012

GENTLEMEN:

WE ARE SENDING YOU

☒ Attached ☐ Under separate cover via _____ the following items

☐ Copy of letter ☐ Prints ☐ Plans ☐ Examples ☐ Change order

☐ _____

COPIES	DATE	NO.	DESCRIPTION
1	—		Interstate 10 Drainage Design Reports list

THESE ARE TRANSMITTED as checked below:

☐ For your use ☐ Accepted as submitted ☐ Resubmit _____ copies for approval

☒ As requested ☐ Accepted as noted ☐ Submit _____ copies for distribution

☐ For review and comment ☐ Returned for corrections ☐ Return _____ corrected prints

☐ _____

REMARKS The attached list shows the Drainage Design Reports that are available for the requested study area. Additional Urban Highway Drainage Design Reports are available through Ray Jordan in the Urban Highway section here at ADOT.

COPY TO

PREPARED BY Clifton Guest

Clifton Guest

20-4404 1/75 (FORMERLY 62-9500)

If enclosures are not as noted, kindly notify us at once.

ARIZONA DEPARTMENT OF TRANSPORTATION

HIGHWAYS DIVISION
206 South Seventeenth Avenue Phoenix, Arizona 85007

ROSE MOFFORD
Governor
CHARLES L. MILLER
Director

THOMAS A. BRYANT, II
State Engineer

March 14, 1990

MAR 1990
RECEIVED
WLB-PHX

Mr. Jeff S. Erikson
Project Engineer
WLB Group
333 East Osborn Road, Suite 380
Phoenix, AZ 85012

RE: ESTRELLA FREEWAY, LOOP 303
S.R. 85 TO I-17
White Tanks/Agua Fria Area Drainage Master Plan
FCD NO. 89-50
WLB NO. 289036
ADOT NO. H087701D

Dear Mr. Erikson:

This letter is in response to your January 16, 1990 letter concerning the Maricopa County Flood Control District's Agua Fria Area Drainage Master Study (AFADMS).

Tim Wilson with the Urban Highway Section will serve as a contact for the Arizona Department of Transportation (ADOT) in the coordination of the AFADMS. Cella-Barr Associates is the general consultant under contract with ADOT for the location and design studies of the Estrella Freeway Corridor. Jim Nelson will serve as a contact for Cella-Barr Associates. Specific requests for any information should be made through Tim Wilson.

We will be able to meet with you regarding this matter at your convenience. Please contact our office at 255-7545 to schedule a date and time.

Sincerely,

ARTHUR V. MAY, P.E.
Corridor Engineer
Urban Highway Section

AVM:TGW:km

cc: Cella-Barr Associates

146

ARIZONA DEPARTMENT OF WATER RESOURCES
15 South 15th Avenue, Phoenix, Arizona 85007
Telephone (602) 542-1553
Fax (602) 256-0506

FIFE SYMINGTON
Governor

ELIZABETH ANN RIEKE
Director

April 15, 1991

John Matticks
Assistant Administrator
Office of Risk Assessment
Federal Insurance Administration
Federal Emergency Management Agency
Washington, D.C. 20472

Dear John:

Over the past couple of years the Department has been reviewing the hydrology of all flood studies completed by the Maricopa County Flood Control District (MCFCD). I now feel this is unnecessary with the completion of their new Hydrology Manual and the increased expertise of their staff. This dovetails nicely with my plans to concentrate on Arizona's rural communities who in general, lack the technical expertise to adequately review hydrologic and hydraulic analysis.

Please accept MCFCD submitted studies as if we had reviewed them, although the Department is always available if sspecial problems or question need to be coordinated.

See you in Denver.

Sincerely,

James R. Morris

James R. Morris, P.E.
Chief
Flood Management Section

FLOOD CONTROL DIST
RECEIVED
APR 16 '91
CH ENG
DEP
LR

JRM:bw

cc: Russ Cruff, MCFCD

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Transmittal

Attn: Paul LeBrun

To: Corps of Engineers

3636 N. Central Avenue
Suite 740
Phoenix, Arizona 85012

Date: 8/1/91

Job No.: 239036

Drawing/Spec Reference:

Re: Dysart Drain

We Transmit ☒ Herewith ☐ Under Separate Cover ☐ Via

Material Format		Requested Action	
☐ Letter	☐ Shop Drawing	☐ For Your Approval	☐ Your Review
☐ Memo	☐ Clarification Drawing	☐ For Your Signature	☐ Please Comment
☒ Prints	☐ Modification Drawings	☐ Information	☐ Make Recommendation
☐ Sketch	☐ Specifications	☐ Resubmit	☐ Issue Construction Order
☐ Sample	☐ C.O./C.B.	☒ As Requested	☐
☐	☒ HEC-1 & HEC-2 Runs	☐	☐

Remarks: Paul,

Please find enclosed the revised HEC-1 and HEC-2 runs for Dysart Drain.

This has more accurate info than your previous data. Please discard the old information and review this data. We have identified three major breakouts along the Dysart Drain and these flows were computed using the HEC-2 side weir analysis and then input into the HEC-1 model.

Please call if you need anything else.

Copies To: Greg Rodzenko-FCD

File

By: Jeff S. Erickson
Project Engineer

1.4.4 FEMA Regional Office

Ray Lenaburg was contacted to clarify methodology of mapping check profiles. The following correspondence was generated.

The WLB Group Inc

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April 25, 1990

Mr. Ray Lenaberg
FEMA Regional Flood Insurance Office
Presidio of San Francisco, Building 105
San Francisco, California 94129

Re: White Tanks/Agua Fria Area Drainage Master Study (ADMS)
Maricopa County, Arizona
WLB No. 289036

Dear Ray:

This letter is written to confirm our telephone conversation concerning map check profiles for the above referenced project.

As I explained on the phone, we are in the process of preparing the White Tanks/Agua Fria ADMS for the Flood Control District of Maricopa County. Included in the scope of work for the study is approximately 200 square miles of aerial mapping. The mapping will be used to define floodplain limits along several washes and shallow flooding areas behind railroads, canals, and highways.

We proposed a procedure, to the Flood Control District, for map check profiles which results in less profiles than what would be required per "FEMA Guidelines and Specifications for Study Contractors". They authorized the procedure we proposed but ask that we make sure it would be acceptable to FEMA (see attached correspondence).

In our telephone conversation, you stated that any procedure for checking the mapping that the Flood Control District authorizes is acceptable to FEMA. Therefore, we are proceeding with the map check profiles as proposed to the Flood Control District.

If you have any questions, please let me know.

Sincerely,

Mark T. Gavan

Mark T. Gavan, P.E., R.L.S.
Assistant Vice President

MTG:srm

cc: Greg Rodzenko, Flood Control District of Maricopa County

A:LETTERS/MTG-8936.L2

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The WLB Group Inc

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March 30, 1990

Mr. Greg Rodzenko
Flood Control District of
Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Re: White Tanks/Agua Fria ADMS
Map Check Profiles
WLB No. 289036

Dear Greg:

This letter is written to request your concurrence on our proposed method of field surveying the map check profiles.

Our interpretation of the "FEMA Guidelines and Specifications for Study Contractors" dated September, 1985, would indicate that approximately 135 profiles of one mile in length are required for the 200 square miles of mapping. That would result in 135 miles of profiles which we feel is excessive.

FEMA requires one profile for every three stereo models. We propose to survey the profiles on the mile streets that separate the stereo models. Thereby checking 6 stereo models with each profile. The result would be approximately 70 profiles instead of 135.

We also propose a profile length of 1/4 mile mile instead of one mile as required by FEMA. Typically, the 1/4 mile length will result in profiles that will begin at a control point at the section corner and extend half way to the control point at the quarter section corner. That seems like adequate checking to us.

If you have any questions, please let me know.

Sincerely,

THE WLB GROUP, INC.

Mark T. Gavan

Mark T. Gavan, P.E., R.L.S.
Assistant Vice President

MTG:srm

A:LETTERS/MTG-8936.L

Offices located in Tucson, Phoenix, Las Vegas, and Rancho Cucamonga, California
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FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

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APR 1 0 1990

Mr. Mark T. Gavan, P.E., R.L.S.
Assistant Vice President
The WLB Group
333 E. Osborn Road, Suite 380
Phoenix, Arizona 85012

SUBJECT: FCD 89-70
White Tanks/Agua Fria ADMS
Map Check Profiles

Dear Mark:

This letter is to authorize the procedure for map check profiles on the White Tanks/Agua Fria ADMS.

The procedure outlined in your letter of March 30, 1990 is acceptable, i.e., using 70 profiles of 1/4 mile length each. Please check with FEMA that this procedure is acceptable with them.

Sincerely,

Gregory Rodzenko

Gregory Rodzenko

1.4.5 FEMA Washington

No correspondence was generated with this agency.

1.4.6 FEMA Technical Consultant

Michelle Monde, FEMA TEC at Baker Engineers, was contacted by telephone several times to ask for advise or clarification of submittal items. Final correspondence of review items will be documented in the Final Technical Data Notebook.

1.4.7 Copy of Public Notices

Four public meetings have been held regarding this flood study. Two meetings were held in September of 1990 to inform individuals of the study and two more were held in May of 1992 to present the results of the study. The following information includes the public notices, meeting agendas, and attendance lists.

NEWS RELEASE

PUBLIC INFORMATION OFFICE, MARICOPA COUNTY
111 S. 3rd AVE. PHOENIX AZ 85003

Contact: Terri Mulholland or Lulea Bailey (262-3271)
Sept. 13, 1990
For Immediate Release

COUNTY STUDIES POSSIBLE FLOOD CONTROL PROJECTS

Recent monsoon storms have made headlines throughout the Valley, reminding desert dwellers that they face flood hazards. But a study under way by the Maricopa County Flood Control District will help identify ways of reducing such problems for residents of half a dozen communities west of the Agua Fria River.

The White Tanks-Agua Fria drainage master study by the WLB Group will gather information, identify flood hazard areas and map floodplains for the 242 square-mile area bounded by the White Tank Mountains on the west, Grand Avenue on the north, the Agua Fria on the east, and the Gila River on the south. This includes Avondale, Buckeye, El Mirage, Goodyear, Litchfield Park and Surprise.

Residents are invited to share their experience and observations of local flooding and hear details of the study at two meetings -- 7 p.m. Monday, Sept. 24, at Pioneer Elementary School, 540 La Pasada, Goodyear and 7 p.m. Thursday, Sept. 27, at Dysart High School, 11440 N. Dysart Rd., El Mirage.

Details will include how the study is conducted, what kind of information is being gathered and how the information will be used.

Mapping floodplains involves developing detailed topographic maps to determine where water goes and studying rainfall patterns to determine typical amounts of runoff.

Extensive surveying and aerial mapping have been done for the study, but other factors influencing drainage must be considered, such as soil composition,

-more-

slope and vegetation, and land use. Several major canals for agriculture have changed natural drainage patterns. Residential and commercial development affect drainage by reducing absorption, which increases and speeds up surface flows. Highways and railroads, often elevated, cause ponding.

When the study is completed late in 1991, Flood Control District representatives will meet with residents once again to present the information. The next step is developing an area drainage master plan by selecting various flood-control options that address the problems identified in the study. Options selected would accommodate residential and commercial needs, allow for environmental and aesthetic considerations and have a favorable benefit-cost ratio.

Public meetings also will be held at various stages of the master plan process to brief residents on progress and gather input on possible projects. Throughout the study and plan development, the Flood Control District will involve cities, towns and other jurisdictions like irrigation districts in the problem-solving process.

Supervisor Carole Carpenter, whose area is affected, said, "Since conducting a study and developing a plan is such a lengthy process, it's important that citizens are involved from the beginning. These meetings are a vital first step toward citizen input and intergovernmental cooperation. We invite everyone to take part in a process that means a long-range improvement for their community."

For information, residents may call the Flood Control District at 262-1501.

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FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

BOARD OF DIRECTORS
Betsey Bayless
James D. Bruner
Carole Carpenter
Tom Freestone
Ed Pastor

MEMO TO: Risk Management

FROM: Susan Fitzgerald, Public Involvement Coordinator

SUBJECT: Certificates of Insurance

The Flood Control District is sponsoring two public meetings to discuss the White Tanks-Agua Fria Area Drainage Master Study. We need certificates of insurance in the amount of $1 million for Pioneer School and $1,000 for each occurrence and aggregate for Dysart High School. Please prepare the certificates as follows:

6:00 to 10:00 p.m., September 24, 1990
Pioneer School, 450 East La Pasada, Goodyear

6:00 to 10:00 p.m., September 27, 1990
Dysart High School, 14405 North Dysart Road, El Mirage

Please sent the certificates to me so I can forward them to the schools with other documents.

Susan Fitzgerald

Susan Fitzgerald

INFO: DAB
~~GR~~ D.G 8/27/90

FILE: SMF

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

WHITE TANKS-AGUA FRIA AREA
DRAINAGE MASTER STUDY

PUBLIC MEETING
SEPTEMBER 24, 1990
PIONEER ELEMENTARY SCHOOL

A G E N D A

A social period with refreshments will precede and follow the meeting.

1. INTRODUCTION

Public Involvement Coordinator Susan Fitzgerald will discuss briefly the history and role of the Flood Control District, the study, and introduce the District and consulting firm staff.

2. AREA DRAINAGE MASTER STUDY (ADMS)

District Project Manager Greg Rodzenko will explain the Area Drainage Master Study program, the public involvement role and explain the drainage history forms for residents to fill out.

Mr. Rodzenko will provide an overview of other Area Drainage Master Studies the District has performed.

3. WHITE TANKS-AGUA FRIA ADMS

Mark Gavan of WLB Group, the consultant that is performing the study, will use a slide presentation to explain various aspects of the study, such as mapping, hydrology, floodplains, drainage problems and solutions.

4. QUESTION AND ANSWER PERIOD

Please ask questions specifically about the study. Staff will remain after the program for people with questions about individual situations.

5. CLOSING

White Tanks ADMS

Meeting Attendance Form

Location: PIONEER ELEMENTARY

September 1990

Name	Address	Telephone
DAN/CAROL REDICK	12812 PASADENA AVE., L.P. AZ	935-4373
[illegible]	St Rte 3 Box 137 Arlington, Ariz. 85322	386-2037
BILL ROER	Box 264 D Buckeye AZ 85326	386-2055
Greg Green	Rt. 3 Buckeye Az 85326	386-7341
GARY [illegible]	P.O. Box 226 Buckeye 85326	[illegible]
DENNIS SANDERS	16401 W. HIGHWAY GOODYEAR 85338	932-0095
FOX + SUE KRUEGER	2 miles GOODYEAR AZ 85338	932-0227
DON Hughes	16430 W. Lower Buckeye Rd Goodyear	932-[illegible]
DAN & GAIL LUELLING	2022 S. Cotton Lane Buckeye 85326	932-[illegible]
Mr & Mrs D.L. Jones	19[illegible] W. Hilton Buckeye 85326	386-7833 932-[illegible]
Joni Sandoval	16401 W. Magnolia Goodyear 85338	925-2634 932-0095
[illegible]	[illegible]	[illegible]
JAMES + CONNIE DONOVAN	16418 W. LOWER BUCKEYE RD. GOODYEAR 85338	932-[illegible]
SUSANNA (WILLDAN) STRUBLE	1717 Northern Suite 112 Phoenix	870-7600
Lynn Kartchner	2634 S. 159th Ave. Goodyear	932-1637
Dana Rhodes	16422 W. Lower Buckeye Goodyear	932-[illegible]
Stan Ashby	103 W. Broadway Buckeye, AZ	935-4271
Robert Klein	7101 N. Citrus Rd Waddell AZ 85355	935-5357

White Tanks ADMS
Meeting Attendance Form
Location: Dysart H.S.
September 1990

Name	Address	Telephone
THOMAS E. MCCORMICK	21901 W. HILTON AVE.	386-5975
ERVIN V. CARLSEN	21917 W. HILTON AVE.	386-3373
JACQUE PAPPAS	DAILY NEWS-SUN	9778351
BARBARA ROBEY	206 N. FLORENCE AV. LITCHFIELD PK	935-9148
Glen Vortherms	Maricopa Water Dist PO BOX 260, Waddell, AZ 85355	975-2151
Lynn Anderson	Box 1089 - Peoria, Az-85380	935-4282
Mary Anderson	✓	✓
Margaret Truman Baker	Waddell / Litchfield Park	935-3373
Don BRADTKE	CITY OF AVONDALE	932-1909
[illegible]	" "	" "

FLOOD CONTROL DISTRICT
of
Maricopa County

2801 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 506-1501
Fax (602) 506-4601

D. E. Sagramoso, P.E., Chief Engineer and General Manager

BOARD OF DIRECTORS
P. Ben Arredondo
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James D. Bruner
Carole Carpenter
Tom Freestone

MAY 1 1992

NEWS RELEASE
For immediate use

Contact: Terri Mulholland, 506-7062
(Maricopa County PIO)
Susan Fitzgerald, 506-4837
(Flood Control District PIO)

Study determines floodplain boundaries for West Valley

PHOENIX--The first-ever floodplain maps of the far west Valley will be available for view and comment by residents at two public meetings.

Meetings are scheduled for:
May 11, 7 p.m., Dysart High School Cafeteria, 11440 N. Dysart Rd, El Mirage;
May 12, 7 p.m., Avondale Jr. High School, Central and La Canada, Goodyear.

The maps were developed as part of an ongoing drainage study being conducted for the Flood Control District. The affected area encompasses the cities of Avondale, Buckeye, Goodyear, El Mirage, Litchfield Park and Surprise, as well as Luke Air Force Base. Town Managers have been briefed on the new floodplain boundaries.

The drainage study, performed by the WLB Group for the Flood Control District, delineates for the first time more than 150 linear miles of floodplain in the area bounded by the White Tank Mountains and the Agua Fria River, the Gila River and Grand Avenue. At 260 square miles, this is the largest Area Drainage Master Study (ADMS) yet undertaken by the District.

# # #

AGENDA

White Tanks - Agua Fria Area Drainage Master Study Public Meeting

May, 1992

1. Introductions and Overview

   Susan Fitzgerald, Public Information Officer
   Flood Control District

2. Review of study and mapping process

   Slide Presentation
   Jeff Erickson, P.E., Civil Engineer
   Mark Gavan, P.E., Project Manager
   WLB Group

3. Where do we go from here?

   WLB Group, Jeff Erickson and Mark Gavan

4. Public question and comment period

5. Adjourn

PLEASE PRINT
MEETING ATTENDANCE ROSTER

MEETING: White Tanks-Agua Fria ADMS LOCATION: Dysart High School

DATE: May 11, 1992

NAME	TITLE	ORGANIZATION	PHONE
John Hilliard		Citizen	439-5459
Susan Fitzgerald	PIO	Flood Control	506-1501
JAN OPSTEIN	Hydrologist	FCDMC	"
Greg Rodzenko	Planning	FCDMC	506-1501
Art Semick	Councilman	Youngtown	933-3294
Maureen Semick		"	"
Lynn Kortkhner	Director, Pub. Works	Goodyear	932-1637
Robert Mussehrite	City Mgr. Litchfield Pk	Litchfield Park	935-5033

6910-004 1-92

PLEASE PRINT
MEETING ATTENDANCE ROSTER

MEETING: White Tanks-Agua Fria ADMS LOCATION: Avondale Junior [redacted] High School

DATE: May 12, 1992

NAME	TITLE	ORGANIZATION	PHONE
Glen Northoms	Project Engr.	Maricopa Water District	975-2151
DAN HODGES	ASST. FIELD ENGR	SPTCO	602 629-2250
WALLACE BROWN	RESIDENT	LITCHFIELD PARK	935-4155
Dave Dusi	Engr.	CVL	264-6831
PAUL WALKER	Comm Dev Dir	City of Goodyear	932-3494
Rene Sturgeon	Civil Engineer	B+R Engineering	895-0799
Susan Fitzgerald	PIO	FCD	

Also in attendance were 2 FCD staff who ~~were not~~ are residents of study area -
Don Reynolds
Cathy Register

6910-004 1-92

Flood plain maps are nearly ready

Meetings slated for public comment

By Jeff Nelson
Special to Community

PHOENIX — The path taken by raging storm waters determines where and how housing developments are built, where shopping centers are constructed and where preventive measures, such as a greenbelt or concrete channel must be placed.

The problem, county officials say, is these paths are often discovered too late, after a new development has become submerged in water, sand and mud.

To overcome this, the Flood Control District of Maricopa County has been conducting an Area Drainage Master Study in the county.

The study will determine what areas of the county experience flooding and drainage problems in a 100-year flood, or a flood that has 1 percent chance of occurring in year.

These locations are known as areas within a flood plain.

The first flood plain maps for the far West Valley are nearing completion, according to Susan Fitzgerald, a spokeswoman for the Flood Control District.

Called the White Tanks/Agua Fria Area Drainage Master Study, the maps show the flood plain boundaries in a 260-square-mile area. The study encompasses the cities of Avondale, Buckeye, Goodyear, El Mirage, Litchfield Park and Surprise, as well as Luke Air Force Base.

The study area is bounded by the White Tank Mountains on the west, the Agua Fria River on the west, the Gila River to the south and Grand Avenue to the north.

City and town managers have been briefed on the new flood plain boundaries, she said.

Litchfield Park City Manager Robert Musselwhite said the maps give the city and the city's primary developer, Phoenix-based Suncor Development Co., a better understanding of the potential for flooding in the city.

This knowledge, Musselwhite said, will help the city determine such things as when to require the developer to place a retention basin in a commercial or housing development, or how high above

FLOOD

From Page 1

the sidewalk a home can be built and whether a basement should be allowed.

Goodyear City Manager Steve Cleveland said the new maps will also help determine where development should not go, and where parks and open spaces should be placed.

Flood plain maps of the far West Valley will be available for review and comment by residents at two meetings next week.

Fitzgerald said the public, and particularly residents with chronic flooding problems, are encouraged to attend.

"They can come and find out if they're in a flood plain or if their property is in a flood plain," she said, "or they might want to come out to say, 'You missed a spot, there's always flooding in this spot.'"

Public comment from the two meetings will be included in the makeup of the final map. The final map will be issued to the Federal Emergency Management Agency, which responds to disasters.

"Future developers can look at this and see where drainage patterns go and where water goes so they can stay away from it," Fitzgerald said.

The meetings will be at 7 p.m. Monday at the Dysart High School cafeteria, 11440 N. Dysart Road in El Mirage, and at 7 p.m. Tuesday at the Avondale Junior High School, Central and La Canada in Goodyear.

Northwest COMMUNITY

El Mirage
Glendale
Litchfield Park
Peoria
Phoenix
Sun City
Sun City West
Surprise
Youngtown
Wickenburg

Friday, May 8, 1992

THE PHOENIX GAZETTE

THE ARIZONA REPUBLIC

Flood plain maps are nearly set

By Jeff Nelson
Special to Community

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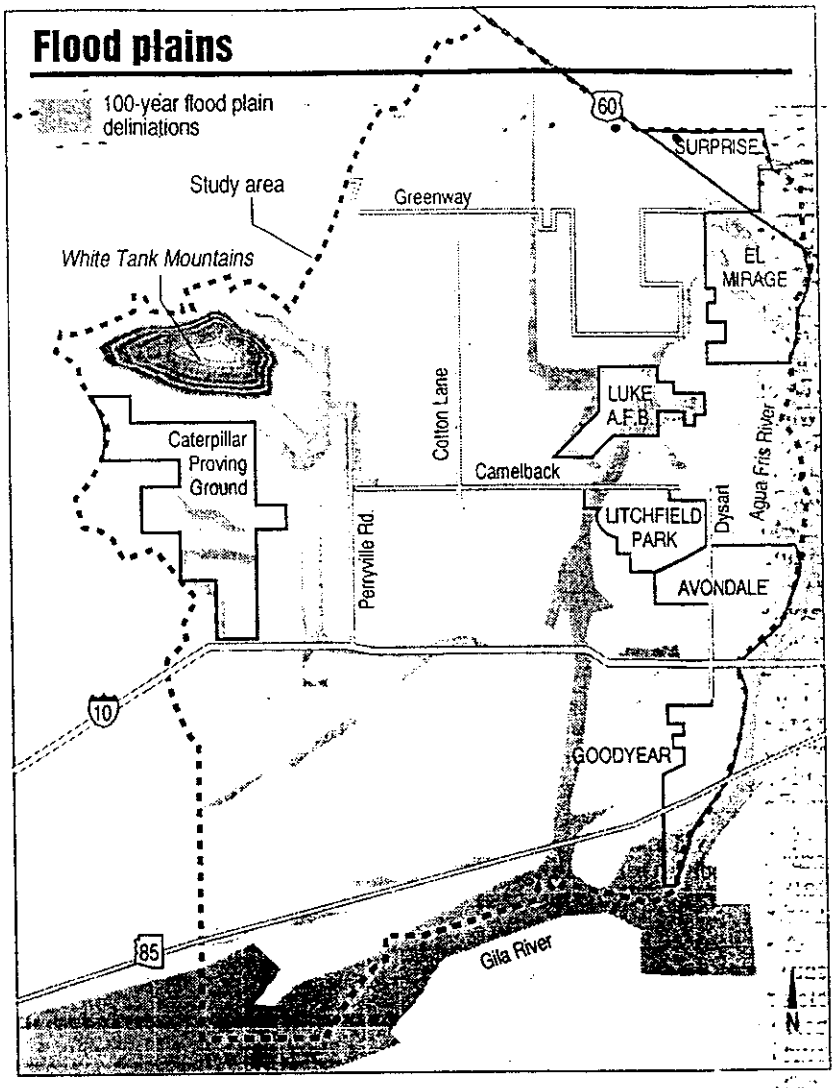
See FLOOD, Page 3

South and Southwest Community sections of R + G



FLOOD

From Page 1

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This knowledge, Musselwhite said, will help the city determine such things as when to require the developer to place a retention basin in a commercial or housing development, or how high above the sidewalk a home can be built and whether a basement should be allowed.

Goodyear City Manager Steve Cleveland said the new maps will also help determine where development should not go, and where parks and open spaces should be placed.

1.5 CONTRACT DOCUMENTS

The scope of work for this flood study is located on the following pages.

SCOPE OF WORK

EXHIBIT A, FCD 89-50

WHITE TANKS/AGUA FRIA AREA DRAINAGE MASTER STUDY

The Engineer shall make the necessary surveys and studies, and shall prepare a report setting forth an Area Drainage Master Plan (ADMP) for stormwater management in the White Tanks/Agua Fria study area. The study area covers the Flood Control District Watershed No. 16, the White Tanks Watershed, and part of the Flood Control District Watershed No. 19, the Lower Agua Fria Watershed. This area is roughly bounded by Grand Avenue on the north, the Agua Fria River on the east, the White Tank Mountains and McMicken Dam on the west, and the Gila River on the south. See Exhibit 1.

The purpose of this study is to identify problems and develop solutions associated with drainage in the existing, developed portions of the watershed. The Consultant, through the use of structural and non-structural methods, should develop solutions to drainage problems by identifying drainage outfalls for existing and proposed drainage/flood control structures. The study products will include:

I. Background Materials

II. Mapping

III. Hydrology

IV. Floodplain Delineations

V. Area Drainage Master Plan

VI. Supporting Documentation

VII. Project Coordination and Study Management

The work shall include the following tasks:

I. Background Materials:

Assemble and review pertinent maps, studies, land use plans, and private master plans, including existing Flood Control District studies within the area. Included in this data search will be the drainage studies for Litchfield Park, plans for three canals, three railroads, airbase drainage, I-10, two Flood Control District structures, CAT proving grounds, Dysart Drain information, Goodyear Airfield, Murphy Dam, drying ponds for Morton Salt (if breached will flood Dysart Drain), Cotton Lane Freeway and private development plans.

II. Mapping:

As identified in Attachment I.

III. Hydrology:

As identified in Attachment II.

IV. Floodplain Delineations:

As identified in Attachment III.

V. Area Drainage Master Plan:

A. The Consultant shall develop a comprehensive list of known flood problems on the watershed. This list will require coordination with the officials from each of the municipalities, transportation agencies, irrigation districts, and other sources. The Consultant will then categorize these problems on the basis of being independent or dependent problems, to be used in identifying drainage alternatives.

B. The Consultant shall prepare a comprehensive drainage inventory of existing drainage facilities in the watershed, their condition, capacity, and ownership, including natural washes. These facilities are to be part of the base map for alternatives. The Consultant should make maximum use of incorporating these facilities, where feasible, as part of the stormwater management plan alternatives (at least two alternatives).

Elements of the alternate plans may include, but are not limited to:

1. Detention or retention basins.

2. Channels and/or pipes.

3. Regulatory or policy changes affecting density or orientation of development, or detention/retention standards.

4. Nonstructural concepts.

5. Combinations of the above.

C. The Consultant shall evaluate the capacity of the existing Dysart-Agua Fria Drain, and if needed, develop alternatives to increase the size to 10-year, 50-year, and 100-year capacity, plus freeboard.

D. The Consultant shall evaluate the capacity and operation of White Tanks Flood Retarding Structures (F.R.S.) No. 3 and No. 4. The Consultant will also provide plans that ensures that all drainage west of the Beardsley Canal alignment enters No. 3, and that all drainage west of Jackrabbit Trail enters No. 4. These plans will include modifications required of the structures to allow for detention of the 100-year storm (24 hour) without emergency spills.

Any modifications to these facilities must meet state dam safety criteria: Supervision of Dams, Reservoirs and Projects, September 1983. Outfall drainage corridors to the Gila and Agua Fria Rivers, and other anticipated channels shall be identified.

E. Evaluate the alternatives in terms of capital costs, effectiveness, environmental impacts, potential for staged construction, acceptability to local residents, and compatibility with other projects and plans. Prepare a generalized working matrix for a ranked comparison of the alternative drainage plans, along with their respective benefits and costs.

F. Present the ranked alternatives to the Flood Control District staff in an oral presentation format. Provide sufficient background and cost information to the decision process for selection of the preferred alternative plan.

G. Recommend, and submit to the District for approval, the design criteria and objectives to be applied during the development of this area under the Area Drainage Master Plan, including:

1. Maximum allowable velocities.

2. Channel characteristics, e.g., alignments and cross sections.

3. Type(s) of drop structures.

4. Provision for runoff in excess of design capacity and maximum depth of flow in streets.

5. Maximum depth of basin and time required to drain basin.

6. Maximum size or frequency-capacity for pipes and box culverts.

7. Selection of dip vs. culvert crossings, and 100-year "all weather" crossings.

8. Water quality

   a) Stormwater runoff: characterize pollutants as a function of precipitation and land use, i.e., rainfall on an industrial area will produce __?__ average pollutants per unit area in the stormwater runoff; likewise in a suburban residential area, and in an agricultural area.

   b) Point source pollution: identify major point sources of pollution, i.e., industrial; generally characterize the pollutants and the manner or circumstances under which they are being introduced to the environment at large.

H. Develop the selected system proposed for the Area Drainage Master Plan, to concept plan level only. The level of detail for the drainage plan will be limited to drainage areas of at least one square mile, or peak flows of not less than 800 cfs, unless extraordinary local conditions warrant Flood Control District participation at a more detailed level.

1. Establish approximate sizes, slopes, profiles, alignments, and plan and profile of proposed channels and pipes at 1"=400'; locations as appropriate for channels, pipes, trunk mains, culverts, and detention/retention basins.

2. Determine the existing and required rights-of-way.

3. Determine critical utility "interferences".

4. Estimate preliminary quantities and costs for each element of the system for the 100-year design flood and the 2-, 10-, 25-, and 50-year level of protection based on size reduction for the approved plan.

5. Estimate maintenance requirements and costs for the recommended ADMP.

6. Recommend a phased program for implementation of the system and estimate the phased program costs, assuming a planning horizon of the year 2015.

7. Water quality-mitigation of stormwater sources/point sources.

   a. Stormwater: industrially zoned areas produce "normal" pollutants as rainwater falls on the buildings and parking lots; likewise with residential and agricultural areas. At present, such pollution is considered "normal, background" pollution. Mitigation would presumably consist of some type of area-wide collection and treatment system.

   b. Point source: major point source polluters would have to mitigate their pollution consistent with EPA and Arizona Department Environmental Quality regulations.

VI. Supporting Documentation will include but is not limited to:

A. Mapping:

1. One complete set of 9" X 9" contact prints of the aerial stereo photographs sequentially numbered and catalogued.

2. One complete set of contour maps, blueline, draft copy for Flood Control District reference during the project, delivered immediately following the topographic mapping.

3. One complete set of contour (base) maps at 1"=400' scale, in reproducible form (mylar); final format.

4. Two transparent overlays for the above mylars: one overlay with the delineated floodplains; the second overlay depicting the various elements of the area drainage master plan.

5. One complete set of contour maps at 1"= 400' scale with the floodplain delineations in reproducible form (mylar); final format.

6. One complete set of contour maps at 1" = 1200' scale, in reproducible form (mylar); final copies.

7. Three transparent overlays for the above mylars: one overlay with the hydrologic subwatersheds; a second overlay with the delineated floodplains; a third overlay depicting the various elements of the area drainage master plan.

8. One complete set of presentation maps: USGS quad sheets mounted on foam board with overlays for the delineated floodplains, drainage patterns, and the subwatersheds.

9. One complete set of mylars for the foldout maps (no larger than 11" x 17") used in the report. One sheet for each of the following: topography, delineated floodplains, sub-basins for the watershed, the various elements of the ADMP, land use patterns (zoning), and hydrologic soil groups.

10. One-half inch magnetic tape formatted at 1600 bpi containing the topographic data and the digitized floodplain/floodway boundaries in either the AutoCAD DXF ASCII format or the Intergraph ISIF ASCII format.

11. Tabular list of control points (ERM's) used with descriptions, elevations, and coordinates.

B. Reports:

REPORT FORMAT:

A. SUMMARY
   1. Description of Study Area
   2. Scope of Project
   3. Selection of Alternative Plan
   4. Recommended Alternative
      a. Proposed Structural Improvements
      b. Non-structural Improvements
      c. Floodplain Management Recommendations
   5. Construction and Maintenance Programs
      a. Costs

B. EXISTING DRAINAGE BASIN CONDITIONS
   1. Basin and Sub-Basin Descriptions
   2. Drainage Conditions
      a. Natural Drainage Features
      b. Existing Improvements
   3. Runoff Concentration Points

C. HYDROLOGY
   1. Rainfall
   2. Peak Discharge Determinations
   3. Flood Routing Methods and Results

D. FLOODPLAINS AND AREAS OF POTENTIAL FLOODING
   1. Summary of Existing Flooding Complaints
   2 Determination of 100-year Floodplains: Methods and Results
   3. Areas and Locations of Potential Flooding

E. BASIN MANAGEMENT ALTERNATIVES
   1. Structural Improvements
   2. Non-Structural Solutions
   3. Floodplain Management Recommendations

F. IMPLEMENTATION PROGRAM
   1. Phasing of Structural and Non-Structural Solutions
   2. Costs

G. REFERENCES

H. PLAN AND PROFILE DRAWINGS

I. LIST OF FIGURES
   Location Map
   Topographic map
   Maps of hydrologic Basins and Sub-Basins
   Flood Routing Hydrographs
   100-year Floodplains and Areas of Potential Flooding
   Land use/zoning map
   Map depicting proposed ADMP
   Map of hydrologic soil groups

J. LIST OF TABLES
   Hydrologic Sub-Basin Characteristics
   Peak Discharges
   Unit Costs for ADMP features
   Projected Costs for future condition watershed conditions and ADMP features
   Elevations of spillways and inverts of key drainage structures; water surface elevations at those same points for the design discharges
   Elevation Reference Marks (ERM's)

VII. Study Management Tasks will include the following:

A. The Consultant shall participate in conferences and liaison with the appropriate officials and agencies during the progress of the work, up to final acceptance by the District. The following is a list of the required meetings for coordination, review, and approval of the work in progress:

   1. Review and approval of study hydrology and preliminary review of mapping.

   2. Regular coordination (at least every three weeks) with the District's Project Manager. Participation in monthly progress meetings with the Review Committee.

3. Milestone coordination meetings in the development of the selected alternative system, held at the 33%, 67% (presentation of the alternatives and identification of the selected alternative system by the Review Committee) and 90% completion points with the Review Committee.

4. Presentation of the final Area Drainage Master Plan and maps to the Review Committee.

5. Coordinate and support the public involvement presentations. There will be three meetings, each given at two locations, at the progress points noted on the attached "Major Task Phases", to inform the public of the status and results of the project. The Consultant will be responsible for the presentations, and all presentation materials, including hand-outs, slides, overheads, and presentation boards. The meetings will be held at locations to be specified.

B. The Consultant shall provide briefing summaries and appropriate printed materials at each of the scheduled meetings in ten (10) copies, and an additional five (5) copies for milestone meetings. The Consultant shall provide meeting summaries or minutes within one week after each meeting for distribution by the District.

The final draft report and draft Executive Summary shall be submitted in ten (10) copies to reach the District 10 days prior to the scheduled presentation briefing. The required maps and one copy of the final Master Plan report shall be submitted to the District for proofing within 14 days following the presentation briefing meeting. It shall include all corrections and address all comments raised during the briefing. The final Master Plan shall be printed and bound, and twenty (20) copies furnished to the District together with thirty (30) copies of the Executive Summary within two weeks of return of the proof copy. A reproducible of each page/sheet of printed material in the report shall be delivered to the District together with the printed copy. A copy of all calculation sheets and computer input data produced by the Consultant in developing the report shall also be furnished at that time. All materials generated to produce this report are the property of the District, although the Consultant may retain a copy for their own use.

The District shall provide any existing data, maps, and plans deemed pertinent by the Consultant and the District in assisting in the progress of the study. The originals of all data, maps, and plans provided by the District and other agencies shall be returned to the District at the time of final contract billing by the Consultant.

ATTACHMENT I

WHITE TANKS/AGUA FRIA AREA DRAINAGE MASTER STUDY

MAPPING

A. Procedures for General Mapping:

1. Prepare topographic mapping to a 2 foot contour interval, with spot elevations on all section line and mid-section line roads. This would be for the entire area excluding the contributing drainage area to White Tanks #3 & #4, as identified on the attached map. The Flood Control District has existing mapping for this portion of the watershed, which must be tied into and noted on the new mapping.

2. Ground Control:

   a. The Consultant shall provide all survey control.

   b. The Consultant shall systematically set panel points and establish horizontal and vertical control throughout the areas to be mapped for use in compilation by the aerial survey contractor. Where readily available, surveys will tie into the State Plane Coordinate System. Field control shall be sufficient to readily allow for compilation of maps by the aerial survey contractor at the desired map scale and contour interval and will be based on the National Geodetic Vertical Data (NGVD).

   c. The horizontal and vertical control points shall be located and marked by the Contractor. The controls for the area mapping shall be in sufficient numbers and shall be in locations which will be compatible with the accuracy of the mapping requirements. The controls shall be of at least third order accuracy. Section corners, quarter corners, and mid-section points shall be used for control points wherever possible. The ground control will be established per Cooper Aerial's control diagram of October 27, 1989. (See proposal diagram).

B. Map Standards:

1. Digital design, contour and planimetric data developed for this project shall be delivered in AutoCAD DXF ASCII format, as specified in Autodesk, Inc., publication TD106-009 (May 7, 1986). Layer names and graphics attributes shall be fully documented by the Consultant. The delivered DXF files shall be compatible with the requirements, and subject to the limitations, of the ESRI DXFARC software translator as detailed in the January 1989 release of the "ARC/INFO users Guide". All DXF file deliveries shall be in ASCII format on industry-standard 1/2" magnetic tape, 2400-foot reels, written in a generic unlabelled COPY format, with specified record-lengths and blocksizes.

- or -

Digital design, contour and planimetric data developed for this project shall be delivered in intergraph ISIF ASCII format, as specified in intergraph publication DIX4110 (May 12, 1985). Layer names and graphics attributes shall be fully documented by the Consultant. The delivered ISIF files shall be compatible with the requirements, and subject to the limitations, of the ESRI SIF2ARC software translator as detailed in the January 1989 release of the "ARC/INFO Users Guide". All ISIF file deliveries shall be in ASCII format on industry-standard 1/2" magnetic tape, 2400-foot reels, written in a generic unlabelled COPY format, with specified record-lengths and blocksizes.

2. The Consultant shall provide permanent non-erasable topographic mylar with a scale of 1-inch equal to 400 feet scale, with a contour interval of 2 feet for all mapping with the exception of section line roads which will have spot elevations. Each manuscript shall

   include a minimum of a north arrow, scale, section corners, current and proposed streets, State Plane Coordinate System, major drainage features, city limits, cross section lines, channel station center line, index map, and reference marks used in ground control. The mapping will have an accuracy such that ninety percent (90%) of all contours shall be within one-half contour of the true elevations and the remaining ten percent (10%) of the contours shall not be in error by more than one contour interval.

3. The Consultant shall provide permanent non-erasable topographic mylars as described above in Section C.2 with delineated floodplains included.

4. Sketch maps no larger than 11" x 17" for the study area, and for each alternative must be included in the narrative report.

5. The Presentation Maps shall be on U.S.G.S. 7.5 minute Quadrangle Maps and include:

   a. The study area: All current and proposed streets, major arterials and freeways, section lines, major drainage features, presently delineated floodplains areas, and city limits. This map shall serve as a base map.

   b. Maps showing the existing drainage patterns, the subwatersheds, and indicating the flows at major intersections and concentration points.

   c. Maps showing the future drainage patterns, if different from existing.

   d. Maps of the floodplain, floodway, and ponding delineations

6. Hydrologic Work Maps should be at a scale of 1 inch = 1200 feet and shall include: reproducible transparent overlay maps of existing drainage patterns, subwatersheds; major flow paths; and general topographic maps.

ATTACHMENT II

WHITE TANKS/AGUA FRIA AREA DRAINAGE MASTER STUDY

HYDROLOGY

A. The Consultant shall use the U.S. Army Corps of Engineers computer program HEC-1 to develop a hydrologic model for the area. Using appropriate hydrologic judgement, sub-basins are to be identified that provide reasonable depiction of the watershed condition. However, unless specific reasons dictate a departure from the above-mentioned method, the largest sub-basin size should not exceed one square mile in size. An appropriate time step and number of ordinates is to be selected that allows for complete calculation of the flood hydrograph without sacrificing resolution of the flood peak. All calculations, or assumptions used in developing sub-basin or routing parameters shall be documented and made a part of the appendix for the hydrology report. The specific hydrologic techniques to be used in this study are:

1. Rainfall	:	100-year 6-hour storm, FCD Distribution(s)
2. Excess	:	Green - Ampt, or Initial and Uniform Loss Rate
3. Unit Hydrograph	:	Clark or appropriate S-graph
4. Time of Concentration	:	Papadakis
5. Routing	:	For floodplain areas or channels where detailed cross sections are being developed normal depth (modified puls), in other areas use Muskingum Routing or kinematic wave.

All hydrologic and hydraulic parameters shall be assessed for realistic values such as velocities and quantities of flows.

B. An existing condition model shall be developed and shall be based on existing land uses as identified at the time of mapping, or other recent area wide aerial mapping. All identified pending drainage improvements will be listed with anticipated completion dates. The Consultant and Flood Control District staff will then meet to identify which features will be assumed to be in place for the purposes of the existing condition hydrology. The assumption will be based on those facilities that are proposed to be in place roughly within one year from the completion of this study, with satisfactory documentation in the model. Significant private and regional retention, and agricultural tailwater sumps shall be incorporated into the model. However, pre-1987 retention for private development shall only be included if it is a common basin (not on lot), and field verification indicates substantial conformance to the approved plans. As this study progresses towards final approval of the hydrology, if any development of 200 acres or larger is approved and construction is imminent the drainage facilities for this feature shall be included in the existing condition hydrology.

C. The future condition hydrology model will identify specific alternatives. Required changes should only be related to changes in land use, modification of routing reaches, or the inclusion/exclusion of specific structures and/or management practices. The future condition model should be based on a fully developed watershed as envisioned by current planning documents (at the time the modeling is initiated), and in general assume that current retention criteria are fully enforced.

D. Specific deviations from this hydrologic scope shall not be undertaken without the specific written concurrence from the Flood Control District.

ATTACHMENT III

WHITE TANKS/AGUA FRIA AREA DRAINAGE MASTER STUDY

FLOODPLAIN DELINEATIONS

A. Prepare topographic mapping to a 2 foot contour interval with a scale of 1" = 400 feet, with spot elevations or 1 foot contours on all section line and mid-section line roads, for all floodplain delineation areas as identified in Section D.4.

B. Procedures for Topographic Mapping of Flood Hazard Areas:

1. Prepare photo-topographic maps to the same specifications as in Section A "Procedures for General Mapping" of this document, or FEMA criteria, whichever is more stringent, for all floodplain delineation areas as identified in Section D.4.

2. Ground Control for Floodplain Delineations:

a. All topographic mapping and survey work shall meet or exceed Federal Emergency Management Agency (FEMA) minimum criteria as defined in FEMA Document 37, Flood Insurance Study Guidelines and Specifications for Study Contractors, Appendix 4, September 1985. This would include, but is not limited to: the establishment of "permanent" elevation reference marks (ERM's); field control; and verification of profiles by the ground survey profile procedure.

b. Horizontal and Vertical Control: Systematically set panel points and establish horizontal and vertical control throughout the area to be mapped for use in compilation by the aerial survey contractor. Where readily available, surveys will tie into State Plane Coordinate System. Field control shall be sufficient, at least one "permanent" point per mile, such point(s) being used as Elevation Reference Marks (ERMs). Surveys will be based on National Geodetic Vertical Datum (NGVD), per FEMA guidelines. "Permanent" survey points shall consist of existing monumentation, such as brass caps or similar survey monuments. Where additional monumentation is needed, survey markers conforming to Maricopa Association of Governments (MAG) Uniform Standard Detail for Public Works Construction, detail 120-1, Type C, shall be placed 2" +/- above grade. Elevation Reference Marks will be labelled on available maps and described in a manner which allow them to be readily located in the field.

c. "As-Built" plans or surveys of all bridges and hydraulic structures are to be obtained by the Study Contractor.

d. The Consultant shall verify profiles for mapped floodplains. The ground survey profile procedure as described in FEMA Document 37 or other methods approved by FEMA.

C. Field Survey:

Cross sections: Stationing will be from left to right looking downstream. Cross sections will be space approximately every 500 feet, unless geographic or structural constraints dictate otherwise. Identification of cross sections will be in river miles, increasing upstream. The channel station centerline will be designated as stations 10,000. The location and alignment of cross sections and channel centerline will be submitted for the Flood Control District's review and approval prior to digitizing cross section data.

D. Floodplain and Floodway Delineation:

1. The Consultant will prepare the study using the guidelines established in "The Flood Insurance Study Guidelines and Specification for Study Contractors', dated September 1985 and "Appeals, Revisions, and Amendments to Flood Insurance Maps", September 1985.

2. The Consultant will conduct a field reconnaissance of all study reaches. This will include observation of channel and floodplain conditions for estimation of Manning's 'N' values; photographic documentation of floodplain characteristics; overflow areas; inspection of levees or other flood control structures; and measurement of bridge dimensions.

3. A written summary of the field inspection, including photographs to document 'N' value estimation will be submitted to the Flood Control District for review and approval.

4. The Consultant will delineate the following floodplains and floodways, to a detailed standard, using the U.S. Army Corps of Engineers HEC-2 Water Surface Profiles computer model, for the 100-year flood event. Interpolated cross sections, and optimization routines for 'N' values, etc., are not to be used without the specific written concurrence from the Flood Control District.

   a. Drainage swale originating within Luke Air Force Base - continuing south between Bullard and Reems Roads to Roosevelt Canal where some flow splits to the west and some flow continues south to the Gila River. Mapping is to be continuous to the Gila River. (14.2 miles).

   b. Caterpillar Proving Grounds - Map 7 major washes originating from the White Tanks continuing southeast and diverted along the Tuthill Dike. Delineate from the western mapping limit to the Tuthill Dike. (11.5 miles).

   c. White Tanks Structures #4 - Mapping of the flood hazard area which develops from the 100-year peak flowing over the spillway, continuing to the Roosevelt Canal. (1.25 miles).

   d. Roosevelt Canal - Ponding limits along the north side of the Canal starting from the Agua Fria continuing west to the study limits. (12.5 miles).

e. Buckeye Canal - Ponding limits along the north side of the Canal starting from the Agua Fria confluence with the Gila River (between Litchfield Road and Bullard Road) and continuing west to the study limits. (9.5 miles).

f. Airline Canal - Ponding limits along the north side of the Canal starting from the Agua Fria continuing west to the outfall. (3.8 miles).

g. Tuthill Dike - Ponding limits along the west side of the dike starting from Bethany Home Road to the junction of White Tanks No. 4. The dike should be evaluated to determine if it meets FEMA criteria for a dike/levee. (5 miles).

h. Jackrabbit Dike - Ponding limits along the west side of the dike starting from Bethany Home Road to the Buckeye Canal. It is to be determined if the dike meets FEMA criteria for a dike/levee. (8.7 miles).

i. Elevated Railroads - Ponding limits behind all elevated railroads. (35 miles).

j. Lower El Mirage Wash - Redelineate from the down-stream limits at the Agua Fria continuing northwest to the railroad. (2.5 miles).

k. Breakout Flows - from the wash west of Beardsley Canal between Olive Avenue and Northern Avenue that flow over the Canal and to the east. (2 miles).

l. Dysart Drain - from the outlet to Reems Road, delineation to include breakout flows to Luke Air Force Base. (4.7 miles).

m. Lower El Mirage Wash Tributary - Redelineate from Lower El Mirage Wash to Litchfield Road. (3 miles).

n. Beardsley Canal - Ponding along the Beardsley Canal from Peoria Avenue to I-10. (8 miles).

o. I-10 - Ponding limits along Interstate 10 from the Agua Fria River to the west study limit. (11 miles).

p. Murphy Dam - Delineate the wash that flows to Murphy Dam. (1 mile).

q. Washes Upstream from White Tanks #3 - Prepare floodplain/floodway delineations on the six major washes upstream of White Tanks #3. Study to the western mapping limit. (13 miles).

Total - 146 Stream Miles

5. Cross section orientation may need to be altered after running of the HEC-2 model to adjust for normality to flow per FEMA criteria.

6. Bridges must be modeled in compliance with HEC-2 modeling requirements for the selected routine. Where multiple bridges occur, each bridge will be modeled separately.

7. For floodplains identified as ponding areas, it is preferable to analyze the area by using the HEC-2 model, which will provide the District with water-surface-elevations. If appropriate, the Consultant shall identify in the ponded floodplains a floodway. The purpose of this floodway is to allow the pond to seek a constant stage throughout the areal extent of the ponds, versus the creation of two independent ponds.

8. Flood zones must be determined according to FEMA criteria.

9. The delineation work shall meet requirements for floodplain delineations as prescribed by FEMA and the Arizona Department of Water Resources.

10. The Consultant shall review pertinent Flood Insurance Studies and Floodplain delineations and include this information when detailing the need for revisions and effects on the floodplain.

11. The Consultant shall prepare a floodplain delineation report to be submitted to FEMA independent from the ADMS report, for FEMA's review and approval. The Consultant is responsible for all changes requested by FEMA and is responsible for gaining report approval.

E. <u>FEMA Coordination:</u>

1. The Consultant will submit coordinated peak discharges to FEMA for review by the Technical Evaluation Contractor (TEC), prior to the submittal of the hydraulic analysis. The Consultant will respond to questions by the TEC and make modifications to the hydrologic model if necessary.

2. The Consultant will submit maps, report, and HEC-2 model to FEMA for review by the Technical Evaluation Contractor (TEC). The Consultant will respond to questions by the TEC and make modifications to maps and HEC-2 analysis if required.

2.3 SURVEY FIELD NOTES

Copies of the field notes for the mapping control are provided in Appendix B. Also included in Appendix B is a map of the study area which shows the ground control points used for the aerial mapping. This map is necessary in order to review the field notes, since the field notes refer to the ground control points.

2.3.1 Horizontal Datum

The horizontal control is based on the NORTH AMERICAN 1927 DATUM. The following Coast and Geodetic Survey monuments were used to establish the horizontal control. Descriptions of the monuments are included in Appendix B.

Name of Survey Monument	State Plane Coordinates X	Y
FRIA	323,802.12	962,668.81
LITCHFIELD	364,498.81	917,079.64
POK	325,284.48	909,915.67
BRADLEY	348,688.75	863,635.23

2.3.2 Vertical Datum

The vertical control is based on the NATIONAL GEODETIC VERTICAL DATUM. The following National Geodetic Survey (NGS) monuments were used to establish vertical control.

Name of Survey Monument	Location	Elevation
BEDROCK-1	Northern Avenue, 2.5 Miles West of Beardsley Canal	1514.92
N-475	Northern Avenue, 2.5 Miles West of Beardsley Canal	1474.15
M-475	Northern Avenue, 1.8 Miles West of Beardsley Canal	1343.22
L-475	Northern Avenue, 0.7 Miles West of Beardsley Canal	1264.23
P-475	Northern Avenue, 2.5 Miles West of Beardsley Canal	1443.02

All of these NGS monuments used for vertical control are located in the foothills of the White Tank Mountains and are not subject to ground subsidence. This is important since parts of the study area have experienced considerable ground subsidence due to groundwater withdrawal for agricultural irrigation purposes.

2.3.3 Land Subsidence

A comparison of the vertical control used for this study versus the 1957 U.S.G.S. Quadrangle Maps revealed subsidence of as much as 18 feet. The part of the study area that experienced the most subsidence is the area bounded by Bethany Home Road on the south, the Beardsley Canal on the west, Greenway Road on the north, and Dysart Road on the east.

Subsidence levels in this area ranged from 5 to 18 feet. It is interesting to note that very little, if any, subsidence occurred along the west edge of the Agua Fria River and in the area south of McDowell Road.

In addition, no significant subsidence occurred in the hilly, White Tank Mountain foothill areas west of the Beardsley Canal. This would be expected since depth to bedrock is shallow west of the Canal.

2.3.4 Elevation Reference Marks

Included in Appendix B is a complete list of elevation reference marks for the study area. These are the same reference marks which appear on the floodplain maps.

2.4 WATERSHED MAPS

Refer to Section 3, Hydrologic Analysis, for a copy of the Drainage Area Map.

The base for the watershed map is the U.S. Geological Survey 7 1/2 Minute Quadrangle Maps. The drainage subbasins were delineated using the 1" = 400' topographic maps and the boundaries were transferred to the quadrangle maps.

2.5 HYDRAULIC ANALYSIS MAPS

The base mapping used for the hydraulic analysis is the 1" = 400', 2' contour interval mapping.

2.6 FIRM, FHBM DRAFT MAPS

The base mapping for the drafts of the FIRM's and the FHBM's is also the 1" = 400', 2' contour interval mapping.

2.7 COMMUNITY MAPS

The floodplain maps provided under separate cover include corporate limits and street names. This information was taken from the "Metropolitan Phoenix Street Atlas", Wide World of Maps, Inc., 1991 Edition. A copy of the 1992 edition is included in this submittal under separate cover.

SECTION 2: MAPPING AND SURVEY INFORMATION

2.1 DESCRIPTION OF MAPPING

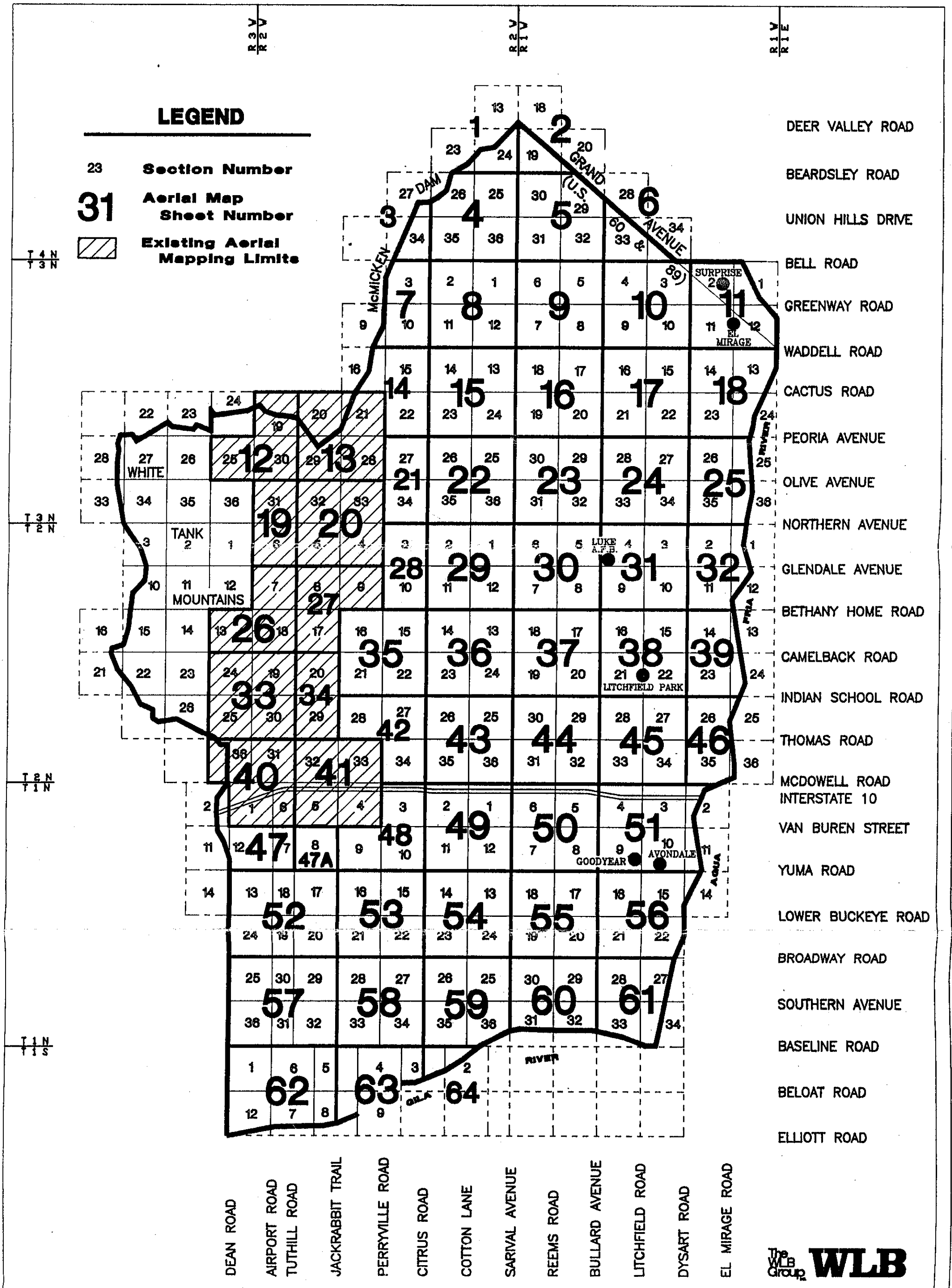
As part of the White Tanks/Agua Fria Area Drainage Master Study, The WLB Group and its mapping subconsultants prepared approximately 200 square miles of topographic mapping. The map scale is 1" = 400' with a 2' contour interval. In addition, the Flood Control District of Maricopa County had previously contracted with Cooper Aerial Survey Company to provide 30 square miles of topographic mapping on the east side of the White Tank Mountains. This mapping was also done at a scale of 1" = 400' and a 2' contour interval. These two mapping efforts nearly cover the entire watershed. Please refer to the following index map for the boundaries of the new and existing mapping. The only area that is not covered is approximately 17 square miles in the White Tank Mountains. In that area, the U.S. Geological Survey 7-1/2 Minute Quadrangle Maps were used to define drainage boundaries and watershed characteristics.

2.2 INDEX OF MAPS

The following map is an index of the 1" = 400' maps used for the White Tanks/Agua Fria ADMS.

WHITE TANKS / AGUA FRIA
AREA DRAINAGE MASTER STUDY

INDEX MAP



SECTION 3: HYDROLOGIC ANALYSIS

3.1 METHOD DESCRIPTION

The hydrologic methodology incorporated in the White Tanks/Agua Fria Area Drainage Master Study (ADMS) utilizes the new "Hydrologic Design Manual for Maricopa County, Arizona" dated April, 1990. This manual is a comprehensive compilation of technical procedures for the estimation of rainfall-runoff which is used for the purpose of designing and analyzing drainage facilities in Maricopa County.

Hydrologic parameters were calculated for each subbasin within the study area. The WLB Group, Inc. created a worksheet utilizing the Lotus 1-2-3 program in which subbasin parameters; such as flow length, slope, land use, soil type, vegetative cover, and soil moisture condition, were used to calculate average Green-Ampt loss rate parameters and lag time for each subbasin. These values were then input into a computer program supplied by the Flood Control District of Maricopa County (FCDMC) called MCUHP2 (Maricopa County Unit Hydrograph Procedure 2) dated October 2, 1990. This program calculates unit hydrographs based on the U.S. Army Corps of Engineers S-graphs that were developed for the Phoenix Area. The program also creates HEC-1 input files that can be utilized within the HEC-1 Flood Hydrograph Package computer program created by the U.S. Army Corps of Engineer's Hydrologic Engineering Center. The HEC-1 program used for this study was the June 1, 1988 version and was acquired directly from the U.S. Army Corps of Engineers Hydrologic Engineering Center.

3.2 PARAMETER ESTIMATION

Due to the large amount of base data generated by this Area Drainage Master Study, separate notebooks for each physical parameter calculated are supplied as appendices to this report and will be referred to when discussing each parameter calculated.

3.2.1 Drainage Area Boundaries

The drainage area for the White Tanks/Agua Fria ADMS is approximately 220 square miles with approximately 2/3 of the watershed draining to the Gila River and 1/3 of the watershed draining to the Agua Fria River. The drainage area is bounded on the north by McMicken Dam and Grand Avenue; on the east by the Agua Fria River; on the south by the Gila River; and on the west by Dean Road and the White Tank Mountains. Several incorporated communities are located within the study area including the Cities of Avondale, El Mirage, Goodyear, Litchfield Park, and Surprise; the Town of Buckeye; Luke Air Force Base; and strip annexed areas of the Cities of Glendale and Phoenix.

Prominent features located within the drainage area are the White Tank Mountains, White Tanks Flood Retarding Structures #3 and #4, Interstate 10, interim Estrella Freeway, Atchison Topeka and Santa Fe Railroad, Southern Pacific Railroad, Airline Canal, Buckeye Canal, Beardsley Canal, Roosevelt Irrigation District Canal, Litchfield Park Detention Facility, Dysart Drain, Tuthill Dike, Bullard Wash, Caterpillar Proving Grounds, Case Proving Grounds, White Tank Mountain Regional Park, Agua Fria River, and Gila River. (Refer to the attached 11" x 17" Study Area Map.)

Subbasins were delineated using 1" = 400', 2-foot contour interval topographic mapping developed for this study by Cooper Aerial and Western Air Maps. Also, aerial photographs were used and field reconnaissance trips were taken to determine subbasin boundaries that were not readily apparent on the maps. Points of concentration that were of particular interest were also used to define subbasin boundaries. Refer to the following 11" x 17" Drainage Area Map. A 1" = 4000' Drainage Area Map is also provided with the hardcopy of the HEC-1 model located in Appendix C under separate cover.

3.2.2 Physical Parameters

3.2.2.1 *Unit Hydrograph Calculation*: The Phoenix Valley S-graph was incorporated per instructions from the FCDMC to calculate unit hydrographs for use within the HEC-1 model. This, along with the use of Green-Ampt loss rate parameters, forms the basis for calculating runoff hydrographs for each subbasin throughout the watershed. The Phoenix Valley S-graph was selected based on the criteria of being applied to a large, mostly undeveloped watershed. The majority of the watershed is in agricultural uses with a lesser degree of desert and mountainous terrain and even fewer areas of urban development.

The Phoenix Valley S-graph was developed by the U.S. Army Corps of Engineers and can be found in "New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology, Part 1", U.S. Army Corps of Engineers, Los Angeles District, October, 1974.

The MCUHP2 program uses the Phoenix Valley S-graph to calculate unit hydrographs. Input requirements for MCUHP2 include basin area, basin lag, and Green-Ampt loss rates.

A number of variables are involved in calculating loss-rate parameters for the Green-Ampt method. The "Hydrologic Design Manual for Maricopa County" describes the steps involved in calculating these parameters and this manual is available from the Flood Control District of Maricopa County upon request. It would be repetitive and cumbersome to relate all of the details involved in this procedure and it is left up to the individual to acquaint themselves with this methodology and to refer to the manual during the following description of procedures if the reader is not familiar with them.

The WLB Group, Inc. created a Lotus 1-2-3 worksheet to help reduce the amount of hand calculations involved in developing the input parameters for MCUHP2. The FCDMC has recently updated this worksheet and now includes it with the new Hydrologic Design Manual for use by its consultants. The following steps were utilized within the worksheet to calculate basin lag time and average Green-Ampt loss rate parameters within each subbasin.

1. Measure flow path length and calculate elevation difference. This may be broken down into incremental elements representing areas of the same hydrologic properties and basin slopes.

2. The representative slope is then calculated according to the following formulas:

$$I = (Li^3 + Hi)^{.5}, \text{ where } i = 1, 2, 3, \ldots n$$

and

L_1, L_2, L_3, etc. Incremental Lengths Along the Longest Flow Path, Miles

H_1, H_2, H_3, etc. Incremental Elevation Differences for Each Length, Feet

and representative slope is then calculated from:

$$\text{Avg. } S = (L + I)^2 \text{ ft/mi}$$

where

L = Total Length of the Longest Flow Path
I = Value From Previous Formula

This average slope formula will take into account differences within a watershed due to varying topographic situations and varying slopes. This formula was taken from the "Hydrology Manual for Engineering Design and Floodplain Management Within Pima County, Arizona". It should be noted that "I" and "S" are usually calculated in feet and feet/feet respectively. But for this study Li was computed in miles and, therefore, S is in feet/mile for use in the lag equation that follows.

3. The lag for each subbasin is then calculated based on a formula created by the U.S. Army Corps of Engineers (1974):

$$\text{Lag} = 1.2\,(L * Lca \div S^{1/2})^{0.38}$$

where

L = Length of Longest Watercourse, miles
Lca = Length Along Longest Watercourse, Measured Upstream to a Point Opposite the Center of the Area, miles
S = Overall Average Slope of Longest Watercourse Between Headwater and Collection Point, ft/mile

Note: To obtain the Lag (in hours) for any area, multiply the lag obtained from the formula by $\bar{n}/.050$ or $20\bar{n}$.

$\bar{n}$ = Visually Estimated Mean of the N (Manning's Formula) Values of all the Channels Within an Area

4. The land use classification is then chosen along with an estimated percentage of vegetative cover and percentage of impervious areas. If the impervious areas are noncontiguous and undeveloped, only 50% of that impervious area is used for calculation purposes as directed by the Flood Control District of Maricopa County.

Aerial photographs were used along with zoning maps to help classify areas of differing land uses. (See the attached 11" x 17" Current Land Use and Zoning Map.) The aerial photographs also helped to define the percentage of vegetative cover in an area. Field investigation, along with numerous photographs, also help document this procedure. (See Appendix D for typical photographs of the area.)

The soil moisture condition for the calculation of DTHETA, and the surface retention loss, IA, are based upon the land use type. For instance, irrigated agricultural land is assumed to be in a saturated condition with a corresponding surface retention loss of 0.50 inches, residential land is assumed to be in a normal moisture condition with a corresponding surface retention loss of 0.12 inches, and desert land is assumed to be in a dry condition with a corresponding surface retention loss of 0.35 inches. These parameters were directed by the Flood Control District of Maricopa County. Refer to the "Hydrologic Design Manual for Maricopa County" for a more indepth discussion of DTHETA.

The rate of hydraulic conductivity to bare ground hydraulic conductivity, CK, is also a function of the percent of vegetative cover. This value was calculated as an average value for each subbasin. Refer to Fig. 4.10 in the "Hydrologic Design Manual for Maricopa County" and to Appendix E, Volume 6 of 15 for examples of the parameter averaging.

5. The next step was to planimeter areas of distinct soil classification within each subbasin and input the percentage of area for each soil group into the worksheet. This was accomplished by using Soil Conservation Service soil survey maps created for Maricopa County. Subbasins were transposed on these maps and distinct soil classification areas were then planimetered. Each soil group has distinct values associated with it for calculation of the Green-Ampt loss rate parameters. These parameters are then averaged based upon the percentage of different soil classifications within each subbasin. Refer to Appendix E, Volume 6 of 15, to see how parameter averaging is performed. The following 11" x 17" Hydrologic Soil Group Map shows locations of various types of hydrologic soil groups within the study area.

6. The average loss rate values, along with basin area and lag time, are then used as input into the FCDMC's computer program MCUHP2 to calculate a unit hydrograph for the HEC-1 model. This was done for each subbasin within the watershed; the corresponding S-graph Parameter sheets for each subbasin are included under separate cover in Appendix E. This appendix also includes a copy of the Soil Loss Rate Tables used in this study. A copy of the MCUHP2 input data as backup documentation to verify that the data was input correctly is located in Appendix F under separate cover.

3.2.2.2 *Channel Routing*: Channel routing throughout the watershed was accomplished by using the normal depth (modified Puls) routing procedure as outlined in HEC-1. This method utilizes an eight point typical cross section along with an average channel slope, channel length and typical Manning's n-values. The 1" = 400', 2-foot contour interval topographic mapping was incorporated to determine typical cross sections and channel geometry.

Two iterations of the HEC-1 model were run to calculate velocities in each routing reach. Initially, velocities were assumed for each routing reach within the watershed. After this initial model had been run, normal depth computations were performed to estimate velocity for each routing reach utilizing the computed discharges. The velocity estimates were based on a trapezoidal channel shape with an average Manning's n-value for the cross section. The resulting velocity estimates were then used to compute the number of steps for each channel routing reach. The number of steps was set equal to (reach length ÷ (average velocity x time interval)). The second iteration of the HEC-1 model was then run to produce the final discharges used in this study. Channel routing parameters are located in Appendix G and Velocity Calculations are located in Appendix H. Both of these appendices are under separate cover.

3.2.2.3 *Stage-Storage Discharge Parameters*: Stage-storage-discharge tables were created to model the numerous ponding areas located throughout the watershed. These areas are typically comprised of ponding behind structures such as dams, roadway embankments, railroad embankments or canal banks. Outfalls from these ponding areas include culverts, bridges, and weir flow over the top of the embankment. A list of existing drainage structures is located in Appendix I under separate cover and can also be found in the HEC-1 input documentation.

Ponding areas were identified using the 1" = 400' topographic mapping. The stage-storage data was computed by planimetering areas between adjacent contours and computing average volumes associated with that area and depth.

Bureau of Public Roads culvert charts were incorporated to calculate outflow from ponding areas where appropriate. The weir flow equation was used when flow overtopped an embankment or overtopped a particular impoundment. Stage-Storage Discharge tables can be found in Appendix I under separate cover.

3.2.2.4 *Diversions*: Numerous diversion tables were also incorporated throughout the watershed. This was due to the fact that a majority of the watershed is fairly flat with no well defined channels to contain the runoff. Consequently, flooding in the study area is characterized by wide, shallow flow paths which are easily diverted along man-made obstructions, such as railroads and irrigation canals.

Agriculture is the predominant practice throughout this area and fields are separated by major mile, half-mile, and farm access roadways. These roadways, along with irrigation canals, tend to pond water at the southeastern corner of the fields. From this point, flows break over the intersection of the two roads and will either continue east at the capacity of that particular road, flow overland to the southeast spreading out into another agricultural field, or flow south at the capacity of that road. It is not uncommon to have a three-way split at these locations.

These types of diversions were calculated by taking a cross section upstream along the centerline of each major road and computing weir flow as it applies to each diversion.

A second type of diversion, using the same cross section method along the centerline of the road, was to model the flow with a normal depth calculation. This was used when weir flow was not applicable at an intersection.

The third type of diversion usually involved a culvert analysis. If an embankment was present and the culvert capacity was exceeded, a diversion would take place above a certain limiting elevation. This diversion was calculated using either weir flow or normal depth methodology depending on the situation.

Finally, the fourth type of diversion would take place at a canal bank. Diversions were calculated by weir flow if the flow was to cross over the top of the canal bank and continue downstream or by normal depth methods if the flow was diverted along the upstream bank of the canal. Diversion tables can be found in Appendix I, under separate cover, and the Drainage Area Map identifies where diversions take place in the watershed. Each diversion is distinctly labeled except for the diverts associated with subbasins 43 and 43-1 through 43-8 - where space limitations on the Drainage Area Map required their exclusion. Refer to the exhibit on the following page for an enlargement of this area.

3.2.2.5 *Hydrograph Combinations*: The HEC-1 model for the White Tanks/Agua Fria ADMS was set up so that the area associated with each hydrograph combination was directly input into the model. The criteria to be followed, as directed by the FCDMC, was to hand calculate the total area that would be contributing to any given concentration point. Diversions were assumed to be contributing the whole area to the next concentration point, therefore, the corresponding area assigned to each concentration point would correspond to the total area of all subbasins that drain, either partially or fully to that point. The calculated areas were checked thoroughly by the FCDMC and concurrence was reached for the areas submitted on the HEC-1 model. This procedure was undertaken because the HEC-1 model assigns an area of zero to the diverts and carries that area to the next concentration point. Because rainfall depth decreases with increases in drainage area, the zero area associated with the diverts would, in some instances, result in overestimating peak discharges.

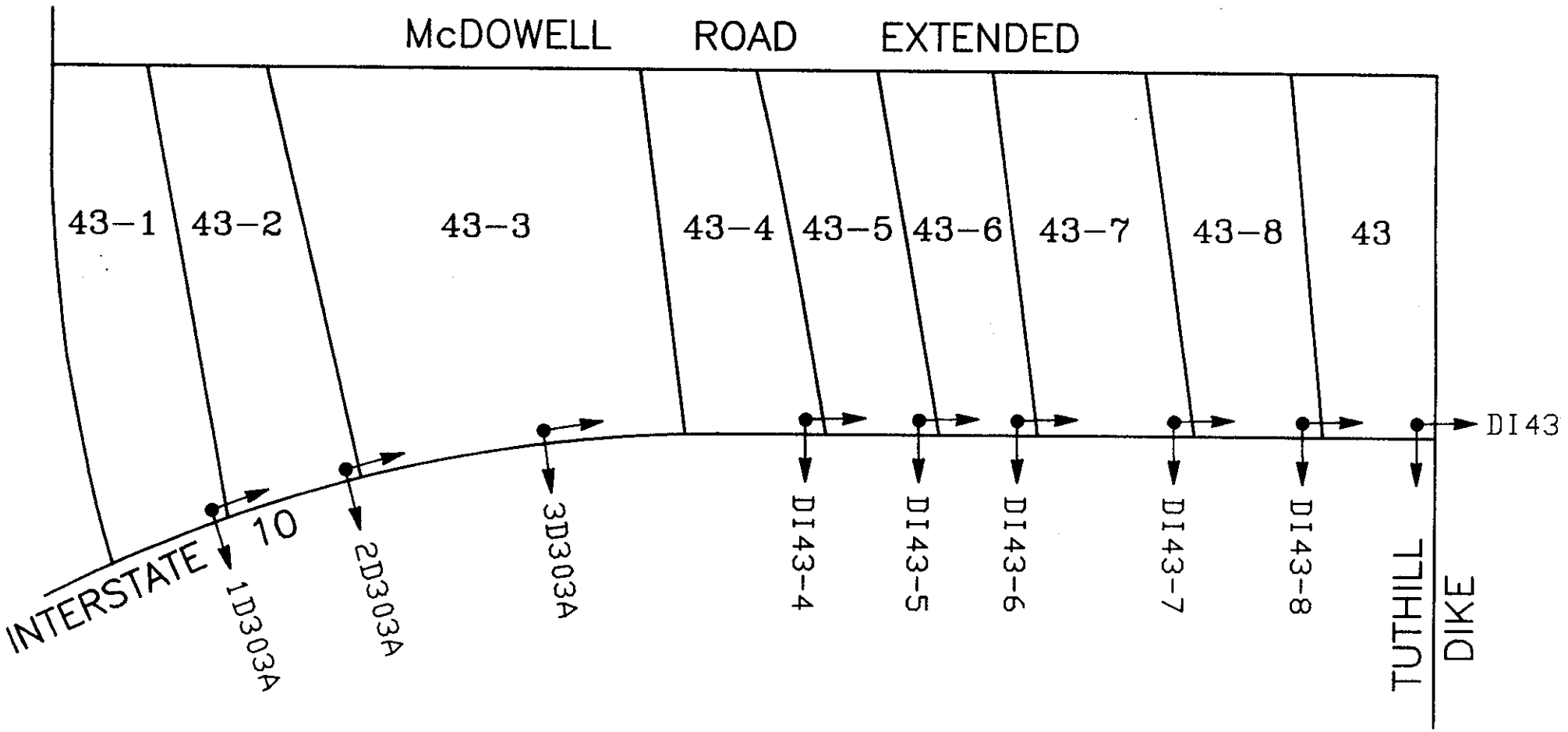
3.2.2.6 *Manning's N-Value Documentation*: Manning's n-value determinations for subbasins and routing reaches within the watershed were made based on field reconnaissance, aerial photographs, picture documentation, and sound engineering judgement. Typical "n" values were designated for agricultural areas, n = .12, and urban areas, n = .03, and these values were mutually agreed upon by The WLB Group and the FCDMC. Desert and mountainous areas have varying "n" values ranging from .03 to .20 and were incorporated based on the hydrologic conditions of that subbasin. Picture documentation of typical basin "n" values and channel and overbank "n" values are presented in Appendix D, under separate cover.

3.2.3 Statistical Parameters

No statistical analysis was performed with the White Tanks/Agua Fria ADMS as stream gage data is not available in this area.

It should be pointed out, however, that the Phoenix Valley S-graph used to compute the unit hydrographs is based on a statistical analysis of streamflow in and around Maricopa County (U.S. Army Corps of Engineer, 1974).

DIVERSIONS FOR SUBBASINS 43 - 43-8



WHITE TANKS / AGUA FRIA AREA DRAINAGE MASTER STUDY

N.T.S.

The WLB Group WLB

3.2.4 Precipitation

Precipitation data for the White Tanks/Agua Fria ADMS was developed from criteria as presented in the "Hydrologic Design Manual for Maricopa County". Initially, The WLB Group was instructed to use the 100-year, 6-hour storm to compute peak discharges. This, along with a new depth-area reduction curve designed for Maricopa County and 6-hour rainfall distribution patterns based upon drainage area, was incorporated into the 100-year model.

Sensitivity analyses were then run and tested against the 100-year, 24-hour storm. The 24-hour storm gave larger peak discharges as the area contributing to a watercourse increased. These discharges also increased uniformly downstream, whereas, the 6-hour storm did not.

The 6-hour storm produced larger peak flows for smaller watersheds (approximately .5 square miles or less), but, as the size of the area increased, the peak flows would, in some cases, decrease in a downstream direction. This was due to the sharp increase in rainfall intensity associated with the rainfall distribution patterns for small drainage areas. This discrepancy was the reason that the 100-year, 24-hour storm was chosen to model the watershed and to ultimately delineate the 100-year floodplains.

Precipitation amounts were developed for different return periods and frequency storms using the procedure stated in the "Precipitation-Frequency Atlas of the Western United States, Volume VIII - Arizona, NOAA Atlas 2," published by the National Weather Service's National Oceanic and Atmospheric Administration. This data is presented in Appendix A in the back of this report. Depth-area reduction of point rainfall was also taken from a graph in NOAA Atlas 2 since the 24-hour storm was used, and the Soil Conservation Service Type II rainfall distribution pattern was used to distribute the rainfall data accordingly.

3.2.5 Gage Data

No stream gages are located in the study area.

3.3 CALIBRATION

Due to the lack of stream gages or precipitation data in the study area, it is difficult to calibrate peak discharges computed in the HEC-1 model. However, a few previous studies have been performed on an isolated basis in different areas of the watershed. The new discharges were compared to the previous values to ascertain whether the results seemed reasonable. The reports and hydraulic analyses that WLB compared its results to are listed as follows:

1. "A Hydrologic Analysis of the White Tanks Flood Retarding Structures #3 and #4", by the Flood Control District of Maricopa County (FCD), October, 1989

INPUT PARAMETER COMPARISONS

Hydrologic Parameters	WLB	FCD
Storm Frequency and Duration	100-Year, 24-Hour	100-Year, 24-Hour
Rainfall Amount	4.03 In.	4.20 In.
Tabulation Interval	5-Minute	15-Minute
Loss Rate	Green-Ampt	SCS Curve Number
Distribution Pattern	SCS Type II	SCS Type II
Areal Distribution	NOAA Atlas II	None
Hydrograph Development	COE Phoenix Valley S-Graph	COE Phoenix Mountain S-Graph SCS Unit Hydrograph
Routing Method	Normal Depth	Normal Depth Kinematic Wave

COMPARISON OF DISCHARGES

Location	Discharges, CFS WLB	FCD
Inflow to White Tanks F.R.S. #3	6649	7640
Inflow to White Tanks F.R.S. #4	6026	5830

These discharges are reasonably close and the differences may be attributed to FCD's rainfall amount of 4.20 inches versus WLB's amount of 4.0 inches. Also, FCD used the SCS Curve Number Loss Rate while WLB incorporated the FCD's new methodology which incorporates Green-Ampt loss rate parameters. Also, a 15 minute time interval was used in the FCD study while a 5 minute time interval was utilized in this study.

2. "Conceptual Drainage Report for Litchfield Park Detention Facility", by Coe and Van Loo, June, 1989.

3. "Flow Estimation to Camelback and Dysart Roads", by Boyle Engineering Corporation, April, 1988.

4. "Hydrologic Evaluation, Litchfield Park Dam, Maricopa County, Arizona", by Dames & Moore, January 1986.

INPUT PARAMETER COMPARISONS

Hydrologic Parameters	WLB	CVL	Boyle	D & M
Storm Frequency & Duration	100/24	100/24	100/24	100/24
Rainfall Amount	4.03 In.	3.75 In.	3.77 In.	3.90 In.
Tabulation Interval	5-Min.	10-Min.	15-Min.	N/A
Loss Rate	Green-Ampt	SCS Curve	SCS Curve	SCS Curve
Distribution Pattern	SCS Type II	SCS Type II	SCS Type II	SCS Type II
Aerial Distribution	NOAA At. II	None	None	N/A
Hydrograph Development	COE Phx. Valley S-Gr	SCS Unit Hydrograph	SCS Unit Hydrograph	N/A
Routing Method	Norm. Depth	Kinematic	Kinematic	N/A

COMPARISON OF PEAK DISCHARGES

Location	WLB	CVL	Boyle	D & M
At Litchfield Park Detention Facility	959	769	525	1031
At Camelback and Dysart Road	1049	953	717	960

Again, these differences can be attributed to modeling techniques and WLB performed a HEC-2 analysis on Dysart Drain to better approximate the actual capacity of this facility and the corresponding breakout flows. Also, WLB had 1" = 400', 2-foot contour interval mapping to better estimate diversions and to delineate the watershed with greater precision.

5. "Conceptual Master Drainage Report for Litchfield Park Development Master Plan", by Coe & Van Loo, September 1989.

6. "Arizona Department of Transportation Interstate 10 Plans, Ehrenberg - Phoenix, Maricopa County I-10-2(34)," September 19, 1985.

INPUT PARAMETER COMPARISONS

Hydrologic Parameters	WLB	CVL	ADOT
Storm Frequency and Duration	100/24	100/6	100/3
Rainfall Amount	4.03 In.	3.15 In.	2.92 In.
Tabulation Interval	5-Minute	10-Minute	N/A
Loss Rate	Green-Ampt	SCS Curve #	SCS Curve #
Distribution Pattern	SCS Type II	SCS Type II	N/A
Areal Distribution	NOAA Atlas II	None	None
Hydrograph Development	COE Phx. Valley S-Graph	SCS Unit Hydrograph	SCS: Part II
Routing Method	Normal Depth	Kinematic Wave	N/A

COMPARISON OF DISCHARGES

Location	CVL	WLB	ADOT
At Reems Road & Northern Ave.	1001	2347	---
Divert E. at Reems Rd. & Northern Ave.	300	812	---
Remainder Flow to the S. at Reems Road and Northern Ave.	701	1536	---
At Camelback Road and Bullard Wash	2941	4243	---
At RID Canal and Bullard Wash	3585	4703	---
At Bullard Wash and I-10	*	5319 Upstream 4450 Downstream	5000 Upstream
At RID Canal and I-10	1347	826	

* Not Computed

The differences here are attributed to different storm durations and associated rainfall amounts, different subbasin divisions, a more intense scrutiny of diversions throughout the watershed, a HEC-2 analysis of Dysart Drain, and use of 1" = 400', 2-foot contour interval mapping over the entire watershed.

A number of sensitivity analyses were also performed to test the assumptions of hydrologic moisture condition and vegetation cover in the agricultural areas. Models were developed assuming fallow field (not planted) with the three different soil moisture conditions - saturated, normal and dry. These three moisture conditions were also used with a fully vegetated condition model. After reviewing these analyses, the FCDMC directed us to use the fully vegetated field in a saturated condition for agricultural areas in the watershed. It was understood that some areas would be fallow in a dry condition, vegetated in a normal or dry condition, etc., but the directed assumption gives an average condition without being too conservative or too under-conservative.

Also, an analysis was performed to determine if the numerous small agricultural reservoirs in the study area should be incorporated in the model. A typical agricultural reservoir was modeled and the results convinced the FCDMC that the storage would be filled during the early part of the storm before the peak arrived, therefore, these reservoirs would not be modeled. Another factor in the decision to not include the reservoirs is that there is no guarantee that they would not be filled in by the farmer or filled with sediment during the storm.

3.4 SPECIAL PROBLEMS/SOLUTIONS

The very nature of the watershed in the White Tanks/Agua Fria ADMS, with vastly differing hydrologic elements, tends to lead to modeling problems.

Initially, the watershed was separated into the following four distinct regions.

1. Watershed draining to White Tanks Flood Retarding Structure #3.
2. Watershed draining to White Tanks Flood Retarding Structure #4.
3. Watershed north of Dysart Drain and Northern Avenue.
4. Watershed south of Dysart Drain and Northern Avenue.

This was done to facilitate the FCDMC's review process and to allow the WLB Group to work on different regions while one was in for review.

This worked reasonably well as volumes of base data were generated in this study. The model was then joined together to create one complete hydrologic model of the entire watershed.

Two future conditions were assumed to be in place for the existing condition model. These assumptions were that the interim Estrella Freeway and Camelback Channel would be in place by the time the study was finished. The interim Estrella Freeway was assumed to collect flows along the west side of the roadway and pass these flows through at either at grade crossings or under the road in culvert crossings. For ease of modeling these were assumed to take place at major mile intersections although some flows may cross over or under at various locations between the intersections. The reason this assumption was made was based on the fact that these flows would eventually collect at the next major mile intersection to the southeast as overland flows naturally collect there now. This assumption was also used along the railroad at Cotton Lane.

The Colter Street Channel will be built by the Maricopa County Department of Transportation along an alignment of Coter Street which is approximately 1/4 mile north of Camelback Road. A Camelback Road alignment was assumed for this HEC-1 analysis which results in slightly larger flows, but does not compromise the integrity of the model. Flows will be collected in the channel from Litchfield Road and along inflow points to the east and are then conveyed to the Agua Fria River.

The Dysart Drain (also known as the Luke Air Force Base Drainage Channel) is located north and east of Luke Air Force Base and was modeled by a HEC-2 split flow analysis. Subsequent breakout flows were then incorporated into the HEC-1 model. Many iterations were required for this analysis to compute final diversion tables for the HEC-1 model.

To make the HEC-1 model a complete unit, it was necessary to route flows around the edge of the watershed in the Agua Fria River and Gila River. Since these are both very wide rivers, the assumption was made to route flows in a 1000 foot wide trapezoidal channel with representative Manning's n-values. The calculated flows are insignificant in comparison to the 100-year flow on the Agua Fria River and the Gila River.

As mentioned previously in this report, numerous diversions and ponding areas were modeled in the White Tank/Agua Fria ADMS. The procedures for modeling these areas are described in section 3.2.2. Of special note are the diversions located at the intersections of Olive Avenue and Beardsley Canal and Northern Avenue and Beardsley Canal. These diversions were modeled previously by the FCDMC in a report entitled "A Hydrologic Analysis of The White Tanks F.R.s #3 & #4". This data was incorporated in the HEC-1 model and into the subsequent HEC-2 analysis.

3.5 FINAL RESULTS/COMPUTER MODEL

The final results of the HEC-1 model are presented in numerical order in the Runoff Summary on the following pages. This is the same Runoff Summary generated by the HEC-1 model but it has been rearranged into numerical order for ease of locating discharges. Final output for the HEC-1 model is located in Appendix C, under separate cover, and another copy of the numerical Runoff Summary is included as well.

Four operations are shown in the Runoff Summary. These are respectively:

A) Runoff hydrographs for each subbasin.
B) Intermediate and final concentration points for combined hydrographs.
C) Diversion hydrographs.
D) Storage routing routines through reservoirs or ponding areas.

Routed flow discharges and returned diversion flows are not shown in this table. The HEC-1 output should be referred to if these discharges are required.

A note about the naming sequence of different operations in the HEC-1 model. Runoff hydrographs are designated as a number, combinations of numbers, or combinations of numbers and letters, ie, 41, 41-1, 41A1.

Final concentration points have the designation CP followed by the watershed number where that particular concentration point is located. Intermediate concentration points are designated as I1CP or 1I, 2I, etc; again, followed by the subbasin number. Concentration points combined in the Agua Fria or Gila River are designated as RCP followed by the subbasin number. It should also be mentioned here that routings in the river reaches are designated as RR standing for river route.

Diversions are designated by D, DI, 1D, 2D, etc. Storage routing through ponding areas or reservoirs is designated by SR with the one exception being the storage routine behind WT#4 which was inadvertently called RS47. Otherwise, these naming schemes stay consistent throughout the model.

Due to the nature and differing hydrologic regions of the watershed, it is difficult to put the model together in a systematic order. The model, therefore, is very complex and difficult to follow. A HEC-1 Key Map was created that breaks out the order in which the model was created. Distinct groups of subbasins make up a hydrologic area that drains to a common concentration point. These areas are numbered and have a corresponding tab in the HEC-1 output hardcopy so that it is easier to identify certain areas within the model that are of particular interest. The key map is located in the front of Appendix C where the HEC-1 hardcopy is located.

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1	HYDROGRAPH AT	1	1342.	12.50	175.	50.	48.	1.94		
2	HYDROGRAPH AT	2	1174.	12.75	206.	58.	56.	1.82		
3	HYDROGRAPH AT	3	828.	12.33	83.	21.	20.	.81		
4	HYDROGRAPH AT	3A	296.	12.33	29.	7.	7.	.29		
5	HYDROGRAPH AT	4	339.	12.25	26.	7.	6.	.30		
6	HYDROGRAPH AT	5	716.	12.25	65.	18.	18.	.72		
7	HYDROGRAPH AT	6	591.	12.08	41.	12.	11.	.45		
8	HYDROGRAPH AT	7	390.	12.08	28.	8.	8.	.31		
9	HYDROGRAPH AT	8	704.	12.33	73.	21.	20.	.81		
10	HYDROGRAPH AT	9	1096.	12.42	127.	36.	35.	1.40		
11	HYDROGRAPH AT	10	1173.	12.75	201.	52.	50.	2.02		
12	HYDROGRAPH AT	11	1313.	12.50	165.	44.	43.	1.56		
13	HYDROGRAPH AT	12	1149.	12.58	156.	40.	39.	1.38		
14	HYDROGRAPH AT	13	1170.	12.42	137.	34.	33.	1.30		
15	HYDROGRAPH AT	14	1163.	12.33	130.	37.	35.	1.47		
16	HYDROGRAPH AT	15	1039.	12.42	117.	32.	30.	1.26		
17	HYDROGRAPH AT	16	1255.	12.42	155.	43.	41.	1.13		
18	HYDROGRAPH AT	17	929.	12.25	110.	27.	26.	1.07		
19	HYDROGRAPH AT	18	923.	12.17	73.	21.	20.	.81		
20	HYDROGRAPH AT	19	622.	12.42	71.	20.	19.	.79		
21	HYDROGRAPH AT	20	861.	12.33	97.	27.	26.	1.07		
22	HYDROGRAPH AT	21	688.	12.42	74.	20.	19.	.79		
23	HYDROGRAPH AT	22	525.	12.25	50.	14.	13.	.57		
24	HYDROGRAPH AT	22A	764.	12.25	69.	18.	17.	.50		
25	HYDROGRAPH AT	23	289.	12.08	18.	5.	5.	.16		
26	HYDROGRAPH AT	24	207.	12.25	18.	5.	4.	.14		
27	HYDROGRAPH AT	25	500.	12.33	50.	12.	12.	.46		
28	HYDROGRAPH AT	26	943.	12.50	115.	30.	29.	1.16		
29	HYDROGRAPH AT	27	999.	12.42	110.	28.	27.	1.00		
30	HYDROGRAPH AT	28	747.	12.50	89.	22.	22.	.86		
31	HYDROGRAPH AT	29	228.	12.25	18.	5.	4.	.22		
32	HYDROGRAPH AT	30	244.	12.33	21.	5.	5.	.28		
33	HYDROGRAPH AT	31	525.	12.50	63.	16.	15.	.71		
34	HYDROGRAPH AT	32	956.	12.42	117.	33.	32.	1.29		
35	HYDROGRAPH AT	33	643.	12.25	59.	17.	16.	.65		
36	HYDROGRAPH AT	34	361.	12.25	33.	9.	9.	.36		
37	HYDROGRAPH AT	35	400.	12.25	35.	10.	10.	.39		
38	HYDROGRAPH AT	36	193.	12.25	16.	4.	4.	.24		
39	HYDROGRAPH AT	37	672.	12.42	85.	24.	23.	.95		
40	HYDROGRAPH AT	38	715.	12.25	64.	17.	17.	.76		
41	HYDROGRAPH AT	39	588.	12.50	74.	18.	18.	.77		
42	HYDROGRAPH AT	40	525.	12.25	48.	13.	13.	.52		
43	HYDROGRAPH AT	41A1	48.	12.00	3.	1.	1.	.02		
44	HYDROGRAPH AT	41A2	60.	12.00	3.	1.	1.	.03		
45	HYDROGRAPH AT	41A3	69.	12.00	4.	1.	1.	.03		
46	HYDROGRAPH AT	41A	91.	12.08	5.	1.	1.	.05		
47	HYDROGRAPH AT	41-1	208.	12.17	14.	3.	3.	.15		
48	HYDROGRAPH AT	41-2	143.	12.17	9.	2.	2.	.10		
49	HYDROGRAPH AT	41	567.	12.42	64.	16.	16.	.58		
50	HYDROGRAPH AT	42	1029.	12.50	131.	33.	32.	1.18		
51	HYDROGRAPH AT	43-1	76.	12.08	5.	1.	1.	.04		
52	HYDROGRAPH AT	43-2	19.	12.00	1.	0.	0.	.01		
53	HYDROGRAPH AT	43-3	107.	12.00	6.	1.	1.	.05		
54	HYDROGRAPH AT	43-4	64.	12.00	3.	1.	1.	.03		
55	HYDROGRAPH AT	43-5	43.	12.00	2.	1.	1.	.02		
56	HYDROGRAPH AT	43-6	45.	12.00	2.	1.	1.	.02		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
57 HYDROGRAPH AT	43-7	45.	12.00	2.	1.	1.	.02		
58 HYDROGRAPH AT	43-8	23.	12.00	1.	0.	0.	.01		
59 HYDROGRAPH AT	43	23.	12.00	1.	0.	0.	.01		
60 HYDROGRAPH AT	44	300.	12.25	25.	6.	6.	.22		
61 HYDROGRAPH AT	45-1	143.	12.08	9.	2.	2.	.08		
62 HYDROGRAPH AT	45	401.	12.42	48.	12.	12.	.38		
63 HYDROGRAPH AT	46-1	184.	12.25	18.	4.	4.	.15		
64 HYDROGRAPH AT	46	651.	12.58	94.	24.	23.	.85		
65 HYDROGRAPH AT	WT3	413.	12.50	54.	14.	13.	.44		
66 HYDROGRAPH AT	WT4	997.	12.25	88.	22.	21.	.77		
67 HYDROGRAPH AT	100	283.	12.58	45.	11.	11.	.26		
68 HYDROGRAPH AT	100A	212.	12.50	31.	8.	7.	.18		
69 HYDROGRAPH AT	101	233.	12.25	22.	5.	5.	.16		
70 HYDROGRAPH AT	102	135.	12.42	17.	4.	4.	.10		
71 HYDROGRAPH AT	102A	525.	12.58	73.	18.	18.	.51		
72 HYDROGRAPH AT	103	286.	12.92	55.	14.	13.	.37		
73 HYDROGRAPH AT	104	236.	12.17	20.	5.	5.	.15		
74 HYDROGRAPH AT	105	354.	12.25	34.	9.	8.	.21		
75 HYDROGRAPH AT	106	871.	12.50	111.	28.	27.	.77		
76 HYDROGRAPH AT	107	398.	13.08	88.	22.	21.	.60		
77 HYDROGRAPH AT	108	478.	13.25	117.	29.	28.	.79		
78 HYDROGRAPH AT	109	536.	13.25	140.	35.	34.	.85		
79 HYDROGRAPH AT	110	270.	12.83	51.	13.	12.	.31		
80 HYDROGRAPH AT	111	443.	12.67	71.	18.	17.	.50		
81 HYDROGRAPH AT	112	534.	13.33	138.	34.	33.	.97		
82 HYDROGRAPH AT	113	431.	13.08	117.	29.	28.	.50		
83 HYDROGRAPH AT	113A	409.	13.08	106.	27.	26.	.50		
84 HYDROGRAPH AT	114	326.	13.00	84.	21.	20.	.38		
85 HYDROGRAPH AT	115	379.	13.08	89.	22.	22.	.49		
86 HYDROGRAPH AT	116	575.	13.50	166.	41.	40.	1.02		
87 HYDROGRAPH AT	117	335.	12.83	63.	17.	16.	.41		
88 HYDROGRAPH AT	117A	195.	12.83	38.	9.	9.	.21		
89 HYDROGRAPH AT	118	126.	12.83	23.	6.	6.	.15		
90 HYDROGRAPH AT	119	600.	13.17	143.	36.	34.	.86		
91 HYDROGRAPH AT	119A	356.	13.17	91.	23.	22.	.47		
92 HYDROGRAPH AT	120	397.	13.25	106.	27.	26.	.54		
93 HYDROGRAPH AT	121A	324.	12.92	60.	15.	14.	.50		
93 HYDROGRAPH AT	121	325.	12.92	60.	15.	14.	.50		
94 HYDROGRAPH AT	122	552.	13.33	146.	37.	35.	.89		
95 HYDROGRAPH AT	123	338.	13.00	74.	18.	18.	.44		
96 HYDROGRAPH AT	124	355.	13.33	97.	24.	23.	.57		
97 HYDROGRAPH AT	125	1044.	12.50	134.	33.	32.	1.00		
98 HYDROGRAPH AT	126	562.	13.33	146.	36.	35.	.95		
99 HYDROGRAPH AT	127	469.	12.08	42.	12.	11.	.22		
100 HYDROGRAPH AT	128	312.	12.92	59.	15.	14.	.41		
101 HYDROGRAPH AT	129	378.	12.67	61.	16.	15.	.43		
102 HYDROGRAPH AT	130	647.	13.25	172.	43.	41.	1.00		
103 HYDROGRAPH AT	131	355.	13.08	79.	20.	19.	.49		
104 HYDROGRAPH AT	131A	354.	13.08	79.	20.	19.	.49		
105 HYDROGRAPH AT	132	271.	13.00	56.	14.	13.	.41		
106 HYDROGRAPH AT	133	328.	13.08	71.	18.	17.	.49		
107 HYDROGRAPH AT	134	334.	13.08	73.	18.	18.	.50		
108 HYDROGRAPH AT	135	315.	13.17	72.	18.	17.	.49		
109 HYDROGRAPH AT	136	315.	13.08	69.	17.	17.	.46		
110 HYDROGRAPH AT	137	307.	13.42	85.	21.	20.	.54		
111 HYDROGRAPH AT	138	587.	13.33	155.	39.	37.	1.00		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
112 HYDROGRAPH AT	139	338.	13.08	77.	19.	19.	.47		
113 HYDROGRAPH AT	140	194.	12.67	32.	8.	8.	.18		
114 HYDROGRAPH AT	141	460.	12.33	47.	12.	11.	.47		
115 HYDROGRAPH AT	141A	202.	12.17	14.	4.	3.	.14		
116 HYDROGRAPH AT	142	351.	13.00	74.	18.	18.	.51		
117 HYDROGRAPH AT	143	354.	13.00	75.	19.	18.	.50		
118 HYDROGRAPH AT	144	351.	13.00	74.	18.	18.	.51		
119 HYDROGRAPH AT	145	328.	13.08	72.	18.	17.	.48		
120 HYDROGRAPH AT	145A	327.	13.08	73.	18.	18.	.49		
121 HYDROGRAPH AT	146	548.	13.17	130.	33.	31.	.90		
122 HYDROGRAPH AT	147	342.	13.00	72.	18.	17.	.50		
123 HYDROGRAPH AT	148	328.	13.00	71.	18.	17.	.48		
124 HYDROGRAPH AT	149	312.	13.08	68.	17.	16.	.48		
125 HYDROGRAPH AT	150	193.	12.75	33.	8.	8.	.23		
126 HYDROGRAPH AT	151	208.	12.75	36.	9.	9.	.25		
127 HYDROGRAPH AT	152	284.	12.92	58.	15.	14.	.35		
128 HYDROGRAPH AT	153	112.	13.33	30.	7.	7.	.20		
129 HYDROGRAPH AT	154	171.	12.58	26.	7.	6.	.17		
130 HYDROGRAPH AT	155	250.	12.75	47.	12.	11.	.26		
131 HYDROGRAPH AT	156	252.	12.75	46.	12.	12.	.30		
132 HYDROGRAPH AT	156A	508.	12.17	46.	13.	13.	.31		
133 HYDROGRAPH AT	157	946.	12.58	150.	37.	36.	.89		
134 HYDROGRAPH AT	158	494.	13.08	108.	29.	28.	.97		
135 HYDROGRAPH AT	158A	114.	13.25	26.	6.	6.	.38		
136 HYDROGRAPH AT	158B	483.	13.00	104.	28.	27.	.67		
137 HYDROGRAPH AT	158C	105.	12.58	17.	4.	4.	.09		
138 HYDROGRAPH AT	158D	560.	12.25	55.	15.	15.	.37		
139 HYDROGRAPH AT	158E	767.	12.17	70.	20.	19.	.45		
140 HYDROGRAPH AT	159	531.	12.33	51.	13.	12.	.58		
141 HYDROGRAPH AT	160	432.	12.42	47.	12.	11.	.39		
142 HYDROGRAPH AT	161	294.	13.00	58.	14.	14.	.50		
143 HYDROGRAPH AT	162	268.	12.50	34.	8.	8.	.25		
144 HYDROGRAPH AT	163	551.	12.92	112.	28.	27.	.75		
145 HYDROGRAPH AT	164	363.	13.08	81.	20.	20.	.49		
146 HYDROGRAPH AT	164A	365.	13.08	82.	20.	20.	.49		
147 HYDROGRAPH AT	165	548.	13.25	133.	33.	32.	.90		
148 HYDROGRAPH AT	166	533.	13.33	142.	36.	34.	.98		
149 HYDROGRAPH AT	167	528.	13.33	141.	35.	34.	.97		
150 HYDROGRAPH AT	168	340.	13.17	79.	20.	19.	.51		
151 HYDROGRAPH AT	169	368.	13.17	89.	22.	21.	.51		
152 HYDROGRAPH AT	170	301.	12.67	53.	14.	13.	.29		
153 HYDROGRAPH AT	171	409.	13.42	115.	29.	28.	.70		
154 HYDROGRAPH AT	172	122.	12.58	18.	5.	4.	.12		
155 HYDROGRAPH AT	173	198.	13.00	39.	10.	9.	.31		
156 HYDROGRAPH AT	173A	191.	12.75	34.	8.	8.	.20		
157 HYDROGRAPH AT	173B	88.	12.83	14.	4.	3.	.20		
158 HYDROGRAPH AT	174	612.	12.25	55.	14.	13.	.45		
159 HYDROGRAPH AT	175	375.	12.25	34.	9.	8.	.28		
160 HYDROGRAPH AT	175A	362.	12.50	42.	10.	10.	.47		
161 HYDROGRAPH AT	176	701.	12.42	79.	20.	19.	.67		
162 HYDROGRAPH AT	176A	378.	13.25	92.	23.	22.	.62		
163 HYDROGRAPH AT	177	336.	13.08	73.	18.	18.	.49		
164 HYDROGRAPH AT	177A	334.	13.08	73.	18.	17.	.49		
165 HYDROGRAPH AT	178	298.	13.08	64.	16.	15.	.44		
166 HYDROGRAPH AT	179	333.	13.08	76.	19.	18.	.46		
167 HYDROGRAPH AT	180	574.	13.33	154.	39.	37.	.99		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
168	HYDROGRAPH AT	181	283.	12.92	54.	14.	13.	.37		
169	HYDROGRAPH AT	181A	249.	13.17	57.	14.	14.	.39		
170	HYDROGRAPH AT	182	212.	12.75	35.	9.	9.	.24		
171	HYDROGRAPH AT	183	258.	12.42	31.	8.	8.	.21		
172	HYDROGRAPH AT	184	531.	13.17	124.	31.	30.	.79		
173	HYDROGRAPH AT	185	465.	13.08	103.	26.	25.	.69		
174	HYDROGRAPH AT	186	187.	13.25	48.	12.	12.	.33		
175	HYDROGRAPH AT	187	81.	12.00	5.	1.	1.	.35		
176	HYDROGRAPH AT	188	240.	12.25	19.	5.	5.	.19		
177	HYDROGRAPH AT	189	521.	12.42	57.	14.	14.	.51		
178	HYDROGRAPH AT	190	657.	12.92	128.	32.	31.	.86		
179	HYDROGRAPH AT	191	610.	13.25	149.	37.	36.	.99		
180	HYDROGRAPH AT	192	346.	13.08	75.	19.	18.	.50		
181	HYDROGRAPH AT	192A	345.	13.08	75.	19.	18.	.50		
182	HYDROGRAPH AT	193	545.	13.25	132.	33.	32.	.91		
183	HYDROGRAPH AT	194	535.	13.42	144.	36.	35.	.99		
184	HYDROGRAPH AT	195	256.	13.50	72.	18.	17.	.49		
185	HYDROGRAPH AT	196	313.	13.00	68.	17.	17.	.47		
186	HYDROGRAPH AT	197	524.	13.50	150.	38.	36.	1.00		
187	HYDROGRAPH AT	198	494.	13.42	140.	35.	34.	.91		
188	HYDROGRAPH AT	199	74.	13.08	16.	4.	4.	.11		
189	HYDROGRAPH AT	200	231.	12.67	37.	9.	9.	.29		
190	HYDROGRAPH AT	201	420.	12.33	44.	11.	11.	.34		
191	HYDROGRAPH AT	202	258.	13.42	73.	18.	18.	.48		
192	HYDROGRAPH AT	203	162.	12.25	14.	4.	3.	.11		
193	HYDROGRAPH AT	204	381.	12.25	32.	8.	8.	.27		
194	HYDROGRAPH AT	205	125.	12.08	8.	2.	2.	.06		
195	HYDROGRAPH AT	206	188.	12.17	14.	3.	3.	.12		
196	HYDROGRAPH AT	207	619.	13.25	153.	38.	37.	1.00		
197	HYDROGRAPH AT	207A	469.	12.42	48.	12.	12.	.50		
198	HYDROGRAPH AT	208	632.	13.17	149.	37.	36.	1.00		
199	HYDROGRAPH AT	209	336.	13.08	73.	18.	17.	.50		
200	HYDROGRAPH AT	209A	335.	13.08	75.	19.	18.	.50		
201	HYDROGRAPH AT	210	296.	13.08	64.	16.	16.	.46		
202	HYDROGRAPH AT	211	309.	13.08	69.	17.	17.	.49		
203	HYDROGRAPH AT	212	279.	13.42	77.	19.	19.	.54		
204	HYDROGRAPH AT	213	373.	12.50	46.	12.	11.	.35		
205	HYDROGRAPH AT	214	208.	12.25	18.	5.	4.	.16		
206	HYDROGRAPH AT	215	419.	12.33	44.	11.	11.	.35		
207	HYDROGRAPH AT	215A	497.	12.42	55.	14.	13.	.45		
208	HYDROGRAPH AT	216	366.	12.92	69.	17.	17.	.51		
209	HYDROGRAPH AT	217	350.	13.00	76.	19.	18.	.49		
210	HYDROGRAPH AT	218	624.	13.25	153.	38.	37.	1.00		
211	HYDROGRAPH AT	219	343.	13.08	76.	19.	18.	.50		
212	HYDROGRAPH AT	220	335.	13.08	75.	19.	18.	.50		
213	HYDROGRAPH AT	221	303.	13.08	69.	17.	17.	.48		
214	HYDROGRAPH AT	221A	175.	13.25	43.	11.	10.	.31		
215	HYDROGRAPH AT	222	541.	13.58	172.	48.	46.	1.10		
216	HYDROGRAPH AT	223	1763.	12.42	327.	110.	106.	1.26		
217	HYDROGRAPH AT	224	1054.	12.33	109.	27.	26.	.80		
218	HYDROGRAPH AT	225	460.	12.42	59.	15.	15.	.43		
219	HYDROGRAPH AT	225A	441.	12.42	50.	12.	12.	.37		
220	HYDROGRAPH AT	226	1573.	12.33	192.	54.	52.	1.18		
221	HYDROGRAPH AT	227	331.	12.25	30.	8.	7.	.23		
222	HYDROGRAPH AT	228	361.	12.33	36.	9.	9.	.28		
223	HYDROGRAPH AT	228A	125.	12.17	8.	2.	2.	.08		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
224	HYDROGRAPH AT	229	724.	12.17	61.	16.	15.	.51		
225	HYDROGRAPH AT	230	56.	12.33	6.	1.	1.	.04		
226	HYDROGRAPH AT	230A	292.	12.17	21.	5.	5.	.18		
227	HYDROGRAPH AT	231	265.	12.42	28.	7.	7.	.35		
228	HYDROGRAPH AT	232	1006.	12.42	109.	27.	26.	.93		
229	HYDROGRAPH AT	233	496.	12.42	56.	14.	13.	.50		
230	HYDROGRAPH AT	234	347.	13.17	81.	20.	19.	.53		
231	HYDROGRAPH AT	235	303.	13.17	71.	18.	17.	.47		
232	HYDROGRAPH AT	236	582.	13.25	146.	37.	35.	1.00		
233	HYDROGRAPH AT	237	300.	13.25	73.	18.	18.	.50		
234	HYDROGRAPH AT	238	314.	13.17	72.	18.	17.	.50		
235	HYDROGRAPH AT	239	313.	13.08	69.	17.	17.	.48		
236	HYDROGRAPH AT	240	282.	13.00	57.	14.	14.	.40		
237	HYDROGRAPH AT	241	1436.	12.50	176.	44.	42.	1.51		
238	HYDROGRAPH AT	242	965.	12.67	143.	36.	34.	1.14		
239	HYDROGRAPH AT	242A	161.	12.08	10.	3.	2.	.09		
240	HYDROGRAPH AT	243	298.	12.33	34.	8.	8.	.24		
241	HYDROGRAPH AT	243A	253.	12.42	31.	8.	8.	.22		
242	HYDROGRAPH AT	243B	64.	12.67	10.	3.	3.	.07		
243	HYDROGRAPH AT	244	200.	12.58	28.	7.	7.	.19		
244	HYDROGRAPH AT	244A	486.	12.17	36.	9.	9.	.31		
245	HYDROGRAPH AT	245	181.	13.17	41.	10.	10.	.40		
246	HYDROGRAPH AT	246	670.	12.50	84.	21.	20.	.75		
247	HYDROGRAPH AT	247	330.	13.08	76.	19.	18.	.50		
248	HYDROGRAPH AT	248	592.	13.25	152.	38.	37.	1.00		
249	HYDROGRAPH AT	249	571.	13.25	143.	36.	35.	1.00		
250	HYDROGRAPH AT	250	316.	13.08	71.	18.	17.	.49		
251	HYDROGRAPH AT	250A	308.	13.17	74.	18.	18.	.51		
252	HYDROGRAPH AT	251	313.	13.17	72.	18.	17.	.50		
253	HYDROGRAPH AT	252	326.	13.17	76.	19.	18.	.50		
254	HYDROGRAPH AT	253	641.	13.17	148.	37.	36.	1.00		
255	HYDROGRAPH AT	253A	291.	12.33	29.	7.	7.	.25		
256	HYDROGRAPH AT	254	556.	12.58	81.	20.	20.	.59		
257	HYDROGRAPH AT	254A	443.	12.08	40.	11.	11.	.22		
258	HYDROGRAPH AT	255	1512.	12.25	152.	41.	40.	.94		
259	HYDROGRAPH AT	256	326.	13.00	72.	18.	17.	.43		
260	HYDROGRAPH AT	257	217.	13.42	62.	16.	15.	.34		
261	HYDROGRAPH AT	258	334.	12.42	37.	9.	9.	.38		
262	HYDROGRAPH AT	258A	150.	12.25	13.	3.	3.	.12		
263	HYDROGRAPH AT	259	242.	12.25	18.	5.	4.	.20		
264	HYDROGRAPH AT	260	536.	12.33	56.	15.	14.	.50		
265	HYDROGRAPH AT	261	264.	13.17	62.	16.	15.	.41		
266	HYDROGRAPH AT	262	616.	13.25	149.	37.	36.	1.03		
267	HYDROGRAPH AT	263	328.	13.08	72.	18.	17.	.50		
268	HYDROGRAPH AT	264	337.	13.00	70.	18.	17.	.50		
269	HYDROGRAPH AT	265	118.	13.33	31.	8.	8.	.22		
270	HYDROGRAPH AT	265A	627.	13.08	137.	34.	33.	.95		
271	HYDROGRAPH AT	266	181.	13.33	48.	12.	12.	.33		
272	HYDROGRAPH AT	267	335.	13.17	79.	20.	19.	.50		
273	HYDROGRAPH AT	268	589.	13.25	147.	37.	35.	.95		
274	HYDROGRAPH AT	269	491.	13.00	101.	25.	24.	.66		
275	HYDROGRAPH AT	270	446.	12.25	49.	14.	13.	.30		
276	HYDROGRAPH AT	271	377.	13.25	96.	24.	23.	.57		
277	HYDROGRAPH AT	271A	95.	13.33	25.	6.	6.	.16		
278	HYDROGRAPH AT	271B	109.	12.83	20.	5.	5.	.15		
279	HYDROGRAPH AT	271C	263.	12.17	22.	5.	5.	.18		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
280 HYDROGRAPH AT	272	231.	12.25	18.	4.	4.	.20		
281 HYDROGRAPH AT	273	671.	12.42	75.	19.	18.	.65		
282 HYDROGRAPH AT	274	329.	13.50	94.	23.	23.	.66		
283 HYDROGRAPH AT	275	136.	12.42	17.	4.	4.	.11		
284 HYDROGRAPH AT	276	187.	13.00	38.	10.	9.	.26		
284 HYDROGRAPH AT	281	487.	13.25	122.	31.	29.	.85		
285 HYDROGRAPH AT	277	570.	13.08	127.	32.	31.	.91		
286 HYDROGRAPH AT	278	537.	13.33	138.	35.	34.	1.00		
287 HYDROGRAPH AT	279	79.	12.25	7.	2.	2.	.05		
288 HYDROGRAPH AT	279A	89.	12.33	10.	3.	2.	.07		
289 HYDROGRAPH AT	279B	46.	12.25	4.	1.	1.	.03		
290 HYDROGRAPH AT	279C	78.	12.25	8.	2.	2.	.05		
291 HYDROGRAPH AT	279D	33.	12.17	3.	1.	1.	.02		
292 HYDROGRAPH AT	280	340.	13.33	89.	22.	21.	.61		
293 HYDROGRAPH AT	280A	53.	12.75	9.	2.	2.	.06		
295 HYDROGRAPH AT	282	161.	12.33	17.	4.	4.	.12		
296 HYDROGRAPH AT	283	102.	13.08	23.	6.	6.	.16		
297 HYDROGRAPH AT	284	326.	13.17	76.	19.	18.	.52		
298 HYDROGRAPH AT	285	56.	12.33	6.	1.	1.	.04		
299 HYDROGRAPH AT	285A	82.	12.33	9.	2.	2.	.06		
300 HYDROGRAPH AT	285B	83.	12.33	9.	2.	2.	.06		
301 HYDROGRAPH AT	286	473.	13.08	109.	27.	26.	.70		
302 HYDROGRAPH AT	287	223.	12.67	36.	9.	9.	.23		
303 HYDROGRAPH AT	287A	331.	12.83	68.	17.	16.	.34		
304 HYDROGRAPH AT	287B	127.	12.50	18.	4.	4.	.10		
305 HYDROGRAPH AT	287C	270.	12.67	57.	14.	14.	.23		
306 HYDROGRAPH AT	287D	239.	12.67	39.	10.	9.	.23		
307 HYDROGRAPH AT	287E	194.	12.58	28.	7.	7.	.20		
308 HYDROGRAPH AT	288	160.	13.25	44.	11.	10.	.22		
309 HYDROGRAPH AT	288A	607.	13.33	188.	47.	46.	.83		
310 HYDROGRAPH AT	288B	179.	13.42	54.	14.	13.	.26		
311 HYDROGRAPH AT	289	519.	13.83	189.	48.	46.	1.00		
312 HYDROGRAPH AT	289A	326.	13.17	77.	19.	18.	.50		
313 HYDROGRAPH AT	290	413.	12.67	64.	16.	15.	.55		
314 HYDROGRAPH AT	291	454.	13.25	112.	28.	27.	.99		
315 HYDROGRAPH AT	292	1004.	12.50	131.	35.	33.	.96		
316 HYDROGRAPH AT	293	679.	12.67	113.	29.	28.	.77		
317 HYDROGRAPH AT	293A	76.	12.50	10.	3.	2.	.07		
318 HYDROGRAPH AT	294	274.	12.17	24.	6.	6.	.18		
319 HYDROGRAPH AT	294A	368.	12.25	34.	8.	8.	.26		
320 HYDROGRAPH AT	295	419.	12.25	41.	10.	10.	.31		
321 HYDROGRAPH AT	295A	114.	12.08	8.	2.	2.	.06		
322 HYDROGRAPH AT	296	565.	13.00	117.	29.	28.	.81		
323 HYDROGRAPH AT	297	240.	13.17	55.	14.	13.	.38		
324 HYDROGRAPH AT	297A	206.	12.92	39.	10.	9.	.27		
325 HYDROGRAPH AT	298	591.	13.00	122.	30.	29.	.84		
326 HYDROGRAPH AT	299	289.	13.00	61.	15.	15.	.40		
327 HYDROGRAPH AT	300	353.	12.83	66.	16.	16.	.39		
328 HYDROGRAPH AT	301	535.	12.17	46.	12.	12.	.30		
329 HYDROGRAPH AT	302	242.	12.17	21.	5.	5.	.15		
330 HYDROGRAPH AT	303	910.	12.67	138.	35.	33.	1.21		
331 HYDROGRAPH AT	303A	531.	12.25	49.	12.	12.	.42		
332 HYDROGRAPH AT	304	1221.	12.33	128.	32.	31.	1.10		
333 HYDROGRAPH AT	305	666.	13.00	141.	35.	34.	.92		
334 HYDROGRAPH AT	306	367.	12.92	71.	18.	17.	.49		
335 HYDROGRAPH AT	307	219.	12.58	30.	8.	7.	.21		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
336 HYDROGRAPH AT	308	270.	12.75	46.	12.	11.	.32		
337 HYDROGRAPH AT	309	666.	13.17	157.	39.	38.	1.07		
338 HYDROGRAPH AT	310	183.	12.58	28.	7.	7.	.19		
339 HYDROGRAPH AT	311	420.	13.33	107.	27.	26.	.73		
340 HYDROGRAPH AT	311A	241.	12.92	46.	12.	11.	.31		
341 HYDROGRAPH AT	312	423.	13.17	97.	24.	23.	.66		
342 HYDROGRAPH AT	313	258.	13.17	62.	16.	15.	.43		
343 HYDROGRAPH AT	314	293.	13.00	61.	15.	15.	.42		
344 HYDROGRAPH AT	315	278.	13.25	68.	17.	16.	.47		
345 HYDROGRAPH AT	316	512.	13.17	122.	31.	29.	.82		
346 HYDROGRAPH AT	317	341.	13.25	82.	21.	20.	.57		
347 HYDROGRAPH AT	318	581.	12.67	97.	27.	26.	.62		
348 HYDROGRAPH AT	319	577.	12.50	84.	23.	23.	.54		
349 HYDROGRAPH AT	320	677.	12.42	88.	25.	24.	.64		
350 HYDROGRAPH AT	321	942.	12.17	74.	18.	18.	.66		
351 HYDROGRAPH AT	322	261.	12.92	51.	13.	12.	.34		
352 HYDROGRAPH AT	323	181.	12.33	18.	5.	4.	.15		
353 HYDROGRAPH AT	324	279.	12.83	53.	13.	13.	.36		
354 HYDROGRAPH AT	325	294.	13.25	74.	18.	18.	.49		
355 HYDROGRAPH AT	325A	404.	13.08	88.	22.	21.	.61		
356 HYDROGRAPH AT	326	373.	13.00	79.	20.	19.	.54		
357 HYDROGRAPH AT	327	373.	13.00	77.	19.	18.	.53		
358 HYDROGRAPH AT	328	521.	12.92	103.	26.	25.	.76		
359 HYDROGRAPH AT	329	496.	12.83	89.	23.	22.	.63		
360 HYDROGRAPH AT	330	467.	12.83	87.	23.	22.	.59		
361 HYDROGRAPH AT	331	489.	13.08	109.	28.	27.	.75		
362 HYDROGRAPH AT	332	353.	13.17	84.	21.	21.	.56		
363 HYDROGRAPH AT	333	388.	13.08	84.	21.	20.	.58		
364 HYDROGRAPH AT	334	452.	13.00	95.	24.	23.	.64		
365 HYDROGRAPH AT	335	277.	12.92	54.	13.	13.	.35		
366 HYDROGRAPH AT	336	552.	13.92	206.	57.	55.	1.28		
367 HYDROGRAPH AT	336A	149.	14.00	54.	14.	13.	.37		
368 HYDROGRAPH AT	336B	168.	12.00	9.	2.	2.	.08		
369 HYDROGRAPH AT	337	744.	12.25	84.	25.	24.	.49		
370 HYDROGRAPH AT	338	260.	12.58	38.	9.	9.	.31		
371 HYDROGRAPH AT	338A	736.	12.42	87.	24.	23.	.77		
372 HYDROGRAPH AT	339	625.	13.17	150.	38.	36.	1.00		
373 HYDROGRAPH AT	340	348.	13.00	72.	18.	17.	.48		
374 HYDROGRAPH AT	341	506.	13.08	113.	28.	27.	.79		
375 HYDROGRAPH AT	342	327.	12.83	60.	15.	14.	.39		
376 HYDROGRAPH AT	342A	240.	13.17	56.	14.	13.	.38		
377 HYDROGRAPH AT	343	323.	13.17	74.	19.	18.	.51		
378 HYDROGRAPH AT	344	343.	13.33	88.	22.	21.	.61		
379 HYDROGRAPH AT	345	296.	13.00	63.	16.	15.	.43		
380 HYDROGRAPH AT	346	403.	12.75	68.	17.	16.	.51		
381 HYDROGRAPH AT	346A	110.	13.00	23.	6.	6.	.16		
382 HYDROGRAPH AT	346B	159.	13.25	39.	10.	9.	.27		
383 HYDROGRAPH AT	346C	135.	12.50	17.	4.	4.	.12		
384 HYDROGRAPH AT	347	671.	13.08	148.	38.	36.	1.02		
385 HYDROGRAPH AT	348	283.	12.75	50.	12.	12.	.34		
386 HYDROGRAPH AT	348A	229.	12.58	32.	8.	8.	.22		
387 HYDROGRAPH AT	348B	355.	13.33	92.	23.	22.	.66		
388 HYDROGRAPH AT	349	654.	13.00	137.	34.	33.	.93		
389 HYDROGRAPH AT	350	106.	13.33	27.	7.	7.	.19		
390 HYDROGRAPH AT	351	607.	12.83	113.	28.	27.	.76		
391 HYDROGRAPH AT	352	150.	12.75	26.	7.	6.	.18		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
392	HYDROGRAPH AT	352A	89.	13.08	20.	5.	5.	.13		
393	HYDROGRAPH AT	353	183.	12.75	32.	8.	8.	.22		
394	HYDROGRAPH AT	354	315.	12.58	45.	11.	11.	.31		
395	HYDROGRAPH AT	355	85.	12.92	18.	4.	4.	.10		
396	HYDROGRAPH AT	355A	46.	12.50	6.	1.	1.	.04		
397	HYDROGRAPH AT	356	346.	12.58	51.	13.	12.	.35		
398	HYDROGRAPH AT	357	132.	12.67	20.	5.	5.	.14		
399	HYDROGRAPH AT	358	119.	13.17	29.	7.	7.	.19		
400	HYDROGRAPH AT	359	104.	12.58	14.	4.	3.	.10		
401	HYDROGRAPH AT	360	168.	13.00	36.	9.	9.	.25		
402	HYDROGRAPH AT	361	138.	13.17	33.	8.	8.	.21		
403	HYDROGRAPH AT	362	339.	12.83	63.	16.	15.	.42		
404	HYDROGRAPH AT	363	360.	13.25	91.	23.	22.	.63		
405	HYDROGRAPH AT	364	559.	13.00	115.	29.	28.	.78		
406	HYDROGRAPH AT	364A	99.	12.25	9.	2.	2.	.07		
407	HYDROGRAPH AT	365	272.	12.83	47.	12.	11.	.38		
408	HYDROGRAPH AT	366	289.	13.00	61.	15.	15.	.44		
409	HYDROGRAPH AT	367	240.	12.08	16.	4.	4.	.15		
410	HYDROGRAPH AT	368	367.	13.58	111.	28.	27.	.83		
411	HYDROGRAPH AT	369	133.	12.25	11.	3.	3.	.15		
412	HYDROGRAPH AT	370	61.	12.33	5.	1.	1.	.07		
413	HYDROGRAPH AT	371	487.	13.67	154.	39.	37.	.93		
414	HYDROGRAPH AT	372	796.	13.75	262.	66.	63.	1.62		
415	HYDROGRAPH AT	373	386.	12.75	68.	17.	16.	.43		
416	HYDROGRAPH AT	374	459.	13.50	137.	34.	33.	.76		
417	HYDROGRAPH AT	375	262.	13.42	74.	19.	18.	.41		
418	HYDROGRAPH AT	376	344.	13.00	75.	19.	18.	.42		
419	HYDROGRAPH AT	377	120.	12.42	15.	4.	4.	.09		
420	HYDROGRAPH AT	377A	188.	13.00	39.	10.	9.	.27		
421	HYDROGRAPH AT	377B	58.	13.58	18.	5.	4.	.12		
422	HYDROGRAPH AT	378	349.	13.42	95.	24.	23.	.78		
422	HYDROGRAPH AT	379	220.	13.58	67.	17.	16.	.46		
423	HYDROGRAPH AT	380	188.	12.83	37.	9.	9.	.20		
424	HYDROGRAPH AT	381	184.	12.92	39.	10.	9.	.21		
425	HYDROGRAPH AT	381A	115.	12.58	17.	4.	4.	.10		
426	HYDROGRAPH AT	381B	67.	12.75	12.	3.	3.	.07		
427	HYDROGRAPH AT	382	539.	13.17	136.	34.	33.	.71		
428	HYDROGRAPH AT	383	154.	12.92	31.	8.	8.	.18		
429	HYDROGRAPH AT	383A	122.	13.00	27.	7.	7.	.16		
430	HYDROGRAPH AT	384	262.	12.83	55.	14.	13.	.29		
431	HYDROGRAPH AT	385	298.	13.17	77.	19.	19.	.39		
432	HYDROGRAPH AT	386	222.	13.25	57.	14.	14.	.35		
433	HYDROGRAPH AT	387	205.	13.00	44.	11.	11.	.25		
434	2 COMBINED AT	CP2	2284.	12.75	370.	100.	96.	3.76		
435	2 COMBINED AT	I1CP3	997.	12.42	110.	28.	27.	1.10		
436	2 COMBINED AT	CP3	2245.	12.92	468.	119.	115.	4.86		
437	2 COMBINED AT	CP5	1053.	12.25	90.	25.	24.	1.02		
438	2 COMBINED AT	I1CP7	1289.	12.17	130.	35.	34.	1.47		
439	2 COMBINED AT	CP7	1668.	12.17	158.	43.	41.	1.78		
440	2 COMBINED AT	I1CP9	2527.	12.33	277.	74.	71.	3.18		
441	2 COMBINED AT	CP9	3227.	12.33	350.	94.	91.	3.99		
442	2 COMBINED AT	I1CP10	3816.	12.75	525.	133.	129.	6.01		
443	2 COMBINED AT	CP10	5141.	12.75	911.	230.	222.	10.87		
444	2 COMBINED AT	CP12	4125.	12.83	861.	217.	209.	12.25		
445	2 COMBINED AT	CP13	1743.	12.58	301.	78.	76.	2.86		
446	2 COMBINED AT	I1CP15	1920.	12.50	246.	68.	66.	2.73		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
447	2 COMBINED AT	CP15	1920.	12.50	246.	68.	66.	3.86		
448	2 COMBINED AT	I1CP17	1738.	12.75	321.	80.	77.	4.93		
449	2 COMBINED AT	CP17	3428.	12.75	621.	158.	153.	7.79		
450	2 COMBINED AT	I1CWT3	3605.	12.83	668.	167.	161.	8.23		
451	2 COMBINED AT	CPWT3	6649.	12.92	1450.	362.	349.	20.48		
452	2 COMBINED AT	CP19	1353.	12.33	144.	40.	39.	1.60		
453	2 COMBINED AT	I1CP21	1933.	12.42	217.	60.	58.	2.39		
454	2 COMBINED AT	CP21	1933.	12.42	217.	60.	58.	3.46		
455	2 COMBINED AT	I1C22A	1111.	12.33	120.	31.	30.	1.07		
456	2 COMBINED AT	CP22A	1108.	12.33	119.	31.	30.	4.69		
457	2 COMBINED AT	CP23	288.	12.08	55.	17.	16.	3.62		
458	2 COMBINED AT	I1CP25	1414.	12.42	169.	44.	42.	5.15		
459	2 COMBINED AT	CP25	1414.	12.42	169.	44.	42.	5.29		
460	2 COMBINED AT	I1CP27	1648.	12.50	225.	57.	55.	2.16		
461	2 COMBINED AT	CP27	3011.	12.50	350.	88.	85.	7.45		
462	2 COMBINED AT	I1CP30	879.	12.50	110.	28.	27.	1.14		
463	2 COMBINED AT	CP30	879.	12.50	110.	28.	27.	1.36		
464	2 COMBINED AT	CP31	1258.	12.58	173.	43.	42.	2.07		
465	2 COMBINED AT	CP33	1003.	12.25	91.	26.	25.	1.01		
466	2 COMBINED AT	I1CP35	1297.	12.25	127.	36.	35.	1.40		
467	2 COMBINED AT	CP35	2155.	12.33	243.	69.	67.	2.69		
468	2 COMBINED AT	I1CP36	2217.	12.42	258.	73.	70.	2.93		
469	2 COMBINED AT	CP36	2886.	12.42	343.	97.	93.	3.88		
470	2 COMBINED AT	CP38	3253.	12.42	407.	113.	109.	4.64		
471	2 COMBINED AT	I1CP39	3708.	12.58	427.	112.	108.	5.41		
472	2 COMBINED AT	CP39	6110.	12.67	765.	197.	190.	12.86		
473	2 COMBINED AT	CP41A2	60.	12.00	3.	1.	1.	.05		
474	2 COMBINED AT	CP41A3	86.	12.08	5.	1.	1.	.08		
475	2 COMBINED AT	CP41A	91.	12.08	5.	1.	1.	.13		
476	2 COMBINED AT	CP41-1	208.	12.17	14.	3.	3.	.28		
477	2 COMBINED AT	CP41-2	143.	12.17	12.	3.	3.	.38		
478	2 COMBINED AT	CP41	567.	12.42	64.	16.	16.	.68		
479	2 COMBINED AT	I1CP42	1454.	12.50	179.	46.	44.	1.70		
480	2 COMBINED AT	CP42	7140.	12.67	934.	241.	232.	14.56		
481	2 COMBINED AT	CP43-1	465.	12.58	42.	11.	10.	1.00		
482	2 COMBINED AT	CP43-2	19.	12.00	1.	0.	0.	1.01		
483	2 COMBINED AT	CP43-3	107.	12.00	6.	1.	1.	1.06		
484	2 COMBINED AT	CP43-4	64.	12.00	3.	1.	1.	1.09		
485	2 COMBINED AT	CP43-5	43.	12.00	2.	1.	1.	1.11		
486	2 COMBINED AT	CP43-6	45.	12.00	2.	1.	1.	1.13		
487	2 COMBINED AT	CP43-7	45.	12.00	2.	1.	1.	1.15		
488	2 COMBINED AT	CP43-8	23.	12.00	1.	0.	0.	1.16		
489	2 COMBINED AT	I1CP43	22.	12.00	1.	0.	0.	1.17		
490	2 COMBINED AT	CP43	6786.	12.83	799.	206.	198.	13.90		
491	2 COMBINED AT	CP45-1	1440.	12.92	90.	22.	22.	13.98		
492	2 COMBINED AT	CP45	1030.	13.08	101.	25.	24.	14.36		
493	2 COMBINED AT	CP46-1	316.	13.25	33.	8.	8.	14.51		
494	2 COMBINED AT	I1CP46	637.	12.58	92.	23.	22.	15.36		
495	2 COMBINED AT	CP46	1737.	12.67	259.	65.	63.	17.43		
496	2 COMBINED AT	I1CWT4	27.	12.67	5.	1.	1.	.05		
497	2 COMBINED AT	I2CWT4	29.	12.67	8.	2.	2.	.07		
498	2 COMBINED AT	I3CWT4	41.	12.58	10.	3.	2.	.09		
499	2 COMBINED AT	I4CWT4	46.	12.58	11.	3.	3.	.10		
500	2 COMBINED AT	I5CWT4	323.	12.25	36.	9.	9.	.32		
501	2 COMBINED AT	I6CWT4	4057.	13.00	747.	195.	187.	14.12		
502	2 COMBINED AT	I7CWT4	552.	13.25	120.	30.	29.	14.36		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
503 2 COMBINED AT	I8CWT4	618.	13.25	153.	38.	37.	14.51		
504 2 COMBINED AT	I9CWT4	1048.	12.25	237.	60.	57.	15.28		
505 2 COMBINED AT	I10WT4	2009.	13.00	491.	123.	119.	18.20		
506 2 COMBINED AT	CPWT4	6026.	13.00	1228.	316.	304.	18.57		
507 2 COMBINED AT	CP100	283.	12.58	70.	19.	18.	.44		
508 2 COMBINED AT	CP101	635.	12.50	95.	24.	23.	.67		
509 2 COMBINED AT	CP102	753.	12.50	112.	28.	27.	.77		
510 2 COMBINED AT	CP107	1044.	12.75	199.	50.	48.	1.37		
511 2 COMBINED AT	1I108	999.	12.58	228.	57.	55.	1.56		
512 2 COMBINED AT	CP108	1158.	12.75	283.	71.	68.	1.93		
513 2 COMBINED AT	CP109	535.	13.25	184.	53.	51.	1.29		
514 2 COMBINED AT	1I112	534.	13.33	157.	40.	38.	1.12		
515 2 COMBINED AT	2I112	765.	13.33	202.	51.	49.	1.62		
516 2 COMBINED AT	CP112	790.	13.33	236.	59.	57.	1.83		
517 2 COMBINED AT	CP113A	560.	13.08	183.	46.	45.	2.33		
518 2 COMBINED AT	CP113	556.	13.08	284.	73.	71.	2.83		
519 2 COMBINED AT	CP114	376.	13.08	140.	36.	35.	3.21		
520 2 COMBINED AT	1I115	1313.	13.17	288.	72.	69.	1.86		
521 2 COMBINED AT	2I115	2429.	13.17	570.	143.	137.	3.79		
522 2 COMBINED AT	CP115	2705.	13.17	688.	173.	167.	7.00		
523 2 COMBINED AT	CP116	892.	13.75	331.	93.	90.	2.31		
524 2 COMBINED AT	1I117	359.	13.33	114.	30.	29.	.72		
525 2 COMBINED AT	2I117	1104.	13.83	439.	122.	118.	3.03		
526 2 COMBINED AT	CP117	1172.	13.75	476.	131.	127.	3.24		
527 2 COMBINED AT	CP119	706.	13.17	169.	42.	41.	1.36		
528 2 COMBINED AT	CP119A	533.	13.25	161.	40.	39.	1.83		
529 2 COMBINED AT	1I120	895.	13.33	266.	67.	64.	2.37		
530 2 COMBINED AT	CP120	1056.	13.33	342.	86.	83.	4.20		
531 2 COMBINED AT	1I121A	465.	12.92	137.	35.	33.	2.33		
532 2 COMBINED AT	CP121A	465.	12.92	146.	37.	36.	2.83		
533 2 COMBINED AT	CP121	708.	13.00	205.	52.	50.	3.33		
534 3 COMBINED AT	1I122	571.	14.33	357.	99.	96.	4.10		
535 2 COMBINED AT	CP122	2976.	13.58	1041.	273.	263.	7.89		
536 2 COMBINED AT	CP124	665.	13.33	171.	43.	41.	1.01		
537 2 COMBINED AT	1I125	1148.	14.08	598.	164.	158.	4.24		
538 2 COMBINED AT	CP125	1318.	14.08	654.	178.	171.	5.25		
539 2 COMBINED AT	1I126	563.	13.33	169.	42.	41.	1.10		
540 2 COMBINED AT	CP126	856.	13.33	381.	101.	98.	6.35		
541 2 COMBINED AT	CP128	537.	13.25	158.	39.	38.	1.77		
542 3 COMBINED AT	1I130	715.	14.42	323.	82.	79.	3.24		
543 3 COMBINED AT	CP130	1703.	13.42	724.	184.	178.	5.54		
544 2 COMBINED AT	CP131A	1102.	13.58	553.	143.	138.	6.03		
545 2 COMBINED AT	1I131	1274.	13.58	621.	163.	157.	6.52		
546 2 COMBINED AT	CP131	1909.	13.50	822.	215.	207.	8.52		
547 2 COMBINED AT	CP132	460.	13.58	208.	56.	54.	8.93		
548 2 COMBINED AT	1I133	579.	13.42	270.	74.	71.	9.42		
549 2 COMBINED AT	CP133	725.	14.33	354.	101.	97.	14.48		
550 2 COMBINED AT	1I134	2655.	14.00	980.	262.	252.	8.39		
551 2 COMBINED AT	2I134	2655.	14.00	980.	262.	252.	8.83		
552 2 COMBINED AT	CP134	2847.	14.00	1106.	298.	287.	15.42		
553 2 COMBINED AT	1I135	885.	14.17	391.	112.	107.	15.63		
554 2 COMBINED AT	CP135	970.	14.08	449.	129.	124.	16.48		
555 2 COMBINED AT	1I136	315.	13.08	139.	39.	37.	1.47		
556 2 COMBINED AT	CP136	931.	14.92	523.	154.	148.	16.94		
557 2 COMBINED AT	1I137	907.	13.50	590.	174.	167.	17.12		
558 2 COMBINED AT	CP137	1306.	14.50	795.	230.	221.	21.72		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
559 2 COMBINED AT	1I138	586.	13.33	312.	93.	90.	6.25		
560 2 COMBINED AT	2I138	613.	13.42	385.	113.	109.	7.35		
561 2 COMBINED AT	CP138	934.	15.58	596.	184.	177.	23.82		
562 2 COMBINED AT	CP138	1329.	15.50	812.	261.	252.	24.02		
563 2 COMBINED AT	1I139	521.	13.33	119.	31.	30.	.69		
564 2 COMBINED AT	2I139	832.	13.83	399.	111.	107.	7.04		
565 2 COMBINED AT	CP139	843.	13.83	429.	119.	115.	7.22		
566 2 COMBINED AT	CP141A	540.	12.33	61.	15.	15.	.62		
567 2 COMBINED AT	CP142	692.	12.50	134.	34.	32.	1.13		
568 2 COMBINED AT	CP143	992.	13.00	209.	52.	50.	1.63		
569 2 COMBINED AT	1I144	858.	13.83	316.	79.	76.	6.05		
570 2 COMBINED AT	CP144	1398.	13.67	524.	131.	126.	7.68		
571 2 COMBINED AT	CP145A	1467.	13.25	563.	141.	136.	8.17		
572 2 COMBINED AT	1I145	1590.	14.00	708.	190.	183.	9.00		
573 2 COMBINED AT	CP145	2946.	14.00	1263.	330.	318.	11.63		
574 2 COMBINED AT	1I146	629.	13.25	259.	72.	69.	15.02		
575 2 COMBINED AT	CP146	1441.	14.08	813.	229.	221.	18.49		
576 2 COMBINED AT	1I147	337.	13.00	118.	36.	35.	14.62		
577 2 COMBINED AT	CP147	2010.	14.42	818.	229.	220.	15.92		
578 2 COMBINED AT	CP148	323.	13.00	134.	35.	34.	17.46		
578 2 COMBINED AT	1I148	323.	13.00	85.	22.	21.	16.40		
579 2 COMBINED AT	CP149	533.	13.58	316.	93.	89.	22.20		
580 2 COMBINED AT	CP150	196.	12.75	51.	13.	13.	17.69		
581 2 COMBINED AT	1I151	312.	12.92	85.	22.	21.	17.94		
582 2 COMBINED AT	CP151	638.	13.75	388.	114.	110.	23.66		
583 2 COMBINED AT	CP152	320.	13.25	170.	48.	46.	24.01		
584 2 COMBINED AT	CP153	423.	16.17	246.	78.	75.	21.92		
585 2 COMBINED AT	CP154	845.	16.33	529.	175.	168.	24.19		
586 2 COMBINED AT	CP155	243.	12.75	45.	11.	11.	24.27		
587 2 COMBINED AT	CP156	829.	14.25	462.	130.	125.	7.52		
588 2 COMBINED AT	1I157	919.	12.58	526.	204.	196.	25.08		
589 2 COMBINED AT	2I157	1258.	17.17	798.	289.	279.	25.08		
590 2 COMBINED AT	CP157	1771.	17.00	1228.	411.	396.	26.25		
591 2 COMBINED AT	1I158	497.	13.08	148.	42.	40.	1.28		
592 2 COMBINED AT	CP158	557.	14.17	219.	62.	60.	2.42		
593 2 COMBINED AT	RC158A	578.	13.25	125.	35.	34.	1.96		
593 2 COMBINED AT	CP158A	577.	13.17	145.	39.	37.	1.14		
593 2 COMBINED AT	CP158B	483.	13.00	119.	33.	31.	.76		
593 2 COMBINED AT	CP160	955.	12.42	98.	25.	24.	.97		
593 2 COMBINED AT	RC158D	1149.	12.33	125.	35.	34.	.82		
594 2 COMBINED AT	CP161	1032.	12.58	156.	39.	38.	1.47		
595 2 COMBINED AT	1I163	726.	13.00	146.	36.	35.	1.00		
596 2 COMBINED AT	CP163	1575.	13.00	301.	75.	72.	2.47		
597 2 COMBINED AT	1I164A	364.	13.08	104.	28.	27.	8.17		
598 2 COMBINED AT	CP164A	1865.	13.17	403.	103.	99.	10.64		
599 2 COMBINED AT	1I164	2141.	13.25	480.	123.	119.	11.13		
600 2 COMBINED AT	CP164	2141.	13.25	790.	202.	194.	15.08		
601 2 COMBINED AT	1I165	858.	15.17	434.	124.	119.	12.53		
602 2 COMBINED AT	2I165	2031.	13.67	1106.	298.	287.	15.98		
603 2 COMBINED AT	CP165	3187.	15.08	1882.	520.	501.	22.84		
604 2 COMBINED AT	1I166	1837.	15.58	799.	253.	243.	16.90		
605 2 COMBINED AT	CP166	1975.	15.58	902.	281.	271.	18.44		
606 2 COMBINED AT	1I167	589.	13.42	263.	78.	76.	24.63		
607 2 COMBINED AT	CP167	1870.	15.92	1044.	336.	324.	25.61		
608 2 COMBINED AT	1I168	330.	13.17	144.	50.	49.	24.17		
609 2 COMBINED AT	2I168	330.	13.17	188.	64.	62.	26.12		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
610 2 COMBINED AT	CP168	515.	13.50	351.	112.	108.	26.47		
611 2 COMBINED AT	1I169	357.	13.17	86.	22.	21.	24.52		
612 2 COMBINED AT	2I169	359.	13.83	131.	33.	32.	25.13		
613 2 COMBINED AT	CP169	491.	13.83	201.	58.	56.	27.24		
614 2 COMBINED AT	CP172	1768.	17.08	1227.	413.	398.	26.37		
615 2 COMBINED AT	CP173	1753.	17.42	1224.	419.	404.	26.68		
616 2 COMBINED AT	RCP173	1826.	17.33	1460.	512.	493.	30.12		
617 2 COMBINED AT	CP173B	496.	14.58	229.	65.	63.	2.62		
618 2 COMBINED AT	RC173B	880.	14.58	354.	99.	96.	3.44		
619 2 COMBINED AT	CP173A	421.	13.58	148.	37.	36.	.90		
620 2 COMBINED AT	RC173A	1822.	17.42	1563.	545.	525.	31.02		
621 2 COMBINED AT	CP175A	687.	12.42	76.	19.	18.	.75		
622 2 COMBINED AT	1I176	700.	12.42	79.	20.	19.	.95		
623 2 COMBINED AT	CP176	792.	12.50	134.	34.	32.	1.40		
624 2 COMBINED AT	CP176A	378.	13.25	92.	23.	22.	3.09		
625 2 COMBINED AT	CP177A	581.	13.17	165.	41.	40.	3.58		
626 2 COMBINED AT	1I177	814.	13.33	237.	59.	57.	4.07		
627 2 COMBINED AT	CP177	902.	13.75	321.	82.	79.	16.68		
628 2 COMBINED AT	CP178	993.	13.42	382.	97.	94.	17.12		
629 2 COMBINED AT	1I179	1156.	13.58	448.	116.	111.	17.58		
630 2 COMBINED AT	CP179	3923.	14.17	2252.	630.	606.	25.34		
631 2 COMBINED AT	1I180	1796.	14.33	1228.	384.	370.	26.33		
632 2 COMBINED AT	CP180	1797.	14.33	1286.	401.	386.	30.29		
633 2 COMBINED AT	CP181	292.	14.75	221.	73.	70.	30.66		
633 2 COMBINED AT	CP181A	1200.	16.25	766.	265.	255.	26.00		
634 2 COMBINED AT	1I182	204.	12.75	34.	8.	8.	30.90		
635 2 COMBINED AT	CP182	903.	16.58	611.	221.	213.	38.46		
636 2 COMBINED AT	1I183	527.	16.42	253.	77.	74.	25.82		
637 2 COMBINED AT	2I183	797.	16.42	395.	124.	120.	26.21		
638 2 COMBINED AT	CP183	1078.	16.42	654.	209.	201.	27.07		
639 2 COMBINED AT	1I184	1051.	17.50	656.	231.	222.	27.86		
640 2 COMBINED AT	2I184	1467.	17.50	944.	332.	319.	40.32		
641 2 COMBINED AT	CP184	1574.	17.50	1104.	388.	373.	41.09		
642 2 COMBINED AT	1I185	487.	13.58	154.	39.	38.	.98		
643 2 COMBINED AT	CP185	552.	13.67	312.	113.	109.	42.07		
644 2 COMBINED AT	CP187	173.	13.67	52.	13.	13.	.68		
645 2 COMBINED AT	RCP187	1801.	17.75	1570.	549.	529.	31.70		
646 2 COMBINED AT	CP188	1457.	12.83	212.	53.	51.	11.08		
647 2 COMBINED AT	1I189	650.	12.67	135.	34.	33.	5.61		
648 2 COMBINED AT	CP189	940.	12.58	192.	48.	46.	6.12		
649 2 COMBINED AT	CP190	1336.	13.08	319.	80.	77.	6.98		
650 3 COMBINED AT	1I191	1774.	13.25	451.	113.	109.	10.07		
651 2 COMBINED AT	CP191	2358.	13.25	593.	149.	144.	12.18		
657 2 COMBINED AT	CP192A	1140.	13.50	177.	44.	43.	12.68		
658 2 COMBINED AT	CP192	1241.	13.75	251.	63.	60.	13.18		
659 2 COMBINED AT	1I193	596.	13.25	156.	39.	38.	14.09		
660 2 COMBINED AT	CP193	2347.	14.50	1229.	312.	301.	36.34		
661 2 COMBINED AT	1I194	514.	13.42	138.	35.	33.	31.19		
662 2 COMBINED AT	CP194	875.	13.50	740.	197.	189.	37.33		
663 2 COMBINED AT	1I195	1482.	15.58	1056.	336.	324.	35.64		
664 3 COMBINED AT	CP195	2559.	15.50	1800.	602.	580.	43.14		
665 2 COMBINED AT	1I196	439.	17.42	313.	124.	119.	38.93		
666 2 COMBINED AT	CP196	945.	17.42	806.	349.	336.	51.75		
667 2 COMBINED AT	CP197	611.	18.42	457.	182.	175.	42.09		
668 2 COMBINED AT	1I198	609.	19.17	440.	169.	163.	42.00		
669 2 COMBINED AT	2I198	846.	19.25	693.	271.	261.	42.98		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
670 2 COMBINED AT	CP198	1418.	19.17	1137.	450.	433.	44.09		
671 2 COMBINED AT	CP200	231.	12.67	53.	13.	13.	.40		
672 2 COMBINED AT	RCP200	1786.	18.08	1550.	552.	532.	32.10		
672 2 COMBINED AT	CP201	2025.	15.67	1298.	385.	371.	43.48		
673 2 COMBINED AT	CP202	565.	13.42	388.	181.	174.	52.23		
674 2 COMBINED AT	CP203	598.	18.92	456.	179.	172.	42.20		
675 2 COMBINED AT	1I204	1413.	19.25	1136.	453.	436.	44.36		
676 2 COMBINED AT	CP204	1721.	19.25	1453.	626.	603.	58.13		
677 2 COMBINED AT	CP205	1721.	19.33	1453.	627.	604.	58.19		
678 2 COMBINED AT	CP206	1722.	19.33	1453.	629.	606.	58.31		
679 2 COMBINED AT	RCP206	3249.	16.67	2901.	1156.	1114.	68.69		
680 2 COMBINED AT	CP207A	1450.	13.00	259.	65.	62.	11.58		
681 2 COMBINED AT	1I208	1921.	13.25	631.	160.	154.	13.18		
682 2 COMBINED AT	CP208	2175.	13.33	746.	188.	182.	14.18		
683 2 COMBINED AT	CP209A	1706.	13.58	519.	131.	126.	14.68		
684 2 COMBINED AT	1I209	1026.	14.08	295.	74.	71.	13.68		
685 2 COMBINED AT	CP209	1285.	14.08	373.	93.	90.	16.18		
686 2 COMBINED AT	1I210	292.	13.08	64.	16.	15.	13.64		
687 2 COMBINED AT	CP210	304.	13.08	123.	31.	30.	16.58		
688 2 COMBINED AT	1I212	1344.	15.50	636.	168.	162.	36.88		
689 2 COMBINED AT	CP212	1350.	15.58	658.	172.	166.	37.23		
690 2 COMBINED AT	CP214	1392.	13.08	276.	69.	67.	11.74		
691 2 COMBINED AT	1I215	874.	12.42	99.	25.	24.	.80		
692 2 COMBINED AT	CP215	1484.	12.42	373.	93.	90.	12.54		
693 2 COMBINED AT	CP217	372.	13.08	111.	28.	27.	1.49		
694 2 COMBINED AT	1I218	614.	13.25	152.	38.	37.	15.18		
695 2 COMBINED AT	CP218	669.	13.25	189.	47.	46.	15.67		
696 2 COMBINED AT	1I219	629.	14.75	321.	92.	89.	14.68		
697 2 COMBINED AT	2I219	1239.	14.67	553.	151.	145.	15.18		
698 2 COMBINED AT	CP219	1270.	14.58	565.	154.	148.	16.67		
699 2 COMBINED AT	1I220	650.	14.42	276.	70.	68.	15.18		
700 2 COMBINED AT	2I220	1115.	14.50	415.	106.	102.	16.68		
701 2 COMBINED AT	CP220	1241.	14.58	476.	121.	117.	18.17		
702 2 COMBINED AT	1I221	490.	15.33	202.	59.	56.	16.66		
703 2 COMBINED AT	2I221	707.	15.25	323.	89.	86.	17.06		
704 2 COMBINED AT	1I221A	276.	14.17	109.	28.	27.	.80		
704 2 COMBINED AT	CP221	1018.	15.17	450.	122.	118.	19.11		
705 2 COMBINED AT	CP221A	1377.	16.17	713.	194.	187.	38.03		
706 2 COMBINED AT	CP222	1380.	16.50	769.	238.	229.	39.13		
707 2 COMBINED AT	1I223	2054.	16.33	1344.	488.	471.	44.74		
708 2 COMBINED AT	CP223	2523.	16.33	1806.	690.	665.	54.67		
709 2 COMBINED AT	CP224	2453.	16.50	1745.	655.	631.	55.47		
710 2 COMBINED AT	1I225	520.	12.42	119.	66.	64.	.43		
711 2 COMBINED AT	CP225	959.	12.42	168.	79.	76.	.80		
712 2 COMBINED AT	1I226	1483.	12.33	180.	51.	49.	53.41		
713 2 COMBINED AT	CP226	1483.	12.33	572.	233.	224.	53.41		
714 2 COMBINED AT	RC228A	3236.	16.83	2897.	1142.	1100.	68.77		
715 3 COMBINED AT	CP229	863.	12.33	127.	32.	31.	1.02		
716 2 COMBINED AT	CP230A	530.	12.25	95.	24.	23.	1.20		
717 2 COMBINED AT	1I230	227.	12.42	33.	8.	8.	1.06		
718 2 COMBINED AT	CP230	672.	12.42	114.	29.	28.	1.24		
719 2 COMBINED AT	CP231	279.	12.50	53.	13.	13.	1.37		
720 2 COMBINED AT	CP233	1685.	12.58	427.	107.	103.	13.04		
721 2 COMBINED AT	1I234	689.	13.08	150.	37.	36.	1.04		
722 2 COMBINED AT	CP234	2084.	12.83	572.	143.	138.	14.08		
723 2 COMBINED AT	1I235	483.	13.42	144.	36.	35.	1.96		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
724 2 COMBINED AT	CP235	1467.	13.42	486.	122.	117.	16.04		
725 2 COMBINED AT	1I236	710.	13.33	187.	47.	45.	16.67		
726 2 COMBINED AT	CP236	1222.	13.58	425.	108.	104.	26.34		
727 2 COMBINED AT	1I237	513.	13.92	195.	51.	49.	16.17		
728 2 COMBINED AT	2I237	519.	13.92	275.	74.	72.	17.17		
729 2 COMBINED AT	CP237	993.	13.83	500.	132.	128.	27.84		
730 2 COMBINED AT	1I238	843.	15.17	453.	130.	126.	17.17		
731 2 COMBINED AT	2I238	918.	15.25	477.	137.	132.	18.67		
732 2 COMBINED AT	CP238	1226.	15.25	751.	213.	205.	30.34		
733 2 COMBINED AT	1I239	701.	15.75	319.	96.	93.	18.65		
734 2 COMBINED AT	2I239	1547.	16.25	757.	217.	209.	19.59		
735 2 COMBINED AT	CP239	2367.	16.17	1410.	406.	391.	31.76		
736 2 COMBINED AT	CP240	634.	16.50	552.	191.	184.	32.16		
737 2 COMBINED AT	1I241	2429.	16.92	1743.	680.	655.	56.98		
738 2 COMBINED AT	2I241	3693.	17.08	2453.	907.	874.	59.77		
739 2 COMBINED AT	CP241	4243.	17.08	2889.	1083.	1044.	78.75		
740 2 COMBINED AT	1I242	1000.	12.67	213.	80.	77.	1.94		
741 2 COMBINED AT	CP242	1049.	12.58	223.	83.	80.	2.03		
742 2 COMBINED AT	CP243B	1111.	12.67	233.	85.	82.	2.10		
743 2 COMBINED AT	CP243A	1261.	12.67	263.	93.	89.	2.32		
744 2 COMBINED AT	CP244A	485.	12.17	49.	13.	12.	1.51		
745 2 COMBINED AT	1I243	338.	12.42	63.	16.	16.	1.98		
746 2 COMBINED AT	CP243	1460.	12.67	324.	108.	104.	4.30		
747 2 COMBINED AT	1I244	582.	12.42	78.	20.	19.	1.70		
748 2 COMBINED AT	CP244	1195.	12.50	192.	48.	47.	1.74		
749 3 COMBINED AT	1I245	1056.	13.25	252.	64.	62.	2.14		
750 2 COMBINED AT	CP245	1860.	13.17	574.	171.	164.	5.05		
751 2 COMBINED AT	RCP245	3300.	17.58	2987.	1219.	1174.	73.82		
752 2 COMBINED AT	CP246	1239.	12.58	193.	48.	46.	1.68		
754 2 COMBINED AT	1I248	1313.	13.42	372.	93.	90.	15.08		
755 2 COMBINED AT	2I248	1602.	13.50	446.	112.	108.	15.58		
756 2 COMBINED AT	CP248	2174.	13.67	679.	170.	164.	17.54		
757 2 COMBINED AT	1I249	559.	13.25	188.	47.	45.	18.54		
758 2 COMBINED AT	CP249	694.	14.00	262.	66.	63.	28.84		
759 2 COMBINED AT	1I250	308.	16.08	145.	45.	43.	26.83		
760 2 COMBINED AT	2I250	411.	16.08	231.	67.	64.	28.33		
761 2 COMBINED AT	CP250	909.	14.42	466.	126.	121.	30.83		
762 2 COMBINED AT	1I250A	314.	14.50	180.	46.	44.	328.35		
763 2 COMBINED AT	2I250A	366.	14.58	214.	55.	53.	30.85		
764 2 COMBINED AT	CP250A	1254.	14.58	675.	180.	173.	33.84		
765 2 COMBINED AT	1I251	301.	13.17	87.	30.	29.	30.84		
766 2 COMBINED AT	2I251	1711.	16.50	937.	258.	249.	32.26		
767 2 COMBINED AT	CP251	2394.	16.50	1543.	424.	408.	35.76		
768 2 COMBINED AT	CP252	439.	16.83	325.	96.	92.	36.26		
769 2 COMBINED AT	1I253	4123.	17.67	2883.	1105.	1064.	79.75		
770 2 COMBINED AT	2I253	4122.	17.67	2883.	1111.	1070.	80.00		
771 2 COMBINED AT	CP253	4121.	17.67	2888.	1114.	1073.	84.50		
772 2 COMBINED AT	CP255	1512.	12.25	183.	50.	48.	1.84		
773 2 COMBINED AT	CP256	325.	13.00	95.	24.	23.	.90		
774 2 COMBINED AT	RCP258	3295.	17.75	2986.	1193.	1149.	74.20		
775 2 COMBINED AT	CP259	243.	12.25	20.	5.	5.	.32		
776 2 COMBINED AT	CP260	1336.	12.75	249.	63.	61.	2.18		
777 2 COMBINED AT	CP261	264.	13.17	62.	16.	15.	.91		
778 2 COMBINED AT	CP262	2219.	13.83	770.	194.	187.	18.57		
779 2 COMBINED AT	CP263	1071.	14.08	423.	108.	104.	19.07		
780 2 COMBINED AT	1I264	324.	13.00	91.	23.	22.	29.34		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
781	2 COMBINED AT	CP264	412.	14.33	212.	54.	52.	30.87		
782	2 COMBINED AT	1I265A	601.	13.08	132.	33.	32.	34.79		
783	2 COMBINED AT	CP265A	604.	13.08	161.	40.	39.	35.01		
784	2 COMBINED AT	1I266	256.	15.42	93.	23.	23.	34.17		
785	2 COMBINED AT	CP266	1964.	16.75	1341.	369.	355.	36.09		
786	2 COMBINED AT	1I267	485.	13.67	353.	109.	105.	36.76		
787	2 COMBINED AT	CP267	2106.	16.92	1534.	434.	418.	37.09		
788	2 COMBINED AT	1I268	4088.	18.00	2880.	1128.	1086.	85.45		
789	2 COMBINED AT	CP268	4703.	17.92	3270.	1269.	1222.	86.28		
790	2 COMBINED AT	CP269	491.	13.00	172.	45.	44.	1.25		
791	2 COMBINED AT	1I270	521.	12.33	89.	25.	24.	.52		
792	2 COMBINED AT	CP270	520.	12.33	92.	26.	25.	3.09		
793	2 COMBINED AT	1I271	1104.	13.50	265.	72.	69.	2.41		
794	2 COMBINED AT	CP271A	284.	13.42	86.	22.	21.	.59		
794	2 COMBINED AT	CP271	1104.	13.50	330.	93.	90.	2.57		
795	2 COMBINED AT	CP271B	109.	12.83	54.	15.	14.	.49		
796	2 COMBINED AT	CP271C	263.	12.17	68.	20.	19.	.79		
796	2 COMBINED AT	1I271C	263.	12.17	67.	20.	19.	.67		
797	2 COMBINED AT	1I272	116.	12.42	12.	3.	3.	.32		
798	2 COMBINED AT	2I272	304.	12.25	29.	7.	7.	.52		
799	2 COMBINED AT	CP272	304.	12.25	39.	14.	13.	1.19		
800	2 COMBINED AT	RCP272	3287.	18.08	2984.	1149.	1107.	75.39		
801	2 COMBINED AT	CP273	1407.	12.83	323.	82.	79.	2.83		
802	2 COMBINED AT	1I274	465.	13.58	134.	33.	32.	1.57		
803	2 COMBINED AT	CP274	687.	13.17	178.	45.	43.	4.40		
804	2 COMBINED AT	CP275	135.	12.42	17.	4.	4.	4.51		
805	2 COMBINED AT	CP276	186.	13.00	38.	9.	9.	4.77		
806	2 COMBINED AT	1I277	569.	13.08	127.	38.	36.	1.82		
807	2 COMBINED AT	2I277	556.	13.08	274.	76.	74.	19.48		
808	2 COMBINED AT	CP277	606.	13.17	279.	78.	75.	24.25		
809	2 COMBINED AT	1I278	1291.	15.50	537.	139.	134.	19.07		
810	2 COMBINED AT	2I278	1645.	15.50	723.	191.	184.	30.87		
811	2 COMBINED AT	CP278	1660.	15.42	793.	224.	216.	31.87		
812	2 COMBINED AT	CP279D	32.	12.17	3.	1.	1.	24.27		
813	2 COMBINED AT	CP279C	76.	12.25	8.	2.	2.	24.32		
814	2 COMBINED AT	CP279B	45.	12.25	4.	1.	1.	24.32		
815	2 COMBINED AT	CP279A	88.	12.42	12.	3.	3.	24.42		
816	2 COMBINED AT	1I279	76.	12.25	7.	2.	2.	24.47		
817	2 COMBINED AT	CP279	1632.	15.58	788.	224.	216.	36.14		
818	2 COMBINED AT	1I280	358.	13.33	104.	26.	25.	36.47		
819	2 COMBINED AT	CP280	826.	16.17	419.	117.	112.	43.77		
820	2 COMBINED AT	CP280A	585.	17.58	389.	119.	114.	43.83		
821	2 COMBINED AT	CP281	742.	13.92	270.	70.	67.	35.86		
822	2 COMBINED AT	CP282	669.	14.17	252.	65.	63.	35.98		
823	2 COMBINED AT	CP283	231.	17.33	154.	48.	47.	36.25		
824	2 COMBINED AT	CP284	1266.	17.58	990.	300.	289.	37.61		
825	2 COMBINED AT	CP285B	229.	17.58	154.	50.	48.	36.31		
826	2 COMBINED AT	1I285A	1257.	17.83	958.	283.	273.	37.67		
827	2 COMBINED AT	CP285A	1259.	17.83	958.	283.	273.	37.89		
828	2 COMBINED AT	CP285	1007.	18.00	719.	190.	183.	37.93		
829	2 COMBINED AT	1I286	4662.	18.50	3264.	1225.	1180.	86.98		
830	2 COMBINED AT	CP286	4662.	18.50	3264.	1235.	1190.	87.20		
831	2 COMBINED AT	1I287	4644.	18.67	3261.	1231.	1186.	87.43		
832	2 COMBINED AT	CP287	5319.	18.58	3724.	1362.	1312.	88.27		
833	2 COMBINED AT	CP287A	618.	13.83	249.	64.	62.	2.42		
834	2 COMBINED AT	1I287B	163.	14.17	68.	18.	17.	.36		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
835	2 COMBINED AT	CP287B	212.	14.25	107.	44.	42.	2.78		
836	2 COMBINED AT	1I287C	599.	16.83	391.	144.	139.	4.32		
837	2 COMBINED AT	CP287C	649.	16.83	430.	167.	161.	7.10		
838	2 COMBINED AT	1I287D	293.	14.00	111.	29.	28.	.73		
839	2 COMBINED AT	CP287D	382.	19.58	345.	149.	143.	7.83		
840	2 COMBINED AT	CP287E	199.	12.58	129.	39.	37.	1.74		
841	2 COMBINED AT	RC287E	3256.	19.17	2973.	1033.	995.	77.13		
842	2 COMBINED AT	CP288A	606.	13.33	188.	48.	46.	2.08		
843	2 COMBINED AT	1I289	598.	16.17	296.	88.	84.	3.09		
844	2 COMBINED AT	CP289	660.	16.17	387.	135.	130.	4.09		
845	2 COMBINED AT	RCP290	3278.	18.33	2980.	1127.	1086.	75.94		
846	2 COMBINED AT	CP291	453.	13.25	120.	32.	31.	1.54		
847	2 COMBINED AT	1I292	1237.	12.75	408.	105.	101.	3.79		
848	2 COMBINED AT	CP292	1704.	13.25	586.	150.	144.	5.36		
849	2 COMBINED AT	1I293	150.	14.25	45.	11.	11.	4.77		
850	2 COMBINED AT	3I293	751.	12.75	419.	116.	112.	25.02		
850	2 COMBINED AT	2I293	677.	12.67	156.	40.	38.	5.54		
851	2 COMBINED AT	CP293	1408.	13.50	693.	185.	178.	25.98		
852	2 COMBINED AT	CP293A	1367.	14.08	668.	181.	174.	26.05		
853	2 COMBINED AT	1I294	56.	13.08	10.	3.	2.	24.32		
854	2 COMBINED AT	CP294	264.	12.17	33.	8.	8.	24.50		
855	2 COMBINED AT	1I294A	730.	15.92	432.	141.	135.	36.40		
856	2 COMBINED AT	2I294A	730.	15.92	432.	141.	136.	36.40		
857	2 COMBINED AT	CP294A	730.	15.92	439.	144.	139.	36.40		
858	2 COMBINED AT	CP295	722.	16.08	447.	151.	145.	36.71		
859	3 COMBINED AT	CP296	674.	18.83	485.	191.	184.	44.76		
860	2 COMBINED AT	CP297	230.	13.17	152.	60.	57.	36.69		
861	2 COMBINED AT	1I297A	246.	18.25	238.	102.	98.	38.16		
862	2 COMBINED AT	CP297A	519.	18.33	432.	153.	148.	38.20		
863	2 COMBINED AT	CP298	4446.	19.83	3602.	1348.	1299.	89.11		
864	2 COMBINED AT	CP302	241.	12.17	22.	10.	9.	1.89		
865	2 COMBINED AT	1RC302	3282.	19.33	3009.	1018.	982.	85.71		
866	2 COMBINED AT	RCP302	3291.	19.42	3019.	1026.	989.	85.71		
867	2 COMBINED AT	1I303	25.	13.58	5.	1.	1.	.05		
868	2 COMBINED AT	2I303	27.	13.67	9.	2.	2.	.08		
869	2 COMBINED AT	3I303	32.	14.42	14.	4.	3.	.13		
870	2 COMBINED AT	4I303	65.	14.25	25.	6.	6.	.28		
871	2 COMBINED AT	5I303	90.	14.50	35.	9.	9.	.38		
872	2 COMBINED AT	6I303	132.	14.50	60.	16.	15.	.96		
873	2 COMBINED AT	CP303	911.	12.67	192.	50.	49.	2.17		
874	4 COMBINED AT	1C303A	530.	12.25	97.	24.	24.	1.48		
875	2 COMBINED AT	CP303A	516.	12.25	94.	24.	23.	18.99		
876	2 COMBINED AT	CP304	1646.	12.42	225.	56.	54.	2.58		
877	2 COMBINED AT	1I306	873.	13.83	360.	95.	92.	26.05		
878	2 COMBINED AT	CP306	974.	13.00	414.	109.	105.	26.54		
879	2 COMBINED AT	CP308	1063.	13.25	448.	118.	114.	26.86		
880	2 COMBINED AT	1I309	645.	13.17	152.	38.	37.	25.57		
881	2 COMBINED AT	CP309	1190.	15.25	590.	185.	178.	27.37		
882	2 COMBINED AT	CP311	721.	13.00	533.	175.	169.	37.44		
883	2 COMBINED AT	CP311A	270.	12.92	54.	14.	13.	.37		
884	2 COMBINED AT	CP312	645.	20.00	476.	200.	193.	45.42		
885	2 COMBINED AT	CP313	247.	13.17	59.	15.	14.	37.12		
886	2 COMBINED AT	CP315	510.	19.92	427.	162.	156.	38.67		
887	2 COMBINED AT	CP316	4438.	20.17	3599.	1333.	1284.	89.93		
888	2 COMBINED AT	CP317	341.	13.25	141.	36.	34.	.97		
889	2 COMBINED AT	CP318	580.	12.67	156.	43.	41.	1.01		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
890	2 COMBINED AT	CP319	578.	12.50	127.	36.	34.	.84		
891	2 COMBINED AT	CP320	675.	12.42	88.	25.	24.	2.53		
892	2 COMBINED AT	RCP320	3313.	19.75	3044.	990.	956.	86.35		
893	2 COMBINED AT	1I321	1309.	12.25	264.	69.	66.	2.83		
894	2 COMBINED AT	CP321	1309.	12.25	306.	80.	77.	4.60		
895	2 COMBINED AT	CP322	1104.	13.00	265.	71.	68.	4.94		
896	2 COMBINED AT	CP323	1600.	12.58	242.	61.	59.	2.73		
897	2 COMBINED AT	CP324	1180.	13.25	221.	57.	55.	3.09		
898	2 COMBINED AT	CP325	880.	13.33	205.	51.	50.	1.41		
899	2 COMBINED AT	CP326	533.	13.08	109.	27.	26.	.75		
900	2 COMBINED AT	CP327	1232.	13.50	513.	136.	131.	27.39		
901	2 COMBINED AT	CP328	1086.	16.50	583.	202.	195.	28.13		
902	2 COMBINED AT	CP329	663.	12.83	117.	30.	29.	.82		
903	2 COMBINED AT	1I330	983.	13.08	582.	195.	188.	38.03		
904	2 COMBINED AT	CP330	1446.	13.08	931.	407.	392.	48.59		
905	2 COMBINED AT	1I331	632.	21.25	473.	201.	194.	46.17		
906	2 COMBINED AT	CP331	632.	21.25	473.	214.	206.	46.54		
907	2 COMBINED AT	CP332	388.	13.75	139.	35.	34.	37.68		
908	2 COMBINED AT	CP333	388.	13.08	137.	36.	35.	1.00		
909	2 COMBINED AT	1I334	4432.	20.33	3596.	1330.	1281.	90.57		
910	2 COMBINED AT	CP334	4915.	20.33	4007.	1473.	1419.	91.31		
911	2 COMBINED AT	CP335	4906.	20.58	4005.	1457.	1403.	91.66		
912	2 COMBINED AT	1I336	311.	13.83	138.	36.	34.	33.31		
913	2 COMBINED AT	2I336	4899.	20.75	4010.	1468.	1414.	94.97		
914	2 COMBINED AT	CP336	4899.	20.75	4013.	1519.	1463.	96.25		
915	3 COMBINED AT	CP337	1120.	12.83	357.	103.	99.	2.34		
916	2 COMBINED AT	CP336A	4897.	20.83	4012.	1527.	1471.	96.62		
917	2 COMBINED AT	CP337B	58.	13.58	17.	5.	4.	.42		
918	2 COMBINED AT	CP338A	734.	12.42	86.	24.	23.	3.30		
919	2 COMBINED AT	CP338	852.	12.67	124.	33.	32.	3.61		
920	2 COMBINED AT	RCP338	3302.	20.42	3039.	918.	887.	87.43		
921	2 COMBINED AT	CP339	1396.	13.33	402.	108.	104.	5.94		
922	2 COMBINED AT	CP340	1090.	13.67	284.	75.	72.	3.57		
923	2 COMBINED AT	CP341	518.	13.92	199.	50.	48.	1.40		
924	2 COMBINED AT	CP342	858.	13.75	263.	66.	64.	1.80		
925	2 COMBINED AT	CP342A	625.	13.58	164.	41.	40.	1.13		
925	2 COMBINED AT	1IC343	1315.	13.75	581.	154.	148.	27.90		
926	2 COMBINED AT	CP343	1821.	13.83	1093.	368.	355.	30.59		
927	2 COMBINED AT	CP344	974.	17.33	580.	216.	208.	28.74		
928	2 COMBINED AT	CP345	756.	13.17	179.	45.	44.	1.25		
929	2 COMBINED AT	CP346A	1404.	13.50	935.	402.	387.	48.59		
930	2 COMBINED AT	CP346B	1268.	14.25	879.	369.	355.	38.58		
931	2 COMBINED AT	CP346C	1244.	14.08	878.	373.	359.	38.31		
932	2 COMBINED AT	1I347	641.	13.08	268.	71.	68.	38.70		
933	2 COMBINED AT	2I347	635.	13.08	300.	88.	84.	52.64		
934	2 COMBINED AT	CP347	635.	13.08	300.	88.	84.	52.98		
935	2 COMBINED AT	CP348A	379.	13.42	165.	44.	42.	1.22		
936	2 COMBINED AT	CP348B	667.	13.42	253.	67.	65.	1.88		
937	2 COMBINED AT	CP349	1430.	13.83	514.	141.	136.	6.87		
938	2 COMBINED AT	1I350	1417.	14.25	532.	148.	143.	7.06		
939	2 COMBINED AT	CP350	1417.	14.25	532.	155.	150.	8.79		
940	2 COMBINED AT	CP351	1053.	14.00	386.	103.	99.	4.33		
941	2 COMBINED AT	CP352	1238.	14.33	486.	130.	126.	3.60		
942	2 COMBINED AT	CP352A	925.	14.75	391.	107.	103.	4.46		
943	2 COMBINED AT	1I353	488.	14.58	216.	58.	56.	1.62		
944	2 COMBINED AT	CP353	1291.	13.92	478.	124.	120.	3.42		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
945	2 COMBINED AT	CP354	604.	13.92	206.	52.	51.	1.44		
946	2 COMBINED AT	1I355	1740.	14.50	1053.	350.	338.	31.04		
947	2 COMBINED AT	2I355	2031.	14.75	1238.	399.	385.	32.48		
948	2 COMBINED AT	CP355	2031.	14.75	1448.	614.	591.	48.68		
950	2 COMBINED AT	CP356	1792.	14.08	1095.	377.	363.	30.94		
951	2 COMBINED AT	CP357	750.	13.42	199.	50.	49.	1.39		
952	2 COMBINED AT	CP358	896.	18.00	829.	263.	253.	41.02		
953	2 COMBINED AT	1I359	404.	13.00	82.	21.	20.	.61		
954	2 COMBINED AT	2I359	1036.	13.33	281.	71.	68.	2.00		
955	2 COMBINED AT	CP359	1281.	14.42	776.	341.	328.	40.58		
956	2 COMBINED AT	1I360	789.	15.83	652.	263.	254.	40.83		
957	2 COMBINED AT	CP360	1026.	15.83	851.	342.	329.	40.83		
958	2 COMBINED AT	1I362	4387.	22.33	3542.	1222.	1177.	101.20		
959	2 COMBINED AT	CP362	4372.	22.42	3568.	1296.	1248.	119.10		
960	2 COMBINED AT	1I363	1777.	21.67	1455.	505.	486.	97.40		
961	2 COMBINED AT	2I363	4372.	22.00	3587.	1233.	1188.	98.56		
961	2 COMBINED AT	3I363	4392.	22.00	3610.	1294.	1246.	100.44		
962	2 COMBINED AT	CP363	4392.	22.00	3610.	1305.	1257.	100.78		
963	2 COMBINED AT	1I364	558.	13.00	157.	41.	39.	1.16		
964	2 COMBINED AT	CP364	3110.	21.17	2554.	970.	934.	97.93		
965	2 COMBINED AT	RCP364	3727.	21.92	2791.	905.	875.	133.69		
966	2 COMBINED AT	1I364A	4895.	20.92	4011.	1514.	1459.	96.69		
967	2 COMBINED AT	CP364A	4895.	20.92	4011.	1516.	1461.	96.77		
968	2 COMBINED AT	1RC367	3300.	20.58	3034.	893.	862.	89.88		
969	2 COMBINED AT	RCP367	3325.	20.58	3058.	986.	953.	89.88		
969	2 COMBINED AT	CP368	367.	13.58	150.	43.	41.	1.27		
970	2 COMBINED AT	RCP368	3332.	21.58	2698.	833.	805.	91.41		
971	2 COMBINED AT	RCP369	3313.	21.42	2769.	828.	800.	90.14		
972	2 COMBINED AT	RCP370	3320.	20.92	3008.	913.	882.	89.99		
973	2 COMBINED AT	1I371	1499.	14.83	595.	179.	173.	9.72		
975	2 COMBINED AT	CP371	1698.	20.67	1315.	556.	536.	65.40		
976	3 COMBINED AT	1I372	1929.	19.42	1515.	502.	483.	55.96		
977	2 COMBINED AT	2I372	2007.	19.42	1554.	530.	510.	56.61		
978	2 COMBINED AT	CP372	2124.	19.42	1616.	602.	580.	61.07		
979	2 COMBINED AT	1I373	1770.	15.92	1354.	520.	501.	49.11		
980	2 COMBINED AT	CP373	1770.	15.92	1354.	520.	501.	49.15		
981	2 COMBINED AT	1I374	1658.	16.92	1333.	500.	482.	49.91		
982	2 COMBINED AT	2I374	1783.	16.83	1376.	516.	497.	50.32		
983	2 COMBINED AT	CP374	1870.	16.83	1405.	533.	513.	50.74		
986	2 COMBINED AT	CP377	120.	12.42	15.	4.	4.	.30		
987	2 COMBINED AT	RC377A	7914.	23.42	5218.	1661.	1603.	156.34		
988	2 COMBINED AT	RC377B	7907.	23.58	4986.	1541.	1487.	156.76		
989	2 COMBINED AT	CP378	4349.	23.25	3184.	1038.	1000.	119.88		
990	2 COMBINED AT	RCP378	7917.	23.25	5339.	1716.	1656.	156.07		
991	2 COMBINED AT	1I379	332.	16.75	189.	70.	67.	61.78		
992	2 COMBINED AT	2I379	332.	16.75	189.	70.	67.	61.88		
993	2 COMBINED AT	3I379	337.	16.75	189.	86.	82.	62.24		
994	2 COMBINED AT	CP379	1797.	21.67	1425.	551.	531.	65.96		
995	2 COMBINED AT	CP380	188.	12.83	44.	13.	13.	.41		
996	2 COMBINED AT	RCP380	7874.	24.75	3565.	1000.	966.	158.18		
997	2 COMBINED AT	RC381B	7878.	24.67	3817.	1058.	1022.	157.77		
998	2 COMBINED AT	CP382	556.	13.17	275.	128.	123.	61.78		
999	2 COMBINED AT	RCP383	7882.	24.50	4063.	1127.	1088.	157.70		
1000	2 COMBINED AT	RC383A	7887.	24.33	4308.	1218.	1175.	157.52		
1001	2 COMBINED AT	CP384	269.	14.33	122.	33.	32.	.65		
1002	2 COMBINED AT	RCP386	7890.	24.25	4401.	1256.	1213.	157.36		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1003 2 COMBINED AT	RCP387	7894.	24.08	4537.	1312.	1266.	157.01		
1004 2 COMBINED AT	CPWTAF	9330.	24.92	4819.	1487.	1435.	183.74		
1010 DIVERSION TO	DCP3	490.	12.92	60.	15.	14.	4.86		
1011 DIVERSION TO	DCP10	1486.	12.75	193.	48.	46.	10.87		
1012 DIVERSION TO	DI43	1441.	12.92	80.	20.	19.	13.90		
1013 DIVERSION TO	DI43-4	21.	12.17	3.	1.	1.	1.09		
1014 DIVERSION TO	DI43-5	19.	12.17	2.	1.	1.	1.11		
1015 DIVERSION TO	DI43-6	6.	12.17	2.	1.	1.	1.13		
1016 DIVERSION TO	DI43-7	20.	12.08	2.	1.	1.	1.15		
1017 DIVERSION TO	DI43-8	12.	12.08	1.	0.	0.	1.16		
1018 DIVERSION TO	DI45-1	325.	13.00	35.	9.	9.	13.98		
1019 DIVERSION TO	DI45	330.	13.17	85.	21.	21.	14.36		
1020 DIVERSION TO	DI46-1	118.	13.42	33.	8.	8.	14.51		
1021 DIVERSION TO	DI119	166.	12.67	26.	7.	6.	.50		
1022 DIVERSION TO	DI120	263.	13.33	79.	20.	19.	1.83		
1023 DIVERSION TO	1D121A	263.	13.33	79.	20.	19.	1.83		
1024 DIVERSION TO	2D121A	58.	13.08	9.	2.	2.	2.33		
1025 DIVERSION TO	1D122	442.	13.08	226.	58.	56.	2.83		
1026 DIVERSION TO	2D122	96.	13.08	20.	5.	5.	3.21		
1027 DIVERSION TO	DI124	338.	13.00	74.	18.	18.	.44		
1028 DIVERSION TO	DI125	200.	13.33	56.	14.	14.	1.01		
1029 DIVERSION TO	DI126	439.	14.08	218.	59.	57.	5.25		
1030 DIVERSION TO	DI128	410.	13.17	98.	25.	24.	1.36		
1031 DIVERSION TO	DI130	0.	.08	0.	0.	0.	1.83		
1032 DIVERSION TO	DI134	2643.	13.58	949.	243.	234.	7.89		
1033 DIVERSION TO	DI136	308.	13.33	87.	22.	21.	1.01		
1034 DIVERSION TO	DI138	439.	14.08	218.	59.	57.	5.25		
1035 DIVERSION TO	DI139	685.	13.33	305.	81.	78.	6.35		
1036 DIVERSION TO	DI144	795.	13.42	242.	61.	58.	5.54		
1037 DIVERSION TO	DI145	1550.	13.50	665.	172.	166.	8.52		
1038 DIVERSION TO	DI146	290.	14.33	141.	40.	39.	14.48		
1039 DIVERSION TO	1D147	145.	14.33	71.	20.	19.	14.48		
1040 DIVERSION TO	2D147	2060.	14.00	741.	193.	186.	15.42		
1041 DIVERSION TO	DI148	172.	14.08	52.	13.	13.	16.48		
1042 DIVERSION TO	DI149	435.	14.50	265.	76.	74.	21.72		
1043 DIVERSION TO	DI150	49.	13.00	20.	5.	5.	17.46		
1044 DIVERSION TO	DI152	205.	13.75	122.	34.	33.	23.66		
1045 DIVERSION TO	DI153	435.	14.50	265.	76.	74.	21.72		
1046 DIVERSION TO	DI154	849.	15.75	529.	172.	166.	24.02		
1047 DIVERSION TO	DI155	0.	.08	0.	0.	0.	24.01		
1048 DIVERSION TO	DI158A	0.	.08	0.	0.	0.	1.14		
1050 DIVERSION TO	DI164	991.	14.00	315.	79.	76.	11.63		
1051 DIVERSION TO	DI165	868.	14.00	365.	91.	88.	11.63		
1052 DIVERSION TO	DI166	1943.	14.42	801.	224.	216.	15.92		
1053 DIVERSION TO	DI167	1753.	15.58	831.	263.	253.	18.44		
1054 DIVERSION TO	1D168	205.	13.75	122.	34.	33.	23.66		
1055 DIVERSION TO	2D168	117.	15.92	51.	14.	13.	25.61		
1056 DIVERSION TO	1D169	0.	.08	0.	0.	0.	24.01		
1057 DIVERSION TO	2D169	140.	13.50	84.	26.	25.	26.47		
1058 DIVERSION TO	DI175A	375.	12.25	34.	9.	8.	.28		
1059 DIVERSION TO	DI176A	0.	.08	0.	0.	0.	2.47		
1060 DIVERSION TO	DI177	294.	13.25	93.	24.	23.	15.08		
1061 DIVERSION TO	1D183	532.	15.92	256.	69.	67.	25.61		
1062 DIVERSION TO	2D183	274.	16.25	150.	47.	46.	26.00		
1063 DIVERSION TO	DI191	0.	.08	0.	0.	0.	3.09		
1064 DIVERSION TO	DI193	2291.	14.17	1121.	280.	270.	25.34		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1065	DIVERSION TO	DI194	0.	.08	0.	0.	0.	25.34		
1066	DIVERSION TO	1D195	1507.	14.33	1086.	340.	328.	30.29		
1067	DIVERSION TO	2D195	292.	14.75	221.	73.	70.	30.66		
1068	DIVERSION TO	DI196	450.	16.58	309.	112.	108.	38.46		
1069	DIVERSION TO	DI197	629.	17.50	441.	155.	149.	41.09		
1070	DIVERSION TO	DI198	629.	17.50	441.	155.	149.	41.09		
1071	DIVERSION TO	1D201	2050.	15.50	1302.	375.	361.	43.14		
1072	DIVERSION TO	DI208	1339.	12.83	490.	123.	119.	12.18		
1073	DIVERSION TO	2D208	476.	13.25	118.	29.	28.	1.00		
1074	DIVERSION TO	DI209A	1541.	13.33	447.	112.	108.	14.18		
1075	DIVERSION TO	DI209	1110.	13.75	225.	56.	54.	13.18		
1076	DIVERSION TO	2D209	288.	13.58	82.	20.	20.	14.68		
1077	DIVERSION TO	DI210	0.	.08	0.	0.	0.	13.18		
1078	DIVERSION TO	2D210	222.	14.08	60.	15.	14.	16.18		
1079	DIVERSION TO	DI212	1536.	14.50	600.	150.	145.	36.34		
1080	DIVERSION TO	DI218	91.	13.08	38.	9.	9.	1.49		
1081	DIVERSION TO	1D219	626.	13.33	297.	76.	73.	14.18		
1082	DIVERSION TO	2D219	58.	13.25	13.	3.	3.	15.67		
1083	DIVERSION TO	1D220	636.	13.58	204.	52.	50.	14.68		
1084	DIVERSION TO	2D220	156.	14.58	64.	16.	16.	16.67		
1085	DIVERSION TO	1D221	550.	14.08	170.	43.	41.	16.18		
1086	DIVERSION TO	2D221	407.	14.58	131.	33.	32.	18.17		
1087	DIVERSION TO	DI225	60.	8.75	60.	53.	51.	54.67		
1088	DIVERSION TO	1D226	600.	17.42	475.	182.	175.	51.75		
1089	DIVERSION TO	2D226	0.	.08	0.	0.	0.	52.23		
1090	DIVERSION TO	DR228A	3212.	17.58	2894.	1064.	1025.	68.77		
1091	DIVERSION TO	1D230	187.	12.33	28.	7.	7.	1.02		
1092	DIVERSION TO	2D230	464.	12.25	81.	20.	20.	1.20		
1093	DIVERSION TO	DI231	172.	12.33	25.	6.	6.	1.02		
1094	DIVERSION TO	DI237	420.	13.25	131.	33.	32.	15.67		
1095	DIVERSION TO	DI238	893.	14.58	408.	114.	110.	16.67		
1096	DIVERSION TO	DI239	733.	14.58	319.	82.	79.	18.17		
1097	DIVERSION TO	DI245	1116.	12.50	162.	40.	39.	1.74		
1098	DIVERSION TO	1D248	990.	12.83	225.	56.	54.	14.08		
1099	DIVERSION TO	2D248	770.	13.42	237.	59.	57.	16.04		
1100	DIVERSION TO	DI249	325.	13.58	78.	20.	19.	26.34		
1101	DIVERSION TO	1D250	403.	13.58	116.	29.	28.	26.34		
1102	DIVERSION TO	2D250	301.	13.83	88.	22.	21.	27.84		
1103	DIVERSION TO	1D250A	306.	13.83	129.	32.	31.	27.84		
1104	DIVERSION TO	2D250A	57.	15.25	20.	5.	5.	30.34		
1105	DIVERSION TO	1D251	160.	15.25	56.	14.	13.	30.34		
1106	DIVERSION TO	2D251	1731.	16.17	888.	228.	220.	31.76		
1107	DIVERSION TO	DI256	119.	13.42	23.	6.	6.	.34		
1108	DIVERSION TO	DI259	19.	12.25	1.	0.	0.	.12		
1109	DIVERSION TO	DI261	0.	.08	0.	0.	0.	.50		
1110	DIVERSION TO	DI262	1955.	13.67	630.	158.	152.	17.54		
1111	DIVERSION TO	DI263	1024.	13.83	356.	90.	87.	18.57		
1112	DIVERSION TO	DI264	77.	14.00	24.	6.	6.	28.84		
1113	DIVERSION TO	DI265A	3.	14.58	1.	0.	0.	33.84		
1114	DIVERSION TO	1D266	276.	14.58	48.	12.	12.	33.84		
1115	DIVERSION TO	2D266	1948.	16.50	1264.	346.	333.	35.76		
1116	DIVERSION TO	DI267	415.	16.83	309.	91.	88.	36.26		
1117	DIVERSION TO	DI271A	211.	13.00	62.	16.	15.	.90		
1118	DIVERSION TO	DI272	122.	12.25	12.	3.	3.	.12		
1119	DIVERSION TO	DI277	98.	13.17	22.	5.	5.	.91		
1120	DIVERSION TO	1D278	676.	13.83	256.	64.	62.	18.57		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1121	DIVERSION TO	2D278	731.	14.08	297.	76.	73.	19.07		
1122	DIVERSION TO	DI280	55.	14.00	20.	5.	5.	35.86		
1123	DIVERSION TO	DI283	236.	16.75	153.	43.	42.	36.09		
1124	DIVERSION TO	DI284	1285.	16.92	973.	284.	273.	37.09		
1125	DIVERSION TO	DI289	887.	13.75	287.	78.	75.	2.57		
1126	DIVERSION TO	DI291	186.	12.92	16.	4.	4.	.55		
1127	DIVERSION TO	DI292	979.	12.92	278.	71.	68.	2.83		
1128	DIVERSION TO	2D292	682.	13.17	178.	45.	43.	4.40		
1129	DIVERSION TO	1D293	90.	12.67	17.	4.	4.	4.51		
1130	DIVERSION TO	2D293	89.	13.00	28.	7.	7.	4.77		
1131	DIVERSION TO	3D293	583.	13.25	279.	78.	75.	24.25		
1132	DIVERSION TO	1D294	31.	12.25	3.	1.	1.	24.27		
1133	DIVERSION TO	2D294	45.	12.50	8.	2.	2.	24.32		
1134	DIVERSION TO	1D294A	10.	12.25	2.	0.	0.	24.32		
1135	DIVERSION TO	2D294A	67.	12.58	12.	3.	3.	24.42		
1136	DIVERSION TO	3D294A	738.	15.67	433.	133.	128.	36.14		
1137	DIVERSION TO	DI297	220.	17.83	153.	50.	48.	36.31		
1138	DIVERSION TO	1D297A	246.	17.83	238.	94.	91.	37.89		
1139	DIVERSION TO	2D297A	274.	18.08	195.	52.	50.	37.93		
1140	DIVERSION TO	DI302	14.	20.58	13.	4.	4.	1.74		
1141	DIVERSION TO	1D303	23.	12.17	3.	1.	1.	.02		
1142	DIVERSION TO	2D303	24.	12.00	2.	0.	0.	.05		
1143	DIVERSION TO	3D303	24.	12.25	5.	1.	1.	.08		
1144	DIVERSION TO	4D303	23.	12.25	5.	1.	1.	.13		
1145	DIVERSION TO	5D303	45.	12.33	11.	3.	3.	.28		
1146	DIVERSION TO	6D303	33.	12.50	12.	3.	3.	.38		
1147	DIVERSION TO	7D303	47.	12.50	26.	7.	6.	.68		
1148	DIVERSION TO	1D303A	282.	12.83	42.	11.	10.	1.00		
1149	DIVERSION TO	D164A	117.	13.67	33.	8.	8.	7.68		
1149	DIVERSION TO	2D303A	10.	12.17	1.	0.	0.	1.01		
1150	DIVERSION TO	3D303A	23.	12.17	5.	1.	1.	1.06		
1151	DIVERSION TO	1D306	894.	13.25	300.	78.	75.	5.36		
1152	DIVERSION TO	2D306	140.	14.17	65.	17.	17.	26.05		
1153	DIVERSION TO	DI320	0.	.08	0.	0.	0.	1.89		
1154	DIVERSION TO	DI321	122.	12.58	45.	11.	11.	2.73		
1155	DIVERSION TO	DI338A	0.	.08	0.	0.	0.	2.53		
1156	DIVERSION TO	DI346C	1273.	13.50	880.	384.	370.	48.59		
1157	DIVERSION TO	DI347	0.	.08	0.	0.	0.	.34		
1158	DIVERSION TO	DI350	100.	14.92	39.	11.	10.	4.46		
1159	DIVERSION TO	DI360	315.	14.25	210.	86.	83.	38.58		
1160	DIVERSION TO	DI363	1782.	20.92	1458.	547.	527.	96.77		
1161	DIVERSION TO	DIGILA	470.	21.17	405.	167.	161.	97.93		
1162	DIVERSION TO	DI367	373.	13.50	335.	103.	99.	2.34		
1163	DIVERSION TO	DI371	1681.	19.42	1302.	492.	474.	61.07		
1164	DIVERSION TO	DI379	63.	19.42	36.	10.	10.	61.07		
1165	ROUTED TO	SRWT3	0.	.08	0.	0.	0.	20.48	1197.48	17.58
1166	ROUTED TO	SR16	0.	.08	0.	0.	0.	1.13	1215.39	24.92
1167	ROUTED TO	SR20	0.	.08	0.	0.	0.	1.07	1456.94	24.92
1168	ROUTED TO	SR21	191.	13.25	40.	12.	11.	3.46	1347.09	14.67
1169	ROUTED TO	SR23	0.	.08	0.	0.	0.	3.62	1279.06	24.92
1170	ROUTED TO	SR24	0.	.08	0.	0.	0.	.14	1217.43	13.25
1171	ROUTED TO	SR25	1374.	12.50	146.	37.	36.	5.29	1214.98	12.50
1172	ROUTED TO	SR27	1642.	12.50	205.	52.	50.	2.16	1214.44	12.50
1173	ROUTED TO	SR29	0.	.08	0.	0.	0.	.22	1168.31	13.17
1174	ROUTED TO	SR38	3220.	12.50	357.	93.	90.	4.64	1300.13	12.50
1176	ROUTED TO	SR41A1	23.	12.17	3.	1.	1.	.02	1135.75	12.17

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1177	ROUTED TO	SR41A2	60.	12.00	3.	1.	1.	.05	1135.66	12.08
1178	ROUTED TO	SR41A3	24.	12.25	5.	1.	1.	.08	1131.18	12.25
1179	ROUTED TO	SR41A	23.	12.25	5.	1.	1.	.13	1126.26	12.25
1180	ROUTED TO	SR41-1	148.	12.33	14.	3.	3.	.28	1116.03	12.33
1181	ROUTED TO	SR41-2	33.	12.50	12.	3.	3.	.38	1101.42	12.42
1182	ROUTED TO	SR41	531.	12.50	64.	16.	16.	.68	1100.02	12.58
1183	ROUTED TO	SR42	6601.	12.75	797.	205.	198.	14.56	1102.23	12.83
1184	ROUTED TO	SR43-1	282.	12.83	42.	11.	10.	1.00	1094.94	12.83
1185	ROUTED TO	SR43-2	10.	12.17	1.	0.	0.	1.01	1093.36	12.17
1186	ROUTED TO	SR43-3	23.	12.17	5.	1.	1.	1.06	1093.79	12.17
1187	ROUTED TO	SR43-4	21.	12.17	3.	1.	1.	1.09	1094.17	12.17
1188	ROUTED TO	SR43-5	19.	12.17	2.	1.	1.	1.11	1091.68	12.17
1189	ROUTED TO	SR43-6	6.	12.17	2.	1.	1.	1.13	1089.99	12.17
1190	ROUTED TO	SR43-7	20.	12.08	2.	1.	1.	1.15	1089.50	12.08
1191	ROUTED TO	SR43-8	12.	12.08	1.	0.	0.	1.16	1086.51	12.08
1192	ROUTED TO	SR43	5503.	12.92	799.	206.	198.	13.90	1092.87	13.00
1193	ROUTED TO	SR45-1	1428.	13.00	90.	22.	22.	13.98	1086.77	13.08
1194	ROUTED TO	SR45	815.	13.17	101.	25.	24.	14.36	1084.85	13.25
1195	ROUTED TO	SR46-1	118.	13.42	33.	8.	8.	14.51	1080.94	12.50
1196	ROUTED TO	SR46	1705.	12.75	259.	65.	63.	17.43	1070.76	12.75
1197	ROUTED TO	RS47	0.	.08	0.	0.	0.	18.57	1040.07	24.92
1198	ROUTED TO	SR138	1327.	15.75	812.	261.	252.	24.02	1048.81	15.75
1199	ROUTED TO	SR139	835.	13.92	428.	118.	114.	7.22	1047.83	14.00
1200	ROUTED TO	SR158A	304.	13.75	78.	20.	20.	1.14	1115.42	13.83
1201	ROUTED TO	SR212	1337.	15.58	626.	161.	155.	36.88	1080.12	15.67
1202	ROUTED TO	SR221A	271.	14.25	95.	24.	23.	.80	1079.02	14.33
1203	ROUTED TO	SR225	84.	13.42	81.	47.	45.	.80	1063.81	13.50
1204	ROUTED TO	SR226	534.	13.08	493.	225.	217.	53.41	1077.10	18.50
1205	ROUTED TO	SR241	4245.	17.08	2888.	1083.	1043.	78.75	1048.40	17.08
1206	ROUTED TO	SR253	4102.	17.75	2881.	1101.	1061.	84.50	1026.53	17.75
1207	ROUTED TO	SR258	0.	.08	0.	0.	0.	.38	969.35	13.67
1208	ROUTED TO	SR259	0.	.08	0.	0.	0.	.32	1010.21	14.08
1209	ROUTED TO	SR268	4695.	18.00	3267.	1242.	1196.	86.28	1012.11	18.08
1210	ROUTED TO	SR269	36.	16.75	17.	5.	5.	1.25	1011.90	24.58
1211	ROUTED TO	SR270	468.	12.42	85.	22.	21.	3.09	1012.77	12.42
1212	ROUTED TO	SR271	904.	13.75	290.	79.	76.	2.57	1014.73	13.83
1213	ROUTED TO	SR271A	284.	13.42	86.	22.	21.	.59	1018.30	13.42
1214	ROUTED TO	SR271C	69.	17.42	26.	6.	6.	.79	1011.57	17.75
1215	ROUTED TO	SR272	0.	.08	0.	0.	0.	1.19	977.27	24.00
1216	ROUTED TO	SR273	1398.	12.92	323.	82.	79.	2.83	1057.40	13.00
1217	ROUTED TO	SR274	682.	13.17	178.	45.	43.	4.40	1045.57	13.25
1218	ROUTED TO	SR275	90.	12.67	17.	4.	4.	4.51	1039.57	12.67
1219	ROUTED TO	SR276	186.	13.00	38.	9.	9.	4.77	1036.06	13.00
1220	ROUTED TO	SR277	583.	13.25	279.	78.	75.	24.25	1025.94	13.25
1221	ROUTED TO	SR279A	67.	12.58	12.	3.	3.	24.42	1015.68	12.58
1222	ROUTED TO	SR279B	47.	12.25	4.	1.	1.	24.32	1020.09	12.25
1223	ROUTED TO	SR279C	45.	12.50	8.	2.	2.	24.32	1021.39	12.50
1224	ROUTED TO	SR279D	32.	12.25	3.	1.	1.	24.27	1023.59	12.25
1225	ROUTED TO	SR279	1634.	15.67	788.	224.	216.	36.14	1015.15	15.75
1226	ROUTED TO	SR280A	583.	17.67	385.	108.	104.	43.83	1008.92	17.75
1227	ROUTED TO	SR280	594.	16.83	390.	117.	112.	43.77	1009.52	16.92
1228	ROUTED TO	SR281	732.	14.00	259.	66.	64.	35.86	1008.53	14.00
1229	ROUTED TO	SR282	562.	14.58	252.	65.	63.	35.98	1002.91	14.58
1230	ROUTED TO	SR284	1259.	17.67	959.	282.	271.	37.61	1011.20	17.75
1231	ROUTED TO	SR285A	1259.	17.83	958.	283.	273.	37.89	1006.58	17.83
1232	ROUTED TO	SR285B	225.	17.75	154.	50.	48.	36.31	1009.07	17.92

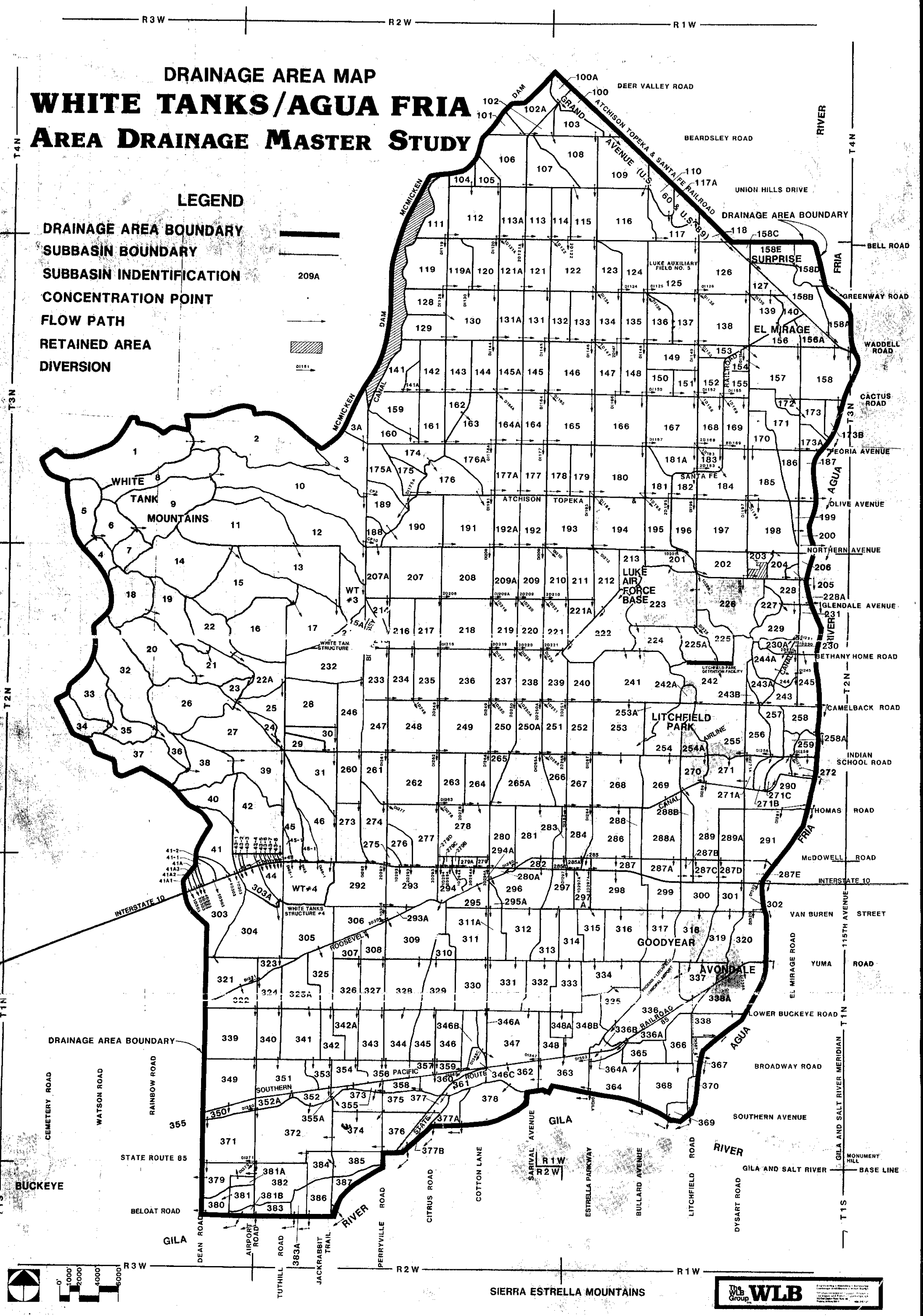
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1233	ROUTED TO	SR285	1007.	18.08	719.	190.	183.	37.93	1003.61	18.08
1234	ROUTED TO	SR287	4450.	19.58	3604.	1346.	1296.	88.27	994.06	19.67
1235	ROUTED TO	SR287A	55.	14.00	55.	27.	26.	2.42	980.87	16.58
1236	ROUTED TO	SR287B	65.	18.75	61.	25.	24.	2.78	979.95	19.67
1237	ROUTED TO	SR287C	376.	19.17	340.	125.	120.	7.10	978.78	19.58
1238	ROUTED TO	SR287D	67.	24.92	62.	25.	24.	7.83	981.50	24.92
1239	ROUTED TO	SR287E	14.	20.58	13.	4.	4.	1.74	985.23	20.25
1240	ROUTED TO	SR290	319.	12.92	64.	16.	15.	.55	996.51	12.92
1241	ROUTED TO	SR291	128.	14.33	115.	32.	31.	1.54	982.80	14.33
1242	ROUTED TO	SR293	1394.	13.58	683.	181.	174.	25.98	1007.92	13.67
1243	ROUTED TO	SR293A	1360.	14.17	641.	171.	165.	26.05	1006.10	14.25
1244	ROUTED TO	SR294	0.	.08	0.	0.	0.	24.50	977.26	18.17
1245	ROUTED TO	SR294A	729.	15.92	439.	142.	137.	36.40	1008.08	16.00
1246	ROUTED TO	SR297	0.	.08	0.	0.	0.	36.69	956.46	24.92
1247	ROUTED TO	SR298	4443.	19.92	3600.	1338.	1289.	89.11	980.27	19.92
1248	ROUTED TO	SR302	39.	12.58	21.	9.	9.	1.89	975.84	12.58
1249	ROUTED TO	SR305	615.	13.17	132.	33.	32.	.92	1003.66	13.17
1250	ROUTED TO	SR306	973.	13.50	414.	107.	103.	26.54	1005.48	13.00
1251	ROUTED TO	SR320	51.	13.25	45.	21.	20.	2.53	959.87	13.17
1252	ROUTED TO	SR321	931.	12.75	224.	58.	56.	4.60	1001.17	12.75
1253	ROUTED TO	SR323	1578.	12.58	219.	55.	53.	2.73	1001.91	12.58
1254	ROUTED TO	SR336	4897.	20.83	4012.	1515.	1459.	96.25	934.34	20.83
1255	ROUTED TO	SR336B	101.	12.17	9.	2.	2.	.08	923.48	12.17
1256	ROUTED TO	SR337	373.	13.50	335.	103.	99.	2.34	958.33	13.50
1257	ROUTED TO	SR338A	646.	12.50	86.	24.	23.	3.30	943.54	12.50
1258	ROUTED TO	SR346B	1271.	14.25	879.	367.	353.	38.58	908.81	14.33
1259	ROUTED TO	SR346C	1220.	14.25	871.	360.	347.	38.31	910.03	14.25
1260	ROUTED TO	SR347	611.	13.17	300.	88.	84.	52.98	908.16	13.17
1261	ROUTED TO	SR348	280.	12.83	49.	12.	12.	.34	908.40	12.83
1262	ROUTED TO	SR348B	645.	13.58	253.	67.	65.	1.88	914.58	13.67
1263	ROUTED TO	SR349	1416.	13.92	514.	141.	136.	6.87	895.06	14.00
1264	ROUTED TO	SR350	1374.	14.42	503.	143.	138.	8.79	889.29	14.42
1265	ROUTED TO	SR351	1003.	14.08	386.	103.	99.	4.33	894.17	14.25
1266	ROUTED TO	SR352	1144.	14.67	461.	121.	117.	3.60	892.88	14.67
1267	ROUTED TO	SR352A	897.	14.92	354.	95.	91.	4.46	891.48	15.00
1268	ROUTED TO	SR353	1288.	13.92	478.	124.	120.	3.42	898.37	16.67
1269	ROUTED TO	SR354	539.	14.17	206.	52.	51.	1.44	895.18	14.25
1270	ROUTED TO	SR355	1824.	15.25	1365.	544.	524.	48.68	895.16	15.33
1271	ROUTED TO	SR355A	0.	.08	0.	0.	0.	.04	892.56	13.92
1272	ROUTED TO	SR356	1775.	14.17	1069.	355.	342.	30.94	899.68	14.17
1273	ROUTED TO	SR358	860.	20.92	791.	219.	211.	41.02	895.47	21.33
1274	ROUTED TO	SR359	803.	15.33	655.	271.	261.	40.58	901.91	15.50
1275	ROUTED TO	SR360	940.	16.25	841.	323.	311.	40.83	895.21	16.42
1276	ROUTED TO	SR361	0.	.08	0.	0.	0.	.21	896.66	16.58
1277	ROUTED TO	SR362	4363.	22.50	3557.	1250.	1204.	119.10	902.86	22.67
1278	ROUTED TO	SR364	3110.	21.17	2554.	961.	926.	97.93	912.34	21.25
1279	ROUTED TO	SR368	221.	14.25	133.	39.	37.	1.27	917.77	14.25
1280	ROUTED TO	SR377	0.	.08	0.	0.	0.	.30	892.51	13.83
1281	ROUTED TO	SR379	1769.	21.83	1408.	538.	518.	65.96	861.15	22.17
1282	ROUTED TO	SR381	78.	13.58	16.	4.	4.	.21	866.93	13.67
1283	ROUTED TO	SR381A	0.	.08	0.	0.	0.	.10	865.43	14.50

3.6 FINAL MODELING RESULTS ON DISKETTES

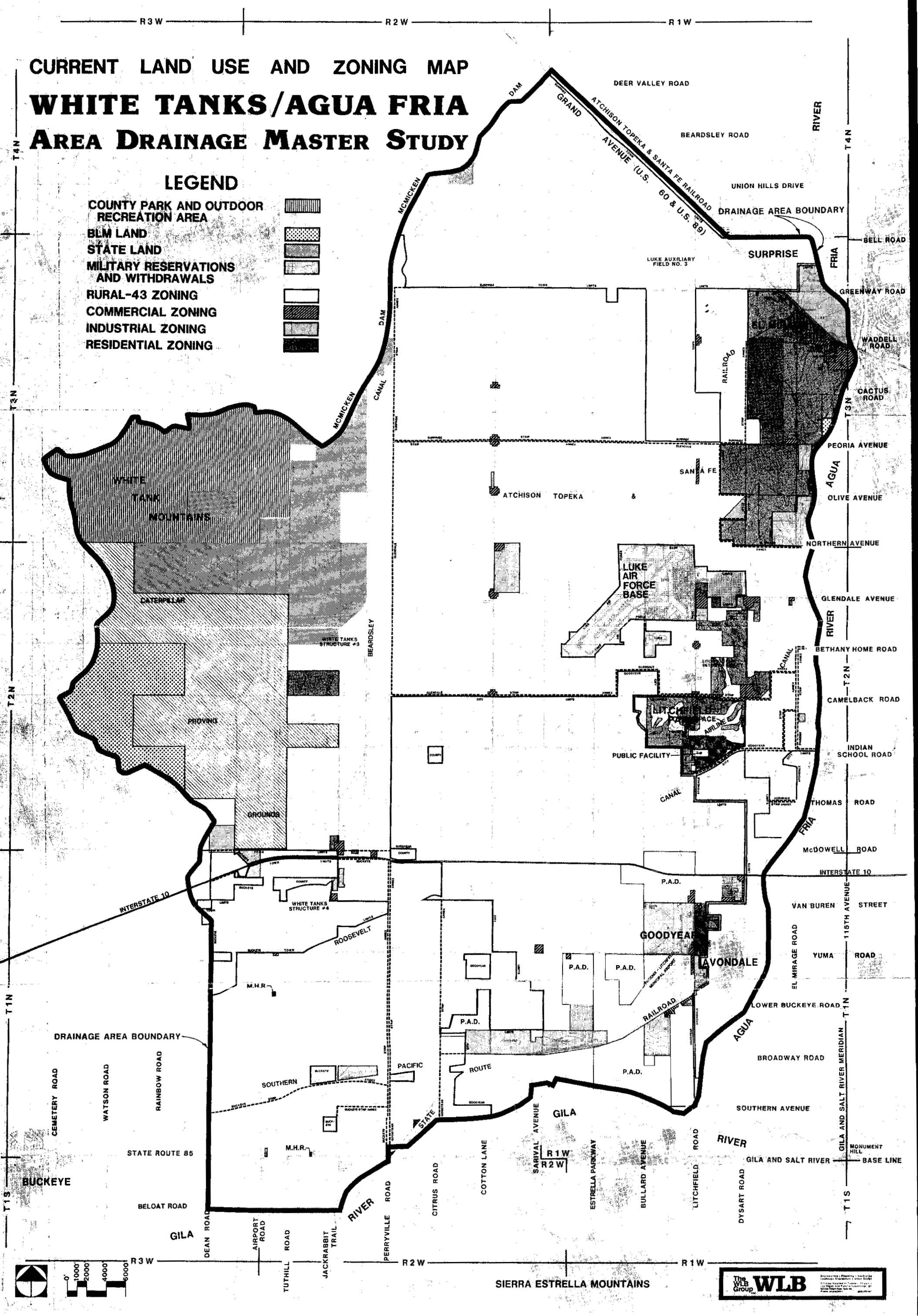
The diskettes for the 100-year, 24-hr input and output files are filed in clear plastic inserts located in the back of this report. The input file is named WTADMS.24 and the output file is named WTADMS.240. The output file has been archived due to its size using the PKARC routine. To unarchive this file type in PKXARC WTADMS24.ARC. The archive programs are also included on the diskettes.

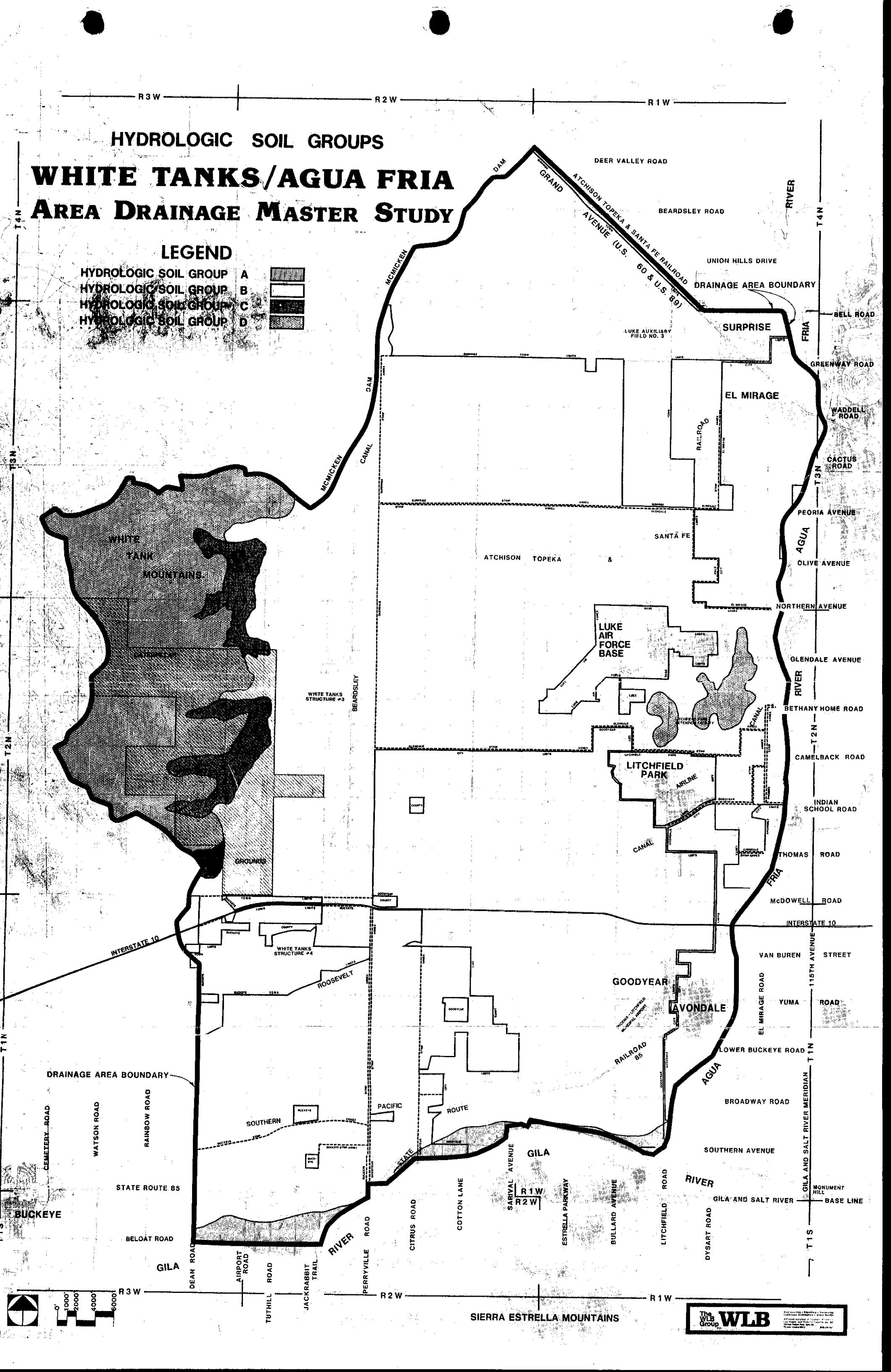
DRAINAGE AREA MAP
WHITE TANKS/AGUA FRIA AREA DRAINAGE MASTER STUDY

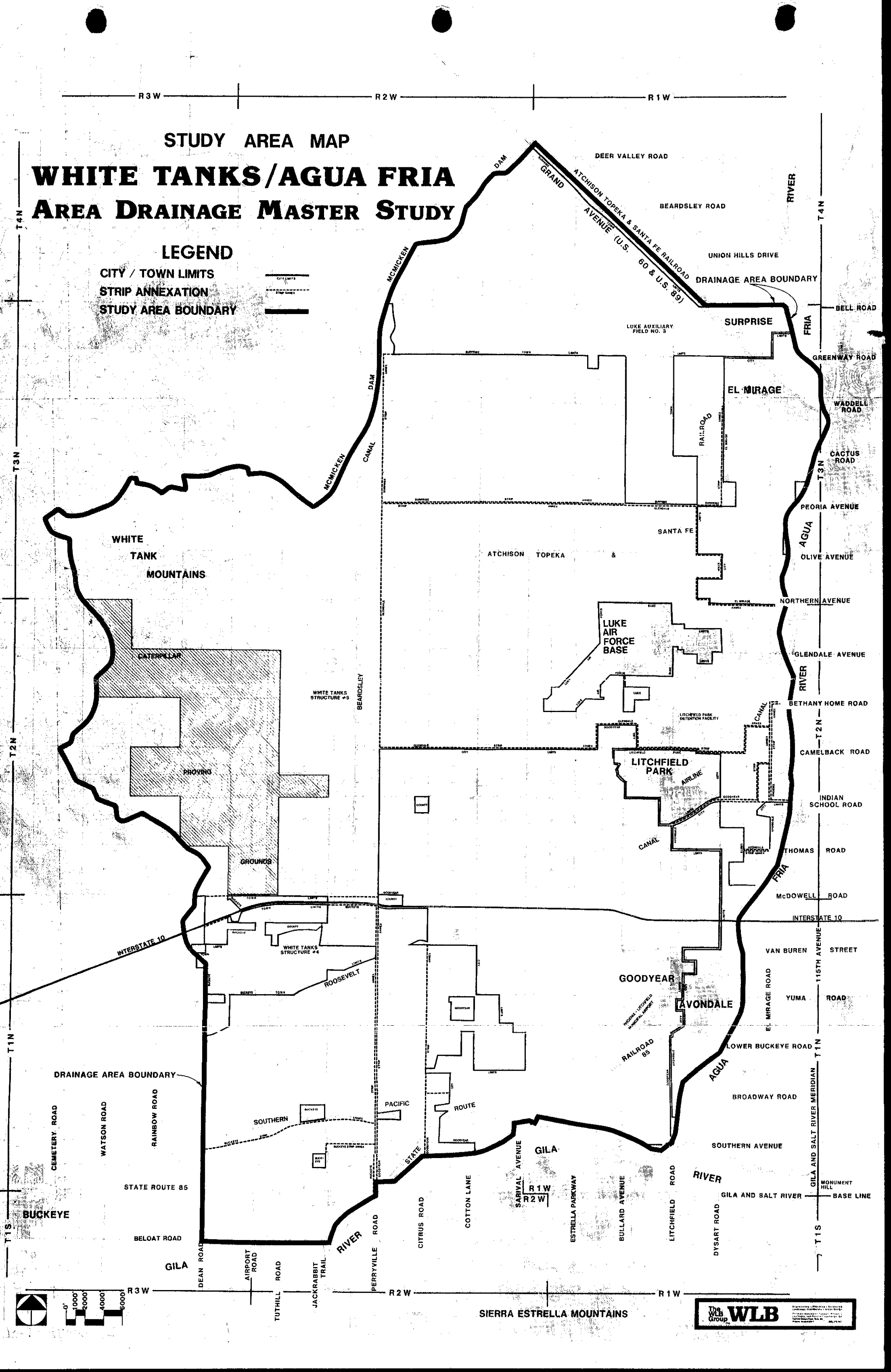


CURRENT LAND USE AND ZONING MAP

WHITE TANKS/AGUA FRIA AREA DRAINAGE MASTER STUDY




HYDROLOGIC SOIL GROUPS
WHITE TANKS/AGUA FRIA
AREA DRAINAGE MASTER STUDY
LEGEND
HYDROLOGIC SOIL GROUP A
HYDROLOGIC SOIL GROUP B
HYDROLOGIC SOIL GROUP C
HYDROLOGIC SOIL GROUP D
R3W
R2W
R1W
T4N
T3N
T2N
T1N
T1S
DEER VALLEY ROAD
BEARDSLEY ROAD
UNION HILLS DRIVE
DRAINAGE AREA BOUNDARY
BELL ROAD
GREENWAY ROAD
WADDELL ROAD
CACTUS ROAD
PEORIA AVENUE
OLIVE AVENUE
NORTHERN AVENUE
GLENDALE AVENUE
BETHANY HOME ROAD
CAMELBACK ROAD
INDIAN SCHOOL ROAD
THOMAS ROAD
McDOWELL ROAD
VAN BUREN STREET
YUMA ROAD
LOWER BUCKEYE ROAD
BROADWAY ROAD
SOUTHERN AVENUE
GILA AND SALT RIVER BASE LINE
MONUMENT HILL
GILA AND SALT RIVER MERIDIAN
115TH AVENUE
EL MIRAGE ROAD
DYSART ROAD
LITCHFIELD ROAD
BULLARD AVENUE
ESTRELLA PARKWAY
COTTON LANE
SARIVAL AVENUE
CITRUS ROAD
PERRYVILLE ROAD
JACKRABBIT TRAIL
TUTHILL ROAD
AIRPORT ROAD
DEAN ROAD
BUCKEYE ROAD
STATE ROUTE 85
WATSON ROAD
RAINBOW ROAD
CEMETERY ROAD
INTERSTATE 10
ROOSEVELT
WHITE TANKS STRUCTURE #4
WHITE TANKS STRUCTURE #3
WHITE TANK MOUNTAINS
CATERPILLAR
GROUNDS
BEARDSLEY
MCMICKEN DAM
MCMICKEN CANAL
GRAND AVENUE (U.S. 60 & U.S. 89)
ATCHISON TOPEKA & SANTA FE RAILROAD
LUKE AUXILIARY FIELD NO. 3
SURPRISE
EL MIRAGE
SANTA FE
ATCHISON TOPEKA &
LUKE AIR FORCE BASE
LITCHFIELD PARK
GOODYEAR
AVONDALE
PHOENIX-LITCHFIELD MUNICIPAL AIRPORT
RAILROAD 85
SOUTHERN PACIFIC ROUTE
GILA RIVER
AGUA FRIA RIVER
SIERRA ESTRELLA MOUNTAINS
BUCKEYE
GILA
0' 1000' 2000' 4000' 6000'
The WLB Group Inc. WLB


STUDY AREA MAP
WHITE TANKS/AGUA FRIA
AREA DRAINAGE MASTER STUDY
LEGEND
CITY / TOWN LIMITS
STRIP ANNEXATION
STUDY AREA BOUNDARY
R3W
R2W
R1W
T4N
T3N
T2N
T1N
T1S
DEER VALLEY ROAD
BEARDSLEY ROAD
UNION HILLS DRIVE
DRAINAGE AREA BOUNDARY
BELL ROAD
GREENWAY ROAD
WADDELL ROAD
CACTUS ROAD
PEORIA AVENUE
OLIVE AVENUE
NORTHERN AVENUE
GLENDALE AVENUE
BETHANY HOME ROAD
CAMELBACK ROAD
INDIAN SCHOOL ROAD
THOMAS ROAD
McDOWELL ROAD
INTERSTATE 10
VAN BUREN STREET
YUMA ROAD
LOWER BUCKEYE ROAD
BROADWAY ROAD
SOUTHERN AVENUE
GILA AND SALT RIVER BASE LINE
MONUMENT HILL
GILA AND SALT RIVER MERIDIAN
115TH AVENUE
GRAND AVENUE (U.S. 60 & U.S. 89)
ATCHISON TOPEKA & SANTA FE RAILROAD
MCMICKEN DAM
MCMICKEN CANAL
AGUA FRIA RIVER
GILA RIVER
SURPRISE
LUKE AUXILIARY FIELD NO. 3
EL MIRAGE
SANTA FE RAILROAD
ATCHISON TOPEKA & SANTA FE RAILROAD
LUKE AIR FORCE BASE
LITCHFIELD PARK DETENTION FACILITY
LITCHFIELD PARK
AIRLINE
CANAL
GOODYEAR
AVONDALE
PHOENIX-LITCHFIELD MUNICIPAL AIRPORT
RAILROAD 85
WHITE TANK MOUNTAINS
CATERPILLAR
PROVING GROUNDS
WHITE TANKS STRUCTURE #3
WHITE TANKS STRUCTURE #4
ROOSEVELT
SOUTHERN PACIFIC ROUTE
STATE ROUTE 85
BUCKEYE
BELOAT ROAD
CEMETERY ROAD
WATSON ROAD
RAINBOW ROAD
DEAN ROAD
AIRPORT ROAD
TUTHILL ROAD
JACKRABBIT TRAIL
PERRYVILLE ROAD
CITRUS ROAD
COTTON LANE
SARIVAL AVENUE
ESTRELLA PARKWAY
BULLARD AVENUE
LITCHFIELD ROAD
DYSART ROAD
SIERRA ESTRELLA MOUNTAINS
0' 1000' 2000' 4000' 6000'
The WLB Group WLB

SECTION 4: HYDRAULIC ANALYSIS

4.1 METHOD DESCRIPTION

The HEC-2 Water Surface Profiles computer program was utilized to compute water surface profiles for the stream reaches where detailed analyses were required. The U.S. Army Corps of Engineers developed this program and the version used for this study was Version 4.6.2, May 30, 1991. This program was purchased from Dodson & Associates in a packaged called PROHEC2.

Since there are many washes delineated in this study, a description of how each starting WSEL was derived will be included in Section 4.5, Description of Streams Studied and Special Problems.

4.2 PARAMETER ESTIMATION

4.2.1 Manning's N-Values

Manning's n-values for each stream reach studied in the White Tanks/Agua Fria ADMS were determined by field reconnaissance and picture documentation, aerial photographs, comparisons to similar studied streams in Maricopa County, Arizona, and sound engineering judgement. Also references were made to Chow's "Open Channel Hydraulics" and "Roughness Characteristics of Natural Channels, Geological Survey Water - Supply Paper 1849". A number of photographs were taken to document typical "n" values for the streams studied in this area. This documentation can be found in Appendix D, under separate cover.

4.2.2 Expansion and Contraction Coefficients

Expansion and contraction coefficients were applied where flows approached and left structures or where flow constrictions or expansions were present due to natural conditions. Coefficients were chosen based upon criteria as stated in the HEC-2 "Water Surface Profiles Manual". These are average values applied where appropriate for each stream studied.

4.2.3 Hydraulic Jump/Drop Analysis

This section is not applicable.

4.3 CROSS-SECTION DESCRIPTION

Cross-sections for each stream studied in this ADMS were constructed from 1" = 400', 2' contour interval topographic mapping prepared for this study. Locations of cross-sections can be found on the floodplain maps submitted separately with this report.

4.4 CALIBRATION

No stream gages or recorded data are available in this area so no calibration was attempted.

4.5 DESCRIPTION OF STREAMS STUDIED AND SPECIAL PROBLEMS

Detailed floodplain and floodway analyses and approximate delineations were performed on many washes and drainage swales within the study area utilizing the U.S. Army Corps of Engineers' HEC-2 Water Surface Profiles computer program. The HEC-1 rainfall-runoff computer program was used to compute peak discharges throughout the watershed and, in some cases, was used to define ponding water surface elevations behind structures such as canals, railroads, and roadway embankments.

Detailed analyses were performed on the following washes using the HEC-2 program. Each was also numbered to provide identification for the HEC-2 input and output file. Refer to the 11" x 17" Floodplain Map on the following page for the locations of these delineations. The washes are as follows:

- Beardsley Canal Wash - Wash 1
- Cholla Wash - Wash 1A
- North Fork Cholla Wash - Wash 1A1
- Waterfall Wash - Wash 1B
- White Tank #3 Wash - Wash 2
- Bedrock Wash - Wash 3
- North Fork Bedrock Wash - Wash 3A
- Jackrabbit Trail Wash - Wash 4
- Tuthill Dike Wash - Wash 5
- Bulldozer Wash - Wash 5A
- Caterpillar Wash - Wash 5B
- Tractor Wash - Wash 5C
- Caterpillar Dike Wash - Wash 5D
- White Granite Wash - Wash 5E
- North Fork White Granite Wash - Wash 5E1
- 191st Avenue Wash - Wash 6
- Perryville Road Wash - Wash 7
- Bullard Wash - Wash 10
- AT&SF Railroad Channel - Agua Fria River to Greenway Road - Wash 12
- Lower El Mirage Wash - Wash 13
- Lower El Mirage Wash Tributary - Wash 13A
- Interstate 10 - Jackrabbit Trail to Tuthill Dike - Wash 14-2
- Litchfield Wash - Wash 21

Detailed studies of ponding areas utilizing the HEC-1 computer model were delineated for the following areas:

- Roosevelt Irrigation District Canal
- Southern Pacific Railroad
- Buckeye Canal
- Agua Fria River Dike - West Side
- Litchfield Park Detention Facility

Approximate delineations were computed using the HEC-2 model for the following areas:

- Cotton Lane Wash - Indian School Road to Olive Avenue - Wash 8
- Cotton Lane Wash - Olive Avenue to Waddell Road - Wash 9
- Bullard Wash - From Gila River to south end of Phoenix-Goodyear Municipal Airport - Wash 10
- Bullard Wash - From south end of Luke AFB to Reems Road - Wash 10
- Interstate 10 - Perryville Road to Jackrabbit Trail - Wash 14-3
- Interstate 10 - RID Canal to Cotton Lane - Wash 14-6
- Dysart Drain - Agua Fria River to Reems Road - Wash 17
- Atchison, Topeka and Santa Fe Railroad Spur - Northern Avenue north to Waddell Road - Wash 18, 19 and 20

Approximate delineations were also computed using normal depth calculations, approximation techniques, and the HEC-1 model for the following areas:

- Ponding behind White Tanks Flood Retarding Structures #3 and #4
- Ponding behind Interstate 10
- Ponding behind Airline Canal
- Approximate delineations of conveyance corridors behind Interstate 10
- Approximate delineations behind Southern Pacific Railroad where appropriate
- Approximate delineation of Bullard Wash breakout west of Estrella Parkway and south of State Route 80
- Approximate delineations of breakouts along the Dysart Drain onto Luke AFB
- Reems Road approximate delineation from Northern Avenue to Beardsley Road

Documentation and results for the HEC-2 analyses and approximate delineations can be found in Appendix J, Volumes 11, 12, and 13 of 15, under separate cover. Also, 8 1/2" x 11" Stream Profiles for the detailed studies can be found in Appendix K, under separate cover.

The following sections will provide a wash by wash description of modeling assumptions and any special problems associated with each analysis. Initially method 4 or method 6 was used to calculate the floodway. The floodway lines were then smoothed using method 1. The floodway was calculated to provide a conveyance corridor with a consistent width and therefore may not always reach the 1 foot rise in water surface elevation allowed for floodway encroachment. This was incorporated per instructions from the Flood Control District of Maricopa County.

4.5.1 Beardsley Canal Wash - Wash 1

Beardsley Canal Wash begins at the retention basin behind White Tanks Flood Retarding Structure #3 and continues north upstream along the west side of Beardsley Canal and terminates just south of McMicken Dam. This backwater analysis was begun at normal depth using the Slope-area method and the floodplain and floodway are tied into the 100-year approximate ponding water surface elevation behind White Tanks Flood Retarding Structure #3. This approximate elevation was taken from the HEC-1 stage-storage-discharge tables.

A few unique conditions exist on this wash. The first one located at Northern Avenue and Beardsley Canal. Approximately 1480 CFS breaks out east over Beardsley Canal at the three cross sections upstream of Northern Avenue. Two undersized culverts are located at Northern Avenue which causes the flows to break out and ponding to occur in the upstream right overbank. The floodplain and floodway are coincident in this area due to the requirements of maintaining existing flow conditions so as to not worsen the breakout situation. In other words, the diversion flow must not be increased due to encroachment and the subsequent rising water surface elevation behind the culverts unless steps are taken to increase the capacity of the culverts and/or raise the left bank protecting Beardsley Canal.

The second unique condition existing on this wash is located at Olive Avenue and Beardsley Canal. Culverts located under Olive Avenue are undersized and approximately 490 CFS breaks out east over Beardsley Canal. Floodplain limits just downstream of Olive Avenue are based upon upstream flow coming over the top of Olive Avenue. Ponding occurs in the right overbank upstream of the Olive Avenue Culverts. Again the floodplain and floodway are coincident in the upstream cross sections due to the requirements of maintaining the existing flow conditions as stated in the previous condition at Northern Avenue.

4.5.2 Cholla Wash - Wash 1A

Cholla Wash begins at the confluence with Beardsley Canal Wash approximately 1/2 mile north of Northern Avenue and continues upstream north and northwest into the White Tank Mountains. The beginning water surface elevation was taken from the Beardsley Canal Wash water surface profile at the confluence with Beardsley Canal Wash. No unique conditions or problems exist in this reach.

4.5.3 North Fork Cholla Wash - Wash 1A1

North Fork Cholla Wash begins at the confluence with Cholla Wash and continues upstream north and northwest into the White Tank Mountains. The beginning water surface elevation was taken from the HEC-2 analysis of Cholla Wash at the confluence of Cholla Wash and North Fork Cholla Wash. No other conditions or problems exist in this reach.

4.5.4 Waterfall Wash - Wash 1B

Waterfall Wash begins at the confluence with Beardsley Canal Wash and continues upstream west and northwest into the White Tank Mountains. The beginning water surface elevation was taken from the HEC-2 analysis of Beardsley Canal Wash at the confluence of Waterfall Wash with Beardsley Canal Wash. The cross section is extended at the beginning of this reach as flow will break out over Beardsley Canal to the east. Approximately 490 CFS will break out; refer to discussion for Beardsley Canal Wash.

Ponding occurs in the right overbank upstream of the Olive Avenue culverts. The floodplain and floodway will be coincident in this location due to the requirements of maintaining existing flow conditions. This is the same condition that was described on Beardsley Canal Wash.

Divided flow occurs between cross sections X1 = 0.873 and X1 = 1.223. The right overbank flow is effective due to the upstream inflow at cross section X1 = 1.352. No other unique conditions or problems exist in this reach.

4.5.5 White Tanks #3 Wash - Wash 2

White Tanks #3 Wash begins at the White Tanks Flood Retarding Structure #3 retention basin and continues upstream north and northwest into the White Tank Mountains. The water surface elevation for the beginning of this run was computed at normal depth by the slope-area method. The floodplain and floodway will be matched into the 100-year ponding water surface elevation behind White Tanks Structure #3 as taken from the HEC-1 model.

This wash flows through a small retention basin created by testing equipment on the Caterpillar Proving Grounds at approximately cross sections X1 = 0.452 to X1 = 0.551. Flow is effective through this basin. Also, the wash flows through a break in the dike surrounding the perimeter of the Caterpillar Proving Grounds at cross section X1 = 1.862. No other unique conditions or problems exist in this reach.

4.5.6 Bedrock Wash - Wash 3

Bedrock Wash begins in the detention basin behind White Tanks Structure #3 and continues upstream west and northwest into the White Tank Mountains. The slope-area method was used to begin the backwater analysis at normal depth. Both the floodplain and floodway were matched into the 100-year ponding water surface elevation behind White Tanks Structure #3 as taken from the HEC-1 model. This wash flows through an earthen embankment created by Case Proving Grounds equipment at cross section X1 = 0.395. No other unique conditions exist in this reach.

4.5.7 North Fork Bedrock Wash - Wash 3A

North Fork Bedrock Wash begins at the confluence with Bedrock Wash and continues upstream west and northwest into the White Tank Mountains. The beginning water surface elevation was taken from HEC-2 analysis of Bedrock Wash where it joins the confluence with North Fork Bedrock Wash. No unique conditions or problems exist on this wash.

4.5.8 Jackrabbit Trail Wash - Wash 4

Jackrabbit Trail Wash begins in the detention basin behind White Tanks Flood Retarding Structure #4 and continues upstream north along the west side of Jackrabbit Trail to the limit of study at Medlock Drive, approximately 1000 feet north of Camelback Road Extended. A split flow analysis was run along the length of Jackrabbit Trail to compute final discharges that would be used in the final HEC-2 analysis. This split flow analysis is included in Appendix J, Volume 11 of 15, under separate cover, for review.

The backwater analysis was started at normal depth using the slope-area method. Both the floodplain and floodway are matched into the 100-year ponding water surface elevation behind White Tanks Structure #4 as obtained from the HEC-1 model. There are a number of areas along Jackrabbit Trail where flows exceed the capacity of the channel and overtop the road. Following is a list of the areas where these flows will break out:

1. Approximately 250 CFS will break out to the east over Jackrabbit Trail from cross section X1 = 0.440 to X1 = 0.566. Break out flows then continue overland to the southeast as sheet flow.

2. There are five 10' x 4' box culverts located underneath the eastbound off-ramp of Interstate 10 and four 12' x 4.5' box culverts are located under the westbound on-ramp of Interstate 10. A concrete lined channel connects these two culverts. There are also four 12' x 4.5' box culverts conveying flows underneath McDowell Road. Some flow will break out to the east over Jackrabbit Trail at cross section X1 = 1.159 to X1 = 1.348 upstream of McDowell Road, however these are very small amounts (less than 10 CFS). Breakout flows will then continue overland to the southeast as sheet flow.

3. Approximately 390 CFS will breakout to the east over Jackrabbit Trail from cross section X1 = 1.631 to X1 = 1.818. Break out flows continue overland to the southeast as sheet flow. The wash flows through a man-made retention basin east of the Caterpillar Proving Grounds buildings at cross sections X1 = 2.973 to X1 = 3.154.

4. Approximately 152 CFS will break out to the east over Jackrabbit Trail between cross section X1 = 4.016 and X1 = 4.152.

5. The last break out flow that occurs on this wash is at cross section X1 = 4.152 where 187 CFS breaks out over the top of Jackrabbit Trail.

No other unique conditions or problems exist on this reach.

4.5.9 Tuthill Dike Wash - Wash 5

Tuthill Dike Wash begins in the detention basin behind White Tanks Flood Retarding Structure #4 and continues west upstream approximately 1/2 mile to the Tuthill Road alignment, then turns north and continues along the west side of Tuthill Dike and terminates approximately 1/2 mile north of Camelback Road Extended. The backwater analysis was started at normal depth using the slope-area method behind White Tanks Flood Retarding Structure #4. Both the floodplain and floodway were then matched into the 100-year ponding water surface elevation behind White Tanks Structure #4 as obtained from the HEC-1 model.

Flow is conveyed through four 10' x 4' box culverts underneath Interstate 10. The capacity of these culverts is not sufficient to handle the flows collected at this point, and approximately 1440 CFS flows over the dike to the east.

The effective flow option was incorporated for the five cross sections upstream of the Interstate 10 culverts where there is a large ponding area in the right overbank. Actual mapped floodplain limits correspond to the calculated water surface elevation and are shown at the correct limits on the floodplain workmap. The HEC-2 model will only show the effective flood limits.

The wash flows through a man-made retention basin on the Caterpillar Proving Grounds at cross section X1 = 1.313 to X1 = 1.362. Effective flow limits are imposed to model this situation correctly. The map limits are shown on the floodplain map to correctly depict the actual ponding area. This is also the confluence with Bulldozer Wash.

The confluence of Caterpillar Wash with Tuthill Dike Wash is located at cross section X1 = 2.563 and the confluence of Tractor Wash with Tuthill Dike Wash is located at cross section X1 = 3.250.

The wash flows through another man-made retention basin on the Caterpillar Proving Grounds at cross section X1 = 3.344 to X1 = 3.535 and flow is effective in this area. The limit of the study at the upstream end of Tuthill Dike Wash is also the confluence with Caterpillar Dike Wash at cross section X1 = 4.725.

No other unique conditions or problems exist on this wash.

4.5.10 Bulldozer Wash - Wash 5A

Bulldozer Wash begins at the confluence with Tuthill Dike Wash and continues upstream west and northwest into the White Tank Mountains. The beginning water surface elevation was taken from the Tuthill Dike Wash HEC-2 analysis where is joins the confluence with Bulldozer Wash.

The wash flows through a man-made retention basin on the Caterpillar Proving Grounds at cross section X1 = 0.000 to X1 = 0.120. Flow in the left overbank from cross sections X1 = 0.705 to X1 = 0.810 is effective due to upstream inflow. There is a minor retention basin on the Caterpillar Proving Grounds in the right overbank at cross section X1 = 0.988 to X1 = 1.085. The flow in this reach is effective due to upstream inflow. The wash flows through an opening in the Caterpillar Proving Grounds perimeter dike at cross section X1 = 1.224. No other unique conditions or problems exist in this reach.

4.5.11 Caterpillar Wash - Wash 5B

Caterpillar Wash begins at the confluence with Tuthill Dike Wash and is located approximately 1/4 mile north of Thomas Road Extended. This wash continues upstream west and northwest into the White Tank Mountains. The beginning water surface elevation was computed from the Tuthill Dike Wash HEC-2 model where it joins at the confluence with Caterpillar Wash.

Divided flow occurs from cross section X1 = 0.164 to X1 = 0.384. This situation is due to upstream inflow and the condition of a wide undefined floodplain. This flow is effective.

Ponding occurs in the right overbank between cross sections X1 = 0.898 and X1 = 0.971 and this flow is assumed to be ineffective. Ponding also occurs in the right overbank at cross section X1 = 1.188 and is defined as ineffective flow and limits of effective flow are designated by encroachment stations identified on the ET 6.1 record.

A divided flow situation occurs from cross section X1 = 1.553 to X1 = 1.916. These areas are effective, however, due to upstream inflow at X1 = 1.984. No other unique conditions or problems exist in this reach.

4.5.12 Tractor Wash - Wash 5C

Tractor Wash begins at the confluence with Tuthill Dike Wash at approximately the intersection of Indian School Road Extended and Tuthill Dike. This wash then continues upstream to the northwest into the White Tank Mountains. The beginning water surface elevation was taken from the HEC-2 analysis of Tuthill Dike Wash where it joins the confluence with Tractor Wash. The cross section is extended at X1 = 0.037, however this can be disregarded as the floodplain ties into the Tuthill Dike Wash floodplain. Ponding occurs in the right overbank area at cross section X1 = 0.305 and is ineffective flow as defined by expansion criteria stated in the HEC-2 manual.

Recent flooding evidence indicates that the road will wash out at cross section X1 = 1.210 and flow will continue to the east. This is the reason that the entire flow is taken to the east at this cross section. Topographic mapping does not reflect this situation and the extended cross section message should be disregarded at this location.

The wash flows through a man-made retention basin on the Caterpillar Proving Grounds at cross section X1 = 1.531. Flow is effective in this basin. No other unique conditions or problems exist on this wash.

4.5.13 Caterpillar Dike Wash - Wash 5D

Caterpillar Dike Wash begins at the confluence with Tuthill Dike Wash and continues upstream to the west to the limit of study at the dike of a man-made retention basin on the Caterpillar Proving Grounds. The beginning water surface elevation was taken from the HEC-2 analysis of Tuthill Dike Wash where it joins the confluence with Caterpillar Dike Wash.

Cross sections X1 = 0.084 to X1 = 0.273 are extended due to the overtopping of the man-made dike located in this area. The wash flows through a roadway dip section in the Caterpillar Proving Grounds at cross sections X1 = 0.416 to X1 = 0.432. No other unique conditions or problems exist on this wash.

4.5.14 White Granite Wash - Wash 5E

White Granite Wash begins in a retention basin on the Caterpillar Proving Grounds and continues upstream west into the White Tank Mountains. The beginning water surface elevation is calculated by using the slope-area method to calculate a normal depth elevation. The floodplain and floodway are then matched into the 100-year ponding water surface elevation in the retention basin as defined by the HEC-1 model.

A divided flow situation occurs at cross sections X1 = 0.324 to X1 = 0.408. This is due to upstream inflow and therefore makes the overbank flow effective. The confluence with North Fork White Granite Wash is located at cross section X1 = 0.585. No other unique conditions or problems exist in this reach.

4.5.15 North Fork White Granite Wash - Wash 5E1

North Fork White Granite Wash begins at the confluence with White Granite Wash and continues upstream northwest into the White Tank Mountains. The beginning water surface elevation was taken from cross section X1 = 0.585 from the White Granite Wash HEC-2 run which is the confluence with North Fork White Granite Wash. No other unique conditions or problems exist on this wash.

4.5.16 191st Avenue Wash - Wash 6

191st Avenue Wash was delineated beginning at Interstate 10, approximately 1/2 mile east of Jackrabbit Trail and then continues upstream north along the 191st Avenue alignment to the limit of study at approximately Bethany Home Road Extended. Discharges for the final HEC-2 run are taken from the split flow analysis which is included for review in Appendix J, Volume 12 of 15. The discharges vary frequently throughout the reach based on the number of breakouts that occur. Refer to the output from the HEC-2 split flow analysis run to verify discharges within the final HEC-2 run. Documentation is also provided within the split flow run to show how discharges were calculated in each reach.

A revised HEC-1 analysis was also computed in this particular area to calculate discharges minus the break out flows as they occur in each mile stretch. This analysis was performed so that discharges could be computed on a mile to mile basis along 191st Avenue Wash taking out diversions at each mile location. This is not reflected in the HEC-1 analysis that is submitted for review, but the results do not differ enough to warrant a complete redo of the HEC-1 model.

Divided flows, wherever they occur, are due to the braided channel type of the wash. Encroachment stations are generally limited to floodplain stations due to the potential of increasing the break out flows over the left bank.

Both the floodplain and floodway are matched into the ponding water surface elevation computed in the HEC-1 model behind Interstate 10 at cross sections X1 = 0.000. The first cross section is located approximately 200 feet north of Interstate 10 and approximately 101 CFS will break out and continue overland as sheet to the southeast from cross section X1 = 0.000 to X1 = 0.863. Approximately 149 CFS will break out between cross sections X1 = 1.184 and X1 = 1.654 and flows will continue overland to the southeast as sheet flow.

Approximately 405 CFS will break out between cross sections X1 = 2.140 and X1 = 2.241 and flows will also continue to the southeast overland as sheet flow. Approximately 354 CFS will break out between cross sections X1 = 2.555 and X1 = 3.161 and flows will continue overland to the south as sheet flow. And finally, approximately 472 CFS will break out between cross sections X1 = 3.161 to X1 = 3.829 and flows will continue overland to the southeast as sheet flow. No other unique conditions or problems exist in this reach.

4.5.17 Perryville Road Wash - Wash 7

The beginning limit of study for Perryville Road Wash is located 1/2 mile west of Citrus Road along the north side of Camelback Road in an agricultural reservoir. The wash continues upstream west 1/2 mile to Perryville Road, then continues north along Perryville Road to the limit of study at Northern Avenue where flows break out from the Beardsley Canal Wash. The starting water surface elevation and discharges are taken from a HEC-2 split flow analysis. This split flow analysis is included in Appendix J, Volume 12 of 15, and should be referred to when reviewing the discharges as input into the final HEC-2 model.

The detailed study begins at an agricultural reservoir located 1/2 mile west of Citrus Road along the north side of Camelback Road. Both the floodplain and floodway limits are coincident in most cases, however, small incremental rises in water surface elevation are allowed to smooth the floodway without overtopping the limiting elevations along the roadway.

Where cross sections are extended, the split flow analysis was used to calculate the actual discharge that would be contained and then conveyed to the south and east along Perryville Road.

Approximately 830 CFS breaks out of the agricultural reservoir between cross sections X1 = 0.000 to X1 = 0.177 and continues south as sheet flow. Approximately 385 CFS breaks out from cross sections X1 = 0.938 to X1 = 1.225. Break out flows continue overland to the southeast as sheet flow. Approximately 108 CFS breaks out between cross sections X1 = 1.548 to X1 = 1.737. Break out flows continue overland to the southeast as sheet flows.

At cross section X1 = 3.549 the wash backs up behind the existing irrigation canal and weirs over the top of the canal and continues to the southeast. Both floodplain and floodway widths are coincident for the next five upstream cross sections due to ponding constraints in this area. The limit of the detailed study is located at Northern Avenue and Beardsley Canal Wash where diversion flows break out from the west over Beardsley Canal. No other unique conditions or problems exist in this reach.

4.5.18 Cotton Lane Wash - Indian School Road to Olive Avenue - Wash 8

This is an approximate study that begins at the intersection of Indian School Road and the AT&SF Railroad spur and continues upstream north along the west side of the AT&SF Railroad spur to the limit of the approximate study at Olive Avenue. Major concentration points were located at every mile intersection point along the AT&SF Railroad spur. Diversions were then computed at these points to divert flow that exceeds the capacity behind the railroad. However, there are locations along the length of the railroad where flow will go over the top between the mile intersection points. To simplify the hydrology model, these diversions were combined and assumed to occur at the major mile intersection points.

Two profiles were run for this particular wash to calculate an approximate floodplain delineation. Discharges for the first profile were as derived from the 100-year, 24-hour HEC-1 model. Discharges for the second profile are the capacities which limit the water surface elevation over the railroad to no more than 1.5 feet in any given area and are then used for the approximate floodplain delineation. Breakout flows over the railroad, along the length of the wash, will continue south in the ditch between Cotton Lane and the railroad, and along Cotton Lane itself. Any flows that exceed the capacity of Cotton Lane will continue overland to the southeast as sheet flow.

No other unique conditions or problems exist on this wash.

4.5.19 Cotton Lane Wash - Olive Avenue to Waddell Road - Wash 9

This is an approximate study that begins at Olive Avenue and continues upstream north along the west side of the AT&SF Railroad spur to approximately 1/2 mile north of Waddell Road which is the limit of this approximate study. Major concentration points were located at every mile intersection point along the AT&SF Railroad spur. Diversions were then computed at these points to divert flows that would exceed the capacity behind the railroad spur. However, there are locations along the length of the railroad where flow will go over the top between the mile intersection points. To simplify the hydrology model, these diversions were combined and assumed to occur at the major mile intersection points.

Two profiles are computed to calculate an approximate delineation for this wash. Discharges for the first profile are as derived from the 100-year, 24-hour HEC-1 model. Discharges for the second profile and the capacities which limit the water surface elevation extension over the top of the railroad to no more than 1.5 feet are then used for the approximate floodplain delineation. Break out flows over the railroad, along the length of the wash, will continue south in a ditch between Cotton Lane and the railroad and along Cotton Lane itself. Any flows that exceed the capacity of Cotton Lane will continue overland to the southeast as sheet flow. No other unique conditions or problems exist on this wash.

4.5.20 Bullard Wash - Wash 10

Bullard Wash delineations begin at Buckeye Canal and the Agua Fria River and continue north upstream to Reems Road and Northern Avenue. This is a combination detailed and approximate study where the approximate study is also documented by the HEC-2 model.

Discharges for the approximate study from cross sections X1 = 0.000 to X1 = 2.297 were originally taken from the HEC-1 model, however they have been reduced to reflect breakouts that occur in this stretch and are based upon the capacity of the channel plus a maximum of 1 foot over the top where capacity is exceeded. These situations are documented throughout the HEC-2 model.

Due to the complexity of the downstream reach, cross sections X1 = 0.000 to X1 = 2.297, an approximate delineation has been shown. However, the HEC-2 model was set up to include these cross sections to aid in producing a reasonable approximate delineation. This approximate delineation is as shown on the maps and is based on engineering judgement reflecting existing topography and using information from the HEC-2 model to estimate where breakout flows will occur. Also, an approximate delineation has been shown for the uppermost reach of the Bullard Wash starting at cross section X1 = 10.269 to X1 = 14.023. The HEC-2 model was also utilized to estimate the approximate delineation in this reach.

A detailed analysis was performed on Bullard Wash from cross sections X1 = 2.371 to X1 = 6.320 and from X1 = 9.189 to X1 = 10.197. The area between cross section 6.320 and cross section 9.189 is currently under construction. No floodplain has been prepared for this area of Bullard Wash. However, the HEC-2 analysis was continued based on existing conditions at the time of the aerial mapping to compute water surface elevations upstream and downstream of this reach. The discharge in field 4 of the QT cards is the actual discharge derived from the 100-year, 24-hour HEC-1 model. This discharge is for comparison purposes only and no profile is being computed for it. Discharges for profiles 1 and 2 are used to delineate the floodplain and floodway respectively. These discharges are based upon the capacity of the channel plus a maximum of 1 foot over the top where capacity is exceeded. These situations are documented throughout the HEC-2 model.

Approximate floodplain delineations begin at cross section X1 = 0.000. This discharge has been reduced to the capacity of the channel plus a maximum of 1 foot over the limiting elevation of the right overbank. Exceeded flow will continue as sheet flow to the west. Any water surface elevation extension at cross sections X1 = 0.047 to X1 = 0.616 is due to the limiting elevation in the right overbank. Divided flow occurs between cross section X1 = 0.384 and X1 = 0.668. These flows are effective, however, due to upstream inflow at cross section X1 = 0.668. Any water surface elevation extensions from cross sections X1 = 0.668 to X1 = 1.378 is due to the limiting elevation at the left overbank.

The discharge at cross section X1 = 1.081 has been reduced by the same amount that exceeded the capacity of the channel at the upstream reach from cross section X1 = 1.474 to X1 = 1.823. That exceeded flow will return at approximately cross section X1 = 0.964. Flows will weir over the top of the Southern Pacific Railroad due to the undersized culverts located under the railroad in this reach.

A divided flow situation occurs at cross section X1 = 1.153 to X1 = 1.435, however, these flows are effective due to upstream inflow at cross section X1 = 1.435.

The discharge at cross section X1 = 1.474 has been reduced to the capacity of the channel plus a maximum of 1 foot over the limiting elevation in the right overbank. Exceeded flow will return approximately at cross section X1 = 0.964. Any water surface elevation extension from cross section X1 = 1.474 to X1 = 1.823 is due to the limiting elevation at the right overbank.

A divided flow situation occurs between cross sections X1 = 1.893 and X1 = 2.371. These flows are effective due to upstream inflow. Extended water surface elevations between cross sections X1 = 2.058 to X1 = 2.185 are due to the limiting elevations in the left overbank.

The approximate floodplain delineation ends at cross section X1 = 2.297 and the detailed floodplain and floodway analysis begins at cross section X1 = 2.371.

Divided flow occurs at cross section X1 = 2.680, however, these flows are effective due to upstream inflow at cross section X1 = 2.883. Divided flow occurs for the next two upstream cross sections beginning with cross section X1 = 3.291 and these flows are effective due to upstream inflow at cross section X1 = 3.602.

Divided flow again occurs for the next two upstream cross sections beginning at cross section X1 = 3.702 and these flows are also effective due to upstream inflow at cross section X1 = 4.101. Bullard Wash flows under a seven span bridge at Interstate 10 at cross sections X1 = 5.430 to X1 = 5.460.

Divided flow occurs for the next two upstream cross sections beginning at X1 = 5.727 and these flows are effective due to upstream inflow at cross section X1 = 5.960.

Divided flow occurs at cross section X1 = 6.674 and this flow is effective due to upstream inflow at cross section X1 = 6.877.

The Roosevelt Irrigation District Canal is located at cross section X1 = 6.864. Flow will weir over the canal and go through a small overchute at this point. Effective flow limits are imposed for the next four upstream cross sections.

The water surface elevation extension at cross section X1 = 7.045 is due to a limiting of elevation at the left overbank. Divided flow occurs for the next four upstream cross sections.

Divided flow also occurs for the next three upstream cross sections beginning at X1 = 8.636. These flows are effective due to upstream inflow at cross section X1 = 8.965.

The discharge at cross section X1 = 9.641 has been reduced by the same amount that exceeded the capacity of the channel at the upstream reach from cross section X1 = 10.629 to X1 = 10.639. The exceeded flow will return at cross section X1 = 9.544.

The discharge at cross section X1 = 9.898 has been reduced by the same amount that exceeded the capacity of the channel at the upstream reach from cross section X1 = 10.269 to X1 = 10.639. That exceeded flow will return at cross section X1 = 9.544.

The detailed floodplain and floodway analysis ends at cross section X1 = 10.197. An approximate floodplain delineation begins at cross section X1 = 10.269 and will continue upstream to Northern Avenue. This discharge has been reduced to the capacity of the channel plus a maximum of 1 foot over the limiting elevation in the right overbank from cross section X1 = 10.493 to X1 = 10.639. The exceeded flow will return at cross section X1 = 9.544.

Water surface elevation extensions from cross sections X1 = 10.493 to X1 = 10.639 are due to the limiting elevation of the right overbank. Water surface elevation extensions at cross sections X1 = 12.150 to X1 = 12.534 are also due to the limiting elevation of the left overbank.

The discharge at cross section X1 = 12.633 has been reduced to the capacity of the channel plus a maximum of 1 foot over the limiting elevation in the left overbank. The exceeded flow will return at cross section X1 = 12.534. Any water surface elevation extensions at cross sections X1 = 12.633 to X1 = 13.161 are due to the limiting elevation of the left overbank.

Divided flow occurs over the next three upstream cross sections beginning at X1 = 13.248, however, these flows are effective due to upstream inflow at cross section X1 = 13.556.

Divided flow occurs at cross section X1 = 13.556 and the next four upstream cross sections due to a sheet flow situation in this reach.

The approximate delineation for Bullard Wash ends at Northern Avenue at cross section X1 = 14.023. No other unique conditions or problems exist on this reach.

4.5.21 AT&SF Railroad Channel - Agua Fria River to Greenway Road - Wash 12

The starting water surface elevation was started at normal depth utilizing the slope-area method. The Atchison Topeka and Santa Fe Railroad Channel begins at a bridge crossing under the AT&SF Railroad just west of the Agua Fria River bridge and continues upstream to the northwest along the north side of the AT&SF Railroad to the limit of study at Greenway Road. Both the floodplain and floodway match into existing 100-year delineations on the Agua Fria River computed by Jerry R. Jones and Associates, on February 6, 1989, at approximately cross section X1 = 1.085 and X1 = 0.231 respectively. No other unique conditions or problems exist on this channel.

4.5.22 Lower El Mirage Wash - Wash 13

Lower El Mirage Wash begins at the Agua Fria River and continues upstream northwest to the limit of study at the Atchison Topeka and Santa Fe Railroad spur. The beginning water surface elevation was started at normal depth utilizing the slope-area method. Both the floodplain and floodway match into the existing Agua Fria River delineations at cross sections X1 = 0.386 and X1 = 0.153 respectively. The entire cross section is effective flow for the next two upstream cross sections due to upstream inflow at cross section X1 = 0.153.

The wash flows through the Pueblo El Mirage Golf Course between cross sections X1 = 0.000 to X1 = 0.696. The wash then flows through a series of agricultural tanks between cross sections X1 = 0.985 to X1 = 1.251.

The entire cross sectional flow at cross section X1 = 1.716 is effective due to upstream inflow at cross section X1 = 1.817. A divided flow will occur at cross section X1 = 2.284 due to overtopping of the main channel bank into the lower overbank, therefore, flow is effective. The limit of the detailed study is located at the AT&SF Railroad spur at cross section X1 = 2.571. No other unique conditions or problems exist in this reach.

4.5.23 Lower El Mirage Wash Tributary - Wash 13A

Lower El Mirage Wash Tributary begins at the confluence with Lower El Mirage Wash and continues upstream north and northwest to the limit of the study at the intersection of Litchfield Road and Greenway Road. The beginning water surface elevation is taken from the HEC-2 model of Lower El Mirage Wash at cross section X1 = 1.259 which is the confluence with Lower El Mirage Wash Tributary.

Cross section X1 = 1.363 is located at the Atchison Topeka and Santa Fe Railroad spur. The wash flows through an agricultural tank between cross sections X1 = 2.302 to X1 = 2.497. The limit of the detailed study is located at cross section X1 = 3.257 which is the intersection of Greenway Road and Litchfield Road. No other unique conditions or problems exist in this reach.

4.5.24 Interstate 10 - Jackrabbit Trail to Tuthill Dike - Wash 14-2

This delineation begins at Jackrabbit Trail and continues west along the north side of Interstate 10 to Tuthill Road Extended. The beginning water surface elevation was computed at normal depth utilizing the slope-area method. The floodplain and floodway are matched into the Jackrabbit Trail Wash floodplain and floodway delineations.

The discharge at cross section X1 = 0.931 is equal to the diversion over Tuthill Dike at Interstate 10. Discharges are reduced accordingly due to flow through Interstate 10 culverts as the wash continues downstream. No other unique conditions or problems exist in this reach.

4.5.25 Interstate 10 - Perryville Road to Jackrabbit Trail - Wash 14-3

This is an approximate study that begins at Perryville Road along the north side of Interstate 10 and continues upstream west along the north side of Interstate 10 to Jackrabbit Trail. This approximate delineation is documented by a HEC-2 run. The approximate delineation is matched into the floodplain delineated for 191st Avenue Wash at cross sections X1 = 0.338 to X1 = 0.447. No other unique problems or conditions exist in this reach.

4.5.26 Interstate 10 - R.I.D. Canal to Cotton Lane - Wash 14-6

This is an approximate delineation that begins at the Roosevelt Irrigation District Canal crossing at Interstate 10 and continues upstream along the north side of Interstate 10 to Cotton Lane. The approximate floodplain delineation is matched into the 100-year ponding water surface elevation behind Interstate 10 and the Roosevelt Irrigation District Canal. This approximate floodplain delineation is documented by a HEC-2 analysis. No other unique conditions or problems exist in this reach.

4.5.27 Interstate 10 - Tuthill Dike to Approximately 1 1/2 Miles West Along the North Side of Interstate 10 -

Approximate delineations were performed on the conveyance corridors and ponding areas in this reach. The approximate delineations begin at Tuthill Dike Road on the north side of Interstate 10 and continue 1 1/2 miles to the west along the north side of Interstate 10. Approximate delineations for the ponding areas were defined by stage-storage-discharge tables within the HEC-1 model. These tables were computed by hand using culvert discharge nomographs and stage storage relationships computed from the topographic mapping.

Ponding areas occur behind Interstate 10 with each associated culvert crossing. Small dikes help to confine the flow to these culverts. Once these dikes are overtopped flow will continue to the east along the north side of Interstate 10 to the next ponding area. Approximate delineations were calculated using Manning's Equation for normal depth flow to connect these ponding areas. Discharges for these calculations were taken from the HEC-1 model.

4.5.28 Interstate 10 - Citrus Road to Perryville Road

This was an approximate study on ponding areas behind Interstate 10 in this reach and approximate delineations were conducted for the connecting conveyance corridors. The approximate delineations for these ponding areas and connecting conveyance corridors begin at Citrus Road and continue upstream to the west along the north side of Interstate 10 to Perryville Road.

Ponding water surface elevations were taken from the HEC-1 computer model from stage-storage-discharge tables, and normal depth calculations were used to delineate approximate floodplains which were then connected between the ponding water surface elevations.

Discharges for the normal depth calculations were taken from the HEC-1 model results at pertinent points of concentration and were then prorated upstream to a definite separation of watershed boundaries or to the next upstream point of concentration. Channel configuration data was taken from the 1" = 400' scale topographic mapping.

4.5.29 Interstate 10 - Cotton Lane to Citrus Road

This was an approximate study of ponding areas behind Interstate 10 and approximate delineations were also performed for the connecting conveyance corridors between these ponding areas. The approximate delineations begin at Cotton Lane on the north side of Interstate 10 and continue upstream to the west to Citrus Road.

Ponding water surface elevations were taken from the HEC-1 computer model from the stage-storage-discharge tables and normal depth calculations were then used to delineate approximate floodplains for the connecting conveyance corridors. Stage-storage-discharge tables were computed from culvert nomographs and stage storage relationships were computed from the topographic mapping.

Discharges for the normal depth calculations were taken from the HEC-1 model results at pertinent points of concentration and were then prorated upstream to a definite separation of watershed boundaries or to the next upstream point of concentration. Channel configuration data was taken from the 1" = 400' scale topographic mapping.

4.5.30 Interstate 10 - Estrella Parkway to Sarival Avenue

This was an approximate study of ponding areas behind Interstate 10 and approximate delineations were also performed for the connecting conveyance corridors between these ponding areas. The approximate delineations begin at Estrella Parkway on the north side of Interstate 10 and continue west upstream to Sarival Avenue.

Ponding water surface elevations were taken from the HEC-1 computer model from the stage-storage-discharge tables and normal depth calculations were then used to delineate approximate floodplains for the connecting conveyance corridors. Stage-storage-discharge tables were computed from culvert nomographs and stage storage relationships were computed from the topographic mapping.

Discharges for the normal depth calculations were taken from the HEC-1 model results at pertinent points of concentration and were then prorated upstream to a definite separation of watershed boundaries or to the next upstream point of concentration. Channel configuration data was taken from the 1" = 400' scale topographic mapping.

4.5.31 Interstate 10 - Bullard Wash to Estrella Parkway

This is an approximate delineation that begins at Bullard Wash on the north side of Interstate 10 and continues west upstream to Estrella Parkway. The HEC-2 model for Bullard Wash was used to confirm the ponding water surface elevation behind Interstate 10. Normal depth calculations were then used to calculate an approximate floodplain to the west and this was then matched into the Bullard Wash floodplain delineation.

Channel configuration data was taken from 1" = 400' topographic mapping. Discharges for the normal depth calculations were taken from the HEC-1 model results at pertinent points of concentration.

4.5.32 Interstate 10 - Detention Basin Delineations Between Dysart Road and Bullard Avenue

Detailed ponding water surface elevation delineations begin on the north side of Interstate 10 at Dysart Road and continue upstream to approximately Bullard Avenue in the detention basins behind Interstate 10. The HEC-1 model was used to compute stage-storage-discharge relationships for each of the detention basins behind Interstate 10 in this area. A backwater analysis was then computed through the 48" stormdrain outlet to calculate an actual water surface elevation in the detention basins. This is documented in the Appendix J, Volume 12, of 15.

To compute the final 100-year water surface elevation of the detention basins, a backwater analysis was performed by hand through the 48" stormdrain that outlets downstream into the Agua Fria River. A total head loss was calculated based on friction slope and junction losses in the storm drain. This head loss was then added to the downstream 10-year water surface elevation in the Agua Fria River to compute the actual 100-year ponding water surface elevation in the detention basins.

Since the four Interstate 10 detention basins are in series the assumption was made that the water surface elevation of the downstream basin was the controlling water surface elevation for all the basins, even though timing is slightly different for each of the peaks in each of the basins. This is a conservative assumption as the HEC-1 model cannot accurately model this situation.

4.5.33 Dysart Drain - Wash 17

An approximate study was performed on Dysart Drain beginning on the west side of the Agua Fria River, 1/2 mile north of Glendale Avenue, and continuing west along the north boundary of Luke Air Force Base to the limit of study at Reems Road and Northern Avenue. Discharges are taken from the HEC-1 model and these include many diversions which occur across the top of Dysart Drain as the capacity of the channel is exceeded. These diverts were verified with the HEC-2 model and then plugged back into the new HEC-2 run to calculate an approximate floodplain.

The backwater analysis was started at normal depth using the slope-area method and the approximate floodplain is matched into the existing 100-year Agua Fria River floodplain at this location.

Ponding occurs in the left overbank area for the next two upstream cross sections beginning at cross section X1 = 0.350. Flows are assumed noneffective in this reach.

Ponding occurs in the left overbank area for the next nine upstream cross sections beginning at cross section X1 = 1.533. Flows are assumed noneffective outside the expansion and contraction limits as stated in the HEC-2 manual. Extended cross sections from cross section X1 = 1.533 to X1 = 1.937 are confined by a wall in the south bank of Dysart Drain. Therefore, the approximate floodplain is limited at that point. Flows break out to the south over Dysart Drain between cross sections X1 = 1.937 to X1 = 2.153. Extended cross section in this area indicate channel capacity is exceeded and approximately 600 CFS is diverted south at this point.

Flows also break out to the south over Dysart Drain between cross sections X1 = 2.834 to X1 = 3.317. Approximately 250 CFS is diverted onto Luke Air Force Base in this reach. Extended cross sections in this area indicate channel capacity is exceeded.

Flows break out to the south over Dysart Drain between cross sections X1 = 4.070 to X1 = 4.465. Approximately 1536 CFS is diverted south. Extended cross sections in this area reflect overtopping of Northern Avenue. The limit of the approximate study is located at cross section X1 = 4.565.

4.5.34 Atchison Topeka and Santa Fe Railroad Wash - Northern Avenue to 1/2 Mile North of Olive Avenue - Wash 18

An approximate delineation was performed on this wash and begins approximately 1/4 mile west of Litchfield Road on Northern Avenue and continues upstream north along the west side of the Atchison Topeka and Santa Fe Railroad spur to 1/2 mile north of Olive Avenue. The approximate floodplain delineation is matched into the approximate floodplain delineation for Dysart Drain.

Extended water surface elevations within this reach indicate the AT&SF Railroad will be overtopped as that is the controlling grade. Breakout flows will continue south and then east in a ditch on the east side of the railroad.

A box culvert is located at cross section X1 = 0.943 at Olive Avenue. There is a 5.3 foot drop in invert elevation from the upstream cross section to the invert of the culvert. This is due to the drop structure located just upstream of the culvert entrance. This drop structure is drowned out during the 100-year event.

4.5.35 Atchison Topeka and Santa Fe Railroad Wash - 1/2 Mile West to 1/2 Mile East of Litchfield Road, 1/2 Mile North of Olive Avenue - Wash 19

This is an approximate delineation beginning at the center of Section 28, Township 3 North, Range 1 West, and continuing upstream east to approximately the center of Section 27, Township 3 North, Range 1 West. This approximate delineation is documented by a HEC-2 analysis. Discharges for the first profile in the HEC-2 analysis are as derived from the 100-year, 24-hour HEC-1 model. Discharges in the second profile are the capacities which limit the water surface elevation extension over the Atchison Topeka and Santa Fe Railroad spur to no more than 1 foot, and are used for an approximate floodplain delineation. Breakout flows over the railroad along the length of the railroad, will continue overland to the south and east as sheet flow.

4.5.36 Atchison Topeka and Santa Fe Railroad Wash - Approximately 1/2 Mile West of Litchfield Road and 1/2 Mile South of Peoria Avenue to 3/4 Mile North of Cactus Road - Wash 20

This is an approximate delineation that begins at approximately the center of Section 27, Township 3 North, Range 1 West, and continues north upstream along the west side of the Atchison Topeka and Santa Fe Railroad spur to approximately the center of Section 15, Township 3 North, Range 1 West. The approximate floodplain was documented by a HEC-2 analysis and discharges of the first profile are as derived from

the 100-year, 24-hour HEC-1 model. Discharges in the second profile are the capacities which limit the water surface elevation extension over the AT&SF Railroad to no more than 1 foot and are then used for an approximate floodplain delineation. Breakout flows over the railroad along the length of the railroad, will continue overland to the southeast as sheet flow.

4.5.37 Litchfield Wash - Wash 21

Litchfield Wash is a detailed analysis that begins at the Litchfield Park Detention Facility and heads upstream to the northwest to the limit of study at Litchfield Road. Cross sections X1 = 0.000 to X1 = 0.496 are taken from the Litchfield Park Detention Facility plans. Floodplain and floodway limits are matched into the water surface elevation of the detention facility as computed in the HEC-1 model. No other unique conditions or problems exist on this reach.

4.5.38 Ponding Areas Behind the West Side of the Agua Fria River Dike

A detailed analysis of the ponding area delineations begin at Lower Buckeye Road and continue north along the west side of the Agua Fria River Dike to Indian School Road. Culvert nomograph charts and weir flow equations were utilized along with stage storage relationships developed from the 1" = 400' topographic mapping to model the ponding areas behind the Agua Fria River Dike within the HEC-1 model.

These relationships were developed where applicable when a pipe drain flows into the Agua Fria River through the dike based on a 10-year water surface elevation in the Agua Fria River, while a 100-year storm event occurs to the west of the Agua Fria River Dike. Otherwise, water surface elevations are computed based on culvert nomograph charts and/or weir flow calculations that will convey flows south across or under major roads.

4.5.39 Approximate Ponding Areas Behind Airline Canal

Approximate delineations were performed on the Airline Canal beginning at approximately the south corner of Section 21, Township 2 North, Range 1 West, and continuing upstream east and northeast to just east of the west quarter corner of Section 12, Township 2 North, Range 1 West. Approximate delineations are based on the nearest whole foot elevation above the top of the highest point on the Airline Canal, as shown on the 1" = 400' topographic mapping. This corresponds closely to previous FIRM mapping in this area however, it is changed slightly because of the more definitive mapping. NOTE: After the Colter Alignment Channel has been completed the opportunity may exist to remove or reduce some of the ponding area limits. This should be explored further when actual channel construction has been completed.

4.5.40 Detailed Analysis on Ponding Areas Behind Buckeye Canal and Approximate Delineations for Conveyance Corridors Where Applicable

Delineations begin at approximately the northwest corner of Section 36, Township 1 North, Range 3 West, and continue along the north side of Buckeye Canal to the east where it ends at approximately the south quarter corner of Section 28, Township 1 North, Range 1 West. Stage-storage-discharge tables within the HEC-1 model were utilized to calculate the water surface elevation in the ponding areas behind the Buckeye Canal. Weir flow equations were used to calculate any breakouts over the top of the canal and stage-storage-discharge relationships were calculated from 1" = 400' topographic mapping and plugged into the HEC-1 model.

The normal depth method for approximate delineations was incorporated to calculate approximate delineations for conveyance corridors where applicable. A large approximate delineation was computed between Estrella Parkway to just west of Sarival Avenue based upon the large amount of flow that breaks out across Estrella Parkway from Bullard Wash.

4.5.41 Approximate Delineations of Breakouts South Over Dysart Drain

Approximate delineations were performed on the breakouts that go south over Dysart Drain and continue through Luke Air Force Base. These eventually connect back into Bullard Wash. This is in the proximity of Sections 3, 4, 5, 8, 9, 10, and 17, Township 2 North, Range 1 West. Approximate delineations for the breakouts south over Dysart Drain occur on Luke Air Force Base which has areas of highly populated housing, ponding areas, and large undefined flow paths. Approximate delineations were computed using discharges from the HEC-1 model, ponding water surface elevations, and normal depth techniques through the Base south of these breakouts. These approximate delineations join together and will connect back into Bullard Wash south of Luke Air Force Base.

4.5.42 Litchfield Park Detention Facility

Litchfield Park Detention Facility is located in Section 15, Township 2 North, Range 1 West, Maricopa County, Arizona. A detailed analysis of the ponding area was performed utilizing the HEC-1 program. A combination of weir flow calculations and culvert outflow calculations were incorporated with the HEC-1 level pool routing routine and stage-storage-relationships were calculated from the 1" = 400' topographic mapping.

4.5.43 Reems Road - Northern Avenue to Beardsley Road

An approximate delineation was performed on Reems Road and begins at Northern Avenue and continues north along the Reems Road alignment to Beardsley Road. Reems Road is an inverted crown road and flows are conveyed downstream on it. An approximate delineation was based on the top of left bank controlling elevation and a few normal depth calculations were computed along the length to confirm this delineation.

Diverts occur at some of the major mile intersection points and these were modeled by the HEC-1 model. Documentation for the normal depth calculations are provided in the Appendix J, Volume 12 of 15.

4.5.44 Roosevelt Irrigation District Canal Ponding Areas

A detailed analysis of the ponding areas behind the Roosevelt Irrigation District Canal begin at Dean Road at approximately the southwest corner of Section 13, Township 1 North, Range 3 West, and continue upstream along the north side of the canal to the Agua Fria River at approximately the midsection of Section 25, Township 2 North, Range 1 West. Ponding areas behind the Roosevelt Irrigation District Canal occur throughout the watershed as the canal runs the width of this particular drainage area. Stage-storage-discharge tables were utilized within the HEC-1 model to compute 100-year water surface elevations behind the canal. Calculations for weir flow over the top of the canal were calculated by hand along with the use of 1" = 400' scale topographic mapping to compute the storage behind the canal.

4.5.45 Ponding Behind the Southern Pacific Railroad

A detailed analysis of the ponding areas and approximate delineations for conveyance corridors were performed behind the Southern Pacific Railroad. These delineations begin at approximately the southwest corner of Section 25, Township 1 North, Range 3 West, and continue to the east-northeast to approximately the south quarter corner of Section 11, Township 1 North, Range 1 West. The HEC-1 model was used with stage-storage-discharge routines to compute water surface elevations behind the Southern Pacific Railroad. Use of culvert nomographs, weir flow calculations, and 1" = 400' topographic mapping to calculate the storage areas, was incorporated to compute the water surface elevations.

The normal depth method was incorporated to calculate approximate floodplains behind the Southern Pacific Railroad between Dean Road and Airport Road and between Citrus Road and Cotton Lane. Some of the ponding areas behind the Southern Pacific Railroad have two or three culvert locations along a mile reach. These culvert capacities were combined together to compute one stage-storage-discharge table which was then incorporated into the HEC-1 model. Although invert elevations may differ somewhat between the culverts, the overall ponding water surface elevation behind the railroad

will act as a single pond, therefore affecting the culverts equally. The ponding area behind the Southern Pacific Railroad that is coincident with Bullard Wash, was modeled by a HEC-2 approximate delineation because of the complexity of the area. This area is included in the Bullard Wash HEC-2 analysis.

4.5.46 White Tanks Flood Retarding Structure #3

White Tanks Flood Retarding Structure #3 is located in Sections 4, 5, 8 and 9, Township 2 North, Range 2 West, Maricopa County, Arizona. An approximate analysis of the ponding area was performed on this structure as directed by the Flood Control District of Maricopa County. Stage-storage-discharge relationships were calculated from 1" = 400' topographic mapping and plugged into the HEC-1 model to compute the 100-year water surface elevation. No flow goes over the spillway during the 100-year event.

4.5.47 White Tanks Flood Retarding Structure #4

White Tanks Flood Retarding Structure #4 is located in Sections 5 and 6, Township 1 North, Range 2 West, Maricopa County, Arizona. An approximate delineation of the ponding area was performed as directed by the Flood Control District of Maricopa County. Stage-storage-discharge relationships were calculated from 1" = 400' topographic mapping and plugged into the HEC-1 model to compute the 100-year water surface elevation. No flow goes over the spillway during the 100-year event.

Study documentation abstracts for each of the above described areas can be found in Appendix L, under separate cover.

4.6 FLOODWAY MODELING

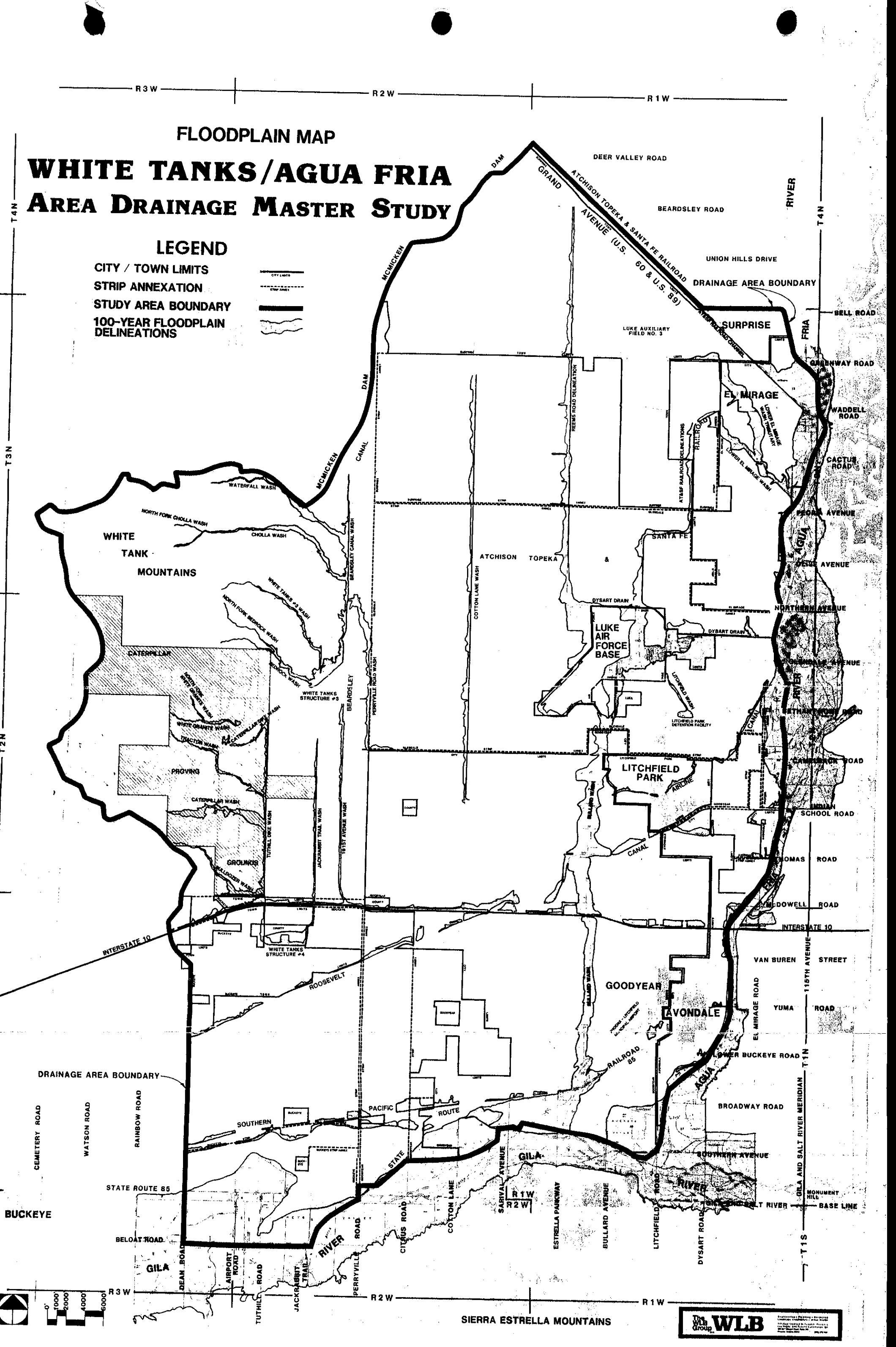
This study delineated floodways on many washes. Method 10.4 or Method 10.6 was initially incorporated to define floodways within each reach studied. These floodways were then fine tuned with Method 1 to set floodway limits while at the same time encroaching only so as to keep the rise in water surface elevation below 1 foot. See the Study Abstracts in Appendix L, Volume 15 of 15 of this report for a description of floodway modeling on each particular wash.

4.7 FINAL RESULTS / COMPUTER RUNS

The hardcopy output for each of the delineated floodplains and approximate delineations are located in Appendix J, Volumes 11 of 15, 12 of 15, and 13 of 15 under separate cover. Also, 1" = 400', 2-foot contour interval Floodplain Maps are submitted under separate cover with this report.

4.8 FINAL MODELING RESULTS ON DISKETTES

One diskette with all the HEC-2 input files for this study is located in a clear plastic insert at the back of this report.


FLOODPLAIN MAP
WHITE TANKS/AGUA FRIA
AREA DRAINAGE MASTER STUDY
LEGEND
CITY / TOWN LIMITS
STRIP ANNEXATION
STUDY AREA BOUNDARY
100-YEAR FLOODPLAIN DELINEATIONS
R3W
R2W
R1W
T4N
T3N
T2N
T1N
T1S
DEER VALLEY ROAD
BEARDSLEY ROAD
UNION HILLS DRIVE
DRAINAGE AREA BOUNDARY
BELL ROAD
GREENWAY ROAD
WADDELL ROAD
CACTUS ROAD
PEORIA AVENUE
BLUE AVENUE
NORTHERN AVENUE
GLENDALE AVENUE
BETHANY HOME ROAD
CAMELBACK ROAD
INDIAN SCHOOL ROAD
THOMAS ROAD
MCDOWELL ROAD
INTERSTATE 10
VAN BUREN STREET
YUMA ROAD
LOWER BUCKEYE ROAD
BROADWAY ROAD
SOUTHERN AVENUE
BASE LINE
MONUMENT HILL
GILA AND SALT RIVER MERIDIAN
115TH AVENUE
DEAN ROAD
AIRPORT ROAD
TUTHILL ROAD
JACKRABBIT TRAIL
PERRYVILLE ROAD
CITRUS ROAD
COTTON LANE
SARIVAL AVENUE
ESTRELLA PARKWAY
BULLARD AVENUE
LITCHFIELD ROAD
DYSART ROAD
BELOAT ROAD
STATE ROUTE 85
BUCKEYE
CEMETERY ROAD
WATSON ROAD
RAINBOW ROAD
SOUTHERN
PACIFIC
ROUTE
STATE
ROOSEVELT
GOODYEAR
AVONDALE
EL MIRAGE
SURPRISE
LUKE AIR FORCE BASE
LUKE AUXILIARY FIELD NO. 3
LITCHFIELD PARK
LITCHFIELD PARK DETENTION FACILITY
ATCHISON TOPEKA & SANTA FE
SANTA FE
RAILROAD
RAILROAD 85
ATCHISON TOPEKA & SANTA FE RAILROAD
AVENUE (U.S. 60 & U.S. 89)
GRAND
MCMICKEN DAM
MCMICKEN CANAL
WHITE TANK MOUNTAINS
WATERFALL WASH
NORTH FORK CHOLLA WASH
CHOLLA WASH
WHITE TANKS #3 WASH
NORTH FORK BEDROCK WASH
BEDROCK WASH
WHITE TANKS STRUCTURE #3
BEARDSLEY CANAL WASH
BEARDSLEY
PERRYVILLE ROAD WASH
COTTON LANE WASH
REEMS ROAD DELINEATIONS
ATSF RAILROAD DELINEATIONS
LOWER EL MIRAGE WASH
DYSART DRAIN
LITCHFIELD WASH
CATERPILLAR
PROVING GROUNDS
NORTH FORK WHITE GRANITE WASH
WHITE GRANITE WASH
CATERPILLAR DIKE WASH
TRACTOR WASH
CATERPILLAR WASH
BULLDOZER WASH
TUTHILL DIKE WASH
JACKRABBIT TRAIL WASH
19TH AVENUE WASH
WHITE TANKS STRUCTURE #4
CANAL
AIRLINE
GILA RIVER
AGUA FRIA RIVER
SIERRA ESTRELLA MOUNTAINS
0' 1000' 2000' 4000' 6000'
The WLB Group
WLB

SECTION 6: REFERENCE MATERIALS

6.1 OTHER PUBLISHED FLOOD STUDIES

1. Litchfield Park Airport Drainage, Thomas-Hartig & Associates, Inc. October 17, 1985.

2. Hydrology for Wigwam Creek Colter Street Alignment Interceptor Channel, DNA Inc., December 1989.

3. Hydrology for Special Study of Luke Air Force Base, Arizona, for L.A. District Corps of Engineers, PRC Toups Corporation, January 1979.

4. Hydrologic Evaluation Litchfield Park Dam Maricopa County, Arizona, for Litchfield Park Properties, Dames & Moore, January 1986.

5. Wittmann Area Drainage Master Study, Part A: Hydrology and Hydraulics, The WLB Group, Inc., March 10, 1989.

6. A Hydrologic Analysis of the White Tanks F.R.S. #3 and #4, Flood Control District of Maricopa County, October 1989.

7. Hydraulic Report, Perryville Road - Bullard Road, I-10-2(34)L STA 6451 to STA 6472, Arizona Department of Transportation, January 8, 1975.

8. Hydrologic Section I, I-10-2(34)C, STA 6451 to STA 6472, Arizona Department of Transportation, September 1974.

9. Hydrologic and Hydraulic Analysis, Perryville - Bullard Road, I-10-2(34)C, Arizona Department of Transportation, January 1975.

10. Drainage Studies, I-10-2(31) Site 11-6, Hayes & Dashney, June 1968.

11. Drainage Design Concept Development Report, I-IG-10-2(37)C, Bullard - Dysart Road, Dibble & Associates, April 1975.

6.2 PREVIOUS FEMA STUDIES

1. Gila River Flood Insurance Study - Gillespi Dam to Bullard Avenue, by Dames & Moore for Flood Control District of Maricopa County, May 1988.

2. Gila River Flood Insurance Study - Bullard Avenue to 115th Avenue, U.S. Army Corps of Engineers, March 1984.

SECTION 5: EROSION/SEDIMENT TRANSPORT

No erosion/sediment transport analyses were performed.

SECTION 6: REFERENCE MATERIALS

6.1 OTHER PUBLISHED FLOOD STUDIES

1. Litchfield Park Airport Drainage, Thomas-Hartig & Associates, Inc. October 17, 1985.

2. Hydrology for Wigwam Creek Colter Street Alignment Interceptor Channel, DNA Inc., December 1989.

3. Hydrology for Special Study of Luke Air Force Base, Arizona, for L.A. District Corps of Engineers, PRC Toups Corporation, January 1979.

4. Hydrologic Evaluation Litchfield Park Dam Maricopa County, Arizona, for Litchfield Park Properties, Dames & Moore, January 1986.

5. Wittmann Area Drainage Master Study, Part A: Hydrology and Hydraulics, The WLB Group, Inc., March 10, 1989.

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8. Hydrologic Section I, I-10-2(34)C, STA 6451 to STA 6472, Arizona Department of Transportation, September 1974.

9. Hydrologic and Hydraulic Analysis, Perryville - Bullard Road, I-10-2(34)C, Arizona Department of Transportation, January 1975.

10. Drainage Studies, I-10-2(31) Site 11-6, Hayes & Dashney, June 1968.

11. Drainage Design Concept Development Report, I-IG-10-2(37)C, Bullard - Dysart Road, Dibble & Associates, April 1975.

6.2 PREVIOUS FEMA STUDIES

1. Gila River Flood Insurance Study - Gillespi Dam to Bullard Avenue, by Dames & Moore for Flood Control District of Maricopa County, May 1988.

2. Gila River Flood Insurance Study - Bullard Avenue to 115th Avenue, U.S. Army Corps of Engineers, March 1984.

3. Agua Fria River Floodplain Maps for Flood Control District of Maricopa County, Jerry R. Jones & Associates, February 1989.

4. Flood Insurance Study, Maricopa County, Arizona and Unincorporated Areas, Federal Emergency Management Agency, September 4, 1991.

5. Flood Insurance Study, Maricopa County, Arizona and Unincorporated Areas, Volumes 1 - 7, Federal Emergency Management Agency, Revised: September 4, 1991.

6.3 OTHER APPLICABLE STUDIES

1. Geophysical, Geohydrological, and Geochemical Reconnaissance of the Luke Salt Body, Central Arizona, Geological Survey Professional Paper 753, G. Eaton, D. Patterson, and H. Schumann, United States Geological Survey, 1972.

2. White Tank Mountain Regional Park Plans, Maricopa County Parks and Recreation Department, February 1973.

3. Southern Pacific Transportation, Company Railroad Plans, Southern Pacific Railway, March 1926.

4. Interstate 10 Highway Plans, I-10-2(32), Cemetery Road - Perryville Road, Arizona Department of Transportation, October 17, 1977.

5. Interstate 10 Highway Plans, I-10-2(34), Perryville Road - Bullard Avenue, Arizona Department of Transportation, September 11, 1985.

6. Maricopa County Road Plans, Maricopa County Highway Department, Many Roadways, Various Dates.

7. Luke Air Force Base Drainage Plans including the Dysart Drain, Corps of Engineers, July 10, 1958.

8. Preliminary Economic Analysis Procedure, Flood Control District of Maricopa County.

9. Phase I Inspection Report for White Tanks Retarding Dam No. 4, by Ertec Western Inc. for the Arizona Department of Water Resources, August 1981.

10. Phase I Inspection Report for White Tanks Retarding Dam No. 3, by Ertec Western Inc., for the Arizona Department of Water Resources, August 1981.

11. Various Reports and Studies from the Flood Control District of Maricopa County.

12. Original White Tanks Data, Soil Conservation Services, January 14, 1954.

13. White Tanks Watershed Protection Project, Agua Fria Watershed, Maricopa County, Arizona, Soil Conservation Services, April 1954.

14. Basis of Design for Phoenix-Litchfield Municipal Airport Taxiway, Storm Drain and Electrical Work, Dibble & Associates, January 1972.

15. Master Drainage Report for a 1200 Acre Parcel Southeast of Peoria Avenue and Reems Road, for Spencer Development Corporation, Carter Associates, Inc., December 1988.

16. City of Glendale Policies for Lands West of the Agua Fria River, City of Glendale, December 1989.

17. White Tanks Agua Fria Technical Guide, Maricopa County Department of Planning and Development, January 1982.

18. Large Scale Developments, Maricopa County Department of Planning and Development, Spring 1989.

19. Little Rainbow Valley, Land Use Plan, by BRW Inc. and Sunregion Assoc. for Maricopa County Department of Planning and Development, June 1989.

20. Estrella Land Use Plan, Maricopa County Department of Planning and Development, May 1989.

21. Maricopa County Comprehensive Plan, Volume I, Background, Maricopa County Department of Planning and Development, January 1983.

22. White Tanks Agua Fria - Policy and Development Guide, Maricopa County Department of Planning and Development, December 28, 1982.

23. Maricopa Association of Governments Westside Joint Land Use Study, Barnard Dunkelberg & Company and Mestre Greve Associates, May 1988.

24. City of Avondale General Plan, Gruen & Associates, June 28, 1990.

25. Airport Master Plan Update for Phoenix-Litchfield Municipal Airport, Coffman Associates, Inc., July 1986.

26. Phoenix-Litchfield Municipal Airport Master Plan Report, Landrum & Brown and Henningson, Durham & Richardson, Inc., July 1978.

27. Pebble Creek Final Planned Area Development, B & R Engineering, Inc., December 1991.

28. Flow Estimation to Camelback and Dysart Roads, for Suncor Development Company, Boyle Engineering Corporation, April 7, 1988.

29. Conceptual Drainage Report for Litchfield Park Detention Facility, Coe & Van Loo, June 1989.

30. Concept Master Drainage Report Litchfield Park Development Master Plan, Coe & Van Loo, September 1989.

31. Technical Memorandum - Estrella Freeway Drainage Alternatives; Cotton Lane Section, Cella Bar Associates, March 3, 1988.

32. Estrella Freeway Hydrologic Investigation Report, Cella Barr Associates, July 14, 1987.

33. Grand Avenue Widening - Beardsley Canal to Thunderbird Road - Initial Drainage Report, Kimley-Horne and Associates, Inc., February 1990.

34. Hydrogeology of the Western Part of the Salt River Valley Area, Maricopa County, Arizona, United State Geological Survey, Brown & Pool, 1989.

6.4 PUBLISHED/UNPUBLISHED HISTORICAL FLOOD INFORMATION

1. Flood on February 1980 along the Agua Fria River, Maricopa County, Arizona B. Thompson, USGS, June 1980.

2. Report of Flood Conditions at Phoenix - Summer of 1951, Caterpillar Tractor Co., September 11, 1951.

6.5 REFERENCED TECHNICAL PAPERS/DOCUMENTS PERTAINING TO METHODOLOGY USED

1. Soil Survey of Maricopa County, Arizona, Central Part, Soil Conservation Service, September 1977.

2. Soil Survey of Aquila - Carefree Area, Parts of Maricopa and Pinal Counties, Arizona, Soil Conservation Services, 1981.

3. HEC-1 Flood Hydrograph Package, User's Manual, U.S. Army Corps of Engineers, Hydrologic Engineering Center, September 1988.

4. HEC-2 Water Surface Profiles, User's Manual, U.S. Army Corps of Engineers, Hydrologic Engineering Center, February 1991.

5. Roughones Characteristics of Natural Channels, Geological Survey Water - Supply Paper 1849, U.S.G.S., 1967.

6. Open Channel Hydraulics, Ven Te Chow, 1959.

7. Handbook of Hydraulics, Brater and King, Sixth Edition, 1976.

8. ProHEC2 Program Documentation, Dodson & Associates, April 1991.

9. Maricopa County Zoning Maps, Maricopa County, January 30, 1990.

10. City of El Mirage Official Zoning Map, July 2, 1987.

11. City of Goodyear Zoning Maps, January 5, 1987.

12. Town of Buckeye Existing Zoning Maps, 1989.

13. Litchfield Park Section 36 Zoning District Map, 1989.

14. Town of Surprise Zoning Maps, March 1989.

15. Time of Concentration in Small Rural Watersheds, C. Papadakis and M. Kazan, August 1986.

16. Hydrologic Design Manual for Maricopa County, Arizona, Flood Control District of Maricopa County, April 1990.

17. New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology, Part 1, U.S. Army Corps of Engineers, Los Angeles District, October 1974.

SECTION 7: CROSS-REFERENCING AND LABELING INFORMATION

7.1 OTHER STUDIES IMPACTED

There are a few previous flood studies that are impacted by the results of the White Tanks/Agua Fria ADMS. The washes and areas affected by this study and narration of the changes incorporated are as follows:

Lower El Mirage Wash - FIRM Panel 1605 of 4350 Lower El Mirage Wash was previously studied for a small stretch between Cactus Road and Dysart Road. This restudy expanded the limits of the delineation beginning at the Agua Fria River and continues upstream to the limit of detailed study at the Atchison, Topeka and Santa Fe Railroad spur south of Waddell Road. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

Lower El Mirage Wash Tributary - FIRM Panels 1165 and 1605 of 4350 Lower El Mirage Tributary was previously studied for a small stretch between Cactus Road and Waddell Road. This restudy expanded the limits of the delineation beginning at the confluence with Lower El Mirage Wash and continues upstream to the limit of detailed study at the intersection of Litchfield Road and Greenway Road. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

Atchison, Topeka and Santa Fe Railroad Channel - FIRM Panels 1165 and 1605 of 4350 This channel was delineated previously from the Agua Fria River to Dysart Road. The new detailed study begins at the AT&SF railroad bridge just northwest of the Agua Fria River and continues upstream to the limit of detailed study at Greenway Road. This limit was requested by the Flood Control District of Maricopa County and should match into the previous approximate delineation northwest of Greenway Road. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

White Tanks Flood Retarding Structure #3 - FIRM Panel 1600 of 4350 A new approximate delineation was computed for the area behind White Tanks Flood Retarding Structure #3. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

White Tanks Flood Retarding Structure #4 - FIRM Panel 2055 of 4350 A new approximate delineation was computed for the area behind White Tanks Flood Retarding Structure #4. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

Airline Canal - FIRM Panels 1615 and 2080 of 4350 A new approximate delineation was computed for the area behind Airline Canal. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

Roosevelt Irrigation District Canal - FIRM Panels 2050, 2055, 2060, 2065 and 2080 of 4350 A detailed analysis was performed on the ponding areas behind Roosevelt Canal and will supersede the approximate delineations calculated previously. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

Buckeye Canal - FIRM Panels 2050, 2065, and 2070 of 4350 A combination of detailed analyses on ponding areas and approximate delineations for conveyance corridors was performed behind stretches of the Buckeye Canal. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

Agua Fria River Dike Ponding Areas - West Side - FIRM Panels 2015, 2080, and 2090 of 4350 A new detailed analysis was performed on the ponding areas on the west side of the Agua Fria River Dike. This study will supersede the previous detailed study on these ponding areas. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

Ponding in the City of Goodyear - FIRM Panel 2090 of 4350 A detailed analysis of the ponding area located on Litchfield Road, north of the State Route 85, will supersede the approximate delineation here. The new delineation incorporates a more detailed hydrologic analysis along with the corresponding discharges calculated for the White Tanks/Agua Fria ADMS and is based on new modeling methodology stated in the "Hydrologic Design Manual for Maricopa County". Also, this study utilizes the new 1" = 400', 2 foot contour interval topographic mapping developed for this ADMS. This study will supersede the previous delineation.

7.2 KEY TO CROSS SECTION LABELING

Cross sections for all HEC-2 analyses in this study are labeled based on river miles. Cross section data was taken from 1" = 400', 2 foot contour interval mapping prepared for this ADMS. No letters were provided for these cross sections as it will be left up to the FEMA technical evaluation coordinator to pick appropriate cross sections for inclusion on the FIRM maps. A table for each HEC-2 analysis is incorporated in the Study Documentation abstracts located in Appendix L, Volume 15 of 15.

SECTION 8: DRAFT FIS REPORT - REVISED TEXT

Insert 1, Page 3, After Paragraph 8, Volume 1 of 7

The hydrologic and hydraulic analyses for many previously undefined washes in Maricopa County, Arizona, including revised studies of Lower El Mirage Wash, Lower El Mirage Wash Tributary, Atchison, Topeka and Santa Fe Railroad Channel, White Tanks Flood Retarding Structure #3 and #4, Airline Canal, Roosevelt Irrigation District Canal, Buckeye Canal, Agua Fria River Dike Ponding Areas - West Side, along the Agua Fria River, and ponding in the City of Goodyear were performed by The WLB Group, Inc. under contract to the Flood Control District of Maricopa County as a portion of the White Tanks/Agua Fria Area Drainage Master Study (ADMS). These analyses were completed in May 1992.

Revision 1, Table 1, Page 7, Volume 1 of 7

Change limits of study description for Lower El Mirage Wash and Lower El Mirage Wash Tributary to the following descriptions respectively:

- From confluence with Agua Fria River to Atchison, Topeka and Santa Fe Railroad spur southwest of the intersection of Waddell Road and Dysart Road.
- From confluence with Lower El Mirage Wash to intersection of Greenway Road and Litchfield Road.

Insert 2, Table 1, Page 10, After Last Entry, Volume 1 of 7

Beardsley Canal Wash	From White Tanks Flood Retarding Structure #3 to 3.7 miles upstream.
Cholla Wash	From confluence with Beardsley Canal Wash to 4.2 miles upstream.
North Fork Cholla Wash	From confluence with Cholla Wash to 2.5 miles upstream.
Waterfall Wash	From confluence with Beardsley Canal Wash to 3.5 miles upstream.
White Tank #3 Wash	From White Tanks Flood Retarding Structure #3 to 3.2 miles upstream.
Bedrock Wash	From White Tanks Flood Retarding Structure #3 to 2.4 miles upstream.

North Fork Bedrock Wash	From confluence with Bedrock Wash to 1.7 miles upstream.
Jackrabbit Trail Wash	From White Tanks Flood Retarding Structure #4 to 4.2 miles upstream.
Tuthill Dike Wash	From White Tanks Flood Retarding Structure #4 to 4.7 miles upstream.
Bulldozer Wash	From confluence with Tuthill Dike Wash to 2.6 miles upstream.
Caterpillar Wash	From confluence with Tuthill Dike Wash to 2.2 miles upstream.
Tractor Wash	From confluence with Tuthill Dike Wash to 2.7 miles upstream.
Caterpillar Dike Wash	From confluence with Tuthill Dike Wash to .9 miles upstream.
White Granite Wash	From retention basin on Caterpillar Proving Grounds to 1.4 miles upstream.
North Fork White Granite Wash	From confluence with White Granite Wash to .7 miles upstream.
191st Avenue Wash	From Interstate 10 to 4.0 miles upstream.
Perryville Road Wash	From agricultural reservoir located on Camelback Road 1/2 mile west of Citrus Road to 3.7 miles upstream.
Bullard Wash	From south end of Phoenix-Goodyear Municipal Airport to south end of Luke Air Force Base.
Interstate 10	From Jackrabbit Trail to Tuthill Dike, .9 miles upstream.
Litchfield Wash	From Litchfield Park Detention Facility to 1.1 miles upstream.

Roosevelt Irrigation District Canal	For ponding areas behind the Roosevelt Irrigation District Canal from the Agua Fria River to Dean Road.
Southern Pacific Railroad	For ponding areas behind the Southern Pacific Railroad from the west side of the Agua Fria River to Dean Road.
Buckeye Canal	For some ponding areas located behind Buckeye Canal from Sarival Avenue to Dean Road.
Agua Fria River Dike Ponding Areas - West Side	For ponding areas along the west side of the Agua Fria River Dike from the Gila River to Indian School Road.
Litchfield Park Detention Facility	For ponding in the Litchfield Park Detention Facility located in Section 15, Township 2 North, Range 1 West, Maricopa County, Arizona.
Interstate 10 Detention Basins	For ponding in the detention basins located along the north side of Interstate 10 from Dysart Road to Bullard Avenue.

<u>Insert 3, Table 2, Pages 11, 12, 13, and 14, Insert Alphabetically with Existing Table, Volume 1 of 7</u>

Cotton Lane Wash
Interstate 10
Atchison, Topeka and Santa Fe Railroads Spur
Bullard Wash
Dysart Drain
Airline Canal
Reems Road
White Tanks Flood Retarding Structure #3
White Tanks Flood Retarding Structure #4

Note: Lower El Mirage Wash can be deleted from Table 2.

Insert 4, Page 18, After Paragraph 5, Volume 1 of 7

Beardsley Canal Wash flows south along the west side of Beardsley Canal from approximately McMicken Dam to the White Tanks Flood Retarding Structure #3 in the central part of Maricopa County.

Cholla Wash flows east/southeasterly from the White Tank Mountains to the confluence with Beardsley Canal Wash.

North Fork Cholla Wash flows easterly from the White Tank Mountains to the confluence with Cholla Wash.

Waterfall Wash flows east/southeasterly from the White Tank Mountains to its confluence with Beardsley Canal Wash.

White Tanks #3 Wash flows east/southeasterly from the White Tank Mountains to the detention basin behind White Tanks Flood Retarding Structure #3.

Bedrock Wash flows east/southeasterly from the White Tank Mountains to the detention basin behind White Tanks Flood Retarding Structure #3.

North Fork Bedrock Wash flows southeasterly from the White Tank Mountains to its confluence with Bedrock Wash.

Jackrabbit Trail Wash flows south to the detention basin behind White Tank Flood Retarding Structure #4.

Tuthill Dike Wash flows south along the west side of Tuthill Dike to the detention basin located behind White Tanks Flood Retarding Structure #4.

Bulldozer Wash flows east/southeasterly from the White Tank Mountains to its confluence with Tuthill Dike Wash.

Caterpillar Wash flows east from the White Tank Mountains to its confluence with Tuthill Dike Wash.

Tractor Wash flows east/southeasterly from the White Tank Mountains to its confluence with Tuthill Dike Wash.

Caterpillar Dike Wash flows east/northeasterly from the White Tank Mountains to its confluence with Tuthill Dike Wash.

White Granite Wash flows east from the White Tank Mountains to a retention basin in the Caterpillar Proving Grounds.

North Fork White Granite Wash flows southeasterly from the White Tank Mountains to its confluence with White Granite Wash.

191st Avenue Wash flows south along 191st Avenue alignment to Interstate 10.

Perryville Road Wash flows south from Beardsley Canal Wash and Northern Avenue to an agricultural reservoir located 1/2 mile west of Citrus Road on the north side of Camelback Road.

Bullard Wash flows from the west side of Luke Air Force Base southerly to its confluence with the Gila River.

The Atchison, Topeka and Santa Fe Railroad Channel flows southeasterly along the north side of the railroad to its confluence with the Agua Fria River.

Lower El Mirage Wash and Lower El Mirage Wash Tributary flows southeasterly to the confluence with the Agua Fria River.

Flows along the north side of Interstate 10 from the breakout at Tuthill Dike flow easterly to Jackrabbit Trail, along the north side of Interstate 10.

Litchfield Wash flows southeasterly from Litchfield Road to the Litchfield Park Detention Facility.

All of these washes are located between the White Tank Mountains, the Gila River, and the Agua Fria River, in central Maricopa County, Arizona.

Insert 5, Page 25, After Paragraph 1, Volume 1 of 7

White Tanks Flood Retarding Structure #3 and White Tanks Flood Retarding Structure #4 were built in 1954 by the Soil Conservation Service. The structures are now maintained by the Flood Control District of Maricopa County. These structures provide some protection from flooding in the White Tank Mountains.

The Litchfield Park Detention Facility was built in 1991 and provides some protection for the City of Litchfield Park.

The Dysart Drain (also known as Luke Air Force Base Drainage Channel) was built in the 1956 to protect Luke Air Force Base. Subsidence in the area, due to groundwater withdrawal, has decreased the capacity of this channel considerably from its original design capacity.

Insert 6, Page 28, After Paragraph 3, Volume 1 of 7

Peak discharge-frequency relationships for the White Tanks/Agua Fria Area Drainage Master Study (ADMS) which includes many washes in central Maricopa County, were computed using the U.S. Army Corps of Engineers, Hydrologic Engineering Center's HEC-1 Flood Hydrograph Package. Also, the new "Hydrologic Design Manual for Maricopa County" was utilized to incorporate new methodology for computing discharges in this ADMS.

Insert 7, Table 3, Page 43, After Last Entry, Volume 1 of 7

Flooding Source and Location					
Beardsley Canal Wash	Drainage Area (Square Miles)	Peak (Cubic Fe 10-Year	50-)		
At White Tanks F.R.S. #3	12.56	---[1]	---[1]		
At 0.465 Miles Upstream of White Tanks F.R.S. #3	12.25	---[1]	---[1]	4125	---[1]
Downstream of Northern Ave. Culverts (1486 CFS is diverted over Beardsley Canal to the east.)	10.87	---[1]	---[1]	3655	---[1]
Upstream of Northern Ave. Culverts	10.87	---[1]	---[1]	5141	---[1]
At the confluence with Cholla Wash	6.01	---[1]	---[1]	3816	---[1]
Downstream of Olive Ave. Culverts (490 CFS is diverted over Beardsley Canal to the east.)	4.86	---[1]	---[1]	1755	---[1]
Upstream of Olive Ave. Culverts (At the Confluence with Waterfall Wash.)	4.86	---[1]	---[1]	2245	---[1]
At 2.330 miles upstream	1.10	---[1]	---[1]	997	---[1]
At Peoria Avenue	.29	---[1]	---[1]	296	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Results

Cholla Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the Confluence with Beardsley Canal Wash.	6.01	---[1]	---[1]	3816	---1
At confluence with North Fork Cholla Wash	3.99	---[1]	---[1]	3227	---[1]
Upstream of the confluence with North Fork Cholla Wash	3.18	---[1]	---[1]	2527	---[1]

1 Not Computed

North Fork Cholla Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the Confluence with Cholla Wash	3.99	---[1]	---[1]	3227	---[1]
Upstream of the confluence with Cholla Wash	0.81	---[1]	---[1]	704	---[1]

1 Not Computed

Waterfall Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the Confluence with Beardsley Canal Wash	4.86	---[1]	---[1]	2245	---[1]
At 0.773 miles upstream	3.76	---[1]	---[1]	2284	---[1]
At 2.169 miles upstream	2.85[2]	---[1]	---[1]	1813[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run

White Tanks #3 Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At White Tanks F.R.S. #3	2.86	---[1]	---[1]	1743	---[1]
At 1.837 miles upstream	1.56	---[1]	---[1]	1313	---[1]
At 2.581 miles upstream	0.78	---[1]	---[1]	656[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run

Bedrock Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At White Tanks F.R.S. #3	4.93	---[1]	---[1]	1738	---[1]
At the Confluence with North Fork Bedrock Wash	3.86	---[1]	---[1]	1920	---[1]
At 1.317 miles upstream	0.63	---[1]	---[1]	520[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run

North Fork Bedrock Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the Confluence with Bedrock Wash	3.86	---[1]	---[1]	1920	---[1]
At 0.147 miles upstream	2.1[2]	---[1]	---[1]	1560[2]	---[1]
At 1.003 miles upstream	1.78[2]	---[1]	---[1]	1362[2]	---[1]
At 1.640 miles upstream	1.47	---[1]	---[1]	1163	---[1]

1 Not Computed

Jackrabbit Trail Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At White Tanks F.R.S. #4	32.10	---[1]	---[1]	1093[3]	---[1]
Downstream of Interstate 10 Culverts.	17.43	---[1]	---[1]	1186[3]	---[1]
Upstream of Interstate 10 Culverts.	17.43	---[1]	---[1]	1186[3]	---[1]
Downstream of McDowell Rd. Culverts.	17.43	---[1]	---[1]	1186[3]	---[1]
Upstream of McDowell Rd. Culverts.	17.43	---[1]	---[1]	1186[3]	---[1]
At Thomas Road	2.07	---[1]	---[1]	1105[3]	---[1]
At Indian School Road	1.36	---[1]	---[1]	726[3]	---[1]
At Camelback Road	0.43	---[1]	---[1]	221[3]	---[1]
At Medlock Drive	0.22	---[1]	---[1]	187[3]	---[1]

1 Not Computed
3 Peak discharges have been derived by performing a HEC-2 split flow analysis along the Jackrabbit Trail Wash.

Tuthill Dike Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At White Tanks F.R.S. #4	14.12	---[1]	---[1]	4057	---[1]
Downstream of Interstate 10 Culverts. (1441 CFS has been diverted over the dike to the east.)	13.90	---[1]	---[1]	4061	---[1]
Upstream of Interstate 10 Culverts.	13.90	---[1]	---[1]	5503[4]	---[1]
At McDowell Road	14.56	---[1]	---[1]	6601	---[1]
At the confluence with Bulldozer Wash.	14.56	---[1]	---[1]	6601	---[1]
At Thomas Road	12.86	---[1]	---[1]	6110	---[1]
At the confluence with Caterpillar Wash.	12.86	---[1]	---[1]	6110	---[1]
At 2.753 miles upstream	7.45	---[1]	---[1]	3011	---[1]
At Indian School Road (at the confluence with Tractor Wash).	7.45	---[1]	---[1]	3011	---[1]
At 3.344 miles upstream of Indian School Road	5.29	---[1]	---[1]	1414	---[1]
At 4.006 miles upstream (just downstream of Camelback Road)	4.99	---[1]	---[1]	1261[2]	---[1]
At the confluence with Caterpillar Dike Wash.	4.69	---[1]	---[1]	1108	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run
4 Decrease due to storage behind culverts and storage in overbanks.

Bulldozer Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with Tuthill Dike Wash	14.56	---[1]	---[1]	6601[3]	---[1]
At 0.178 miles upstream	1.46[2]	---[1]	---[1]	1250[2]	---[1]
At 0.705 miles upstream	1.11[2]	---[1]	---[1]	942[2]	---[1]
At Caterpillar Proving Grounds Road	0.52	---[1]	---[1]	525	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run
3 Decrease due to storage behind culverts and storage in overbanks.

Caterpillar Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with Tuthill Dike Wash	12.86	---[1]	---[1]	6110	---[1]
At 0.077 miles upstream	5.02[2]	---[1]	---[1]	3412[2]	---[1]
At Caterpillar Proving Grounds Road (1.147 miles upstream)	4.64	---[1]	---[1]	3253	---[1]
At 1.764 miles upstream	3.88	---[1]	---[1]	2886	---[1]
At 2.139 miles upstream	0.95	---[1]	---[1]	672	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run

Tractor Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with Tuthill Dike Wash	7.45	---[1]	---[1]	3011	---[1]
At 0.037 miles upstream	2.16	---[1]	---[1]	1648	---[1]
At 0.720 miles upstream	1.66[2]	---[1]	---[1]	1295[2]	---[1]
At 1.196 miles upstream	1.16	---[1]	---[1]	943	---[1]
At Caterpillar Proving Grounds Road (2.042 miles upstream)	0.58[2]	---[1]	---[1]	472[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run

Caterpillar Dike Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with Tuthill Dike Wash	4.69	---[1]	---[1]	1108	---[1]
At 0.273 miles upstream	0.25[2]	---[1]	---[1]	382[2]	---[1]
At 0.504 miles upstream	0.12[2]	---[1]	---[1]	191[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run

White Granite Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the retention basin on Caterpillar Proving Grounds	3.46	---[1]	---[1]	1933	---[1]
At the confluence with North Fork White Granite Wash	3.46	---[1]	---[1]	1933	---[1]
At 0.777 miles upstream	0.39[2]	---[1]	---[1]	344[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Run

North Fork White Granite Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with White Granite Wash.	3.46	---[1]	---[1]	1933	---[1]
At 0.134 miles upstream	1.60	---[1]	---[1]	1353	---[1]

1 Not Computed

191st Avenue Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the downstream end (200 feet north of Interstate 10)	---[1]	---[1]	---[1]	617[4]	---[1]
At McDowell Road	---[1]	---[1]	---[1]	655[4]	---[1]
At Thomas Road	---[1]	---[1]	---[1]	116[4]	---[1]
Downstream of Indian School Road	---[1]	---[1]	---[1]	147[4]	---[1]
Upstream of Indian School Road	---[1]	---[1]	---[1]	147[4]	---[1]
At Camelback Road	---[1]	---[1]	---[1]	564[4]	---[1]
At a 1/2 mile north of Camelback Road	---[1]	---[1]	---[1]	883[4]	---[1]

1 Not Computed
4 Peak discharges have been derived by performing a HEC-2 split flow analysis along 191st Avenue Wash.

Perryville Road Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the limit of study at Camelback Road and 1/2 mile east of Perryville Rd.	13.17	---[1]	---[1]	470[4]	---[1]
At 0.088 miles upstream	13.17	---[1]	---[1]	1129[4]	---[1]
At X1 - 0.177 miles upstream	13.17	---[1]	---[1]	1277[4]	---[1]
At the intersection of Camelback Road and Perryville Road	13.04	---[1]	---[1]	1190[4]	---[1]
At Bethany Home Road	12.54	---[1]	---[1]	1375[4]	---[1]
At Glendale Avenue	11.58	---[1]	---[1]	1450[4]	---[1]
At Northern Avenue	11.08	---[1]	---[1]	1457[4]	---[1]

1 Not Computed
4 Peak discharges have been derived by performing a HEC-2 split flow analysis along Perryville Road Wash.

Bullard Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the beginning of detailed study (2.371 miles upstream from Buckeye Canal)	96.25	---[1]	---[1]	4899	---[1]
At Lower Buckeye Road Extended	91.66	---[1]	---[1]	4906	---[1]
At 2.977 miles upstream	91.31	---[1]	---[1]	4915	---[1]
At 3.167 miles upstream	90.57	---[1]	---[1]	4432	---[1]
At Yuma Road	89.93	---[1]	---[1]	4438	---[1]
At Van Buren Street	89.11	---[1]	---[1]	4446	---[1]
Downstream of Interstate 10 Bridge	88.27	---[1]	---[1]	4446	---[1]
Upstream of Interstate 10 Bridge	88.27	---[1]	---[1]	5319	---[1]
At McDowell Road	87.20	---[1]	---[1]	4662	---[1]
At Thomas Road Extended (Roosevelt Irrigation District Canal)	86.28	---[1]	---[1]	4703	---[1]
At 7.124 miles upstream	85.45	---[1]	---[1]	4088	---[1]
At Indian School Road	84.50	---[1]	---[1]	4121	---[1]
At Camelback Road	78.75	---[1]	---[1]	4243	---[1]
At 9.292 miles upstream	59.77	---[1]	---[1]	3693	---[1]
At 9.641 miles upstream	59.77	---[1]	---[1]	3113[2]	---[1]
At 9.898 miles upstream	39.13	---[1]	---[1]	762[2]	---[1]
Downstream of Bethany Home Road Extended (End of Detailed Study)	39.13	---[1]	---[1]	762[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Results

From Agua Fria River Northwest to Greenway (Wash 12)	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At Atchison, Topeka & Santa Fe Railroad Bridge	1.14	---[1]	---[1]	577	---[1]
At 0.438 miles upstream	0.76	---[1]	---[1]	483	---[1]
At Factory Street	0.76	---[1]	---[1]	483	---[1]

1 Not Computed

Lower El Mirage Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with Agua Fria River	26.68	---[1]	---[1]	1753	---[1]
At El Mirage Road	26.37	---[1]	---[1]	1768	---[1]
At Cactus Road	26.25	---[1]	---[1]	1771	---[1]
At the confluence with El Mirage Wash Tributary	26.25	---[1]	---[1]	1771	---[1]
At 1.348 miles upstream	25.08	---[1]	---[1]	1258	---[1]
At Dysart Road	24.19	---[1]	---[1]	845	---[1]

1 Not Computed

Lower El Mirage Wash Tributary	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with El Mirage Wash	26.25	---[1]	---[1]	1771	---[1]
At 0.045 miles upstream	7.88[2]	---[1]	---[1]	1170[2]	---[1]
At Waddell Road	7.52	---[1]	---[1]	829	---[1]
At AT&SF Railroad	7.22	---[1]	---[1]	843	---[1]
At 1.894 miles upstream	---[1]	---[1]	---[1]	764[2]	---[1]
At the intersection of Greenway Rd. and Dysart Rd.	6.35	---[1]	---[1]	856	---[1]
At 2.615 miles upstream	---[1]	---[1]	---[1]	770[2]	---[1]
At the intersection of Greenway Road and Litchfield Road	5.44	---[1]	---[1]	545	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Results

Interstate 10 - Jackrabbit Trail West to Tuthill Rd. (Wash 14-2)	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At the confluence with Jackrabbit Trail Wash	0.43**	---[1]	---[1]	325[2]	---[1]
At 0.29 miles upstream	14.51	---[1]	---[1]	316	---[1]
At 0.50 miles upstream	14.36	---[1]	---[1]	1030	---[1]
At 0.74 miles upstream	13.98	---[1]	---[1]	1104	---[1]
At 0.931 miles upstream	13.98	---[1]	---[1]	1440	---[1]

** The computed diversion from the next upstream subbasin was 0.0 CFS and therefore the upstream contributing area was considered to be 0.0 sq. mi. Only half of drainage area from subbasin contributes.

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Results

Litchfield Wash	Drainage Area (Square Miles)	Peak Discharge (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
At Litchfield Park Detention Facility	0.43	---[1]	---[1]	520	---[1]
At 0.238 miles upstream	---[1]	---[1]	---[1]	347[2]	---[1]
At 0.597 miles upstream	---[1]	---[1]	---[1]	260[2]	---[1]
At 0.856 miles upstream	---[1]	---[1]	---[1]	78[2]	---[1]

1 Not Computed
2 Interpolated Discharge from White Tanks/Agua Fria ADMS HEC-1 Results

Insert 8, Page 45, After Paragraph 2, Volume 1 of 7

Cross section data for delineations performed in the White Tanks/Agua Fria ADMS were developed from 1:4,800, 2-foot contour interval topographic mapping compiled for the White Tanks/Agua Fria ADMS.

Insert 9, Table 4, Pages 46 and 47, Insert Alphabetically, Volume 1 of 7

Beardsley Canal Wash	.024 - .035	.024 - .07
Cholla Wash	.035 - .07	.03 - .07
North Fork Cholla Wash	.07	.07
Waterfall Wash	.035 - .05	.07 - .10
White Tanks #3 Wash	.035 - .045	.035 - .07
Bedrock Wash	.045 - .05	.035 - .07
North Fork Bedrock Wash	.035 - .045	.07
Jackrabbit Wash	.012 - .03	.012 - .06
Tuthill Dike Wash	.016 - .03	.016 - .05
Bulldozer Wash	.035 - .05	.04 - .07
Caterpillar Wash	.03 - .035	.05 - .07
Tractor Wash	.03 - .035	.035 - .075
Caterpillar Dike Wash	.035	.035 - .07
White Granite Wash	.035	.07
North Fork White Granite Wash	.035	.07
191st Avenue Wash	.012 - .03	.04 - .07
Perryville Road Wash	.022 - .045	.035 - .08
Bullard Wash	.013 - .07	.03 - .07
Atchison, Topeka & Santa Fe Railroad Channel	.035 - .045	.03 - .08
Lower El Mirage Wash	.03 - .045	.035 - .10
Lower El Mirage Wash Tributary	.04 - .045	.07 - .10
Interstate 10 - Jackrabbit Trail to Tuthill Dike	.035	.045 - .05
Litchfield Wash	.025 - .035	.025 - .05

Revision 3, Page 50, Paragraph 5, Volume 1 of 7

Delete Lower El Mirage Wash Tributary from this section.

Interstate 10, Page 48, After Last Paragraph, Volume 1 of 7

Starting water surface elevations for delineations of washes in the White Tanks/Agua Fria ADMS were computed utilizing the slope-area method or from computed elevations at the confluence with another wash.

Insert 11, Table 5, Alphabetically

Floodway Data Tables are found in the HEC-2 computer printout.

Insert 12, Add References Where Applicable

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COMPUTATION DATA SHEET

Subject WHITE TANKS ADMS Prepared by JSE Date 8-9-91

DEPTH AREA RELATIONSHIPS FOR PRECIPITATION DATA
24-HOUR STORMS

AREA (SQMI)	1	10	50	100	200
*REDUCTION %	1.00	.984	.95	.933	.918
RETURN PERIOD					
2-YEAR	1.42	1.40	1.35	1.32	1.30
10-YEAR	2.53	2.49	2.40	2.36	2.32
25-YEAR	2.98	2.93	2.83	2.78	2.74
50-YEAR	3.60	3.54	3.42	3.36	3.30
100-YEAR	4.03	3.97	3.83	3.76	3.70

* Percentage Reduction Data Taken From NOAA Atlas 2 - Precipitation Data for the Southwestern United States - VOLUME VIII, Arizona

e applied to appro-
tions are used f
ipitation, while tu-
es of precipitation-

required to furnish
ns for fixed areas.
the determination
al Weather Service

of recording gages.
crease the number
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usually early morn-
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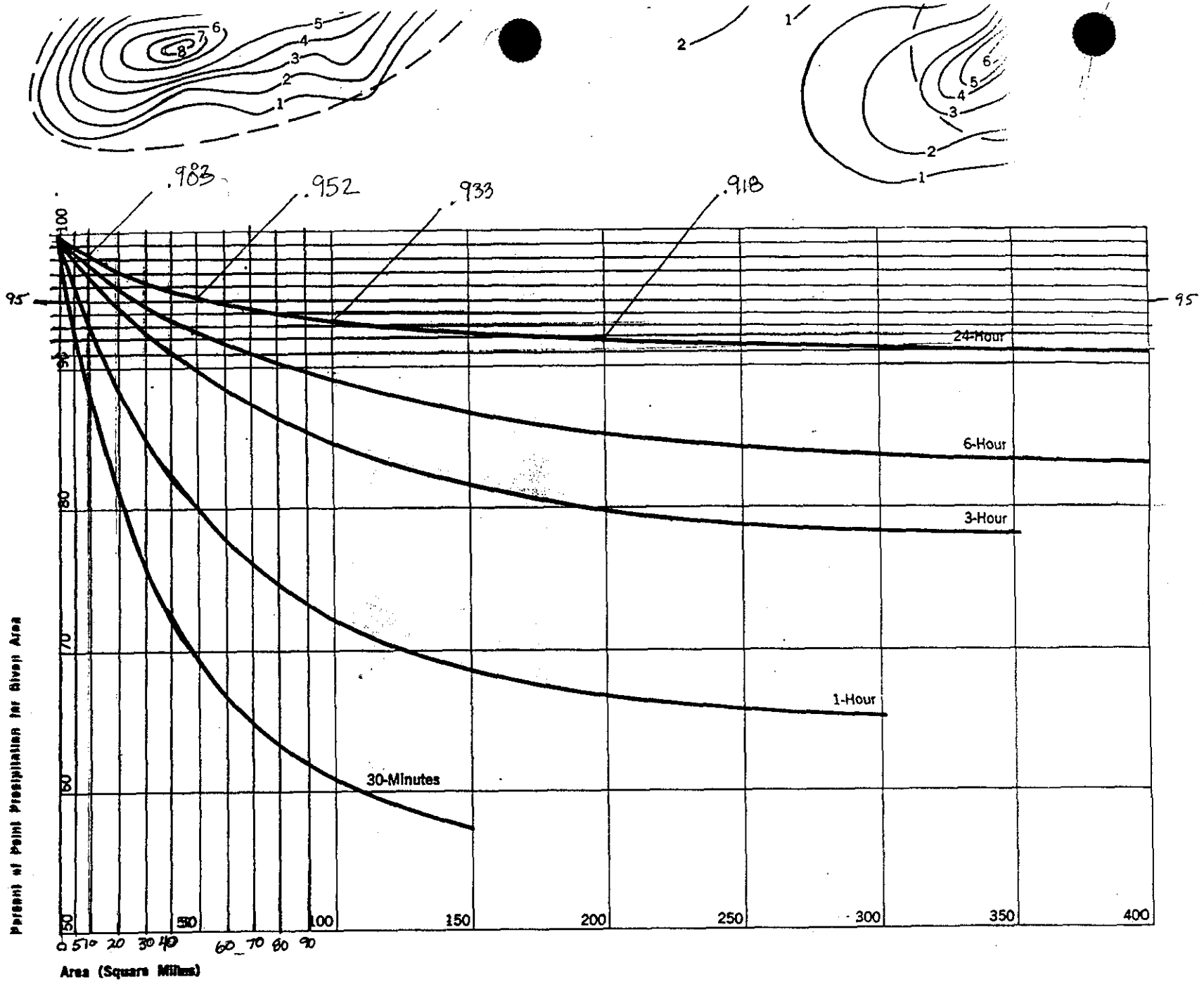

NOAA ATLAS 2 - VOLUME VIII, ARIZONA

Figure 14. *Depth-Area curves.*

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COMPUTATION DATA SHEET

Subject WHITE TANKS/AGUAFRIA ADMS Prepared by JSE Date 1-19-90

DEPTH - DURATION - FREQUENCY CALCULATIONS

LOCATION: Parts of T4N, R2W; T4N, R1W; T3N, R3W; T3N, R2W; T3N, R1W; T2N, R3W; T2N, R2W; T2N, R1W; T1N, R3W; T1N, R2W; T1N, R1W

(Boundaries are Grand Avenue on the North, the White Tanks on the West, the Gila River on the South, and the Agua Fria River on the East)

Using Plates 1-12 of the Maricopa County Hydrology Manuel, 1989 the following rainfall values were estimated.

6-Hour Events		Map Value	Corrected Values.
2-Year,	6-Hour	= 1.19 in	1.20 in
5-Year,	6-Hour	= 1.80 in	1.71 in
10-Year,	6-Hour	= 2.04 in	2.07 in
25-Year,	6-Hour	= 2.54 in	2.44 in
50-Year,	6-Hour	= 2.89 in	2.92 in
100-Year,	6-Hour	= 3.23 in	3.29 in

These are average rainfall amounts interpolated from the Plates. The data is plotted on Figure 25 and the corrections are made above.

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COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-19-90

24-Hour Events

		Map Value	Corrected Value
2-Year, 24-Hour	=	1.40 in.	1.42 in
5-Year, 24-Hour	=	2.08 in.	2.05 in
10-Year, 24-Hour	=	2.56 in.	2.53 in
25-Year, 24-Hour	=	3.14 in	2.98 in
50-Year, 24-Hour	=	3.54 in	3.60 in
100-Year, 24-Hour	=	4.00 in	4.03 in

CALCULATE OTHER DEPTHS FOR VARIOUS FREQUENCIES + DURATIONS

$Y_2 = -.011 + .942 (X_1)(X_1/X_2)$ $Y_2 =$ 24R, 1HR

$Y_{100} = .494 + .755 (X_3)(X_3/X_4)$ $Y_{100} =$ 100-YR, 1-HR

$X_1 =$ 2-YR, 6-HR
$X_2 =$ 2-YR, 24-HR
$X_3 =$ 100-YR, 6HR
$X_4 =$ 100-YR, 24-HR

$Y_2 = -.011 + .942 (1.20)(1.20/1.42)$
$Y_2 = 0.94$ in

$Y_{100} = .494 + .755 (3.29)\left(\frac{3.29}{4.03}\right)$
$Y_{100} = 2.52$ in

1-Hour Events

2-Year, 1-Hour	=	0.94 in
5-Year, 1-Hour	=	1.30 in
10-Year, 1-Hour	=	1.60 in
25-Year, 1-Hour	=	1.89 in
50-Year, 1-Hour	=	2.23 in
100-Year, 1-Hour	=	2.52 in

From Fig. 2.5

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COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-19-90

2-HOUR EVENTS

100-YR, 2-HR = .341 (100-YR, 6-HR) + .659 (100-YR, 1-HR) = (.341)(3.29) + (.659)(2.52)

2-YR, 2-HR = .341 (2-YR, 6-HR) + .659 (2-YR, 1-HR) = (.341)(1.20) + (.659)(0.94)

2-YEAR, 2-HOUR = 1.03 in

5-YEAR, 2-HOUR = 1.45 in

10-YEAR, 2-HOUR = 1.78 in

25-YEAR, 2-HOUR = 2.06 in

50-YEAR, 2-HOUR = 2.57 in

From Fig 2.5

100-YEAR, 2-HOUR = 2.78 in

3-HOUR EVENTS

100-YR, 3-HR = .569 (100-YR, 6-HR) + .431 (100-YR, 1-HR) = (.569)(3.29) + (.431)(2.52)

2-YR, 3-HR = .569 (2-YR, 6-HR) + .431 (2-YR, 1-HR) = (.569)(1.20) + (.431)(0.94)

2-YEAR, 3-HOUR = 1.09 in

5-YEAR, 3-HOUR = 1.56 in

10-YEAR, 3-HOUR = 1.89 in

25-YEAR, 3-HOUR = 2.19 in

50-YEAR, 3-HOUR = 2.63 in

From Fig 2.5

100-YEAR, 3-HOUR = 2.96 in

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12-HOUR EVENTS — From Fig. 2.6

2-YEAR, 12-HOUR = 1.30 in

5-YEAR, 12-HOUR = 1.88 in

10-YEAR, 12-HOUR = 2.29 in

25-YEAR, 12-HOUR = 2.70 in

50-YEAR, 12-HOUR = 3.25 in

100-YEAR, 12-HOUR = 3.66 in

30-MINUTE EVENTS

2YR, 30-MIN = (.82)(2YR, 1-HR) = .82 (0.94) = 0.77 in

100YR, 30-MIN = (.82)(100YR, 1HR) .82 (2.52) = 2.06 in

2-YEAR, 30-MIN = 0.77 in

5-YEAR, 30-MIN = 1.05 in

10-YEAR, 30-MIN = 1.30 in

25-YEAR, 30-MIN = 1.53 in

50-YEAR, 30-MIN = 1.86 in

100-YEAR, 30-MIN = 2.06 in

From Fig 2.5

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COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-19-90

15-MINUTE EVENTS

2-YEAR, 15-MINUTE = .62(2-YR, 1HR) = .62(0.94) = 0.58 in

100-YEAR, 15-MINUTE = .62(100-YR, 1-HR) = .62(2.52) = 1.56 in

2-YEAR, 15-MINUTE = 0.58 in

5-YEAR, 15-MINUTE = 0.80 in

10-YEAR, 15-MINUTE = 0.98 in

25-YEAR, 15-MINUTE = 1.12 in

50-YEAR, 15-MINUTE = 1.38 in

100-YEAR, 15-MINUTE = 1.56 in

From Fig. 2.5

10-MINUTE EVENTS

2-YEAR, 10-MINUTE = .51(2YR, 1HR) = .51(0.94)

=

100-YR, 10-MIN = .34(100-YR, 1-HR) = .51(2.52)

2-YEAR, 10-MINUTE = 0.48 in

5-YEAR, 10-MINUTE = 0.67 in

10-YEAR, 10-MINUTE = 0.81 in

25-YEAR, 10-MINUTE = 0.95 in

50-YEAR, 10-MINUTE = 1.22 in

100-YEAR, 10-MINUTE = 1.29 in

From Fig. 2.5

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COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-19-90

5-MINUTE EVENTS

2-YEAR, 5-MIN = .34(2-YR, 1-HR) = .34(0.94) = 0.32 in

100-YEAR, 5-MIN = .34(100-YR, 1-HR) = .34(2.52) = 0.86 in

2-YEAR, 5-MINUTE = 0.32 in

5-YEAR, 5-MINUTE = 0.47 in

10-YEAR, 5-MINUTE = 0.56 in

25-YEAR, 5-MINUTE = 0.64 in

50-YEAR, 5-MINUTE = 0.77 in

100-YEAR, 5-MINUTE = 0.86 in

From Fig. 2.5

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COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-19-90

DEPTH-DURATION-FREQUENCY CHART FOR THE WHITE TANKS / AGUA FRIA ADMS

DURATION	FREQUENCY 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
5-MIN	0.32	0.47	0.56	0.64	0.77	0.86
10-MIN	0.48	0.67	0.81	0.95	1.22	1.29
15-MIN	0.58	0.80	0.98	1.12	1.38	1.56
30-MIN	0.77	1.05	1.30	1.53	1.86	2.06
1-HR	0.94	1.30	1.60	1.89	2.23	2.52
2-HR	1.03	1.45	1.78	2.06	2.57	2.78
3-HR	1.09	1.56	1.89	2.19	2.63	2.96
6-HR	1.20	1.71	2.07	2.44	2.92	3.29
12-HR	1.30	1.88	2.29	2.70	3.25	3.66
24-HR	1.42	2.05	2.53	2.98	3.60	4.03

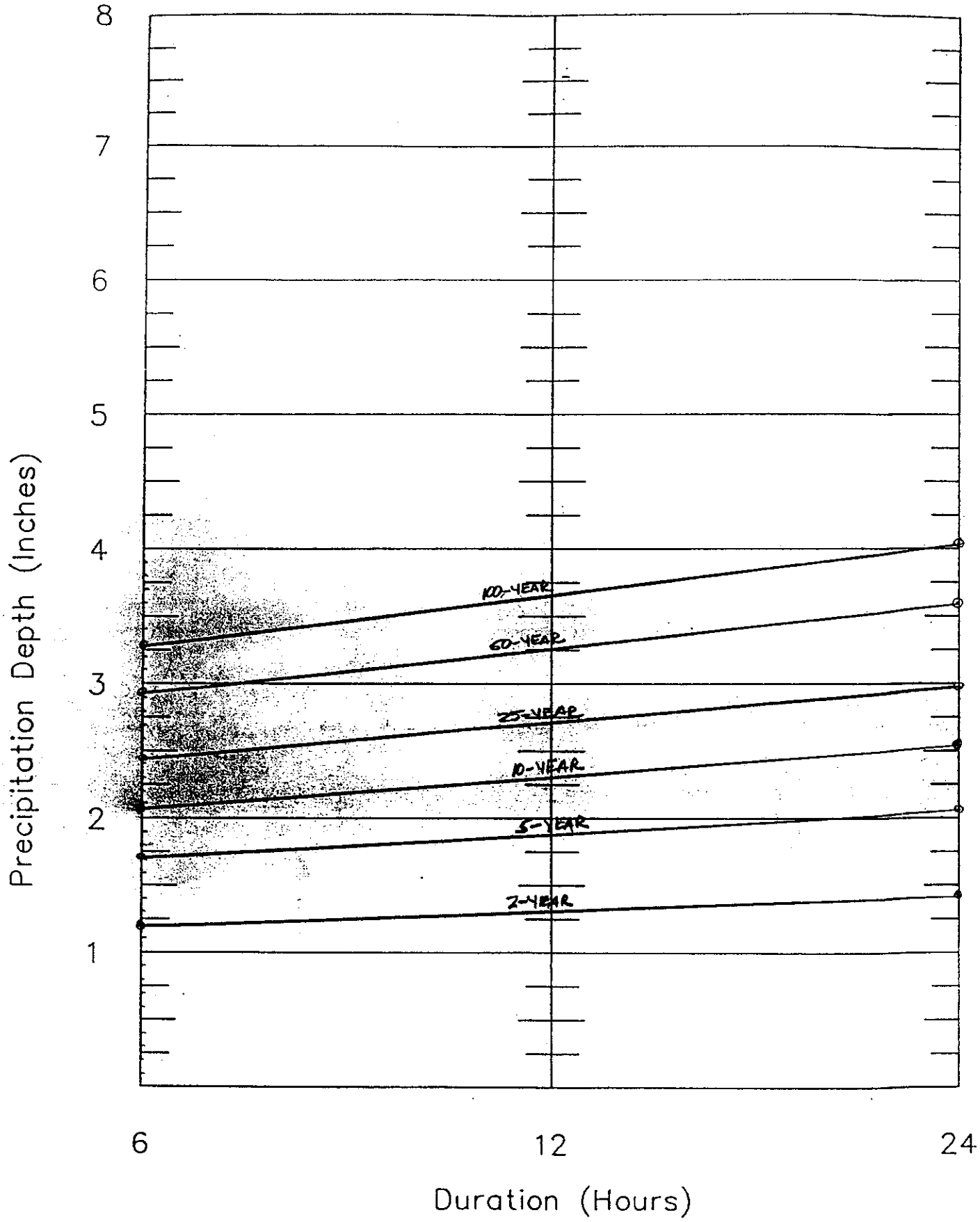

FIGURE 2.6 Precipitation depth–duration diagram (6 to 24 hr).

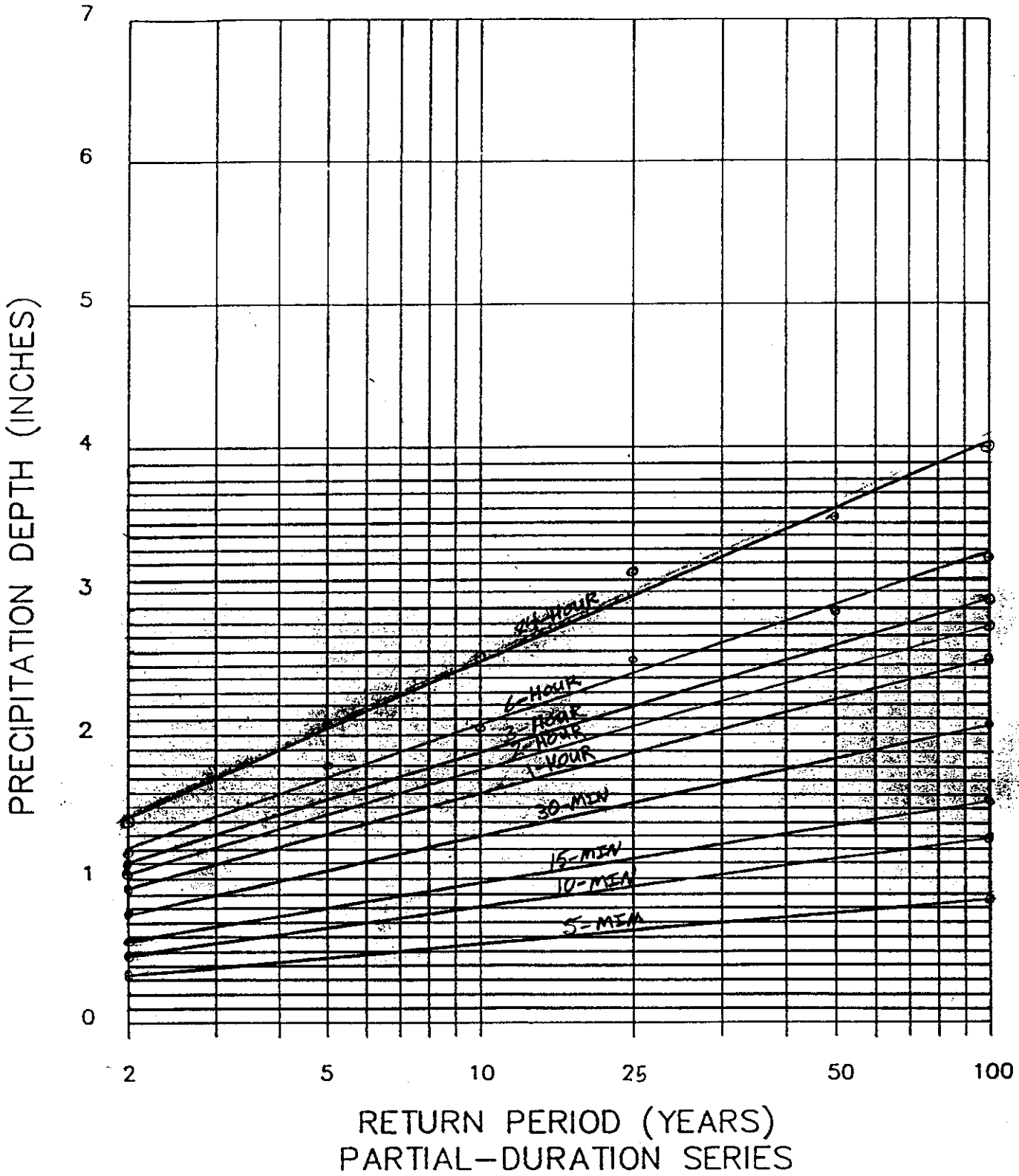

FIGURE 2.5 PRECIPITATION DEPTH VERSUS RETURN PERIOD FOR PARTIAL-DURATION SERIES

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COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-23-90

DESIGN RAINFALL DISTRIBUTION TABLE (100-YR, 6HR)

TIME (HRS)	RAINFALL (IN)	CUM. RAINFALL (IN)
0:00	0.00	0.00
0:15	0.0275	0.0275
0:30	0.0275	0.055
0:45	0.0275	0.0825
1:00	0.0275	0.110
1:15	0.0275	0.1375
1:30	0.0275	0.165
1:45	0.0275	0.1925
2:00	0.0275	0.22
2:15	0.0275	0.2475
2:30	0.045	0.2925
2:45	0.045	0.3375
3:00	0.065	0.4025
3:15	0.065	0.4675
3:30	0.23	0.6975
3:45	0.50	1.1975
4:00	1.56	2.7575
4:15	0.23	2.9875
4:30	0.065	3.0525
4:45	0.065	3.1175
5:00	0.045	3.1625
5:15	0.045	3.2075
5:30	0.0275	3.235
5:45	0.0275	3.2625
6:00	0.0275	3.29

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COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-22-90

INTENSITY - DURATION - FREQUENCY
TABLE AND GRAPH

5-MINUTE DURATION DEPTHS

2-YEAR	$\frac{0.32}{5\text{ min}} \times \frac{60\text{ min}}{\text{Hr}}$	=	3.84 in/hr
5-YEAR	$\frac{0.47}{5} \times \frac{60}{1}$	=	5.64 in/hr
10-YEAR	$\frac{0.56}{5} \times \frac{60}{1}$	=	6.72 in/hr
25-YEAR	$\frac{0.64}{5} \times \frac{60}{1}$	=	7.68 in/hr
50-YEAR	$\frac{0.77}{5} \times \frac{60}{1}$	=	9.24 in/hr
100-YEAR	$\frac{0.86}{5} \times \frac{60}{1}$	=	10.32 in/hr

10-MINUTE DURATION DEPTHS

2-YEAR	$\frac{0.48}{10} \times \frac{60}{1}$	=	2.88 in/hr
5-YEAR	$\frac{0.67}{10} \times \frac{60}{1}$	=	4.02 in/hr
10-YEAR	$\frac{0.81}{10} \times \frac{60}{1}$	=	4.86 in/hr
25-YEAR	$\frac{0.95}{10} \times \frac{60}{1}$	=	5.70 in/hr
50-YEAR	$\frac{1.22}{10} \times \frac{60}{1}$	=	7.32 in/hr
100-YEAR	$\frac{1.29}{10} \times \frac{60}{1}$	=	7.74 in/hr

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COMPUTATION DATA SHEET

Subject WHITE TANK/AGUA FRIA ADMS Prepared by JSE Date 1-22-90

15-MINUTE DURATION DEPTHS

2-YEAR	$\frac{0.58}{15} \times \frac{60}{1}$	=	2.32 in/hr
5-YEAR	$\frac{0.80}{15} \times \frac{60}{1}$	=	3.20 in/hr
10-YEAR	$\frac{0.98}{15} \times \frac{60}{1}$	=	3.92 in/hr
25-YEAR	$\frac{1.12}{15} \times \frac{60}{1}$	=	4.48 in/hr
50-YEAR	$\frac{1.38}{15} \times \frac{60}{1}$	=	5.52 in/hr
100-YEAR	$\frac{1.56}{15} \times \frac{60}{1}$	=	6.24 in/hr

30-MINUTE DURATION DEPTHS

2-YEAR	$\frac{0.77}{30} \times \frac{60}{1}$	=	1.54 in/hr
5-YEAR	$\frac{1.05}{30} \times \frac{60}{1}$	=	2.10 in/hr
10-YEAR	$\frac{1.30}{30} \times \frac{60}{1}$	=	2.60 in/hr
25-YEAR	$\frac{1.53}{30} \times \frac{60}{1}$	=	3.06 in/hr
50-YEAR	$\frac{1.86}{30} \times \frac{60}{1}$	=	3.72 in/hr
100-YEAR	$\frac{2.06}{30} \times \frac{60}{1}$	=	4.12 in/h

1-HOUR DURATION DEPTHS

2-YEAR	$\frac{0.94}{60} \times \frac{60}{1}$	=	0.94 in/hr
5-YEAR	$\frac{1.30}{60} \times \frac{60}{1}$	=	1.30 in/hr
10-YEAR	$\frac{1.60}{60} \times \frac{60}{1}$	=	1.60 in/hr
25-YEAR	$\frac{1.89}{60} \times \frac{60}{1}$	=	1.89 in/hr
50-YEAR	$\frac{2.23}{60} \times \frac{60}{1}$	=	2.23 in/hr
100-YEAR	$\frac{2.52}{60} \times \frac{60}{1}$	=	2.52 in/hr

The WLB Group Inc. WLB

COMPUTATION DATA SHEET

Subject WHITE TANKS / AGUA FRIA ADMS Prepared by JSE Date 1-22-90

2-HOUR DURATION DEPTHS

2-YEAR	$\frac{1.03}{120} \times \frac{60}{1}$	=	0.52 in/hr
5-YEAR	$\frac{1.45}{120} \times \frac{60}{1}$	=	0.73 in/hr
10-YEAR	$\frac{1.78}{120} \times \frac{60}{1}$	=	0.89 in/hr
25-YEAR	$\frac{2.06}{120} \times \frac{60}{1}$	=	1.03 in/hr
50-YEAR	$\frac{2.57}{120} \times \frac{60}{1}$	=	1.28 in/hr
100-YEAR	$\frac{2.78}{120} \times \frac{60}{1}$	=	1.39 in/hr

3-HOUR DURATION DEPTHS

2-YEAR	$\frac{1.09}{180} \times \frac{60}{1}$	=	0.36 in/hr
5-YEAR	$\frac{1.56}{180} \times \frac{60}{1}$	=	0.52 in/hr
10-YEAR	$\frac{1.89}{180} \times \frac{60}{1}$	=	0.63 in/hr
25-YEAR	$\frac{2.19}{180} \times \frac{60}{1}$	=	0.73 in/hr
50-YEAR	$\frac{2.63}{180} \times \frac{60}{1}$	=	0.88 in/hr
100-YEAR	$\frac{2.96}{180} \times \frac{60}{1}$	=	0.99 in/hr

6-HOUR DURATION DEPTHS

2-YEAR	$\frac{1.20}{360} \times \frac{60}{1}$	=	0.20 in/hr
5-YEAR	$\frac{1.71}{360} \times \frac{60}{1}$	=	0.28 in/hr
10-YEAR	$\frac{2.07}{360} \times \frac{60}{1}$	=	0.34 in/hr
25-YEAR	$\frac{2.44}{360} \times \frac{60}{1}$	=	0.41 in/hr
50-YEAR	$\frac{2.92}{360} \times \frac{60}{1}$	=	0.49 in/hr
100-YEAR	$\frac{3.29}{360} \times \frac{60}{1}$	=	0.55 in/hr

The WLB Group Inc. WLB

COMPUTATION DATA SHEET

Subject WHITE TANKS/AGUA FRIA ADMS Prepared by JSE Date 1-22-90

12-HOUR DURATION DEPTHS

2-YEAR $\frac{1.30}{720} \times \frac{60}{1} = 0.11$ in/hr

5-YEAR $\frac{1.88}{720} \times \frac{60}{1} = 0.16$ in/hr

10-YEAR $\frac{2.29}{720} \times \frac{60}{1} = 0.19$ in/hr

25-YEAR $\frac{2.70}{720} \times \frac{60}{1} = 0.23$ in/hr

50-YEAR $\frac{3.25}{720} \times \frac{60}{1} = 0.27$ in/hr

100-YEAR $\frac{3.66}{720} \times \frac{60}{1} = 0.31$ in/hr

24-HOUR DURATION DEPTHS

2-YEAR $\frac{1.42}{1440} \times \frac{60}{1} = 0.06$ in/hr

5-YEAR $\frac{2.05}{1440} \times \frac{60}{1} = 0.08$ in/hr

10-YEAR $\frac{2.53}{1440} \times \frac{60}{1} = 0.10$ in/hr

25-YEAR $\frac{2.98}{1440} \times \frac{60}{1} = 0.12$ in/hr

50-YEAR $\frac{3.60}{1440} \times \frac{60}{1} = 0.15$ in/hr

100-YEAR $\frac{4.03}{1440} \times \frac{60}{1} = 0.17$ in/hr

The WLB Group Inc. WLB

COMPUTATION DATA SHEET

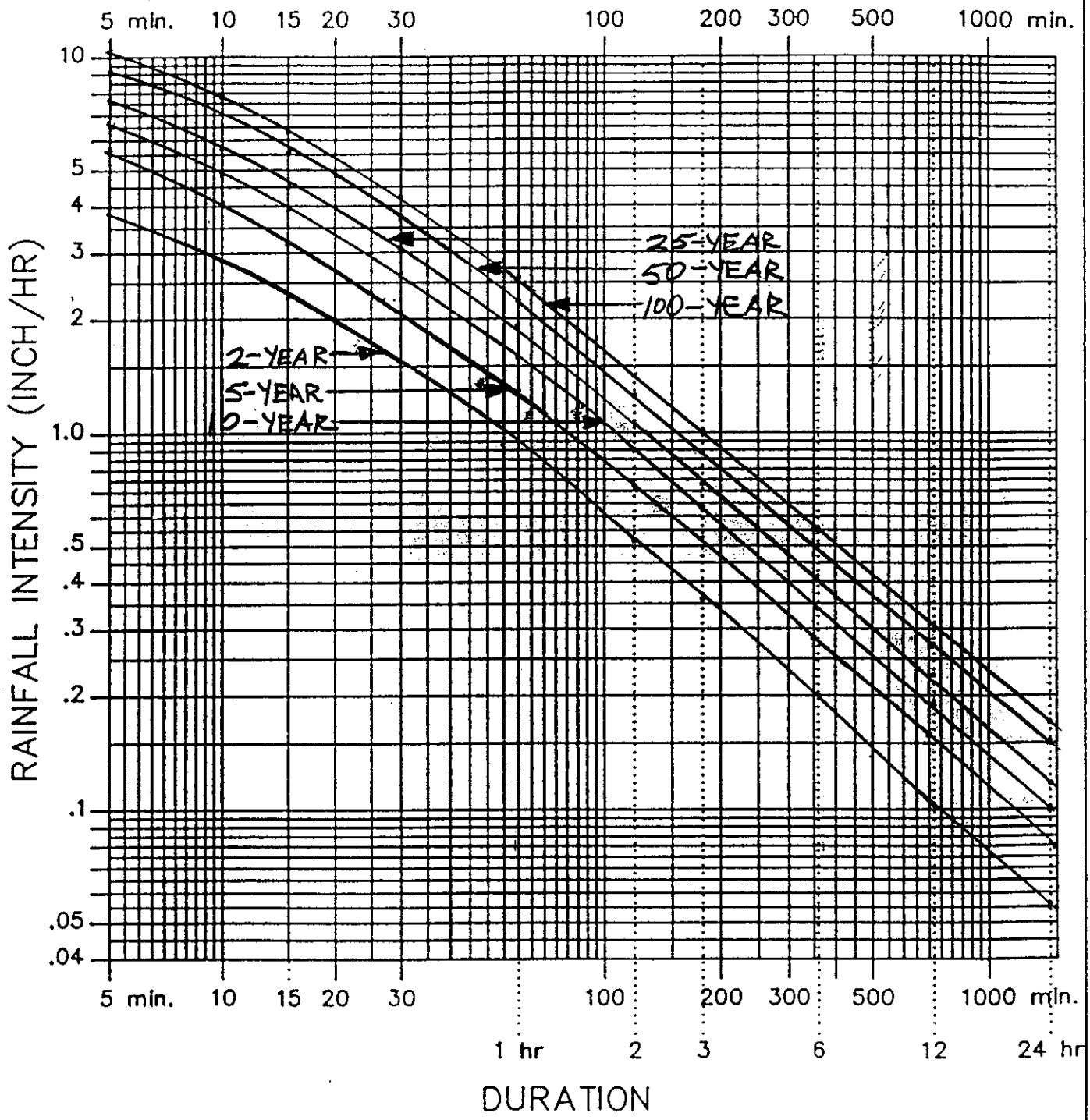
Subject WHITE TANKS/AGUA FRIA ADMS Prepared by JSE Date 1-22-90

INTENSITY - DURATION - FREQUENCY TABLE FOR

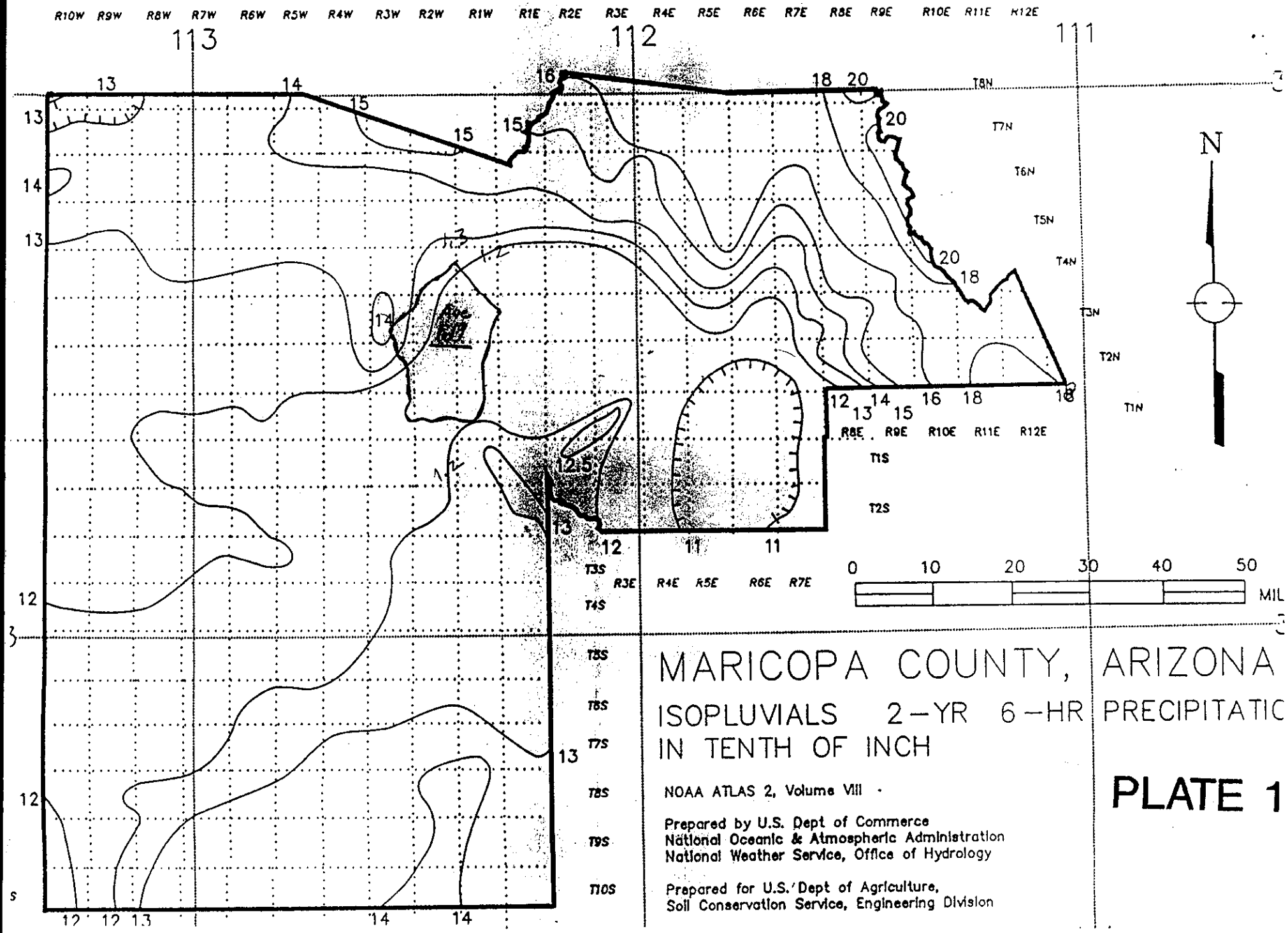
WHITE TANKS / AGUA FRIA ADMS

DURATION	FREQUENCY 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
5-MIN	3.84	5.64	6.72	7.68	9.24	10.32
10-MIN	2.88	4.02	4.86	5.70	7.32	7.74
15-MIN	2.32	3.20	3.92	4.48	5.52	6.24
30-MIN	1.54	2.10	2.60	3.06	3.72	4.12
1-HR	0.94	1.30	1.60	1.89	2.23	2.52
2-HR	0.52	0.73	0.89	1.03	1.28	1.39
3-HR	0.36	0.52	0.63	0.73	0.88	0.99
6-HR	0.20	0.28	0.34	0.41	0.49	0.55
12-HR	0.11	0.16	0.19	0.23	0.27	0.31
24-HR	0.06	0.08	0.10	0.12	0.15	0.17

WHITE TANKS / AUGA FRIA ADMS



RAINFALL INTENSITY–DURATION–FREQUENCY RELATION
FOR MARICOPA COUNTY, ARIZONA



MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 2-YR 6-HR PRECIPITATIC IN TENTH OF INCH

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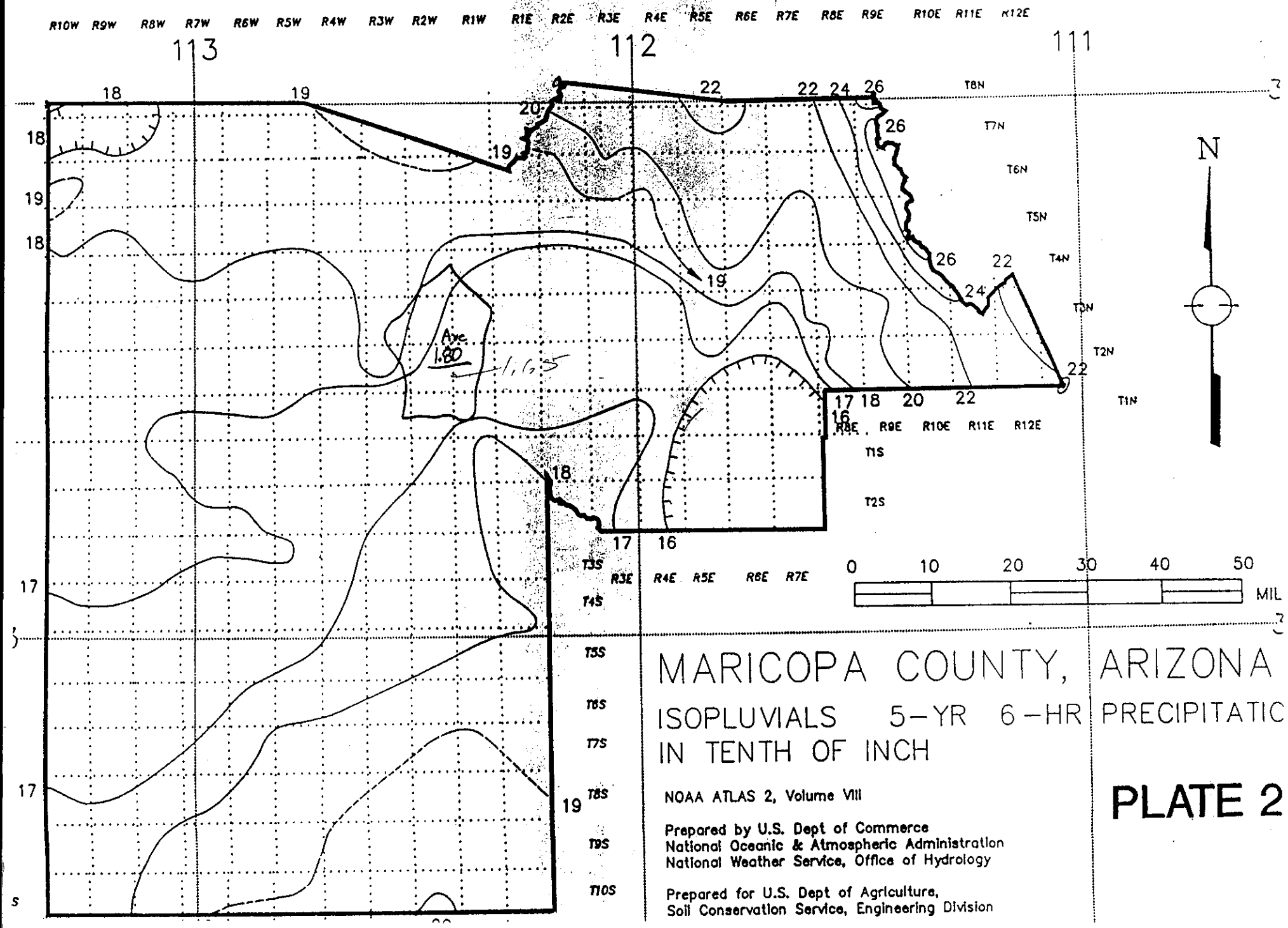
PLATE 1



MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 5-YR 6-HR PRECIPITATIC IN TENTH OF INCH

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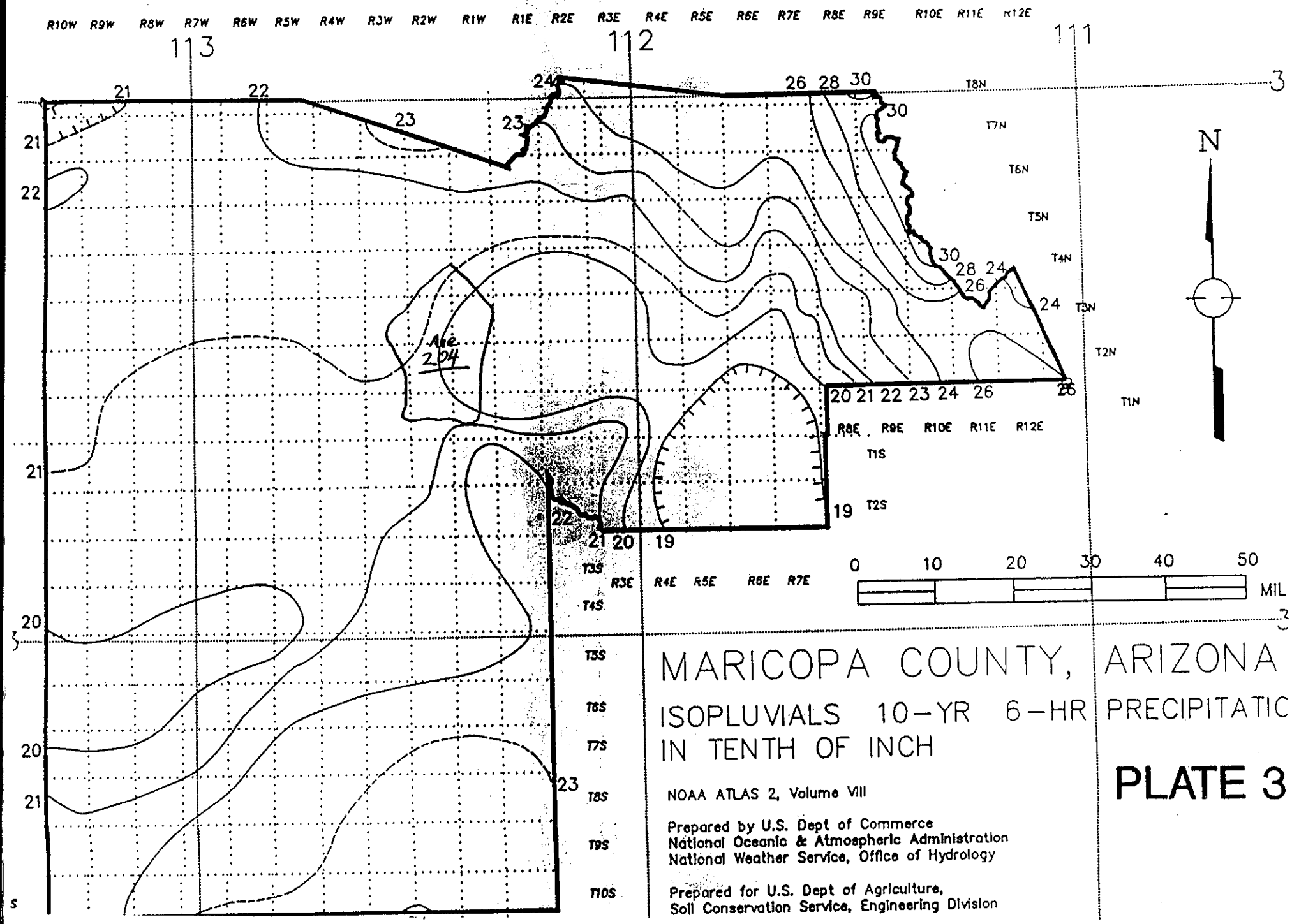
PLATE 2



MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 10-YR 6-HR PRECIPITATIC IN TENTH OF INCH

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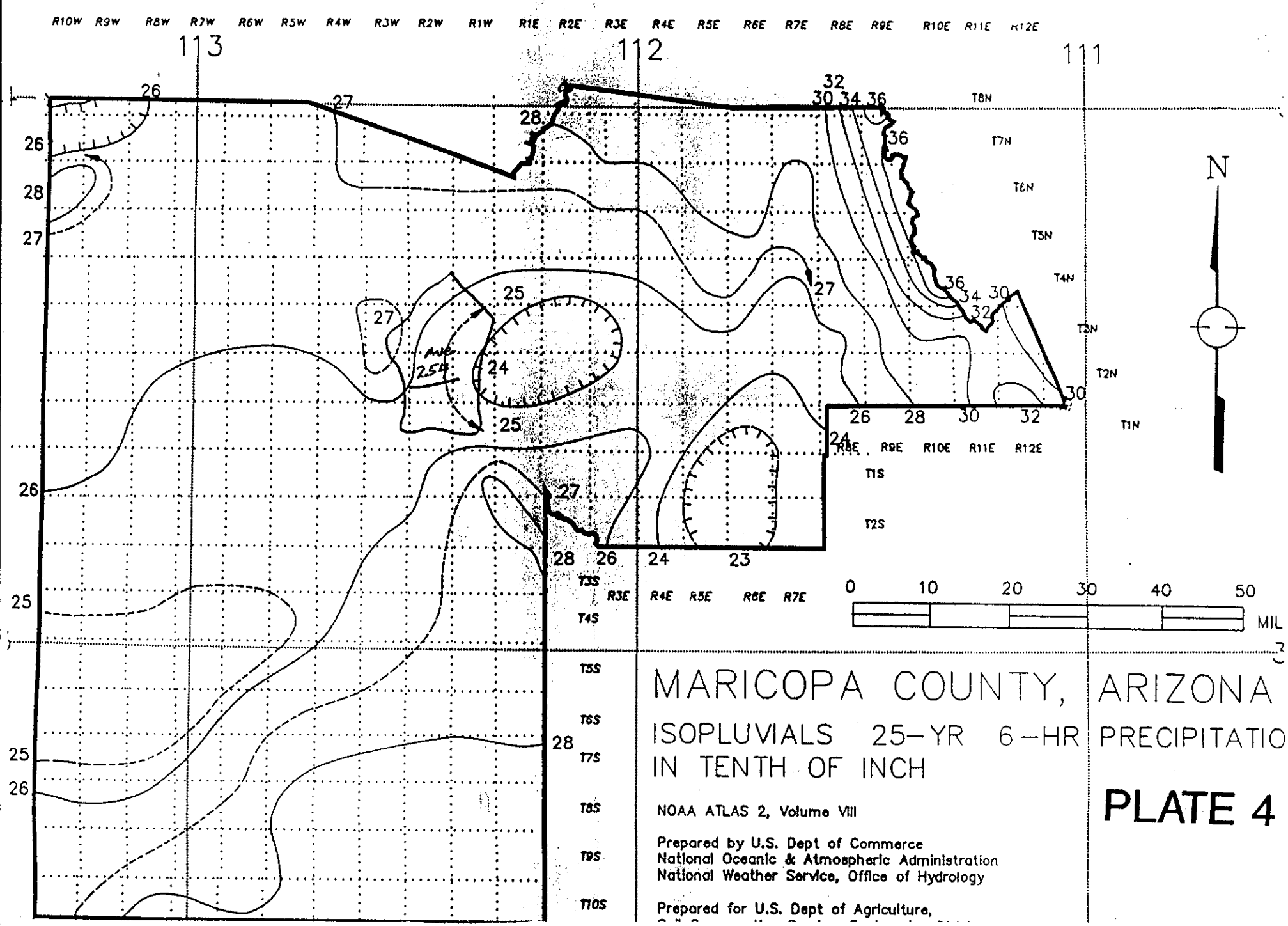

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 25-YR 6-HR PRECIPITATIO IN TENTH OF INCH

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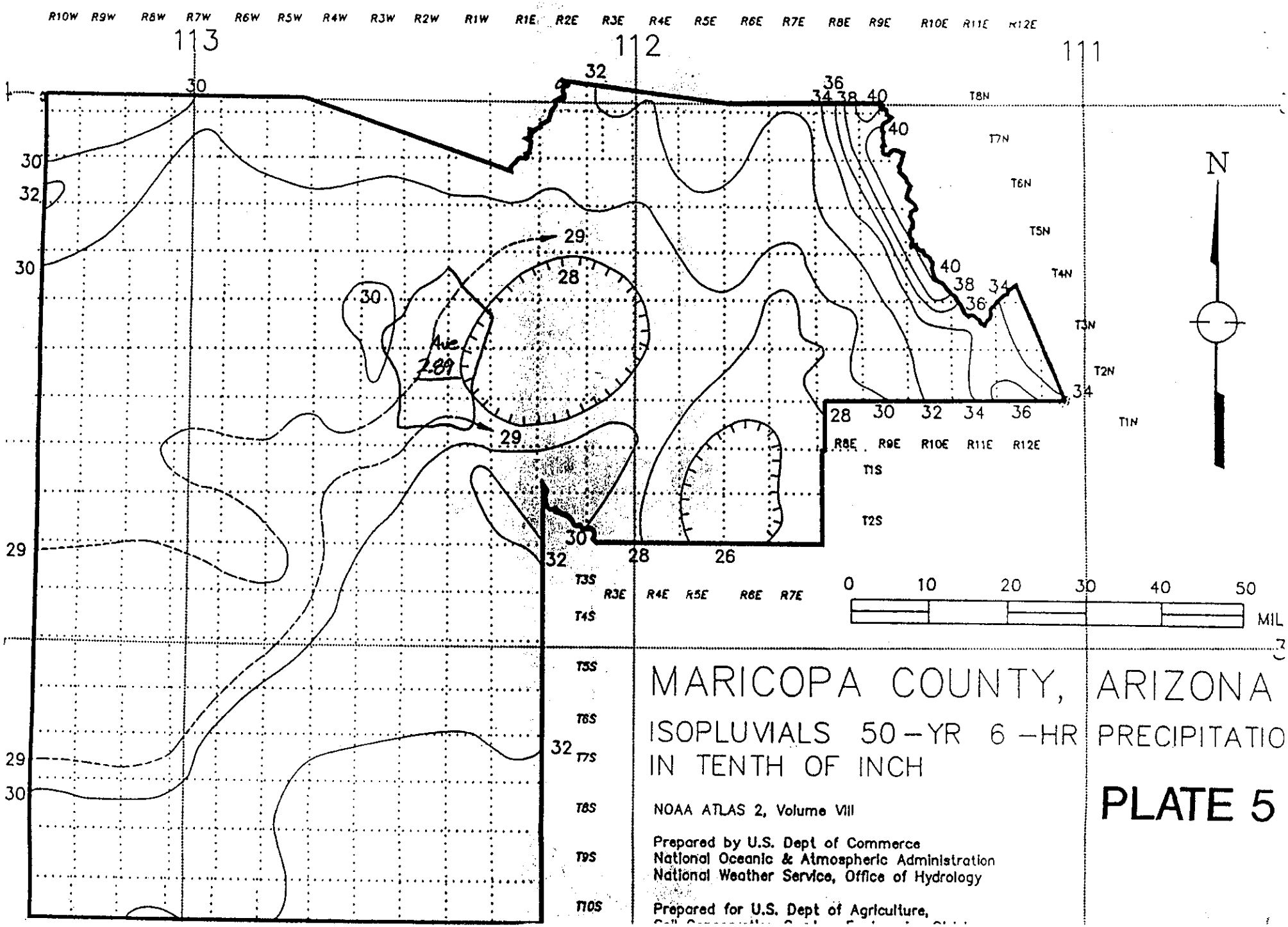

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 50-YR 6-HR PRECIPITATIO IN TENTH OF INCH

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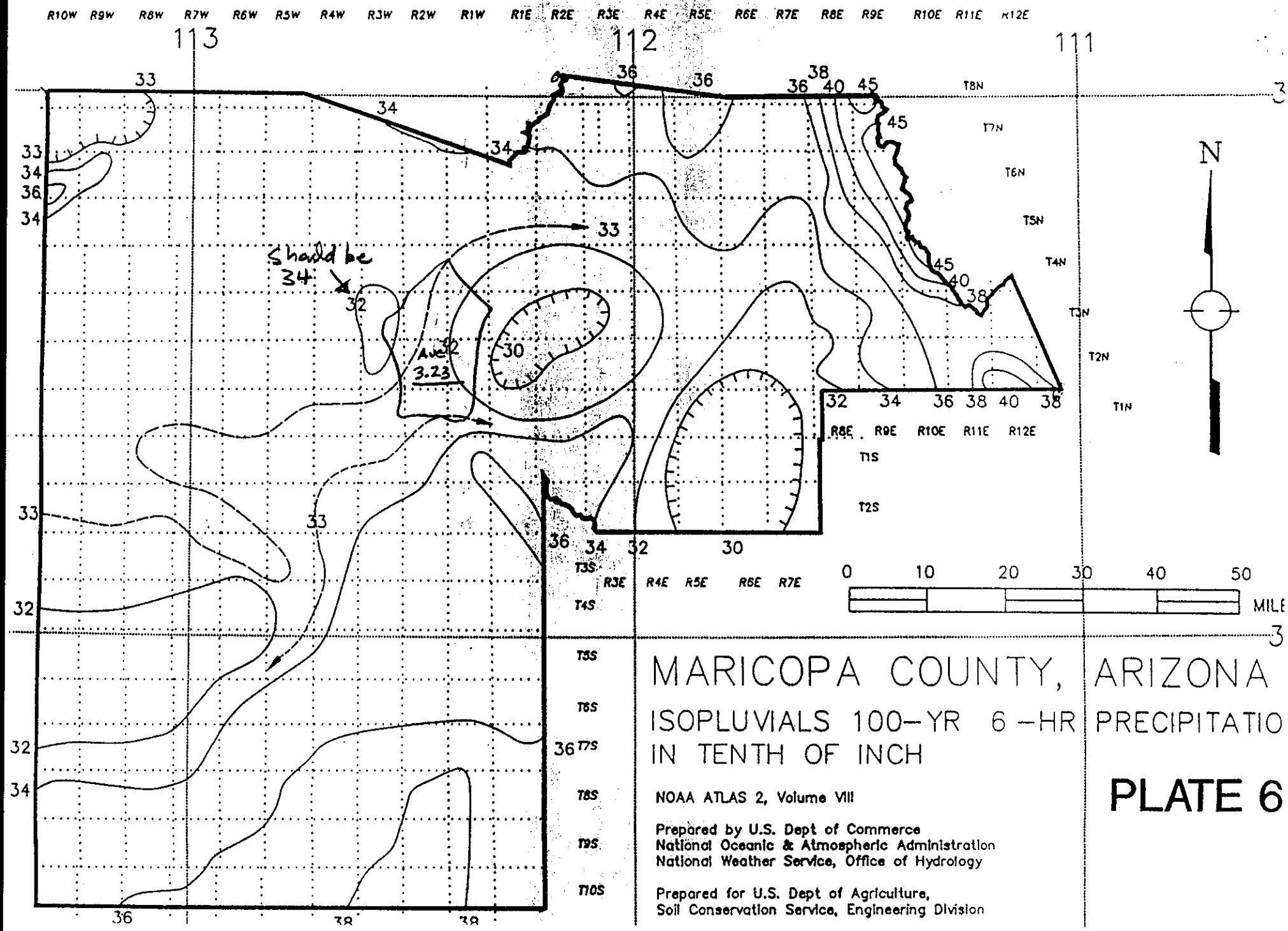

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 100-YR 6-HR PRECIPITATIO IN TENTH OF INCH

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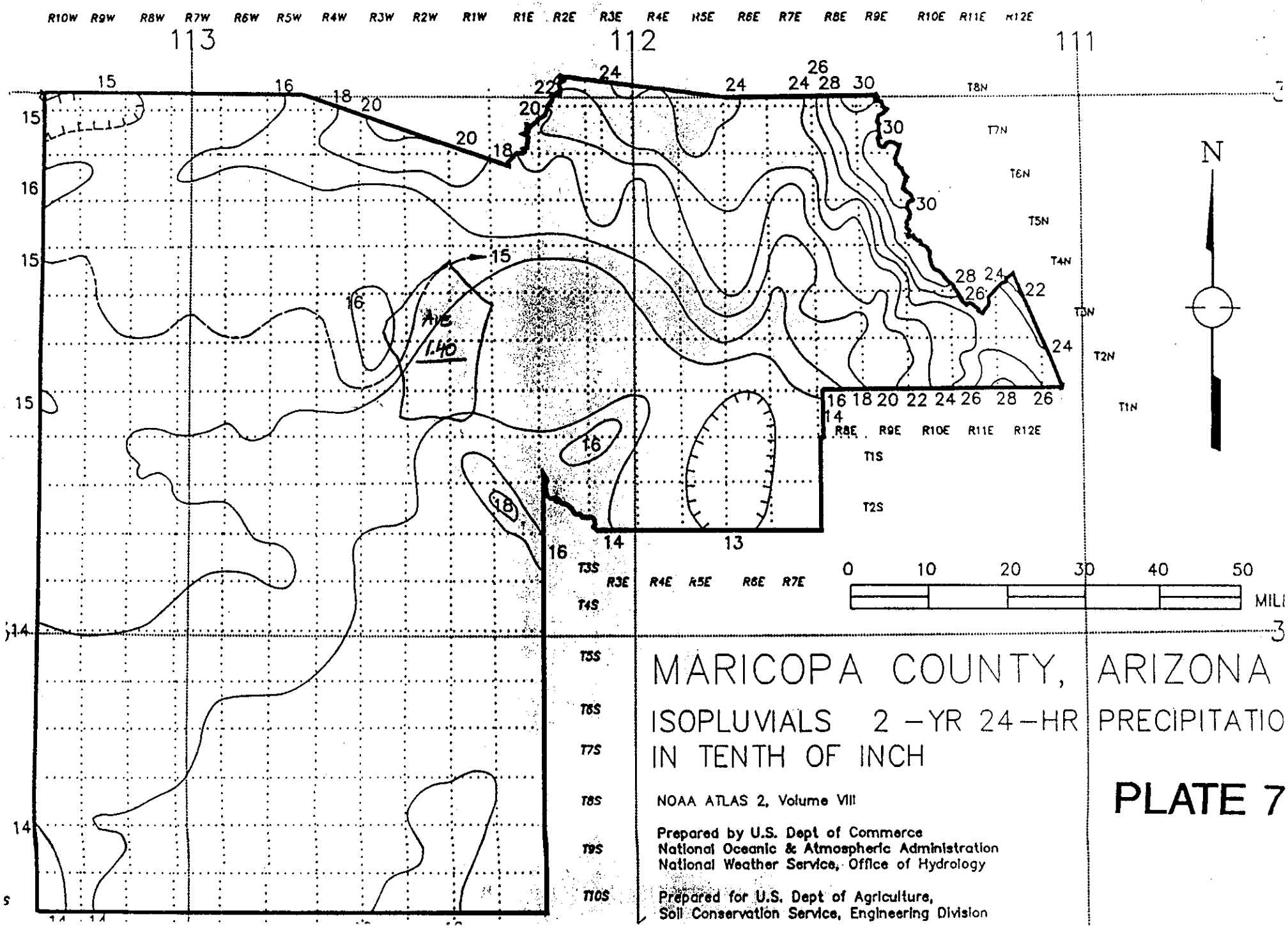

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 2 –YR 24–HR PRECIPITATIO IN TENTH OF INCH

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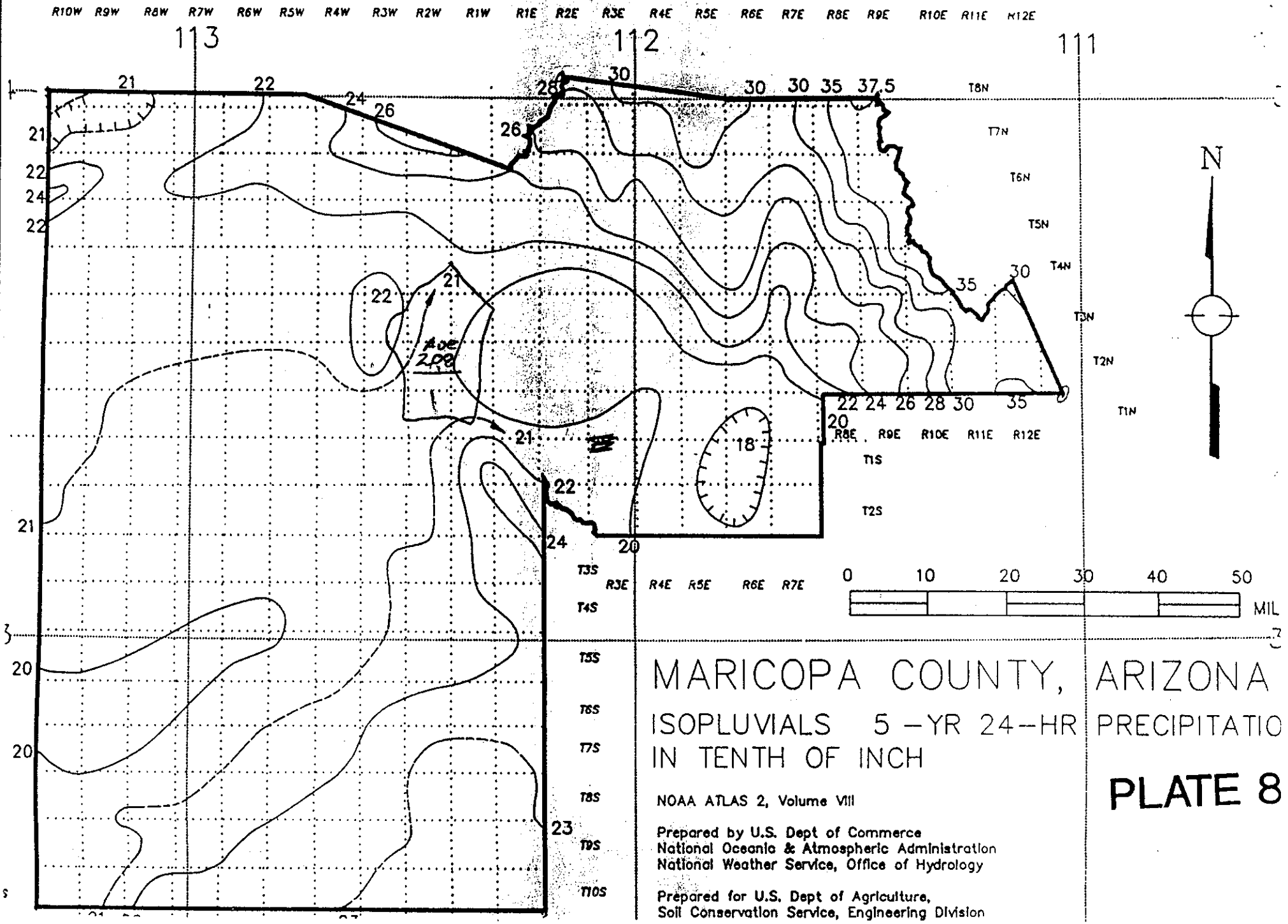

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 5 –YR 24–HR PRECIPITATIO IN TENTH OF INCH

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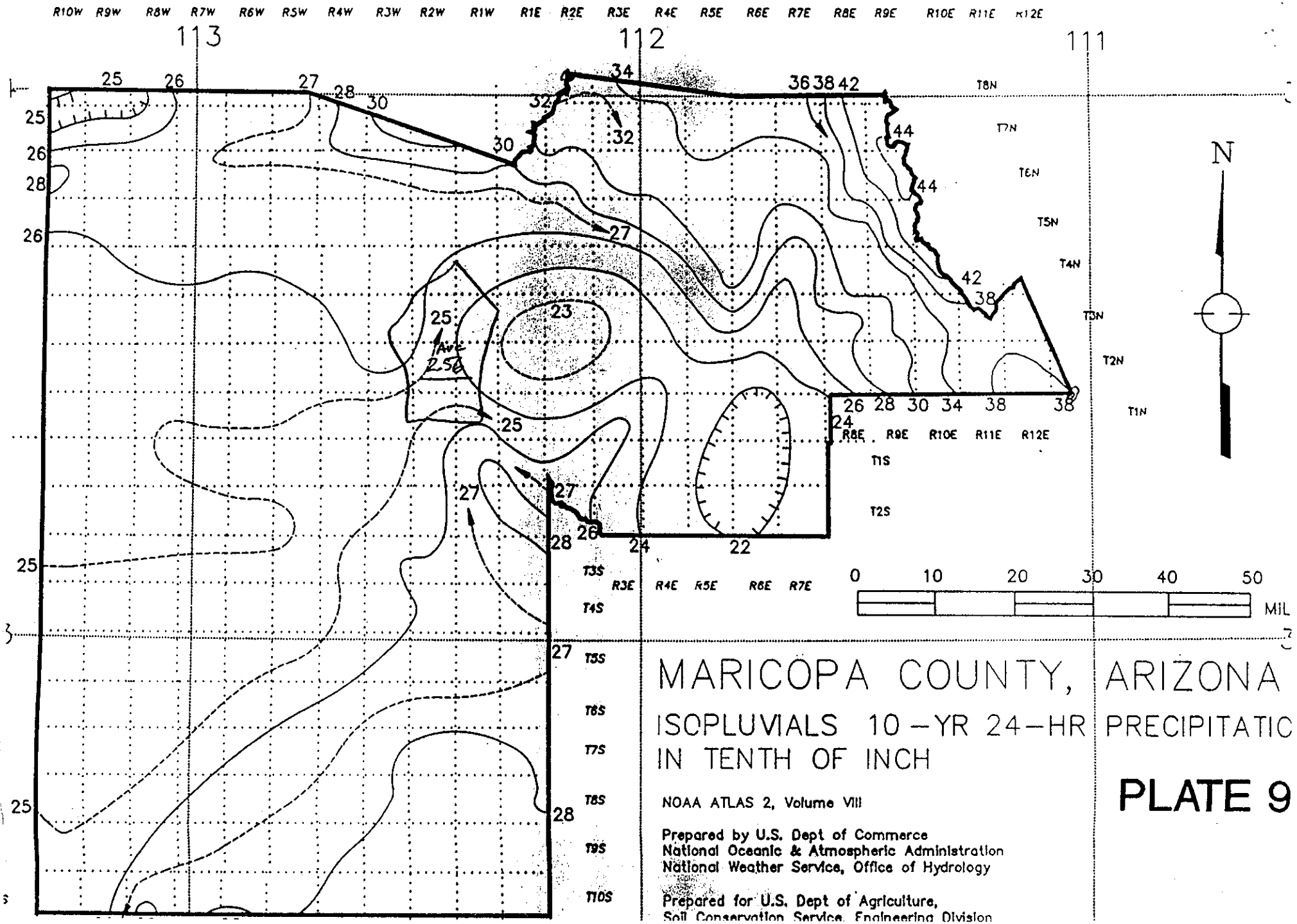

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 10-YR 24-HR PRECIPITATIC IN TENTH OF INCH

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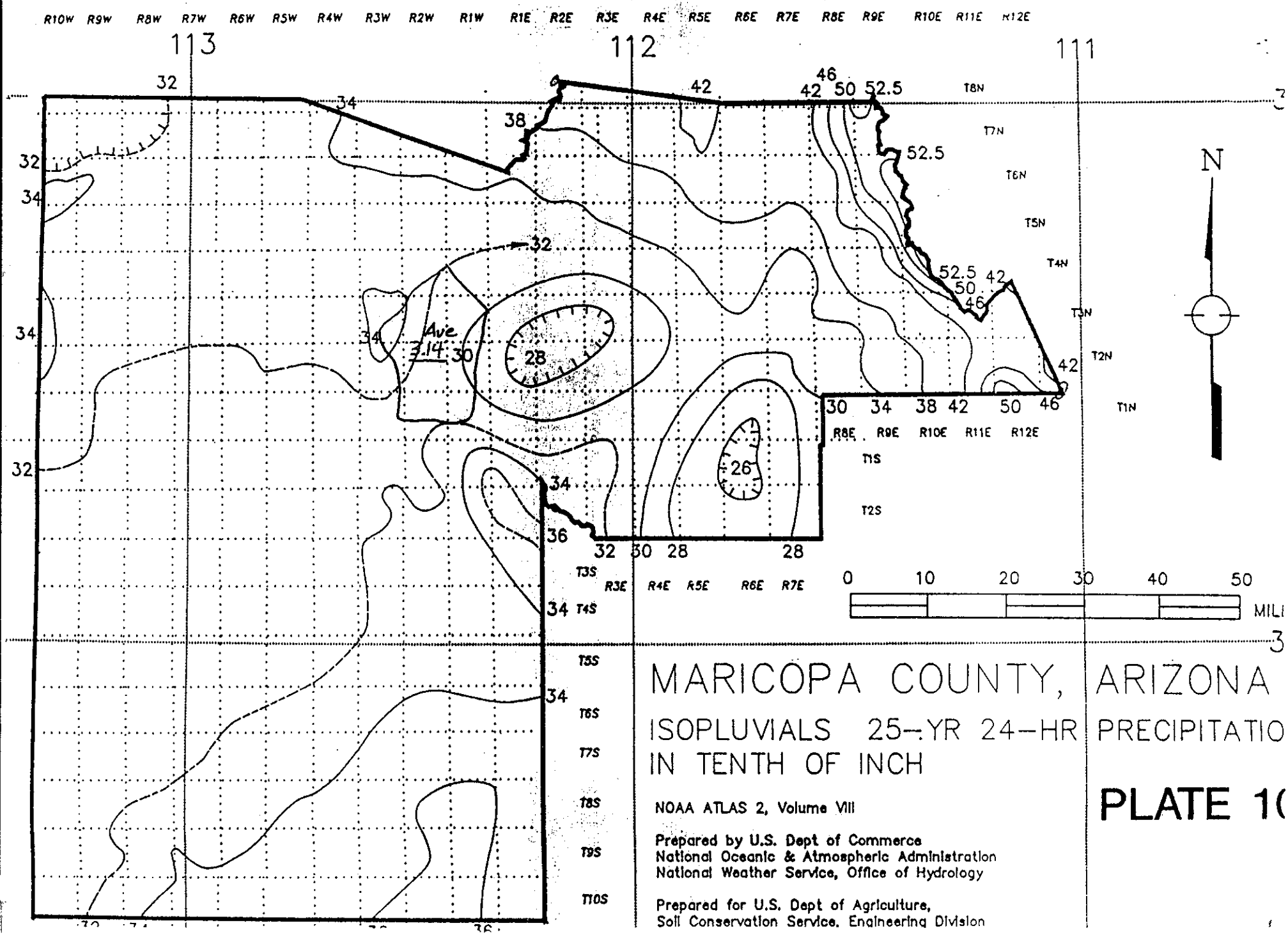

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 25-YR 24-HR PRECIPITATIO IN TENTH OF INCH

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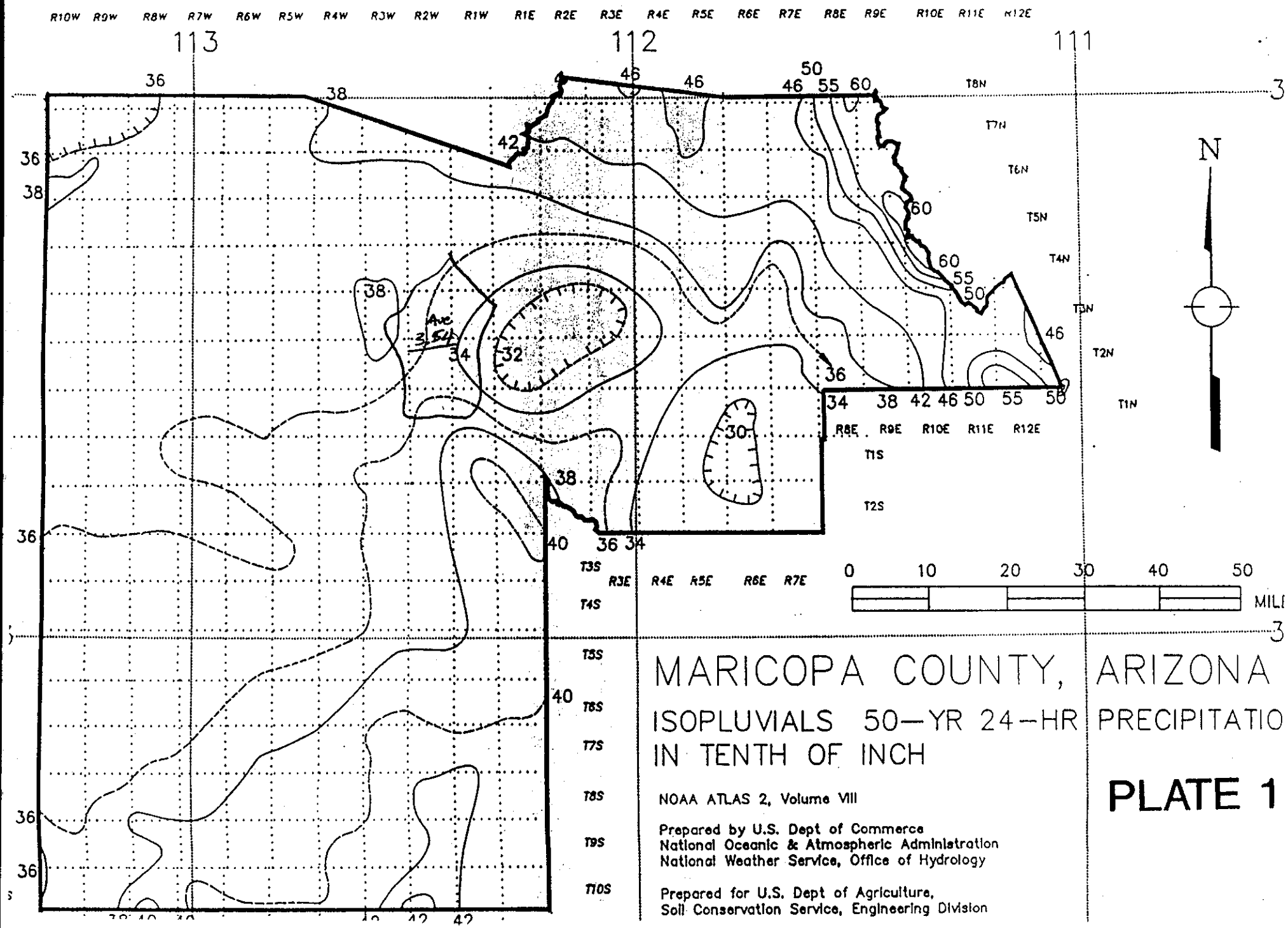

MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 50—YR 24—HR PRECIPITATIO IN TENTH OF INCH

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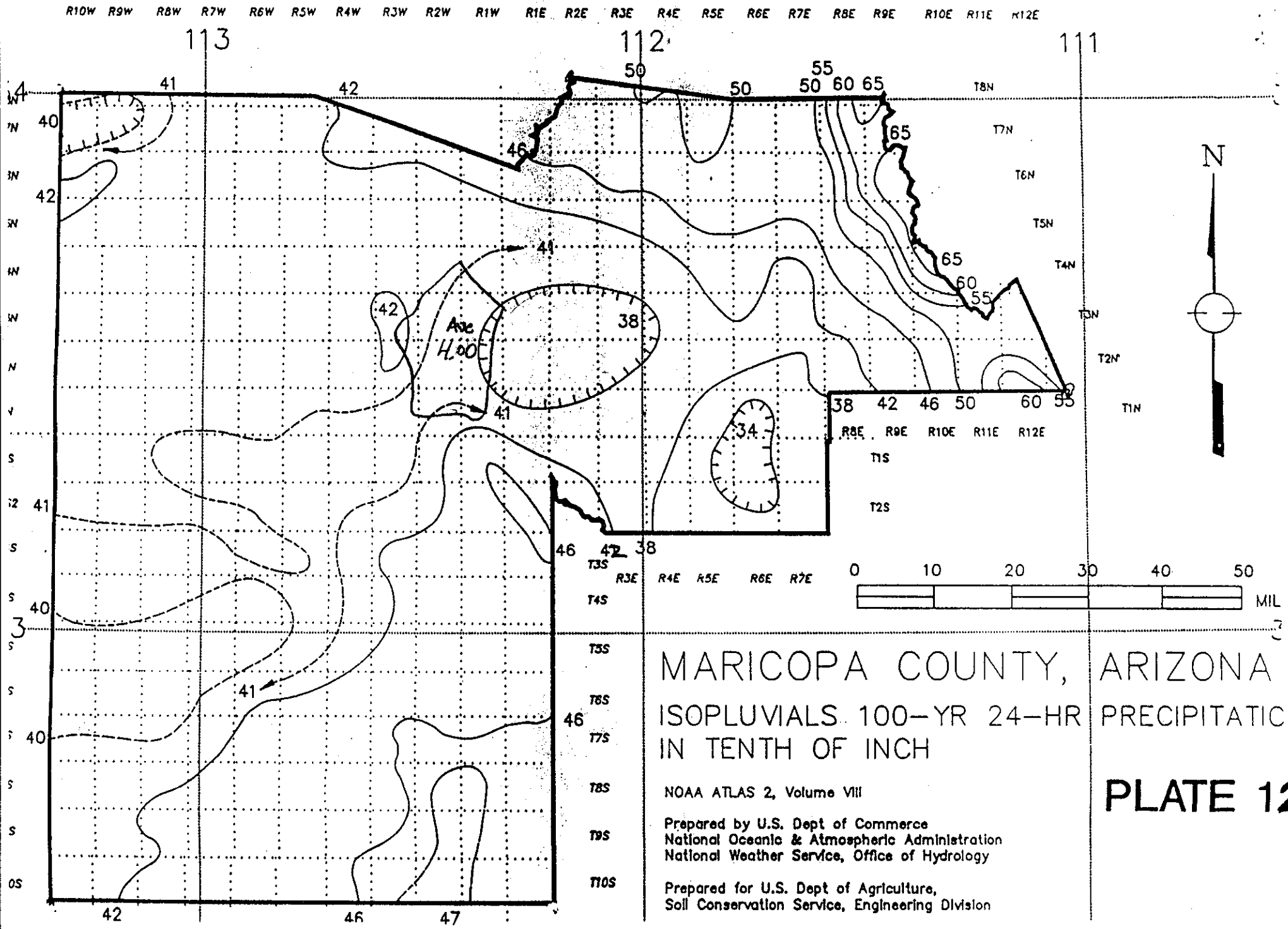

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