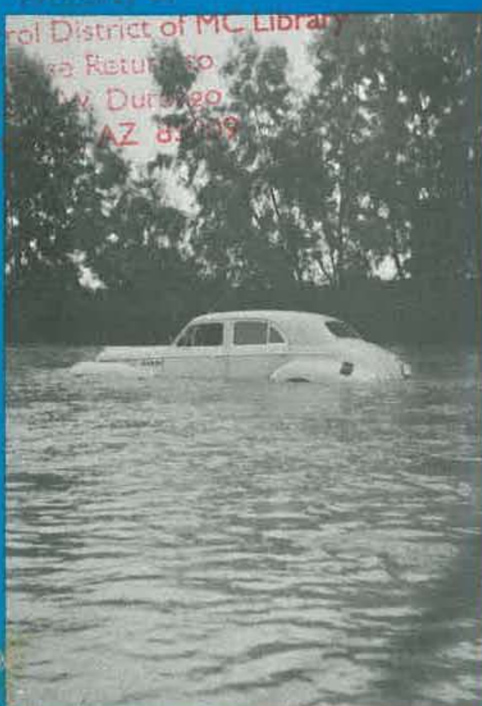
FLOOD CONTROL SURVEY & REPORT

23

study area one
southeastern maricopa county
arizona



prepared by
BENHAM ENGINEERING COMPANY, INC.
consulting engineers phoenix, arizona

FLOOD CONTROL SURVEY AND REPORT

STUDY AREA I

SOUTHEASTERN MARICOPA COUNTY

For

FLOOD CONTROL DISTRICT

of

MARICOPA COUNTY

June 1962

Prepared by

BENHAM ENGINEERING COMPANY, INC.

Consulting Engineers

Phoenix, Arizona

FOUNDED BY
WEBSTER LANCE BENHAM
IN 1909

BENHAM ENGINEERING COMPANY, INC.

SURVEYS •PLANS •CONSTRUCTION SUPERVISION OF CIVIL •MECHANICAL •ELECTRICAL • STRUCTURAL •SANITARY ENGINEERING PROJECTS AND ALL TYPES OF BUILDING CONSTRUCTION

SUITE 623 GUARANTY BANK BUILDING PHOENIX, ARIZONA AM 6-7333

June 11, 1962

Mr. John C. Lowry
Chief Engineer and Manager
Flood Control District
Maricopa County
Phoenix, Arizona

Re: Flood Control Report
Study Area No. 1

Dear Mr. Lowry:

In accordance with our contractual agreement and your subsequent request for additional copies, we submit, herewith, fifty (50) copies of the final draft of our "Flood Control Survey and Report, Study Area No. 1, Southeastern Maricopa County," including tables, maps, and preliminary plans from which the study was derived.

We wish to personally thank you and your staff for the courtesy and co-operation extended to us in the preparation of this report.

We are ready to proceed with final detailed plans and specifications on any of the included projects and to help in their promotion in any manner your office may desire.

Respectfully submitted,

BENHAM ENGINEERING COMPANY, INC.

Chandler J. McCoy

Registered Professional Engineer (Civil)
CERTIFICATE EXPIRES
CHANDLER JERRY McCOY
DEC. 31,
ARIZONA, U. A.

HED:mm

DAVID BLAIR BENHAM, PRESIDENT
CHANDLER J. McCOY, EXEC. VICE PRESIDENT
JOHN F. BENHAM, SECRETARY-TREASURER

PROFESSIONAL ENGINEERS:
• HAROLD E. DITZLER
• WILSON D. CHARLES
• CARL O. POPPINO
• KIRK H. STEALY
• ALLEN G. POPPINO
• JOHN CARNEY
• FRANK RODESNEY
• THOMAS B. SHOEBOTHAM
• THOMAS C. MOORE
• RUSSELL A. REA
• NORMAN B. ASH
• BILL J. BLAIR
• CARL O. JENSEN
• JOHN E. RUTLEDGE
• LEE C. STOLDT
• VINCENT J. SMITH

TABLE OF CONTENTS

TABLES

I GILA RIVER - QUEEN CREEK BASIN

1-1. DESCRIPTION OF BASIN: The Gila River - Queen Creek Basin is a project study designation rather than a cohesive watershed. In general, it is part of the Gila River Basin and contains areas with more or less common flood problems, an exception being the area between Roosevelt Dam and Granite Reef Dam on the south bank which is part of the Salt River Basin. The basin is further broken down into study areas for the purpose of this report. Although these study areas are subject to the flood problems of the whole, they have distincitve individual characteristics and have internal flood problems for which solutions must be sought.

a. Location and Extent. The basin is located in the southeast corner of Maricopa County between the Salt River and the Gila River. It includes areas in Pinal and Gila Counties which contribute to the flood problems of the areas in Maricopa County. The boundary of the basin begins at Roosevelt Dam and follows the Salt River down to Granite Reef Dam, then it angles off to the southwest, passing through Mesa and along the divide of the Salt River Mountains to the Gila River. From this point it follows the Gila River southeast to approximately Gila Butte and then easterly to intercept Hunt Highway at a point due south of Higley. From here the boundary follows the south divide of the Queen Creek watershed westerly to Whitlow Dam; thence northeasterly around the watershed divide of Whitlow Canyon to the Maricopa County - Pinal County Line. It follows this line north and then east to the Gila County line; thence northeast along the Maricopa County - Gila County line back to Roosevelt Dam. The location and extent of the basin and the breakdown into study areas is shown on Plate No. 1, Basin Index Map, in Appendix I-A Hydrologic and Hydraulic Analyses.

b. Streams and Washes. The principal stream of the basin is the Queen Creek Wash. Numerous washes head in the Superstition and Goldfield Mountains but disappear in the large desert area on the east slope of the basin. The flood flows from these washes eventually join Queen Creek in the lower reaches due to diversion effected by the embankment of the R.W.C.D. Main Canal, an irrigation canal which is part of the Roosevelt Conservation District Irrigation System. In the mountainous area above Granite Reef Dam which is part of the Salt River watershed there are several creeks which are of principal interest as sources of runoff for the existing Mormon Flat Reservoir and the Granite Reef Reservoir.

(1) Queen Creek Wash heads in the mountains above Whitlow Reservoir draining 143 square miles. Immediately below the dam, the stream is joined by 40 square miles of drainage from Whitlow Canyon. Queen Creek then traverses a long narrow stretch of desert area to the vicinity of Chandler Heights where it is joined by drainage diverted by the embankment of the Roosevelt Water Conservation District Main Canal and the drainage of the Santan and Goldfield Mountains area. At the point where Queen Creek Wash leaves Maricopa County, the total drainage area is 711 square miles, including 322 square miles partially diverted into Queen Creek by the Roosevelt Canal embankment.

c. Topography. The basin topography is comprised of mountains, desert lands and irrigated valley lands. The mountains are concentrated in the eastern portion of the basin with minor mountains located on the west side of the valley and at the south extremity. The mountains are rugged and precipitous, rising to nearly 5000 feet elevation. Between the valley and the mountains lie the desert lands which have a general slope of about 30 feet per mile. The elevation rises from about 1300 feet at the lower portion to approximately 2000 feet at the foothills of the principal mountains.

The irrigated valley land slopes to the southwest at 3 to 4 feet per mile. In this area are found the population centers of the basin: Mesa, Chandler, Gilbert and smaller communities.

d. Geology and Soils. The mountains are composed of mainly pre-cambrian schists, granites and quartzites; and tertiary cretaceous volcanic rocks. The soils in the higher mountainous zones are residual but those in lower foothills and adjacent desert lands are alluvial though shallow and poorly developed. The desert lands further down toward the valley are deeper and better developed and underlain by a previous strata. The soils of the valley are deep and well developed, highly pervious and underlain by extensive ground water acquifiers.

e. Stream and Wash Characteristics. The streams and washes in the mountaineous portions of the basin are well defined. All are intermittent, responding to even small amounts of rainfall. Leaving the mountains and entering the desert areas, they become increasingly less defined. The smaller washes disappear entirely after a short distance. The washes through the desert have shallow, gravelly beds where the fine material has washed away. Further down in the valley and in the Gila River flood plain, the washes are choked with debris and shift their courses frequently except where they have been confined and channeled by man.

f. Vegetation. There is no appreciable effect by vegetation on retardation of runoff in the basin, except in the irrigated valley where crops may increase slightly the infiltration rate. Desert shrubs are the dominating plant life in the non-irrigated portions. A few stunted trees are scattered among the shrubs consisting of juniper, paloverde, mesquite, ironwood and salt cedar. Conditions along the larger washes favor the growth of oaks, mesquites, cottonwoods and willows. Perennial grasses form a negligible part of the vegetation present, but good covers of annual grasses occur after winter rains. The winter floods normally occur before the annual grasses are up, and summer floods occur after this cover has disappeared.

g. Maps. The basin is well mapped. Two U.S.G.S. maps of a 1;250,000 scale have been published which cover the basin. These maps are entitled Phoenix and Mesa. They have an index code designation of Nl 12-7 and Nl 12-8, respectively. The contour interval is 200 feet supplemented by contours at 100 foot intervals. The incremental areas of the basin are mapped by U.S.G.S. quadrangle sheets of a 1:24,000 scale. The contour interval is 10 feet except for the Goldfield, Arizona Quadrangle which has a contour interval of 20 feet. These maps are from recent surveys and are adequate for the purpose of this report. The basin is also covered by U.S.G.S. quadrangle sheets of 1:125,000 scale and 1:62,500 scale, having a contour interval of 100 feet and 50 feet, respectively. These maps, however, are from older surveys.

1-2 SCOPE OF STUDIES:

a. The flood problems investigated were confined to those in Maricopa County but flood control measures located outside of the county were studied where these measures were required to provide protection within the county. Field investigations of the study area included surveys of existing drainage structures and channels, stream profiles and damsites. Economic appraisal was made of land and structures in the flood problem areas.

b. The area was inspected by the Engineer and the problems and proposed solutions discussed with representatives of the Maricopa County Flood Control District.

c. The investigation was coordinated with the Soil Conservation Service with particular emphasis on the relation between the portion of the basin being studied by that agency and the remaining study areas under investigation. The investigation was also coordinated with the plans and interests of the

Salt River Water User Association with respect to their irrigation system. The desires and flood problems of other local interests were considered in the formulation of the proposed program of flood control measures.

1-3. ECONOMIC DEVELOPMENT:

a. Population. Based on the 1960 census, Maricopa County has a population of 664,000, of which approximately 100,000 reside within the basin. The population of Maricopa County is increasing rapidly and projections indicate that it will reach 1,440,000 in the next 20 years. Similar growth can be expected within the basin. The principal cities and towns of the basin, Mesa, Chandler and Gilbert, have populations of 33,772, 9,531 and 1,700 respectively. The population of Mesa is expected to be 130,000 by 1980, Chandler, 20,200, and Gilbert, 3,500. These projections are from reports prepared by the Maricopa County Planning and Zoning Commission. Urban complexes are developing along the highways in the vicinity of Mesa. This is the pattern which will evolve in other areas as the road building program, now underway, progresses. In the south end of the valley, more intensive land use is taking place in the form of small fruit ranches and acreages. This development is following the pattern set by Chandler Heights. The ultimate urbanization of the entire valley can be expected.

b. Industry and Resources. The principal occupation of the basin in the past has been agriculture, agriculture-oriented industry and businesses. In Mesa and vicinity, non-agricultural employment now predominates with the greatest increase in employment in the past decade occurring in manufacturing. Many of the residents of this area are employed in the Phoenix area. The smaller population centers remain essentially agricultural communities. The increasing trend toward non-agricultural employment can be expected to continue and agricultural employment to remain steady or decline.

c. Agriculture. The agriculture of the basin is based on irrigation and is therefore highly developed. The principal crop is cotton with important acreages of barley, alfalfa and wheat. Citrus fruit growing is important in the area northeast of Mesa and in Chandler Heights at the south side of the basin. The irrigation development consists of 4 projects: the Salt River Project, Roosevelt Conservation District, Queen Creek Irrigation District and the Chandler Heights Citrus Irrigation District. The Salt River Project irrigation system within the basin supplies water for the land lying between Highline Canal on the west, the Eastern Canal on the east, from the divide running through Mesa to Germann Road. Below Germann Road to the county line, the Consolidated Canal is the east boundary and a smaller area is supplied from this canal extending out irregularly to the west as far as Price Road. The Roosevelt Conservation District Irrigation Project lies adjacent to the Salt River Project with the R.W.C.D. Main Canal forming the eastern boundary, and the basin divide the northern boundary, and Germann Road the southern boundary. The Queen Creek Irrigation District is comprised of lands adjacent to Queen Creek above the Consolidated and Eastern Canals. The Chandler Heights Citrus Irrigation District lies upslope of the Queen Creek Irrigation District in the vicinity of Chandler Heights. Both of these last two projects are supplied by water from wells.

d. Transportation. The basin is served by air and rail facilities and by a network of Federal, State and County highways. The Phoenix Skyharbor Airport, located outside the basin, is the principal air transportation facility serving the area. The mainline of the Southern Pacific Railroad runs through Chandler and Mesa. Three branch lines serve the other areas of the basin. In addition to the existing highways, construction will begin soon on an interstate highway connecting Phoenix with Tucson which will pass through the basin along the west side of the irrigated area. This highway will open new areas of the basin to urban development.

e. Mining. In the mountainous areas of the basin there is some mining activity. Mining is not an economic factor in the portion of the basin to be protected by flood control measures.

1-4. CLIMATOLOGY: The Gila River - Queen Creek basin lies in an arid to semi-arid region, typically desert in character. The winters are short and mild; the summers long and hot. The mean annual rainfall ranges from about 8 inches in the valley in the vicinity of Mesa and Chandler to better than 22 inches in the mountainous area of the northeastern extremity. Weather Bureau precipitation and temperature stations in and adjacent to the basin are shown on Plate No. 2, Appendix I-A.

a. Storms. There are three types of storms that occur in this area:

(1) General winter storms of low intensity covering wide areas and of several days duration.

(2) General summer storms that result from convergence, orographic uplift, or frontal lift.

(3) Local thunderstorms including isolated sporatic showers and cloudbursts that are brought about by insolational heating of tropical maritime air invading the region from the Gulf of Mexico or the Gulf of California and South Pacific.

b. Rainfall. The mean annual rainfall for the basin based on U.S. Weather Bureau stations is 8.10 inches. The maximum annual rainfall during the past 30 years occurred in 1941. The annual rainfall is approximately equally distributed between winter and summer.

c. Snowfall. The average annual snowfall distributed throughout the months of December through March, inclusive, for the Gila River - Queen Creek Basin is less than 1.0 inch. Snowfall has never been a contributing factor to major floods in the basin.

d. Temperature. The normal annual temperature for the basin is 69.8 degrees fahrenheit. The highest recorded temperature during the past 40 years near the basin was 121 degrees at Granite Reef Dam, and the lowest temperature was 13 degrees at Granite Reef Dam.

1-5. RUNOFF AND STREAM - FLOW DATA: The principal stream of the basin, Queen Creek Wash, has runoff and stream-flow records available of limited periods in the vicinity of existing Whitlow Reservoir. The flows from the other areas of the basin are of such nature that rainfall-runoff relations must be relied on for the data for flood peaks and volumes with consideration given to the storage induced by man-made barriers and restrictions.

a. Discharge and Gaging Records. The period of record for the gage at Whitlow damsite is from 1896-97; 1916-20 and 1948 to date. The maximum peak discharge during this period was 42,900 c.f.s. on 19 August 1954. Queen Creek was also gaged near Florence Junction from 1939 to 1941. The maximum discharge here during this interval was 13,200 c.f.s. on 7 August 1939.

b. Run-off Characteristics. The runoff of the basin can be divided into three types, characteristic of the following sources:

(1) Runoff from mountain areas is great and concentrates quickly, producing flash floods of sharp peak discharge. The floods are modified considerably as they pass through the desert areas, having relatively gentle slopes, and are further modified in the valley areas below. Much of the volume of the smaller floods is infiltrated in the desert and valley areas.

(2) Runoff from desert areas approach a sheet-flow condition being carried by numerous washes. The desert areas are essentially bare and tend to "slick-over" except in the washes where the finer material has eroded away leaving a gravelly bed.

Infiltration is relatively small except for the gravelly beds of the washes. Intercepting of this runoff by dike and channel sections such as that on the upside of the R.W.C.D. Main Canal, considerably modifies the peak runoff.

(3) In irrigated areas the infiltration rate and capacity is very great. Only the larger storms produce runoff. What runoff occurs during smaller storms is due to that from roads, compacted soil and roofs.

1-6. FLOODS OF RECORD: The basin has experienced a number of floods which have caused major damage in the valley. Floods occurred in 1926, 1930, 1933, 1936, 1941, 1946, 1954, 1959. Floods of major proportions occurred in 1941, 1946, 1954, 1959. The flood of 1954 is the largest flood of record and is important in that the storm that produced this flood has been used in developing the standard project flood.

a. Storm and Flood of 19 August, 1954. On the morning of 19 August 1954, rain caused flooding beginning east of Mesa and spreading down through the highly developed valley land. Runoff from the Superstition, Goldfield and Usery Mountains and part of the Queen Creek watershed contributed to this flooding. The observer at Boyce Thompson Arboretum reported a total of 5.3 inches of rainfall with an intense period of three hours. Florence Junction reported 1-hour and 6-hour periods of 1.8 and 4.2 inches, respectively. The Corps of Engineers have estimated a peak discharge of 42,900 c.f.s. and a volume of 5,300 acre-feet at the Whitlow Reservoir site. The peak discharge was probably considerably reduced from this in the lower portion of the basin in passing through the desert areas and due to storage induced by man-made restrictions and barriers. However, the total volume is estimated to be about 15,000 acre feet. The isohyet of the 19 August

1954 storm is shown on Plate No. 3, Appendix I-A, which was reproduced by permission of the Los Angeles District Corps of Engineers, from the "Isohyet" shown in the Design Memorandum No. 1, Hydrology for Whitlow Ranch Reservoir, Queen Creek, Arizona - Gila River Basin, Arizona and New Mexico.

1-7. STANDARD PROJECT FLOOD: The standard project flood is the flood that may be expected from the most severe combination of meteorological conditions considered reasonably characteristic of the geographical region involved including extremely rare conditions. This flood represents a standard by which the degree of protection selected for a project can be compared with protection provided at similar projects in other localities.

a. Standard Project Flood Development. The Corps of Engineers in the design of the Whitlow Reservoir, on upper Queen Creek, developed a standard project flood applicable to that watershed. The 19 August 1954 storm was centered over the watershed and appropriate infiltration losses applied to obtain the resultant effective rainfall quantities. A 3 hour preceding rain was assumed which accounted for initial losses. The standard project flood hydrograph was derived by applying the effective rainfall to a unit-hydrograph based on the basin's characteristics and drainage area. With permission of the Los Angeles District Corps of Engineers, the Hydrologic Map showing lines of equal mean-seasonal precipitation in inches; the isohyets, showing total precipitation of 19 August 1954 storm; and the lag relationships curve, taken from the "Design Memorandum No. 1, Hydrology for Whitlow Ranch Reservoir, Queen Creek, Arizona, Gila River Basin, Arizona and New Mexico," were reproduced and are shown as Plates No. 4 through No. 6, in Appendix I-A. This storm and procedure, with certain adjustments, were adopted for the development of standard project floods for the study areas of the basin for which flood control measures are indicated. The rainfall was reduced by 25% to account for the smaller orographic enfluence present in the lower portions of the basin as compared to that in the mountains above Whitlow Dam. The modifying effect of man-made restrictions and barriers on the peak flows of the respective standard project floods was taken into account by inflow-storage-discharge relations. The lag and basin characteristics of the study areas are shown in Table No. 1, Appendix I-A. The Standard project flood hydrographs for each study area is shown as Plate No. 7 through Plate No. 12 in Appendix I-A. Pertinent information on standard project floods for the study areas are shown in the following tabulations:

TABLE NO. 1

STANDARD PROJECT FLOODS - GILA RIVER
QUEEN CREEK BASIN

Study Area No.	Drainage Area (sq. miles)	Standard Project Flood: Peak Inflow (c.f.s.)	Standard Project Flood: Modified Peak (c.f.s.)	Standard Project Flood: Volume (ac.-ft.)
1	34	32,200	16,800	5,080
2	110	24,500	9,700	7,930
3	98	14,400	15,600	7,050
4	123	60,700	28,200	15,400
5*	184	58,700	22,800	19,200
6	30	24,000	7,600	4,480

* Includes drainage area of Study Area #4.

b. Project Design Flood Development. The project design flood applicable to a particular project is based on the degree of protection warranted by the type and character of the flood damages to be prevented or the area to be protected. The study areas of the basin contain both areas of intensive agricultural development and urban development. A different degree of protection is warranted in each case. Consequently, an individual project may require the development of one or more separate design floods of different frequency of occurrence. The considerations which enfluence the development of a project design flood are discussed in the following paragraphs:

(1) Although facilities for the drainage of interior areas of an urban area are customarily designed on a relatively low frequency the actual degree of protection afforded is usually quite great due to the relative elevation of improvements to street elevation and the effect of resultant surface storage present. Flood waters are thereby prevented from reaching damaging elevations. But, when the runoff from large built-over areas concentrate, the effect of surface storage is relatively small and protection is reduced. Therefore, outfalls serving urban areas must be designed to provide a high degree of protection. In general, floods of standard project flood proportions should be reduced to a depth that will not cause loss of life or extensive property damage. The critical elevation in urban areas is usually at or near floor levels above this, damage mounts rapidly. Similar protection from runoff of external sources is required.

(2) In agricultural areas the major portion of the average annual crop damages sustained are made up an accumulation of damage from frequent floods. The objective of flood control measures in these areas is usually prevention of frequent floods with an attendant reduction in the flood damages from the more infrequent floods. In irrigated areas the investment in irrigation facilities such as canals, distribution ditches and land leveling is great. Floods of greater magnitude cause a disproportionate amount of damages to such facilities. In this case, a much higher degree of protection from floods originating from outside the area is warranted. Diversion of runoff from external sources around and away from these

areas may be feasible. The development of project design floods for the various degrees of protection entails the determination of the frequency of occurrence of the floods to be expected in the study areas of the basin. The frequency of flood peaks were based on a correlation of rainfall frequency data with a regional analysis of flood frequencies of natural streams. This was further correlated with the drainage basin characteristics used in the development of standard project floods in order to obtain peak flows for various frequencies of occurrence for each of the study areas of the basin. From these correlations, drainage area v.s. c.f.s. per square mile curves were developed for use in determining the peak flows of incremental areas within the study areas. These curves take into account the shape of the drainage area. The flows developed are those to be expected from natural streams having the drainage area and runoff characteristics of the respective study areas. These peak flows would be modified by man-made restrictions and barriers under present conditions and by the floodway characteristics of the proposed projects. Inflow- storage - discharge relations were applied to these peak flows to obtain modified peak discharges used in the design of the proposed projects.

c. Design Criteria. Based on the degree of flood protection warranted by the type and kind of development requiring protection, the future conditions anticipated, and upon the type and kind of flood control measures under consideration, the following guidelines for project development were established:

(1) Channels serving as outfalls for urban areas should be designed

to contain the 50-year flood and reduce the standard project flood depths to 1-2 feet in urban areas giving due consideration to induced storage. The flood peaks should be based on projected development of the area. The outfall channel of this capacity should be carried to a point approximately 1 mile beyond the limits of future development where it should transition into the channel provided for the area as a whole. Channels serving irrigated areas should contain the 5-year peak flow giving due consideration to peak flows from urban development in the area. In desert areas, channels should contain the 2-year peak flow.

(2) Where floodways and diversions consist of a channel and dike section, as will be the usual case, the dike or embankment should provide protection against overtopping by the standard project flood as modified by induced storage in irrigated agricultural areas. Where urban areas are being protected which envolves loss of life and expensive property, the dike or embankment should have adequate freeboard for protection against overtopping by a flood of spillway design proportions.

(3) Reservoirs should generally be designed to contain a 50 to 100 year flood and modify the standard project flood sufficiently to provide adequate protection in urban areas. The spillway should safely pass the modified standard project flood and have sufficient freeboard that the spillway design flood will not overtop the dam.

(4) The general criteria of culvert design capacity for highways and railroads is a once in 25 year flood; county road, once in 10 year flood, and for multi-lane highways, one in 50 year

flood. Bridges should be designed to pass the 50 year flood with 2 to 5 feet of freeboard, depending on the importance of the road, the consequences of exceeding design capacity, and likelihood of clogging by drift. Existing structures should be altered to provide at least design channel capacity.

1-8 EXTENT AND CHARACTER OF FLOODED AREA

a. General Flood Problem - The principal flood problems of the basin are: The flooding of the valley lands by runoff originating in the mountains and desert areas; and, the increasing runoff from the expanding population centers which already exceeds the capacity of existing outfall drainage ways. With respect to the valley lands, these two flood problems may be classified as flooding from external sources and flooding from internal sources. From external sources there are 36,000 acres of irrigated agricultural land and 800 acres of urban area subject to flooding. The proportion of urban flooding to agricultural flooding will change with future development but not the total. From internal sources, there are about 660 acres of irrigated agricultural land and 70 acres of urban area subject to flooding under present conditions. As urban development proceeds, both the urban and agricultural flooding will increase. It is estimated that by 1980 there would be 2100 acres of agricultural land and 480 acres of urban area subject to flooding; and by the year 2010, 4600 acres of agricultural land and 1850 acres of urban area. The present land use is shown on Plate No. 1, Appendix I-B. The various aspects of the present and future flooded areas are discussed in the following paragraphs.

(1) The principal improvement in the basin is the irrigation system and the on-the-farm facilities connected with it. The value of the portion of this improvement subject to flood hazard is estimated to be $3,500,000. Second in importance is the state and county road system. No accurate estimate can be

made of the value of these facilities, but roads sustain heavy damage from external flooding. The flooded urban area consists of all of Gilbert and the smaller communities in the valley and parts of Chandler and Mesa.

(2) The type of flood damage sustained by the various improvements is comprised of breaching of canals, overtopping of roads, inundation of crops and flooding of houses and buildings with attendant damage to their contents. Direct loss of life due to flood waters has not been reported. However, indirect loss of life due to hazardous road conditions is a distinct possibility during even moderate floods. Floods of standard project flood proportions with the probably collapse of existing dikes and resultant sudden release of impounded water would likely result in loss of life and would aggravate hazardous road conditions.

(3) The expanding population centers of Mesa, Chandler and Gilbert, as shown on Plate No. 1 in Appendix I-B, are within the flood problem area. Some measure of flood protection will be a necessary prerequisite for such expansion even without the comprehensive program under study. However, such measures, based on past experiences of other urban communities, will be largely alleviation measures against frequent floods and the hazard from extreme floods would remain. Urban complexes can be expected to spring up along the new highways in areas where no flood history now exists and these areas will encounter flood problems. These flood problems will arise from the concentration of runoff from the desert areas and from increased runoff from the urban complexes themselves.

b. <u>Flooding in Study Area #1</u> - The irrigated lands below Highline Canal and the western Canal in Study Area #2 are subject to flooding from the Salt River Mountains and the intervening desert located in this area. The new interstate highway from Phoenix to Tucson will pass through Study Area #1. The concentration of water by this highway and the expected urban development will further augment the flood problem.

c. Flooding in Study Area #2 - The lower part of this area is subject to flooding from both internal and external sources. At the upper end of the area is Mesa which as it expands into the valley will increase the flooding from internal sources. Some flooding now occurs on the outskirts of Mesa and Chandler. This flooding is both to urban property from inadequate outfall and to irrigation canals and agricultural lands as the increased runoff seeks avenues of escape. The future expansion of the population centers is the chief concern of this area both from the standpoint of external and internal flooding.

d. Flooding in Study Area #3 - At the present, the flood problem here is the breaching of canal dikes and overtopping of roads by runoff from the Superstition and Usery Mountains. In the lower portion, Queen Creek floods large areas of agricultural land and causes damage to irrigation facilities and county roads. Projected expansion of Mesa and later Chandler and Gilbert into this area will create flood problems from internal sources. Adequate outfalls will be required to carry the increased runoff south to Queen Creek.

e. Flooding in Study Area #4 - Flooding from runoff from the Usery and Superstition Mountains causes damage to the Roosevelt Water Conservation District Main Canal and to state and county roads. In the past, considerable flooding of Williams AFB has occurred. Williams AFB has now been protected against all except extreme floods by use of levees and by improving the outfall along the upside of the R.W.C.D. Main Canal to the outfall at Queen Creek. There are some agricultural lands in the upper end of the area and urban development along the highway from Mesa to Apache Junction which is subject to flood hazard.

f. Flooding in Study Area #5 - This area includes the flood plain of Queen Creek and extensive damage to agricultural land and roads occur. The upslope area on the south, which includes Chandler Heights, is subject to flooding from runoff originating in the Santan and Goldfield Mountains. Over-topping of roads and the erosion and inundation of agricultural lands is an important problem here.

g. Flooding in Study Area #6 - The principal damage due to flooding is that done to Hunt Highway. This area is, however, the source of flooding to areas south of Queen Creek in Study Area #5.

h. Flooding in Study Area #7 - This area is under study by the Soil Conservation Service. Location of retention structures here is expected to control the flooding of the Valley from runoff originating in the Usery and Superstition Mountains.

i. Flooding in Study Area #8 - No flood problems are apparent in this area.

j. Summation of Flood Problems - The extent of flooding and the value of property subject to flooding under present and future conditions in the basin are tabulated in the following table:

TABLE NO. 2

EXTENT OR VALUE OF FLOODED PROPERTY IN THE GILA RIVER-QUEEN CREEK BASIN

PRESENT

Study Area	Urban Property (acres)	Agricultural Land (acres)	Irrigation Facilities (value)	Roads & Streets (value)
#1	0	0	$ 450,000	$ 120,000
#2	860	1,700	140,000	2,300,000
#3	0	28,400	2,360,000	3,700,000
#4	0	600	50,000	70,000
#5	10	6,000	500,000	650,000
#6	0	0	0	160,000
TOTAL	870	36,700	$3,500,000	$ 7,000,000
		FUTURE (1980)		
#1	40	0	$ 450,000	$ 145,000
#2	1130	2,660	220,000	2,940,000
#3	60	28,800	2,400,000	3,850,000
#4	40	700	1,000,000	200,000
#5	20	6,100	630,000	700,000
#6	20	0	0	265,000
TOTAL	1310	38,260	$4,700,000	$ 8,100,000
		FUTURE (2010)		
#1	75	0	$ 450,000	$ 185,000
#2	1960	3,400	180,000	5,350,000
#3	420	29,100	2,420,000	4,750,000
#4	155	1,000	1,400,000	555,000
#5	50	6,300	650,000	845,000
#6	10	0	0	315,000
TOTAL	2700	40,900	$5,100,000	$12,000,000

1-9 FLOOD DAMAGES

a. General - The flood damages caused by past floods of record in the valley have been investigated by the Soil Conservation Service and the Corps of Engineers. The Soil Conservation Service in connection with their present studies in Study Area #1 have estimated that the recurrence of the 1954 flood would cause $3,792,000.00 of agricultural damage in the valley excluding the Queen Creek area. Damage to improvements is also estimated to be nearly equal to this damage. They estimate that the flood of 1959 was almost as damaging. Earlier floods were investigated by this agency in a previous report. A portion of this report entitled "Flood Damages" is shown in Appendix I-B. The Soil Conservation Service estimates of the average annual flood damage from floods originating in the Usery and Superstition Mountains area will be shown in their report. The Corps of Engineers in their studies for the existing Whitlow Reservoir estimated the average annual flood damage on Queen Creek. These economic studies by the Corps of Engineers were reported in the survey report for Whitlow Reservoir. For the other two sources of external flooding; the Santan and Goldfield Mountains area at the south end of the valley; and the Salt River Mountains area on the west side of the valley, no flood history is available. By sample area methods and correlation with the estimates of damages from external flooding from other sources, it is estimated that the average annual damage from flooding from the Santan and Goldfield Mountains area is $57,000, and from the Salt River Mountains area, $51,000. The flooding due to internal sources in the study areas covered by this report and the resultant damages is premised on future urban development. The average annual flood damages from internal flooding, based on the next 50 years of development, are estimated to be $218,000.

b. Summary of Flood Damages - The average annual flood damages for the various study areas of the basin showing breakdown as to source of flooding and the respective types of property are tabulated on the following table:

TABLE NO. 3

PROJECTED (50-YEAR ECONOMIC LIFE) AVERAGE ANNUAL FLOOD DAMAGES IN THE GILA RIVER - QUEEN CREEK BASIN

STUDY AREA	URBAN PROPERTY		AGRICULTURAL		IRRIGATION FACILITIES		ROADS AND STREETS	
	External Flooding	Internal Flooding	External Flooding	Internal Flooding	External Flooding	Internal Flooding	External Flooding	Internal Flooding
#1	$ 4,500	$ 3,000	$ 0	$ 0	$ 4,000	$ 0	$ 2,400	$ 700
#2	173,000	77,000	48,000	43,200	2,000	2,400	46,000	26,900
#3	9,200	16,400	797,000	8,800	21,700	400	74,000	7,900
#4	6,000	6,000	16,800	5,100	9,100	0	1,400	4,000
#5	3,000	2,100	168,200	3,900	5,700	300	13,000	1,700
#6	3,300	700	0	0	0	0	3,200	1,200
TOTAL	$199,000	$ 105,200	$1,030,000	$61,000	$42,500	$3,100	$140,000	$42,400

1-10 OTHER FLOOD CONTROL PROJECTS AND STUDIES

a. Soil Conservation Service - This agency has under study a system of detention reservoirs designed to intercept runoff from the Usery and Superstition Mountains. These reservoirs extend from the basin divide on the north along about the 1600 contour to the Queen Creek divide on the south. Most will be in Pinal County but are intended for protection of the valley against external floods. A report is expected to be completed early in 1962.

b. Corps of Engineers - The existing Whitlow Reservoir was planned and constructed by the Corps of Engineers. Extensive studies of the flood problem on Queen Creek have been made of Queen Creek flooding in the report on this reservoir.

c. Ground Water Recharge Study - A feasibility study was made of ground water recharge of flood waters and is made a part of this report. The objective of this study was the investigation of the subsurface geology of the basin and possible methods for recharging of water collected by detention reservoirs to determine the feasibility of conserving flood waters by this means. The report of this study is shown as "Proposed Artificial Recharge Aspects of Flood Control Survey for Southeastern Maricopa County", Appendix I-C.

1-11 IMPROVEMENTS DESIRED

a. General - The principal objective of the flood control study in the basin is the protection of the highly developed valley lands from external flooding and the providing of adequate outfall channels for the increasing runoff from the expanding population centers located therein. The Salt River Users Association have expressed the desire that all flood water be carried by a separate system and not allowed to flow into their irrigation canals. One of the specifications for this report made by the Maricopa County Flood Control District was that multi-purpose use of reservoirs be made where feasible. To this end, ground water recharge of flood waters has been incorporated as a project purpose in connection with all proposed detention reservoirs.

1-12 SOLUTIONS CONSIDERED AND PLAN OF IMPROVEMENT -

a. Flood Problems and Possible Solutions - As discussed under "1-8 EXTENT AND CHARACTER OF FLOODED AREA," the overall flood problem of the basin is the flooding of the valley lands from external sources and increasing internal flooding as a result of higher runoff rates due to expanding population centers. The indicated solution for the prevention of floods from external sources is the storage of flood waters by dams and the diversion of flood flows by dike and channel around the protected area to natural drainage ways. The principal natural drainage way is Queen Creek. The indicated solution for internal flooding is to provide an adequate system of outfall channels to carry flood flows to the Gila River. Other possible solutions such as: flood plain zoning, filling of areas to be built over, evacuation and resettlement, and levees, cannot alone or in combination provide the overall protection required for the basin. This is true principally because of the economic considerations involved but is also true because of the impracticability of effecting some of these measures and anticipating future trends of development; however, these measures may be important adjuncts to the proposed plan in the future. This is especially true of internal flood problems both in the protected areas and in the areas where flood control measures are installed. It should, however, be recognized that it is unrealistic to assume that restrictions imposed by flood plain zoning now, will remain inviolate. The pressure of the problems and needs of future generations can and will alter such restrictions and regulations. Instead, it is more realistic to set forth the problems faced by development in such areas and to establish criteria which govern successful development. Flood plain areas and reservoir areas can be utilized, subject to economic considerations, provided sound engineering concepts are followed.

b. Existing Channels - There is no clearly defined drainage system serving the valley area. The irrigation canals carry some flood waters and the upside of the canal dikes intercept and divert flood water along the canals toward Queen Creek and the Gila River. Queen Creek is the principal drainage course but the canal at the lower end of the valley near West Chandler, now used as a waste-way, also carries flood waters to the Gila River. The channel of Queen Creek has been enlarged and improved, in the past, from a point in the vicinity of the town of Queen Creek to approximately State Highway No. 87. In connection with the protection of Williams AFB, the dike on the upside of the R.W.C.D. Main Canal has been raised and an improved channel constructed from Williams AFB to Queen Creek.

c. Relationship Between Irrigation Canals and Drainage - Drainage channels and the irrigation canals are highly compatible. They may utilize a common dike on the upslope side of the canal. The gentle slope required for irrigation canals is also desirable for drainage channels. At times the irrigation canals may beneficially accept some flood waters. The principal problem is where drainage must cross a canal. This is not insurmountable; however, such crossings are costly and are to be avoided where possible.

d. Multi-purpose Use -Conservation of water is the principal concern in this area. In connection with detention reservoirs, water stored may be used for ground water recharge provided adequate facilities are planned and constructed. As previously mentioned, the irrigation canals may accept some water but this is a very limited use of flood waters. Full conservation will require a well co-ordinated system of ground water recharge facilities.

e. Plan of Improvement. The projects which constitute the plan of improvement for the basin are shown on the Basin Project Map in Appendix I-D. Included in the plan of improvement is the proposed Whitlow Canyon Reservoir, SCS structures, and the projects for the various study areas. The proposed Whitlow Canyon Reservoir is a long-range proposal and consequently no cost estimate or monetary economic evaluation was made. The SCS structures, as previously mentioned, will be covered by a report by that agency. Pertinent data for Whitlow Canyon Reservoir, retention dams and the proposed dikes and channels are shown in Tables 1 and 2 in the "Hydrologic and Hydraulic Analyses", Appendix I-A. The projects for the Study Areas No. 1 through No. 6, and Whitlow Canyon Reservoir are discussed below:

(1) Study Area No. 1 project consists of two retention reservoirs operating in conjunction with dike and diversion channels. Retention Reservoir No. 1 would receive runoff collected by a dike and diversion channel located along the south boundary of the area. Retention Reservoir No. 2 would receive water from a dike along the east side of the Salt River Mountains. The discharge from this reservoir would be carried by the dike and diversion channel to Retention Reservoir No. 1. The project would provide protection from external flooding to the irrigated lands below the Highline Canal down to the proposed outfall channel in Study Area No. 2 and the irrigated lands below Study Area No. 1 in the Gila River Indian Reservation. It would also provide protection for the area between the

new Interstate Highway and the Salt River Mountains in which urban development is expected to proceed rapidly following completion of the highway. The diversion dike along the base of the Salt River Mountains in conjunction with the diversion resulting from construction of the Interstate Highway and that of the embankment on the upslope side of the existing Highline Canal will provide a high degree of protection for the irrigated lands in Study Area No. 2 which now experience flooding from runoff originating in the Salt River Mountains.

(2) Study Area No. 2 project consists of a system of outfall channels for the population centers of Mesa, Chandler and Gilbert. It is sized for future expansion of these communities and the expansion of Tempe into the valley. The outfall channels for Mesa and Gilbert are sized to carry the 50-year flood to a point at approximately the center of Sec. 9, R5E, T1S, Gila and Salt River Base Line, where they join and then transition into the outfall channel serving the area as a whole. The Mesa and Gilbert outfall channel would reduce the Standard Project flood to where only one to two feet of flooding would occur. A similar outfall channel is provided for Chandler to Dobson Road where it transitions into a smaller channel which joins the main outfall channel at a point where it enters the Gila River Indian Reservation. The main channel, which is designed to contain the 5-year flood, assuming ultimate development, should be constructed initially to a point just past the new interstate highway. The extension of this channel to the Gila River could be delayed until urban development progresses further. Right-of-way should be obtained under the initial program. The porposed project provides a basic outfall channel system. Laterals and the usual drainage facilities to tie into this system would be required in connection with the expansion of the population centers.

(3) Study Area No. 3 project consists of dike and diversion channels along the upside of the Consolidated Canal and the upside of the R.W.C.D. Main Canal to outfalls at Queen Creek. These dike and diversion channels are intended to provide protection against external flooding for Study Area No. 2 which originate in

this area. They also would serve as outfall channels as urban development expands into this area. The channel along the upside of the Consolidated Canal is sized on the basis of the anticipated urban development expected to occur in the foreseeable future.

(4) Study Area No. 4 and No. 5 projects consist of: A dike and diversion channel along the upside of the Roosevelt Canal to Queen Creek; a dike and diversion channel along the upside of the planned Central Arizona Project Aquaduct to Queen Creek, approximately in line with the SCS structures. The dike and diversion channel on the upside of the Roosevelt Canal would provide protection against external flooding for the valley areas below. It would also serve as an outfall channel for Williams AFB and future urban developments. The dike and diversion channel on the upside of the planned Central Arizona Project Aquaduct would divert flood waters from the uncontrolled area below the SCS structures to Queen Creek. The channel would serve as a means of collecting and diverting reservoir flows released from the SCS structures to suitable ground water recharge areas. The detention and debris barrier reservoir on Queen Creek would store flood waters from the uncontrolled area below the existing Whitlow Reservoir and the releases from that reservoir. The releases from the detention and debris barrier reservoir would be handled by ground water recharge facilities. The reservoir would also intercept the debris scoured from the Queen Creek channel in the valley areas below. As the coarse debris collected into a delta in the reservoir, it would form a natural ground water recharge element.

(5) Study Area No. 6 project consists of a detention reservoir operating in conjunction with a dike and diversion channel. The dike and diversion channel would lie along the upside of Hunt Highway and would divert the flood waters from the Santan and Goldfield Mountain area to the reservoir for storage and subsequent induction by ground water recharge facilities. This project is intended for the protection of the area in the vicinity of Chandler Heights down to Queen Creek against

external flooding. The dike could be incorporated into a raised road section for Hunt Highway with mutual advantages to both functions.

(6) The Whitlow Canyon Reservoir, in conjunction with the existing Whitlow Reservoir, to be constructed in the future, would establish a high degree of control of flooding from mountain areas of the Queen Creek basin. As the debris delta, mentioned in preceeding Paragraph (4), builds up and begins to function as a natural ground water recharge element, then releases from these two reservoirs could be inducted directly without appreciable storage. The flood control storage remaining after depletion of the debris load would then still be adequate to accommodate flood flows from the uncontrolled area below the proposed Whitlow Canyon Reservoir and the existing Whitlow Reservoir. This is a long-range project, but one that is likely to prove highly desirable in the future.

1-13 COST ESTIMATES

a. Estimates of First Cost - The estimated first cost of the proposed plan of improvement, exclusive of SCS structures, is $11,000,000. The cost of SCS structures and their required right-of-way cost will be covered by that agency in a separate report. The estimates of first cost by study areas are summarized in the following table:

Table No. 4

ESTIMATES OF FIRST COST

Item	First Cost by Study Areas					
	#1	#2	#3	#4	#5	#6
Right-of-Way						
Lands & Damages	$ 707,000	$ 686,000	$ 768,000	$ 387,000	$ 855,000	$ 272,000
Relocations:						
Railroads	0	0	0	0	0	0
Highways	0	0	0	0	0	0
Utilities	2,200	1,800	2,400	0	1,600	1,200
Miscellaneous						
Structures	17,000	442,000	390,000	190,000	125,000	9,000
Dams	500,000	0	0	0	793,000	82,000
Channels & Dikes	110,000	1,294,000	980,000	201,000	200,000	420,000
Engineering & Design	62,920	173,780	137,240	39,100	111,960	51,220
Contingencies @ 20%	125,840	347,560	274,480	78,200	223,920	102,440
Total First Cost	$1,524,960	$2,945,140	$2,552,120	$ 895,300	$2,310,480	$ 937,860

b. Estimates of Annual Cost - The estimated annual costs for construction of the proposed plan of improvement are itemized by Study Areas in the following table:

Table No. 5

ESTIMATED ANNUAL COSTS - 50-year Economic Life

Item	Annual Cost by Study Areas					
	#1	#2	#3	#4	#5	#6
Interest and Amortization	$ 59,180	$ 114,464	$ 99,188	$ 34,796	$ 89,797	$ 36,450
Operation and Maintenance	8,250	6,850	4,500	850	10,930	3,820
Total Annual Charges	$ 67,430	$ 121,314	$ 103,688	$ 35,646	$ 100,727	$ 40,270

1-14 ESTIMATED BENEFITS

a. Benefits - Tangible benefits estimated to accrue to the plan of improvement include reduction in flood damages, future development and increased income from changes in land utilization. There would be other benefits of an intangible nature, not susceptible to monetary evaluation. Benefits are summarized below:

(1) Benefits from flood damage reduction consist of the estimated reduction of average annual damage to crops and structures. These benefits, exclusive of those that accrue to the SCS structures, amount to $467,200 annually, of which $218,000 are crop benefits and $249,200 are structural benefits. The Soil Conservation Service will estimate the average annual benefit in flood damage reduction due to their structures.

(2) Future development in the flood problem area of the basin during the next 50 years is estimated to result in an increase in the value of structures of $76,600,000, without protection from floods. These increases are based on population trends, past and future agricultural production, and construction trends in the region under study and smilar areas. On the basis of these increased values, it is estimated that the benefits from reduction in flood damages to future development in the area would amount to $497,700 annually.

(3) Increased land utilization will result from the flood protection afforded by the proposed projects for the basin. Initially this will consist of a change in agricultural use, but eventually a change in land use to urban development will result. Benefits from this item were measured by the increase in net income to be expected from improvements in the flood problem area. The net increase in value of improvements is estimated to amount to $9,900,000 in the next 50 years. The estimated benefit from increased land utilization would amount to $374,800 annually.

(4) Intangible benefits not susceptible to evaluation in terms of average monetary value would be realized through the development of the proposed plan of

improvement. Such benefits would include the reduction in interrupted transportation and delay in harvesting crops, enhancement of the general welfare and security of the people, and a lessening of the hazard of epidemics. These benefits would be real and of significant importance in the problem area.

b. Summary of Tangible Benefits - Estimated average annual benefits for the plan of improvement by Study Areas are summarized in the following table:

Table No. - 6

ESTIMATED AVERAGE ANNUAL BENEFITS

	Study Area					
	#1	#2	#3	#4	#5	#6
Flood losses prevented -	$ 9,200 (1)	$256,800	$ 60,800	$26,400	$114,000	$ (2)
Future development -	7,500 (1)	220,200	210,200	32,800	27,000	(2)
Increased land utilization -	44,800 (1)	52,100	41,300	50,000	186,600	(2)
Total	$61,500	$529,100	$312,300	$109,200	$327,600	

Note: (1) Additional benefits of project are to areas in Study Area #2.
(2) Benefits of project are to areas in Study Area #5.

1-15 FORMULATION OF FLOOD CONTROL PLAN

a. General - The proposed plan of improvement is a comprehensive program of flood control measures designed to provide flood protection for the basin and to provide a basic system of drainage channels to meet present needs and the needs of future urban development of the area. Close co-ordination of the various elements is required to obtain a sound functional project. Of immediate urgency is the need for protection from external flooding. Almost equally important is the need for a system of drainage channels for internal drainage and for uncontrolled areas to insure that the benefits from detention reservoirs and diversion are fully realized.

b. Co-operation with Federal Programs - Implementation of the proposed plan of improvement will require close co-operation with Federal programs to effect maximum economy and to obtain the most effective flood control. The usual requirements for local participation in Soil Conservation Service projects must be met as will be set forth in the report being prepared by that agency. Specific opportunities for co-operation with Federal programs to effect economy and better flood control are discussed in the following paragraphs.

(1) The Soil Conservation Service structures are a prerequisite to the installation of other flood control measures in the area to be protected on the east side of the valley. At the present time, they tentively plan a closed conduit from the structures down to the drainage channel on the upside of the R.W.C.D. Main Canal in the vicinity of Williams AFB, to safely carry the reservoir releases to an outfall. By providing a collection or diversion channel to carry these releases along the contour to areas where recharge facilities can be located, this costly conduit can be eliminated and at the same time water conserved. Close co-ordination of construction of the collection and diversion channel with the construction of the SCS structures would be required to effect this economy.

(2) The Bureau of Reclamation plans an irrigation canal, designated on the Basin Project Map as the Central Arizona Project Canal. By constructing the

proposed dike and diversion channel in conjunction with the construction of this canal, savings can be effected by an overall balance of earth quantities and in eliminating drainage structures otherwise required. Construction of the proposed Queen Creek Detention and Debris Barrier Reservoir will reduce the size of the drainage structure required under the Bureau of Reclamation canal. Co-ordination of construction schedules would be required to make this possible.

c. <u>Design and Construction of Non-Federal Program</u> - The projects which comprise the plan of improvement for the Non-Federal Program fall into three categories, as follows:

(1) Projects required to extend the control and protection against external flooding, provided by the Federal Program of SCS structures and the existing Whitlow Reservoir, to the entire valley.

(2) Projects required to insure benefits afforded by the flood control measures against external flooding are fully realized, now and in the future.

(3) Projects required to provide protection against internal flooding due to increased runoff from expanding population centers.

The specific projects which make up the Non-Federal Program grouped according to the foregoing categories and in that order, are discussed in the following paragraphs:

(4) The projects in category (1) are:

(a) The proposed Queen Creek Detention and Debris Barrier Reservoir operating in conjunction with the SCS structures would complete the protection from external flooding from the east side of the basin. This project would cost $1,499,000. The annual charges would be $73,248 as compared to the estimated annual benefits of $349,100. The resultant benefit/cost ratio is 4.76 to 1.

(b) The proposed Hunt Highway dike and diversion channel with the detention reservoir, in Study Area #6, are the components of the project required to provide protection from external flooding from the Santan and Goldfield Mountain area. This project would cost $937,860. The annual charges would be $40,270 as compared to the estimated annual benefits of $74,000. The benefit/cost ratio is 1.84 to 1.

(c) In the Salt River Mountain area, Study Area #1, the proposed dike and diversion channel operating in conjunction with the proposed detention reservoir would provide protection from external flooding on the west side of the basin. This project would cost $1,524,960. The annual charges would be $67,430 as compared to the estimated annual benefits of $174,700. The benefit/cost ratio is 2.59 to 1.

(5) The projects in category (2) are:

(a) The proposed dike and diversion channel on the upslope side of the Central Arizona Project Aquaduct will divert flood flows from the uncontrolled area below the SCS structures to Queen Creek and will serve as a collection channel for carrying reservoir releases to recharge facilities in the vicinity of Queen Creek. This project would cost $487,000. The annual charges would be $18,357 as compared to the estimated annual benefits of $109,200, giving a benefit/cost ratio of 5.95 to 1.

(b) The proposed dike and diversion channel on the upslope side of the R.W.C.D. Main Canal would divert flood flows from the large uncontrolled desert area, lying between the R.W.C.D. Main Canal and the Central Arizona Project Aquaduct, to Queen Creek. This project would cost $1,706,780. The annual charges would be $70,184 as compared to the estimated annual benefits of $104,100. The benefit/cost ratio would be 1.48 to 1.

(c) The proposed dikes and diversion channels, in Study Area #3, would divert flood flows from uncontrolled areas to outfall at Queen Creek. The uncontrolled areas, now agricultural, can be expected to be partially build-over in the foreseeable future. The cost of this project would be $2,552,120. The annual charges would be $103,688 as compared to estimated annual benefits of $208,200. This gives a benefit/cost ratio of 2.01 to 1.

(6) The project in category (3) is:

(a) The proposed system of outfall channels, Study Area #2, would provide protection from internal flooding from increased runoff from the expanding population centers of Mesa, Chandler and Gilbert, and carry flood flows from this area to the Gila River. The cost of this project would be $2,945,160. The annual charges would be $121,314 as compared to the estimated annual benefits of $426,800. The benefit/cost ratio is 3.52 to 1.

d. Recommendations for Acquisition of Existing Flood Control Facilities - The acquisition of the following existing flood control facilities is recommended:

(1) The waste-way channel from the vicinity of West Chandler to the Gila River should be acquired. This facility is held at present by the Salt River Water Users Association but the possibility of it reverting to former owners in the immediate future is imminent.

(2) All of the Queen Creek channel in Maricopa County including access right-of-way, should be obtained. The proper care and maintainance of this important drainage way will have a direct bearing on the effectiveness of the proposed flood control measures.

(3) The improved dike and diversion channel from a point in the vicinity of Williams AFB to Queen Creek is needed.

e. Master Plan For Future Development - As discussed under "I-12 SOLUTIONS CONSIDERED AND PLAN OF DEVELOPMENT," a detailed master plan is not practical. However, guide-lines for future development in flood plains and reservoir areas can be established which will realistically control such development. Guide-lines can also be established to set forth the drainage facilities that should be provided with any development in the uncontrolled areas of the basin. These guide-lines are discussed in the following paragraphs.

(1) Any improvements can safely be constructed in the flood plain adjacent

Table No. - 7

SUMMARY OF RECOMMENDED PROGRAM

Project	Classification	Cost	B/C Ratio
SCS structures	urgent	*	*
Outfall channel system in Study Area No. 2	urgent	2,945,160	3.52:1
Queen Creek detention and debris barrier dam	urgent	1,499,000	4.76:1
Detention reservoir and dike and diversion channel in Study Area No. 6	immediate	937,860	1.84:1
Detention reservoir and dike and diversion channel in Study Area No. 1	immediate	1,524,960	2.59:1
Dike and diversion channel upside of Roosevelt Canal	immediate	1,706,780	1.48:1
Dike and diversion channels in Study Area No. 3	immediate	2,552,120	2.01:1
Extension of outfall channel to Gila River - Study Area No. 2	long range		
Whitlow Canyon Reservoir	long range		

* The cost of S.C.S. structures and Flood Control District r/w cost will be covered by the Agency in a separate report.

to the dike and channel diversions, outfall channels and in reservoir areas provided the minimum floor elevation or critical damage elevation is no more than 1 to 2 feet below the water surface elevation of the Standard Project Flood. The water surface elevation of the Standard Project Flood will normally be at approximately the top of dike or 2 to 3 feet below the top of dam. Dikes protecting urban areas will be from 2 to 3 feet above the Standard Project Flood water surface elevation. Areas may be filled to attain the required minimum floor elevations or critical damage elevations provided the fill is borrowed in and adjacent to the channel or, in reservoir areas, from the effective flood control pool area.

(2) Where extensive development is undertaken in the unprotected desert areas, dike and diversion channels should be provided upslope of the development to collect and divert external flood flows to suitable drainage ways. Similar dike and channels should be provided below the development to collect the increased flows in order that development below will not be adversely affected.

(3) Development in the protected areas of the valley should have adequate internal drainage facilities to carry flood flows to the proposed outfall channel system.

f. Summary of Recommended Program - The projects that make up the recommended program of flood control for Study Area No. I, Southeastern Maricopa County, the Queen Creek-Gila River Basin, are summarized in the following table:

FLOOD CONTROL SURVEY AND REPORT

STUDY AREA NO. I

SOUTHEASTERN MARICOPA COUNTY

APPENDIX I-A

HYDROLOGIC AND HYDRAULIC ANALYSIS

FLOOD CONTROL DISTRICT

MARICOPA COUNTY

Prepared By

BENHAM ENGINEERING COMPANY, INC.

CONSULTING ENGINEERS

PHOENIX, ARIZONA

APPENDIX I-A

HYDROLOGIC AND HYDRAULIC ANALYSIS

TABLE OF CONTENTS

TABLES

PLATES

PLATES

PLATES

APPENDIX I-A

HYDROLOGIC AND HYDRAULIC ANALYSIS

INTRODUCTION

1. General - The purpose of this appendix is to present the basic hydrologic and hydraulic data used in the development of the flood control report for the basin. A further purpose is the discussion of methods used in the hydrologic and hydraulic analysis.

DETERMINATION OF PROJECT DESIGN FLOODS

2. General - The basin and its streams are of such nature that synthetic methods must be used in the determination of project design flood flows. This involves correlation of rainfall data and frequencies with runoff characteristics of drainage areas, regional analysis of stream flood flow data for similar streams and areas, and synthetic unit-hydrograph methods. A further consideration is the effect of man-made barriers on flood flows. Dikes, bridges and culverts, and highway and railroad fills induce storage modifying the peaks of flood flows.

3. Standard Project Floods - The synthetic unit-hydrograph method, as used by the Corps of Engineers in development of the standard project flood for Whitlow Reservoir on Queen Creek, was adapted for determination of the standard project floods applicable to the various study areas of the basin. The unit-hydrograph method takes into account the shape of the area and the slope of the terrain as basin characteristics. The basin characteristics determined for the various study areas are shown in Table No. 1 of this appendix. Corrections for the differences in annual rainfall and for the differences in infiltration rates for desert and irrigated lands as compared to mountain areas, such as that above Whitlow Reservoir, were required. The resultant standard project flood hydrographs, are shown on Plates No. 7 through No. 12, are those

to be expected from natural streams. The effect of ~~existing man~~-made barriers when taken into account by use of inflow-storage-discharge relations result in the modified hydrographs as shown on the aforementioned plates. A further modification could be expected in connection with the proposed flood control program. This, however, will eventually be offset by the building over of large areas of the basin.

4. Project Design Floods - The flood flows used as the basis of design on a project are based on the magnitude of the flood to be expected from a given area and its frequency of occurrence. The degree of protection necessary is related to the type of property to be protected and the type and amount of flood damages. Consequently, values for flood peaks of several different frequencies may be required. Flood frequencies and magnitudes were determined by first correlating rainfall intensity frequency data obtained from Weather Bureau Publication No. 28, with the run-off characteristics of each of the study areas of the basin. These rainfall runoff frequency relations were further correlated with the standard project flood applicable to each study area to obtain the frequency of floods of various magnitudes. Design criteria was established for the various types of flood control measures contemplated. To facilitate the determination of design flows for incremental areas within the study areas, "drainage area verses c.f.s. per square mile" curves were developed which take into account the shape of the drainage area. The values determined from these curves were adjusted by inflow-storage-discharge relations to reflect the flow characteristics of the flood control measures under consideration as compared to natural streams. These design flows for incremental areas are shown in "Pertinent Data - Channels", Table No. 2.

Table No. 1

Gila River - Queen Creek Basin

Lag and Characteristics of Study Areas of Basin

Study Area	Drainage Area (sq. miles)	L (miles)	Lca (miles)	S (ft./mi.)	L.Lca / S-1/2	Lag* (hours)
1	34	6.4	3.0	62	2.4	1.5
2	110	14.4	7.2	6	42.3	4.6
3	98	22.8	11.4	4	130.0	7.0
4	123	11.6	4.8	30	10.2	2.7
5	184**	17.0	11.5	19	44.7	4.7
6	30	9.5	5.0	66	5.8	2.2

* See Pl. 6 for lag curve

** Includes area of Area 4

Table No. 2

Pertinent Data - Channels By Study Areas

Reach	Length (feet)	Bottom Width (feet)	Depth (feet)	Dike Height (feet)	Design Discharge (c.f.s.)	S.P.F. Discharge (c.f.s.)
Study Area No. 1						
1 (to R.D.(#2)	4,000	10	4-0	8-12	200-2yr.	1,400
2 (to R.D.(#2)	13,000	10	0	12	260-2yr.	3,600
3 (to R.D.(#1)	6,000	-20	5	7	260-2yr.	1,900
4 (to R.D.(#1)	10,600	20	5-3	7-6	580-2yr.	4,600
5 (from west)	10,600	10	5	7	360-2yr.	3,400
Study Area No. 2						
1-Mesa lateral	9,250	50-100	8	3	2200-50yr.	4,000
2-Mesa lateral	7,950	100	8	3	4000-50yr.	7,500
1-Gilbert lateral	15,850	25	6	3	600-50yr.	1,100
1-Main channel	2,640	100-30	8	6	1600-5yr.	5,600
2-Main channel	10,600	30	8	6	1600-5yr.	9,700
1-Chandler lateral	5,300	25	3-6	3	600-50yr.	1,100
2-Chandler lateral	7,950	25	6-8	3	1600-50yr.	3,000
3-Chandler lateral	2,640	25-10	8	3-6	900-5yr.	3,300
4-Chandler lateral	25,100	10	8	6	900-5yr.	3,600
3-Main channel	31,700	30	8	6	1600-5yr.	9,700

Study Area No. 3

Reach	Length (feet) (Data)	Bottom Width (feet)	Depth (feet)	Dike Height (feet)	Design Discharge (c.f.s.)	S.P.F. Discharge (c.f.s.)
Consolidated Canal:						
1	10,000	15	5	3	330-5yr.	1,100
2	15,300	15-35	5	3-4	600-5yr.	2,000
3	18,500	35-40	5	4	660-5yr.*	2,200
4	24,600	40	5	4-5	600-5yr.	2,400
5	29,100	40	5	5	600-5yr.	3,000
6	10,600	40	5	5	600-5yr.	3,400

(*Urban development)

Reach	Length (feet) (Data)	Bottom Width (feet)	Depth (feet)	Dike Height (feet)	Design Discharge (c.f.s.)	S.P.F. Discharge (c.f.s.)
Eastern Canal:						
1	19,000	10	3	4	80-5yr.	1,500
2	14,800	10-15	3	4	110-5yr.	2,000
3	16,400	15-20	3	4-5	170-5yr.	2,900
4	26,400	20	3-4	5	210-5yr.	3,600
5	44,000	20	4	5	240-5yr.	4,200

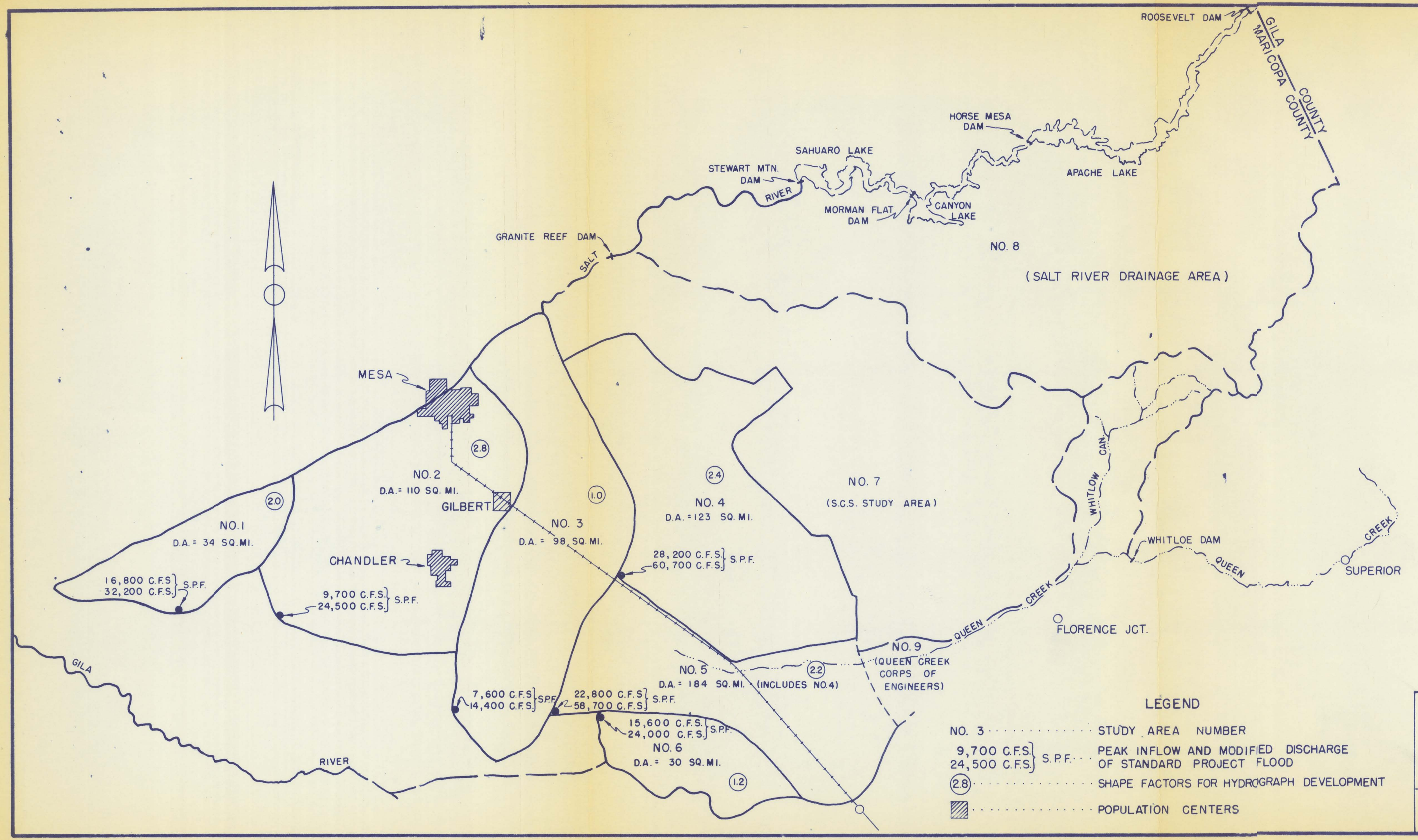
Study Area Nos. 4 & 5

Reach	Length (feet) Data	Bottom Width (feet)	Depth (feet)	Dike Height (feet)	Design Discharge (c.f.s.)	S.P.F. Discharge (c.f.s.)
Roosevelt Canals:						
1	10,600	15	4	4	190-2yr.	1,570
2	14,800	15-25	4-5	4-5	470-2yr.	2,000
3	14,800	25-40	5	5-6	650-2yr.	3,300
4	27,000	40-65	5	6-8	1,000-2yr.	4,700
5	31,700	65	5	8	1,000-2yr.	4,700
Central Arizona Project Aquaduct:						
1	22,700	15	3	3	110-2yr.	520
2	40,700	15-20	3-4	3-4	250-2yr.	780
3	27,000	20	4	4	250-2yr.	780
4	22,700	20	4-5	4-8	360-2yr.	3,000
5 (from south)	19,000	20	5	4-8	360-2yr.	3,000
Study Area No. 6						
1	7,400	20	5	8	600-2yr.	4,400
2	12,150	20	5	8	680-2yr.	4,600
3	15,800	20-30	5	8	800-2yr.	5,400
4	18,500	30	5	8	1,600-2yr.	11,000

Table No. 3

Pertinent Data - Dentention Reservoirs in Non-Federal Program

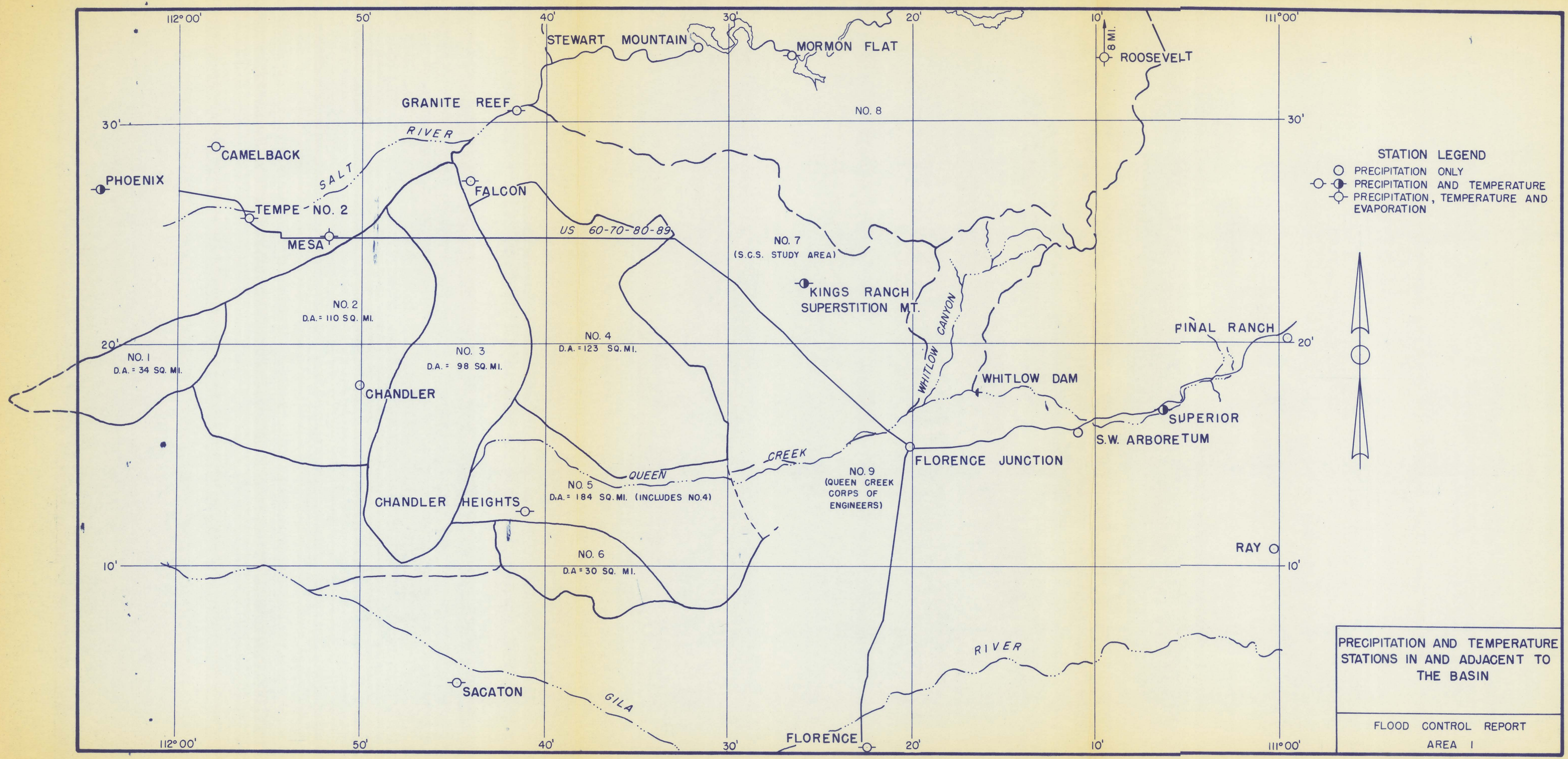
Data	Reservoir: Queen Creek	Study Area No. 1 RD #1	Study Area No. 1 RD #2	Study Area No. 6	Whitlow Canyon
Drainage Area in Sq. Miles	262.0	26	8	30	38.7
Type of Dam	Earthfill	Earthfill	Earthfill	Earthfill	Earthfill
Height of Dam	33'	35'	30'	45'	96'
Approximate Length	21,000	29,200	1,100	6,000	758
Volume of Fill	1,640,000	1,008,541	70,000	300,000	635,807 cy
Flood Control Storage -					
Capacity	15,980	6,000	2,000	4,500	5,888
Area	1,273 Ac.	550	200	230	128 Ac.
Elevations	1,595	1,153	1,290	1,433	2,094
Spillway Capacity					
Primary	524	70	20	60	80
Auxiliary	27,000 c.f.s.	34,000 c.f.s.	8,000 c.f.s.	30,000 c.f.s.	12,000 c.f.s.



BASIN INDEX MAP

FLOOD CONTROL REPORT
AREA I

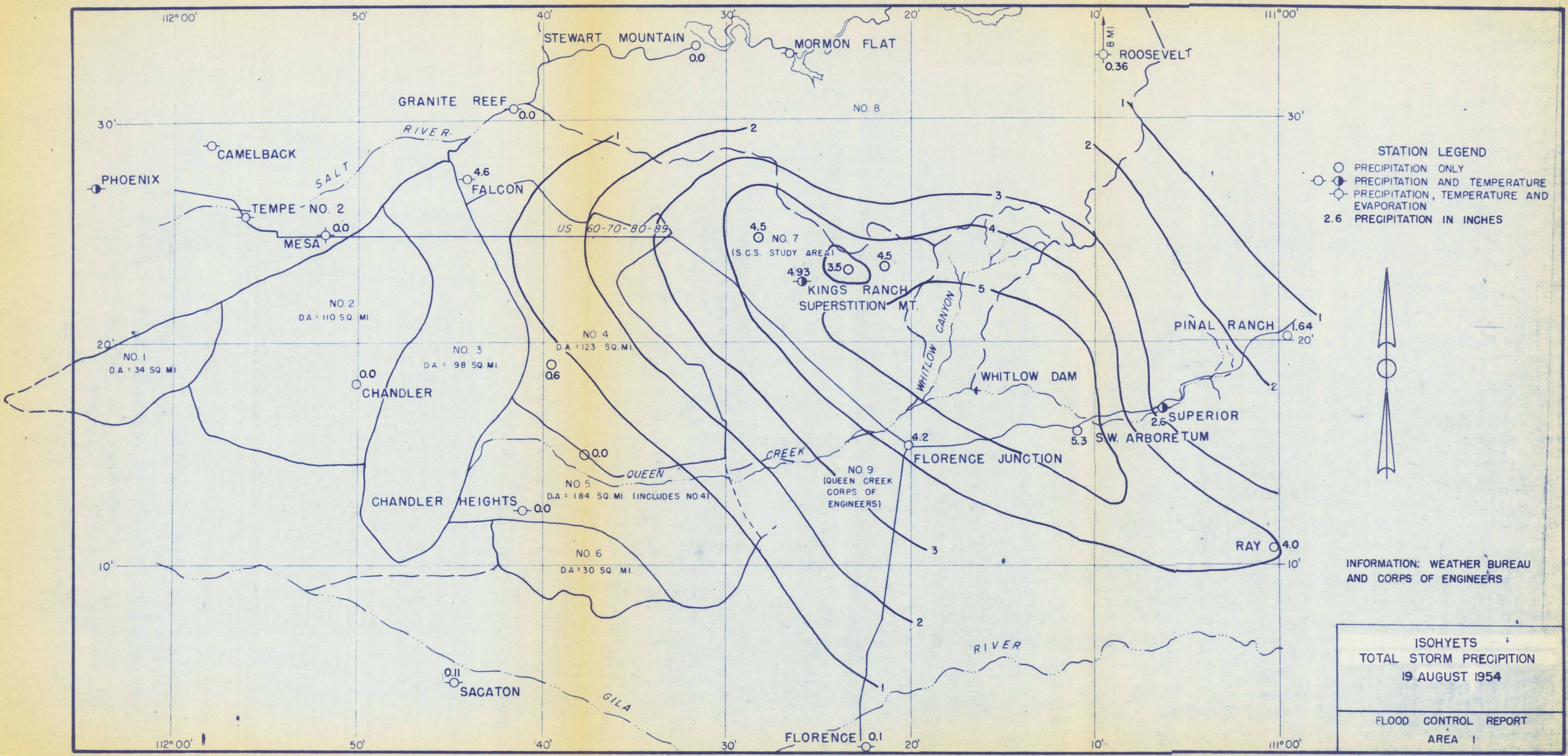
APPENDIX - A, PLATE NO. I



PRECIPITATION AND TEMPERATURE STATIONS IN AND ADJACENT TO THE BASIN

FLOOD CONTROL REPORT
AREA I

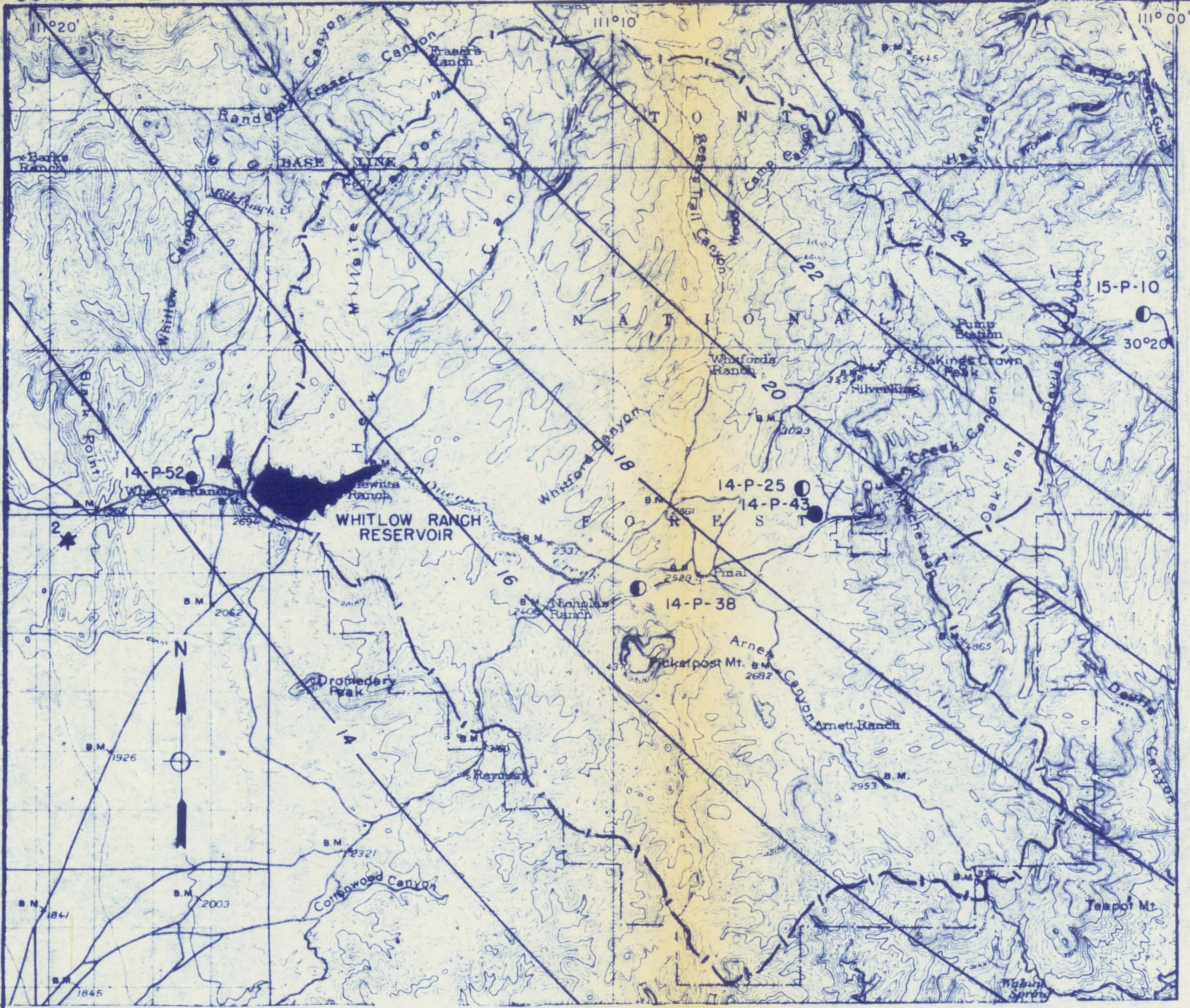
APPENDIX – A, PLATE NO. 2



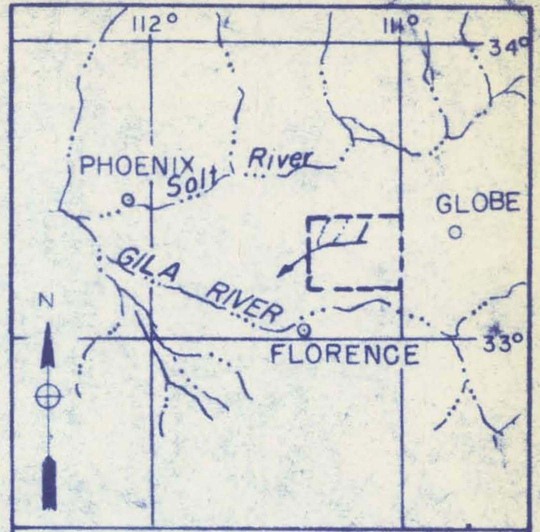
APPENDIX - A PLATE NO. 3

CORPS OF ENGINEERS

U. S. ARMY



SCALE 0 1 2 3 4 5 MILES



VICINITY MAP

SCALE 0 10 20 30 40 50 MILES

AREA COVERED BY MAP

LEGEND

BOUNDARY OF PROJECT DRAINAGE AREA.

14 LINE OF EQUAL MEAN-SEASONAL PRECIPITATION IN INCHES.

PROJECT FLOOD-CONTROL BASIN.

14-P-52 PRECIPITATION STATION AND NUMBER (RECORDING).

14-P-25 PRECIPITATION STATION AND NUMBER (NON-RECORDING).

1 STREAM-GAGING STATION AND NUMBER.

2 STREAM-GAGING STATION AND NUMBER (DISCONTINUED)

NOTE:

ISOHYETS BASED ON GILA RIVER BASIN PRECIPITATION RECORDS (1868-1939)
CONTOUR INTERVAL 100 FEET.
DATUM IS MEAN SEA LEVEL.

GILA RIVER BASIN, ARIZ., & N. MEX.
QUEEN CREEK, ARIZ.

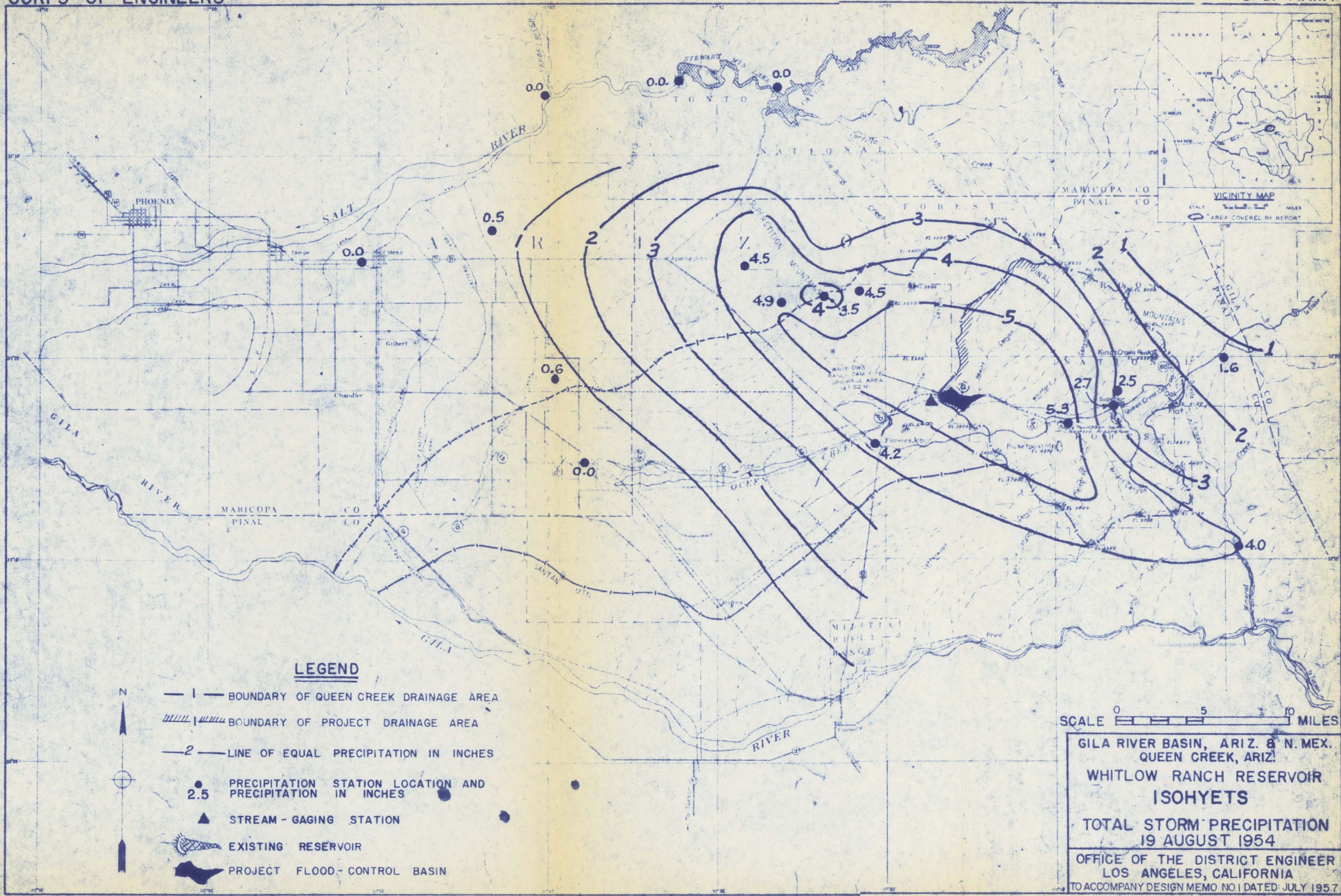
WHITLOW RANCH RESERVOIR

HYDROLOGIC MAP

OFFICE OF THE DISTRICT ENGINEER
LOS ANGELES, CALIFORNIA
TO ACCOMPANY DESIGN MEMO NO. I DATED JULY 1957

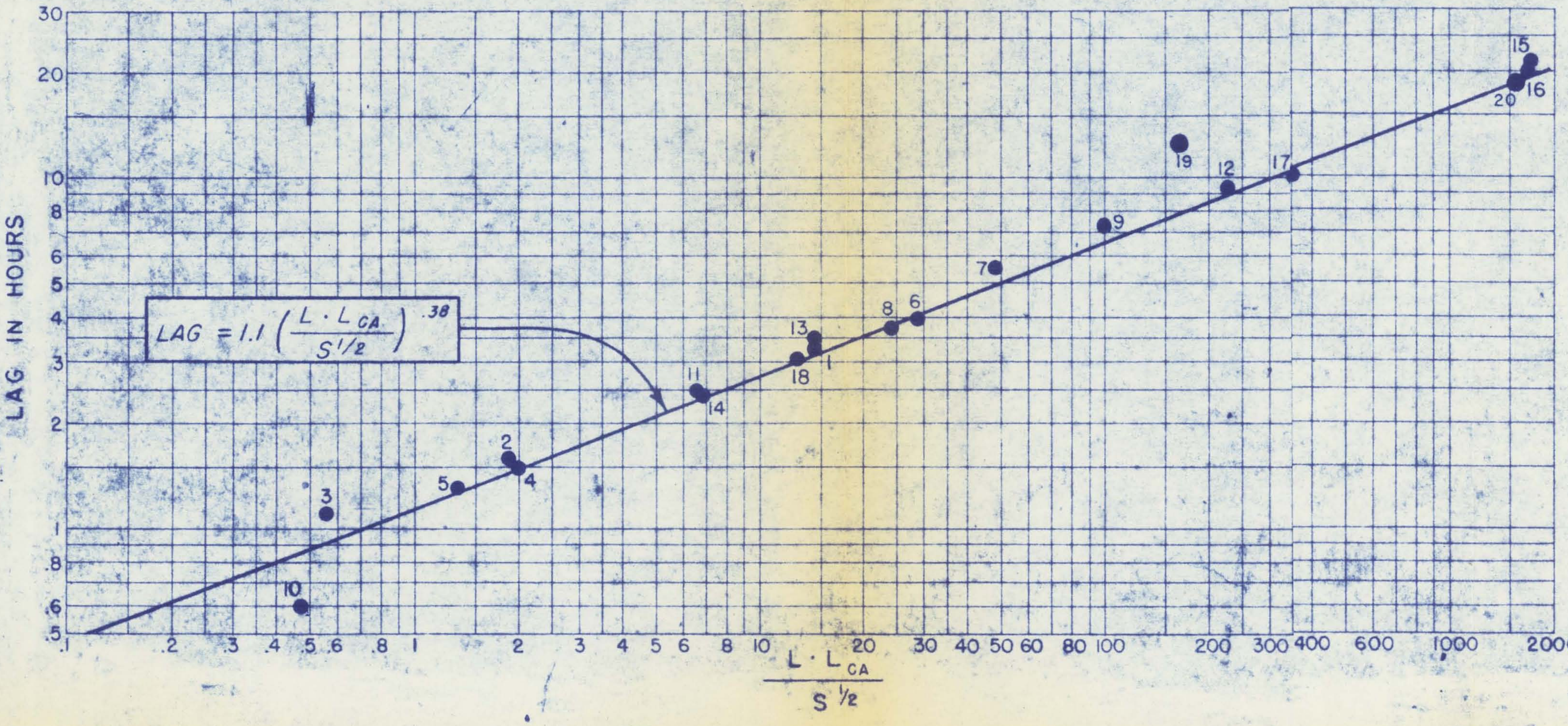
Incl 1

CORPS OF ENGINEERS U.S. ARMY



APPENDIX - A PLATE NO 5

	DRAINAGE AREA SQ. MI.	L MILES	L_{CA} MILES	S FT/MI.	LAG HOURS
1. SAN GABRIEL RIVER AT SAN GABRIEL DAM, CALIF.*	162	23.2	11.6	350	3.3
2. WEST FORK SAN GABRIEL RIVER AT COGSWELL DAM, CALIF.	40.4	9.3	4.2	450	1.6
3. SANTA ANITA CREEK AT SANTA ANITA DAM, CALIF.	10.8	5.8	2.5	690	1.1
4. SAN DIMAS CREEK AT SAN DIMAS DAM, CALIF.	16.2	8.6	4.8	440	1.5
5. EATON WASH AT EATON WASH DAM, CALIF.	9.5	7.3	4.4	600	1.3
6. MURRIETA CREEK AT TEMECULA, CALIF.	220	27.2	10.3	95	4.0
7. SANTA CLARA RIVER NEAR SAUGUS, CALIF.	355	36.0	15.8	140	5.6
8. TEMECULA CREEK AT PAUBA CANYON, CALIF.**	168	26.0	11.3	150	3.7
9. SANTA MARGARITA RIVER NEAR FALLBROOK, CALIF.	645	46.0	22.0	105	7.3
10. EAST FULLERTON CREEK AT FULLERTON DAM, CALIF.	3.1	3.2	1.7	140	0.6
11. TUJUNGA CREEK AT BIG TUJUNGA DAM NO. 1, CALIF.	81.4	15.1	7.3	290	2.5
12. SANTA MARGARITA RIVER AT YSIDORA, CALIF.	740	61.2	34.3	85	9.5
13. LOS ANGELES RIVER AT SEPULVEDA DAM, CALIF.	152	19.0	9.0	145	3.5
14. PACOIMA WASH AT PACOIMA DAM, CALIF.	27.8	15.0	8.0	315	2.4
15. GILA RIVER AT CONNER NO. 4 DAM SITE, ARIZONA	2840	131	71	29	21.5
16. SAN FRANCISCO RIVER AT JUNCTION WITH BLUE RIVER, ARIZONA***	2000	130	74	32	20.6
17. BLUE RIVER NEAR CLIFTON, ARIZONA***	790	77	37	65	10.3
18. SAN VICENTE CREEK AT FOSTER, CALIF.	75	18.2	7.4	111	3.2
19. SANTA ANA RIVER AT PRADO DAM, CALIF.#	1,466	68	26	115	13.0
20. SALT RIVER NEAR ROOSEVELT, ARIZ.	4,310	160	66	45	18.6



TERMINOLOGY

L = LENGTH OF LONGEST WATERCOURSE.

L_{CA} = LENGTH OF LONGEST WATERCOURSE, MEASURED UPSTREAM, TO POINT OPPOSITE CENTER OF AREA.

S = OVER-ALL SLOPE OF DRAINAGE AREA BETWEEN HEADWATERS AND COLLECTION POINT.

LAG = ELAPSED TIME FROM BEGINNING OF UNIT RAINFALL TO INSTANT THAT SUMMATION HYDROGRAPH REACHES 50% OF ULTIMATE DISCHARGE.

* EXCLUDES AREA ABOVE COGSWELL DAM.

** PALOMAR MOUNTAIN PORTION. ENTIRE AREA IS 319 SQUARE MILES, OF WHICH 151 SQUARE MILES DID NOT CONTRIBUTE APPRECIABLE FLOOD FLOWS DURING THE FLOODS ANALYZED.

*** UNIT-GRAPH STUDY BASED ON RUNOFF RECORDS FOR SAN FRANCISCO RIVER AT CLIFTON (DRAINAGE AREA = 2790 SQ. MI.)

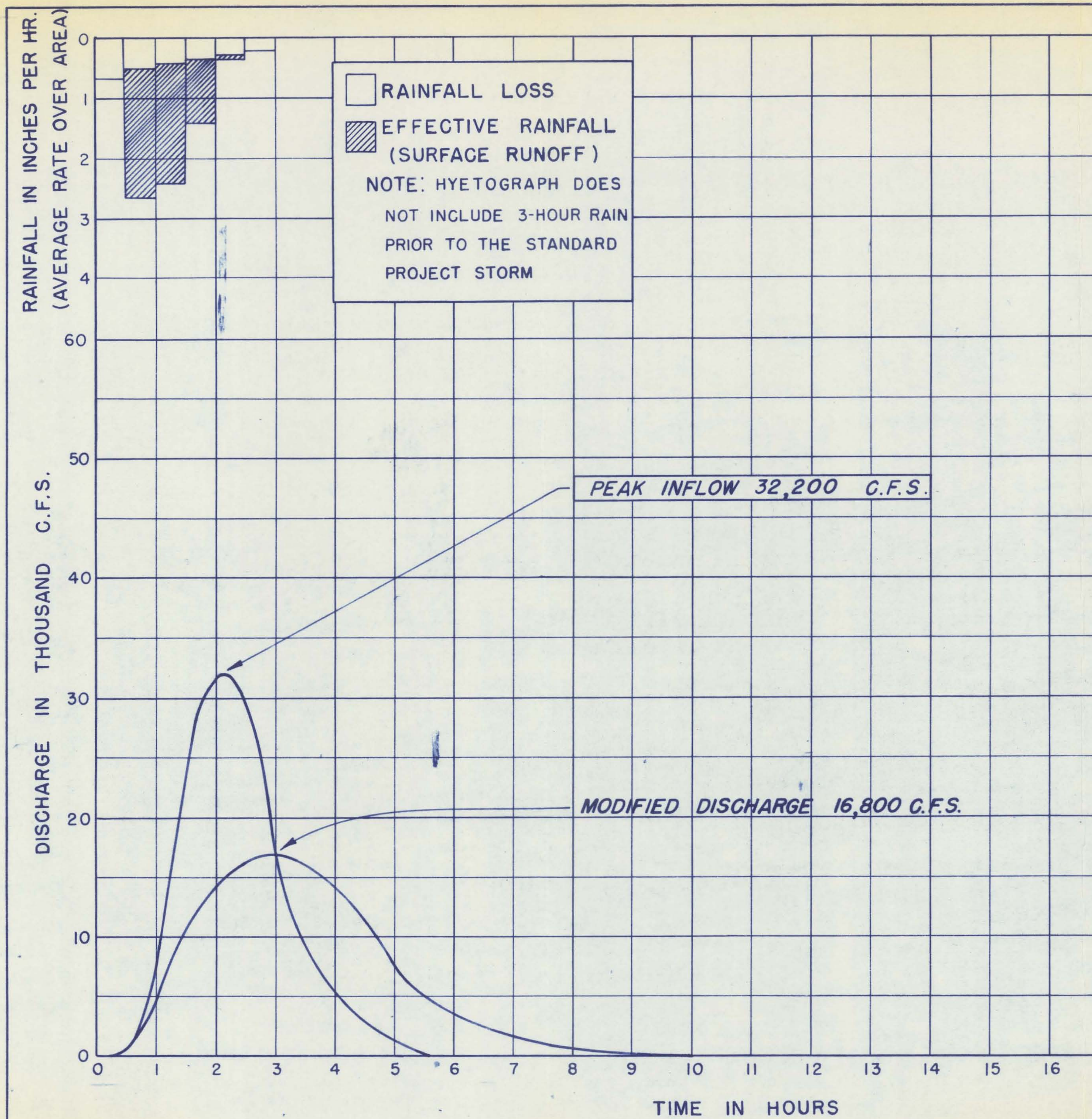
\# EXCLUDES 25 SQUARE MILES TRIBUTARY TO BALDWIN LAKE AND 767 SQUARE MILES TRIBUTARY TO LAKE ELSINORE.

GILA RIVER BASIN, ARIZONA AND NEW MEXICO
QUEEN CREEK, ARIZONA
WHITLOW RANCH RESERVOIR

LAG RELATIONSHIPS

OFFICE OF THE DISTRICT ENGINEER
LOS ANGELES, CALIFORNIA

TO ACCOMPANY DESIGN MEMO NO. 1 DATED: JULY 1957



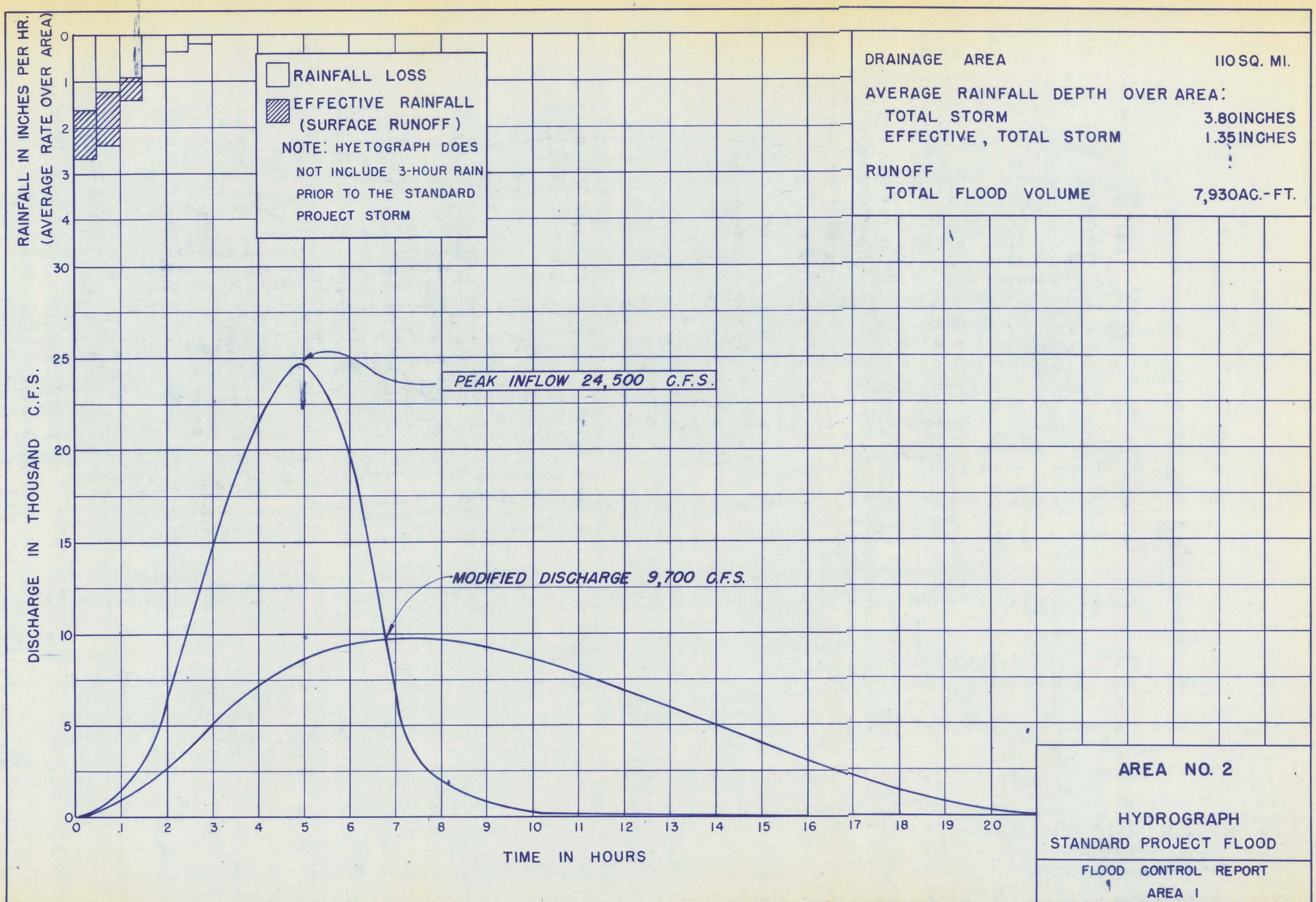
DRAINAGE AREA	34 SQ. MI.
AVERAGE RAINFALL DEPTH OVER AREA:	
TOTAL STORM	3.80 INCHES
EFFECTIVE, TOTAL STORM	2.80 INCHES
RUNOFF	
TOTAL FLOOD VOLUME	5,080 AC.-FT.

AREA NO. I

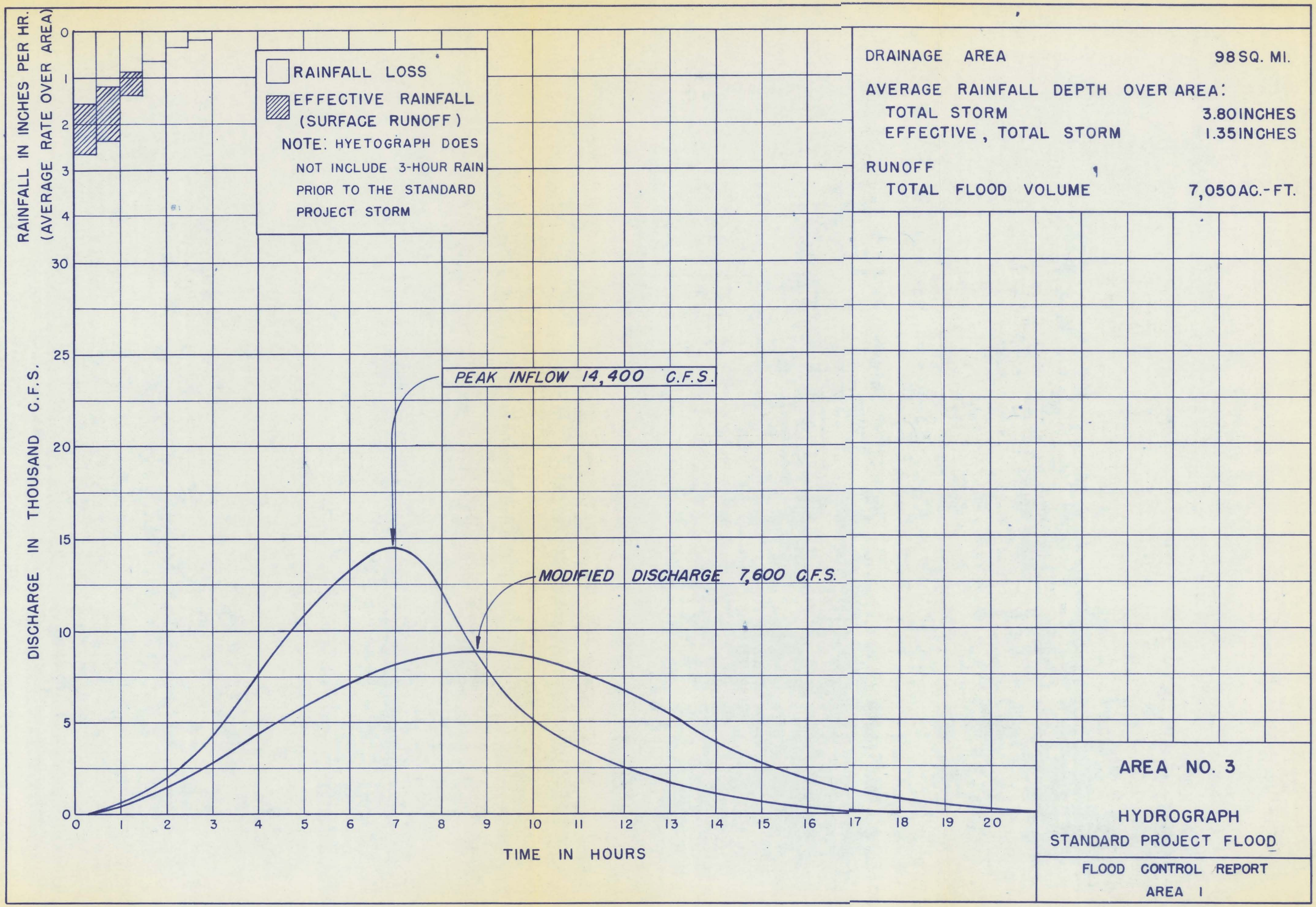
HYDROGRAPH

STANDARD PROJECT FLOOD

FLOOD CONTROL REPORT
AREA I



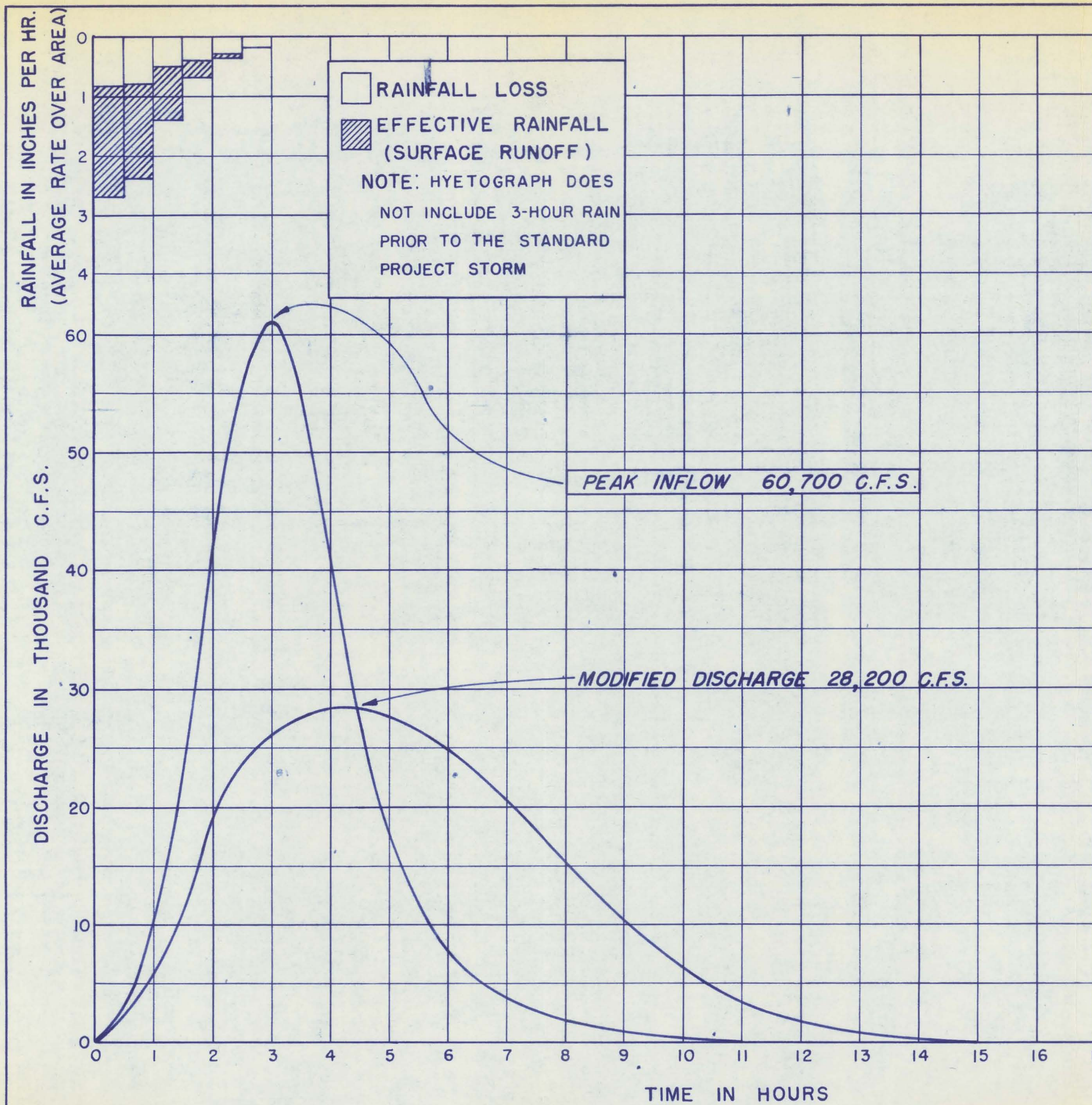
APPENDIX-A, PLATE NO. 8



AREA NO. 3

HYDROGRAPH
STANDARD PROJECT FLOOD

FLOOD CONTROL REPORT
AREA I



DRAINAGE AREA 123 SQ. MI.

AVERAGE RAINFALL DEPTH OVER AREA:

TOTAL STORM 3.80 INCHES

EFFECTIVE, TOTAL STORM 2.35 INCHES

RUNOFF

TOTAL FLOOD VOLUME 15,400 AC.-FT.

AREA NO. 4

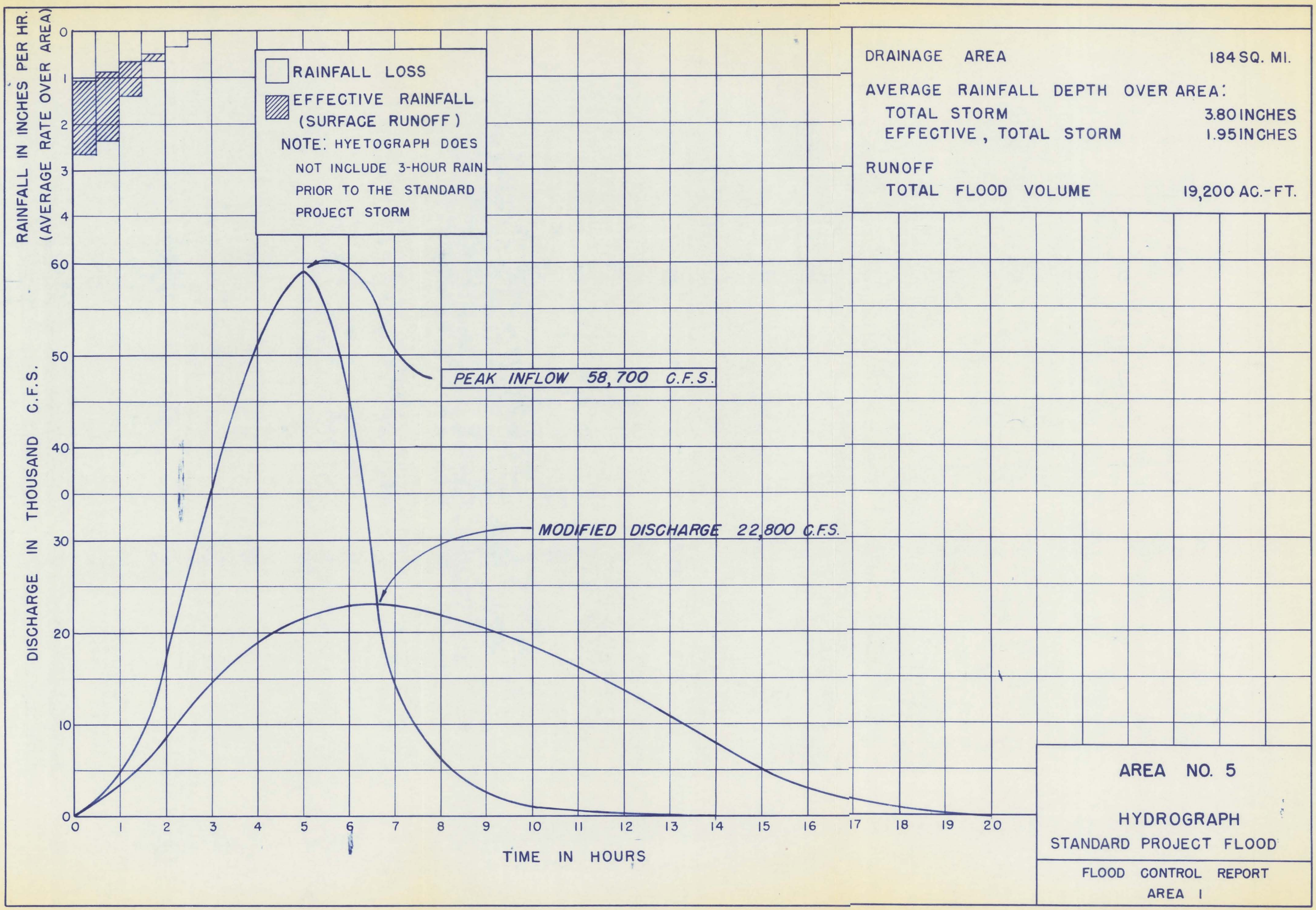
HYDROGRAPH

STANDARD PROJECT FLOOD

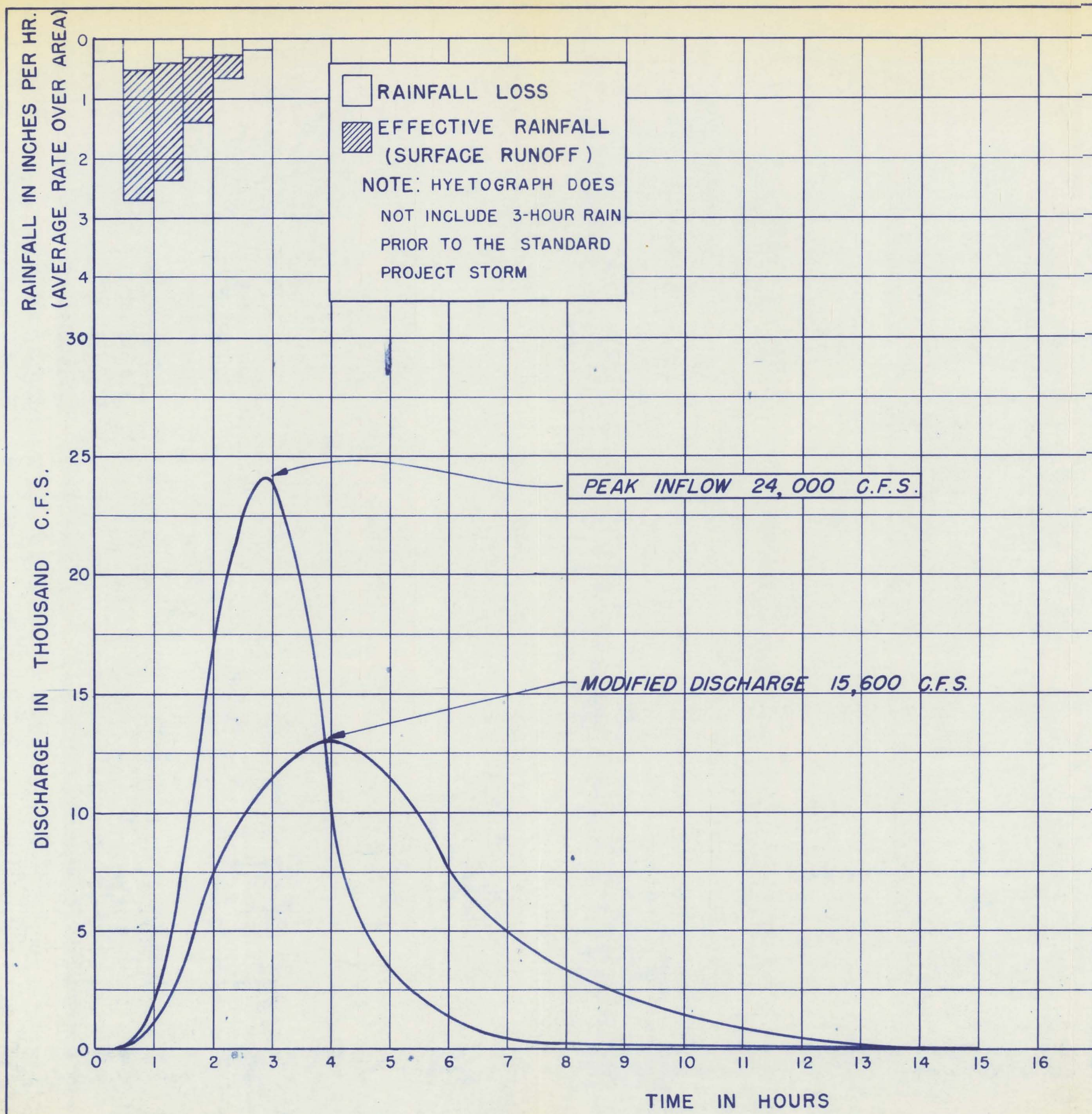
FLOOD CONTROL REPORT

AREA I

APPENDIX - A, PLATE NO. 10



RAINFALL IN INCHES PER HR.
(AVERAGE RATE OVER AREA)
0
1
2
3
4
RAINFALL LOSS
EFFECTIVE RAINFALL
(SURFACE RUNOFF)
NOTE: HYETOGRAPH DOES NOT INCLUDE 3-HOUR RAIN PRIOR TO THE STANDARD PROJECT STORM
DRAINAGE AREA 184 SQ. MI.
AVERAGE RAINFALL DEPTH OVER AREA:
TOTAL STORM 3.80 INCHES
EFFECTIVE, TOTAL STORM 1.95 INCHES
RUNOFF
TOTAL FLOOD VOLUME 19,200 AC.-FT.
DISCHARGE IN THOUSAND C.F.S.
60
50
40
0
30
20
10
0
PEAK INFLOW 58,700 C.F.S.
MODIFIED DISCHARGE 22,800 C.F.S.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
TIME IN HOURS
AREA NO. 5
HYDROGRAPH
STANDARD PROJECT FLOOD
FLOOD CONTROL REPORT
AREA 1

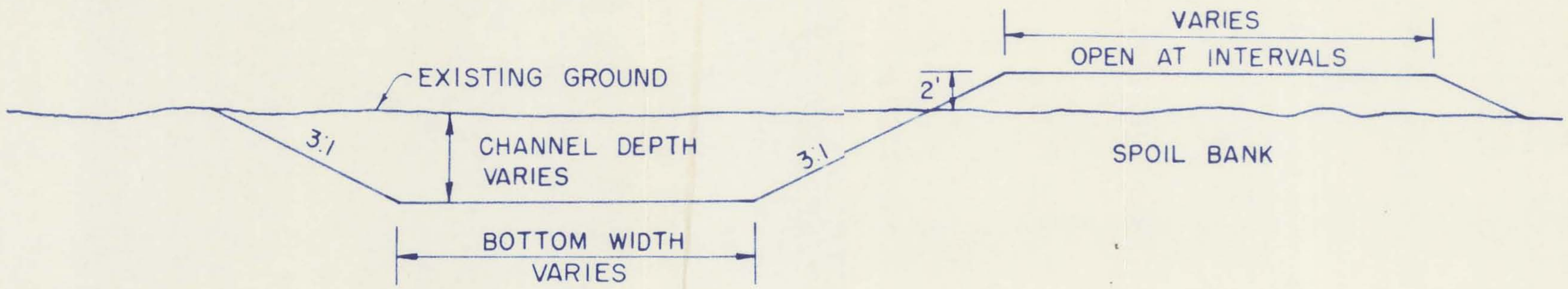


DRAINAGE AREA	30 SQ. MI.
AVERAGE RAINFALL DEPTH OVER AREA:	
TOTAL STORM	3.80 INCHES
EFFECTIVE, TOTAL STORM	2.80 INCHES
RUNOFF	
TOTAL FLOOD VOLUME	4,480 AC.-FT.

AREA NO. 6

HYDROGRAPH
STANDARD PROJECT FLOOD

FLOOD CONTROL REPORT
AREA I

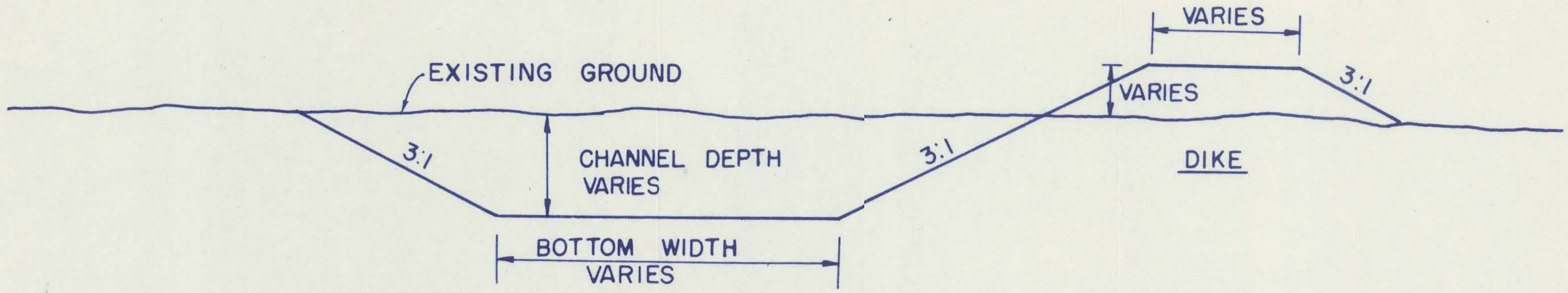


TYPICAL CHANNEL SECTION

SEE PLAN FOR DIMENSIONS

TYPICAL CHANNEL SECTION
FLOODWAY

FLOOD CONTROL REPORT
AREA I

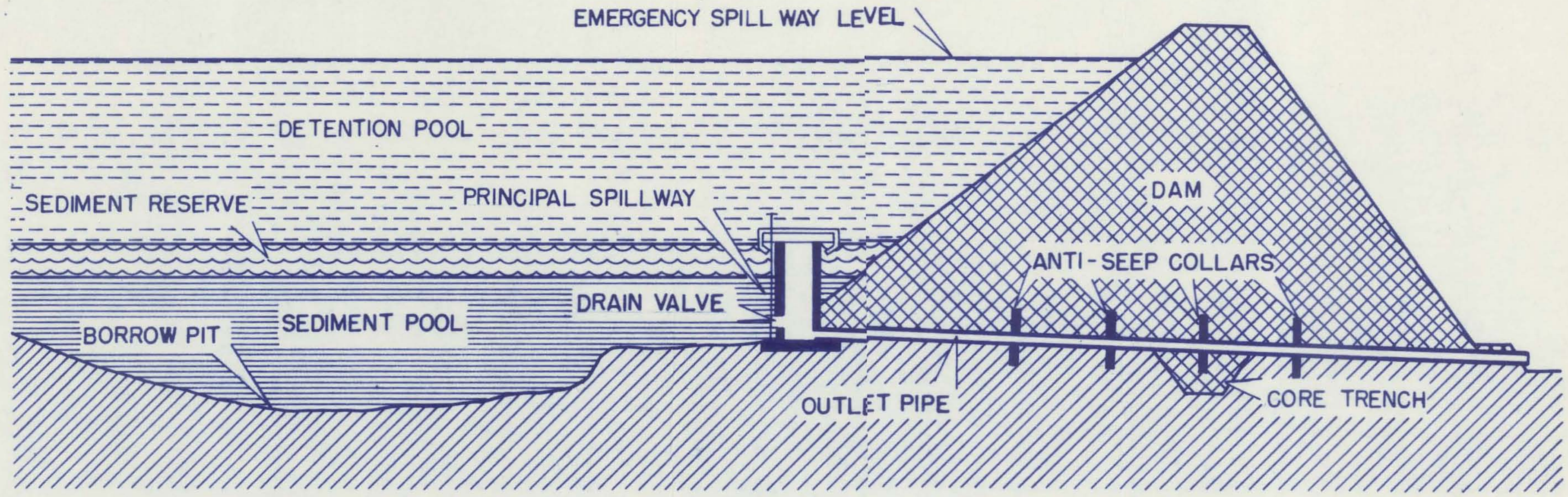


TYPICAL CHANNEL SECTION

SEE PLAN FOR DIMENSIONS

TYPICAL CHANNEL AND DIKE SECTION

FLOOD CONTROL REPORT
AREA I

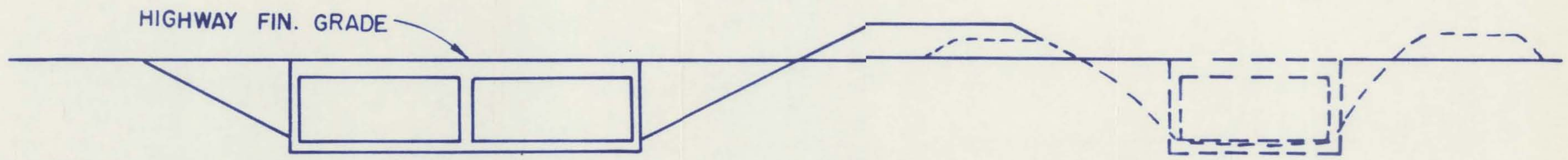


SECTION OF A TYPICAL FLOODWATER RETARDING STRUCTURE

U.S. DEPARTMENT OF AGICULTURE, SOIL CONSERVATION SERVICE

TYPICAL FLOODWATER
RETARDING STRUCTURE

FLOOD CONTROL REPORT
AREA I



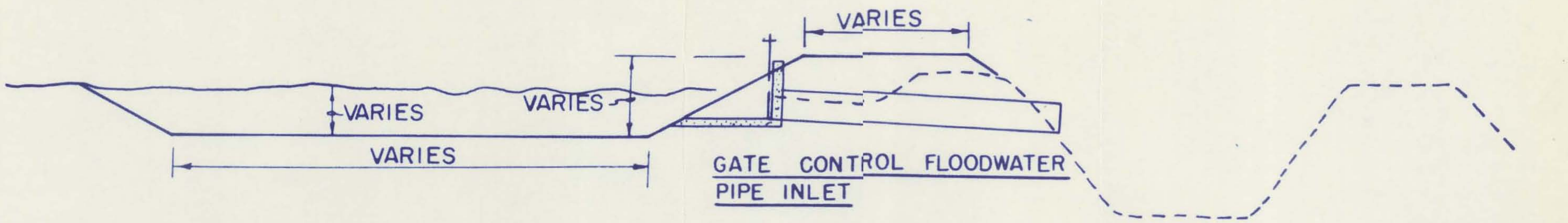
FLOODWAY STRUCTURE
DIMENSIONS & OPENING, VARIABLE

EXISTING CANAL STRUCTURE

TYPICAL COUNTRY ROAD CROSSING

FLOOD CONTROL REPORT
AREA I

APPENDIX A, PLATE NO. 16

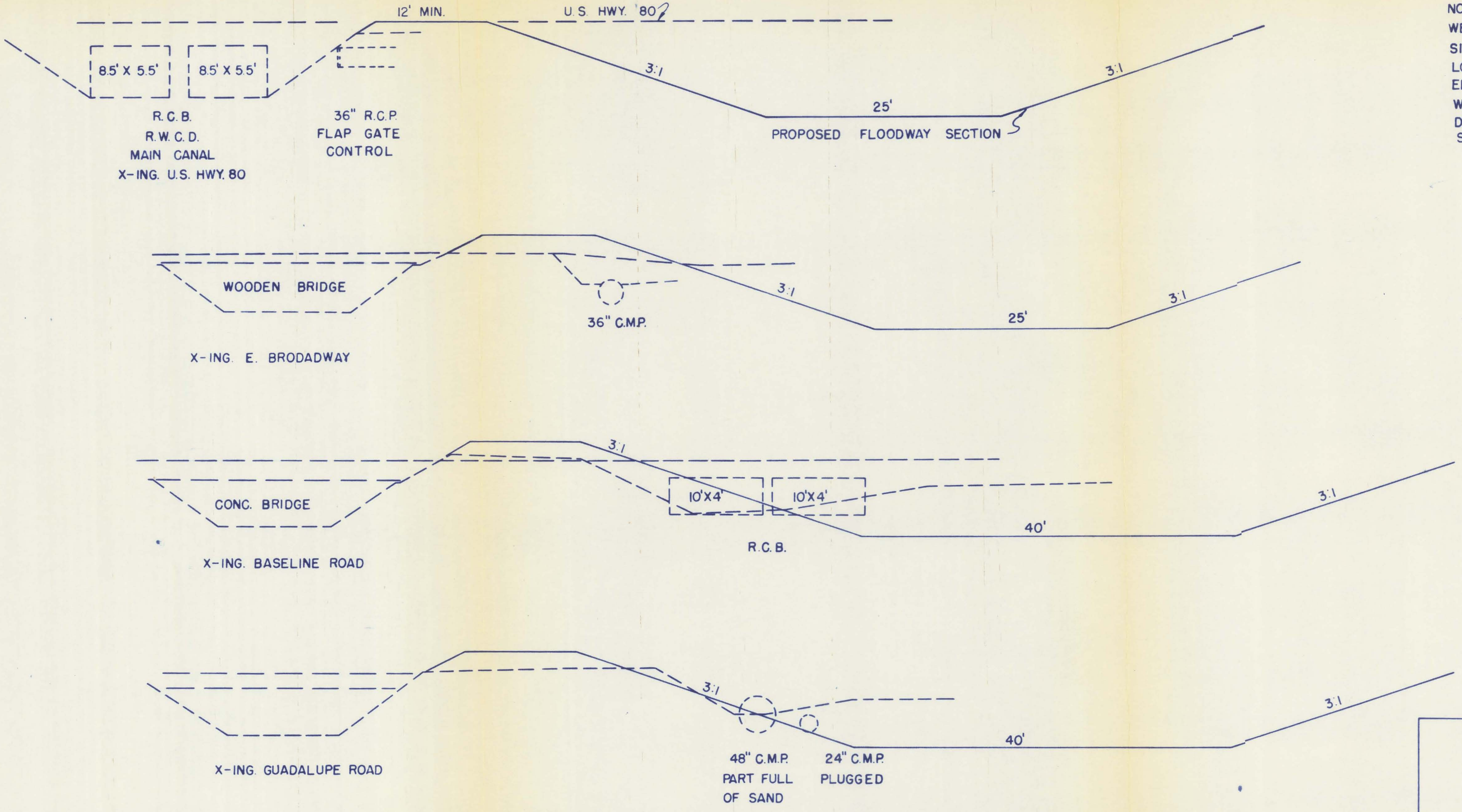


FLOODWAY CHANNEL
SEE PLAN FOR DIMENSIONS

GATE CONTROL
FLOODWATER
PIPE INLET

FLOOD CONTROL REPORT
AREA I

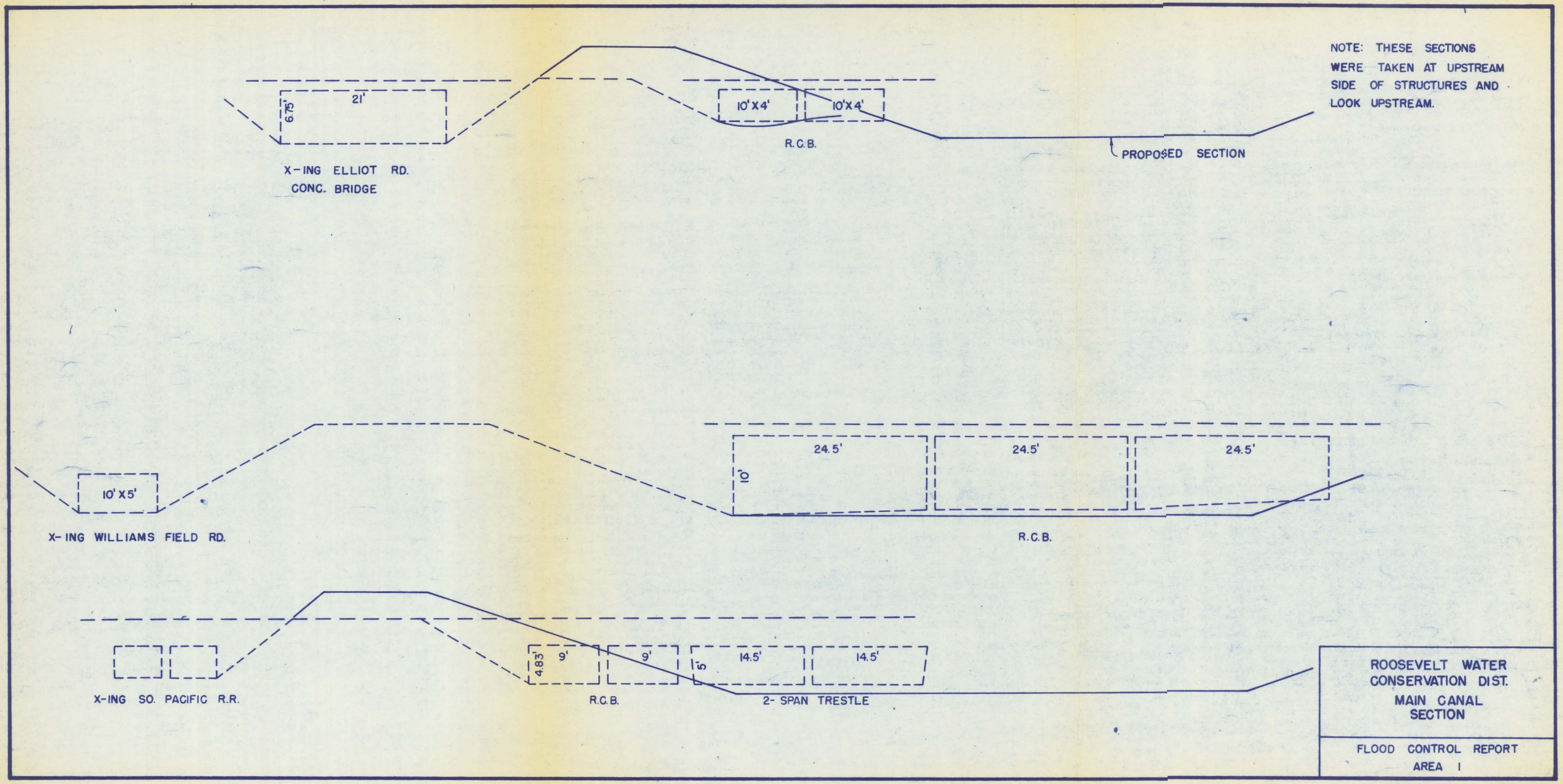
APPENDIX A, PLATE NO. 17



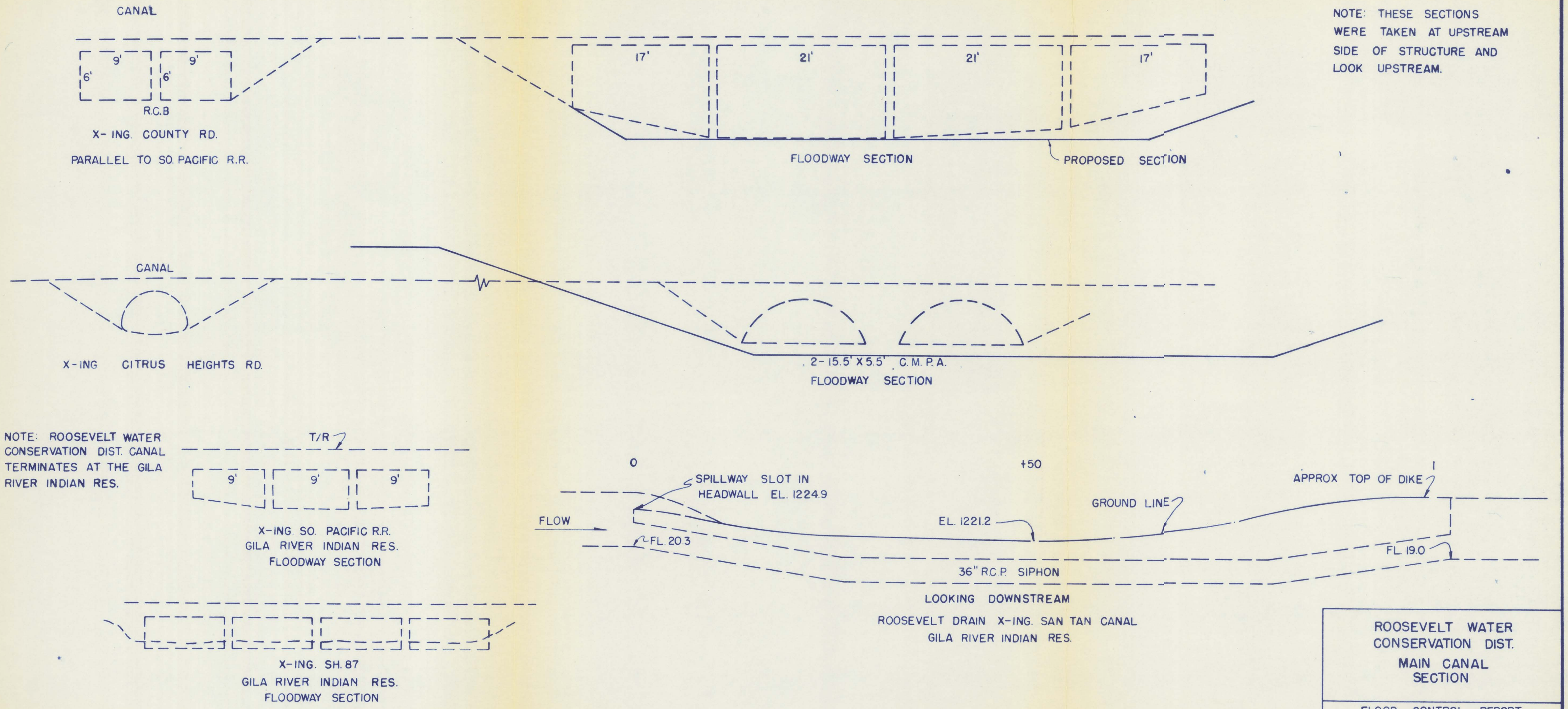
NOTE: THESE SECTIONS WERE TAKEN AT UPSTREAM SIDE OF STRUCTURES AND LOOK UPSTREAM. ACTUAL ELEV. AND LOCATION OF FLOODWAY SECTIONS WOULD BE DETERMINED BY DETAILED SURVEY AND PLANS.

ROOSEVELT WATER CONSERVATION DIST.
MAIN CANAL SECTION

FLOOD CONTROL REPORT
AREA I



APPENDIX - A, PLATE NO. 19

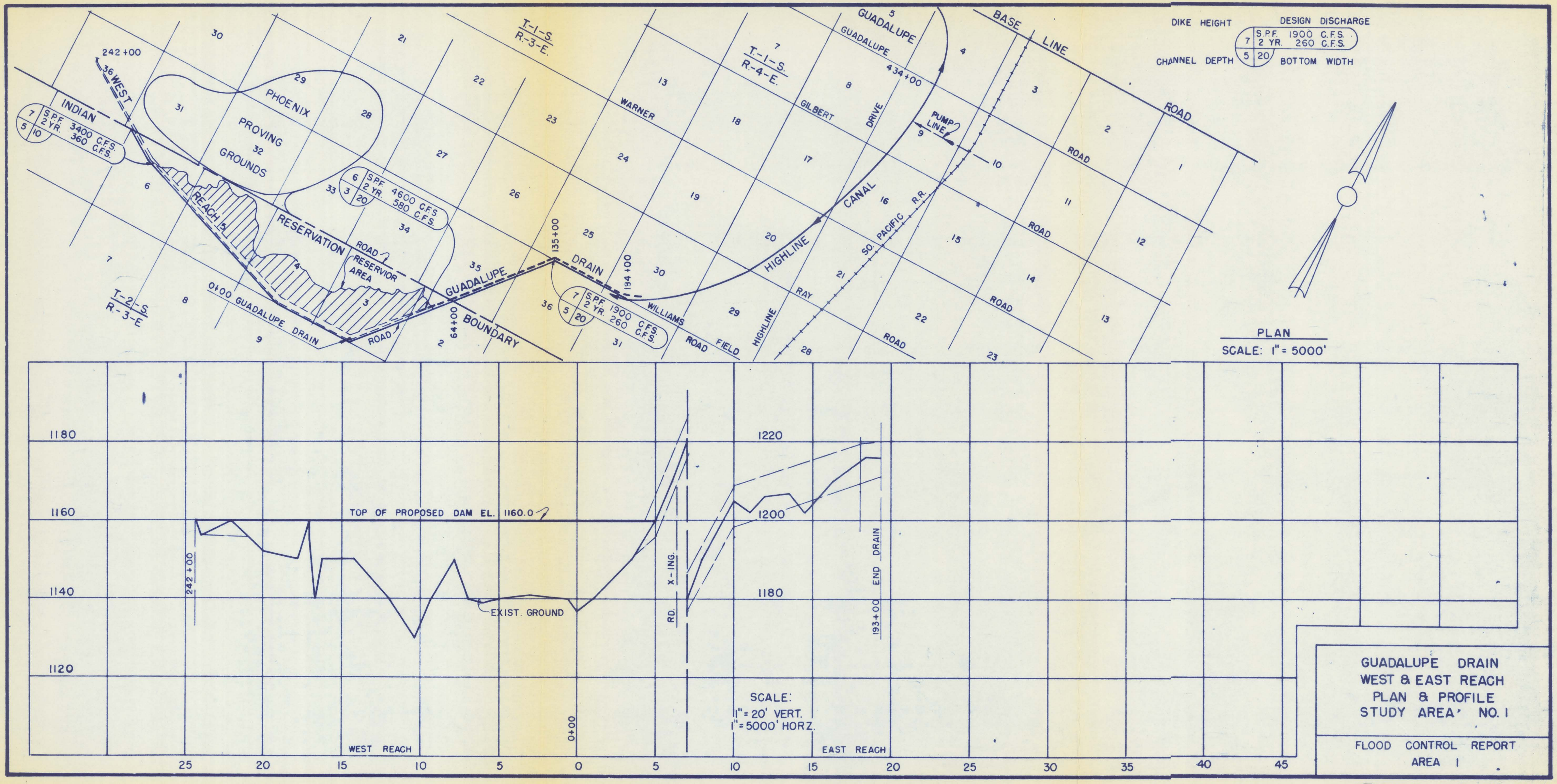


ROOSEVELT WATER CONSERVATION DIST.

MAIN CANAL SECTION

FLOOD CONTROL REPORT

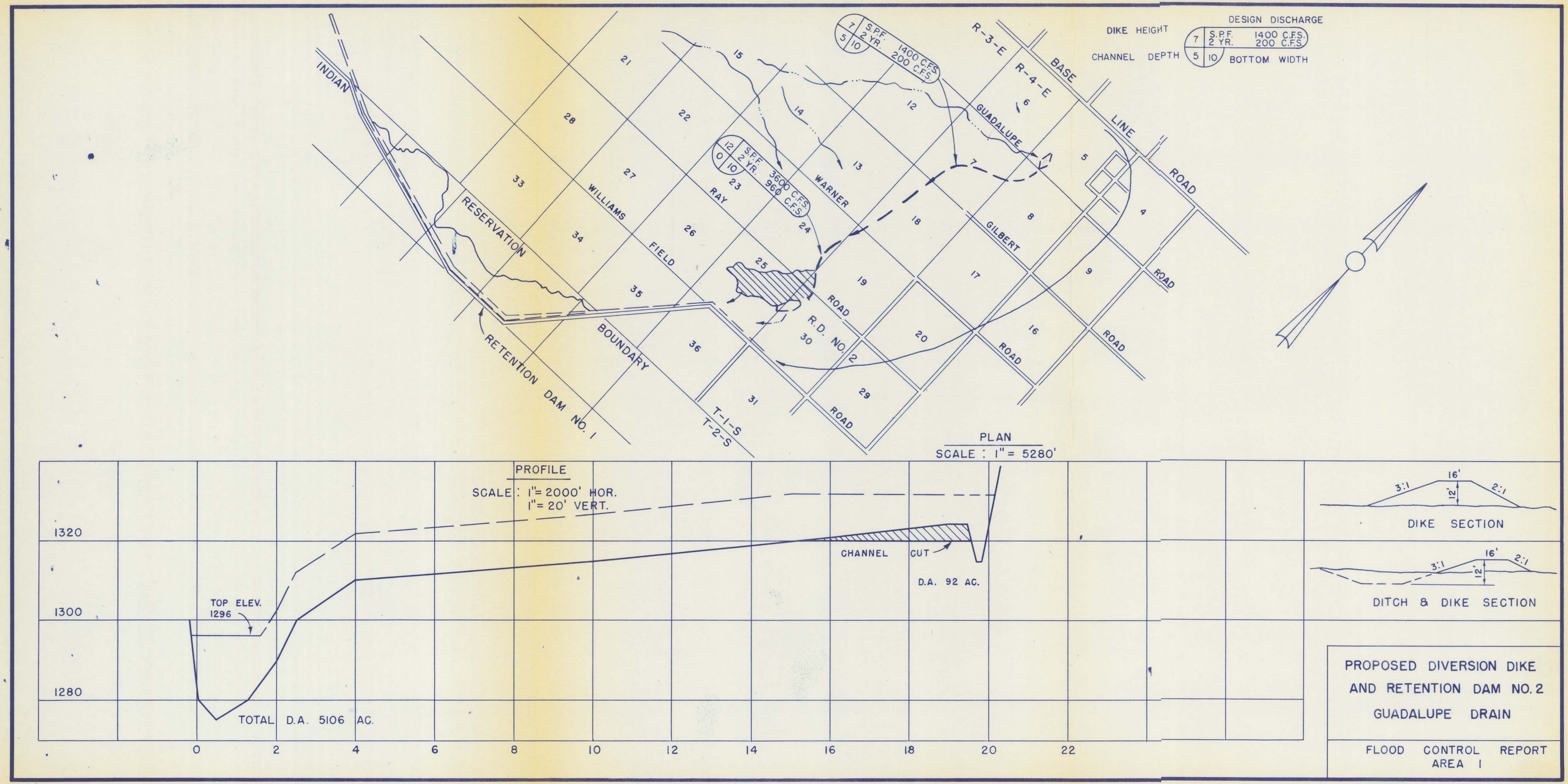
AREA I



GUADALUPE DRAIN
WEST & EAST REACH
PLAN & PROFILE
STUDY AREA NO. 1

FLOOD CONTROL REPORT
AREA 1

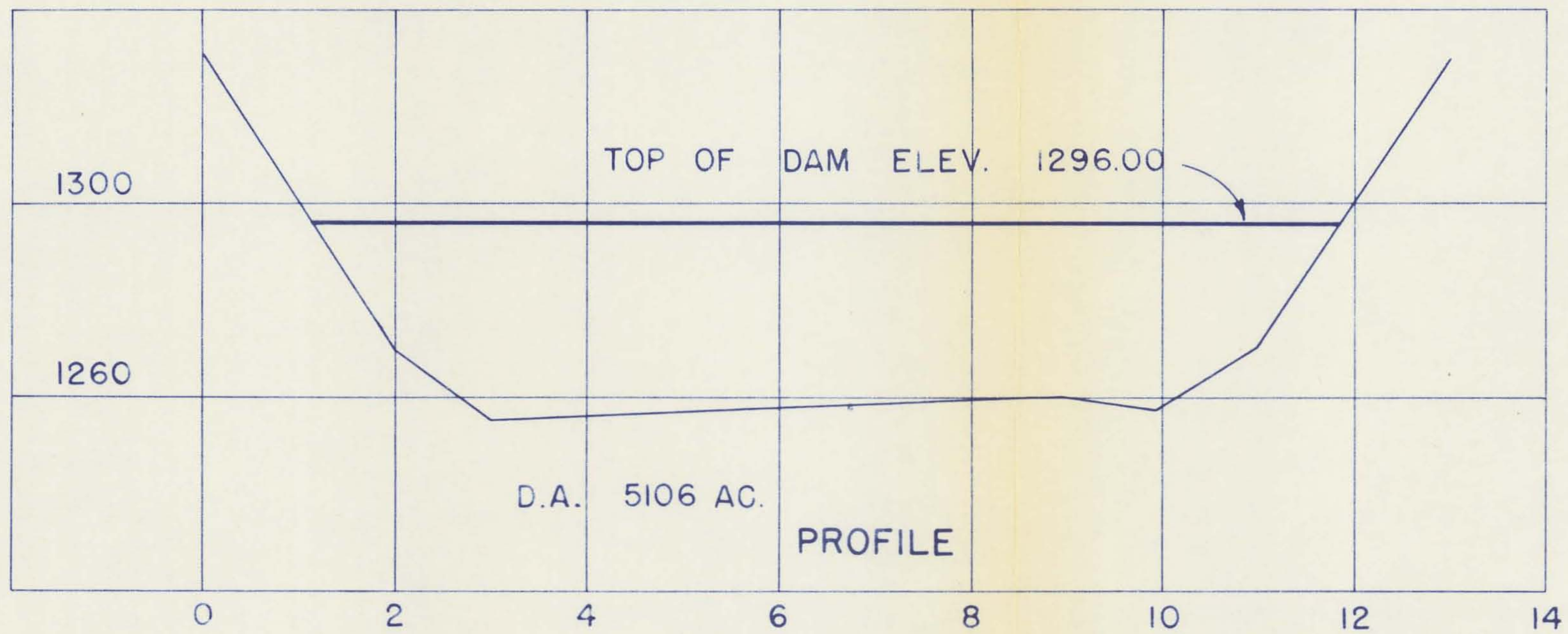
APPENDIX - A, PLATE NO. 21



PROPOSED DIVERSION DIKE AND RETENTION DAM NO. 2 GUADALUPE DRAIN

FLOOD CONTROL REPORT AREA 1

RETENTION DAM NO. 2
GUADALUPE DRAIN

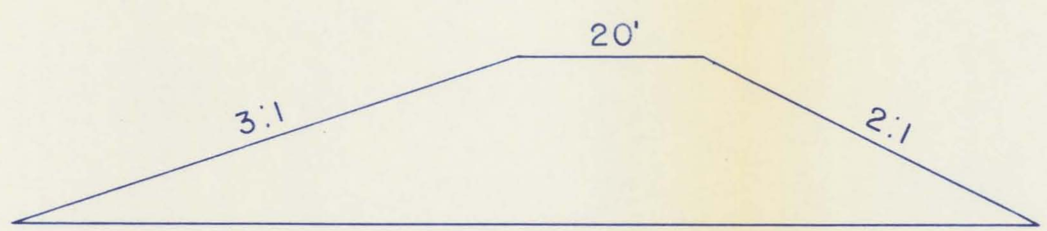


PROFILE

SCALE

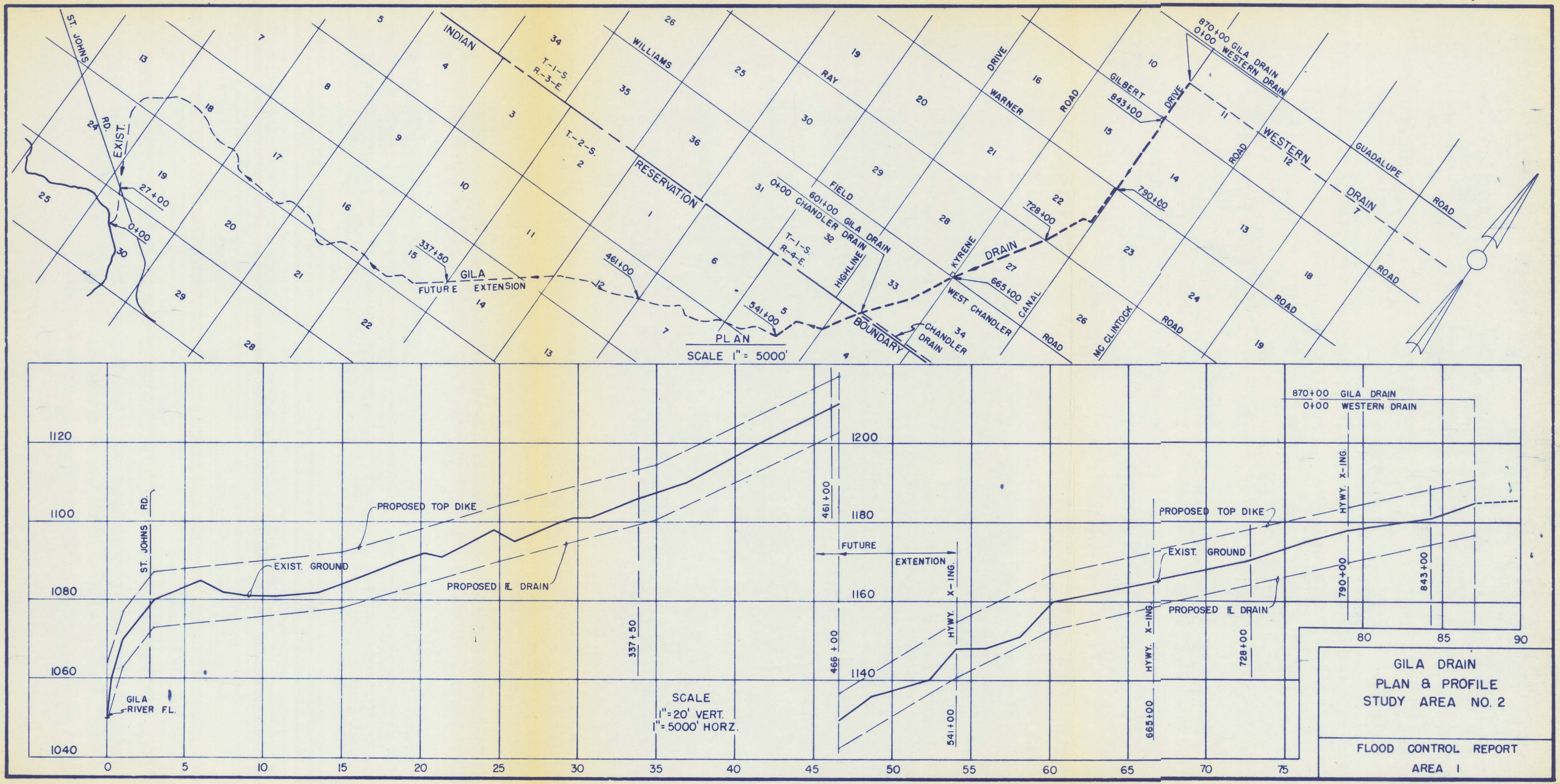
1" = 200' HORZ.

1" = 40' VERT.

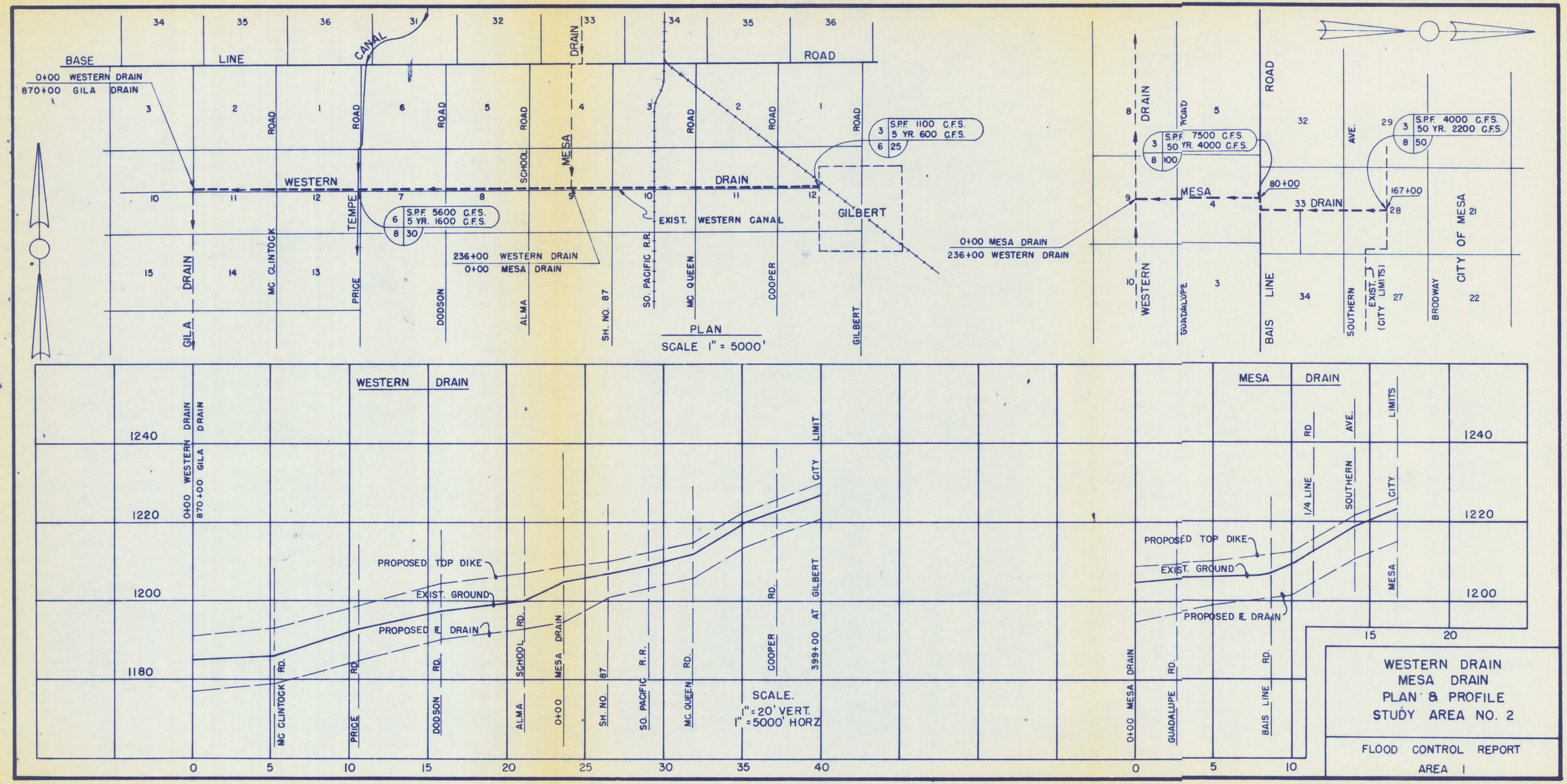


TYPICAL SECTION

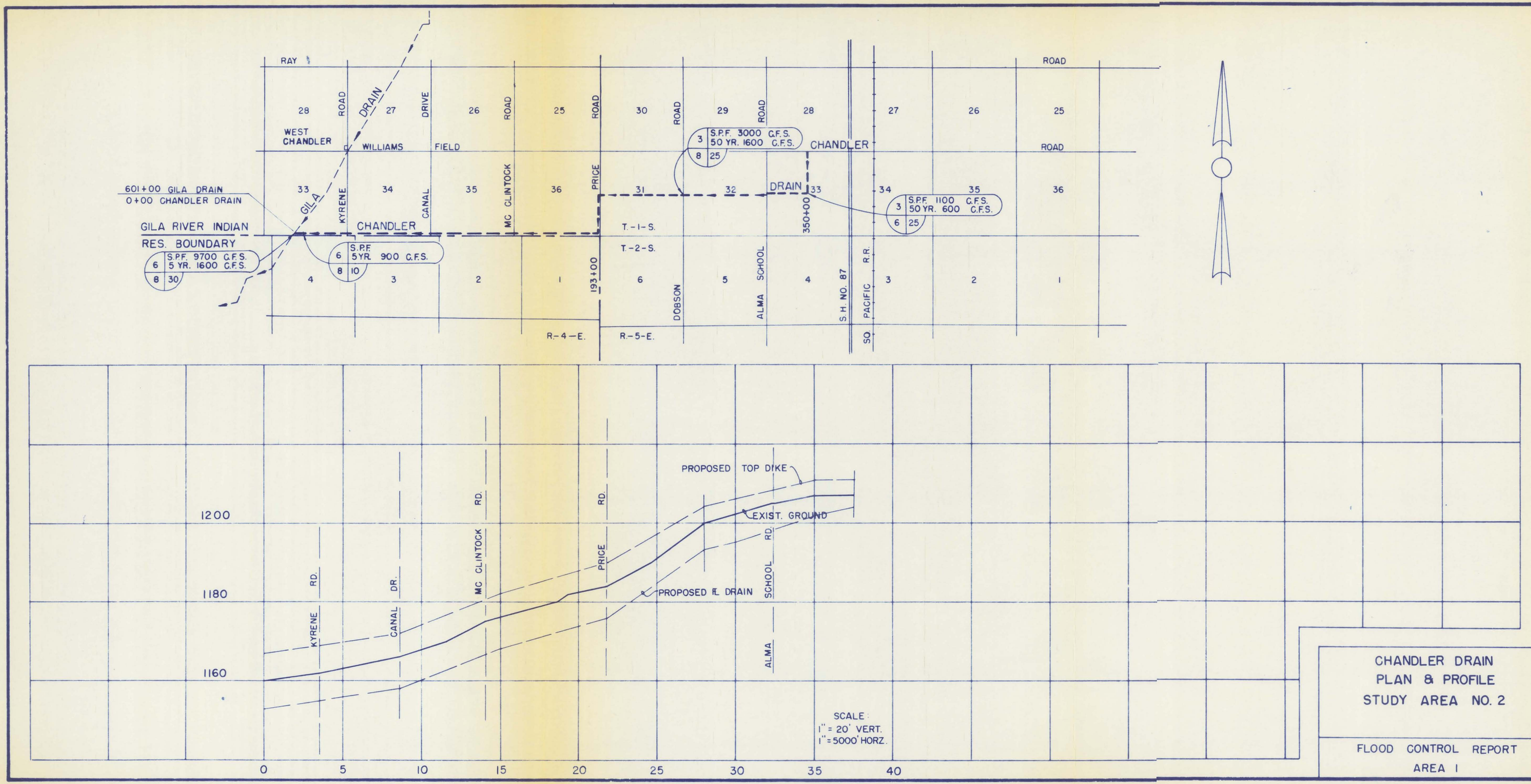
RETENTION DAM NO. 2 GUADALUPE DRAIN
FLOOD CONTROL REPORT AREA I



APPENDIX - A, PLATE NO. 22



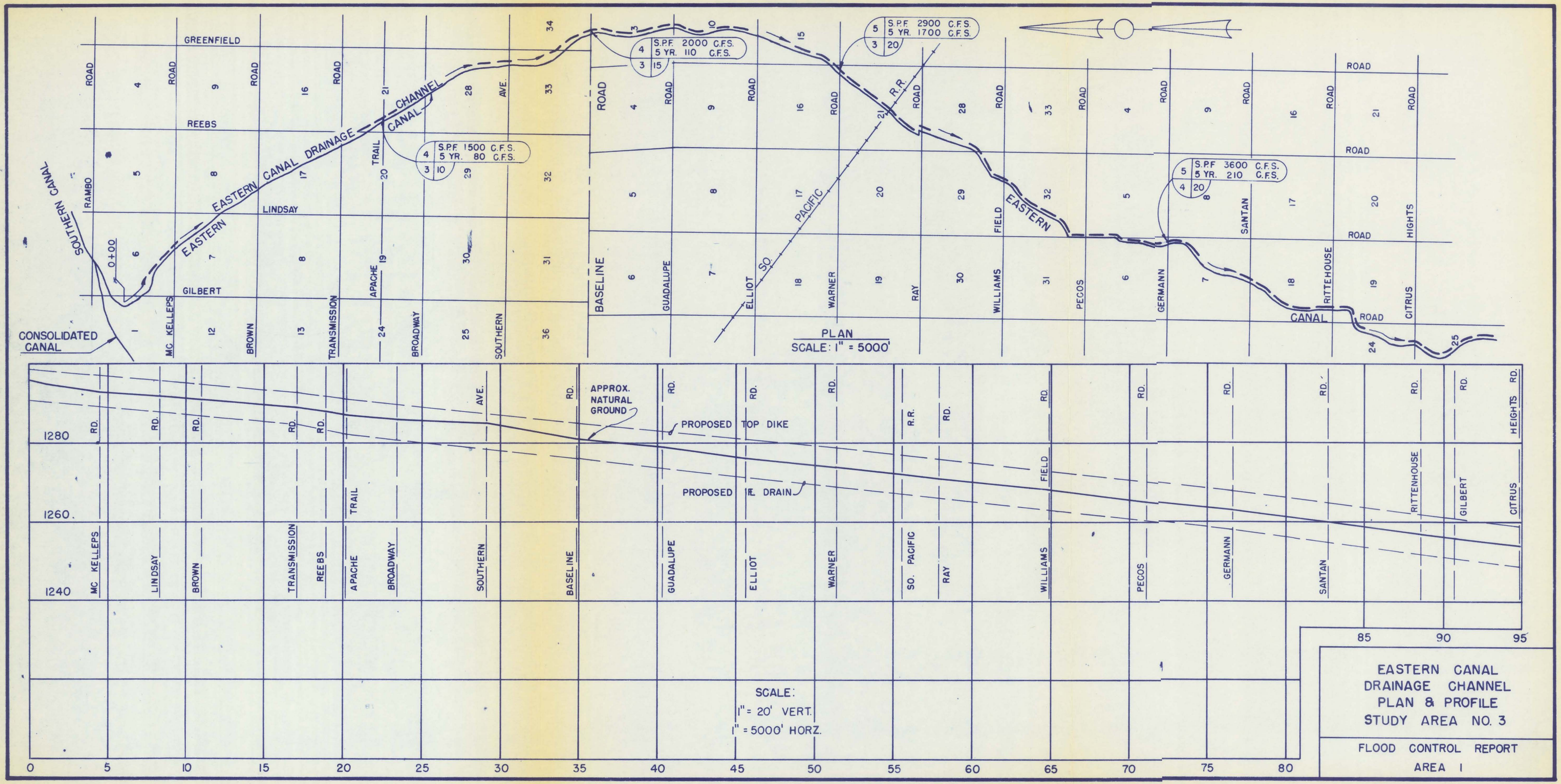
APPENDIX - A, PLATE NO. 22-A



CHANDLER DRAIN
PLAN & PROFILE
STUDY AREA NO. 2

FLOOD CONTROL REPORT
AREA I

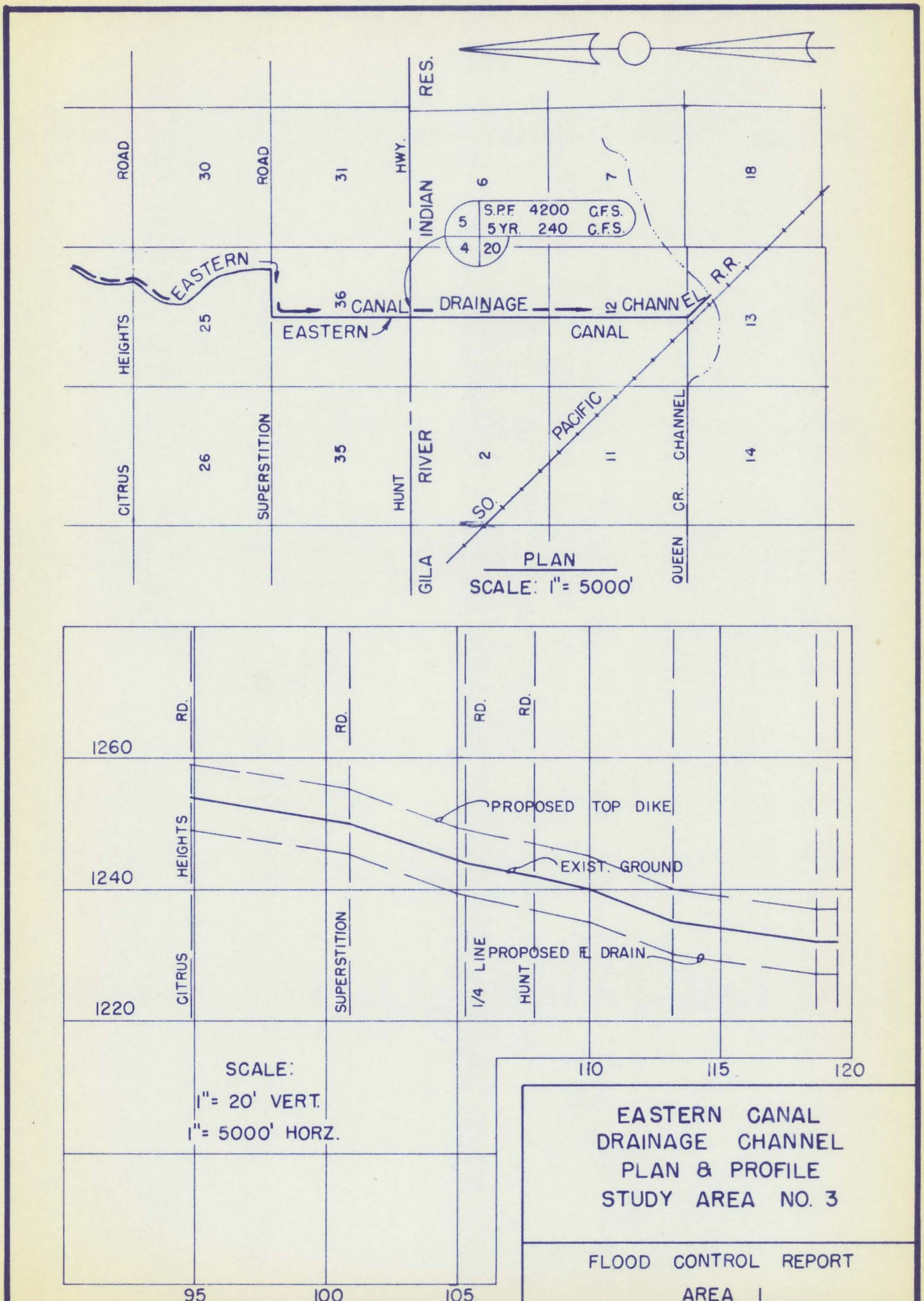
APPENDIX - A, PLATE NO. 22-B



EASTERN CANAL DRAINAGE CHANNEL PLAN & PROFILE STUDY AREA NO. 3

FLOOD CONTROL REPORT AREA I

APPENDIX – A, PLATE NO. 23



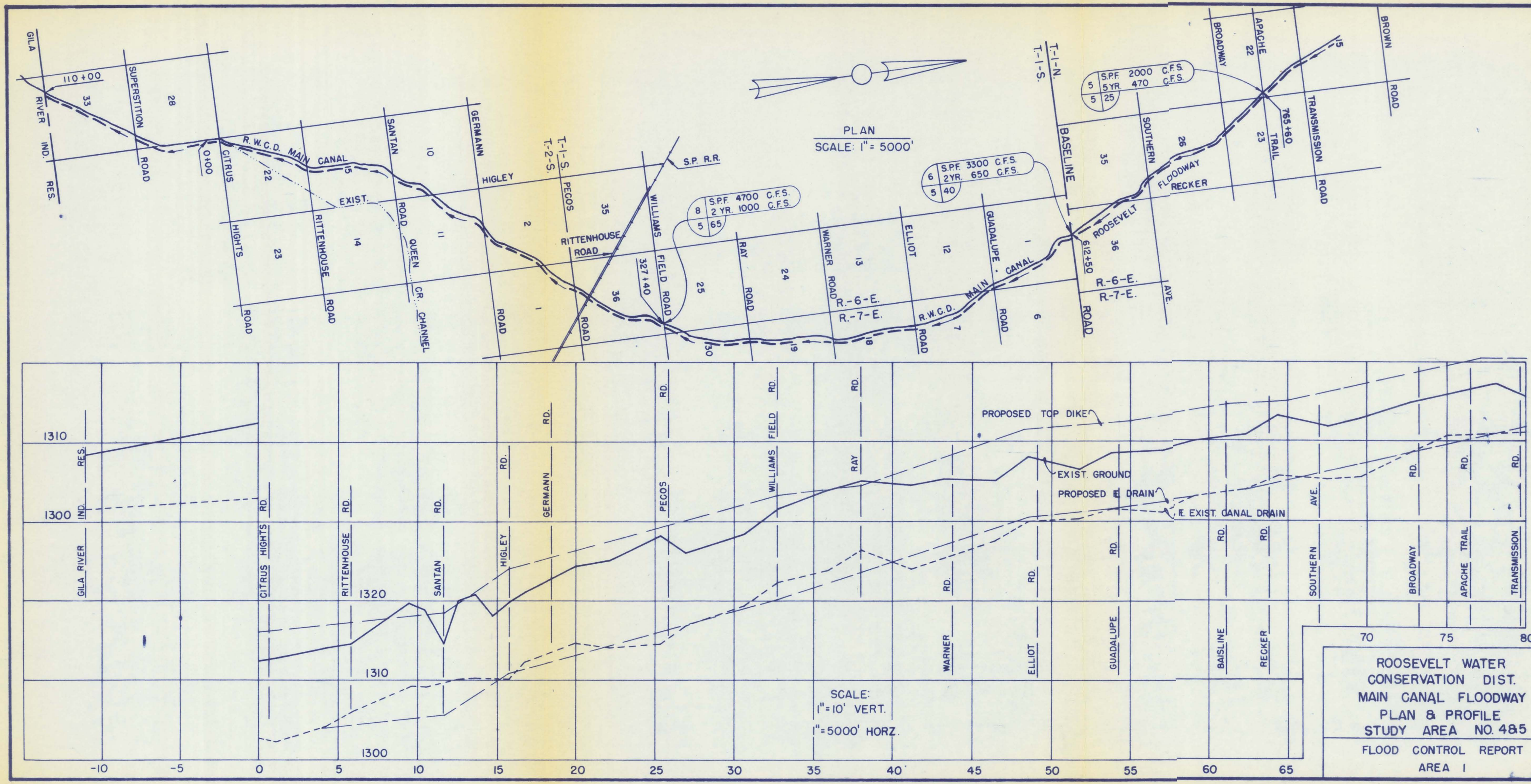
PLAN
SCALE: 1" = 5000'

SCALE:
1" = 20' VERT.
1" = 5000' HORZ.

EASTERN CANAL
DRAINAGE CHANNEL
PLAN & PROFILE
STUDY AREA NO. 3

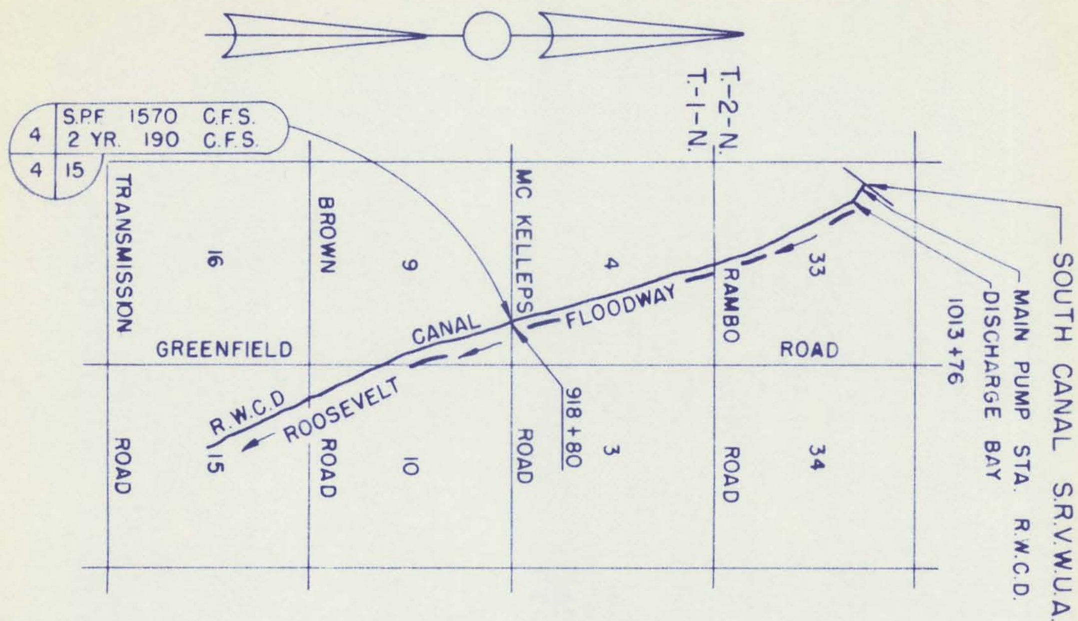
FLOOD CONTROL REPORT
AREA I

APPENDIX – A, PLATE NO. 23



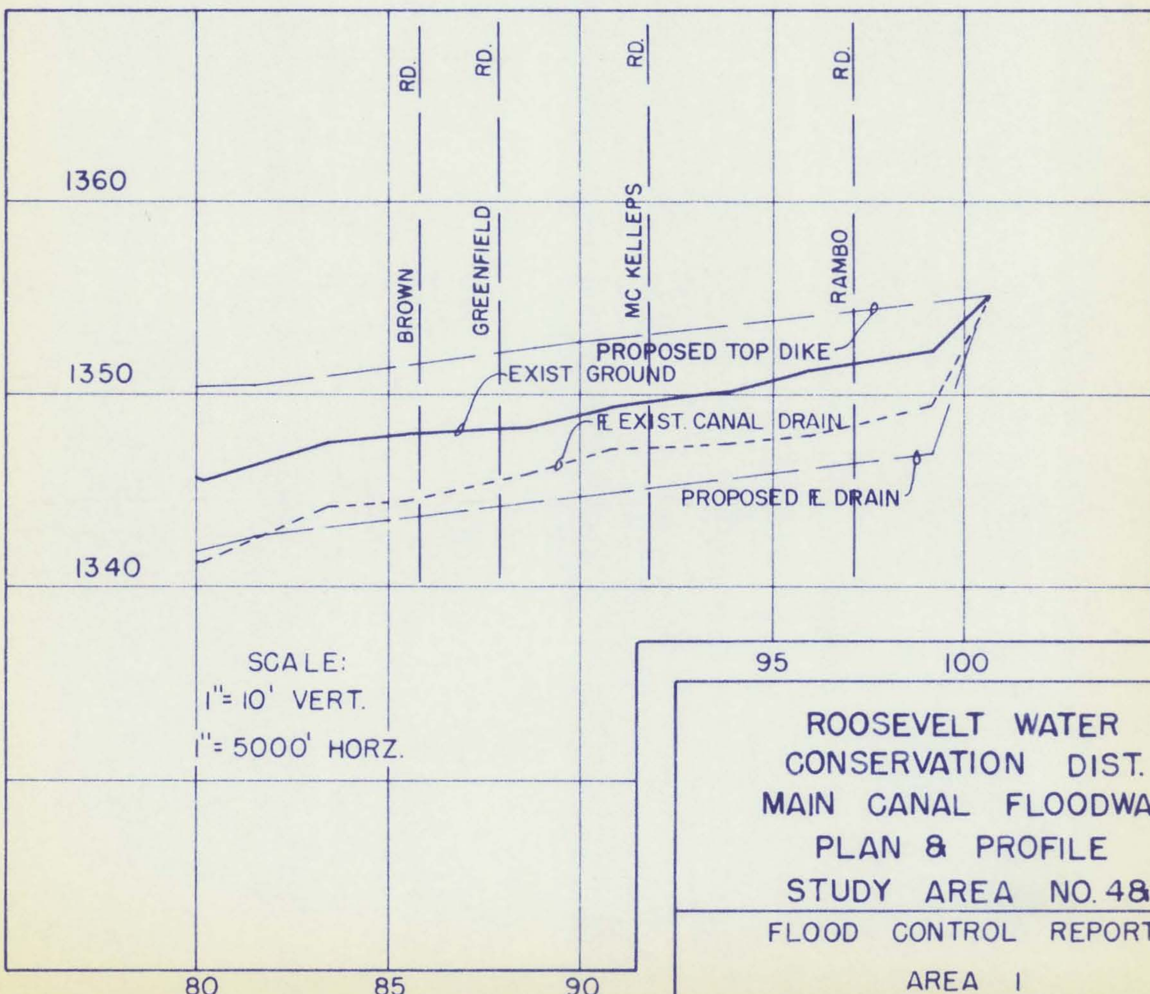
ROOSEVELT WATER CONSERVATION DIST.
MAIN CANAL FLOODWAY
PLAN & PROFILE
STUDY AREA NO. 4&5

FLOOD CONTROL REPORT
AREA I



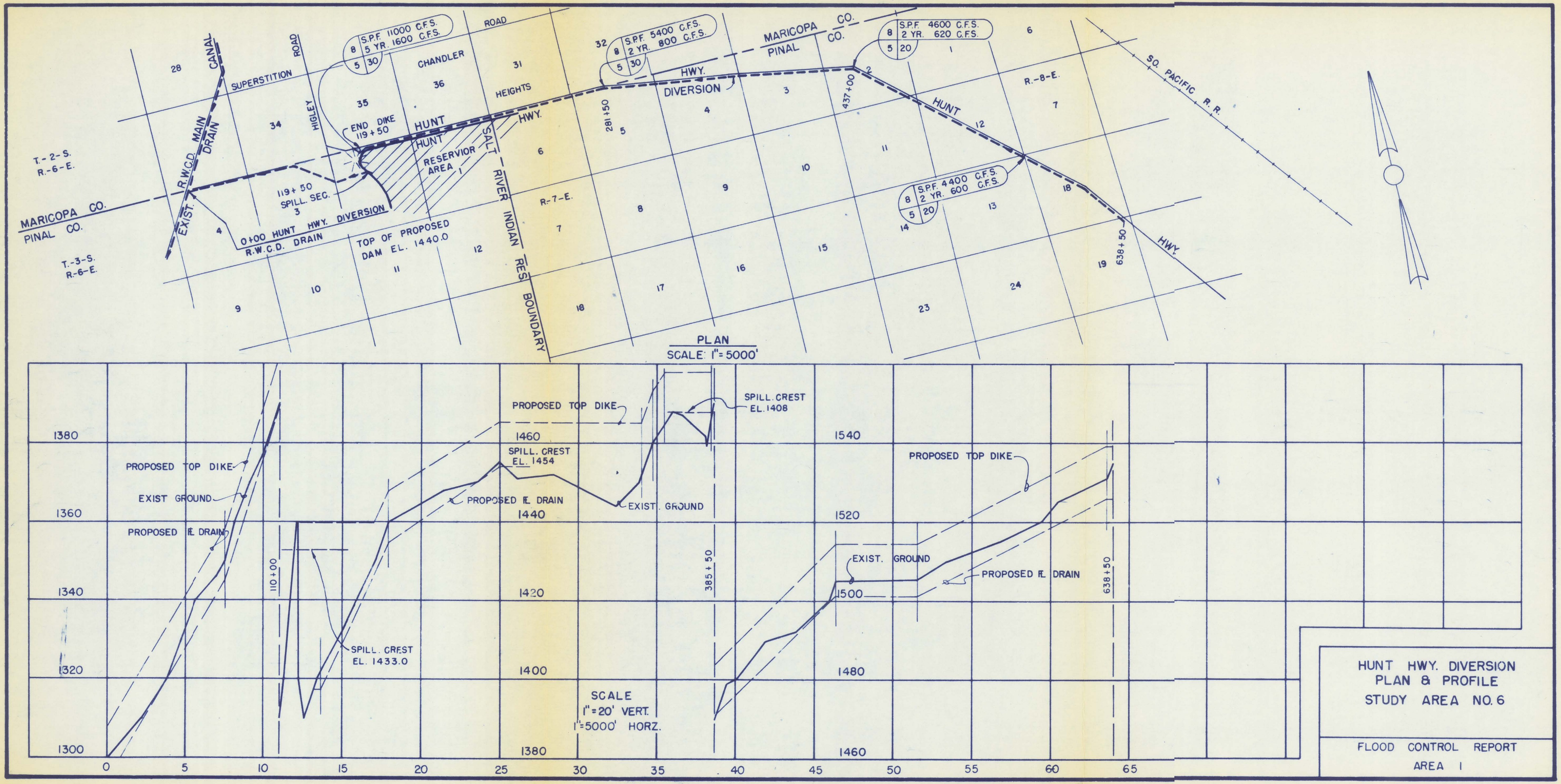
PLAN

SCALE: 1" = 5000'



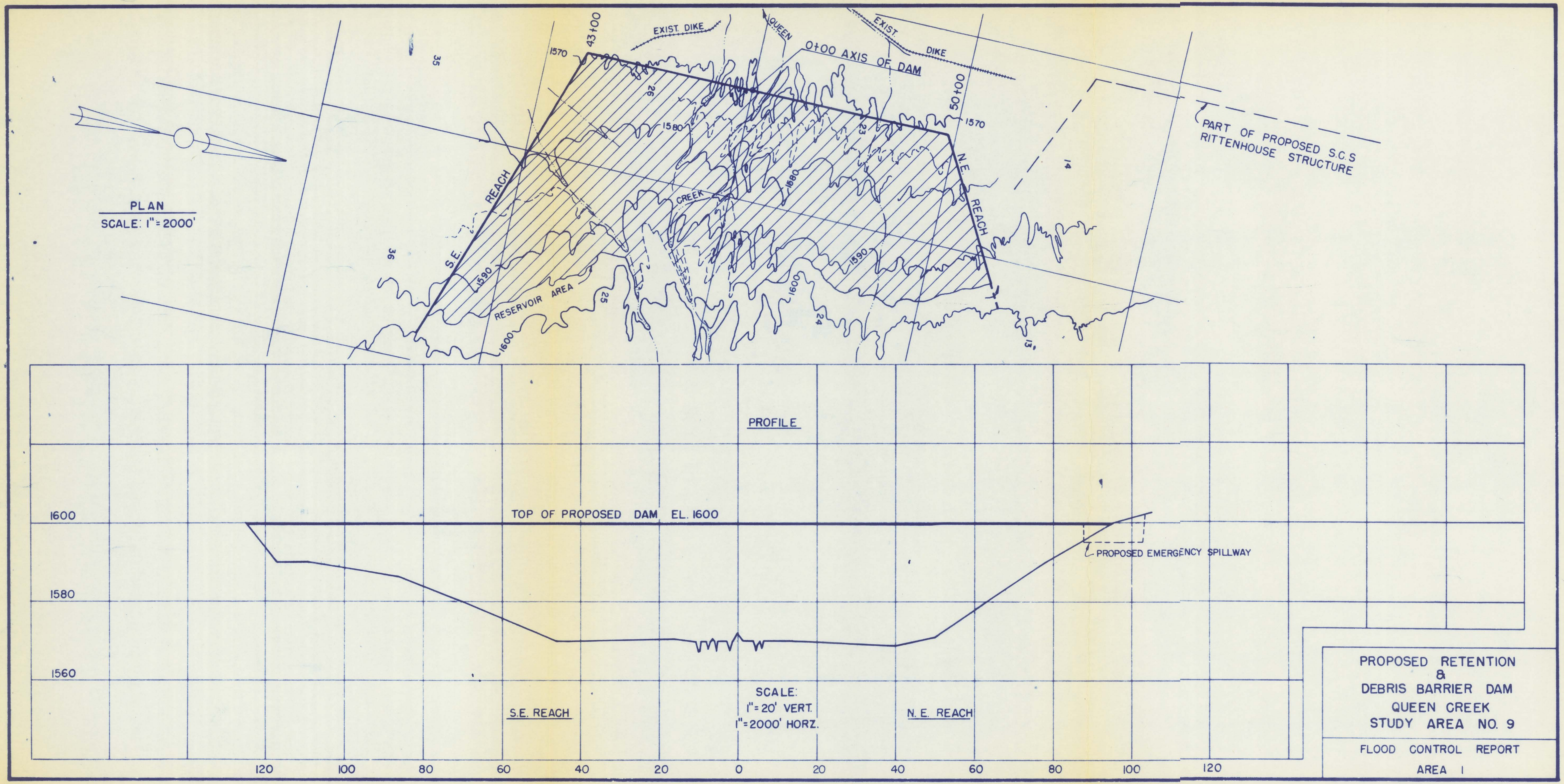
ROOSEVELT WATER CONSERVATION DIST.
MAIN CANAL FLOODWAY
PLAN & PROFILE
STUDY AREA NO. 4&5

FLOOD CONTROL REPORT
AREA I



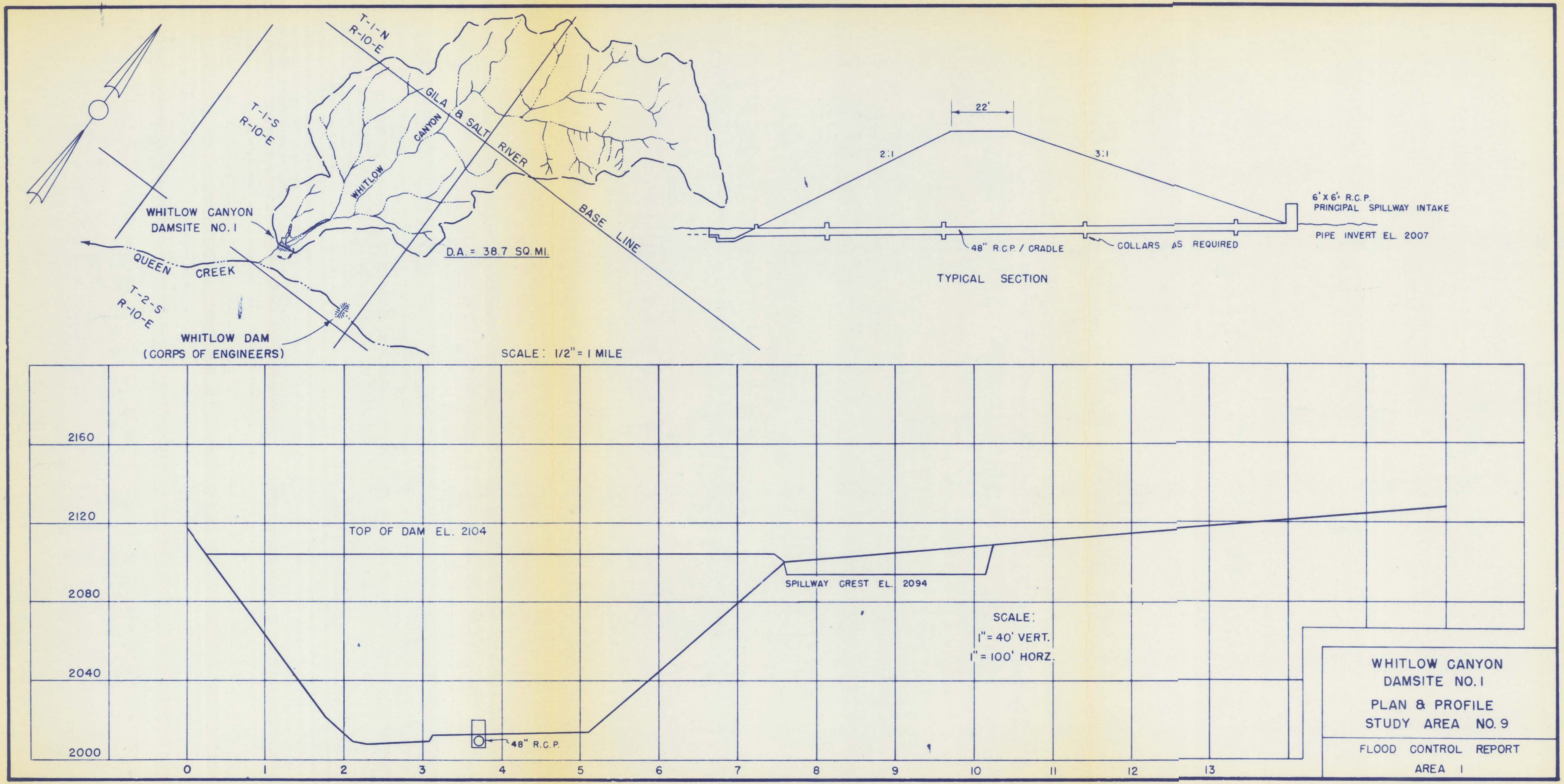
HUNT HWY. DIVERSION
PLAN & PROFILE
STUDY AREA NO. 6

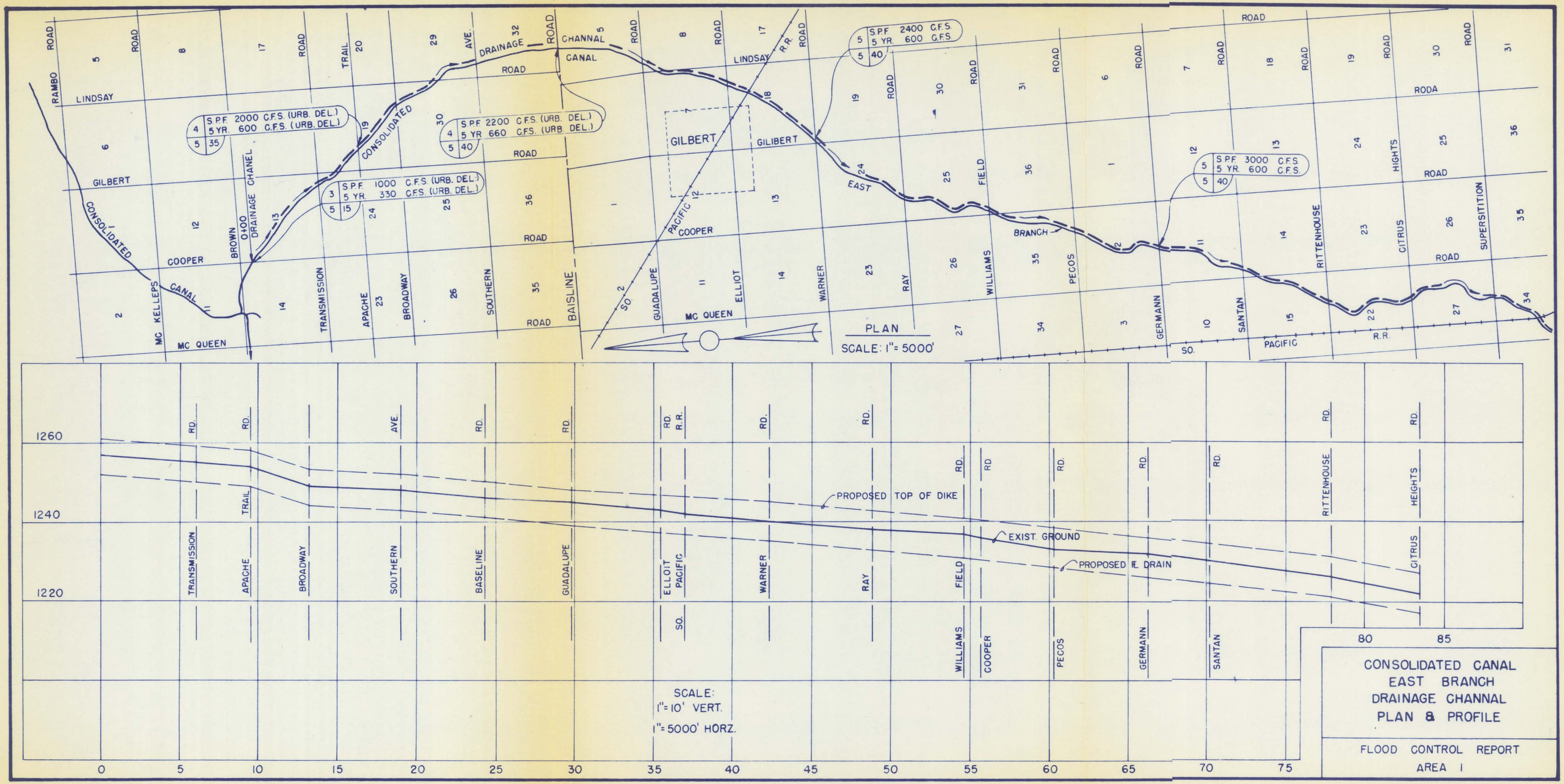
FLOOD CONTROL REPORT
AREA 1



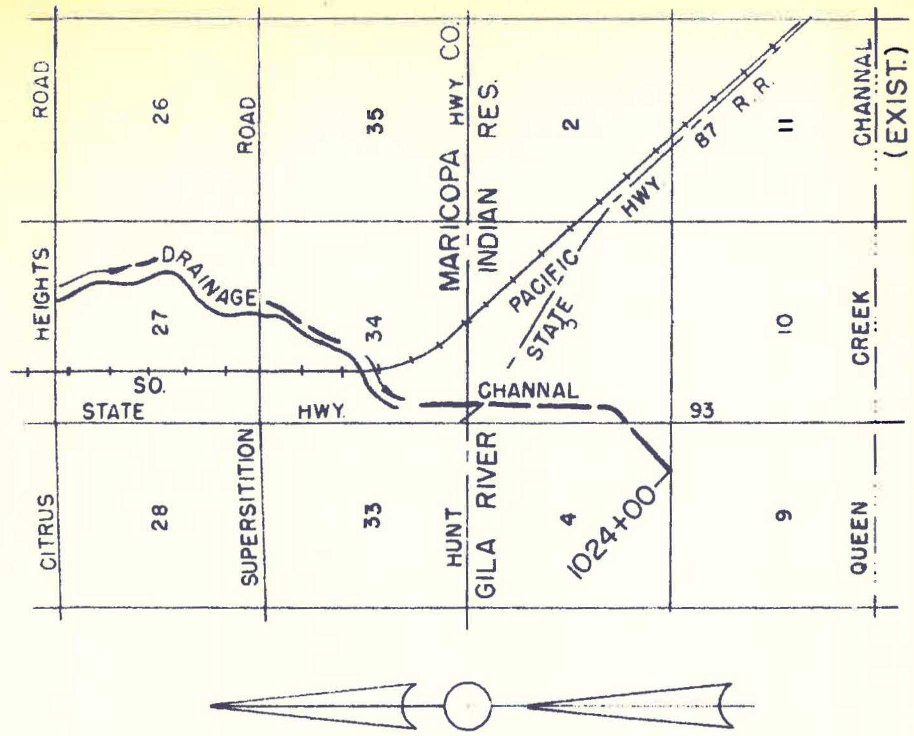
PROPOSED RETENTION & DEBRIS BARRIER DAM
QUEEN CREEK
STUDY AREA NO. 9

FLOOD CONTROL REPORT
AREA I


T-1-N
R-10-E
T-1-S
R-10-E
T-2-S
R-10-E
GILA & SALT RIVER BASE LINE
WHITLOW CANYON
WHITLOW CANYON DAMSITE NO. 1
QUEEN CREEK
WHITLOW DAM (CORPS OF ENGINEERS)
D.A. = 38.7 SQ. MI.
SCALE: 1/2" = 1 MILE
22'
2:1
3:1
6' X 6' R.C.P. PRINCIPAL SPILLWAY INTAKE
PIPE INVERT EL. 2007
48" R.C.P. / CRADLE
COLLARS AS REQUIRED
TYPICAL SECTION
2160
2120
2080
2040
2000
0 1 2 3 4 5 6 7 8 9 10 11 12 13
TOP OF DAM EL. 2104
SPILLWAY CREST EL. 2094
48" R.C.P.
SCALE:
1" = 40' VERT.
1" = 100' HORZ.
WHITLOW CANYON DAMSITE NO. 1
PLAN & PROFILE
STUDY AREA NO. 9
FLOOD CONTROL REPORT
AREA 1

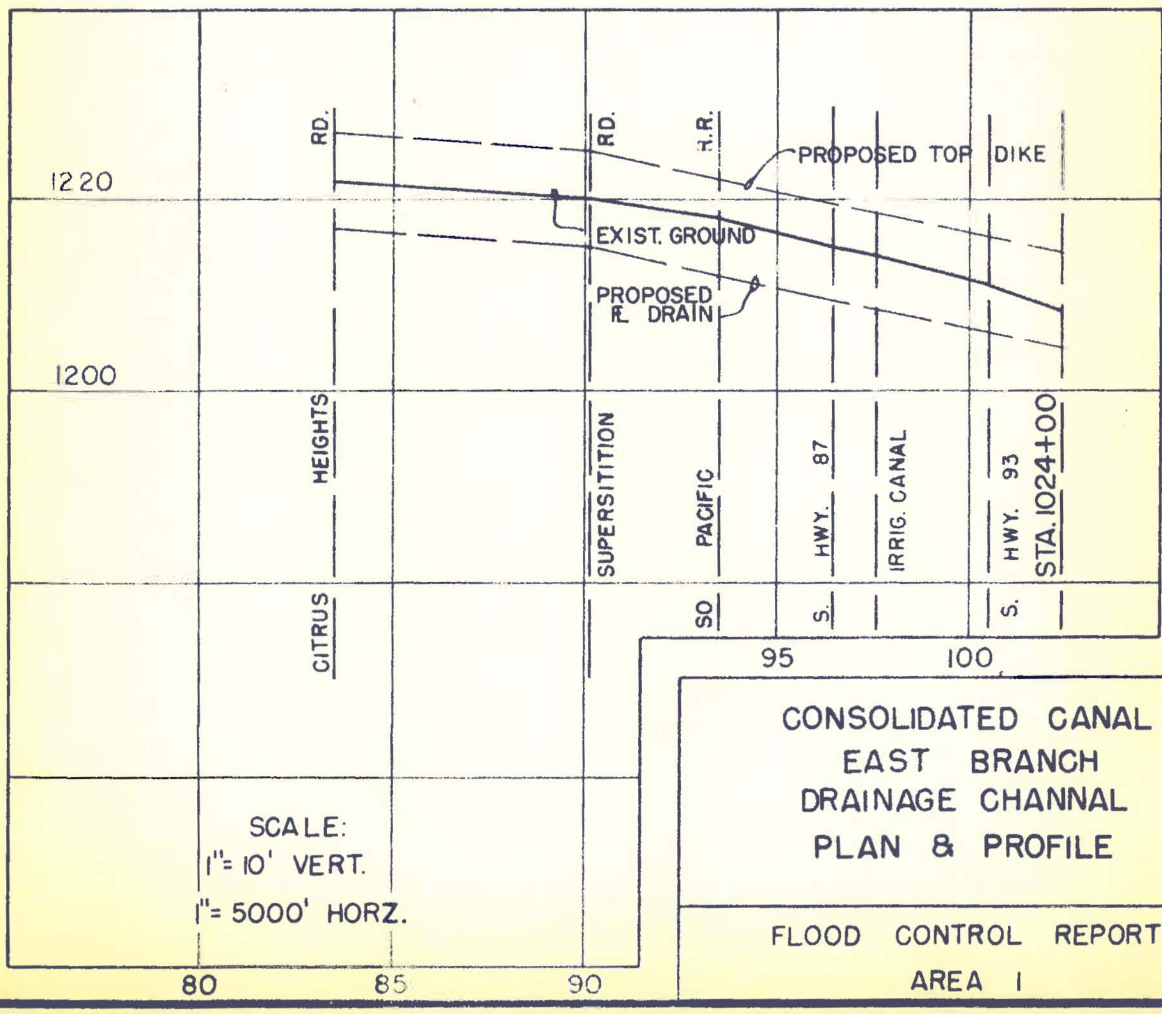


APPENDIX A, PLATE NO. 28



PLAN

SCALE: 1" = 5000'



CONSOLIDATED CANAL
EAST BRANCH
DRAINAGE CHANNAL
PLAN & PROFILE

FLOOD CONTROL REPORT

AREA I

APPENDIX A PLATE NO. 28

FLOOD CONTROL SURVEY AND REPORT

STUDY AREA NO. I

SOUTHEASTERN MARICOPA COUNTY

APPENDIX I-B

ECONOMIC BASE STUDY

FLOOD CONTROL DISTRICT

MARICOPA COUNTY

Prepared by
Benham Engineering Company Inc.
Consulting Engineers
Phoenix, Arizona

APPENDIX I-B

ECONOMIC BASE STUDY

TABLE OF CONTENTS

APPENDIX I-B
ECONOMIC BASE STUDY

INTRODUCTION

1. General: The purpose of this appendix is to present the considerations and methods of analysis used in the economic base study made in the development of the flood control report for the basin.

ECONOMIC ANALYSIS

2. General: Economic justification of a comprehensive program of flood control for the basin depends only in part on the past history of flooding and flood damages. The future conditions to be expected during the economic life of the project is an extremely important phase of any economic analysis. The nature and amount of future flood damages requires a careful projection of population, industrial and commercial activity and the trends of development. The reports on the economic growth and development of the urban centers of the basin prepared by the Maricopa County Planning and Zoning Department were used as a basis for anticipating the flood damage to be expected under these changing conditions.

3. Economic Growth: The basin, along with the rest of Maricopa County, has experienced a tremendous expansion in population and in industrial and commercial activity in recent years. Continued expansion of urban areas can be expected with eventual urbanization of much of the basin. This will have a marked effect on the source of flooding and on the kind and amount of future flood damages. The present land use of the basin is shown on Land Use Map, Plate No. 1. The projection of future expansion is also shown on Plate No. 1.

4. Flood Damages: The Soil Conservation Service and the Corps of Engineers have extensively studied the past history of flooding and flood damages of the basin. These Federal agencies have made a number of reports on these studies

At present, the Soil Conservation Service is preparing a report on Study Area No. 7, planning detention reservoirs to control floods originating in the Superstition Mountains. The flood damages determined by these studies were used in evaluating the proposed program with respect to present conditions. Flood damages to be expected under future conditions were evaluated by the same methods utilized by these agencies. An extract from a Soil Conservation Service report which demonstrates these techniques, is shown as Extract No. 1, "Flood Damages".

5. Cost-Benefit Determinations: The cost-benefit analysis of the proposed program is based on an economic life of 50-years. The methods of analysis conform to those used by the Soil Conservation Service and the Corps of Engineers. The resultant cost-benefit ratios are considered conservative. Past experience has shown that, in general, the true economic benefits of projects of this nature cannot be anticipated. The effect of the expanding economy of the nation, as a whole, and this area in particular, and the effect of advances in the general technology over the next 50 years, can only vaguely be defined. The program itself opens new economic opportunities, the import of which cannot be realized at this time. Consequently, the actual worth of the proposed program of flood control for the basin may be several times that indicated by the benefit-cost ratios shown.

FLOODWATER DAMAGES

Types of damage considered -- The principal classes of property damaged by floods in the Queen Creek Basin are agricultural property, irrigation works, highways, urban property, and railroads and other public utilities. Refer to map 9 for flood damage areas. Damages are distributed among the various classes of property as follows:

	Percent
Farms	67.9
Irrigation systems	18.4
Highways	6.0
Urban property	5.8
Railroad & utilities	1.4
Other	0.5
	100.0

Source of information -- Information upon which floodwater damages are based was secured in part from flood-damage schedules taken in 1938 for the period of 1926 to 1938, covering approximately 25 percent of the farm area subject to damage. Where possible, at least one farmer was contacted in each square mile. Because of the short length of tenure of some farmers, a few sections were not covered by a schedule. However, the sample was large enough that it offers a good basis upon which to estimate flood damages. In addition, damages caused by the 1941 and 1946 floods were estimated on a basis of information gathered from irrigation districts and from spot checks in the farming areas affected. All damage estimates were adjusted to 1948 prices by appropriate indexes.

Kinds of farm damages -- Floods damage crops in a number of ways. Alfalfa hay that has been cut and still in the field is washed away or rots on the ground. Some crops are killed by drowning or scalding. The quality and yield of crops are reduced by introduction of weeds and deposition of sediment. Land damage consists principally of washing and sediment deposition. Other farm damage is the destruction of irrigation laterals and field borders. Two important types of indirect damage are (1) disruption of irrigation services due to breaks in irrigation canals and damage to farm irrigation systems, causing reductions in crop yields; (2) introduction of weeds by the floodwaters, necessitating additional expense for cultivation and in some cases delay in planting alfalfa on land that has been fallowed for fall planting.

Damage to farms by a small flood -- The 1936 flood and also 1933, 1941, and 1946 floods, herein considered, are small floods. They offer a basis for determining average flood damages caused by small flood. The 1933, 1941, and 1946 floods were essentially the same type both with respect to area inundated damage incurred. The average direct farm damage caused by these floods is estimated at $134,000. The 1936 flood inundated a larger area with consequent farm damages of $284,000 because of its breaking into the farm area further north than the other floods. The "average" small flood is considered as a composite of these two types, and the damages resulting from a small flood are based on the flood frequence of the 1936 flood type in relation to frequencies of other small floods. It is expected that one flood of every five will be of the 1936 type. Thus the average damage to agriculture resulting from a small flood is estimated as follows:

Direct farm damage	$134,000
Indirect farm damage	31,000
Total damage	$165,000

Damage to farms by a medium flood -- A medium flood will not only inundate more area than a small flood but will cause more damage per acre. It is assumed that the damage per acre by a medium flood would be 50% greater than on the acreage inundated by a small flood and that the same damage per acre would be incurred on the additional acreage inundated by a mediim flood. To illustrate the method used, it is estimated that the average small flood will inundate about 9,000 acres, resulting in damage of $165,000, or about $18.00 per acre. The average medium flood will inundate 16,000 acres. Thus the damage that may be expected from a medium flood are calculated as follows:

9,000 acres @ $27.00 ($18.00 x 150%)	=	$243,000
7,000 acres @ $18.00	=	126,000
Total damage		$369,000

The assumptions used above are substantiated by a comparison of the 1936 flood with other small floods. The 1936 floods not only inundated a greater area but also caused greater damage per acre. The direct damage for a medium flood is estimated at $295,000 and indirect damages of $74,000.

Damage to farms by a large flood -- Area inundated and damaged per acre would be increased proportionately by a large flood. It is estimated that a large flood would inundate from 20,000 to 25,000 acres, with damage ranging from $18.00 to $36.00 per acre. The resultant damage is estimated as follows:

Direct farm damage	$528,000
Indirect farm damage	105,000
Total damage	$633,000

Damage to farms by a great flood -- A great flood would probably inundate as much as 30,000 acres. Damages would be great, perhaps complete crops loss would be sustained on a considerable acreage depending on the season when the flood struck. Crops not destroyed would be damaged by lack of irrigation water because of destruction of the irrigation system. Even after the 1936 flood it was two weeks before irrigation water flowed in the Roosevelt Water Conservation District canal south of the railroad. A great flood would put the canal out of use for a longer time. The damage caused by a great flood is estimated as follows:

Direct farm damage	$707,000
Indirect farm damage	218,000
Total damage	$925,000

LARGE IRRIGATION SYSTEMS

Kind of damage considered -- Up to 1949 the Roosevelt Water Conservation District has invested almost $80,000 to protect its irrigation system. That the protection afforded has not been adequate is shown by the fact that during the period from 1930-1945 the district spent about $55,000 for canal repairs necessitated primarily by floods. One hundred twenty-seven breaks were made in the main canal during the 1933 flood. Because of the existing flood hazard, no attempt has been made to repair the canal lining and water losses by seepage have increased. Loss of irrigation water because of seepage, which occurs to some extent every year irrespective of floods, is considered as an indirect damage.

Direct damage to irrigation systems -- Direct damage includes damage to irrigation systems such as canals and laterals, structures, and pumping equipment and wells. It also includes damage to flood control structures built by the irrigation companies to protect the main canal. On the basis of experienced damages, direct damage to irrigation works varies from an estimated $41,000 for an average small flood to $139,000 for a great flood.

Indirect damage to irrigation systems -- Officials of the Roosevelt Water Conservation District indicate that the present flood hazard makes it infeasible to replace canal lining washed out by floods. As a result, seepage losses have increased by an estimated 800 acre-feet annually. Because this loss must be made up by additional pumping, the cost of pumping is used as the basis for evaluating this type of damage. The cost of pumping (1948) is estimated at 4 cents per acre-foot of lift. For an average lift of 190 feet the cost is estimated at $7.60 per acre-foot. The total cost for pumping 800 acre-feet is estimated at $6,800 annually. No determination is made of the effect of the additional pumping on ground-water level. Interruption of irrigation services because of damage to irrigation systems is included as an indirect farm damage.

DAMAGE TO HIGHWAYS & ROADS

Extent of highway and roads -- About 200 miles of road are subject to flood damage. Farm roads are located at mile intervals. Many miles of these roads are hard surfaced. One Federal highway and one State highway traverse the area.

Damages to highways and roads -- Highways and roads leading away from canals from water courses for flood flows when the canals are over-topped. These flows cause some damage to the surface of the road, and more important, they do considerable damage to the shoulders of the road. Indirect damage are important. Highwater and washouts necessitate detours. Damaged roads resulting from a flood add to the cost of traffic moving over them. The road and highway damages are estimated to vary from $93,000 for a great flood to $9,300 for a small flood. About one fourth of these amounts are indirect damages. A small flood will isolate Williams Air Force Base for 24 hours or more causing considerable inconvenience to employees who commute to and from work.

DAMAGE TO URBAN AREA

Communities damages -- The towns Higley and Gilbert have been flooded in the past. Although Higley is flooded by practically every flood, damage is nominal because of the low property valuation. Gilbert's most serious flood since the construction of the Roosevelt Water Conservation District canal occurred in 1933 when damages were over $20,000. Of the total, $2,200 was indirect damage. The 1933 flood is classed as a small one, but past experience shows that all small floods do not damage or even reach Gilbert. It is possible that the dike above Roosevelt Canal, constructed since 1936, is sufficient to prevent some small flood flows from reaching Gilbert. The 1946 flood with a low runoff and low peak flow was intercepted at the outskirts of Gilbert by the Eastern Canal. It is probable that not over the two small floods in 100 years would cause damage. Large floods would cause considerable damage in urban areas. The estimates urban damage varies from $4,600 for a small flood to $98,000 for a great flood. About 15% of this amount is indirect damage.

DAMAGE TO RAILROADS & OTHER UTILITIES

Extent of damage -- Power lines and telephone lines, although rather numerous throughout the flood damage area, are not subject to severe damage. Occasionally, a few poles are washed out or loosened to the extent they have to be reset. This repair cost, however, is nominal. Some damage has occurred on the Mesa-Magma branch line of the Southern Pacific Railroad. The 1946 flood softened the roadbed near Higley. A major flood would do considerable damage to a section of this line. Such a flood would probably also damage the Southern Pacific main line near Serape, south of Chandler. The estimated damage to railroads and other utilities varies from $500 for a small flood to $40,000 for a great flood.

DAMAGE TO AIR FORCE INSTALLATIONS

Williams Air Force Base -- This military base is located at the mouth of Sand Tank Wash. Dikes are now being planned to divert floodwaters around the airfield. Maintenance of the dikes, throughtout strictly a floodwater damage, entails additional costs necessitated by the flood hazard. The necessity for these dikes would be eliminated by the recommended flood control program. Therefore, the annual maintenance cost estimated at $600 per year is considered a floodwater damage.

DAMAGE BY FLOODS IN LOWER QUEEN CREEK

Floodwater damages from Lower Queen Creek -- The proceeding estimates of floodwaters damage includes only those damages arising from flood flows from the Superstition-Bulldog area and Santan Mountains, the latter flows damaging Chandler Heights. The flows from Sonoqui Wash intermingling with flows from Lower Queen Creek (below the authorized Whitlow Dam) cause additional damage estimated at $11,900 annually. The estimate is based on a preliminary estimate by the Corps of Engineers of $11,000 annually, based on 1946 prices. This estimate was adjusted to 1948 prices levels by the use of price and cost indexes. These damages are distributed among the various types in the tabulation summarizing floodwater damages.

AVERAGE ANNUAL FLOODWATER DAMAGE

Summary of floodwater damage -- The estimate of average annual floodwater damage is derived from damage-frequency curves. Damage-frequency curves were developed by plotting monetary flood damages against their respective frequencies as shown in the preceding section on hydrology. Total floodwater damage in the watershed, exclusive of those that will be prevented by the authorized Whitlow Dam, amount to an estimated $135,000 annually as follows:

Type of Floodwater Damage	Average Annual Damage		
	Direct	Indirect	Total
Agricultural			
Farms	$ 70,590	$ 21,160	$ 91,750
Irrigation systems	18,770	6,080	24,850
Subtotal	$ 89,360	$ 27,240	$116,600
Non-agricultural			
Highways & roads	$ 6,100	$ 2,000	$ 8,100
Urban & rural non-farm property	6,830	1,040	7,870
Railroads & other utilities	1,460	420	1,880
Military bases	600	---	600
Subtotal	$ 14,990	$ 3,460	$ 18,450
Total annual floodwater damages	$104,350	$ 30,700	$135,050


LEGEND
S.C.S. PROPOSED STRUCTURES
PROPOSED HUNT RD. DIVERSION
PROPOSED DRANAGE CHANNEL
PRESENT CITY LIMITS
1980 CITY LIMIT
2010 CITY LIMIT
PRIVATE
CULTIVATED
URBAN
STATE
FEDERAL
FOREST
TONTO NATIONAL FOREST
MARICOPA CO.
PINAL CO.
KEY MAP
SCALE
1 MILE
LAND USE MAP
& PROJECTION OF
URBAN AREAS OF BASIN
FLOOD CONTROL REPORT
AREA I
APPENDIX I-B PLATE NO. 1

FLOOD CONTROL SURVEY AND REPORT

STUDY AREA NO. I

SOUTHEASTERN MARICOPA COUNTY

APPENDIX I-C

PROPOSED ARTIFICAL RECHARGE ASPECTS

OF FLOOD CONTROL SURVEY

FLOOD CONTROL DISTRICT

MARICOPA COUNTY

Prepared by
Benham Engineering Company Inc.
Consulting Engineers
Phoenix, Arizona

APPENDIX I-C

PROPOSED ARTIFICAL RECHARGE ASPECTS

OF FLOOD CONTROL STUDY

TABLE OF CONTENTS

APPENDIX I-C

PROPOSED ARTIFICIAL RECHARGE ASPECTS OF FLOOD CONTROL SURVEY FOR SOUTHEASTERN MARICOPA COUNTY

INTRODUCTION

1. General: One of the most critical problems in the Southwest at present is the diminishing ground water supply. The year-to-year decline in volume of water available - the average yearly overdraft on the ground water reserves in Arizona alone is approximately four million acre-feet - is illustrative of the inability of precipitation to naturally recharge water to the aquifers at a rate commensurate with the withdrawal. In order to maintain even the present population and prevent further land subsidence in this region, a method must be found to slow or stop the depletion of the ground water supply. Artificial ground water recharge can be the method by which this is accomplished.

2. Runoff: At present, flood runoff generally flows uncontrolled overland and down natural desert drainages causing, at times, considerable damage en-route. Very little, if any, beneficial use is made of the flood runoff on the ground surface, and probably only a small amount ever reaches the ground water table. Most of the water is lost by evapotranspiration. To control flood run-off by means of detention basins, and put excess water underground by artificial means would in some cases reverse the downward trend of the water table, and in all areas slow the rate of depletion of ground water reservoirs.

3. Sewage Effluent: Artificial recharge facilities can be planned so sewage effluent can be given minimum treatment and handled along with flood runoff by means of these facilities.

4. Other Benefits: Other arguments for underground storage of water suggested by our new mode of life are namely: There is less danger from fallout contamination and bombing, and sabotage would be more difficult.

GEOLOGICAL ANALYSIS OF THE AREA

1. General: In this section a geological analysis of the area is made with regard to method of investigation and general description of the geology.

2. Method of Investigation:

a. General: Geological maps of Maricopa and Pinal Counties published by the Arizona State Bureau of Mines provide information on the broad geological setting of the area of study. These maps, however, fail to provide sufficiently detailed information on the nature of the Quaternary sedimentary fill which would be the recharge aquifer. To study the fill and locate suitable recharge aquifers, drillers logs of water wells throughout the area were studied. Approximately 600 logs were reviewed and 190 selected for use. Logs typical of those selected are given in Appendix C, Table No. 3. These logs were obtained from the files of the U. S. Geological Survey, Arizona State Land Department, and the Chandler Heights Irrigation District. Location of the wells used are shown in Appendix C, Plate No. 1.

b. Drillers Logs: In reviewing the drillers' logs, special emphasis was placed on locating a sedimentary facies near land surface which is coarse textured and covers a broad area. Such a unit was sought because it could be easily recharged and could transmit large quantities of water. Isopach maps, Appendix C, Plates No. 1 and 2 give the depth of overburden and thickness of the gravel bed selected. Contours of the bottom of the gravel unit indicate the geologic structure of the area. Appendix C, Plate No. 4.

c. Shallow Borings: Accuracy of the isopach map of the overburden was checked by 10 shallow borings made in areas where recharge facilities appeared feasible. In all cases where the boring could be completed, the gravel bed was intersected by the drill at the depth indicated by the map, or at a slightly shallower depth. (Appendix C, Table No. 1). Logs of these borings are given in Appendix C, Table No. 2.

3. General Geological Description: The general description of lithology and geologic structure is presented in the following sections for parts of the area under survey where sufficient data are available.

a. Lithology: Drillers' logs indicate the unit selected for recharge to be a clean gravel locally mixed with small amounts of finer grained material. Near Mesa this description was found true in borings into the gravel.

West of the Town of Queen Creek, the unit chosen was described by drillers as a sand. Moderate grading of the unit is indicated in this area by a greater volume of finer grained intermixed sediment. This description was confirmed by bore hole information. East of the Town of Queen Creek and along Queen Creek both drillers' logs and additional bore holes indicate a rapid coarsening of sediment toward the head waters of Queen Creek. This change is not accompanied by better sorting, although grading of the sediments is not as pronounced and results in less fine grained sand and silt with the gravel.

Locally, throughout the unit selected silt and clay beds occur among the gravels. These sedimentary facies are so small and irregular in extent they could not be traced using logs of water wells, except in the area north of Queen Creek in T 2 S, R 7 E. Here, silt and clay beds are more prominent in the unit selected and are shown on a geologic cross section drawn through the area. (Appendix C, Plate No. 5).

b. Structure: Structurally the unit follows the general topographic fall of the land surface from east to west. This feature is shown on geologic sections in Appendix C, Plate No. 5. The true profile of land surface is not on the cross sections, but rather, points of equal thickness of overburden, aquifer material, and structure of the bottom of the aquifer are shown. These features were determined from Appendix C, Plates Nos. 2, 3, and 4.

Folds and faults in the area qould not be detected because of the nature of the sediments and type of subsurface information available.

c. Volume of the Aquifer -

From data given in Appendix C, Plate No. 3 and depth to water data available from the Geological Survey, the following hydrologic characteristics have been determined for various sections of the recharge unit.

Area	Extent (mi^2)	Average Thickness (ft)	Volume ($10^9 ft^3$)	Assumed Specific Intake	Recharge ($10^9 ft^3$)	Recharge (10^3 acre-ft)
Queen Creek	30	50	45	15%	7	160
Higley	20	50	30	20%	4*	90*
Salt River-Gilbert-Chandler-Mesa	110	100	310	20%	62	1,400

*Estimated one-third decrease in recharge capacity based presence ground water

For the area south of the Salt River Mountains insufficient data is available to make an interpretation of the geologic structure and lithology of the subsurface sediments. However, logs of the few water wells drilled in the area indicate an extention of the recharge unit to this locale where it is exposed at land surface. Based on the presence of the recharge aquifer and depth to water data taken from one well, we believe ample recharge capacity exists in this area.

HYDROLOGIC ANALYSIS

1. General: The hydrologic data used in determining size of recharge facilities is shown in Appendix C, Table No. 4.

Recharge rates and design characteristics for dry and wet pits suggested for use (Appendix C, Plates No. 6&7), are based on results obtained through laboratory and field tests carried out in Arizona and elsewhere in the United States by governmental and private agencies. However, the results of research under way are needed to determine whether or not a pea gravel filter and developed "schmutzdecke" in the wet pits can operate successfully under prevailing conditions. The chemical and physical characteristics of the filter and/or flood water may require alteration to make filtering effective.

2. Cotton Gin Trash: Tests by the Agricultural Research Service, United States Department of Agriculture in California has shown that cotton gin trash incorpor-

ated into the top layer of soil increases infiltration rates about five fold for a seven-month period of flooding (3). The organic matter must be decomposed, however, before it is effective (2nd year) and then remains effective for about five years.

3. Mesa-Chandler Area: The design recharge rates and optimm yields, which is essentially sewage effluent, for the facilities proposed for the Mesa and Chandler Recharge Areas are based on sewage effluent rates adapted from the Tucson system, and estimated flood runoff which could be handled by these facilities.

4. Stage Development: The recharge rates suggested, Appendix C, Plate No. 6, are conservative when compared with rates experienced elsewhere; hence the facilities should be developed in stages at each location to determine actual site requirements with regard to capacity.

PROPOSED RECHARGE FACILITIES

1. General: The proposed recharge facilities to be discussed in this section fall into two categories. The first group are facilities to handle controlled flood runoff from detention basins providing, besides the very important function of conserving water, a means of disposing of flood water. The second group are facilities which would not only handle treated waste water such as domestic and industrial sewage effluent and tail-water from irrigated areas, but would also conserve undetained flood runoff during the short periods when flow is available. It is realized that only a small reduction is possible in flood peak discharge and volume.

a. Recharge Areas: The recharge areas have been selected after careful consideration of the availability of water,geology, value of land, utility, and possible beneficial use of water recharged.

b. Chlorination: Chlorination is required to kill microorganisms and microflora which cause a reduction in infiltration rates with time. For dry pits recharging only short periods of time, occasional heavy doses of chlorine would probably be satisfactory. However, for wet pits, (Appendix C, Plate No. 6), recharging constantly, a steady application of 3 to 4 ppm chlorine would be required.

2. Primary Proposed Recharge Facilities: The primary proposed recharge facilities for handling detained flood runoff would be developed in the following four areas.

a. Upper Queen Creek Recharge Area: This area would encompass T 2 S, R 8 E, Section 21, and east one-third Section 20. The design characteristics of the recharge pits are given in Appendix C, Plate No. 7 and Appendix C, Plate No. 6. The development of this recharge area has potential beyond just the conservation of flood runoff from Queen Creek in that flows from the Soil Conservation Service (SCS) detention basins south of Apache Junction could be recharged in this area, possibly alleviating the load in Roosevelt Canal when overtaxed. Further, if desired, water could be diverted from the Central Arizona Project Aqueduct at this point and recharged. Besides providing flood control aspects the recharged water replenishes badly depleted ground water basins which supply cities like Chandler Heights and Queen Creek, and irrigation districts in the area. (Appendix C, Plate No. 5).

b. Upper Salt River Recharge Area: This area would consist of a strip 300 feet wide and two miles long on the left bank of the Salt River starting west of the outfall of the flood channel draining water from the SCS detention basins north of Apache Junction. A diversion structure on the channel would direct water into the recharge pits. The design characteristics of the recharge pits are given in Appendix C, Plate No. 7 and Appendix C, Plate No. 6. Recharging the water by means of pits is recommended as opposed to emptying directly into the Salt River since the water can be placed deep enough in the aquifer to prevent unbeneficial evapotranspiration losses. Water could also be diverted from the Central Arizona Project Aqueduct at this point and recharged. Further, if desired, water could be released from the Salt River Project reservoir system and recharged in this area.

c. Lower Queen Creek Recharge Area: This area would utilize land in T 2 S, R 6 E, between the Roosevelt Canal and Queen Creek in Sections 10, 11, 15, and 22. The design characteristics of the recharge pits are given on Appendix C, Plates No. 6 and 7. Besides handling detained flood runoff from the Santan Mountains area (above Hunt Highway) and water released into the Roosevelt Canal from SCS detention basins south of Apache Junction, this area could be used for recharging sewage effluent and part of the uncontrolled flood runoff from Williams Air Force Base. Further, if desired, additional water from the Roosevelt Canal could be recharged in this area. If the flood runoff from the SCS detention basins south of Apache Junction are not released for recharge, the facilities in this area should be designed for flood runoff from the Santan Mountain area and Williams Air Force Base. Again, besides providing flood control aspects, the water recharged replenishes badly depleted ground water basins which supply cities like Chandler Heights and irrigation districts in the area.

d. Salt River Mountains Recharge Area: This area would be located in T 2 S, R 3 E, southwest corner Section 21. The design characteristics of the recharge pits are given in Appendix C, Plates No. 6 and 7. Besides handling controlled flood runoff from the Salt River Mountains, this area could be used to recharge sewage effluent from Mesa, Gilbert, and Chandler and part of the undetained flood runoff in the Gila Drain. Further geologic exploration in this area is strongly urged because, if the aerial extent of the gravel recharge unit exposed at land surface is considerable, low-cost basins could be constructed to recharge large quantities of flood runoff diverted from the Gila Drain. Water recharged at this point would replenish ground water basins which supply wells in this area.

3. Secondary Proposed Recharge Facilities: The secondary proposed recharge facilities for handling treated waste water such as domestic and industrial sewage effluent (assuming detergents can be removed), irrigation tail-water, and part of the undetained flood runoff could be developed in the following two areas.

a. Mesa Recharge Areas: Because of geologic considerations these recharge areas would have to be located in T 1 N, R 5 E, north central edge of Section 31, and in T 1 S, R 5 E, northeast corner of the northwest quadrant of Section 4. Both of these locations are within the projected future development of the City of Mesa. The design characteristics of the recharge facilities are given in Appendix C, Plates No. 6 and 7. Because of the continuous flow of sewage effluent, the quantity of water that can be recharged is quite large in comparison with quantities handled by facilities primarily concerned with flood control. Undetained flood runoff from Mesa could in part, however, be recharged by these facilities. The water recharged would benefit cities like Mesa and Chandler, and irrigation districts in the area.

b. Chandler Recharge Area: Because of geologic conditions in this area consideration should be given to recharge by shafts instead of pits. A suggested system might consist of a 15 acre grass covered park located west of Chandler. The park should be graded to drain into a one acre lake which is underlain by a pea gravel filter. Filtered water would move downward through the pea gravel into collection pipes which would supply gravel back-filled shafts drilled through 80 feet of overburden and 140 feet of recharge unit. The controlled filtration rate for an acre of pea gravel is 20 acre-feet per day (2). Thus four feet of flood runoff (60 acre feet) detained in the park area would be recharged in a three day period. In addition, treated sewage effluent from Chandler could be handled by this recharge facility. The water conserved would be of direct benefit to Chandler and irrigation districts in the area. (Appendix C, Plate No. 5).

ECONOMIC ANALYSIS

1. General: The following economic analysis is based on estimations and assumptions. This analysis, however, does indicate overall economic feasibility of ground water recharge in the survey area.

2. Determination of Cost Per Acre-Foot of Water Recharged: For determination of cost per acre-foot of water, the lower Queen Creek Recharge Area was selected as an area which would reflect the average conditions of all recharge areas. (Appendix C, Plates No. 6 and 7.

50 Year Economic Life

First Cost:

Cost of Land: 230 ac. @ $500/ac		$ 115,000
Cost of excavating recharge pits @ $0.15/yd		252,000
Intake channel and control structures		75,000
	Sub-total	$ 442,000
Contingencies @ 10%		44,000
	Sub-total	$ 486,000
Engr. and Admin. @ 10%		49,000
	Sub-total	$ 535,000
Interest during construction @ 6%		35,000
	Grand Total	$ 570,000

Annual Cost:

Amortization and Interest @ 3%		$ 22,100
Operation & Maintenance		
Every 10 yrs remove 0.5 ft sediment and 0.5 ft affected aquifer from pits		$ 1,000
Conditioning of pits with gin trash every 5 yrs		$ 500
Chlorination @ $1.00/ac ft of water recharged		$ 5,000
Administration		$ 1,000
	Sub-total	$ 29,600
Misc. @ 5%		1,500
	Grand Total	$ 31,100
Cost per acre-foot of water recharged (4,530 ac ft of water recharged per yr, Appendix C, Plate No. 6 -		$ 6.87

3. Cost-Benefit Ratio: The cost-benefit ratio calculations are based upon the following assumptions and estimations.

a. The cost of operation and maintenance, administration. etc., remains the same during the economic life.

b. Cost to lift 1 ac ft/ft of lift is $0.04.

c. Present average lift in survey area is 250 ft (based of US Geol. Survey data).

d. Drop in ground water levels (1) in the survey area is 5 ft/yr.

e. Overdraft on ground water reserves in survey area is 100,000 ac ft/yr.

f. Volume of water to be recharged artifically (Appendix C, Plate No. 6) is 20,000 ac ft.

Volume of above water which would be recharged naturally - 10,000 ac ft/yr.

Volume of water to be effectively recharged artifically - 10,000 ac ft/yr.

g. Decrease in drop in ground water levels due to effective artificial recharge: $\frac{10,000}{100,000}$ X 5 = 0.5 ft/yr.

h. Total volume of water pumped in survey area is 700,000 ac ft/yr.

i. Benefit first year from decreased lift: 0.04 X 0.5 = 0.02 ac ft.

Total for water pumped = 0.02 X 700,000 = $ 14,000

Benefits second year: 0.04 X 1.0 = 0.04 ac ft.

Total for water pumped = $0.04 X 700,000 = $28,000

j. Cost for artificial recharge: 20,000 X $ 7.00 = $ 140,000 / yr.

k. Cost for artificial recharge per ac ft of water pumped in the survey area: $140,000/700,000 = $0.20/ ac ft.

As seen from items i and k above and in Appendix C, Plate No. 8, a deficit will occur during the first ten years of the project if interest is disregarded. The benefits, reduction in pumping costs due to the decreasing rate of water table decline, are equal to the annual cost of recharge at the end of this ten years.

An interest rate of 5% was used in the present worth analysis shown in Appendix C, Table 5, to obtain a benefit-cost ratio of about 3:1. The benefit-cost ratio will depend upon the interest rate used.

4. Discussion:

a. Assumed and estimated values used in the above economic analysis are believed to be of a conservative nature.

b. The benefits which would be realized through prevention of flood damages and saving in cost of channels by artificial recharge of flood runoff have not been considered in the above analysis.

c. Further, the benefits which would be realized because agriculture in the survey area could remain in operation at the present scale for four extra years because of the delay in the drop of the water table below the economic level have not been considered in the above analysis.

SUMMARY AND CONCLUSIONS

1. The presence of the excellent coarse-grained recharge unit, with limited overburden in many areas, presents the opportunity for pit recharge, which is certainly one of the better methods available with regard to economic and technical considerations.

2. Artificial ground water recharge, when used in conjunction with detention facilities, can be effective for disposing as well as conserving flood runoff.

3. The design rates used are conservative where compared with rates experienced elsewhere. Hence, the facilities should be developed in stages to determine actual site requirements with regard to capacity.

4. The recharge areas have been selected after careful consideration of the availability of water, geology, value of land, utility, and possible beneficial use of water recharged.

5. A complete economic analysis should be made; however, the analysis presented although based on numerous estimations and assumptions, which are believed to be on the conservative side, indicates that artificial recharge as presented is economically favorable.

REFERENCES CITED

1. Hardt, W. P. Stulik, M.S., and Bocher, M.B., 1960 Report on Water in Arizona, Spring 1959 to Spring 1960: U.S. Geol. Survey - Arizona State Land Dept. Water Resources Report No. 7, P. 36-43

2. Feiple, L. R., 1959, Effectiveness of Coarse-Grained Media for Filtration: Journal Amer. Wtr. Wks. Assn., Vol. 51, pp. 749-760.

3. Shiff, L., 1956, The Darcy Law In The Selection of Water-Spreading Systems For Ground-Water Recharge: Extract from Publication No. 41 of The International Hydrology Associations' Darcy Symposium, p. 99-110.

4. Sutter, M., 1956, The Peoria Recharge Pit; Its Development and Results; Journal Irrig. and Drain. Div., Am. Soc. of Civil Engr., Vol. 82, IR3, pp. 1102-17.

APPENDIX C

TABLES

PLATES

APPENDIX C

TABLE NO. 2

Test Bore Hole Logs

APPENDIX C

TABLE 1

Test Bore Hole Results:

No.	Location	Predicted Depth To Gravel (ft)	Actual Depth To Gravel(ft)	Remarks
1.	T1S, R5E Sec 9 bcb*	40 - 50	48	
2.	T1S, R5E Sec 7 cbb	40 - 60	44	
3.	T1N, R5E Sec 33, caa	20 - 30		Unable to penetrate caliche below 15' depth.
4.	T2S, R7E Sec 19, bbb	20 - 40	22	
5.	T1N, R5E Sec 33, daa	20 - 40		Unable to penetrate caliche below 19' depth.
5b.	25 ft. West of #5	20 - 40		Unable to penetrate caliche below 19' depth.
6.	T2S, R7E Sec 17, ccd	20 - 30	25	
7	T2S, R7E Sec 25, abb	10 - 20	20	
8	T2S, R6E Sec 10, ddd	20 - 40	18	
9	T2S, R8E Sec 22, bcc	10 - 20	15	
10	T2S, R9E Sec 20, a	-	45	

* Lower case letters indicate location with section according to the U. S. G. S. well numbering system.

APPENDIX "C"

TABLE NO. 2

Test Bore Hole Logs

#1 Location: T1S, R5E, Sec. 9, bcd
Date: Nov. 28, 1961

Depth Below Ground Surface (ft.)	Lithology
0 - 15	Sand with silt
15 - 20	Caliche cemented sand & gravel
20 - 24	Clay with caliche
24 - 47	Clay with caliche & some pebbles
47 - 48	Medium Grained Sand
48 - 49	Medium sand with coarse gravel (1/2-3/4")
49 - 51	Medium sand with gravel & cobbles (2")

#2 Location: T1S, R5E, Sec. 7 cbb
Date: Nov. 28, 1961

0 - 12	Silty sand with clay
12 - 18	Sand & gravel with caliche
18 - 28	Clay with caliche
28 - 37	Clay with caliche & pebbles
37 - 44	Medium grained sand with silt
44 - 50	Gravel (1/2-3/4") Cobbles (2'-4") & coarse sand

#3 Location: T1N, R5E, Sec. 33, caa
Date: Nov. 28, 1961

0 - 5	Sandy silt
5 - 10	Silty sand
10 - 13	Sand
13 - 15	Caliche cemented medium sand, gravel (1/2-3/4") & cobbles (3")

Stopped drilling at 15 ft. because of hardness of caliche.

#4 Location: T2S, R7E, Sec. 19,bbb
Date: Nov. 29, 1961

Depth Below Ground Surface (ft.)	Lithology
0 - 10	Silty sand
10 - 13	Silty sand
13 - 17	Sand with clay
17 - 22	Sand with clay & caliche
22 - 30	Fine to medium grained sand At 27' - 1' of clay & caliche mixed with the sand, but at 29' sand became coarse & was mixed with pea gravel.

#5a Location: T1N, R5E, Sec. 33,daa
Date: Nov. 28, 1961

0 - 4	Sand with silt
4 - 14	Clay & sand with caliche
14 - 18	Sand cemented with caliche
18 - 19	Sand, gravel & cobbles cemented with caliche
	Stopped drilling at 19' because of hardness of caliche

#5b Location: T1N, R5E, Sec. 33,daa - 25 feet west of test hole #52.
Date: Nov. 28, 1961

0 - 10	Clay with sand
10 - 12	Clay with sand & caliche
12 - 15	Hard caliche cemented sand
	Stopped at 15' because of hardness

#6 Location: T2S, R7E, Sec. 17,ccd
Date: Nov. 29, 1961

0 - 9	Silty sand
9 - 12	Silty clay with sand
12 - 17	Medium to fine grained sand with pea gravel (red).
17 - 25	Silty clay with some caliche
25 - 30	Medium to fine grained sand with clay lenses up to 8" thick

TABLE NO. 2 (Cont.)

#7 Location: T2S, R7E, Sec. 25,abb
Date: Nov. 29, 1961

Depth Below Ground Surface (ft.)	Lithology
0 - 2	Silty sand
2 - 13	Medium grained sand with layers of silt up to 3" thick
13 - 18	Cobbles, gravel & sand with some silt
18 - 20	Clay & silt with pebbles
20 - 30	Sand with pebbles

#8 Location: T2S, R6E, Sec. 10,ddd
Date: Nov. 29, 1961

0 - 18	Silty sand
18 - 25	Dry loose, medium to coarse sand with some gr[illegible]
	Stopped drilling because of caving.

#9 Location: T2S, R8E, Sec 23,bcc
Date: Nov. 29, 1961

0 - 15	Silty sand
15 - 23	Cobbles, gravel & sand with some silt which decreased with depth.

#10 Location: T2S, R9E, Sec. 20, a
Date: Nov. 29, 1961

0 - 4.5'	Silty sand
4.5 - 7	Boulders with sand & silt
	Stopped drilling at 7' because of coarseness of material.

APPENDIX C

TABLE NO. 3

Logs of Representative Wells

TABLE NO. 3

Logs of Representative Wells:

Location: (A-1-6) 7 ccb
Source of Well Log: U. S. Geological Survey-Ground Water Branch

Depth Below Surface (ft.)	Lithologic Description
0 - 2	Soil
2 - 9	Clay & caliche
9 - 11	Caliche & gravel
11 - 52	Boulders
52 - 92	Sand, gravel & Boulders
92 - 98	Gravel & clay
98 - 206	Gravel & cemented gravel & sand
206 - 216	Clay

Location: (A-1-6) 36 aaa
Source of Log: USGS - Ground Water Branch

0 - 195	Sand & clay
195 - 220	Boulders
220 - 240	Sand
240 - 280	Boulders
280 - 320	Clay
320 - 450	Sand & clay

TABLE NO. 3 (cont.)

Location: (D-1-4) 14 ccc
Source of Log: USGS - Ground Water Branch

Depth Below Surface (ft.)	Lithologic Description
0 - 15	Soil
15 - 60	Caliche
60 - 174	Boulders
174 - 182	Clay & Boulders
182 - 212	Clay
212 - 223	Cemented sand & boulders
223 - 261	Clay

Location: (D-1-5) 24 dbb
Source of Log: USGS - Ground Water Branch

0 - 15	Soil
15 - 52	Hard clay
52 - 100	Sandy clay
100 - 102	Hard clay
102 - 128	Caliche
128 - 185	Conglomerate
185 - 200	Cemented gravel
200 - 215	Sand & boulders
215 - 275	Boulders
275 - 300	Clay
300 - 315	Sand & boulders
315 - 520	Clay

TABLE NO. 3 (cont.)

Location: (D-1-6) 15 baa
Source of Log: USGS - Ground Water Branch

Depth Below Surface (ft.)	Lithologic Description
0 - 20	Caliche
20 - 25	Sand
25 - 60	Clay
60 - 65	Caliche
65 - 87	[illegible]
87 - 110	Hard caliche
110 - 113	Sandy clay
113 - 142	Gravel & boulders
142 - 146	Clay
146 - 166	Clay & caliche
166 - 288	Sand, gravel & boulders
2[illegible] - [illegible]	Sandstone & clay
3[illegible] - [illegible]	Sand & gravel
340 - 383	Clay
383 - 386	Sand & gravel
386 - 390	Clay
390 - 396	Sand
396 - 41[illegible]	Clay

TABLE NO. 3 (Con.t)

Location: (D-2-3) 1 aaa
Source of Log; USGS - Ground Water Branch

Depth Below Surface (ft.)	Lithologic Description
0 - 3	Soil
3 - 23	Sandy clay
23 - 26	Sand
26 - 40	Caliche
40 - 50	Sand, clay & gravel
50 - 93	Sandstone
93 - 211	Large gravel & boulders
211 - 215	Clay
215 - 242	Sand, gravel & boulders
242 - 268	Clay
268 - 288	Coarse gravel
288 - 371	Clay with hard shells

Location: (D-2-3) 20 dbc
Source of Log: Arizona State Land Department

0 - 4	Soil
4 - 8	Caliche
8 - 16	Fine sand
16 - 30	Caliche
30 - 49	Coarse sand
49 - 163	Gravel
163 - 176	Clay & sand
176 - 190	Gravel
190 - 196	Clay & sand
196 - 202	Fine sand
202 - 220	Clay & caliche
220 - 231	Fine sand
231 - 232	Clay

TABLE NO. 3 (cont.)

Location (D-2-6) 11 bc
Source of Log: USGS - Ground Water Branch

Depth Below Surface (ft.)	Lithologic Description
0 - 7	Soil
7 - 17	Hardpan
17 - 36	Dry sand & gravel
36 - 60	Caliche
60 - 74	Dry sand & gravel
74 - 100	Clay with gravel lenses
100 - 107	Sand & gravel
107 - 136	Clay & caliche
136 - 143	Sand & gravel
143 - 149	Clay.
149 - 160	Sand
160 - 162	Hard cement
162 - 206	Clay & caliche
206 - 220	Hard caliche
220 - 301	Clay & caliche

Location: (D-2-7) 15 da
Source of Log: USGS - Ground Water Branch

0 - 3	Silt
3 - 20	Sandy clay
20 - 24	Gravel
24 - 36	Clay & conglomerate
36 - 42	Clay with gravel
42 - 46	Sand
46 - 52	Clay & gravel
52 - 60	Clay streaks
60 - 69	Sand & gravel

TABLE NO. 3 (CONT.)

Location: (D-2-7) 15 da (cont'd)
Source of Log: USGS - Ground Water Branch

Depth Below Surface (ft.)	Lithologic Description
69 - 92	Conglomerate with clay
92 - 108	Clay & caliche
108 - 114	Sandy clay
114 - 126	Gravel
126 - 135	Clay & sand
135 - 160	Sandy clay with conglomerate streaks
160 - 165	Gravel to 3"
165 - 168	Clay & caliche
168 - 183	Gravel to 5"
183 - 193	Clay
193 - 224	Sand & gravel to 2"
224 - 237	Clay
237 - 245	Sand & gravel to 2"
245 - 270	Clay & caliche
270 - 278	Sandy clay
278 - 302	Sandy clay
302 - 307	Clay & gravel to 1"
307 - 316	Sandy clay & gravel
316 - 342	Clay
349 - 355	Sand & gravel to 1"
355 - 378	Clay
378 - 382	Fine sand
382 - 389	Gravel to 1"
389 - 440	Clay

TABLE NO. 3 (cont.)

Location: (D-2-7) 31 abc
Source of Log: Chandler Heights Irrigation District Office

Depth Below Surface (ft.)	Lithologic Description
0 - 15	Sandy clay
15 - 75	Sand & gravel
75 - 135	Sandy clay & gravel
135 - 280	Sand & gravel
280 - 345	Sandy clay
345 - 380	Sand, gravel & clay
380 - 385	Granite wash
385 - 410	Sandy clay
410 - 430	Clay
430 - 470	Sand - hard
470 - 545	Cemented sand & rock
545 - 605	Gravel, sand & rock
605 - 655	Sand, clay & gravel
655 - 675	Sand & granite wash
675 - 730	Sand & gravel

Location: (D-2-8) 21 d
Source of Log: USGS - Ground Water Branch

0 - 5	Soil
5 - 12	Clay & gravel
12 - 115	Gravel & boulder
115 - 134	Clay & gravel
134 - 210	Dry gravel
210 - 225	Clay
225 - 230	Clay
230 - 264	Gravel with cemented sand at the top
264 - 340	Clay

APPENDIX C

TABLE NO. 4

Reservoir Yield Data and Recharge Water Siltation Loads

Area	Drainage Area (square miles)	Average Annual Runoff (acre-feet)	Optimum Yield (acre-feet)	Design Recharge Rate (c.f.s.)	Silt Load (ac - ft.) per yr.
SCS #1 (south of Apache Junction)	133	4,000	3,680	160	2.1
SCS #2 (north of Apache Junction)	40	1,240	1,130	160	.7
Whitlow Reservoir	143	6,480	6,100	830*	3.7
Queens Creek (above 1600' contour including Whitlow Reservoir)	217	9,100	8,700	1260	5.2
Santan Mountain Area (above Hunt Highway)	30	930	850	140	.5
Salt River Mountains Area (above High Line Canal)	34	1,180	1,090	140	.7
Total	454	16,450	15,450	[illegible]	9.2

*Design discharge from Whitlow Reservoir represents inflow in Queen Creek Detention & Debris Barrier Reservoir, the design discharge for which is shown on Line 4.

APPENDIX C

TABLE 5

Present Worth Analysis Giving Cost-Benefit Ratio With Interest @ 5%

		Net Cost	Present Worth Factor	Accum. Present Worth
1960:	1	0.20	0.952	0.190
	2	0.18	0.907	0.354
	3	0.16	0.864	0.492
	4	0.14	0.823	0.607
	5	0.12	0.783	0.701
	6	0.10	0.746	0.776
	7	0.08	0.711	0.833
	8	0.06	0.677	0.873
	9	0.04	0.645	0.899
1970:	10	0.02	0.614	0.911
	11	0.00	0.585	---

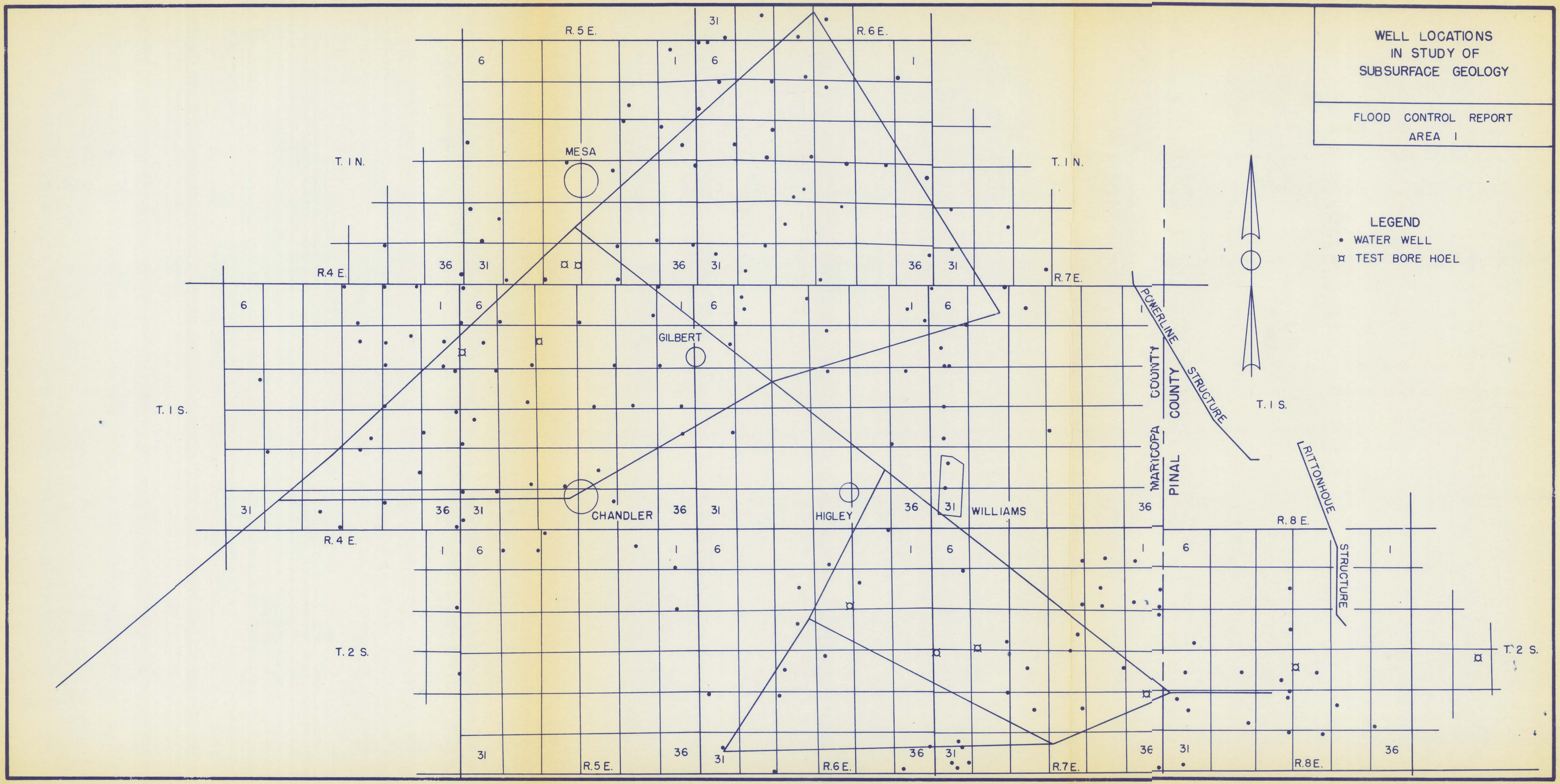
Total Present Worth
of Net Costs = 0.911

Cost-Benefit Ratio = 2.817 ÷ 0.911

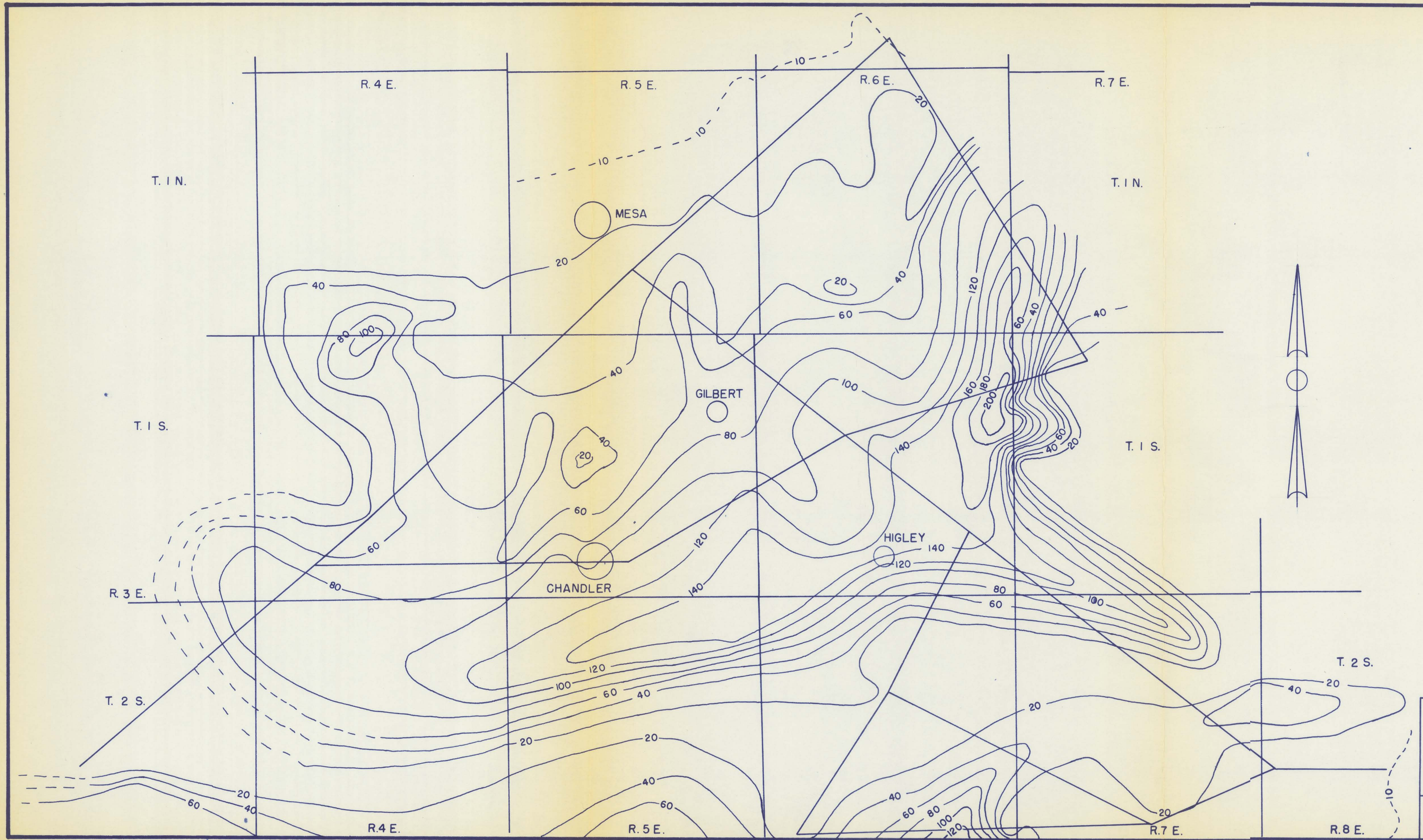
= 3.09 or 3

		Net Cost	Present Worth Factor	Accum. Present Worth
	11	0.00	0.585	0.000
	12	0.02	0.557	0.011
	13	0.04	0.530	0.032
	14	0.06	0.505	0.603
	15	0.08	0.481	0.101
	16	0.10	0.458	0.147
	17	0.12	0.436	0.199
	18	0.14	0.416	0.257
	19	0.16	0.396	0.321
1980:	20	0.18	0.377	0.388
	21	0.20	0.359	0.460
	22	0.22	0.342	0.536
	23	0.24	0.326	0.614
	24	0.26	0.310	0.694
	25	0.28	0.295	0.777
	26	0.30	0.281	0.861
	27	0.32	0.268	0.947
	28	0.34	0.255	1.034
	29	0.36	0.243	1.121
1990:	30	0.38	0.231	1.209
	31	0.40	0.220	1.297
	32	0.42	0.210	1.385
	33	0.44	0.200	1.473
	34	0.46	0.190	1.560
	35	0.48	0.181	1.647
	36	0.50	0.173	1.734
	37	0.52	0.164	1.819
	38	0.54	0.157	1.904
	39	0.56	0.149	1.987
2000:	40	0.58	0.142	2.070
	41	0.60	0.135	2.151
	42	0.62	0.129	2.231
	43	0.64	0.123	2.309
	44	0.66	0.117	2.386
	45	0.68	0.111	2.462
	46	0.70	0.106	2.536
	47	0.72	0.101	2.609
	48	0.74	0.096	2.680
	49	0.76	0.092	2.750
2010:	50	0.78	0.087	2.817

Total Present Worth of Net Benefits = 2.817



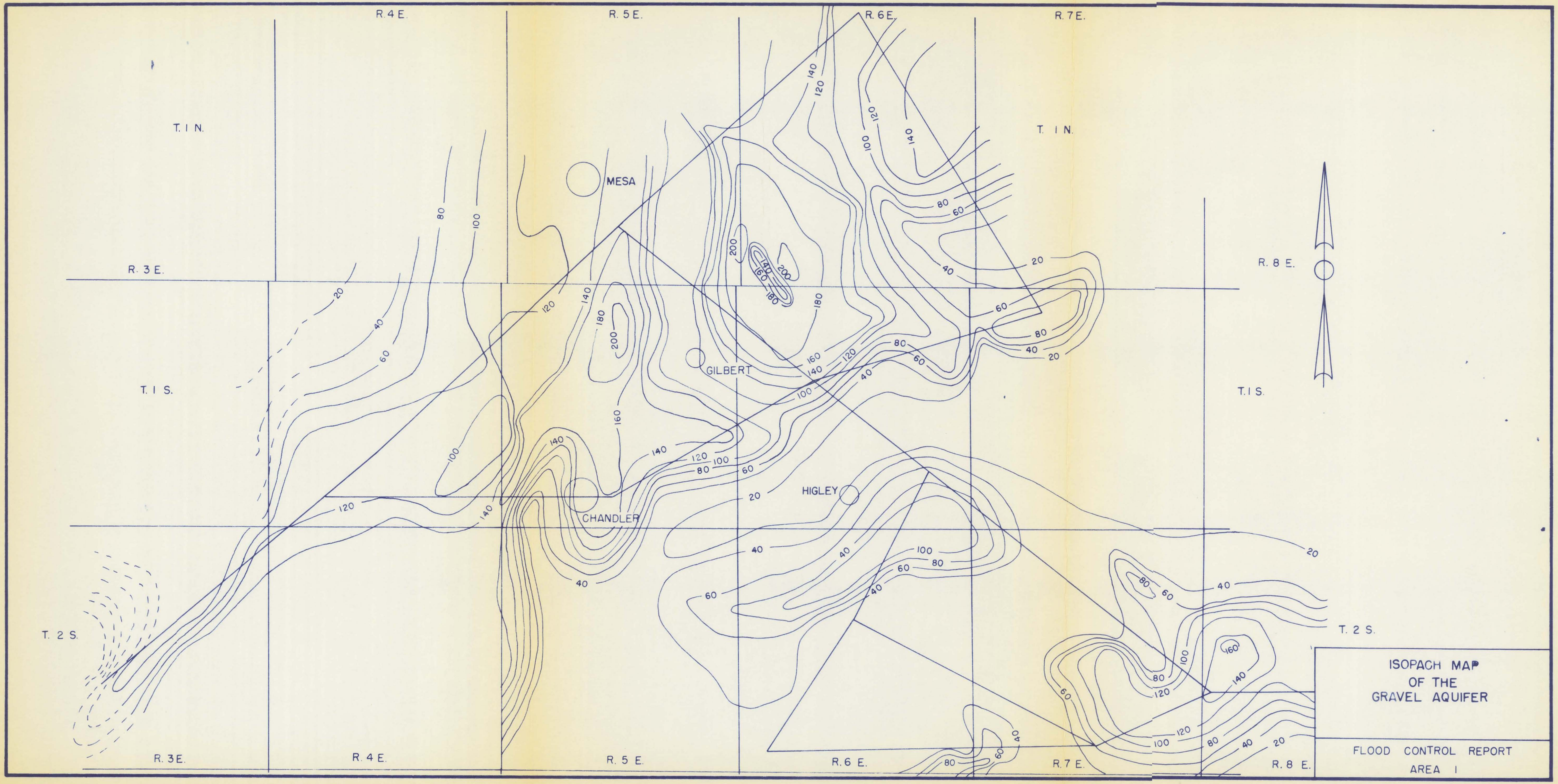
APPENDIX C , PLATE NO. I



DEPTH TO TOP OF GRAVEL FROM SURFACE

FLOOD CONTROL REPORT
AREA 1

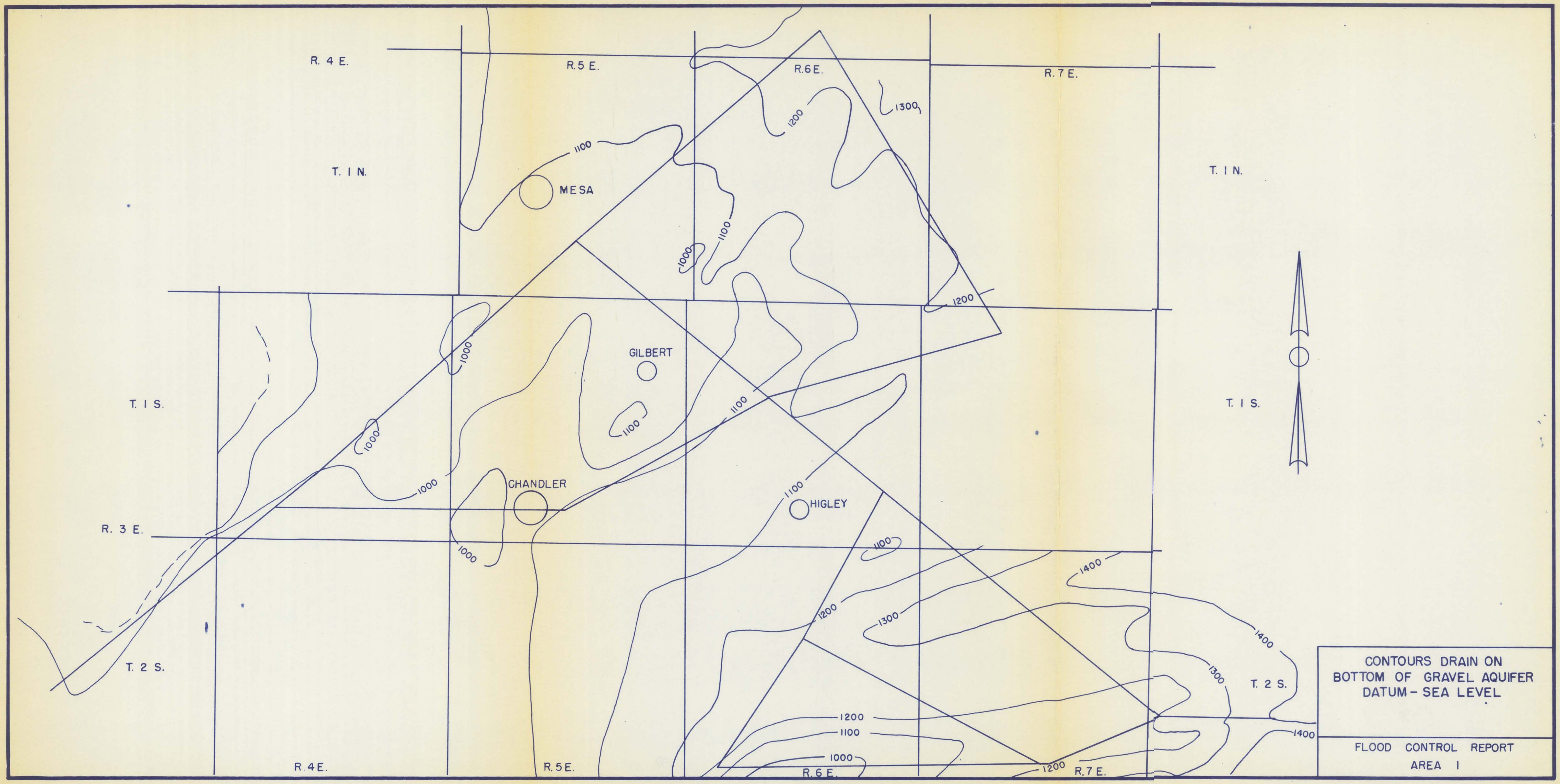
APPENDIX C, PLATE NO. 2



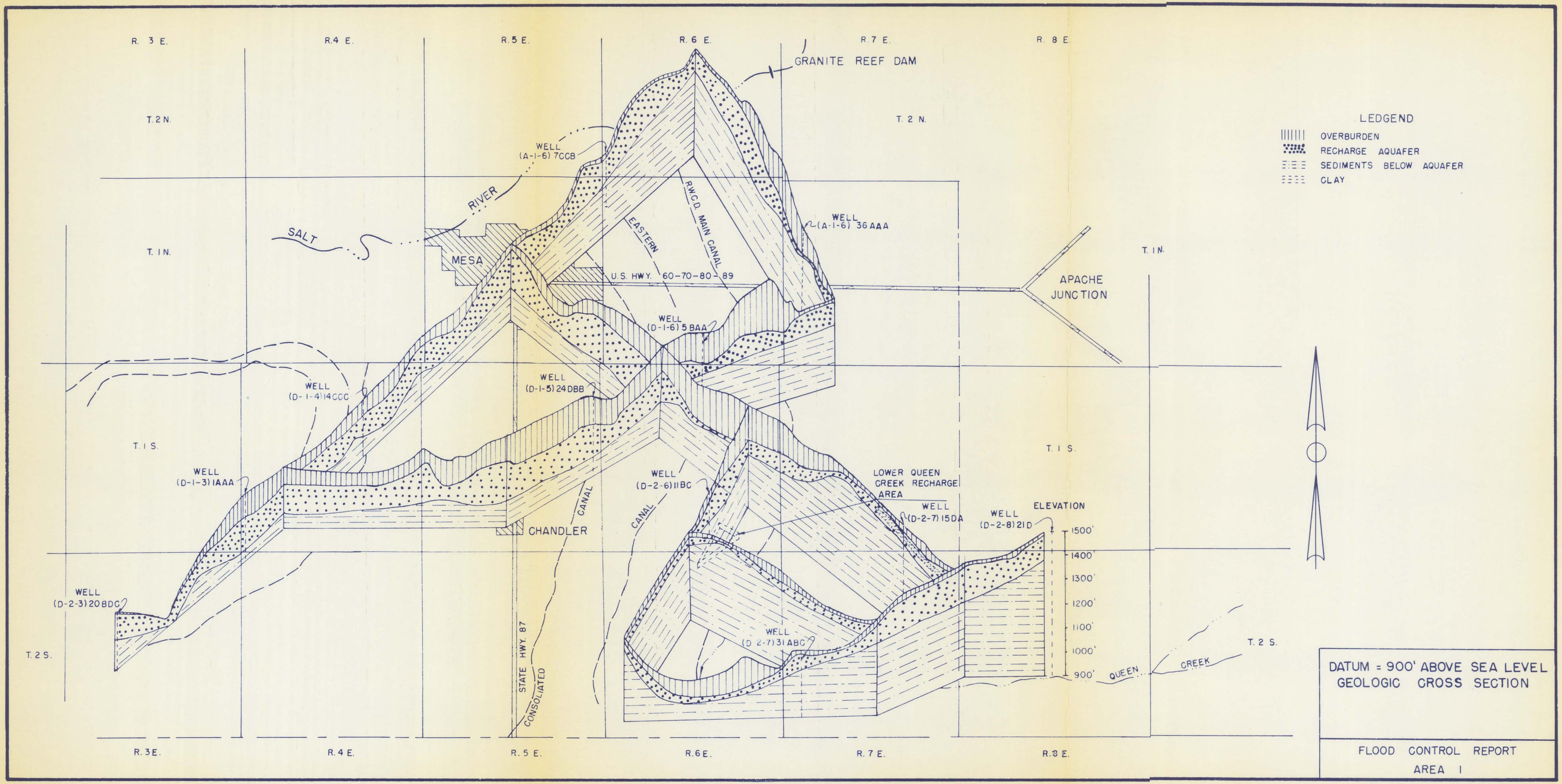
ISOPACH MAP OF THE GRAVEL AQUIFER

FLOOD CONTROL REPORT AREA 1

APPENDIX C, PLATE NO. 3



APPENDIX C, PLATE NO. 4



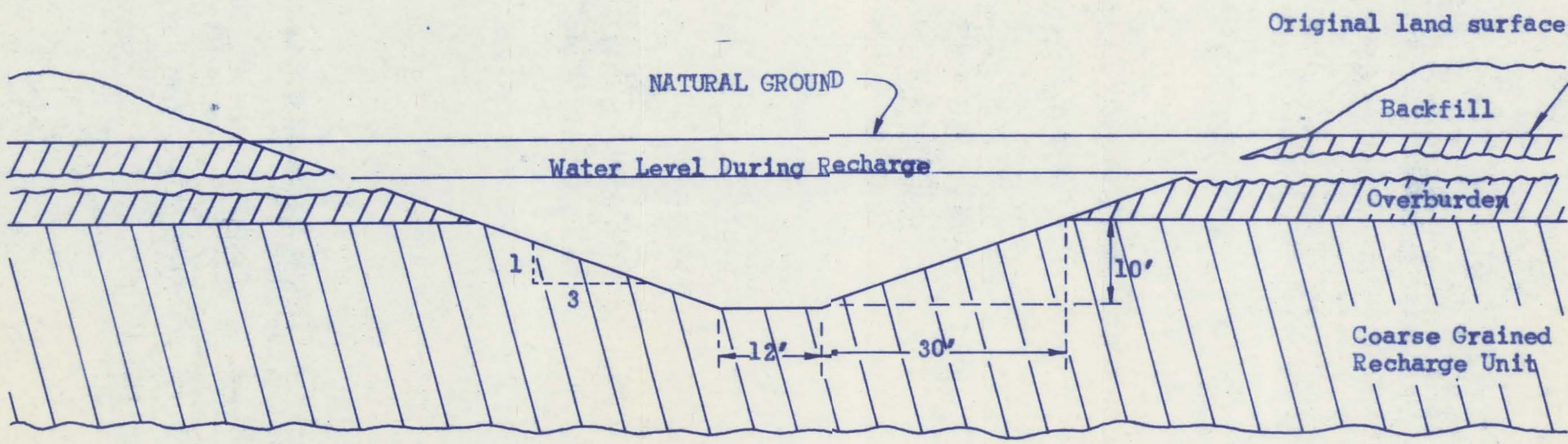
DATUM = 900' ABOVE SEA LEVEL
GEOLOGIC CROSS SECTION

FLOOD CONTROL REPORT
AREA 1

APPENDIX C, PLATE NO. 5

Cross Section of Recharge Pit & Design Features

Recommendations For Dry Pit: Bottom
Length of 2,300 ft. (provides recharge
area of four acres/Pit).



NOTE: End section provides 4,545 sq.ft. of recharge area - 520 ft. bottom length provides recharge area of one acre.

1. DRY PITS: Recharge rate - 20 acre-ft/acre/day using 3" of cotton gin trash spaded into pit surface. Recommended for following recharge areas - Upper Queen Creek, Upper Salt River, Lower Queen Creek, and Salt River Mountains.

2. WET PITS*: Recharge rate - 25 acre-ft/acre/day using 6" of pea size gravel (1/4"-1/2"). Recommended for Mesa Recharge Areas.

*Differentiation of pit type is based on the fact that "dry pits" receive flow only during flood runoff period and "wet pits" receive continuous flow throughout the year. Pits, if saturated continuously, develop a schmutzdecke (responsible for filtering sediment) and thus retain higher recharge rates.

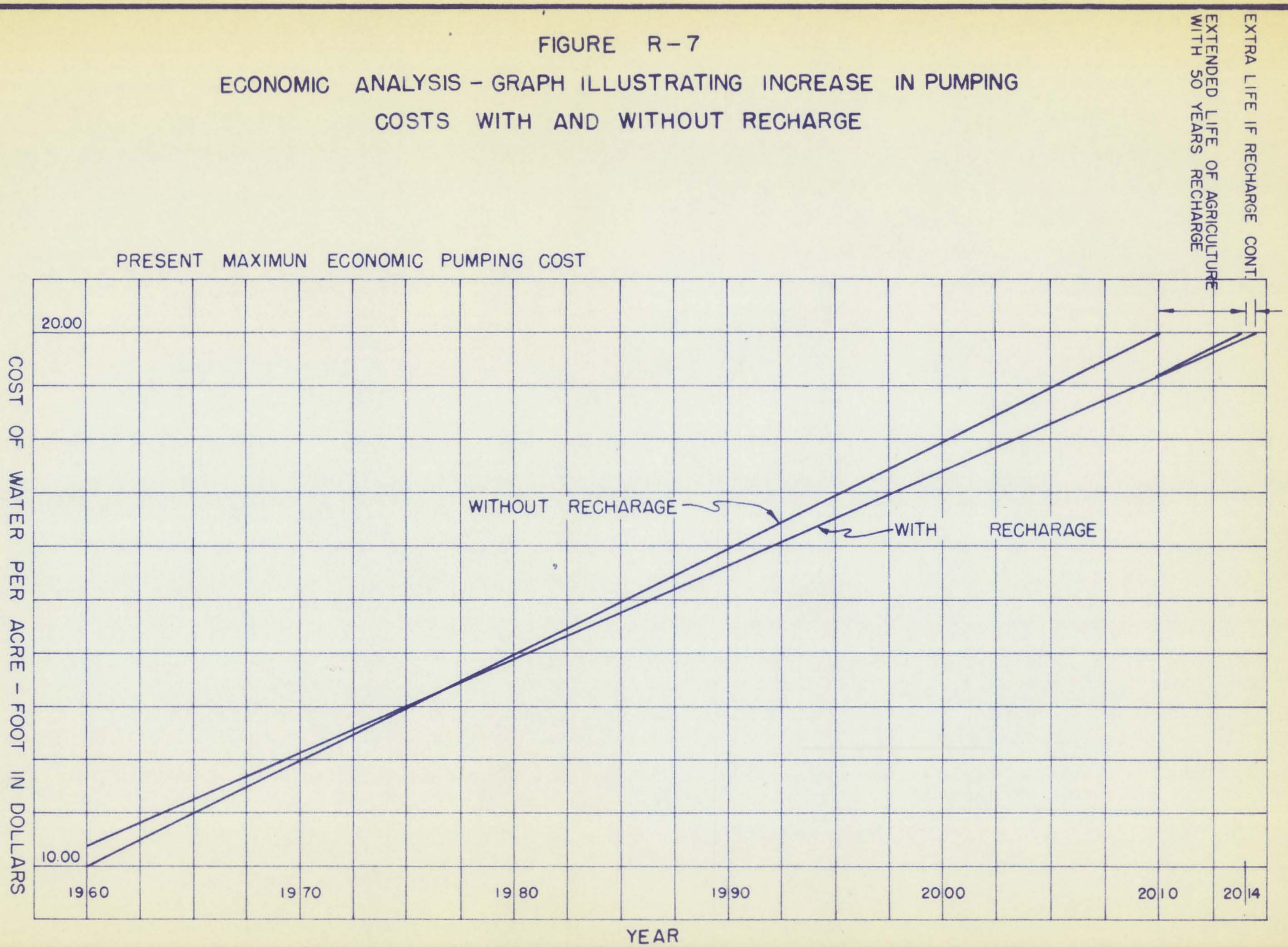
Characteristics of Recharge Areas & Recharge Pits

Recharge Area	Description: Legal	Description: Size (Acre)	Depth of Overburden (ft)	Thickness of Aquifer (ft)	Drainage Area: Description	Drainage Area: Size (Sq Mi)	Design Recharge Rate (cfs)	Optimum Yield (acre-ft./yr.)	Pit Recharge Area (Acres)	Total Recharge Rate - (Ac-ft/day)
Upper Queen Creek	T 2S, R8E Sec. 21 & East 1/3 Sec. 20	850	18	100	Queen Creek (Including Whitlow Reservoir)	220	1,180	8,700	120	2,400
Upper Salt River	Strip 300' by 2 mi. South bank Salt River starting West of SCS Flood Channel	80	10	100	SCS Detention Basins North of Apache Junction	40	160	1,130	16	320
Lower Queen Creek	T 2S, R6E Between Roosv. Canal & Queen Creek in Sec. 10, 11, 15, & 22	400	18	50	SCS Detention * Basins South of Apache Junction	133	500	3,680	50	1,000
		100			Santan Mt. Area (above Hunt Hwy)	30	120	850	12	240
		500			Totals:	163	620	4,530	62	1,240
Salt River Mountains	T 2S, R3E Sec. 21, SW Corner	60	0	60	Salt River Mountains (above High Line Canal)	34	150	1,090	15	300
Mesa Recharge Areas	T 1N, R5E Sec. 31 North Central Edge	7	20 - 30	100 - 120	West Mesa	5	10	1,200	0.8	20
	T 1S, R5E Sec. 4, N.E. Corner NW Quad.	22	20 - 30	140 - 160	Central Mesa	16.5	33	4,100	2.7	66

* If the flood runoff from the SCS detention basins south of Apache Junction are not released for recharge, the facilities in this area should be designed only for flood runoff from the Santan Mts. Area.

FIGURE R-7

ECONOMIC ANALYSIS - GRAPH ILLUSTRATING INCREASE IN PUMPING COSTS WITH AND WITHOUT RECHARGE



APPENDIX I-D PLATE NO. I