VOLUME I - REPORT
FINAL DAMBREAK ANALYSIS FOR THE HARQUAHALA AND SADDLEBACK FLOOD RETARDING STRUCTURES
Project No. FCD-88-66

Prepared for:
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

Prepared by:
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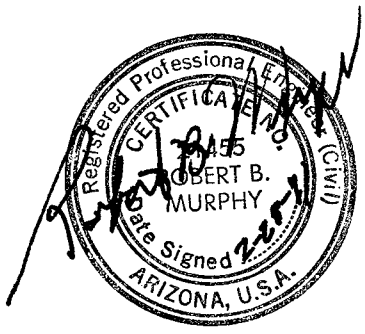

February 1991
Carter Job No. 06-016-83394-02

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CARTER ASSOCIATES, INC.

February 25, 1991

Mr. Steve Waters
Project Manager and Hydrologist
FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
3335 W. Durango
Phoenix, AZ 85009

RE: FINAL DAMBREAK ANALYSIS FOR THE HARQUAHALA AND SADDLEBACK FLOOD RETARDING STRUCTURES

Dear Steve:

Carter Associates, Inc. is pleased to present five (5) copies of the Final Dambreak for the Harquahala Flood Retarding Structure (HQFRS) and Saddleback Flood Retarding Structure (SBFRS).

As specified in the contract scope of work, this submittal consists of all the calculations and assumptions used in the preparation of the Final Evacuation Plan (contained herein and under separate cover). All supporting data and computer model outputs are presented in the Appendices (Volume II) of this report. Floppy disks (5) of all computer generated work products are also provided at this time.

It is a continued pleasure to work with the District on this interesting project. We look forward to answering any questions you may have on our work.

Sincerely,

CARTER ASSOCIATES, INC.

Robert B. Murphy, P.E.
Senior Hydrologist/Project Manager

5080 North 40th Street • Suite 300 • Phoenix, Arizona 85018 • (602) 955-0900

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ABBREVIATIONS

Abbreviation	Definition
AMC	antecedent moisture condition
CAP	Central Arizona Project
cfs	cubic feet per second
csm	cubic feet per second per square mile
DARF	drainage-area-reduction factor
FCDMC or FCD	Flood Control District of Maricopa County
FRS	Flood Retarding Structure
GRAD	Granite Reef Aqueduct Dike, Reach 6
HEC	Hydrologic Engineering Center, Corps of Engineers
HMR	Hydrometerological Report
HQFRS	Harquahala Flood Retarding Structure
LF	lineal feet
NOAA	National Oceanic and Atmospheric Administration
NWS	National Weather Service
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RCP	reinforced concrete pipe
SCS	Soil Conservation Service
sq.mi.	square mile
SQFRS	Saddleback Flood Retarding Structure
Tc	time of concentration
TL	time of lag
USGS	U.S. Geological Survey

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INTRODUCTION

The Flood Control District of Maricopa County ("District") conducted this dambreak analysis on the Harquahala Flood Retarding Structure (HQFRS) and the Saddleback Flood Retarding Structure (SBFRS) as part of the dam certification requirements set forth by the Arizona Department of Water Resources (ADWR). The purpose of the study was to develop an evacuation plan for residents downstream of the HQFRS and SBFRS should one of these structures fail. The analysis was based on the available technical and structural information, hydrologic and hydraulic methodologies and current ADWR dam safety criteria.

The preparation of an evacuation plan requires that estimates be made of the travel time and limits of flood inundation resulting from the sudden release of water from a failed dam. The limits of inundation will be approximated by first estimating the probable maximum flood (PMF) coming to the FRS. Second, failure hydrographs will be developed by modeling dam failures (breach analysis). Third, failure hydrographs were routed through the study area (Dambreak analysis). Each of the three elements of the evacuation plan will be presented in a separate section within this report. The evacuation plan will be discussed in the fourth and final section of this report.

The purpose for the hydrologic analysis in this report is to present the basic hydrology used to estimate the PMFs in this study. The District has designated the HQFRS and SBFRS as being "medium" in size and significant in hazard. Additionally, the District specified the use of the half PMF for the dambreak analysis.

To aid in the estimation of the PMF, investigations of previous studies were made. Numerous studies were completed previously during the design phase of the HQFRS or the SBFRS.

The hydrologic section also describes and locates the primary and secondary structures. A determination was made as to which primary and/or secondary structures have a direct impact on the PMF.

The area of watersheds tributary to the HQFRS and the SBFRS were estimated. The topographic characteristics of the tributary watersheds along with the soil, vegetative cover and land use characteristics were investigated and utilized appropriately to estimate the PMFs.

To estimate the PMF, this study used the HEC-1 computer program and the SCS option. Curve numbers, lag times and probable maximum preciptations were calcuated for HQFRS and SBFRS watershed areas. When the routing of design flows was necessary, the Muskingum

method or the level pool method was used. The diversion routine in HEC-1 was utilized to modify the PMF to obtain the half PMF.

The purpose of the breach analysis in this report is to estimate HQFRS and SBFRS failure hydrograph information. To estimate the breach parameters, the microversion of the National Weather Service Breach Model, dated January 1987, was used. The information presented in this section includes:

1. Routing of the 6-hour and 72-hour 1/2 probable maximum flood (PMF) hydrograph through each FRS to estimate the potential for overtopping.
2. Based on the type of failure mechanism selected from Step 1 above, estimate breach parameters such as breach size, shape and formation time.
3. Using the parameters from the breach model for the hydrograph which produced the highest peak discharge, run the Dambreak-88 model for each failure to estimate failure hydrographs for 3 breach locations on the HQFRS and 2 breach locations on the SBFRS.

The dambreak section of this report presents the routing of the hydrographs resulting from the piping failure of the Harquahala and Saddleback FRSs. This was accomplished by using the Boss Corporation version of the National Weather Service model DAMBRK. This model was used to estimate the peak discharge, peak stage and

floodwave travel time at specific cross-sections downstream of each structure. This section also describes the inputs, methods and assumptions that were used in this analysis and the results obtained.

The evacuation plan presents an overview of the potential social and economic impacts anticipated for the five dambreak failures. This section will also qualitatively address potential damage due to dambreak flood inundation.

PROJECT AUTHORIZATION

The project was authorized by a contract between the District and Carter Associates, Inc. dated March 12, 1990 and has a District designation of FDC 88-66.

HYDROLOGY

Previous Studies

There have been numerous hydrologic, hydraulic, and design studies done on the HQFRS and SBFRS. The majority of the HQFRS studies were done by the Soil Conservation Service (SCS), while the majority of the SBFRS studies were prepared by the SCS and the PRC TOUPS CORPS (PRC, Ref. 8). Several of these studies or reports have been reviewed and pertinent information and/or data was used in this study. The information and/or data is presented and referenced in other sections of this report. Studies and reports have been listed within the references.

Project Description

The HQFRS and SBFRS are located in northwestern Maricopa County. The HQFRS is located north of I-10 and west of Burnt Mountain (Figure 1). The Central Arizona Project (Granite Reef Aqueduct Reach 5B) lies just south and parallel to the HQFRS. Additional descriptions of the HQFRS features such as length, height, primary and secondary outflow structures, etc. are found in Table 1.

The SBFRS is located south of I-10 and northwest of Saddleback Mountain, in Township 2 North, Range 8 West, Sections 17, 20, 21, 27, 28 and 34.

The HQFRS and the SBFRS are the two primary structures in this study and in the project. There are also eight secondary structures. The primary structures (HQFRS and SBFRS) are those structures on which failure will be analyzed under the half PMF and conditions.

Of the eight secondary structures, two are defined as those barriers that retain, detain or divert rainfall runoff to or away from the primary structures. These two secondary structures are the Granite Reef Aqueduct Dike, Reach 6 (GRAD) and I-10 (Figure 2). The remainder of the secondary structures are defined as those man-made physical barriers that may detain, retain or divert flood waters from specific downstream areas of the HQFRS or SBFRS. A listing of the secondary structures are as follows:

1. Link Canal
2. Westside Canal Dike
3. Centennial Levee
4. Eagletail Levee
5. Harquahala Floodway
6. Saddleback Diversion
7. I-10
8. Granite Reef Aqueduct Dike

The GRAD and I-10 are the secondary structures discussed further in this report. The remaining structures will be considered in subsequent reports.

Of the eight secondary structures, the GRAD and I-10 are the only structures that impact on the primary structures. The GRAD intercepts storm runoff from approximately 54.7 square miles (sq.mi.) of watershed, a portion of which would have contributed directly to the SBFRS. It was reported (Ref. 8) that the GRAD is a compacted, earth-filled levee approximately 20 feet high with a detention capacity of about 7,500 acre-feet (ac-ft) below the crest of dike (el. 1,399). Flow is released downstream from the dike through five ungated 72-inch conduit overchutes. The SBFRS is impacted by only the western-most overchute immediately east and adjacent to Burnt Mountain (Figure 2). The function of the GRAD is to prevent rainfall runoff from draining into the Granite Reef Aqueduct (Reach 6).

Interstate 10 crosses many of the watersheds that drain to the SBFRS. The interstate consists of two asphalt traffic lanes (plus shoulders) per east or west traffic flow. The two sets of lanes are raised 1 to 3 feet from the desert floor and are separated by a natural desert median (Ref. 26 and 27). I-10 has numerous drainage structures that convey runoff from the tributary watersheds and the applicable GRAD overchute to the SBFRS.

Although I-10 crosses many of the SBFRS watersheds, this structure will not be analyzed as to its impact of impeding stormwater runoff from reaching the SBFRS. The main reason for this assumption is

that the drainage structures under the interstate were designed for up to 1-hour, 10-year rainfall events. During the 6-hour or 72-hour PMF event, these structures will be inundated and the peak flows will continue across the interstate to the SBFRS relatively unimpeded. Additionally, this study's objective is to make assumptions that are conservative with respect to estimating limits of inundation.

Watershed Description

The watersheds' tributary to the HQFRS and the SBFRS generally have similar topography, soil types, vegetative cover and land uses.

Topgraphy and Watershed Areas. There are four subwatersheds (Subwatersheds 1 through 4) which drain to the HQFRS (Figure 3). The topography for these subwatersheds vary from the steep rugged slopes of the Big Horn Mountains to the gently sloping alluvial fans and valley plains. These watersheds drain north to south beginning in steep canyons and washes and extend to the deep foothill gullies and into shallow, braided and meandering alluvial fan washes. In the upper reaches of the watershed, the ground slopes are as steep as 70 percent with values generally ranging from 20 to 40 percent. The general slopes in the foothill gullies range from 1.3 to 5.7 percent. The typical slopes for the alluvial fan channels range from 0.9 to 2.1 percent.

These subwatersheds range in size from 11.25 sq.mi. to 48.63 sq.mi. The HQFRS subwatersheds with their respective tributary areas are given in Table 2 and are shown on Figure 3.

The topographic characteristics of the watersheds' tributary to the SBFRS are similar to those of the HQFRS. Except for Subwatershed 13, Subwatersheds 5 through 12 typically drain in a south to southwesterly direction. Subwatershed 13 drains from Saddleback Mountain in a northwesterly direction to the SBFRS. The SBFRS subwatersheds drain from the steep canyons and washes in the Big Horn Mountains, Belmont Mountains and Saddleback Mountains to deep foothill gullies and into the shallow, braided washes of the alluvial fans. In the upper reaches of the SBFRS watershed, the steep canyons and washes slope as much as 70 percent with values generally ranging from 15 to 67 percent. The general range of slopes in the gullies of the foothills is between 1 and 10 percent and 0.40 to 2 percent in the shallow washes of the alluvial fans.

A portion of the watershed tributary to the SBFRS is regulated. Of the 76.93 square miles, only 22.20 square miles drain unregulated to the SBFRS (Subwatersheds 8 through 13). Subwatersheds 5 through 7, 54.73 square miles, drain to the Granite Reef Aqueduct dike where the runoff is detained and discharged through five 72-inch overchutes. Only one of the five overchutes discharges to a subwatershed that is tributary to the SBFRS. This overchute is located near Burnt Mountain and drains into

Subwatershed 8 (Figure 3). The remaining overchutes discharge to the east into the Tonopa Desert. Table 3 lists the subwatersheds tributary to the SBFRS and their respective areas.

The selection of subwatershed boundaries and tributary areas were based largely on the SCS Supplemental Report for the HQFRS (Ref. 3) and the PRC report on the SBFRS (Ref. 8). Watershed boundaries were modified based on field reconnaissance by District and Carter Associates' staff and by close examination of the USGS topography maps. All modifications were in the subwatersheds triburary to the SBFRS (Subwatersheds 8 through 12). The changes to the subwatershed areas are given in Table 4.

Other modifications resulting from field reconnaissance by District and Carter staff were made due to channel splitting. This occurs in Subwatershed 8. The assumption was made by the District and Carter staff that 50 percent of the stormwater will continue into Watershed Basin 9 while the remainder will be diverted and flow southwest into Subwatershed 10. The Hydrology Map shows the location and direction of the flow split.

Soils. The soil types within the watersheds tributary to the HQFRS and the SBFRS can be found in two basic topographic areas: (1) mountains, butts or low hills; and (2) alluvial fans and valley plans. The mountains, butts and low hills have soil types consisting of gravelly loams and rock outcroppings (Ref. 26).

The soil survey of Maricopa County categorizes the alluvial fans and valley plains into two basic associations: (1) soils formed in recent alluvium; and (2) soils formed in old alluvium. The most recent alluvial fans and valley plains (recent alluvium) consist of loams, sandy loams and clay loams. The older alluvial fans and valley plans consist of loams, clay loams and gravelly loams.

Vegetation. The variety of vegetation throughout the subwatersheds remain relatively consistent. Typical vegetation in the areas where the rock outcroppings predominate are creosote bush, bursage, cactus and scattered mesquite and Paloverde trees. In the areas where recent alluvial fans and valley plains exist, the predominating vegetation is creosote bush, cactus, annual weeds, grasses, scattered mesquite and Paloverde trees.

Land Use. The land use within the areas tributary to the HQFRD and SBFRS have remained relatively unchanged over the last 20 years. Range lands and mining claims are the two predominanting land uses. The range lands are located chiefly in the alluvial fans and valley plains. The mining claims are situated in the rock outcrop mountain zones.

Watershed Modeling

The following section of this report describes the hydrologic modeling of the watersheds tributary to the HQFRS and the SBFRS.

The watersheds contributing to the HQFRS or the SBFRS were modeled separately, but the same methods and procedures were used. The existing models (Ref. 3 and Ref. 8) were revised to create consistent hydrologic characteristics. Figure 5 and 6 are the model diagrams for the HQFRS and the SBFRS.

Computer Model. In conformance with the contract scope of work, the Corps of Engineers' HEC-1 computer program, utilizing the SCS unit hydrograph option was used to model the HQFRS and SBFRS watersheds. The IBM 512k version (revised February 1985) of HEC-1 was utilized and run on a 286 AT class computer.

Model Inputs. The SCS option in the HEC-1 computer model requires the following input data to estimate runoff:

A. Tributary Area
B. Curve number
C. Lag time
D. Precipitation amount and temporal distribution
E. Channel and Level Pool Routing

Where a specific runoff needed to be routed in a channel through a subwatershed, the Muskingum method was utilized. Where runoff was to be routed through a reservoir, the level pool or Modified PULS method was used.

Triburary Area. The area of subwatersheds tributary to the HQFRS or SBFRS is that area that contributes direct runoff to a point of concentration. The areas for the HQFRS were obtained from Ref. 3. Table 2 lists the watershed designations and areas in square miles for the HQFRS.

The subwatershed areas tributary to the SBFRS were obtained from Ref. 8. Table 3 lists the watershed designations and areas in square miles. Watershed boundaries were modified as discussed in the Watershed Description section of this report. Table 4 lists the changes from the PRC report.

Curve Number. The SCS curve number (CN) is based on the vegetative cover type and density as well as hydrologic soil group. The CNs that were reported in Ref. 3 and Ref. 8 were reviewed and found to be reasonable. These CNs were used in this study. However, based on the SBFRS subwatershed tributary area modifications, the CNs were correspondingly modified (Table 5). The CN modification calculations are given in Appendix C.

Lag Time. The lag time in the SCS option is described as the lag in hours between the center of mass of rainfall excess and the peak of the unit hydrograph. The lag time is equal to the time of concentration (Tc) times 0.6.

The Tc is defined as the runoff travel time from the hydrologically most distant point of the watershed to the point of concentration at the watershed outlet. The Tc was computed for each subwatershed. The open channel flow method presented in the Ref. 28 was utilized to calculate the Tc.

The open channel flow method was developed for specific washes in this study. The washes are identified as blue-dashed lines on the USGS Topographic Maps. This method was based on the Manning equation for open channel flow, assuming bank-full conditions.

Channels for each subwatershed were divided into reach lengths with uniform slopes. Where possible, each reach length was set between a change in elevation of 200 feet. The Manning equation was used to solve for the velocity of a given reach. An incremental Tc for a given reach was then calculated by dividing the velocity times 3,600 into the reach length. The Tc for a subwatershed was then calculated by adding the incremental Tc's. The Tc value for each subwatershed was converted to the SCS lag time. The Tc and lag time calculations are given in Appendix D.

Channel geometry was not readily available on the USGS Map. Channel geometry for the various regions such as mountains and alluvial fans was assumed based on field reconnaissance of washes in the middle and lower regions of the watersheds.

The methods mentioned above were utilized on SBFRS (Ref. 8). The SCS Report on the HQFRS also appears to have used a similar procedure for estimating Tc and lag time.

Probable Maximum Precipitation. There were two probable maximum precipitation (PMP) events estimated in this study. These were the 6- and 72-hour storm events. A "general storm" analysis was performed for the 72-hour PMP. The general storm was centered over both the HQFRS and the SBFRS. A "local storm" analysis was performed for the 6-hour PMP. Two local storms were analyzed; one centered over the HQFRS and one centered over the SBFRS.

Using the Hydrometeorological Report No. 49 (HMR49, Ref. 24) procedures, the general PMP for the month of August was selected as the 72-hour flood event. The HMR49 general storm PMP computation sheet was used to calculate the total PMP. Appendix E contains the data and calculation sheets. By using the above method, the general 72-hour PMP was estimated to be 15.4 inches. The temporal distribution of the 72-hour PMP is illustrated on Figure 7.

The 72-hour precipitation was distributed hourly by first plotting a depth-duration curve from the 6-, 18-, 36-, 48- and 72-hour cumulative precipitation values calculated from Ref. 24 procedures and the 1-, 2-, 3-, 4-, and 5-hour cumulative precipitation from the procedures in Ref. 25. Cumulative precipitation was then

interpolated from the depth-duration curve for each 3-hour increment. The 3-hour precipitation distribution was subsequently obtained using the U.S. Bureau of Reclamation procedure described in the Flood Hydrology Manual (Ref. 23). In this procedure, the maximum hourly precipitation is placed at two-thirds the storm duration (i.e., the 48th hour for the 72-hour storm). The one hour precipitation distribution was then obtained by interpolation.

The HMR49's procedure for local storm PMP was also used. The HQFRS and SBFRS 6-hour PMP was estimated to be 9.5 inches and 10.1 inches, respectively. The 6-hour PMP was distributed according to the procedures outlined in Ref. 24, Table 4.7. The hourly precipitation was further distributed evenly into 15 minute increments with the exception of the maximum hourly, which was distributed according to Table 4.8 in Ref. 24. The HQFRS and SBFRS 6-hour PMP distributions are illustrated on Figure 8 and Figure 9, respectively.

The calculations for the general and local PMPs are given in Appendix E.

Channel and Level Pool Routing. Two routing methods were used to compute dowmstream hydrographs. The Muskingum method was used for channel routing where storage routing was not a factor. The level pool reservoir routing was used to route storm runoff through structures such as the GRAD.

The Muskingum method within the HEC-1 was used to route the storm waters from flow split through the dowmstream watershed or to route the discharge hydrographs from the GRAD structures through dowmstream subwatersheds.

Two parameters are used in the Muskingum method. The first is the coefficient "X", a function of storage. The range of this coefficient are 0.0 for a reservoir condition and 0.5 for minimum flow attenuation. For purposes of this study, a value of 0.3 was selected since the channels appear to be narrow with moderate overbank flow and overbank storage will be insignificant.

The second input is the coefficient "K". This coefficient is the travel time of the floodwave peak through the entire reach. To estimate this coefficient, the open channel flow method used to calculate the Tc was utilized (see Lag Time section in this report). This method estimates channel velocity (V). Channel velocity is then converted into the celerity of elementary discharge wave (VM=1.67*V). The travel time along the measured flow paths can then be calculated by dividing the reach length by VM*3600 seconds/hour. Estimated values for "X" and "K" for each reach are provided in Appendix F.

Routing of the 6-hour and 72-hour PMF hydrographs was performed at the GRAD using the level pool reservoir routing method. The input data to the HEC-1 for this type of routing was obtained from Ref.

8. As previously discussed, the GRAD has five 72-inch overchutes which discharge across the Granite Reef Aqueduct in Reach 6. Of these five overchutes, only one is a tributary to the SBFRS. The Ref. 8 data indicated that the crest of the GRAD is 1,398.0. Flow over the GRAD was estimated for pool water surface elevations from 1,398.0 to 1.399.0. The Ref. 8 report indicated that the GRAD has a second crest elevation of 1,399.0. It became apparent in the process of using the SBFRS model that the 6-hour PMF inflow hydrograph will overflow the stretch of GRAD that will drain into associated SBFRS subwatersheds. Therefore, additional weir calculations were prepared for the GRAD for water surface elevations from 1,399.0 to 1,400.0. These additional elevation versus discharge data were added to the calculations in Ref. 8. Appendix G contains the weir calculations and Table 6 contains the results of the five 72-inch overchutes and weir flow.

Probable Maximum Flood

In this section of the report, the results of the hydrologic analysis conducted on the watersheds tributary to the HQFRS and the SBFRS are described.

In this study, the estimated probable maximum flood (PMF) and volumes for the 6-hour and 72-hour PMF and half PMF are presented. The inflow half PMF hydrographs from the HQFRS and the SBFRS watersheds were estimated by utilizing the HEC-1 computer program and the input data presented in the preceeding section. The half

PMF was estimated by using the diversion inflow table (DI record) in the HEC-1 computer program. Appendix A and B contain the input and output data for the HQFRS and the SBFRS PMF and half PMF, respectively. Table 7 lists the PMF and half PMF for the HQFRS and the SBFRS. Figures 10 through 13 illustrate the associated half PMF hydrographs.

The half PMF peak discharges for the 6-hour PMF were greater than the 72-hour PMF events for both FRSs (Table 7). This occurence was expected since the 6-hour PMP is approximately two-thirds of the 72-hour PMP. The runoff volumes entering the FRSs as a result of the 6-hour and 72-hour PMP are presented in Table 8.

As expected, the 72-hour PMP produces more runoff volume at the flood retarding structures than the 6-hour PMP. This occurrence was anticipated because the 72-hour PMP is larger than the 6-hour PMP.

Computer diskettes containing both input and output files for all runs accompany the submittal of this report. Hard copies of the 6- and 72-hour output files can be found in Appendix A and B.

Summary

Hydrologic models using the HEC-1 computer program were prepared for the drainage areas above the HQFRS and SBFRS. Within the model, the SCS unit hydrograph and curve number criteria were

utilized. The Muskingum routing method was used for all channel routing and the level pool routing was used to route flows through the Granite Reef Aqueduct Dike. These models were used to develop half of the 6-hour and 72-hour PMF.

Peak discharges for the flooding events are presented in Table 7. The 6-hour PMP storm generates the greatest peak discharge. However, as expected, the 72-hour PMP storm produces the greatest volume. Peak volumes for the flooding events are presented in Table 8.

BREACH ANALYSIS

Introduction

The purpose of this section is to present the HQFRS and SBFRS failure hydrograph information. The information discussed in this section includes:

1. Routing of the 6-hour and 72-hour 1/2 probable maximum flood (PMF) hydrograph through each FRS to estimate the potential for overtopping.
2. Estimating breach parameters such as breach size, shape and formation time. This was based on the type of failure mechanism selected from Step 1 above,
3. Using the parameters from the breach model for the hydrograph which produced the highest peak discharge, run the Dambreak-88 model for each failure scenario to estimate failure hydrographs for 3 breach locations on the HQFRS and 2 breach locations on the SBFRS.

Routing of 1/2 PMF Hydrographs

The 1/2 PMF hydrographs resulting from the 6-hour and 72-hour probable maximum precipitation (PMP) events were estimated and presented in the Hydrology section of this report. The respective hydrographs were routed through the HQFRS and SBFRS to estimate the potential for overtopping. The HEC-1 computer model was used to

prepare this analysis. The detailed input to the model for each FRS is presented in Volume II, Appendix A and B of this report.

The results of the flood routing indicated that neither the HQFRS or the SBFRS overtopped. The maximum stage obtained in the routing analysis is given on Table 9. We concluded from this analysis that the failure mechanism for each structure would be piping.

Failure Parameters

The failure parameters that were input into DAMBRK-88 were approximated using the microversion of the National Weather Service Breach Model dated January 1987. The failure parameters approximated using this model were final breach dimensions and time for breach formation. A copy of the input and output variable names is provided in Appendix H.

The basic dam shape data was presented in the Hydrology section of this report. The properties of the materials comprising the structures were selected based on data provided in references 2, 3, 4, 5, 6, 7, 8 and 9, professional judgment. A detailed summary of the values selected for each of the required input variables are given in Appendix I and J.

Our analysis consisted of running the breach model for the 1/2 PMF hydrographs resulting from the 6-hour and 72-hour PMP for each

structure. The breach analysis was conducted at three locations on the HQFRS and two locations on the SBFRS (Figure 14).

The analysis at each breach location consisted of failing the structure via piping for various initial water surface elevations. The elevation at which the piping failure started were also varied. These variations were conducted for both the 6-hour and 72-hour events. The indicator used in selecting the inflow hydrograph, the breach size and the time for the breach to form was the peak breach discharge.

The peak breach discharge from the HQFRS was obtained for the 1/2 PMF resulting from a 72-hour PMP (Table 10). The elevation at which piping was started for the HQFRS breaches was selected to occur at about 1/2 the distance between the crest and the bottom of the dam. The initial reservoir water surface elevation at the time piping started was selected to correspond to the maximum reservoir stage during HEC-1 routing of the 72-hour 1/2 PMF event (Appendix A). The peak discharge from the SBFRS was also obtained for the 1/2 PMF resulting from a 72-hour PMP. The piping elevation was selected to occur near the bottom of the SBFRS and the initial water surface was selected to be at the maximum stage estimated during the HEC-1 routing of the 72-hour, 1/2 PMF event (Appendix B). A detailed summary of the output for each of the breach locations is given in Appendix I and J.

DAMBREAK ANALYSIS

The routing of hydrographs resulting from the piping failure of the Harquahala FRS and the Saddleback FRS was conducted using the Boss Corporation version of DAMBRK. This micro-computer program is based around a highly optimized version of two original programs written in 1984 and 1988 by Professor D.L. Fread, Senior Hydrologist with the Hydrologic Research Laboratory, National Weather Service, NOAA, Silver Spring, Maryland 20910. It is our understanding that the Flood Control District of Maricopa County has this model and that it is acceptable in lieu of the National Weather Service version.

The DAMBRK model was run for piping failures at each of the five assumed failure locations for the 1/2 PMF resulting from the 72-hour PMP. This model was also used to estimate the peak discharge, peak stage and floodwave travel time at specific cross-sections downstream of each structure. The following section describes the inputs, methods and assumptions that went into this analysis and the results obtained.

DAMBRK Model Inputs

The primary inputs to this model include:

1. Type of failure;
2. In-flow hydrograph;
3. Reservoir elevation, discharge, storage;

4. Breach formation time;
5. Breach final geometry;
6. Downstream cross-sections;
7. Manning Roughness coefficients; and
8. Computation Option No. 13.

The type of failure, inflow hydrograph, reservoir characteristics, breach formation time, and final breach geometry were discussed in detail in previous sections. The input data used in DAMBRK for each of the breach locations is given in Appendix K and L. A summary of DAMBRK peak breach discharge for each of the five breach locations is given in Table 11.

The cross-sections used in the routing analysis were developed from three sources. Cross-sections in the areas adjacent to the Harquahala FRS were prepared using topography developed as part of this study and U.S.G.S. 15-minute topographic maps. Cross-sections used in the areas adjacent to the Saddleback FRS were prepared using U.S.G.S. 15-minute series quadrangle topographic maps (contour interval 40 feet) and 1 inch equals 400 feet (contour interval 4 feet) flood insurance study work maps by Cella Barr Associates and dated February 1989 provided by the District. The remaining study area below both of the FRSs was cross-sectioned using flood insurance study work maps prepared by Cella Barr Associates dated June 1988.

Each of the flood hydrographs routed from the selected breach locations had the potential to be impacted by secondary structures. The routed hydrography from the east Harquahala breach crosses the CAP Canal and I-10. We have assumed in concurrence with District staff that the impact of these barriers would not significantly impact flood routing.

The routing of the middle Harquahala breach had the potential to be impacted by the CAP Canal and I-10. With the concurrence of District staff, we make the conservative assumption that both the CAP and I-10 would have no impact on the flood hydrograph routing.

The west breach flood hydrograph routing had the potential to be impacted by the CAP Canal, I-10 and the Centennial Levee. The levee would, under west breach flood hydrograph routing, be overtopped and fail. The levee would probably fail prior to the arrival of the peak of the west breach flood hydrograph. Until the levee failed, flow would be diverted to the west into the Centennial Wash. The routing problem under conditions of split flow such as might happen are extremely complex and beyond the capabilities of the DAMBRK model. For purposes of this study, we have assumed that the levee would fail early in the flood hydrograph and as such, would not have a significant impact on flow rates stages and travel time downstream.

The Saddlesback FRS failure hydrograph routings did not consider any significant secondary structures. The flow paths were generally unobstructed over their entire length.

Manning Roughness Coefficients

The Manning roughness coefficient ("n") was selected based on field inspection and aerial photographs of the flow paths for flood hydrograph routings. The "n" values were adjusted up or down in some instances to improve model stability and to produce model convergence. The Manning roughness coefficients selected range from approximately 0.04 to 0.05 (Table 12).

Floodplain and Off-Channel Storage

The flow paths assumed for each of the routed hydrographs were not naturally well-defined alluvial channels with sufficient conveyance to carry the peak flow. The flow paths adjacent to the dams (between the dam and I-10 or irrigated fields) were relatively flat in the lateral direction with very small channels longitudinally (down slope). The flow paths intersecting the valley section (between I-10 to Base Line Road) have sides that slope up to the east and west. This lateral slope up to the east and west gradually provides a very broad shallow flow path. The flow path(s) in the southern portion of this valley, south of Base Line Road, are somewhat more defined. This flow path(s) has shallow channel sides that slope up and become part irrigated fields or

break off into a parallel alluvial channel system. The channel banks are not sufficient to convey the peak flow of a breached FRS.

Due to the condition just described, there was no floodplain portion of the flow path typical of many riverine systems. The relatively flat lateral slope of the valley resulted in modeling the area between the flood retarding structures and Baseline Road as a large continous channel. Storage modeling was implemented south of Baseline Road. Storage of flood waters may occur as a result of overflow into other parallel channels (cross-section location 26.69 - 29.62). The detailed input data on the areas where this storage occurred is given in Appendix K and L.

Output from DAMBRK Model

The output from the five dambreak routing analyses are summarized on Tables 13 through 17. The detailed Dambreak computer runs for each of these five routing analyses are given in Appendix K and L.

EVACUATION PLAN

This section presents the results of the Dambreak on downstream areas. These results include limits of inundation, floodwave travel and social and economic impacts. Additionally, a brief discussion of potential evacuation routes is presented.

Dambreak Flooding Characteristics

The following describes the Dambreak flood characteristics on downstream areas. These characteristics include:

1. Limits of inundation along flow path of failure hydrographs through downstream locations.
2. Time from peak discharge from the dam failure to maximum stage elevation at downstream locations.
3. Maximum stage elevation at downstream locations.
4. Time to flood elevation.

Limits of Inundation

The limits of inundation for each of the five routings are plotted on Figures 15 through 19. The limits of inundation were plotted based on the stages estimated at specific cross-sections along the routed failure hydrograph flow path. A separate figure showing limits of inundation was developed for each of the five routed hydrographs. Stages between cross-section longitudinally were

interpolated and adjusted as appropriate. The limits of inundation represent the surface coverage based on water surface elevation at specific cross-sections.

The longitudinal downstream limits of the study areas were selected in concurrence with District staff to be at approximately Mullens Cut. This location is approximately 24.1 miles from the west Harquahala breach location and 15.9 miles from the south Saddleback breach location.

Travel Time

The travel time or time to maximum stage (basis for limits of inundation) was estimated as the time difference between the maximum stage at the most upstream cross-section to the maximum stage at each downstream cross-section. This travel time is given on Figures 15 through 19 at the specific cross-section. The time from the start of rainfall and from the beginning of dam failure to the maximum stage at each cross-section is given in Tables 13 through 17.

Time to Flood Elevation

The time to flood elevation is also presented on Figures 15 through 19. Time to flood elevation were estimated at locations where the potential for residential/commercial floods and evacuation may be necessary (Tables 13 through 17). The time to flood elevation

presented is the time from the beginning of dam failure to the time it would take the flood water to reach a specified depth at selected downstream locations. For the HQFRS DAMBRK analysis, this depth was selected to be 2 to 3 feet. The depth was selected based upon the Flood Control District's request of 2 feet, channel geometry and model stability.

The SBFRS DAMBRK produces relatively small flows and thus, the maximum stage at downstream locations does not reach 2 feet in all appropriate cross-sections. To generate meaningful flood elevation times, the flood elevations were selected such that flow depths ranged from as low as 0.2 feet at Section 17.75 (SBFRS, North DAMBRK) to 4.2 feet at Section 24.16 (SBFRS, North and South DAMBRK). Generally, the flood depths were maintained at 2± feet.

Social and Economic Impacts

The following section presents a brief overview of the potential social and economic impacts anticipated for each of the five dambreak scenarios. This overview qualitatively addresses potential damage due to the sudden failure of the Harquahala FRS or the Saddleback FRS.

Current Land Use and Development

The study area can be divided into two distinct areas (Figure 20). The area north of I-10 is undeveloped. The area south of I-10 has extensive agricultural activity including the irrigation infrastructure associated with this activity. There is limited residential and commercial development in the area south of I-10.

Impacts North of I-10

The impact of the Harquahala failure scenarios on the area from I-10 north would potentially be limited to damages to I-10 and irrigation infrastructures. The east failure has the potential to significantly impact the CAP Canal, the Harquahala Floodway and I-10. The CAP, in all likelihood, would sustain significant damage and could be out of service for the length of time needed to make repairs. Because of the importance of the CAP as a water conveyor, this kind of downtime, particularly if it occurs during the critical summer months, would be significant.

The high flow rates anticipated with each of the three Harquahala failure scenarios could potentially cause significant damage to I-10. This damage is likely to require diversion of traffic for up to several weeks while interim repairs are made.

The damage caused by an east failure on the Harquahala Floodway could be significant but it would probably be repaired at the same

time the Harquahala FRS and CAP are being repaired. This would generally mean the floodway would be in operation by the time the FRS went back into service.

A middle failure could significantly impact the CAP, I-10, the Link Canal and the Westside Canal and Dike. The downtime on the Link and West Canal would only have significant impact on agricultural operations locally if failure were to occur during the high demand summer months. The impacts on the CAP canal would be similar to those anticipated as a result of the east failure scenario. The impact on I-10 would be as discussed above.

A failure at the west end of the Harquahala FRS could significantly damage a portion of the Salome Road. The type of damage anticipated on Salome Road should be such that the road could be made temporarily passable very quickly, only causing a short term interruption in traffic. The impacts to the CAP Canal and I-10 would be similar to those anticipated as a result of the east and middle failure. The Centennial Levee could potentially sustain significant damage. This damage could be repaired at the same time other repairs are made to the CAP and I-10.

Impact South I-10

The area south of I-10 within the limits of inundation of the five failure hydrograph flow paths has extensive agricultural

development. The area has limited residential and commercial development.

The impacts to the agricultural and range lands would potentially consist of crop damage and irrigation system damage. The approximate area of agricultural or range land inundated during each of the five failure scenarios is listed in Table 18. The impact would be most significant if failure were to occur during the time crops were maturing.

The residential and commercial structures impacted by each of the five failure scenarios are given on Figure 20 and Table 18.

Evacuation Plan

The following section outlines the potential travel routes for evacuation of impacted residential and commercial structures. The routes suggested lead directly to areas our estimates indicate will be outside the limits of inundation for each of the five scenarios. These evacuation routes are only meant to provide temporary refuge until the flood stage subsides in several hours. Lists of impacted structures (Figure 20) and potential evacuation routes for each are given in Table 19.

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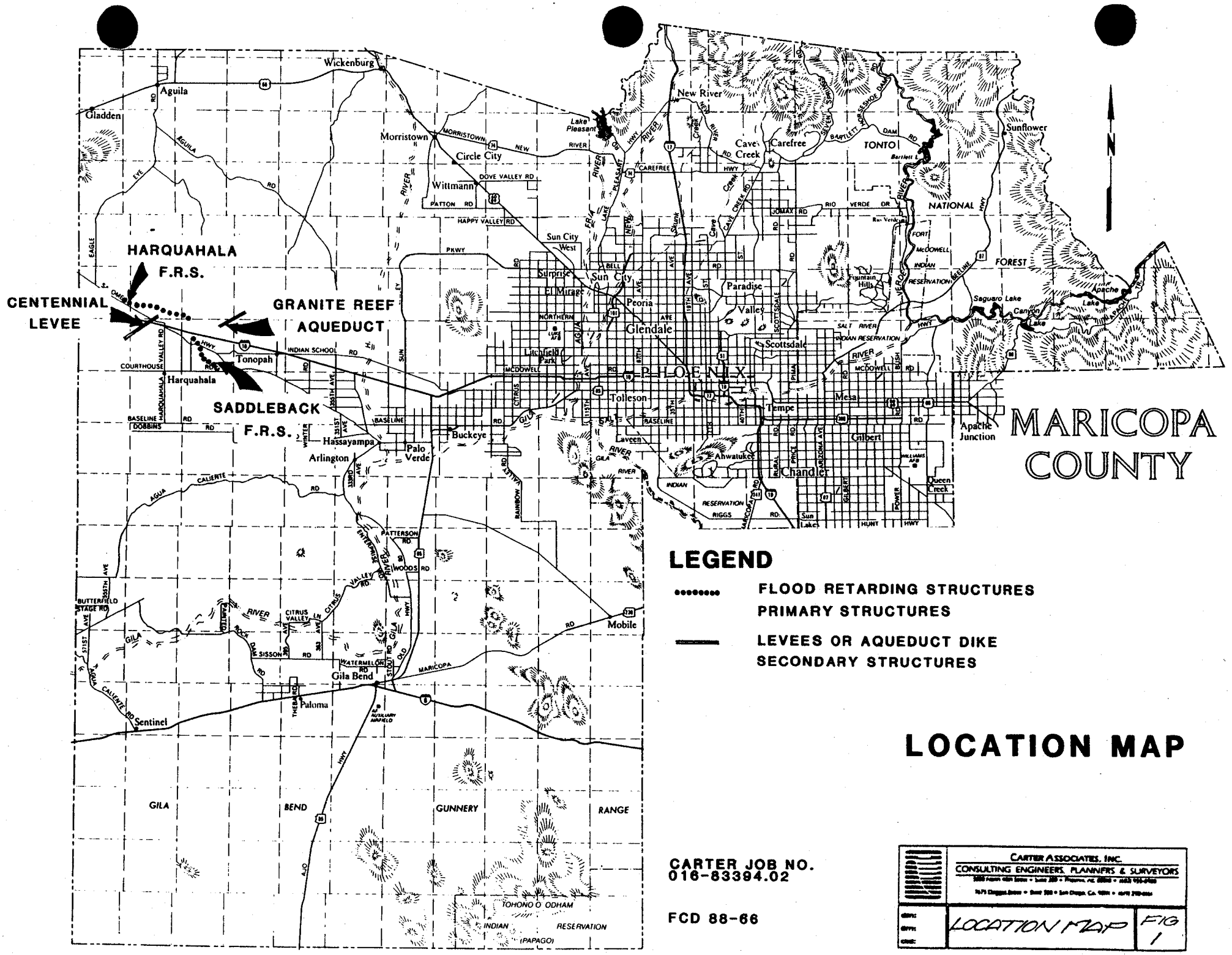

LEGEND

- FLOOD RETARDING STRUCTURES
- PRIMARY STRUCTURES
- LEVEES OR AQUEDUCT DIKE
- SECONDARY STRUCTURES

LOCATION MAP

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016-83394.02

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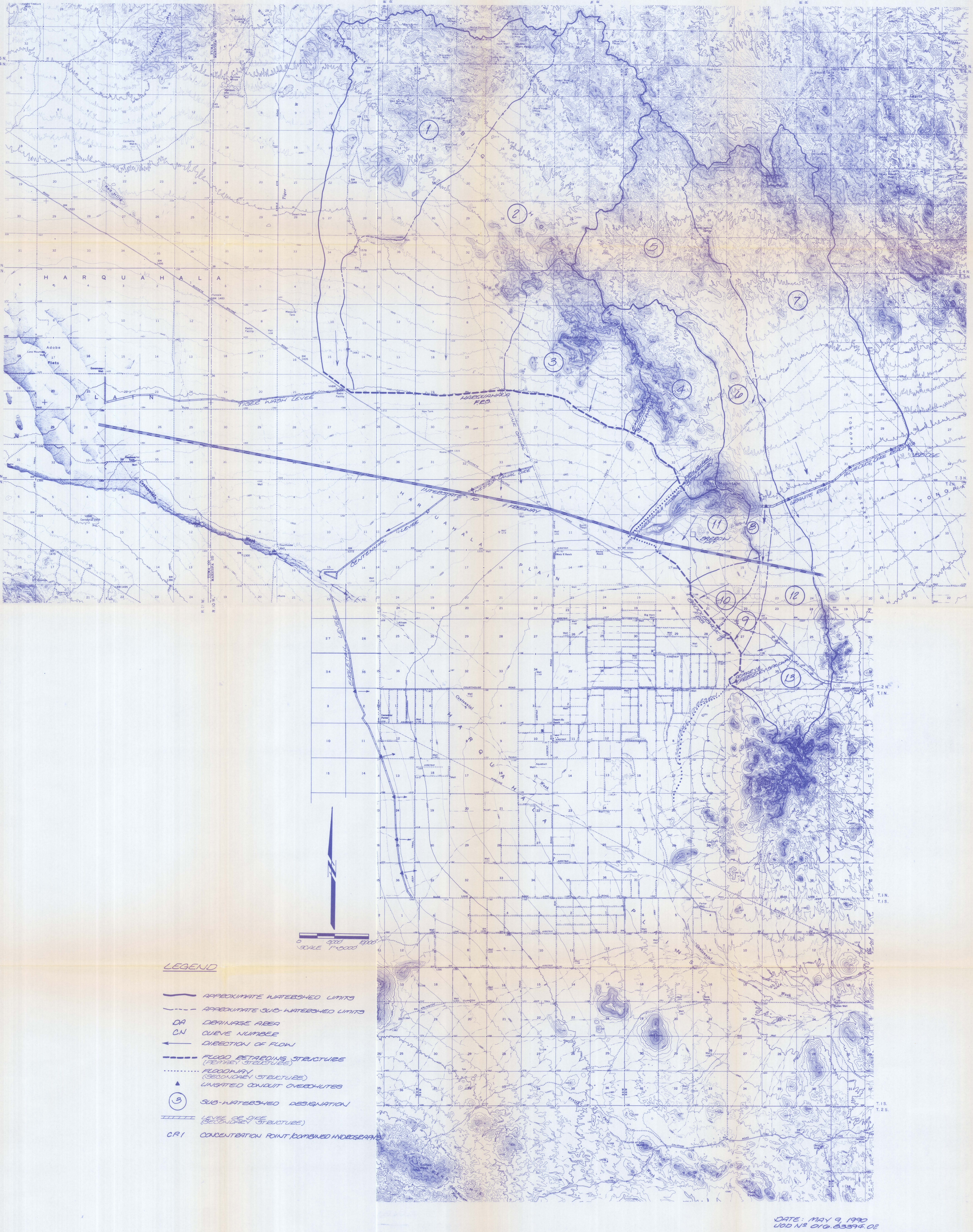
LOCATION MAP | FIG 1

DAMBREAK ANALYSIS
OF
FAR-WEST MARICOPA COUNTY STRUCTURES
HYDROLOGY MAP



DATE: MAY 9, 1990
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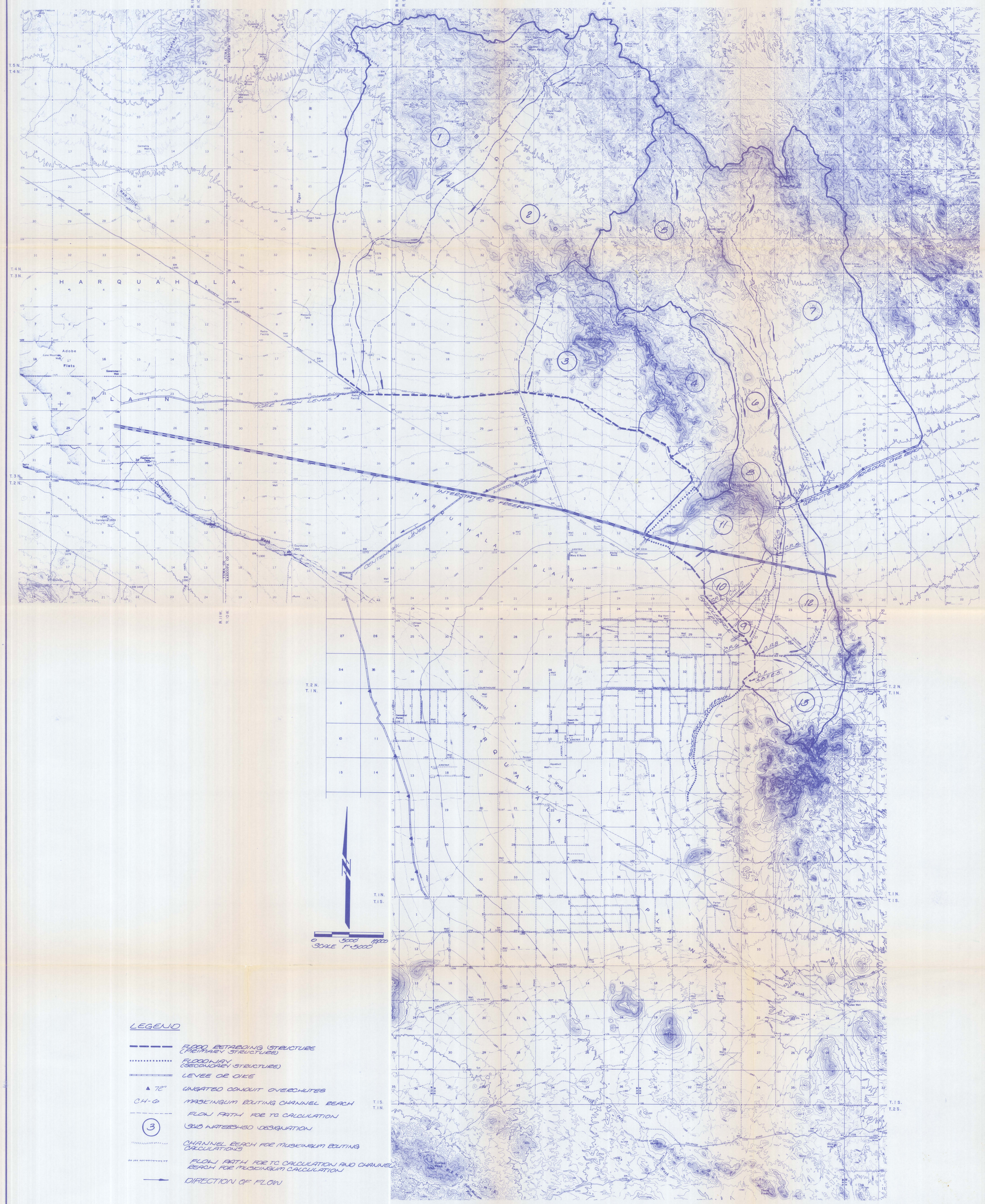
dsn:
drn:
ckd:

PRIMARY & SECONDARY STRUCTURE LOCATION MAP

FIGURE 2

F.C.D.M.C. CONTRACT F.C.D. 88-66

DAMBREAK ANALYSIS OF FAR-WEST MARICOPA COUNTY STRUCTURES
STRUCTURE & REACH LOCATION MAP



LEGEND

- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- FLOODWAY (SECONDARY STRUCTURE)
- LEVEE OR DIKE
- ▲ 72" UNGATED CONDUIT OVERCHUTES
- CH-6 MASKINGUM ROUTING CHANNEL REACH (T.1S. T.1N.)
- FLOW PATH FOR TC CALCULATION
- (3) SUB WATERSHED DESIGNATION
- CHANNEL REACH FOR MUSKINGUM ROUTING CALCULATIONS
- FLOW PATH FOR TC CALCULATION AND CHANNEL REACH FOR MUSKINGUM CALCULATION
- DIRECTION OF FLOW

DATE: JULY 9, 1990
JOB NO. 0160.83394.02

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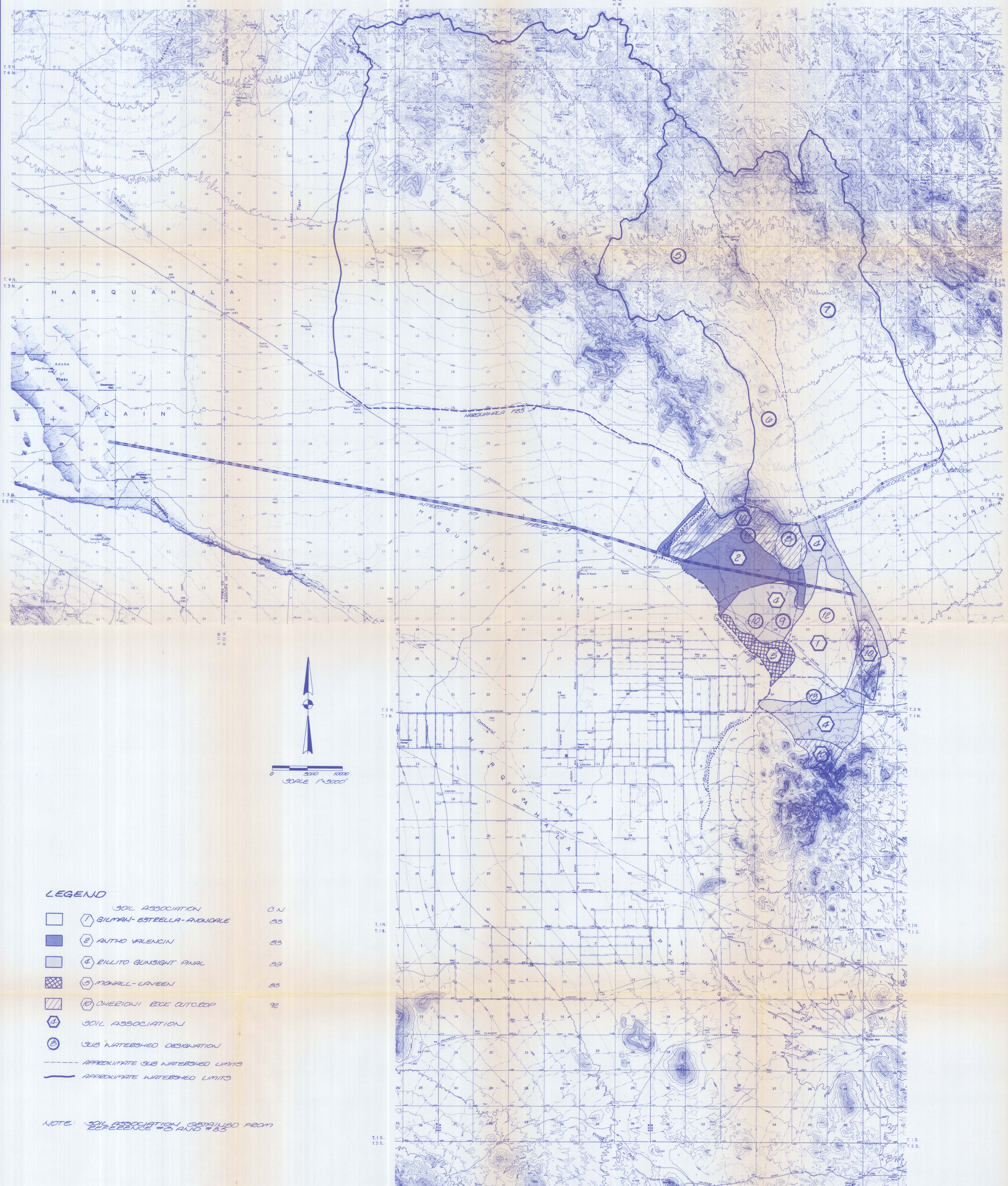
dsn:
drn:
ckd:

HYDROLOGY MAP

FIG. 3

F.C.D.M.C. CONTRACT F.C.D. 88-66

DAMBREAK ANALYSIS
OF
FAR-WEST MARICOPA COUNTY STRUCTURES
S.C.S. SOILS MAP FOR S.B.F.R.S.



LEGEND

	SOIL ASSOCIATION	C.N
	1 GILMAN-ESTRELLA-AVONDALE	83
	2 ANTHO VALENCIN	83
	4 RILLITO GUNSIGHT PINAL	83
	5 MOHALL-LAVEEN	83
	10 CHERIONI ROCK OUTCROP	92

4 SOIL ASSOCIATION

8 SUB WATERSHED DESIGNATION

------ APPROXIMATE SUB WATERSHED LIMITS

—— APPROXIMATE WATERSHED LIMITS

NOTE: SOIL ASSOCIATION OBTAINED FROM REFERENCE #8 AND #85

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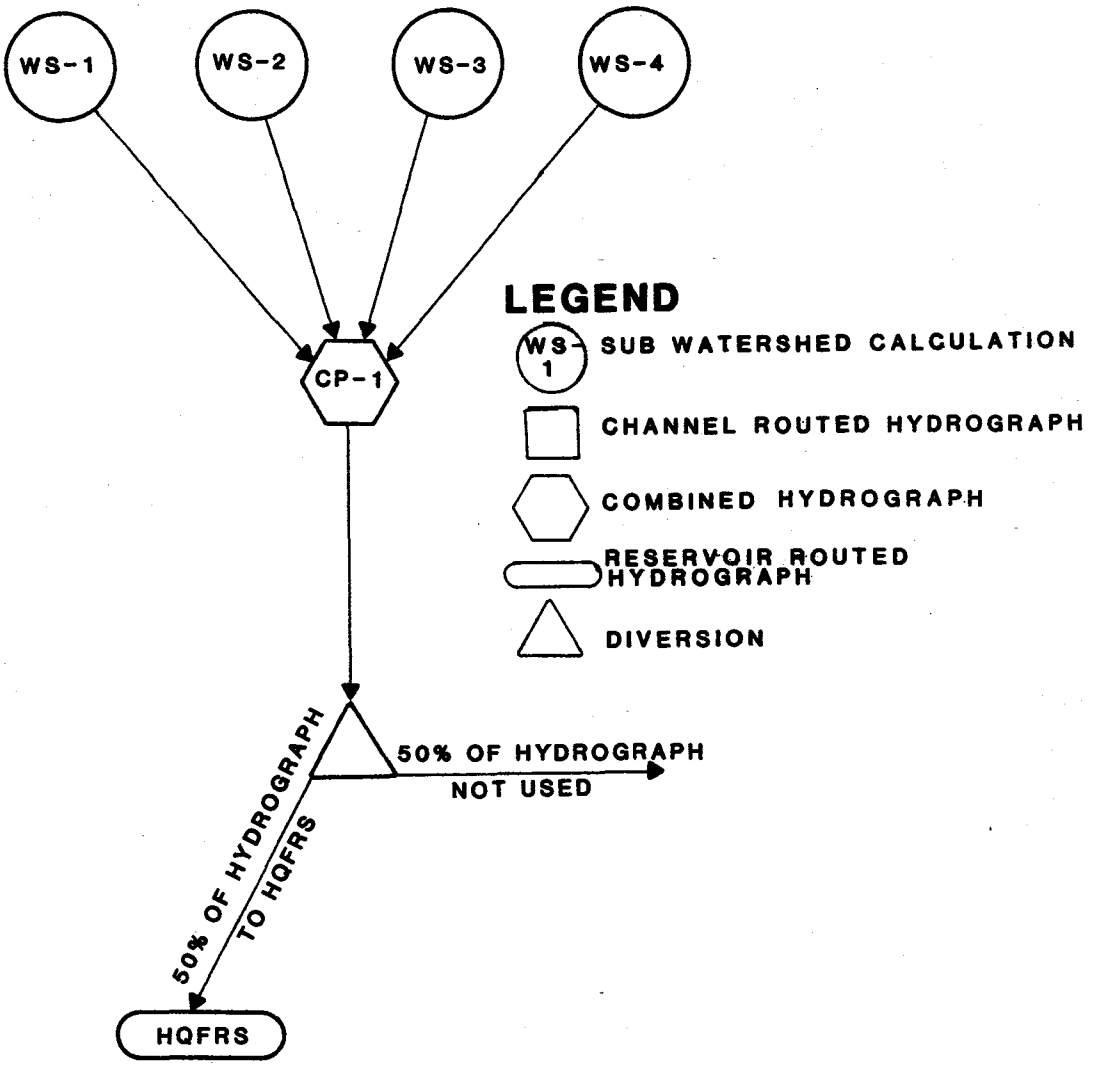
drn:
ckd:

SCS SOILS MAP FOR SBFRS

FIG. 4

F.C.D.M.C. CONTRACT F.C.D. 88-66

HEC-1 ROUTING SCHEMATIC
FOR
HARQUAHALA FLOOD
RETARDING STRUCTURE WATERSHED



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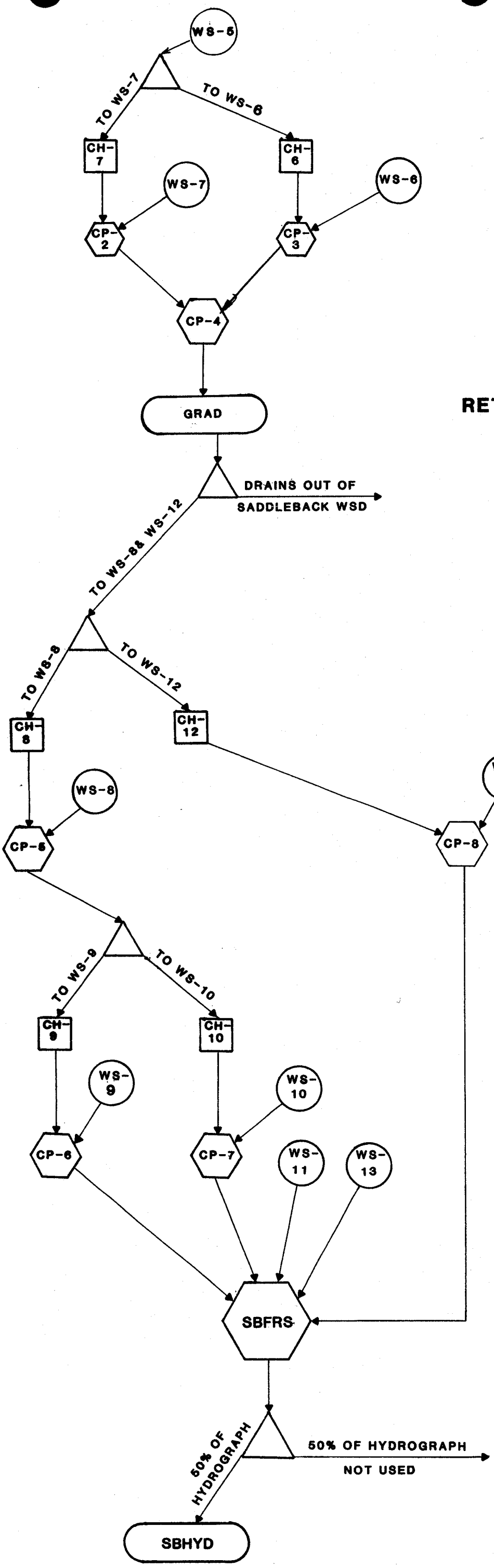
dsn: drn: ckd:	HEC-1 ROUTING SCHEMATIC FOR HARQUAHALA FLOOD RETARDING STRUCTURE	FIGURE 5



HEC-1 ROUTING SCHEMATIC FOR SADDLEBACK FLOOD RETARDING STRUCTURE WATERSHED

LEGEND

- WS-5 — SUB WATERSHED CALCULATION
- CHANNEL ROUTED HYDROGRAPH
- COMBINED HYDROGRAPH
- RESERVOIR ROUTED HYDROGRAPH
- DIVERSION

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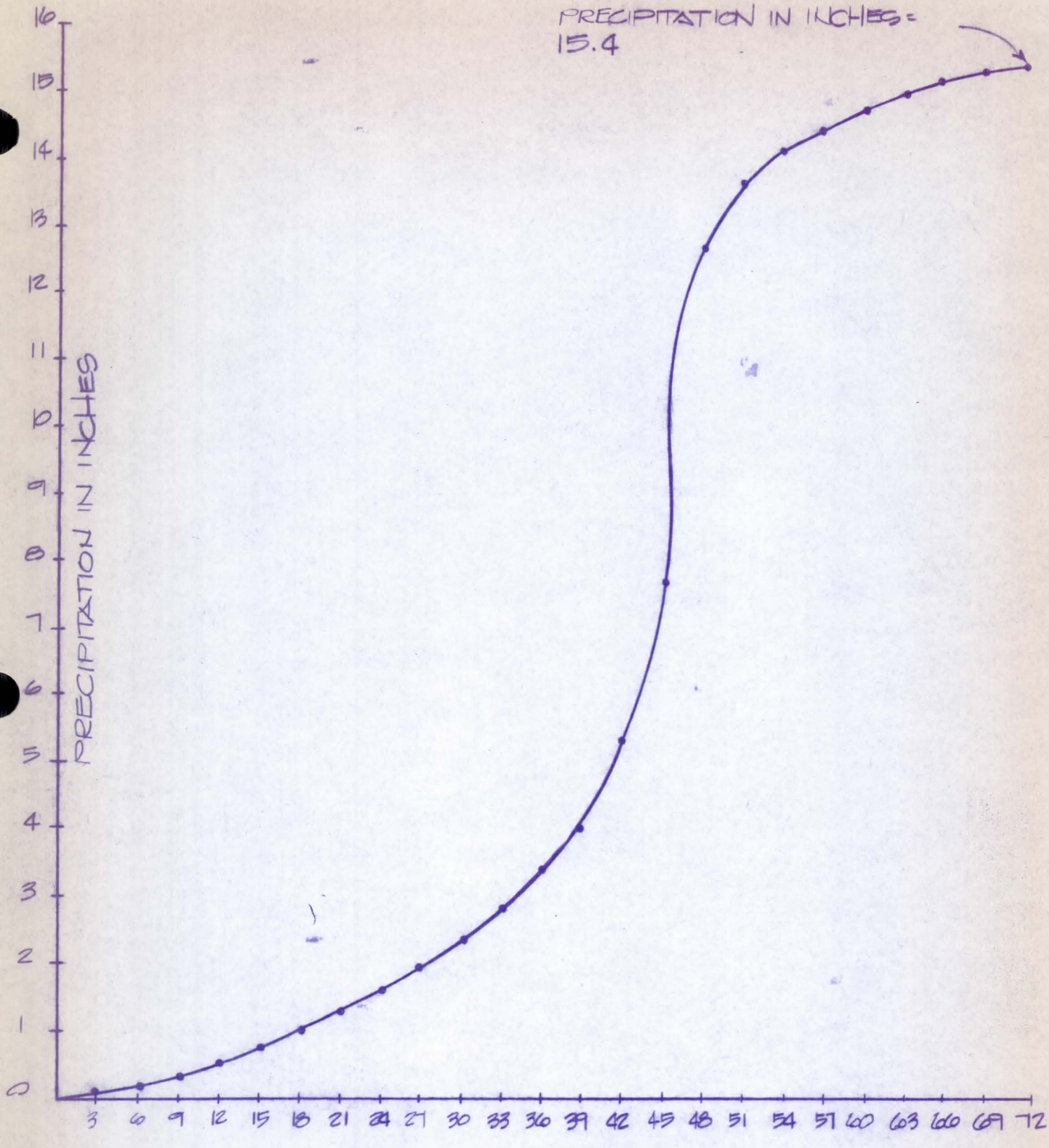
dsn:	HEC-1 ROUTING SCHEMATIC FOR SADDLEBACK FLOOD RETARDING STRUCTURE WATERSHED	FIG. 6
drn:		
ckd:		



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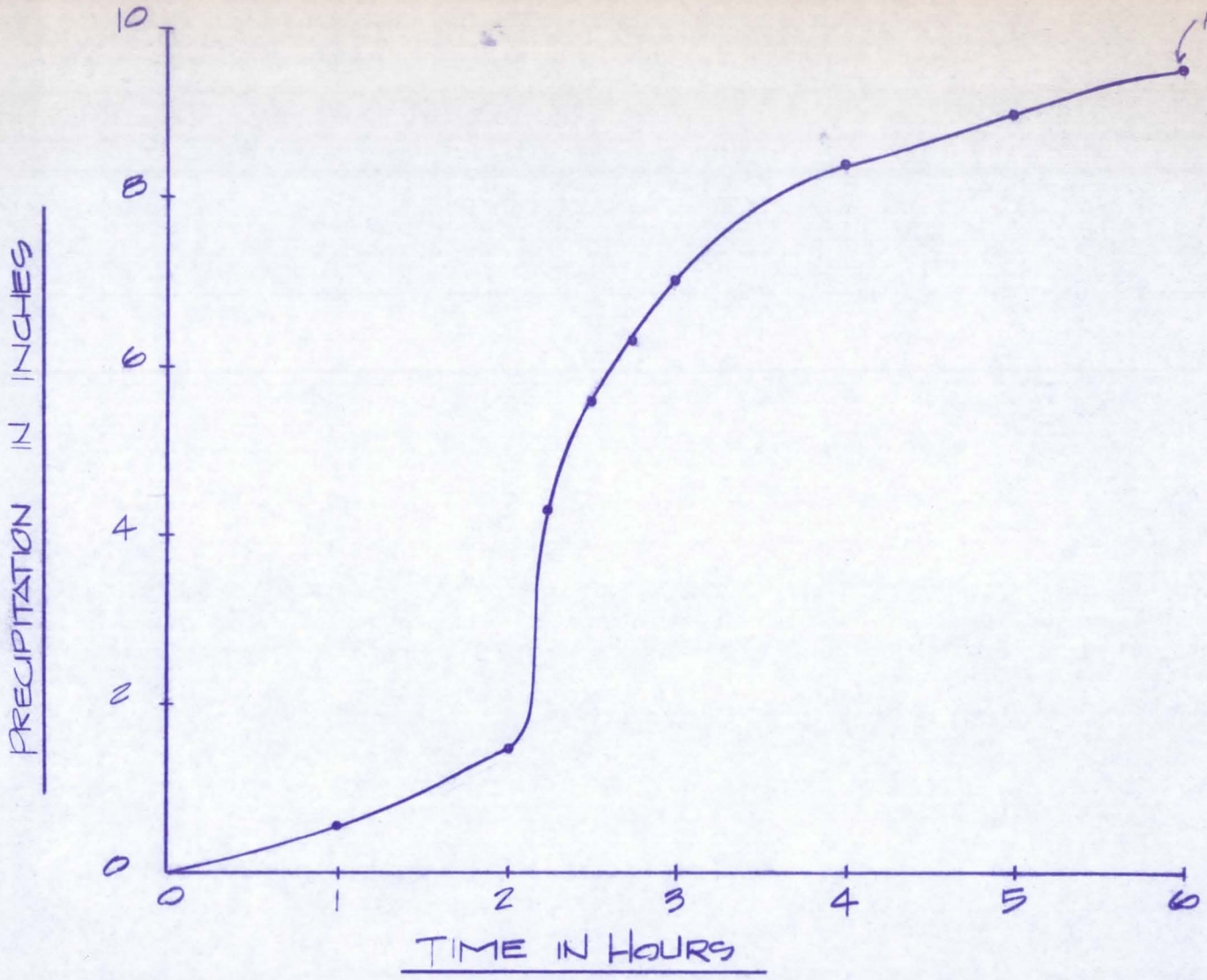
dsn: BM
drn: BH
ckd:

72 HOUR GENERAL STORM PROBABLE MAX. PERCIP. DISTRIBUTION

FIG. 7

FCD 88-660



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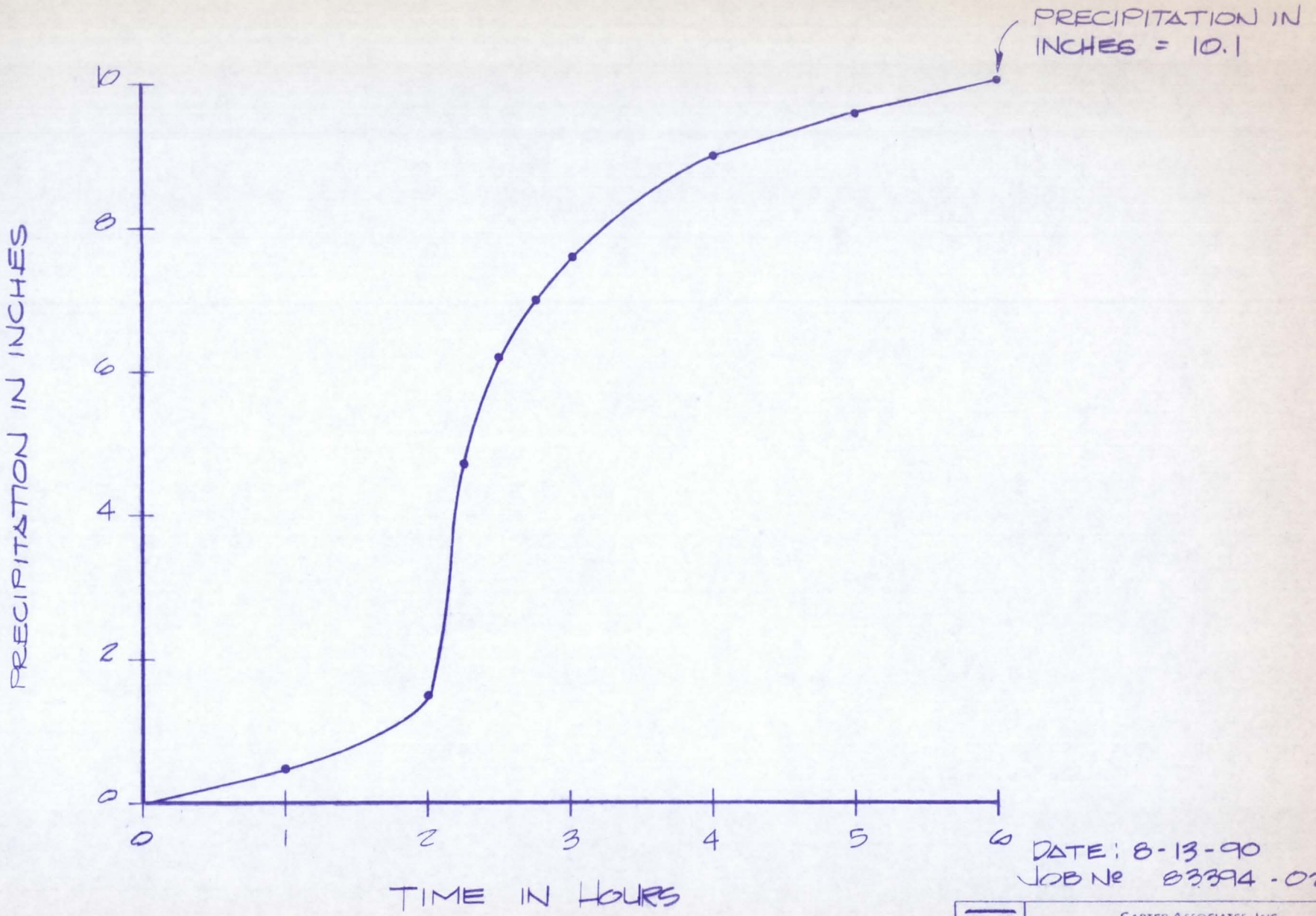
dsn: RM
drn: BH
chk. BHM

HARQUAHALA FRS
6-HR. LOCAL STORM
PROBABLE MAX. PRECIP.
DISTRIBUTION

FIG. 8

FCD 88-606



DATE: 8-13-90
JOB Nº 83394-02

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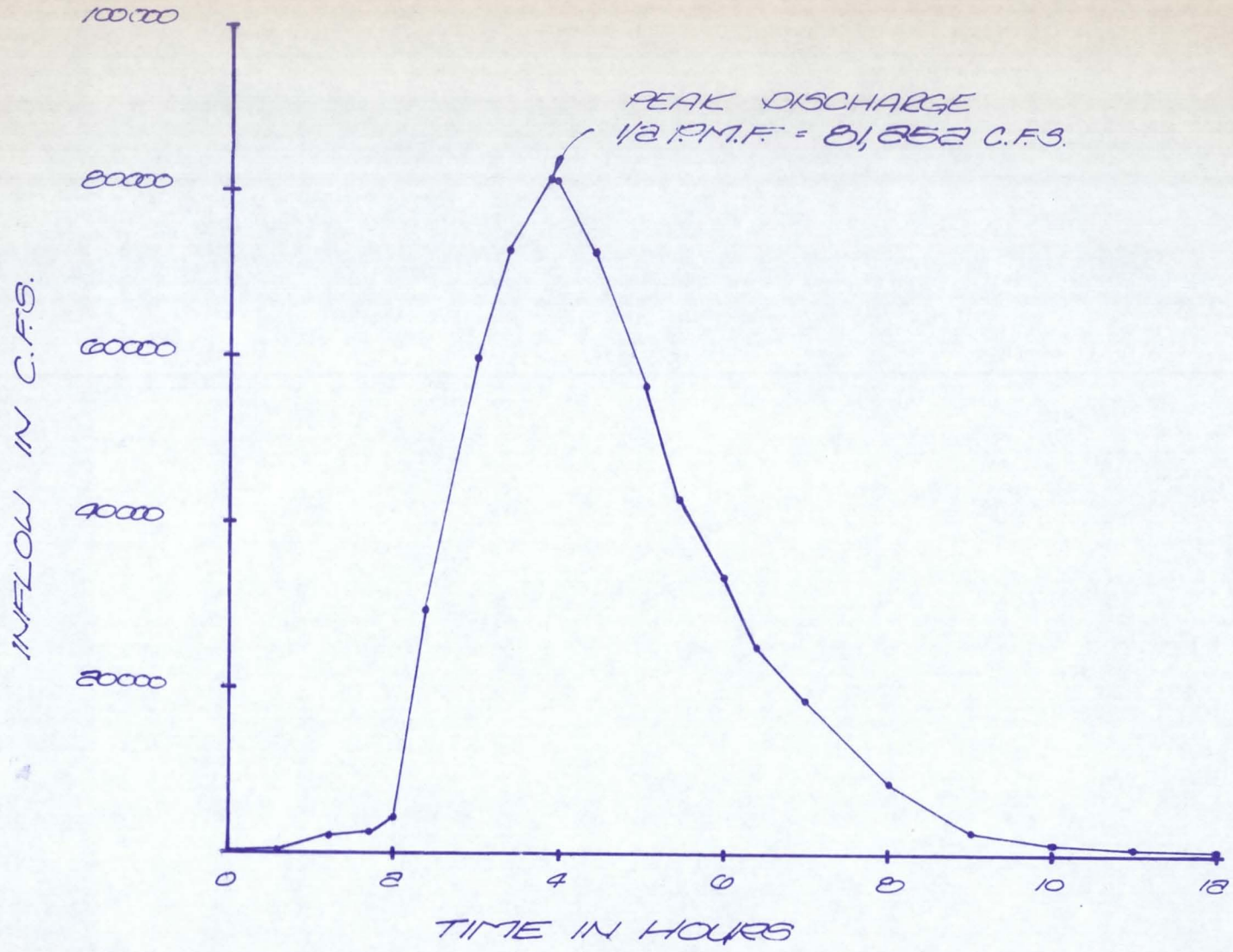
dsn: RM
drn: BH
ckd: BM

SADDLEBACK FRS
6 HR LOCAL STORM
PROBABLE MAX. PRECIP
DISTRIBUTION

FIG. 9



DATE: 8-13-90
JOB Nº 016-83394-02

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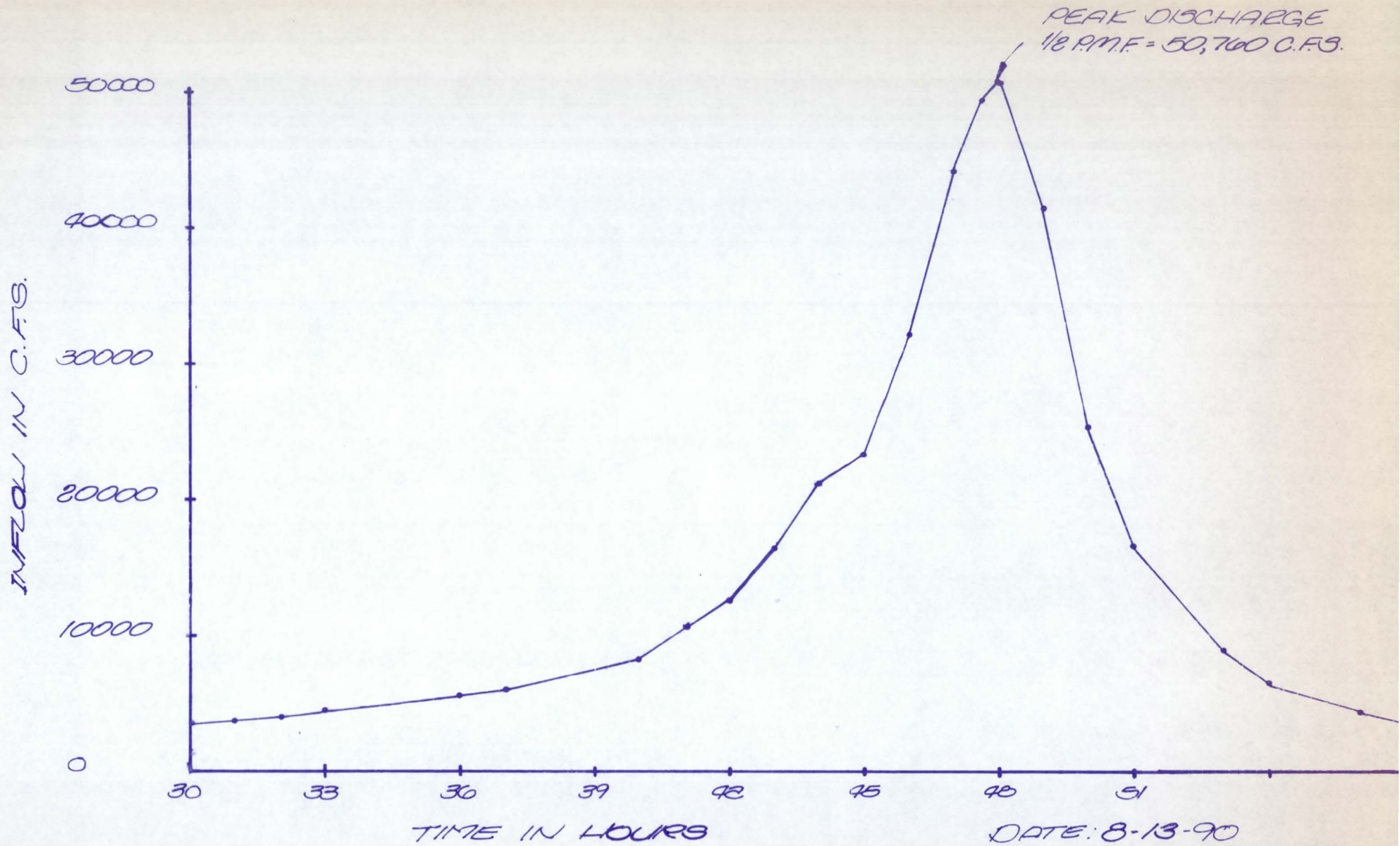
dsn. B.M
drn. B.H
ckd. B.M

HARQUAHALA FRS
6 HOUR 1/2 P.M.F.
INFLOW HYDROGRAPH

FIG. 10



DATE: 8-13-90

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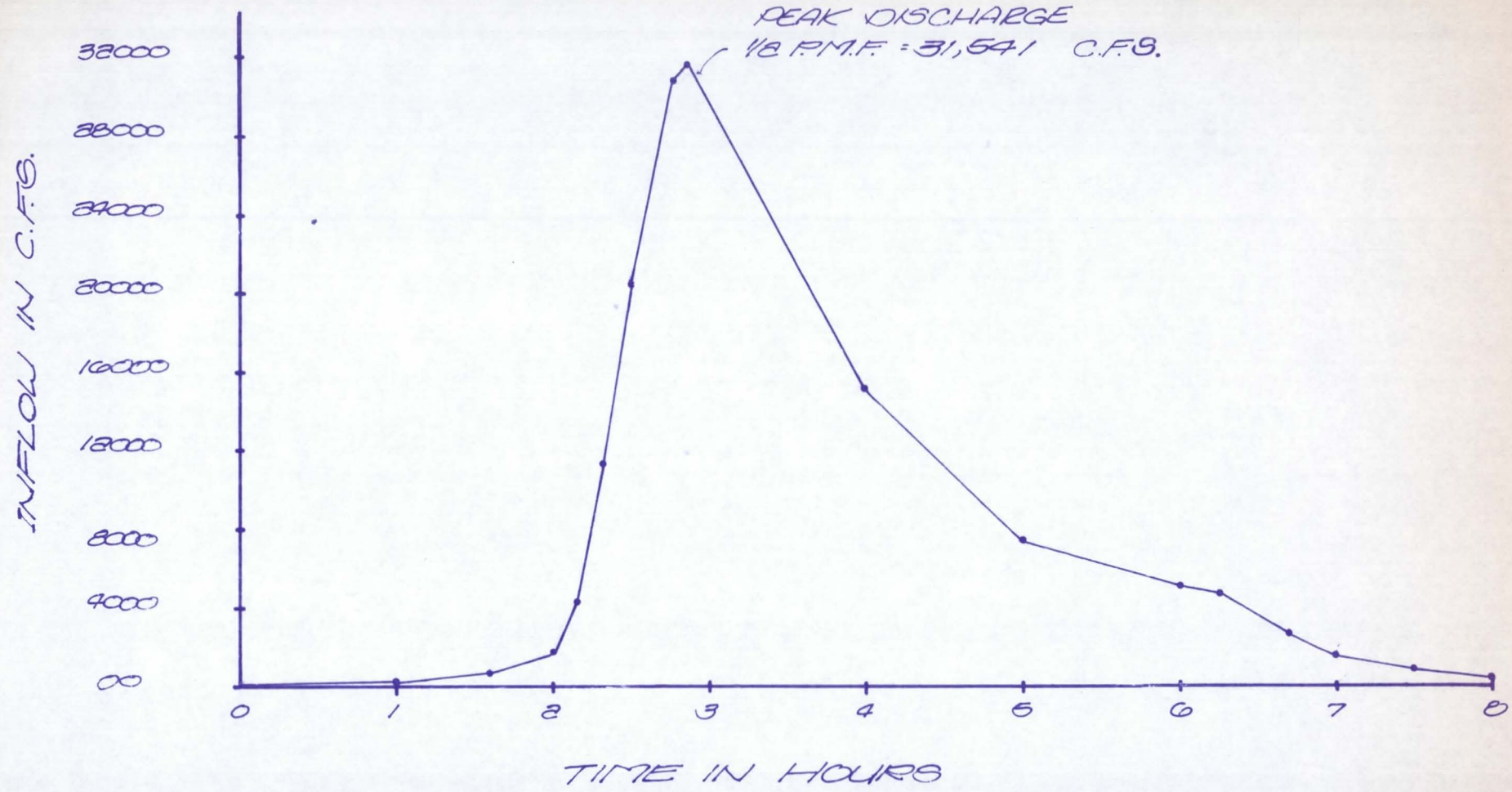
dsn: BM
drn: BH
ckd: BM

HARQUAHALA F.R.S.
72 HOUR 1/2 P.M.F.
INFLOW HYDROGRAPH

FIG. 11



DATE: 8-13-90

JOB Nº 016-83394-02

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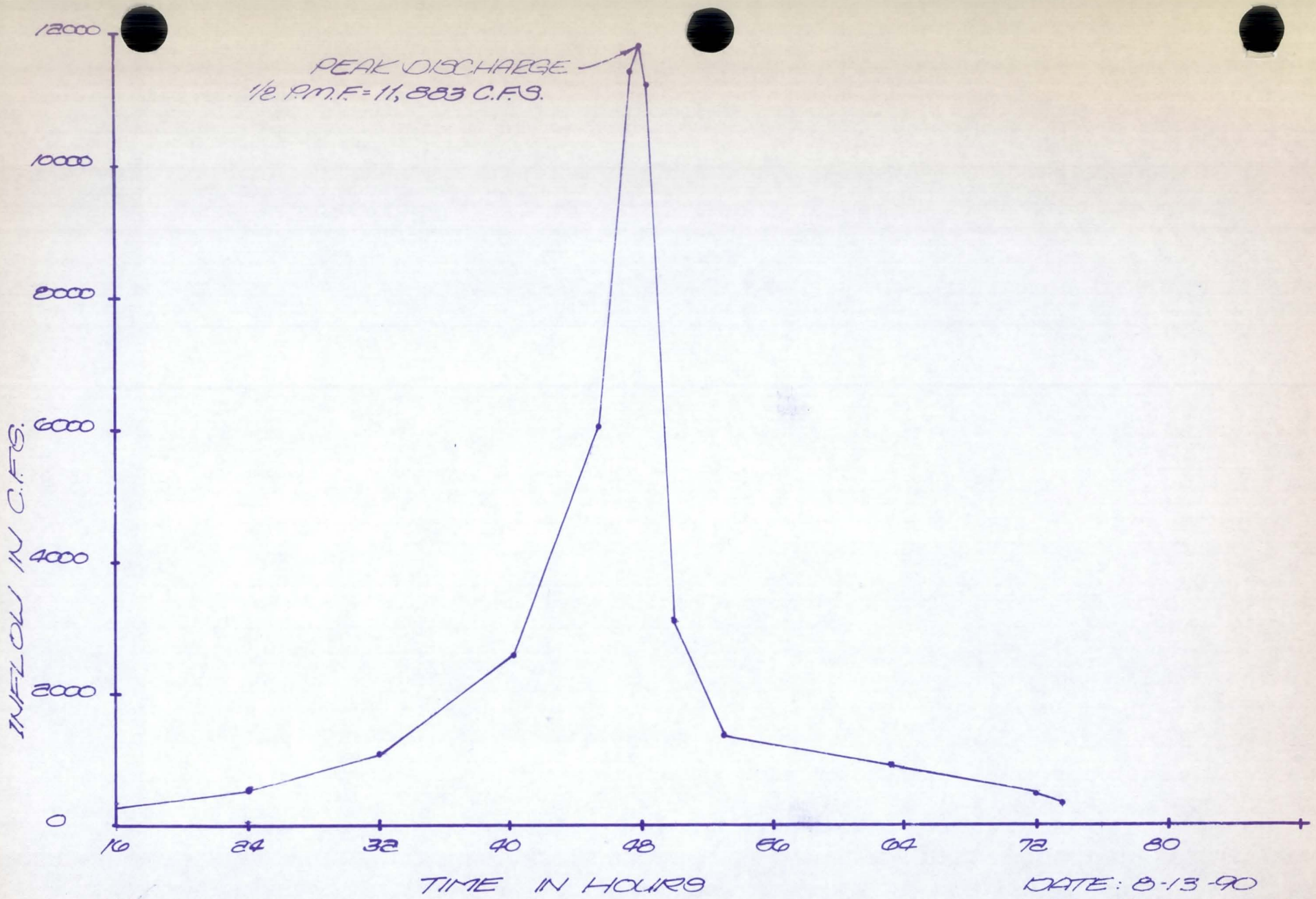
dsn: BM
drn: BH
ckd: BM

SADDLEBACK FRS
6 HOUR 1/2 P.M.F.
INFLOW HYDROGRAPH

FIG. 18



DATE: 8-13-90

JOB Nº 010-83394-02

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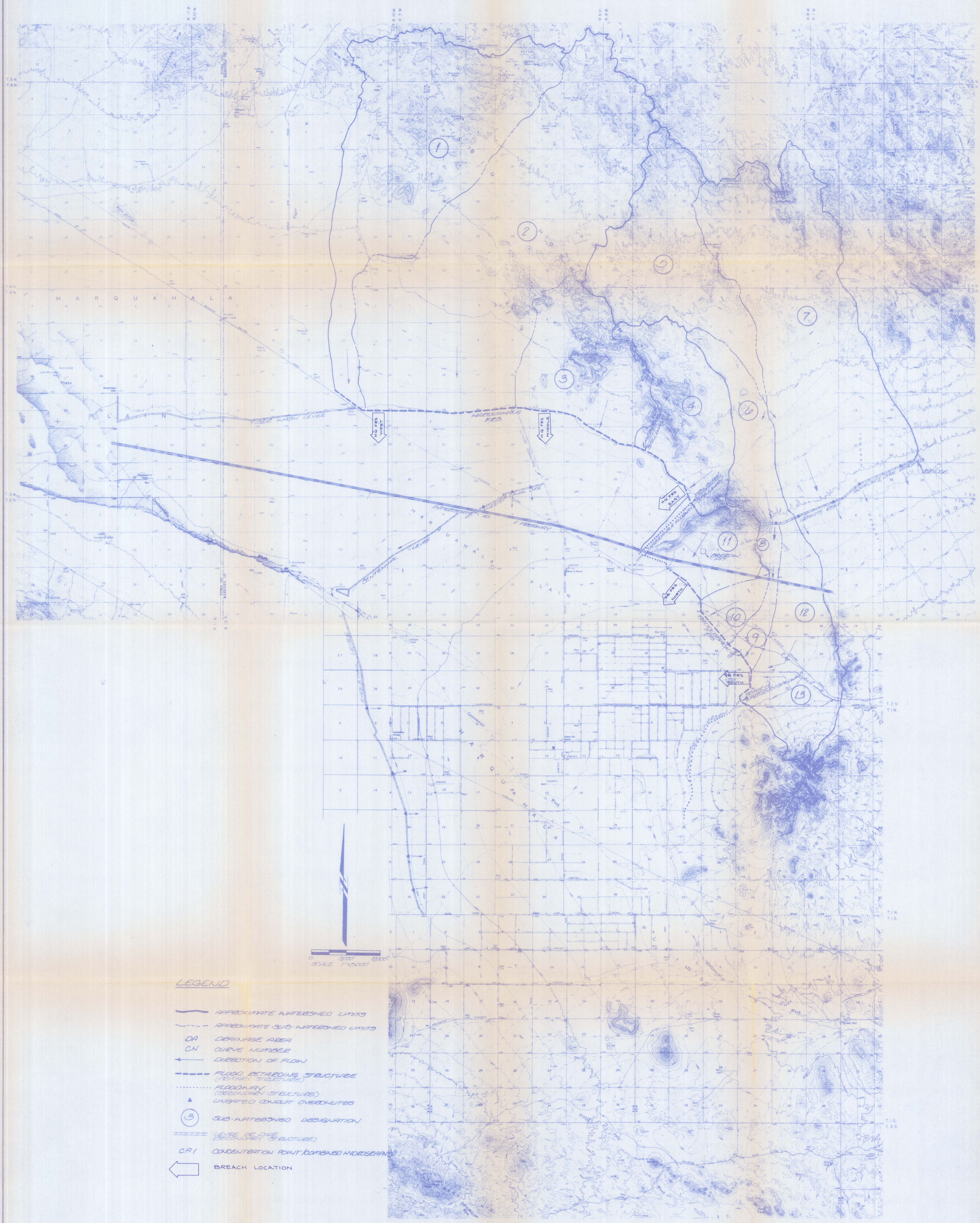
dsn:
drn:
ckd:

SADDLEBACK F.R.S.
72 HOUR 1/2 P.M.F.
INFLOW HYDROGRAPH

FIG. 13

DAMBREAK ANALYSIS
OF
FAR-WEST MARICOPA COUNTY STRUCTURES



LEGEND

- APPROXIMATE WATERSHED LIMITS
- APPROXIMATE SUB-WATERSHED LIMITS
- DA — DRAINAGE AREA
- CN — CURVE NUMBER
- DIRECTION OF FLOW
- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- FLOODWAY (SECONDARY STRUCTURE)
- UNGATED CONDUIT OVERCHUTES
- (3) — SUB-WATERSHED DESIGNATION
- LEVEE OR DIKE (SECONDARY STRUCTURE)
- CP1 — CONCENTRATION POINT/COMBINED HYDROGRAPH
- BREACH LOCATION

DATE: Oct. 24, 1990
JOB Nº 010.83395.03

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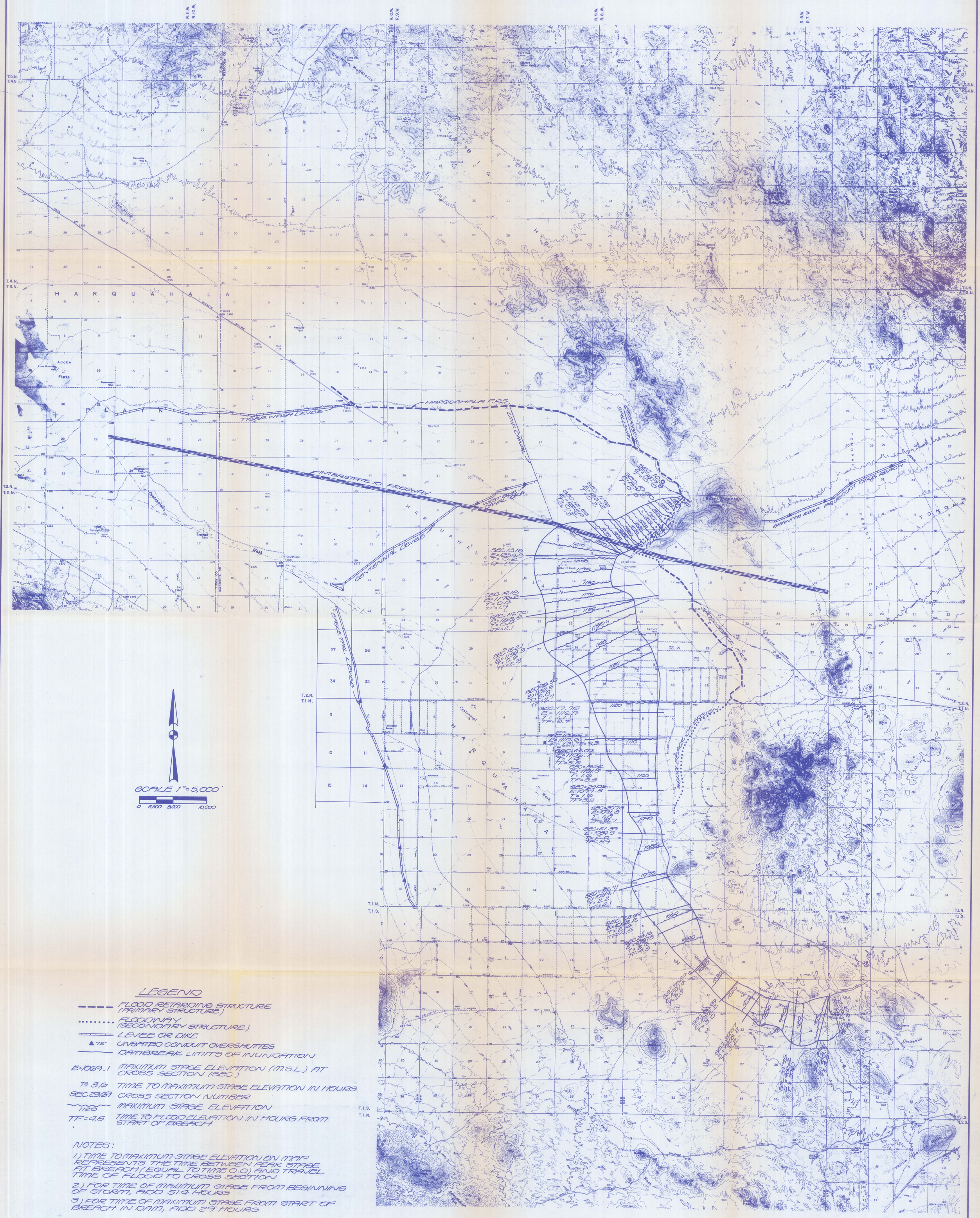
dsn:
drn:
ckd:

BREACH LOCATION MAP

FIGURE 14

F.C.D.M.C. CONTRACT FCD 88-66

DAMBREAK ANALYSIS
OF
FARWEST MARICOPA COUNTY STRUCTURES
FLOOD INUNDATION LIMITS



SCALE 1"=5,000'

0 2,500 5,000 10,000

LEGEND

- ---- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- ..... FLOODWAY (SECONDARY STRUCTURE)
- ═══ LEVEE OR DIKE
- ▲72" UNGATED CONDUIT OVERSHUTTES
- —— DAMBREAK LIMITS OF INUNDATION
- E=1069.1 MAXIMUM STAGE ELEVATION (M.S.L.) AT CROSS SECTION (SEC.)
- T=3.6 TIME TO MAXIMUM STAGE ELEVATION IN HOURS
- SEC 23.69 CROSS SECTION NUMBER
- 1128 MAXIMUM STAGE ELEVATION
- TF=4.5 TIME TO FLOOD ELEVATION IN HOURS FROM START OF BREACH

NOTES:

1) TIME TO MAXIMUM STAGE ELEVATION ON MAP REPRESENTS THE TIME BETWEEN PEAK STAGE AT BREACH (EQUAL TO TIME 0.0) AND TRAVEL TIME OF FLOOD TO CROSS SECTION

2.) FOR TIME OF MAXIMUM STAGE FROM BEGINNING OF STORM, ADD 51.4 HOURS

3.) FOR TIME OF MAXIMUM STAGE FROM START OF BREACH IN DAM, ADD 2.9 HOURS

DATE: JAN 1991
JOB NO# 016-83395.03

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drn:
ckd:

LIMITS OF INUNDATION
EAST BREACH
HARQUAHALA F.R.S.

FIG. 15

F.C.D. 88-66

DAMBREAK ANALYSIS
OF
FARWEST MARICOPA COUNTY STRUCTURES
FLOOD INUNDATION LIMITS

R.11.W. R.10.W. R.10.W. R.9.W. R.9.W. R.8.W. R.8.W. R.7.W.

T.5.N. T.4.N. T.4.N. T.3.N. T.3.N. T.2.N. T.2.N. T.1.N. T.1.N. T.1.S.

H A R Q U A H A L A P L A I N

H A R Q U A H A L A

SEC. 8.68
E = 1917.8
T = 0.0

HARQUAHALA F.R.S.

SEC 8.92
E=1866.10
T=0.00

SEC 9.68
E=1836.9
T=0.2

SEC 10.53
E=1285.6
T=0.2

SEC 11.67
E=1247.4
T=0.4

SEC 12.52
E=1286.4
T=0.6
TF=1.3

SEC 13.16
E=1608.0
T=0.8
TF=1.9

SEC 14.18
E=1175.9
T=0.8
TF=2.1

SEC 14.70
E=1068.2
T=0.8
TF=2.8

SEC 15.21
E=1169.1
T=1.0
TF=2.5

SEC 16.50
E=1152.1
T=1.0
TF=2.8

SEC 17.75
E=1110.4
T=1.0
TF=3.3

SEC 18.53
E=1110.00
T=1.8+TF=3.6

SEC 19.00
E=1109.9
T=1.0, TF=3.7

SEC 19.52
E=1107.0
T=2.0
TF=3.7

SEC 20.08
E=1089.9
T=2.2
TF=3.7

SEC 20.78
E=1090.7
T=2.4
TF=3.9

SEC 21.33
E=1089.4
T=2.6
TF=4.1

SEC 22.71
E=1089.5
T=2.8
TF=4.5

SEC 23.68
E=1091.8
T=3.2
TF=4.7

SEC 24.10
E=1089.2
T=3.4
TF=4.5

INTERSTATE 10 FREEWAY

TIBER WASH LEVEE

CENTENNIAL LEVEE

EAGLE TAIL LEVEE

HARQUAHALA FLOOD CONTROL DIST. CANAL

EMERGENCY SPILLWAY

SPILLWAY DIVERSION

GRANITE REEF AQUEDUCT PIPELINE REACH

SCALE 1"=5,000'

0 2,500 5,000 10,000

LEGEND

- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- FLOODWAY (SECONDARY STRUCTURE)
- LEVEE OR DIKE
- ▲ 72" UNGATED CONDUIT OVERSHUTTES
- DAMBREAK LIMITS OF INUNDATION
- E=1069.1 MAXIMUM STAGE ELEVATION (M.S.L.) AT CROSS SECTION (SEC.)
- T=3.6 TIME TO MAXIMUM STAGE ELEVATION IN HOURS
- SEC 23.68 CROSS SECTION NUMBER
- 1090 MAXIMUM STAGE ELEVATION
- TF=4.5 TIME TO FLOOD ELEVATION IN HOURS FROM START OF BREACH

NOTES:

1.) TIME TO MAXIMUM STAGE ELEVATION ON MAP REPRESENTS THE TIME BETWEEN PEAK STAGE AT BREACH (EQUAL TO TIME 0.0) AND TRAVEL TIME OF FLOOD TO CROSS SECTION.

2.) FOR TIME OF MAXIMUM STAGE FROM BEGINNING OF STORM ADD 51.2 HOURS.

3.) FOR TIME OF MAXIMUM STAGE FROM START OF BREACH IN DAM, ADD 2.7 HOURS.

DATE: JAN. - 1991
JOB NO# 016-83395.03

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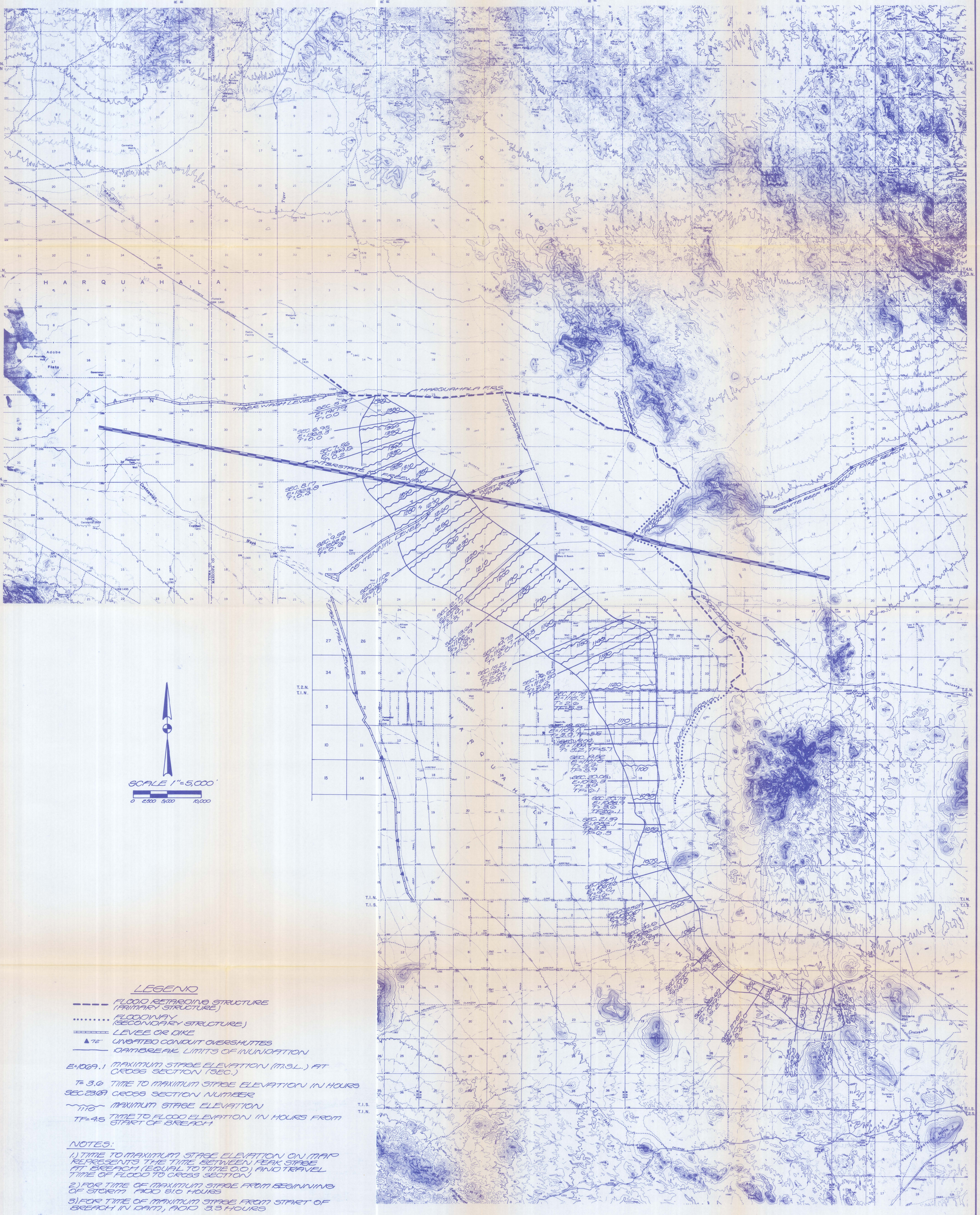
dsn:
drn:
ckd:

LIMITS OF INUNDATION
MIDDLE BREACH
HARQUAHALA F.R.S.

FIG. 16

F.C.D. 88-66

DAMBREAK ANALYSIS OF FARWEST MARICOPA COUNTY STRUCTURES FLOOD INUNDATION LIMITS



LEGEND

- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- FLOODWAY (SECONDARY STRUCTURE)
- LEVEE OR DIKE
- ▲72" UNGATED CONDUIT OVERSHUTTES
- DAMBREAK LIMITS OF INUNDATION

E=1069.1 MAXIMUM STAGE ELEVATION (M.S.L.) AT CROSS SECTION (SEC.)

T= 3.6 TIME TO MAXIMUM STAGE ELEVATION IN HOURS

SEC 23.89 CROSS SECTION NUMBER

1110 MAXIMUM STAGE ELEVATION

TF=4.5 TIME TO FLOOD ELEVATION IN HOURS FROM START OF BREACH

NOTES:

1.) TIME TO MAXIMUM STAGE ELEVATION ON MAP REPRESENTS THE TIME BETWEEN PEAK STAGE AT BREACH (EQUAL TO TIME 0.0) AND TRAVEL TIME OF FLOOD TO CROSS SECTION

2.) FOR TIME OF MAXIMUM STAGE FROM BEGINNING OF STORM ADD 51.0 HOURS

3) FOR TIME OF MAXIMUM STAGE FROM START OF BREACH IN DAM, ADD 3.3 HOURS

DATE: JAN.-1991
JOB NO.# 016-83395.03

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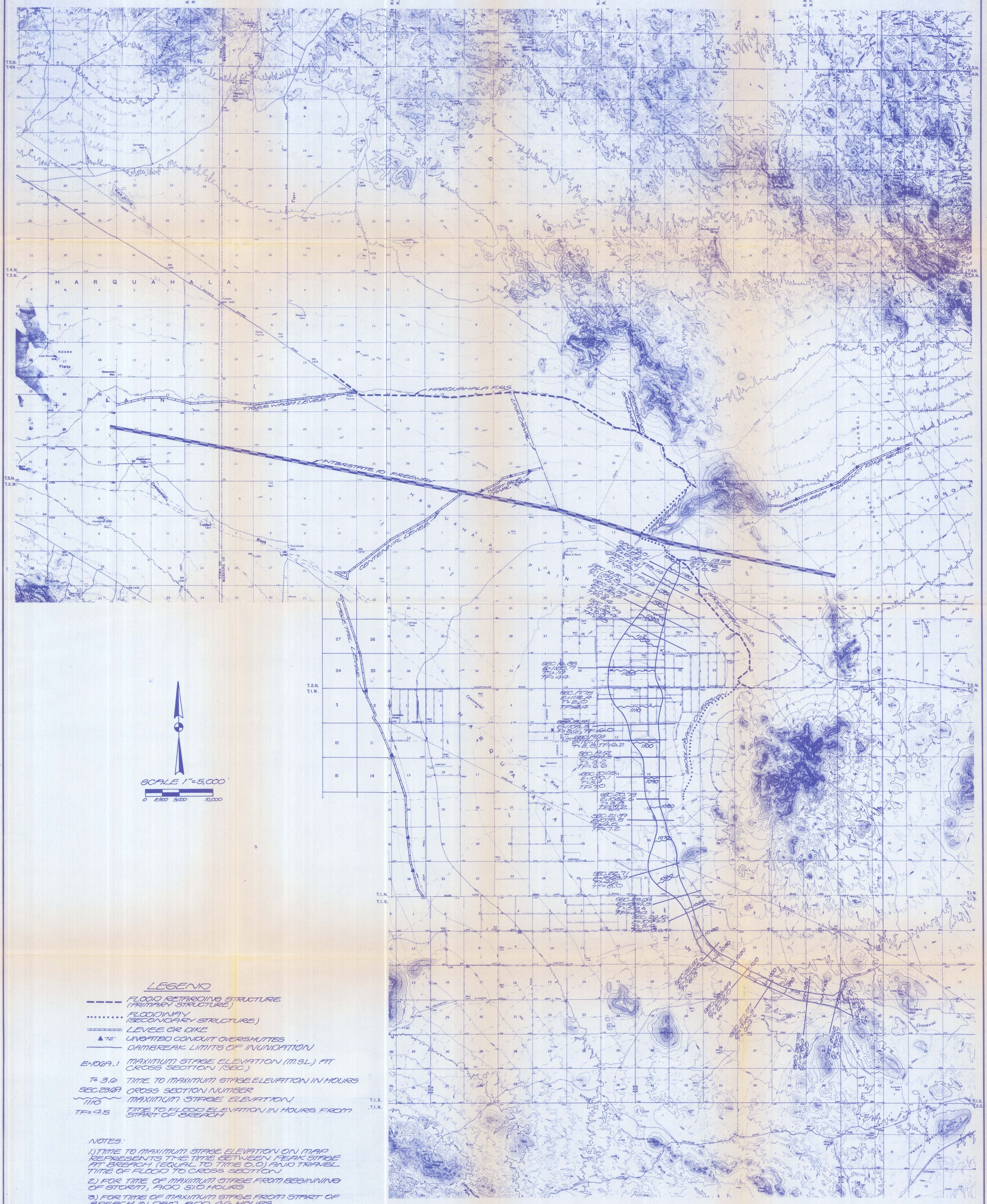
dsn:
drn:
ckd:

LIMITS OF INUNDATION WEST BREACH HARQUAHALA F.R.S.

FIG. 17

FCD 88-66

DAMBREAK ANALYSIS
OF
FARWEST MARICOPA COUNTY STRUCTURES
FLOOD INUNDATION LIMITS



LEGEND

- ---- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- ..... FLOODWAY (SECONDARY STRUCTURE)
- ═══ LEVEE OR DIKE
- ▲72" UNGATED CONDUIT OVERSHUTTES
- ——— DAMBREAK LIMITS OF INUNDATION
- E=1069.1 MAXIMUM STAGE ELEVATION (M.S.L.) AT CROSS SECTION (SEC.)
- T=3.6 TIME TO MAXIMUM STAGE ELEVATION IN HOURS
- SEC.23.89 CROSS SECTION NUMBER
- 1110 MAXIMUM STAGE ELEVATION
- TF=9.5 TIME TO FLOOD ELEVATION IN HOURS FROM START OF BREACH

T.1.S.
T.1.N.

NOTES:

1) TIME TO MAXIMUM STAGE ELEVATION ON MAP REPRESENTS THE TIME BETWEEN PEAK STAGE AT BREACH (EQUAL TO TIME 0.0) AND TRAVEL TIME OF FLOOD TO CROSS SECTION

2) FOR TIME OF MAXIMUM STAGE FROM BEGINNING OF STORM, ADD 51.0 HOURS

3) FOR TIME OF MAXIMUM STAGE FROM START OF BREACH IN DAM, ADD 4.6 HOURS

DATE: JAN. 1991
JOB NO. 016-83395.03

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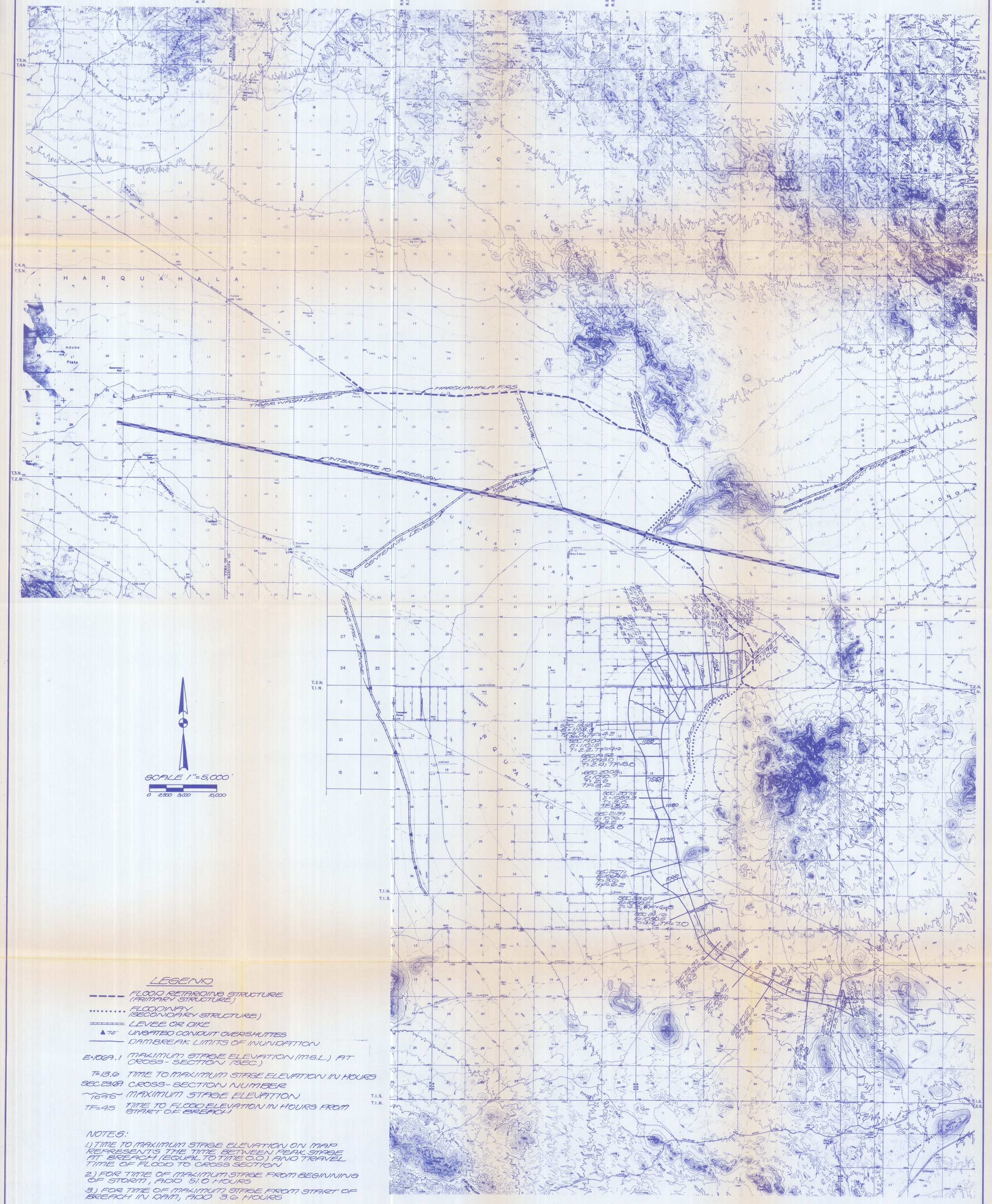
dsn:
drn:
ckd:

LIMITS OF INUNDATION
NORTH BREACH
SADDLEBACK F.R.S.

FIG. 18

F.C.D. 88-66

DAMBREAK ANALYSIS
OF
FARWEST MARICOPA COUNTY STRUCTURES
FLOOD INUNDATION LIMITS



LEGEND

- ---- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- ..... FLOODWAY (SECONDARY STRUCTURE)
- ═══ LEVEE OR DIKE
- ▲ 72" UNGATED CONDUIT OVERSHUTTES
- ——— DAMBREAK LIMITS OF INUNDATION
- E=1069.1 MAXIMUM STAGE ELEVATION (M.S.L.) AT CROSS-SECTION (SEC.)
- T=13.0 TIME TO MAXIMUM STAGE ELEVATION IN HOURS
- SEC 23.69 CROSS-SECTION NUMBER
- ~1090 MAXIMUM STAGE ELEVATION
- TF=4.5 TIME TO FLOOD ELEVATION IN HOURS FROM START OF BREACH

NOTES:

1.) TIME TO MAXIMUM STAGE ELEVATION ON MAP REPRESENTS THE TIME BETWEEN PEAK STAGE AT BREACH (EQUAL TO TIME 0.0) AND TRAVEL TIME OF FLOOD TO CROSS SECTION

2.) FOR TIME OF MAXIMUM STAGE FROM BEGINNING OF STORM, ADD 51.0 HOURS

3.) FOR TIME OF MAXIMUM STAGE FROM START OF BREACH IN DAM, ADD 3.6 HOURS

DATE: JAN-1991
JOB NO. 016-83395.03

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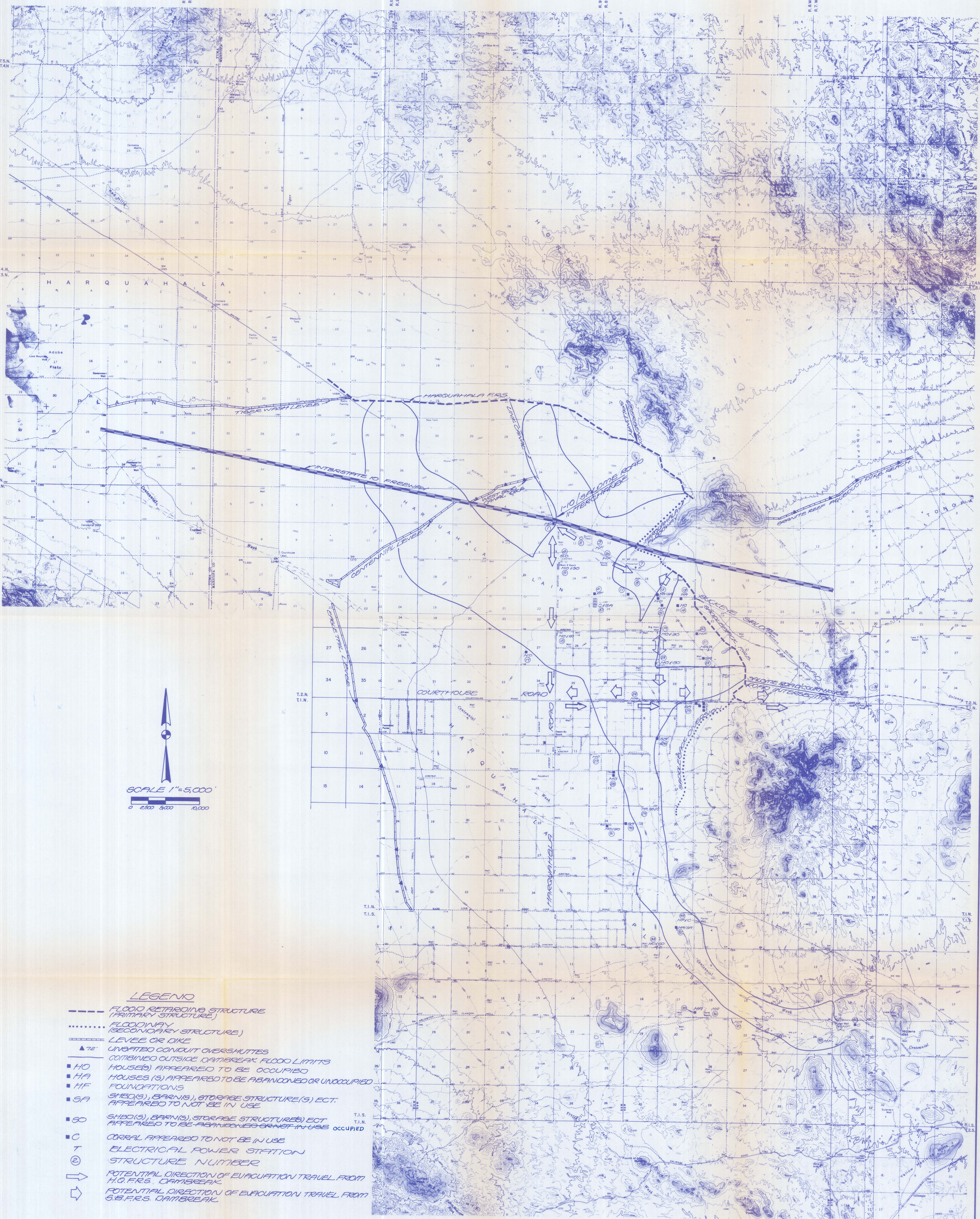
LIMITS OF INUNDATION
SOUTH BREACH
SADDLEBACK F.R.S.

FIG. 19

F.C.D. 88-66

DAMBREAK ANALYSIS OF FARWEST MARICOPA COUNTY STRUCTURES



LEGEND

- FLOOD RETARDING STRUCTURE (PRIMARY STRUCTURE)
- FLOODWAY (SECONDARY STRUCTURE)
- LEVEE OR DIKE
- ▲72" UNGATED CONDUIT OVERSHUTTES
- COMBINED OUTSIDE DAMBREAK FLOOD LIMITS
- ■ HO HOUSE(S) APPEARED TO BE OCCUPIED
- ■ HA HOUSES (S) APPEARED TO BE ABANDONED OR UNOCCUPIED
- ■ HF FOUNDATIONS
- ■ SA SHED(S), BARN(S), STORAGE STRUCTURE(S) ECT. APPEARED TO NOT BE IN USE
- ■ SO SHED(S), BARN(S), STORAGE STRUCTURE(S) ECT APPEARED TO BE ~~ABANDONED OR NOT IN USE~~ OCCUPIED
- ■ C CORRAL APPEARED TO NOT BE IN USE
- T ELECTRICAL POWER STATION
- ② STRUCTURE NUMBER
- ⇨ POTENTIAL DIRECTION OF EVACUATION TRAVEL FROM H.Q.F.R.S. DAMBREAK
- ⇨ POTENTIAL DIRECTION OF EVACUATION TRAVEL FROM S.B.F.R.S. DAMBREAK

DATE: JAN - 1991
JOB NO# 016-83395.03

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dsn:
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ckd:

STRUCTURE LOCATION MAP — FIG. 20

FCD 88-66

TABLE 1
PRINCIPAL FEATURES OF HQFRS AND SBFRS

	Principal Features	HQFRS	SBFRS
1.	Watershed area (sq.mi.)	103.3	76.9
2.	Type of FRS	compacted earthfill	same
3.	Crest of FRS (MSL)	1419.7 - 1422.2	1193.0 - 1194
4.	Length of FRS (ft)	62,058	27,270
5.	Maximum height (ft)	46.7	20.3
6.	Top width (ft)	14	12
7.	Principal Discharge Structure	48" RGRCP	10'X8' CBC
8.	Principal Discharge Structure Invert	1387.3	1176.9
9.	Maximum principal spillway discharge	500	1315
10.	Emergency discharge structure	spillway	N/A
11.	Emergency spillway length (ft)	150	N/A
12.	Emergency spillway crest-elevation (MSL)	1,408.4	N/A
13.	Emergency spillway top elevation (MSL)	1,419.7	N/A
14.	Emergency structure discharge	15,607	N/A
15.	Total discharge (principal + emergency discharge structures)	16,107	1315
16.	Maximum storage below dam crest (ac-ft)	12,000	7500

TABLE 2
HARQUAHALA FLOOD RETARDING STRUCTURE
SUBWATERSHEDS

Subwatershed Designation	Basin Area Sq.Mi.
1	30.54
2	48.63
3	12.87
4	11.25
TOTAL	103.29

TABLE 3
SADDLEBACK FLOOD RETARDING STRUCTURE
SUBWATERSHEDS

Subwatershed Designation	Basin Area (sq.mi.)
5	16.01
6	5.65
7	33.07
8	0.90
9	1.67
10	2.11
11	6.35
12	6.71
13	4.46
TOTAL	76.93

TABLE 4
SBFRS SUBWATERSHED AREA MODIFICATION

Original Subwatershed Designation	Original Area (sq.mi.)	Subwatershed Designation	Modified Area (sq.mi.)	Modification
5A[1]	4.32[2]	8	0.90	additional area from (4) 11, lose area to 5B[1](12)
5A[1]	4.32	9	1.67	lost area to 5B (12)
5C	2.01	10	2.11	added area from 4 (11)
4	6.90	11	6.35	lost area to 5A (8 & 10)
5B	4.51	12	6.71	additional area from 5A (8 & 9)

[1]Watershed area and designation from Ref. 8.

[2]Areas divided due to split flow. See Figure 3.

TABLE 5

MODIFIED CURVE NUMBERS

Subwatershed Designation	Modified Curve Number
1	89
2	87
3	88
4	88
5	90
6	89
7	90
8	92
9	86
10	84
11	87
12	85
13	88

NOTE: Sub-watersheds 8 through 12 were modified per this report. See Appendix C for calculations.

TABLE 6
GRANITE REEF AQUEDUCT DIKE
DISCHARGE REVISION

	Structures Draining to Subwatershed 8 and/or 12		Structures Draining to Tonopa Desert				
Ele.	72" Pipe (cfs)	Weir (cfs)	4-72" Pipe (cfs)	Weir (cfs)	V-Notch (cfs)	Total (cfs)	Vol. (ac-ft)
1380.60	0	0	0	0	0	0	50.0
1381.50	-	0	60	0	0	60	65.0
1382.00	5	0	100	0	0	105	105.3
1384.00	50	0	320	0	0	370	271.3
1386.00	125	0	640	0	0	765	590.0
1388.00	220	0	1,020	0	0	1,240	1101.3
1390.00	300	0	1,300	0	0	1,600	1815.5
1392.00	360	0	1,530	0	0	1,890	2987.0
1394.00	410	0	1,720	0	0	2,130	4300.0
1396.00	450	0	1,890	0	0	2,340	5570.0
1398.00	495	0	2,030	0	2,600	5,125	6940.0
1398.25	500	0	2,045	7,020	3,450	13,015	7080.0
1298.50	505	0	2,065	19,850	4,360	26,780	7220.0
1398.75	510	0	2,080	36,470	5,340	44,400	7360.0
1399.00	515	0	2,100	56,150	6,370	65,135	7500.0
1399.25	520	1,068	2,120	78,472	7,468	89,648	7640.0
1399.50	525	3,022	2,140	103,155	8,610	117,485	7780.0
1400.00	535	8,548	2,192	158,820	11,050	181,145	8060.0

TABLE 7

PEAK DISCHARGE AND ONE-HALF PEAK DISCHARGE AT THE HQFRS AND SBFRS (HEC-1 MODEL)

PMF Event	HQFRS Peak Flow (cfs)	HQFRS One-Half Peak Flow (cfs)	SBFRS Peak Flow (cfs)	SBFRS One-Half Peak Flow (cfs)
6-Hour	162,504	81,252	63,083	31,541
72-Hour	101,520	50,760	23,766	11,883

TABLE 8

RUNOFF VOLUMES TO HQFRS AND SBFRS (HEC-1 MODEL)

Flooding Event	Flood Retarding Structure HQFRS (ac-ft)	Flood Retarding Structure SBFRS (ac-ft)
6-Hour	22,038	5,483
72-Hour	38,140	9,040

TABLE 9

SUMMARY OF ROUTING OF 1/2 PMF HYDROGRAPHS
(HEC-1 MODEL)

FRS	ELEV. DAM CREST	PMP STORM	ELEV. MAX STAGE
Harquahala (HQ)	1419.70	6-hour	1415.41
		72-hour	1418.48
Saddleback (SB)	1193.33	6-hour	1190.83
		72-hour	1192.07

TABLE 10

SUMMARY OF BREACH ANALYSIS
(BREACH EROSION MODEL)

FRS	Breach Location	PMP	Initial W.S. Elev.	Initial Breach Elev	Peak Discharge (cfs)	Breach Botton Width	Time for Breach
HQ	East	6-hr.	1415.4	1397.8	146437	307	3.2
		72-hr.	1418.5	1397.8	217237	420	3.1
HQ	Middle	6-hr.	1415.4	1404.9	120725	578	3.6
		72-hr.	1418.5	1404.9	203186	788	3.1
HQ	West	6-hr.	1415.4	1407.9	71888	563	4.5
		72-hr.	1418.5	1407.9	137821	793	3.5
SB	North	6-hr.	1190.80	1183.50	6537	122	5.8
		72-hr.	1192.00	1183.50	11835	201	5.4
SB	South	6-hr.	1190.80	1176.90	11844	64	3.4
		72-hr.	1192.00	1176.90	18061	100	3.7

TABLE 11

SUMMARY OF DAMBRK FAILURE HYDROGRAPH ANALYSIS

FRS	BREACH LOCATION	PMP	PEAK DISCHARGE (cfs)
HQ	East	72-hour	185,613
HQ	Middle	72-hour	155,853
HQ	West	72-hour	119,249
SB	North	72-hour	12,300
SB	South	72-hour	18,585

TABLE 12

MANNING "N" VALUES

Area(s)	Depth	"n" Value	Comments
Desert	0 thru 4.0	0.05	Desert area north of agricultural area
	4.01 and higher	0.04	Desert area north of agricultural area
Agricultural	0 to 3.99	0.05	
	4.0 and higher	0.04	
Desert	varies		Manning "n" value determined by terrain and vegetation
	0 to 8.0	0.05	
	varies 2.0 and higher	0.04	

TABLE 13
TIMES TO MAXIMUM FLOOD STAGE & FLOOD ELEVATION FOR HARQUAHALA FRS-EAST BREACH

CROSS-SECTION NO.	MAXIMUM FLOOD STAGE			FLOOD ELEVATION
	FROM START OF STORM (hrs.)	FROM START OF BREACH (hrs.)	FROM MAXIMUM STAGE AT BREACH (hrs.)	FROM START[1] OF BREACH (hrs.)
10.0	51.4	2.9	0.0	
10.8	51.4	2.9	0.0	
11.92	51.6	3.1	0.2	
12.55	51.6	3.1	0.2	1.3
13.16	51.6	3.1	0.2	1.7
14.18	51.8	3.3	0.4	1.9
14.70	51.8	3.3	0.4	2.1
15.21	52.0	3.5	0.6	2.3
16.50	52.2	3.7	0.8	2.7
17.75	52.5	4.0	1.1	3.1
18.55	52.6	4.1	1.2	3.3
19.04	52.8	4.3	1.4	3.3
19.52	53.0	4.5	1.6	3.5
20.05	53.0	4.5	1.6	3.5
20.73	53.2	4.7	1.8	3.7
21.39	53.4	4.9	2.0	3.9
22.71	53.6	5.1	2.2	4.3
23.69	53.8	5.3	2.4	4.3
24.16	54.0	5.5	2.6	4.5
25.54	54.2	5.7	2.8	
26.69	54.6	6.1	3.2	
27.57	54.6	6.1	3.2	
28.75	55.0	6.5	3.6	
29.62	55.2	6.7	3.8	
30.10	55.4	6.9	4.0	

[1]For selected sections only.

TABLE 14
TIMES TO MAXIMUM FLOOD STAGE & FLOOD ELEVATION
FOR HARQUAHALA FRS-MIDDLE BREACH

CROSS-SECTION NO.	MAXIMUM FLOOD STAGE			FLOOD ELEVATION
	FROM START OF STORM (hrs.)	FROM START OF BREACH (hrs.)	FROM MAXIMUM STAGE AT BREACH (hrs.)	FROM START[1] OF BREACH (hrs.)
8.62	51.2	2.7	0.0	
8.92	51.2	2.7	0.0	
9.64	51.4	2.9	0.2	
10.53	51.4	2.9	0.2	
11.67	51.6	3.1	0.4	
12.52	51.8	3.3	0.6	1.3
13.16	51.8	3.3	0.6	1.9
14.18	52.0	3.5	0.8	2.1
14.7	52.0	3.5	0.8	2.3
15.21	52.2	3.7	1.0	2.5
16.5	52.4	3.9	1.2	2.9
17.75	52.8	4.3	1.6	3.3
18.55	53.0	4.5	1.8	3.5
19.04	53.0	4.5	1.8	3.7
19.52	53.2	4.7	2.0	3.7
20.05	53.4	4.9	2.2	3.9
20.73	53.6	5.1	2.4	3.9
21.39	53.8	5.3	2.6	4.1
22.71	54.0	5.5	2.8	4.5
23.69	54.4	5.9	3.2	4.7
24.16	54.4	5.9	3.2	4.7
25.54	54.6	6.1	3.4	
26.69	55.0	6.5	3.8	
27.57	55.2	6.7	4.0	
28.75	55.4	6.9	4.2	
29.62	55.8	7.3	4.6	
30.10	55.8	7.3	4.6	

[1]For selected sections only.

TABLE 15
TIMES TO MAXIMUM FLOOD STAGE & FLOOD ELEVATION
FOR HARQUAHALA FRS-WEST BREACH

CROSS-SECTION NO.	MAXIMUM FLOOD STAGE			FLOOD ELEVATION
	FROM START OF STORM (hrs.)	FROM START OF BREACH (hrs.)	FROM MAXIMUM STAGE AT BREACH (hrs.)	FROM START[1] OF BREACH (hrs.)
5.97	51.85	3.3	0.0	
6.95	51.8	3.3	0.0	
7.56	52.0	3.5	0.2	
8.17	52.2	3.7	0.4	
9.54	52.4	3.9	0.6	
11.06	52.8	4.3	1.0	
12.70	53.2	4.7	1.4	3.7
13.64	53.4	4.9	1.6	3.9
14.73	53.8	5.3	2.0	4.3
15.21	53.8	5.3	2.0	4.7
16.50	54.2	5.7	2.4	4.9
17.75	54.4	5.9	2.6	5.3
18.55	54.8	6.3	3.0	5.5
19.04	55.0	6.5	3.2	5.7
19.52	55.2	6.7	3.4	5.9
20.05	55.2	6.7	3.4	6.1
20.73	55.4	6.9	3.6	6.1
21.39	55.6	7.1	3.8	6.3
22.71	55.8	7.3	4.0	6.7
23.69	56.2	7.7	4.4	6.9
24.16	56.4	7.9	4.6	6.9
25.54	56.6	8.1	4.8	
26.69	57.0	8.5	5.2	
27.57	57.2	8.7	5.4	
28.75	57.4	8.9	5.6	
29.62	57.6	9.1	5.8	
30.10	57.6	9.1	5.8	

[1]For selected sections only.

TABLE 16
TIMES TO MAXIMUM FLOOD STAGE & FLOOD ELEVATION
FOR SBRRS-NORTH BREACH

CROSS-SECTION NO.	MAXIMUM FLOOD STAGE			FLOOD ELEVATION
	FROM START OF STORM (hrs.)	FROM START OF BREACH (hrs.)	FROM MAXIMUM STAGE AT BREACH (hrs.)	FROM START[(1)] OF BREACH (hrs.)
13.53	51.0	4.6	0.0	
14.01	51.2	4.8	0.2	1.4
14.41	51.2	4.8	0.2	2.6
14.87	51.4	5.0	0.4	3.2
15.33	51.6	5.2	0.6	3.4
15.79	51.8	5.4	0.8	3.8
16.88	52.4	6.0	1.4	4.4
17.75	53.0	6.6	2.0	4.4
18.55	53.6	7.2	2.6	6.0
19.04	53.8	7.4	2.8	6.2
19.52	54.2	7.8	3.2	6.6
20.05	54.4	8.0	3.4	7.0
20.73	54.8	8.4	3.8	7.2
21.39	55.0	8.6	4.0	7.2
22.71	55.6	9.2	4.6	8.0
23.69	56.2	9.8	5.2	8.4
24.16	56.4	10.0	5.4	8.6
25.54	56.8	10.4	5.8	
26.69	57.2	10.8	6.2	
27.57	57.4	11.0	6.4	
28.75	57.8	11.4	6.8	
29.62	58.0	11.6	7.0	
30.10	58.2	11.8	7.2	

[(1)]For selected sections only.

TABLE 17
TIMES TO MAXIMUM FLOOD STAGE & FLOOD ELEVATION FOR SBRRS-SOUTH BREACH

CROSS-SECTION NO.	MAXIMUM FLOOD STAGE			FLOOD ELEVATION
	FROM START OF STORM (hrs.)	FROM START OF BREACH (hrs.)	FROM MAXIMUM STAGE AT BREACH (hrs.)	FROM START[1] OF BREACH (hrs.)
14.17	51.0	3.6	0.0	
14.61	51.2	3.8	0.2	2.2
15.06	51.2	3.8	0.2	2.6
15.54	51.4	4.0	0.4	2.8
16.66	52.0	4.6	1.0	3.2
17.25	52.2	4.8	1.2	3.8
18.55	53.0	5.6	2.0	4.2
19.04	53.2	5.8	2.2	4.4
19.52	53.4	6.0	2.4	5.0
20.05	53.6	6.2	2.6	5.2
20.73	54.0	6.6	3.0	5.4
21.39	54.2	6.8	3.2	5.6
22.71	54.6	7.2	3.6	6.2
23.69	55.2	7.8	4.2	6.8
24.16	55.4	8.0	4.4	7.0
25.54	55.8	8.4	4.8	
26.69	56.0	8.6	5.0	
27.57	56.2	8.8	5.2	
28.75	56.6	9.2	5.6	
29.62	56.8	9.4	5.8	
30.10	57.2	9.8	6.2	

[1]For selected sections only.

TABLE 18
AREA OF FLOODED AGRICULTURAL AND RANGE LAND*

Dambrk Scenario	Agricultural Area (ac)	Range Land Area (ac)
HQFRS East	8100	6000
HQFRS Middle	7900	5900
HQFRS West	6500	11940
SBFRS North	2700	2800
SBFRS South	2500	2800

*All areas are from I-10 south to end of project.

TABLE 19

RESIDENTIAL AND COMMERCIAL STRUCTURES

Structure No.[A]	Structure Description	Appeared Occupied	Appeared Abandoned	Within Flood-limits[B]	Evac-uation Route(s)[C]
1	Trailor/Shed	X	-	6	A
2	Corral	-	x	1,2	
3	Foundation	-	x	1,2	
4	Shed	x	-	1,2	A
5	House/Shed	x	-	1,2	A
6	Foundation	-	x	1,2	
7	Transformer	-	-	7	
8	Foundation	-	x	7	
9	Foundation	-	x	1,2	
10	Shed/Corral	-	x	1,2,3	
11	Shed	-	-	1,2	
12	Trailer/Shed	-	x	4	
13	House	-	x	7	
14	House	x	-	7	A or B
15	House	-	x	4	
16	Trailer	x	-	4	A or B
17	House	x	-	3	A
18	House/Shed	x	-	1,2,3	A
19	House/Shed	x	-	4	A or B
20	Foundation	-	x	7	
21	House	-	x	7	
22	Shed	-	x	7	
23	House/Shed	x	-	1,2,3,4,5	B
24	House	-	x	1,2,3,4,5	
25	House/Shed	x	-	5	B
26	Shed	-	x	1,2,3,4,5	
27	House	x	-	N/A	
28	House/Shed	x	-	N/A	
29	House/Shed	-	x	1,2,3,4,5	
30	Shed	-	x	N/A	
31	House/Shed	x	-	N/A	
32	House/Shed	-	x	1,2,3,4,5	
33	Foundation	-	x	1,2,3	
34	House/Shed	x	-	N/A	
35	House/Shed	-	x	N/A	
36	House/Shed	-	x	N/A	
37	House	-	x	N/A	
38	Foundation	-	x	1,2,3	

TABLE 19 (continued)

RESIDENTIAL AND COMMERCIAL STRUCTURES

Structure No.[A]	Structure Description	Appeared Occupied	Appeared Abandoned	Within Flood-limits[B]	Evac-uation Route(s)[C]
39	Foundation	-	x	1,2,3	
40	Corral	-	-	1,2,3,4,5	

(A) Structure location given on Figure 7.

(B) Floodlimits:
1. HQFRS - East
2. HQFRS - Middle
3. HQFRS - West
4. SBFRS - North
5. SBFRS - South
6. HQFRS - Dambreak other locations
7. SBFRS - Dambreak other locations

(C) Potential evacuation routes:
1. Travel north to I-10/Salome Road interchange or south and east to Salome Road/Courthouse Road interchange.
2. Travel south and west to Harquahala Road/Courthouse Road interchange or south and east to Salome Road/Courthouse Road interchange.