DICK PERREAULT

PRELIMINARY HYDRAULIC DESIGN MEMORANDUM

CHANNELIZATION ELEMENTS

STRUCTURAL FLOOD CONTROL ALTERNATIVES

For

Central Arizona Water Control Study
U.S. Army Corps of Engineers
Los Angeles District

By

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TABLE OF CONTENTS

		Page
I.	INTRODUCTION	I-1
	Background	I-1
	Report Organization	I-2
II.	LEVEES	II-1
	General Description	II-1
	Hydraulic Design Criteria and Considerations	II-1
	Water Surface Profile Calculations	II-6
	Results of Hydraulic Analysis	II-8
III.	CHANNELS	III-1
	General Description	III-1
	Hydraulic Design Criteria and Considerations	III-1
	Preliminary Analysis	III-4
	Water Surface Profile Calculations	III-6
	Results of Hydraulic Analysis	III-7
IV.	GREENBELT FLOODWAYS	IV-1
	General Description	IV-1
	Hydraulic Design Criteria and Considerations	IV-2
	Preliminary Analysis	IV-3
	Water Surface Profile Calculations	IV-7
	Results of Hydraulic Analysis	IV-7

I. INTRODUCTION

BACKGROUND

This memorandum documents the hydraulic design of channelization elements performed for the Stage II - Structural Flood Control Alternatives Report - Central Arizona Water Control Study. Three previous Camp Dresser & McKee (CDM) reports* provide background information on the channelization options as well as dam and flood outlet options also being considered.

Three basic channelization alternatives were analyzed: levees, channels, and greenbelt floodways. For each alternative, four designs were prepared, corresponding to design flows of 50,000 cfs, 100,000 cfs, 200,000 cfs and 300,000 cfs. Levees were designed for two reaches as shown in Figure I-1, Reach A - the Salt River from 35th Avenue to upstream of Country Club Drive (approximately 24 miles) and Reach B - The Gila River from Gillespie Dam to the confluence with the Salt, and the Salt from the confluence to 91st Avenue (approximately 39 miles). Channels and Greenbelt Floodways were designed for Reach A only.

* 1. "Stage II Structural Flood Control Alternatives - Conceptual Plans," January 28, 1980,
2. "Stage II Structural Flood Control Alternatives - Hydraulic Design and Cost Estimating Methodology," February 5, 1980,
3. "Appraisal Level Cost Study," February 20, 1980

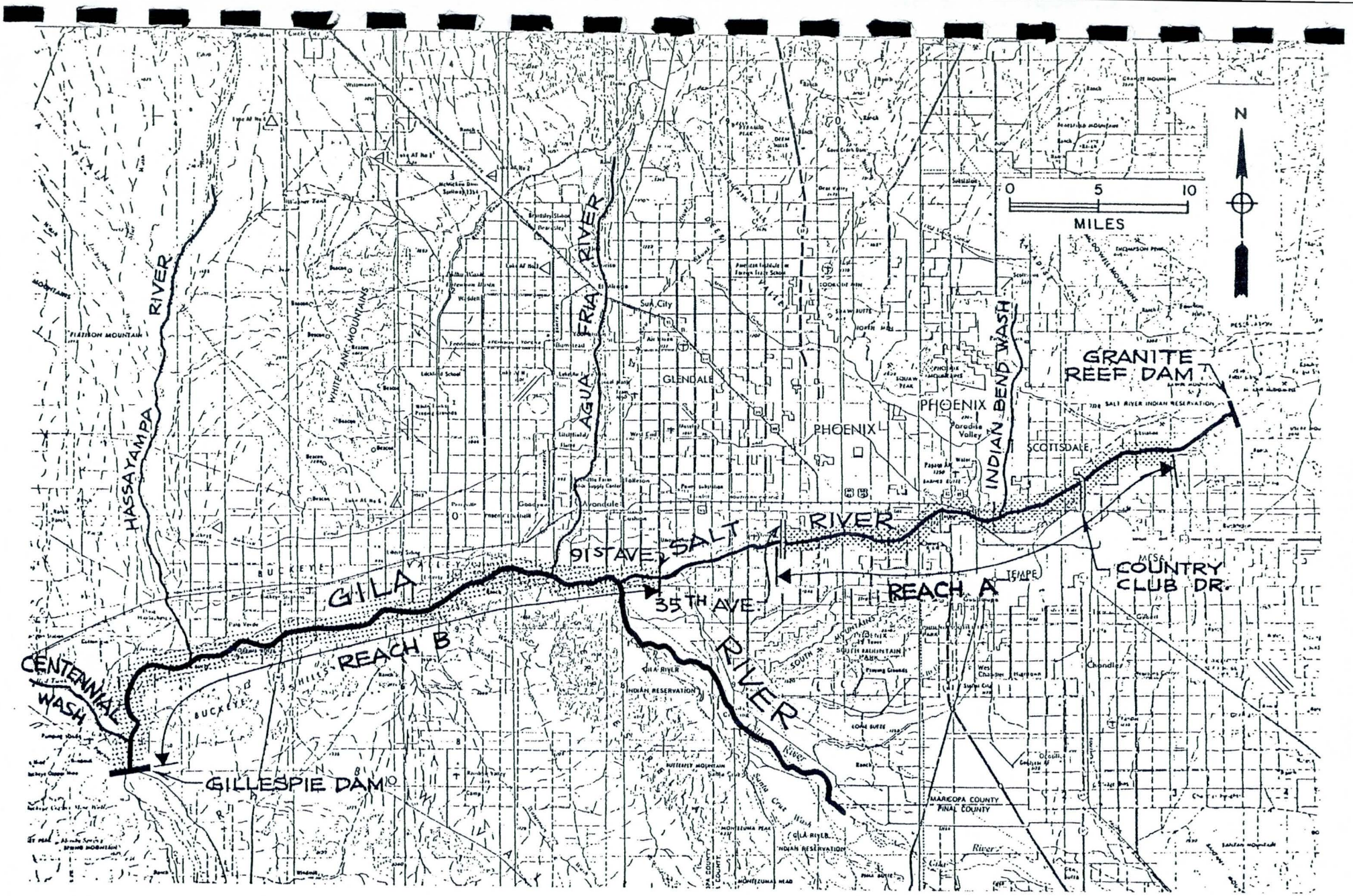


FIGURE I-1

STUDY REACHES

REPORT ORGANIZATION

This report is organized into four main sections consisting of this introduction followed by sections on levees, channels and greenbelt floodways. Each of the sections will briefly introduce the alternatives, list key hydraulic criteria and design considerations, describe the selection of alignments and the hydraulic analysis procedure, and discuss the results of the analysis. In addition to the text, the following accompanying drawings comprise this hydraulic design report:

1. Plan and Profile Drawings

   Levees alternative, channel alternative and greenbelt alternative. Each for 300,000 cfs.

   Scale: 1" = 1000' horizontal
   1" = 10' vertical

   Number of Sheets: Reach A 5 total
   Reach B 7 total

   Copies: One blue-line print of each sheet

2. Profile Drawings (Work Sheets)

   Levee alternative, channel alternative and greenbelt floodway alternative. Each for four flows: 50,000; 100,000; 200,000 and 300,000 cfs.

   Scale: 1" = 1000' horizontal
   1" = 10' vertical

Number of Sheets: Reach A 6 total for levees
5 total for channels
6 total for greenbelts
Reach B 7 total for levees

Copies: One reproducible of each sheet

3. Plan Drawings

Levee alternative, channel alternative and greenbelt floodway alternative. Each for four flows: 50,000; 100,000; 200,000 and 300,000 cfs. These are marked up plan and profile drawings that show channel alignments and levee tie-in points.

Scale: 1" = 1000' horizontal
1" = 10' vertical

Number of Sheets: Reach A 5 total
Reach B 7 total

Copies: One blue-line print of each sheet

4. Topographic Maps

These 1977 maps show channel and levee alignments and the location of HEC-2 cross-sections. These maps were the source of ground elevations.

Scale: 1" = 400"

Number of Sheets: Reach A 13 total
Reach B 14 total

Copies: One blue-line print of each sheet

5. Profile Drawing

This profile was used to analyze channel grades.

Scale: 1" = 5000'

Number of Sheets: One only

Copies: Two blue-line prints of the sheet

6. HEC-2 data listings and original computer output for all four designs, 15 major runs plus several specialized runs used to analyze transitions and to establish starting water surface elevations.

i. Alma School Road. The situation at Alma School Road is comparable to that at 24th Street and the same approach was taken in modeling this crossing.

j. Country Club Drive. The proposed 130,000 cfs bridge was assumed to be built. No excavation was modelled due to lack of any information on proposed plans. Since the vicinity is presently very disturbed by gravel pits, it is felt that some grading or channelization will be needed to provide for a smooth inlet condition here.

7. Local drainage facilities will be designed using the following criteria, in the absence of any specific contemporaneous flow data:

a. The local drainage facilities must be sized for the 100-year event on the local drainage area. The Salt and Gila Rivers are assumed to by dry when this event occurs, since it corresponds to a summer event.

b. There must also be the capability to handle the 10-year local event while the Salt and Gila Rivers are flowing at design capacity (winter event). The local drainage facilities are currently being designed based on these criteria and the water surface profiles developed for the Salt and Gila Rivers.

8. Confluences with the following major tributaries will be treated by tieing in levees to high ground on one or both sides of the tributary:

Centennial Wash: Gila River levee ties in on the north side of the Wash and marks the downstream end of the project. The small area to the south of Centennial Wash down to Gillespie Dam is very low and not protected by this project.

Hassayampa River: Gila River levees tie-in upstream and downstream from this river. The Hassayampa River is elevated well above the Gila River and will not be affected by backwater from any of the levee designs.

Agua Fria River: Gila River levees tie-in to both banks of the river and extend upstream to high ground.

Indian Bend Wash: The Salt River levees tie-in to the existing Indian Bend Wash levees at Princess Drive.

The size and extent of the tie-in levees was approximated by extending the Salt or Gila River water surface back into the tributaries assuming only minor flow (and headloss) in the tributary. Water surface elevations in the tributaries were only roughly approximated due to lack of contemporaneous hydrology data and channel topo for the tributaries.

WATER SURFACE PROFILE CALCULATIONS

All water surface profiles were computed using the HEC-2 computer program. Data listings and outputs are included with this report. The following items are included:

1. Listing of the Gila* Levee - data for Runs 3-8 on Reach B. The data is divided into three parts:
   Reach B1 - Salt River from confluence to 5 miles upstream of confluence.
   Reach B2 - Arlington area to confluence with Salt River.
   Reach B3 - Gila River from Gillespie Dam to Arlington.

2. Listing of the Salt* Levee - data for Runs 1-8 on Reach A. The data is divided into three parts:
   Reach A1 - Salt River from Mesa area to Granite Reef Dam.
   Reach A2 - Salt River from 43rd Avenue area to Mesa area.
   Reach A3 - Salt River from 5 miles above confluence with Gila to 43rd Avenue area.

3. Reach B Runs 4 and 5 - These runs are for 300,000 cfs and Reach B1 only. The purpose of these two runs was to determine if all the Salt River flow would enter the leveed floodway if the 91st Avenue Treatment Plan was used as a tie-in point. Run 4 assumes existing conditions and Run 5 assumes levees downstream. The findings show that the 91st Avenue tie-in is acceptable, but the results are very close and further studies may be needed in later planning stages.

4. Reach B Runs 3, 6, 7, 8 - These are the Reach B runs (Reaches B1 and B2) for the four design flows. Critical depth at Gillespie Dam was the downstream control. A flow of 180,000 cfs rather than 200,000 cfs was used for Run 8. After this run was made, it was decided that 200,000 cfs should be used for Reach A levees as well as for channels and greenbelts. The water surface profiles for these cases are plotted on the profiles included with this report.

5. Reach A Runs 1-4 - These runs, one for each design flow, were for Reach A3 only to establish starting water surface elevations for Runs 5-8. Reach A3 is below the project reach and therefore has no levee encroachment. However, the encroachment stations supplied with the data by the Corps were retained to be conservative. These encroachment stations are generally very far out from the river and limit the floodway to observed effective flow areas.

6. Reach A Runs 5-8 - These are the Reach A runs (Reaches A1 and A2) for the four design flows. The water surface profiles for these runs are plotted on the profiles included with this report. Since the Corps supplied the HEC-2 data and calibrated friction factors, the basic representation was not altered except as noted earlier. There are certain areas in which the representaion appears to be weak in that cross-sections are not perpendicular to the flow direction and that substantial divided flow and other non-representative conditions occur. To some extent, these conditions are probably compensated for by the calibrated friction factors. However, the presence of gravel pits and makeshift levees on Reach A greatly complicates the hydraulics in this reach and makes a good representation of actual flow conditions very difficult. This fact should be kept in mind when analyzing the water surface profiles for Reach A.

RESULTS OF HYDRAULIC ANALYSIS

The results of the hydraulic analysis are presented in computer output and on plotted profiles. The profiles also show the ground elevations at the left and right bank protential levee locations. Where the water surface was above the ground elevation, the topo maps were checked to determine whether this overflow would inundate any developed areas and result in flood damages. In those cases in which the overflow reach is short and causes no damages, no levee is called for. In such cases, the elevation corresponding to a "non-damaging" situation is indicated on the profiles. Also shown on the profiles for all four design flows is the height of levee required and tie-in points. Where existing levees contain the computed water surface, the existing levee elevations are also shown.

The most significant finding was that in Reach A, the Salt River can pass up to 100,000 cfs with no flood damages. This is largely due to the relatively steep gradient in this reach, the lack of vegetation, the man-induced constriction of flow to an unnaturally narrow floodway, and particularly by the mining of large quantities of riverbed materials by the gravel operators and the cutoff of sediment supply by upstream reservoirs. These conditions do not exist on the Gila River and the extent of flooding is much greater, requiring levees for flows as low as 50,000 cfs to prevent damages.

This analysis should be adequate for the purpose of developing Stage II cost estimates. If further study is warranted in Stage III it is recommended that better quality, more recent topographic data be used and more care be taken in developing the river representation for HEC-2. Also, more up-to-date information about the proposed new bridges would be needed. Finally, low-flow channel improvements in some key reaches should be studied as a potential cost-effective alternative to higher levees.

2. The side slopes of 3 horizontal to 1 vertical and back slopes of 2.5 to 1 were used as per the recommendations contained in the Corps' "Preliminary Design Report" dated March, 1980.

3. The maximum allowable velocity in the earth-bottom channels was taken to be 12 ft/sec. at a Manning "n" of 0.030 and a fixed bed, as recommended by the Corps.

4. Channel heights were established to provide a minimum of 3 feet of freeboard above the design water surface elevations.

5. The channel grades were set to insure a basically entrenched channel, i.e., the top of the channel banks are below adjacent ground levels. The grades were also set to result in low-flow (non-pressure) conditions at all existing and proposed bridges to be incorporated into the plans.

6. The following small bridges were assumed to be removed and replaced with dip crossings:

35th Avenue
7th Avenue
7th Street
Hohokam Expressway
Hayden Road

The following existing or proposed new bridges were incorporated into the channel plans if possible. If not possible due to the size or location of the bridge, a new bridge or enlargement of the existing or proposed bridge was assumed:

19th Avenue (proposed)	SPRR (existing)
Central Avenue (existing)	Abandoned Bridge (existing)
16th Street (proposed)	Mill Avenue (existing)
24th Street (proposed)	Scottsdale Road (proposed)
I-10 (existing)	Alma School Road (proposed)
	Country Club Drive (proposed)

Piers were modeled with a debris accumulation of two feet on each face of the piers, except for supercritical flow.

7. Supercritical flow must have a Froude number over 1.13 and subcritical flow must have a Froude number below 0.86 to insure stable flow. Supercritical flow must be maintained through the bridges (Class C flow). A Manning "n" of 0.014 was used for concrete channels.

8. Local drainage facilities will be designed using the following criteria, in the absence of any specific contemporaneous flow data:

a. The local drainage facilities must be sized for the 100-year event on the local drainage area. The Salt River is assumed to be dry when this event occurs, since it corresponds to a summer event.

b. There must also be the capability to handle the 10-year event while the Salt River is flowing at design capacity (winter event).

The local drainage facilities are currently being designed based on these criteria and the water surface profiles developed for the Salt River.

9. Since the water surface in the Salt River will be lowered as a result of the channelization, no backwater calculations on Indian Bend Wash were necessary. A grade stabilization structure on Indian Bend Wash will be required at the confluence.

PRELIMINARY ANALYSIS

The Salt River profile drawing at a 1" = 5,000' scale included with this report shows the variations in bank and invert slope over the reach from the confluence with the Gila River to Granite Reef Dam. The straight line drawn through the existing top of bank elevations indicates that from about 35th Avenue to Granite Reef Dam there is a relatively constant grade of 0.00190. The deviations from this line are relatively small. The existing channel invert, on the other hand, varies considerably in slope over the same reach. These variations can be explained in terms of several factors, including natural topography and geology as well as the man-made influences of gravel mining and constriction of flow by bridges and development along the river.

The existing riverbed varies widely in shape and width as well as slope. It is unlikely that the river profile shown, which is based on 1977 topography, represents a stable equilibrium grade. Recent severe flooding, gravel mining, and the fact that flows and sediment loads are controlled by upstream reservoirs complicate any analysis of equilibrium in this reach. In any case, since the earth-bottom channel would confine all flows to a uniform cross-section, there is no justification for varying the slope of the channel in response to the existing slope changes. A uniform channel shape and slope will insure that whatever sediment is carried into the channel from upstream can be transported to the downstream end of the project. A slope of 0.00190 was selected, based on the general ground slope in the project reach.

Below 35th Avenue, there is a decrease in slope of both the channel and the adjacent ground. This decrease in slope is accompanied by a widening of the flood plain. This is a natural occurrence and would continue to occur after channelization of the Salt River.

One alternative to a uniform natural slope was considered: a controlled slope utilizing a series of drop structures to create a "stair-step" invert. This would control velocities, but was found to be less cost-effective than the natural slope option. This conclusion is documented in previously submitted CDM appraisal level reports.

The 200,000 cfs channel is earth-bottom and requires a 1,400 foot bottom width to have a velocity under 12 ft/sec. At uniform flow, the average velocity is 11.3 ft/sec and the Froude Number is 0.57. The invert of the channel was set 18 feet below the top of bank line at a slope of 0.00190, as shown on the Salt River Profile. This depth was arrived at after considering the uniform depth of flow, the required freeboard allowance, and the additional depth of flow caused by backwater upstream of bridges. This grade insures that the water surface will remain below the adjacent ground level and that no levees will generally be required. Since the top of the existing bank is up to about a thousand feet or more from the earth-bottom channel bank, this area would be filled using the excess excavated soil from the channel excavation rather than building a levee along the channel.

The width of the 200,000 cfs channel was kept constant except near Central Avenue, where the bottom width was reduced to 834 feet to avoid having to demolish several buildings and replace the existing bridge which has a rated capacity of over 200,000 cfs. Other bridges were assumed to be over 1,500 feet wide.

The channel design for 300,000 cfs is earth-bottom only upstream of the Hohokam Expressway area. At a slope of 0.00190, the channel must have a bottom width of 2,000 feet to maintain velocities below 12 ft/sec. At uniform flow, the average velocity is 11.6 ft/sec and the Froude Number is 0.58. This earth-bottom reach was set at the same level as the 200,000 cfs channel in that reach. A concrete channel was designed downstream from Hohokam Expressway to near 35th Avenue. The size and slope of this super-critical flow channel were not dictated by allowable velocities or the need

to maintain a slope of 0.00190. Rather, the controlling factors were the need to establish stable supercritical flow, maintain supercritical flow through the bridges, and transition smoothly to subcritical flow both upstream and downstream from the concrete channel. These requirements resulted in a channel with a bottom width of 600 feet and a slope of 0.00176 (Velocity 28.1 ft/sec, Froude Number 1.28) and at a lower elevation than the 200,000 cfs earth-bottom channel, as shown on the Salt River Profile.

WATER SURFACE PROFILE CALCULATIONS

Following the preliminary analysis of channel hydraulics, the HEC-2 computer program was used to develop water surface profiles for the 200,000 cfs and 300,000 cfs channels. The stationing of the cross-sections correspond to stationing along the channel centerlines. The HEC-2 representation of the existing topography was not used. Although the centerline for the 200,000 cfs and the 300,000 cfs channels were not always along the same alignment, it was found that the differences in the stationing on the two centerlines were so small that they could be ignored. The elevations on the HEC-2 runs are not the actual elevations, but are 914 feet lower. This occurs because the initial computer runs were based on elevation 100 feet at 35th Avenue (station 50 + 00). When the channel invert elevation was later set at 1014 feet for 200,000 cfs, this convention was adopted. Therefore, to get the actual elevation at any point, 914 feet must be added to the elevations called out in the HEC-2 output.

The following HEC-2 runs, with data listings, are included with this report:

Run 1 This is the run for the 200,000 cfs channel for all of Reach A, excluding the transitions at both ends. The water surface for this run is plotted on the 1" = 1,000' profile for 200,000 cfs included with this report.

Run 2 This run shows the upstream transition from the 200,000 cfs channel to the existing river conditions upstream of Country Club Drive. The 1" = 1,000' profile also shows the water surface elevations from this run as well as the required levees.

Run 3 This is the downstream transition run for 200,000 cfs below 19th Avenue. The resulting water surface profile is plotted on the 1" = 1,000' profile.

Run 4 This is the supercritical run for the 300,000 cfs concrete channel from just upstream of Hohokam Expressway to 35th Avenue. This run indicates the point in the transition where supercritical flow is established. The results are plotted on the plan and profile drawings.

Run 5 This run is for the subcritical portion of the 300,000 cfs channel above Hohokam Expressway. The resulting water surface profile is shown on the plan and profile drawings.

Run 6 This is the upstream transition run from the 300,000 cfs channel to the existing river conditions upstream of Country Club Drive. The water surface elevations from this run are plotted on the plan and profile drawings.

RESULTS OF HYDRAULIC ANALYSIS

The HEC-2 analysis was used to refine the channel designs until they produced acceptable results. The following were the most significant findings:

The 200,000 cfs earth-bottom channel exhibits a generally smooth subcritical profile, and bridge losses are moderate. Two exceptions occur: at the Central Avenue Bridge and at the upstream transition. At Central Avenue, the channel is constricted and a substantial amount of backwater occurs upstream and flow passes through critical at the bridge. For this reason, additional channel height is required both upstream and downstream from the bridge to contain the high waters and the undular hydraulic jump. The Froude Number of the flow immediately downstream from the bridge is 1.33. No supercritical HEC-2 run was made, but approximate hand calculations indicate the jump will occur within 50 feet of the bridge and the first wave will require an additional 4 feet of freeboard for about 100 feet.

At the upstream transition, the flow goes through critical depth and is marked by some rapid changes in water surface and velocity. This occurs largely because the energy grade line in this area has been lowered to allow the water surface to pass under the proposed bridges. However, the same sort of behavior occurs in this area even without the channel, as can be seen in the levee analysis. The river in this area is extremely disturbed, and the HEC-2 representation is not very good. At later stages in the planning process it is recommended that the HEC-2 representation be improved using newer topography and the problem be analyzed in more detail. The likely solution would be to allow this area to adapt itself to the downstream channelization by slight headcutting and changes in bed configuration. The same holds true for the 300,000 cfs channel upstream transition.

The 300,000 cfs design works quite well in the subcritical reach, with acceptable bridge losses. In the supercritical reach, however, considerable difficulty was experienced at bridge crossings. Since the flow does not have extremely high velocities for supercritical flow, the width of the bridge piers is highly critical. Above a certain total pier width, the flow does not remain supercritical through the bridge and becomes subcritical for some distance upstream. The resulting high water and undular hydraulic jump is not acceptable in a channel designed for supercritical flow. It was

found that no more than 8 feet of total pier width is allowable in the 600 foot wide channel. This would correspond to 4 two-foot wide piers at 120 foot centers, and no allowance for debris accumulation. At supercritical velocities, debris accumulation should not be a problem. The proposed new bridges generally call for 3-foot piers at 125 foot centers. With design modifications to a smaller pier width, some of the proposed bridges would be acceptable. However, the existing Central Avenue and I-10 Bridges would need to be replaced with new structures, since they have too many piers for this supercritical channel design.

Superelevation was checked in the supercritical flow channel. The sharpest bend has a radius of curvature of 6,000 feet, which will result in some 2.7 feet of superelevation. Generally, the superelevation will be less than a foot.

The transition from supercritical to subcritical flow near 35th Avenue would occur as an undular hydraulic jump. The channel was designed such that the energy grade line of the supercritical flow meets the energy grade line of the backwater (based on the levee analysis) near 35th Avenue. A detailed design of the transition using HEC-2 was not considered necessary at this planning stage. The channel terminates by flaring out rapidly to the width of the levee floodway previously established. This sudden expansion would be designed to trigger the jump. Analysis indicates the solitary wave will extend 28 feet above the invert if it occurs in a 600 foot wide section. However, the effective width of flow at the jump will be somewhat greater than this and the wave may be smaller. Since the levees required in the levee plan (assuming only backwater conditions and no transition) provide sufficient freeboard for the jump, it was assumed that these levees would extend upstream to the transition.

The transition from subcritical to supercritical flow near Hohokam Expressway was handled as a rapid contraction and slope increase over a 1,000 foot distance. An HEC-2 analysis was made of the transition in both subcritical and supercritical runs. The transition was not modelled in detail, but

the results show the flow will be subcritical through the transition and turn supercritical when flow enters the 600 foot wide concrete channel. Because of the potential for large standing waves resulting from the contraction, additional freeboard is provided for a distance of 1,000 feet from the start of the concrete channel. A more detailed analysis of this transition will be required in later planning stages.

GENERAL DESCRIPTION

The concept of greenbelt floodways has undergone several changes in the course of this study. The greenbelt floodways were originally conceived to be entrenched, swale-type grass-lined waterways with drop structures to mantain velocities under 7 feet per second. Parks and other recreational facilities would be located within the floodway. After appraisal level studies for channels and levees had been completed, it became apparent that an entrenched channel with such a low velocity would require an extremely large amount of excavation. The cost of this excavation and the disposal of the excess excavation would be unreasonably high relative to other flood control alternatives.

The concept was then altered to a leveed waterway having an earth-bottom low-flow channel with revetted banks. The low flow channel would be sized to carry enough of the flow at moderate velocities to reduce the velocity in the overbank (between the low-flow channel banks and the levees) to under 7 feet per second. The overbank greenbelt area would be graded to prevent flow concentrations that could produce erosive velocities.

The finding that designs for flows of 50,000 cfs and 100,000 cfs are unnecessary (see Levees) leaves only the 200,000 cfs and 300,000 cfs design flows for the greenbelts. At these high flows, much of the project reach (Reach A) was found to be unsuited to the greenbelt floodway concept. Only the wider areas and those having few gravel pits and bridge crossings were potentially suited to the concept. Therefore, the greenbelt floodways plan presented here consists of a modified levee system with three non-contiguous greenbelt sub-reaches.

The low-flow channels are similar in design to those designed for the channel alternatives: earth-bottom with 3 to 1 soil cement side slopes. The levees are similar to those designed for the levee alternatives, except that the thickness and depth of soil cement protection is reduced and a landscape fill having a 4 to 1 slope is placed over the soil cement on the river side of the levees. The back slope of the levees is also 4 to 1. This reduced slope is to facilitate access and maintenance and to minimize the visual impacts of the levees. Typical sections are shown on the plan and profile drawings.

HYDRAULIC DESIGN CRITERIA AND CONSIDERATIONS

1. The levees forming the outside boundaries of the greenbelt floodways were generally located along the same alignments as the levees in the levee plans, and according to the same criteria. In one area upstream from the Sky Harbor Airport, the south side levee was relocated farther south for the greenbelt since the expanded floodway area contains no existing development.

2. The velocity in the greenbelt area was limited to 7 feet per second, assuming a Manning "n" of 0.030 in both the low-flow channel and the greenbelt, as modelled using HEC-2.

3. The levees were modelled using the floodway encroachment option number 1 of HEC-2. The height of levees was set to provide a minimum of 3 feet of freeboard above the water surface elevation calculated assuming a Manning "n" of 0.030 in the low-flow channel and 0.050 in the greenbelt. The 0.050 value allows for obstructions related to recreational development in the greenbelt, which would be excluded from the low-flow channel. Outside the greenbelt floodway areas,

the levee heights were determined by providing 3 feet of freeboard above the water surface computed using the same Manning "n" values used for the levee alternative hydraulic analysis.

4. The future "without project" condition was the base condition on which the greenbelt modifications were made. The same assumptions were made as were made for the levee analysis.

5. Local drainage and confluence criteria were the same as for the levee analysis.

6. The low-flow channel alignments were established using the same criteria as used in the channel alternative analysis.

7. All flow was kept in the stable subcritical range. Since the flow characteristics in the low-flow channel and in the greenbelts were so different, separate Froude Numbers were calculated for each. The overall floodway Froude Number was considered to have no real significance.

8. The water surface profiles were calculated assuming that all levees and all considered greenbelts were built.

PRELIMINARY ANALYSIS

The first step in the analysis was the identification of areas that are suitable for greenbelts. In order to be suitable for a 200,000 cfs or 300,000 cfs greenbelt, the area must be very wide, to allow the flow velocities to be under 7 feet per second. There must also be no bridge constrictions in the greenbelt reach. This requirement is necessary because of the high velocities created by the reduction in the width of the floodway needed to pass the flow under a relatively narrow bridge opening. High velocities occur not only at the

bridge, but also in the transitions both upstream and downstream from the bridge. With very wide floodways and relatively narrow bridge openings, the transition areas are very long and may leave little or no suitable greenbelt area between closely spaced bridges.

The presence of large gravel pits also severely limits the potential greenbelt areas. These pits often lower the elevations in the overbank areas to at or below the existing invert elevations, essentially removing the potential greenbelt land. A final consideration in siting the greenbelts has to do with the existing channel shape. Where the existing low-flow channel is very deep and quite narrow, the velocities in that channel are very high. To control velocities, either the low-flow channel has to be filled or the overbank area has to be lowered considerably. Both alternatives are very costly in that the former will raise the water surface elevation over the existing condition and require higher levees and possibly new bridges while the latter is costly in terms of excavation and creates problems in transitioning the greenbelt to upstream and downstream overbank areas.

The ideal greenbelt area is characterized by an existing channel configuration and slope that distributes the flow between the low-flow channel and the overbank in a way that results in low velocities in the overbanks and moderate velocities in the low-flow channel. The area should be wide, relatively uniform in section, not too highly braided, and free of bridges and large gravel pits. Although such conditions are quite common in many rivers, they are rare in this reach of the Salt River.

After reviewing the hydraulic analysis for the levee plans previously described and considering all the above noted criteria, three reaches were selected for further study as promising greenbelt areas. These areas are shown in detail on the plan and profile drawings, and are:

1. Phoenix-Tempe Greenbelt - extending from the proposed Sky Harbor Airport channelization project upstream to the Tempe bridge crossing.

2. Mesa Greenbelt - extending from Price Road to near Alma School Road.

3. Indian Greenbelt - extending from Stapley Drive to Val Vista Drive at the upstream end of Reach A.

After these areas were selected, low-flow channel alignments were determined and the profiles of the existing invert and average overbank elevations were plotted and "best-fit" straight lines drawn through the points. It was found that the points generally fell near the respective lines, and the two lines were roughly parallel and 13 to 15 feet apart. However, the three areas were found to have not only varying widths (levee locations were fixed), but also varying slopes. The Phoenix-Tempe Greenbelt has a slope of 0.0013, the Mesa Greenbelt has a 0.00268 slope and the Indian Greenbelt has a slope of 0.00175. Since altering the natural slopes in these areas would be extremely costly and create problems in transitioning to the adjacent non-greenbelt areas, these slopes were taken as fixed, and the analysis concentrated on varying the size of the low-flow channel and, to a lesser extent, the elevation of the overbank areas.

The preliminary hydraulic analysis was performed using a small computer program that computes uniform flow channel and overbank velocities given user-input channel and overbank sizes and friction factors. An example output from the program is shown as Table IV-1. Using the average floodway width, the natural slope, and the design friction factors, each potential greenbelt was checked for velocity and depth of flow prior to the more detailed HEC-2 analysis. The results of the uniform flow program were found to agree well with the HEC-2 results and saved a considerable amount of time and expense in arriving at acceptable designs.

TABLE IV-1

EXAMPLE OUTPUT FROM UNIFORM FLOW GREENBELT
FLOODWAY ANALYSIS PROGRAM

TOTAL FLOW 200000.
DEPTH OF CHANNEL 15.0
BOTTOM WIDTH OF CHANNEL 600.0
BOTTOM WIDTH OF FLOODWAY 5000.0
MANNINGS N 0.030

DEPTH ABOVE CHANNEL FT	TOTAL DEPTH FT	SLOPE FT/FT	VELOCITY IN CHANNEL FT/SEC	VELOCITY IN OVERBANK FT/SEC	AVERAGE VELOCITY FT/SEC	FLOW WEIGHTED VELOCITY FT/SEC	FLOW IN CHANNEL CFS	FLOW IN OVERBANK CFS	MEAN FROUDE NUMBER	FROUDE NUMBER IN CHANNEL	FROUDE NUMBER IN OVERBANK	LOW FLOW CHANNEL CAPACITY CFS	LOW FLOW CHANNEL VELOCITY FT/SEC
2.0	17.0	0.0023365	15.15	3.80	10.19	13.94	167447.	32553.	1.24	0.70	0.47	134079.8	13.86
2.1	17.1	0.0022347	14.87	3.83	9.94	13.62	165466.	34534.	1.20	0.68	0.46	131126.7	13.55
2.2	17.2	0.0021374	14.61	3.87	9.70	13.31	163502.	36498.	1.16	0.67	0.46	128241.7	13.25
2.3	17.3	0.0020446	14.35	3.90	9.47	13.01	161558.	38442.	1.12	0.65	0.45	125425.2	12.96
2.4	17.4	0.0019560	14.09	3.92	9.25	12.71	159635.	40365.	1.08	0.64	0.44	122677.1	12.68
2.5	17.5	0.0018714	13.84	3.94	9.04	12.42	157736.	42264.	1.04	0.63	0.44	119997.2	12.40
2.6	17.6	0.0017908	13.59	3.96	8.84	12.14	155862.	44138.	1.01	0.61	0.43	117384.9	12.13

TOTAL FLOW 300000.
DEPTH OF CHANNEL 15.0
BOTTOM WIDTH OF CHANNEL 800.0
BOTTOM WIDTH OF FLOODWAY 5000.0
MANNINGS N 0.030

DEPTH ABOVE CHANNEL FT	TOTAL DEPTH FT	SLOPE FT/FT	VELOCITY IN CHANNEL FT/SEC	VELOCITY IN OVERBANK FT/SEC	AVERAGE VELOCITY FT/SEC	FLOW WEIGHTED VELOCITY FT/SEC	FLOW IN CHANNEL CFS	FLOW IN OVERBANK CFS	MEAN FROUDE NUMBER	FROUDE NUMBER IN CHANNEL	FROUDE NUMBER IN OVERBANK	LOW FLOW CHANNEL CAPACITY CFS	LOW FLOW CHANNEL VELOCITY FT/SEC
2.0	17.0	0.0033058	18.20	4.51	13.26	17.12	263087.	36913.	1.42	0.82	0.56	211339.8	16.67
2.1	17.1	0.0031809	17.93	4.57	12.97	16.79	260723.	39277.	1.38	0.81	0.55	207308.6	16.36
2.2	17.2	0.0030608	17.66	4.63	12.70	16.48	258365.	41635.	1.34	0.79	0.55	203355.2	16.04
2.3	17.3	0.0029452	17.39	4.68	12.43	16.16	256016.	43984.	1.30	0.78	0.54	199480.3	15.74
2.4	17.4	0.0028342	17.13	4.72	12.18	15.86	253680.	46320.	1.26	0.76	0.54	195684.4	15.44
2.5	17.5	0.0027276	16.87	4.76	11.94	15.56	251359.	48641.	1.23	0.75	0.53	191967.6	15.15
2.6	17.6	0.0026252	16.62	4.79	11.71	15.27	249056.	50944.	1.19	0.74	0.52	188329.9	14.86
2.7	17.7	0.0025269	16.37	4.82	11.48	14.98	246772.	53228.	1.16	0.72	0.52	184770.8	14.58
2.8	17.8	0.0024326	16.12	4.84	11.27	14.70	244509.	55491.	1.12	0.71	0.51	181289.9	14.30

WATER SURFACE PROFILE CALCULATIONS

Following the preliminary analysis with the uniform flow program, the greenbelts were coded for HEC-2 analysis and run to check for velocity and required levee heights. The runs cover all of Reach A, with greenbelts in the three areas selected and levees elsewhere. Cross-sections altered to represent greenbelts are identified by profile stationing and are therefore easily differentiated from existing condition sections which are identified by river mile.

The following HEC-2 runs, with data listings, are included with this report:

Run 1 - 200,000 cfs, Manning "n" 0.030 in both low-flow channels and greenbelts.

Run 2 - 300,000 cfs, Manning "n" 0.030 in both low-flow channels and greenbelts.

Run 3 - 200,000 cfs Manning "n" 0.030 in low-flow channels, 0.050 in greenbelts.

Run 4 - 300,000 cfs, Manning "n" 0.030 in low-flow channels, 0.050 in greenbelts.

The greenbelts were modified until acceptable results were obtained from the HEC-2 analysis. The HEC-2 outputs included with the report represent the final designs.

RESULTS OF HYDRAULIC ANALYSIS

In spite of the differences in the slope and width of the three greenbelts, it was found that acceptable results could be achieved in all three areas with a low-flow channel having a bottom width of 600 feet for 200,000 cfs and 800 feet for 300,000 cfs. In general, meeting the velocity criteria of 7 feet per second in the greenbelt was not the determining factor. Instead, maintaining

moderate velocities in the low-flow channels necessitated wider channels than would be required simply to control greenbelt velocities. As the low-flow channel is widened, however, excavation cost increases and greenbelt land is lost to the channel as a higher percentage of the flow is carried in the low-flow channel. Also, the water surface is lowered considerably, complicating the transitions into and out of the greenbelts. The trade-off involves what is acceptable for a low-flow channel velocity versus the integrity of the greenbelt concept. It was found that keeping channel velocities below 12 feet per second (as in the earth-bottom channel alternatives) was either not possible or would require extremely wide channels. Therefore, channel velocities were kept as close to 12 feet per second as possible, but were allowed to climb as high as 15 feet per second in some cases. If the greenbelt concept is carried on for further analysis in later planning stages, this problem should be studied further and the design concept modified if required to avoid these higher velocities.

A summary of the key hydraulic parameters is presented in Table IV-2. The variations in flow depth and velocity within the same greenbelt floodway are caused by variations in the width ofthe floodways. The Mesa Greenbelt has a much wider range of flow depths and velocities than do the other two greenbelts because of a substantial amount of backwater at its downstream end. This backwater accounts for the higher depths and lower velocities, results from a natural constriction, and is quickly dissipated in the greenbelt. The Mesa Greenbelt is also unique in that it has a much steeper slope than either of the other two greenbelts or the river in general. This slope creates problems for controlling velocities in both the channel and the greenbelt. Therefore, it was necessary to excavate the overbank area 8 feet. Without this excavation, the low-flow channel would be 15 feet deep, but almost all the flow would end up in the channel at very high velocities. The Mesa Greenbelt is not as well suited to being a greenbelt and will be much more costly to construct and probably less stable than the other two greenbelts. The overbank areas of the Phoenix-Tempe and Indian Greenbelts do not need to be lowered, but must be graded to their average elevation.

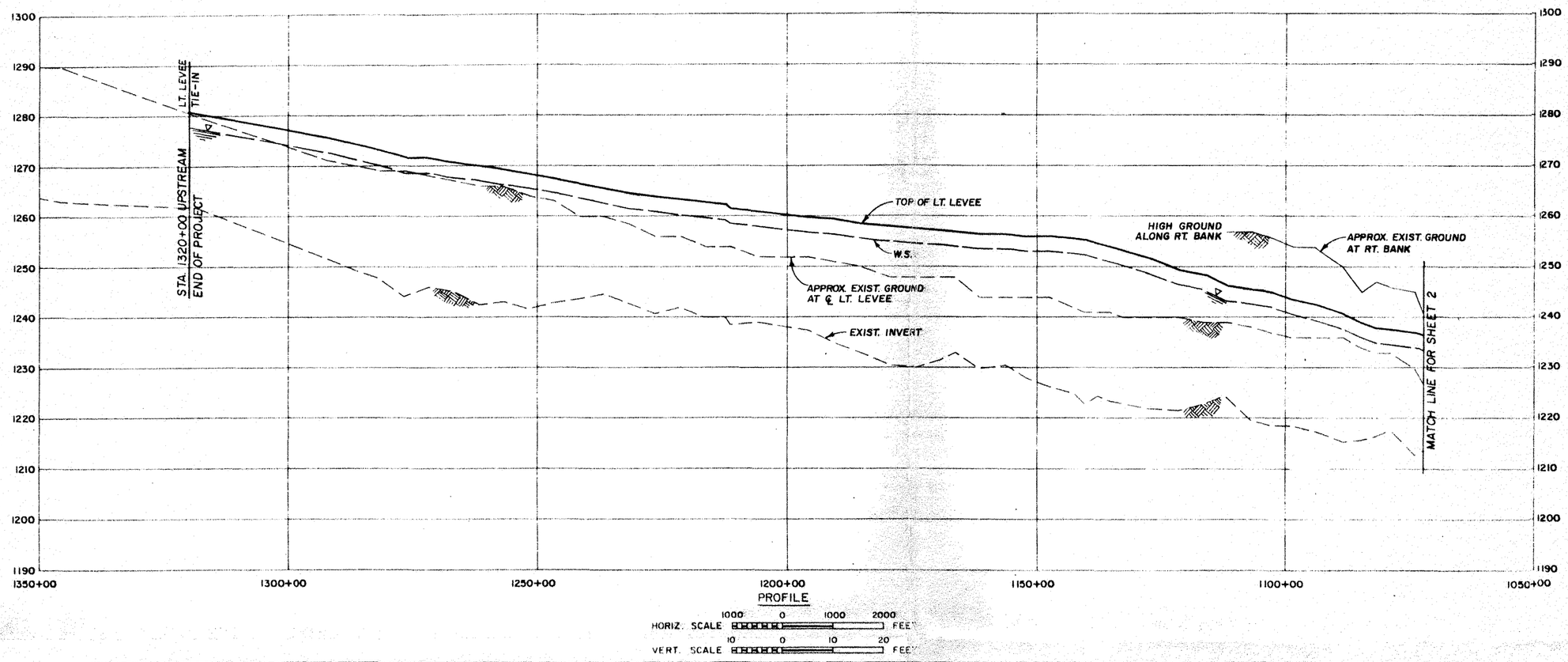
TABLE IV-2

KEY HYDRAULIC PARAMETERS
GREENBELT FLOODWAYS

	Phoenix-Tempe Greenbelt				Mesa Greenbelt				Indian Greenbelt			
Flow (cfs)	200,000	200,000	300,000	300,000	200,000	200,000	300,000	300,000	200,000	200,000	300,000	300,000
Channel "n"	.030	.030	.030	.030	.030	.030	.030	.030	.030	.030	.030	.030
Greenbelt "n"	.030	.050	.030	.050	.030	.050	.030	.050	.030	.050	.030	.050
Channel Bottom Width (ft)	600	600	800	800	600	600	800	800	600	600	800	800
Slope (ft/ft)	.0013	.0013	.0013	.0013	.00268	.00268	.00268	.00268	.00175	.00175	.00175	.00175
Channel Depth (ft)	14	14	14	14	7	7	7	7	13	13	13	13
Flow Depth (ft)	17.9-19.4	18.8-20.0	19.3-21.0	20.2-21.7	11.6-18.1	12.5-18.3	12.7-21.8	14.0-22.0	15.7-19.7	16.2-19.5	16.4-23.3	17.3-23.1
Channel Velocity (ft/sec)	10.7-13.7	11.7-13.8	11.6-14.5	12.6-14.9	4.1-13.7	5.8-14.7	4.4-14.4	6.2-15.2	7.8-12.8	10.0-13.3	7.1-13.6	9.2-14.0
Greenbelt Velocity (ft/sec)	4.5-5.8	3.0-3.7	5.4-7.0	3.8-4.5	3.0-7.0	2.5-5.0	3.4-7.0	2.8-5.8	3.9-4.4	2.6-3.1	4.2-5.1	3.1-3.5
% of Flow in Channel	69-83	73-88	66-84	75-90	27-53	34-61	27-50	38-59	50-66	61-70	50-63	61-69

An interesting result in all three greenbelts is that the water surface elevations are lower than for the levee alternatives. This is because the reduced efficiency of the overbanks is more than outweighed by the increased flow area and efficiency of the low-flow channel. Also, the depth of flow for the case where the greenbelt friction factor is 0.05 is not very much higher than that for the case of 0.03. The higher overbank friction factor forces more flow into the efficient low-flow channel.

The greenbelt areas (having velocities below 7 feet per second) are shown as the shaded areas on the plan and profile sheets. The invert toes of the low-flow channels are also shown.



PROFILE

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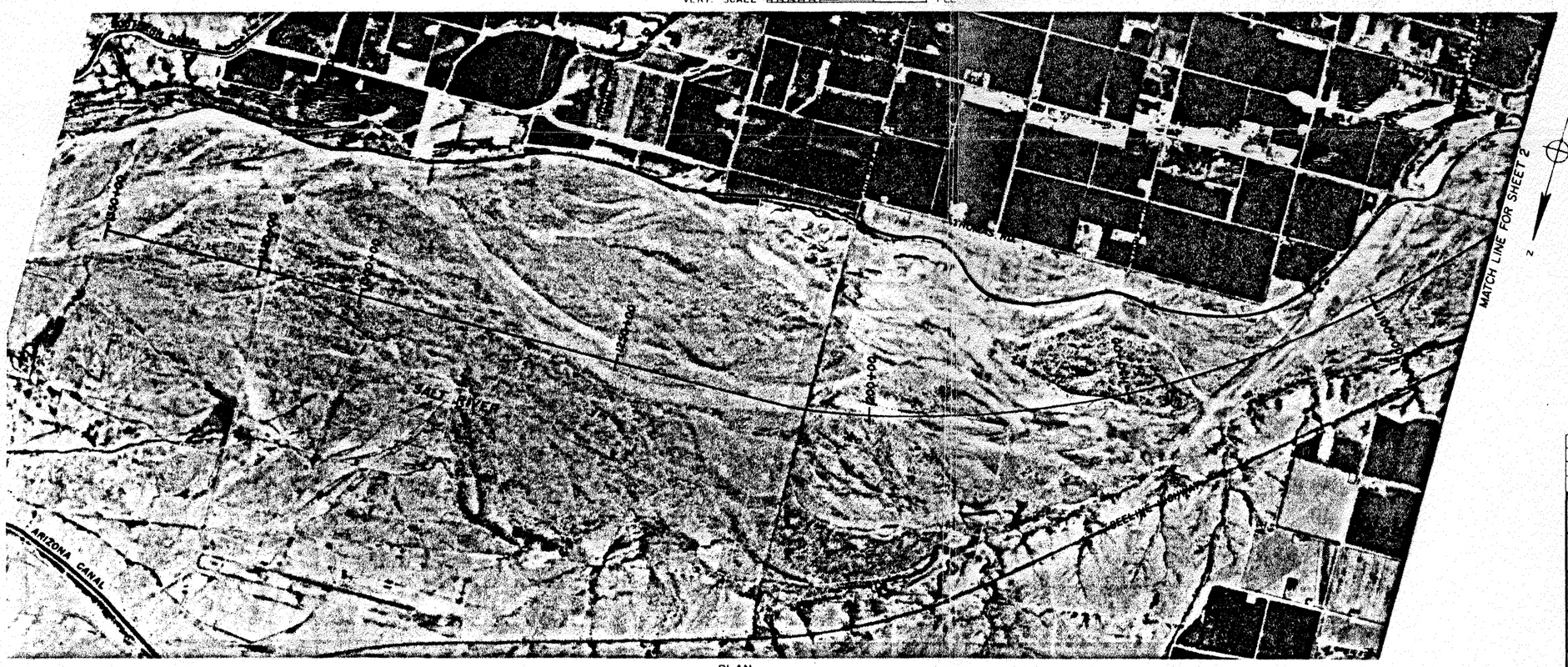
VERT. SCALE 10 0 10 20 FEET



PLAN

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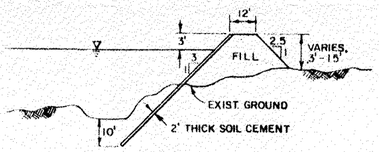
PRELIMINARY



TYPICAL SECTION
LT. LEVEE
NOT TO SCALE

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

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1333 South Broadway
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CORPS OF ENGINEERS

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CENTRAL ARIZONA WATER CONTROL STUDY
STRUCTURAL FLOOD CONTROL ALTERNATIVES

GILA RIVER AND SALT RIVER CHANNELIZATION

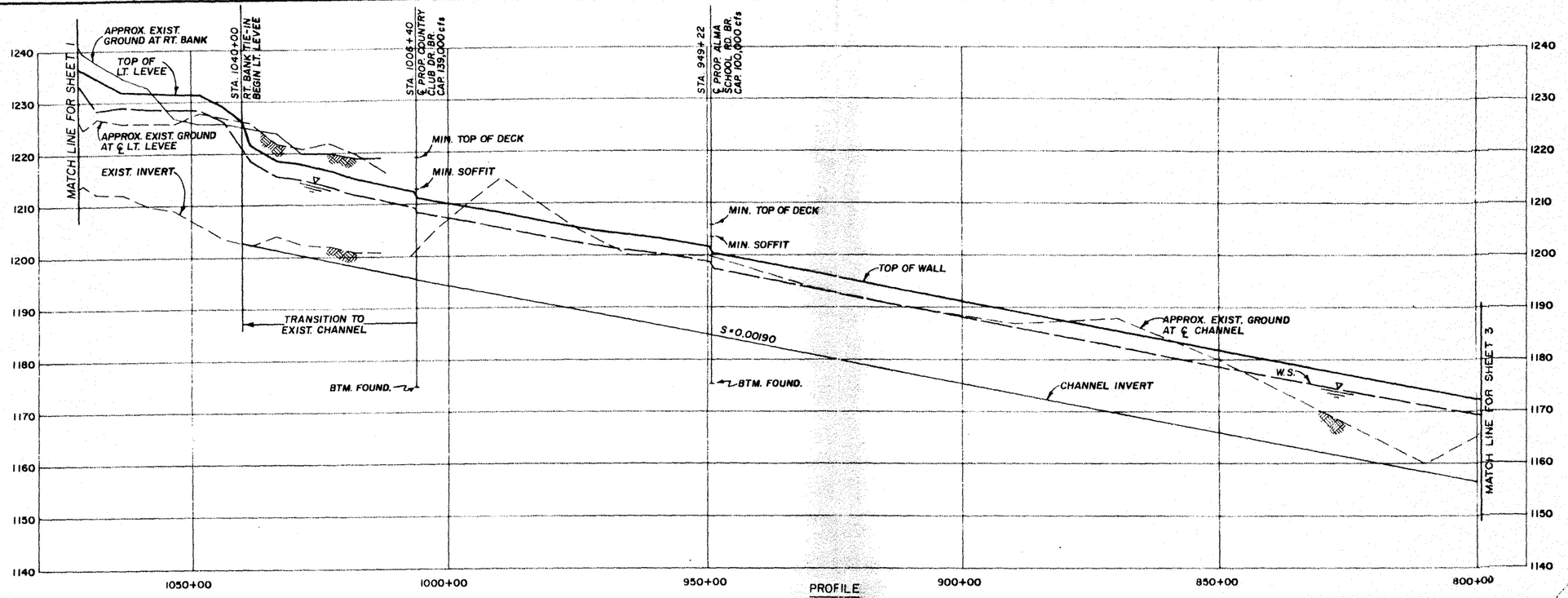
CHANNELS
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
COUNTRY CLUB DRIVE TO 35th AVENUE

SUBMITTED BY:
DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO.

SHEET 1 OF 5

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



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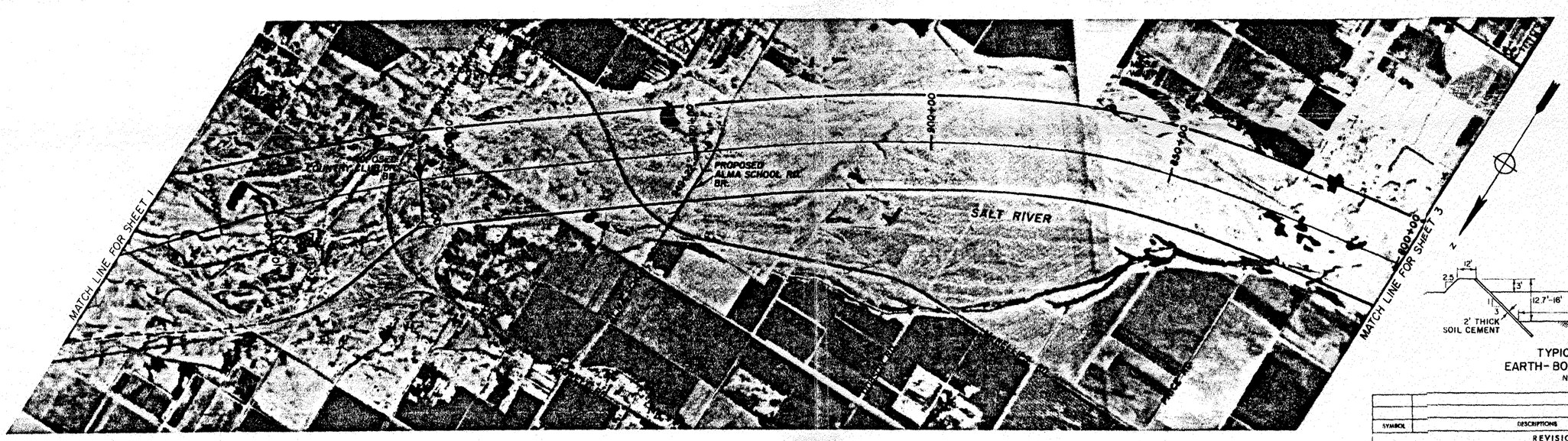

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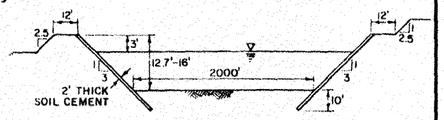

TYPICAL SECTION
EARTH-BOTTOM CHANNEL
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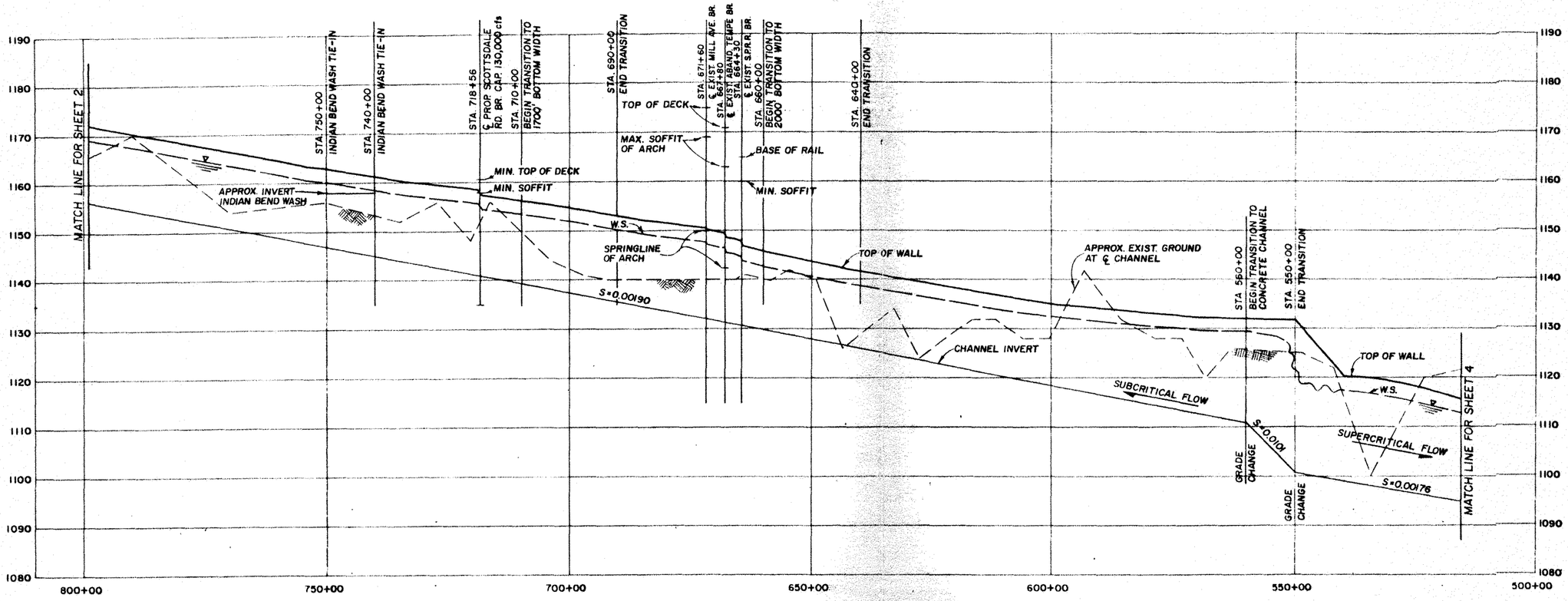
SHEET 2 OF 5

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE

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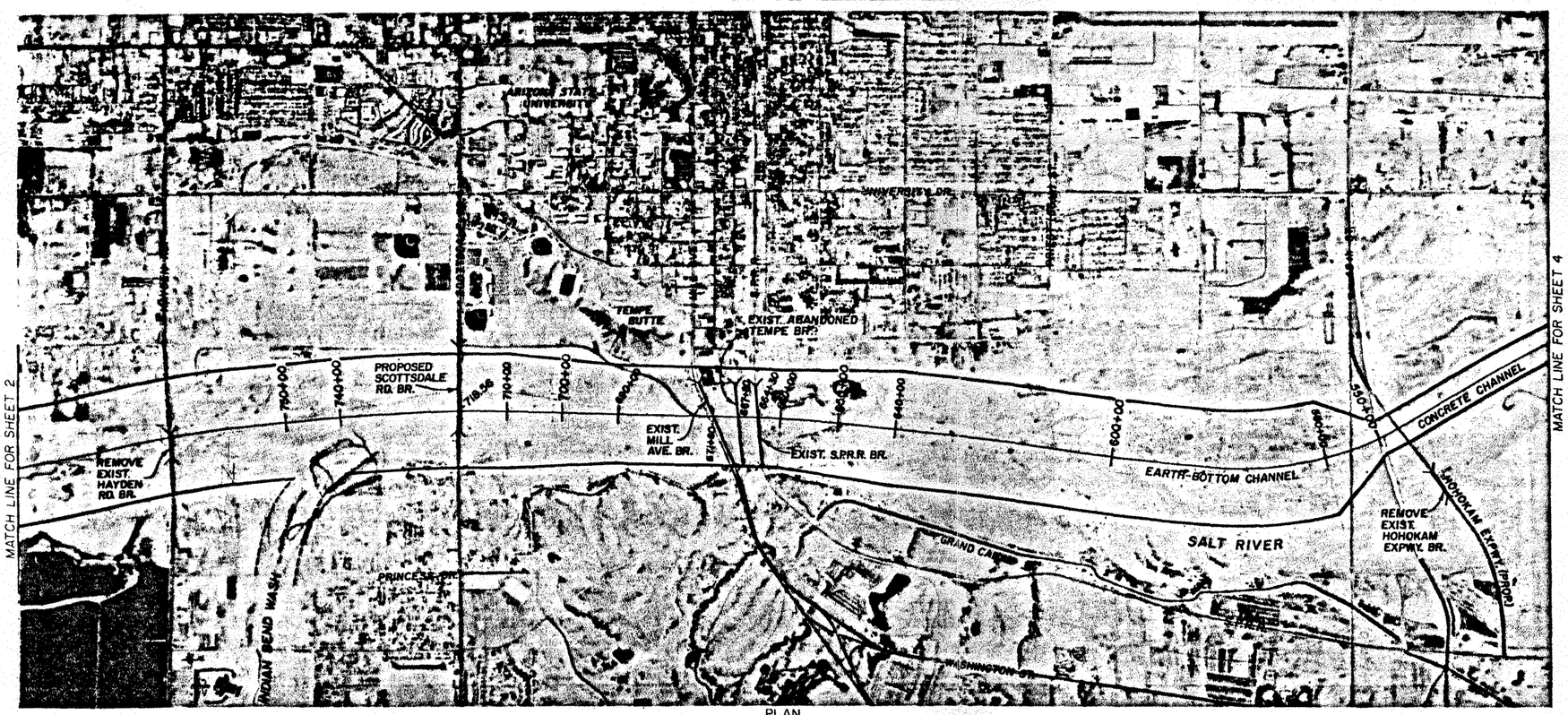

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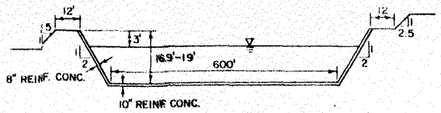
PRELIMINARY



TYPICAL SECTION
CONCRETE CHANNEL
NOT TO SCALE

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GILA RIVER AND SALT RIVER CHANNELIZATION

CHANNELS
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
COUNTRY CLUB DRIVE TO 35th AVENUE

SUBMITTED BY
DATE APPROVED
CHIEF, MANOF

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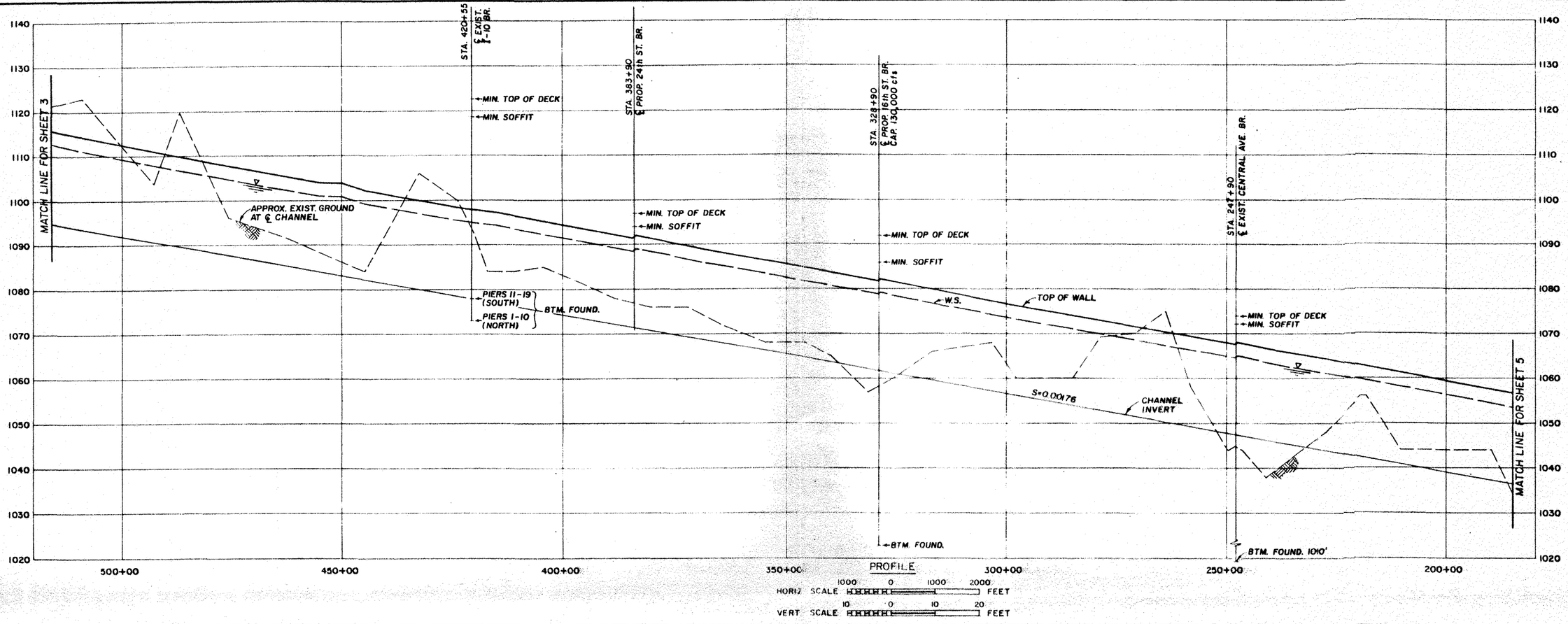
DISTRICT FILE NO.

SHEET 3 OF 5

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE

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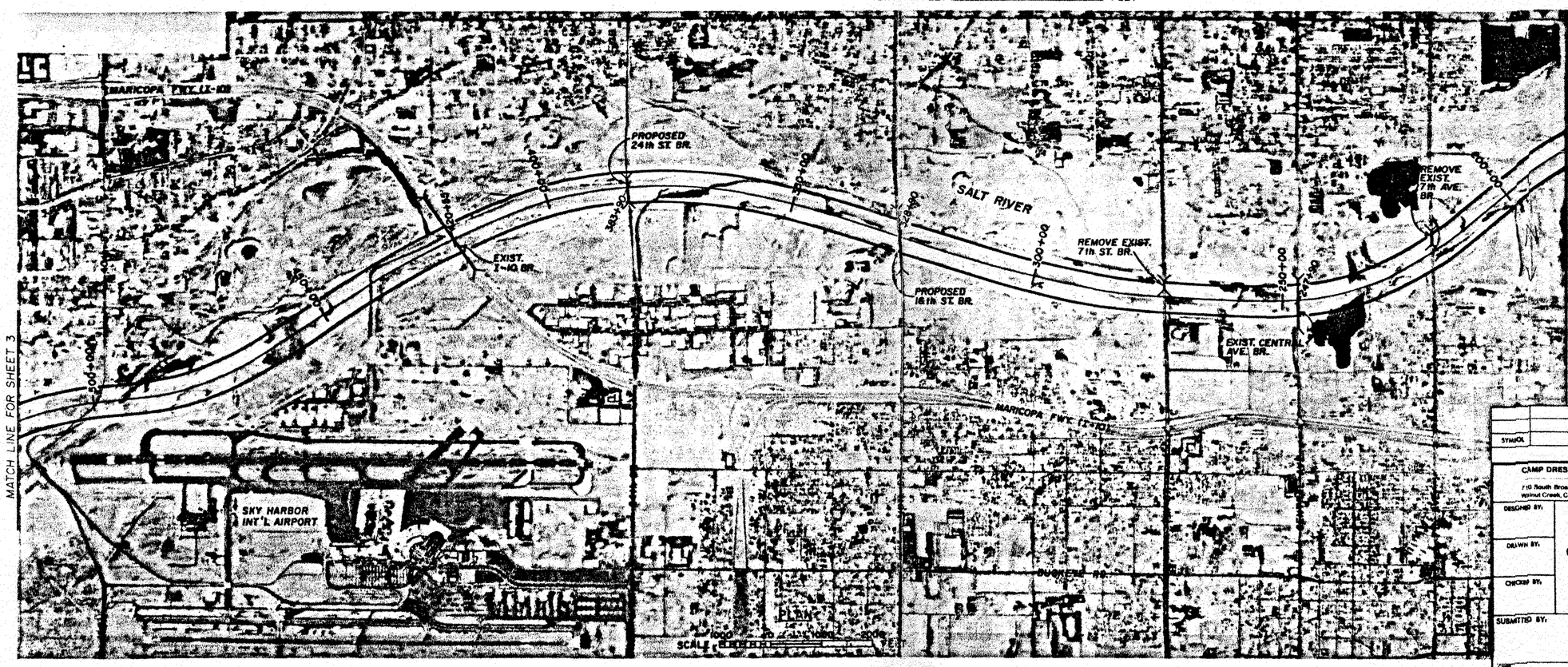

PRELIMINARY

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CHANNELS
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
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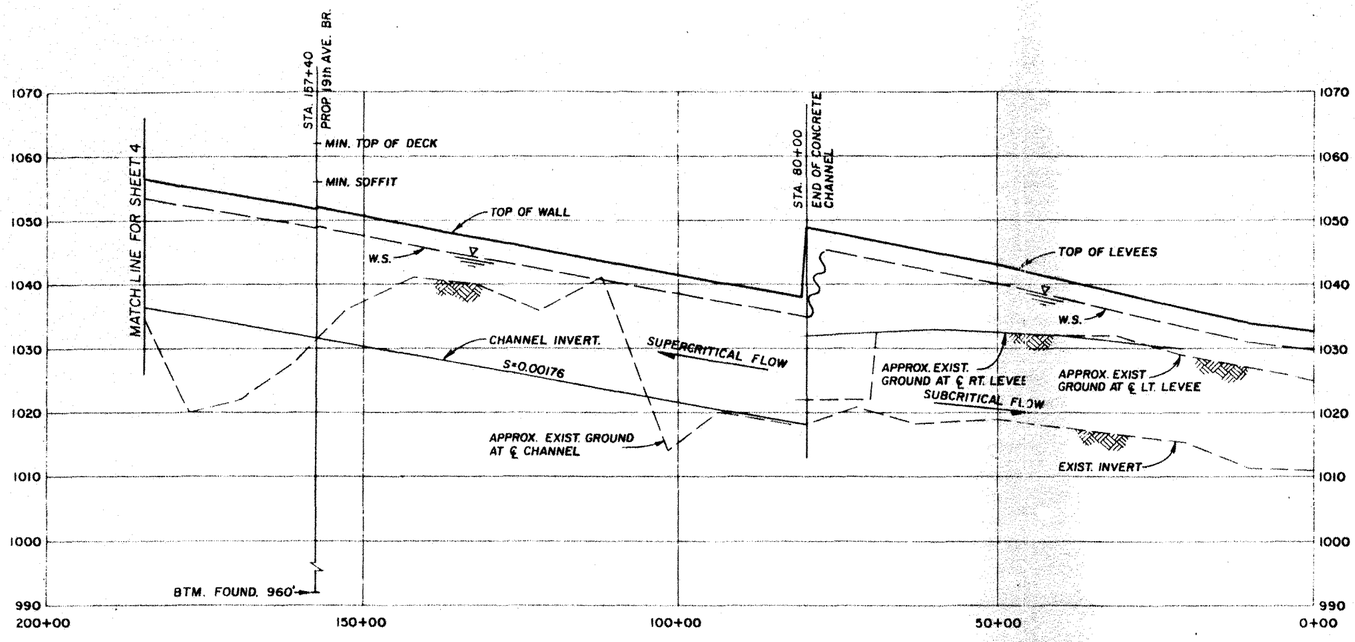
SHEET 4 OF 5

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE

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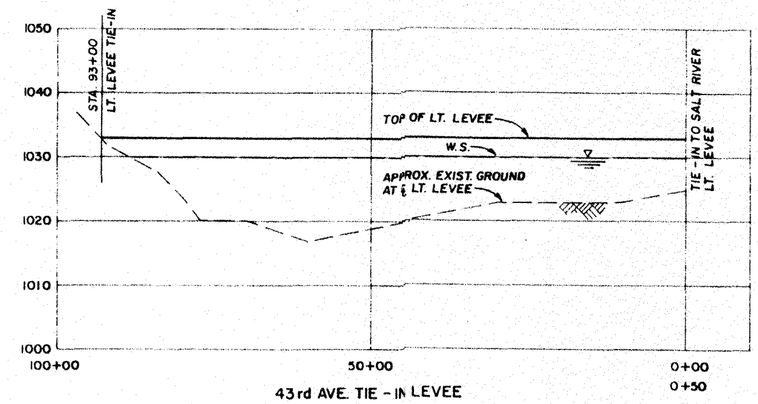
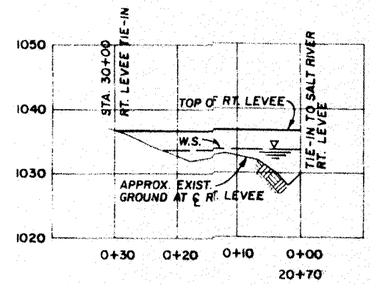


43rd AVE. TIE - IN LEVEE



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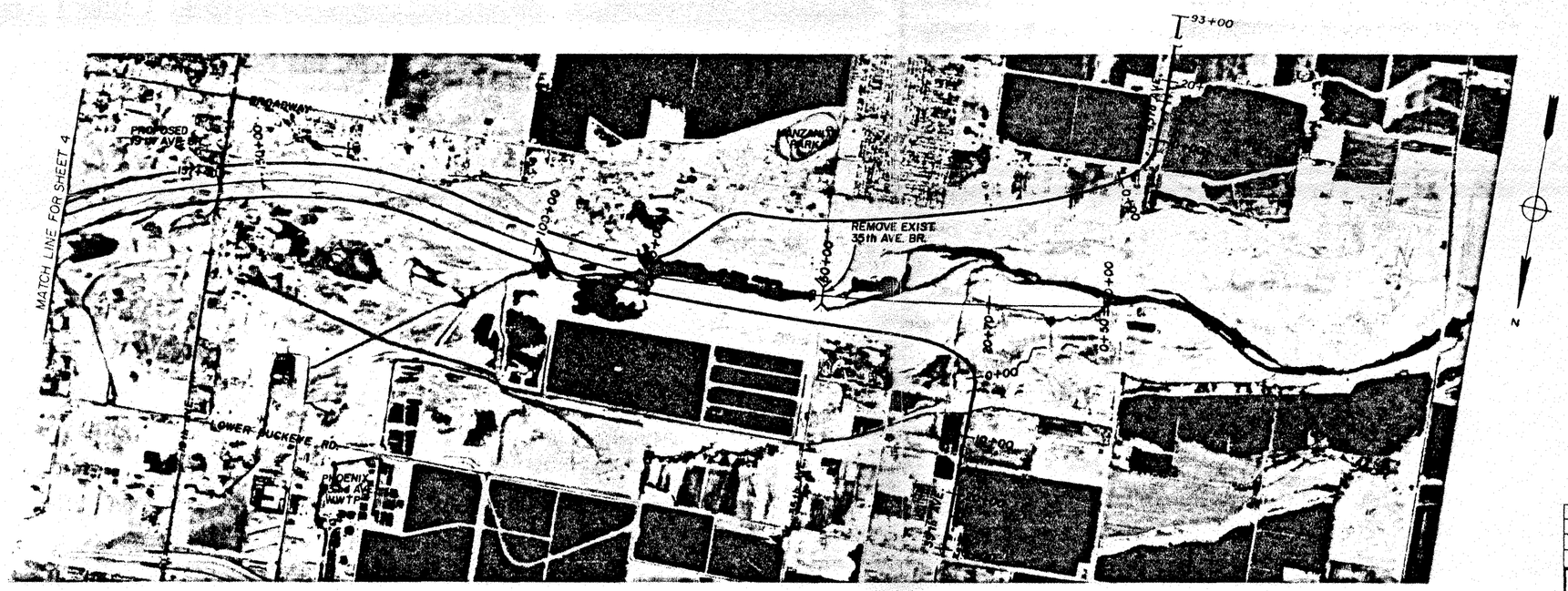

PLAN

SCALE 1000 0 1000 2000 FEET

PRELIMINARY

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
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CHANNELS
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
COUNTRY CLUB DRIVE TO 35th AVENUE

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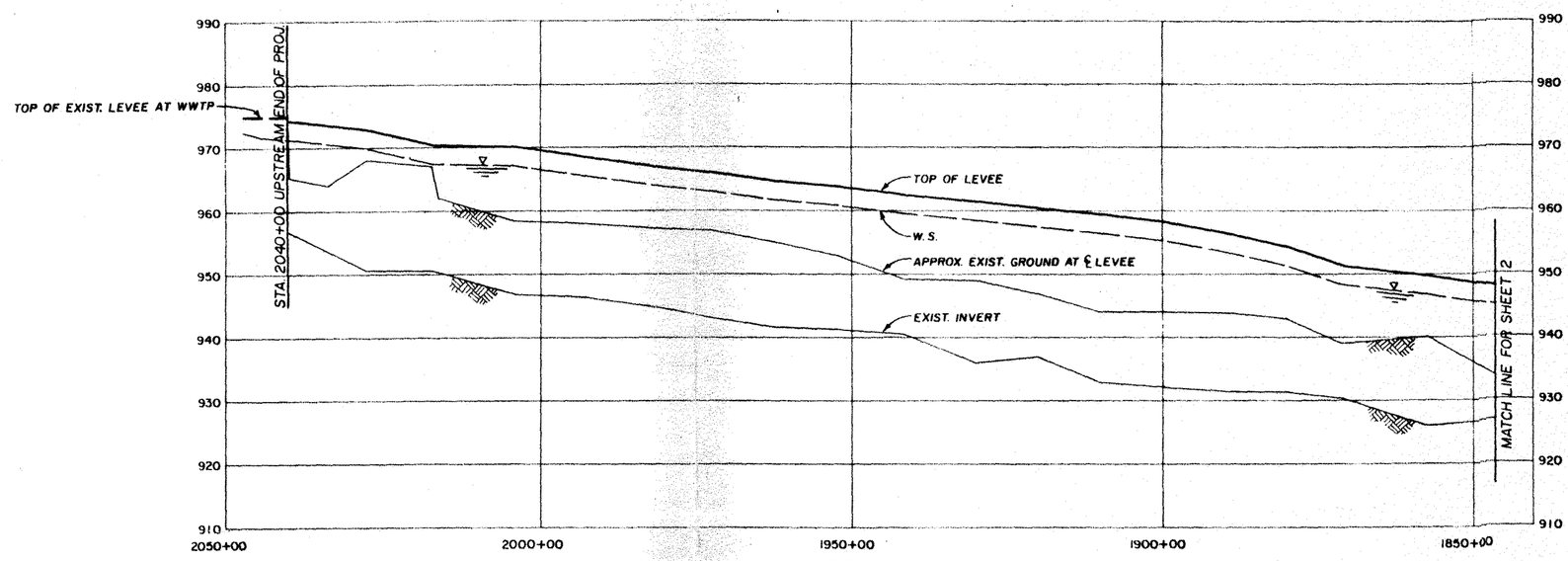
SHEET 5 OF 5

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE

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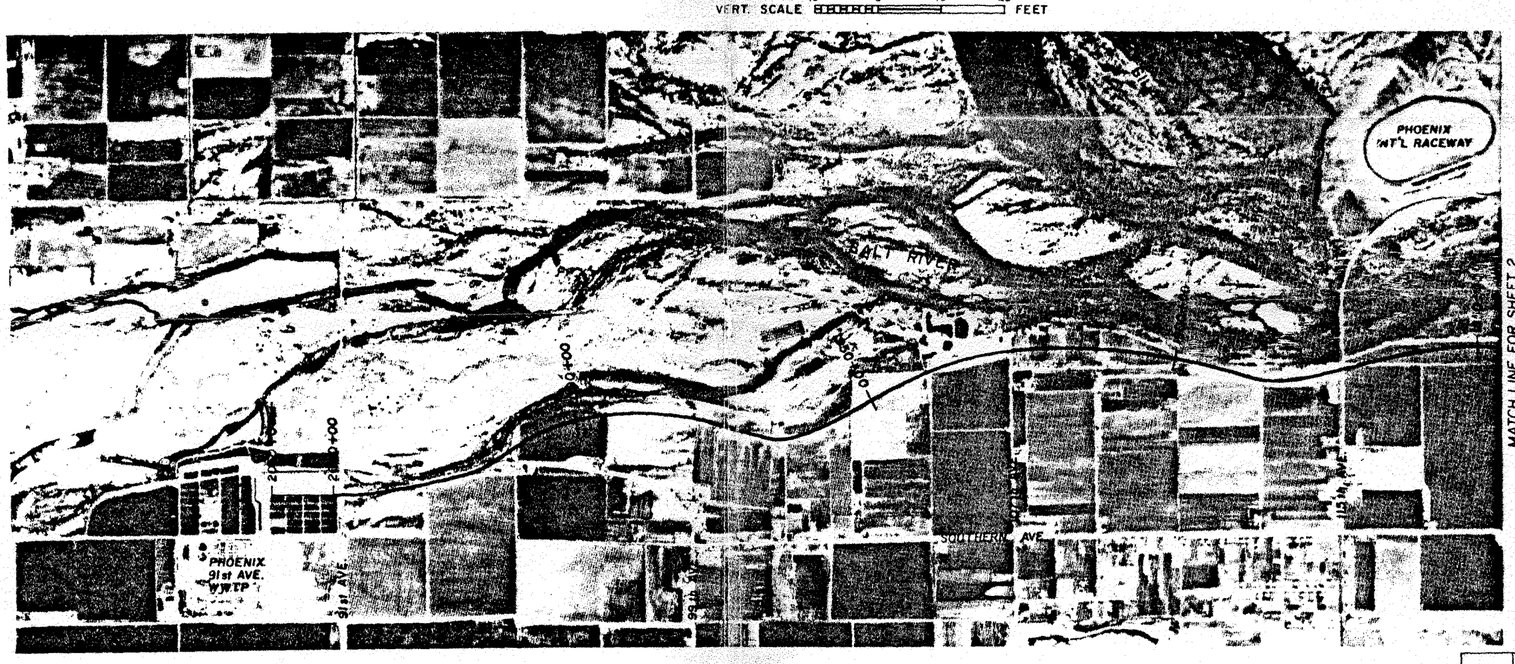

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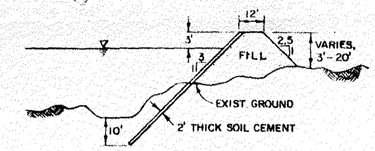

TYPICAL SECTION
LEVEES
NOT TO SCALE

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STRUCTURAL FLOOD CONTROL ALTERNATIVES

GILA RIVER AND SALT RIVER CHANNELIZATION

LEVEES
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
91st AVENUE TO GILLESPIE DAM

SUBMITTED BY:
DATE APPROVED:
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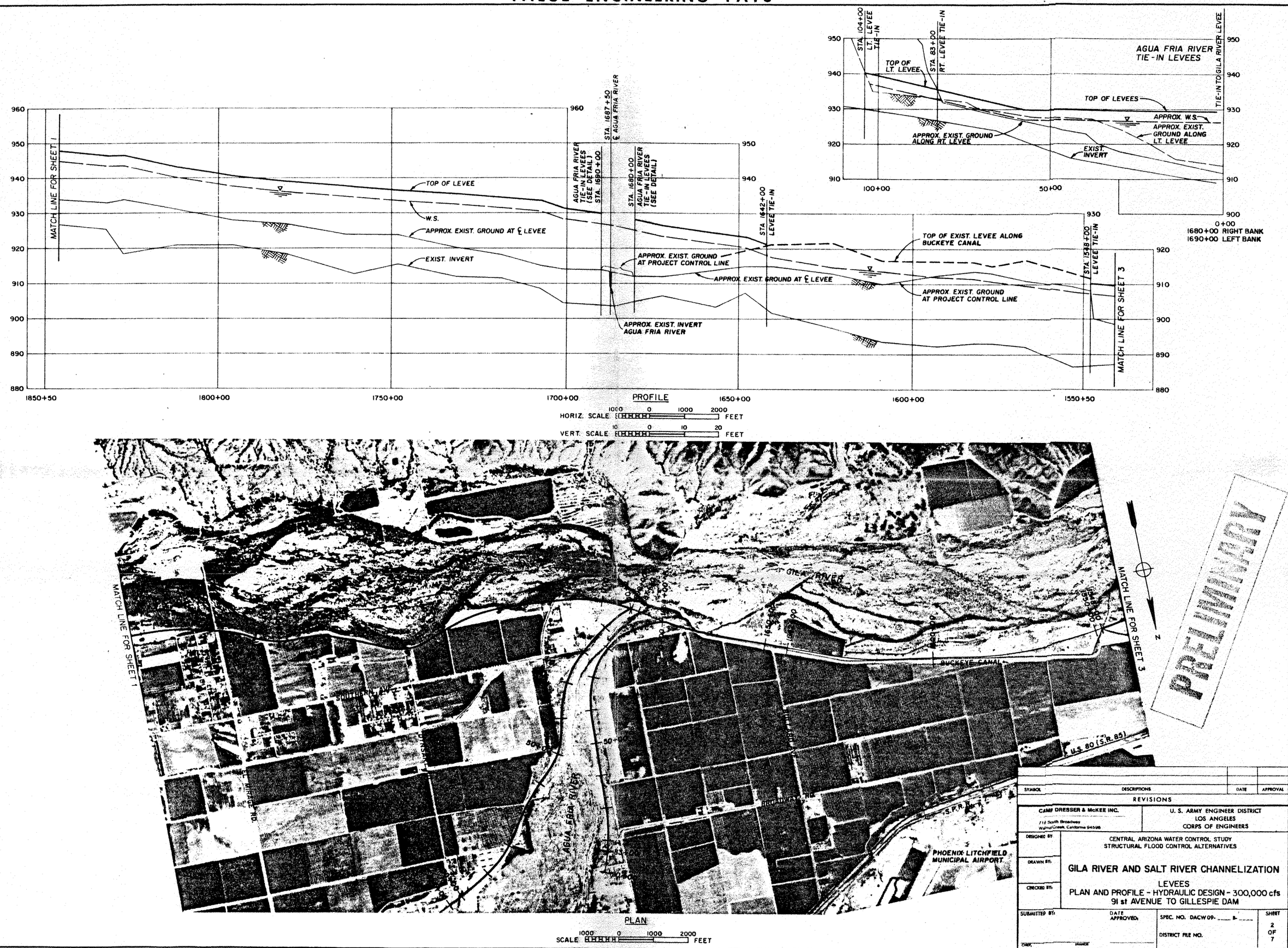
SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO.

SHEET 1 OF 7



SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

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LOS ANGELES
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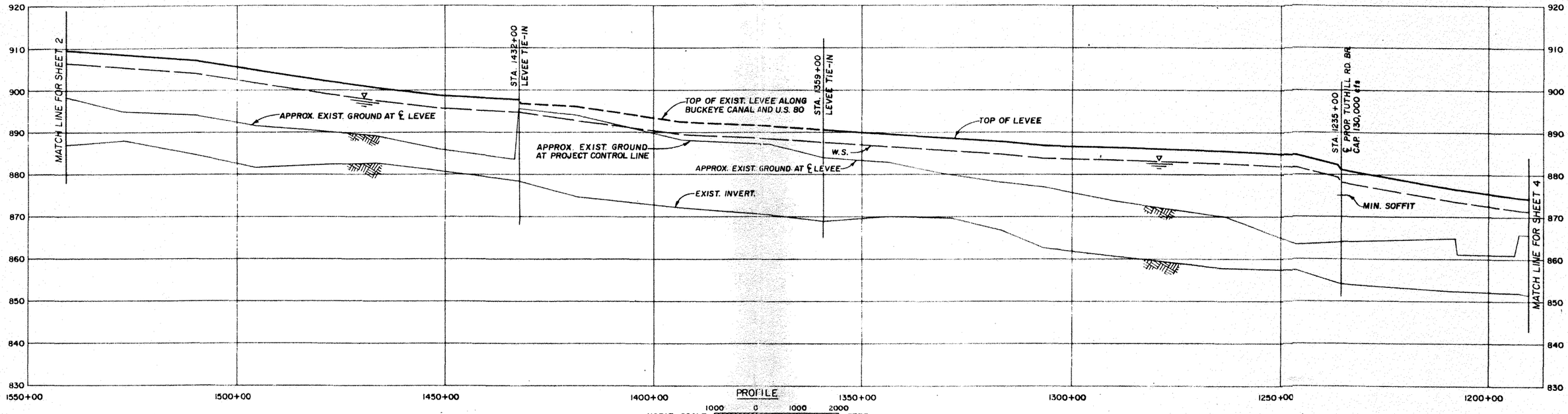
STRUCTURAL FLOOD CONTROL ALTERNATIVES

GILA RIVER AND SALT RIVER CHANNELIZATION
LEVEES
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
91 st AVENUE TO GILLESPIE DAM

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09-____-B-____ SHEET 2 OF 7
DISTRICT FILE NO.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE

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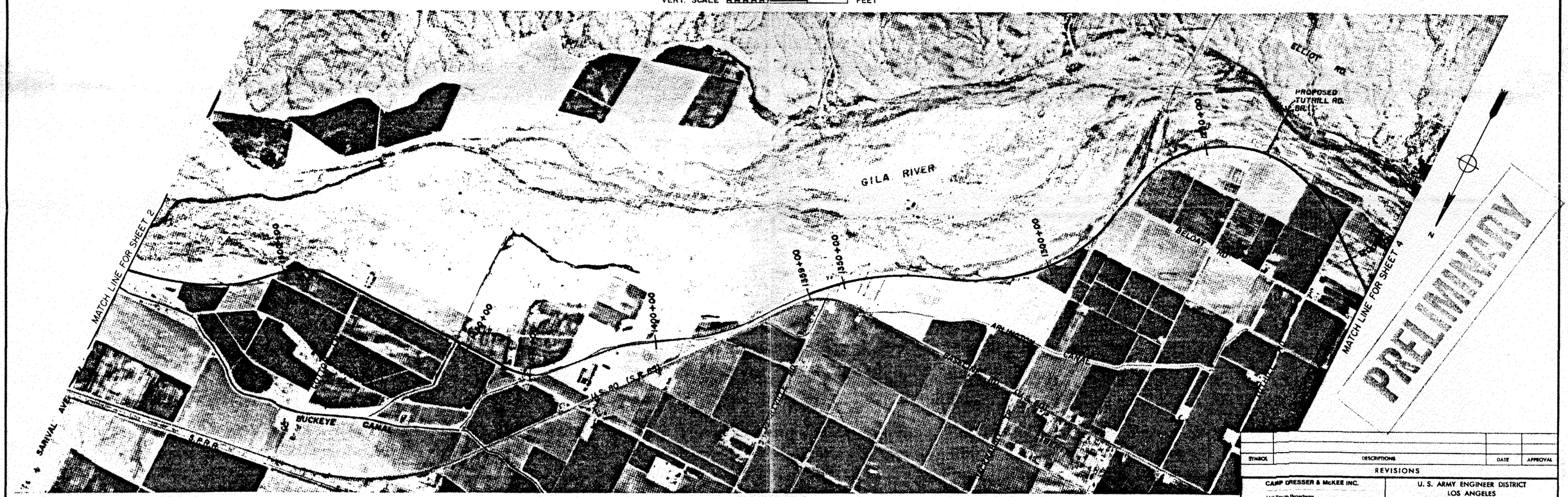

PLAN

SCALE 1000 0 1000 2000 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

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STRUCTURAL FLOOD CONTROL ALTERNATIVES

GILA RIVER AND SALT RIVER CHANNELIZATION

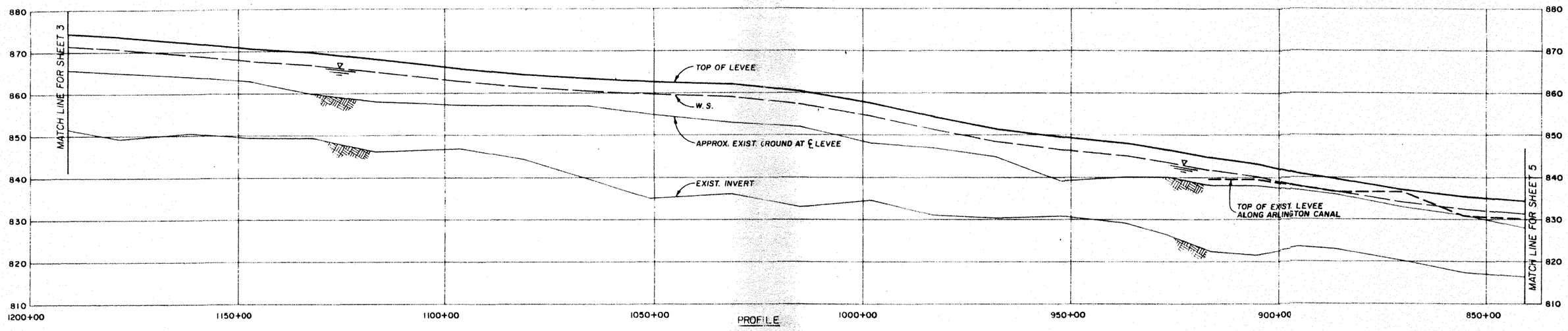
LEVEES
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
91 st AVENUE TO GILLESPIE DAM

SUBMITTED BY:
DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO.

SHEET 3 OF 7

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



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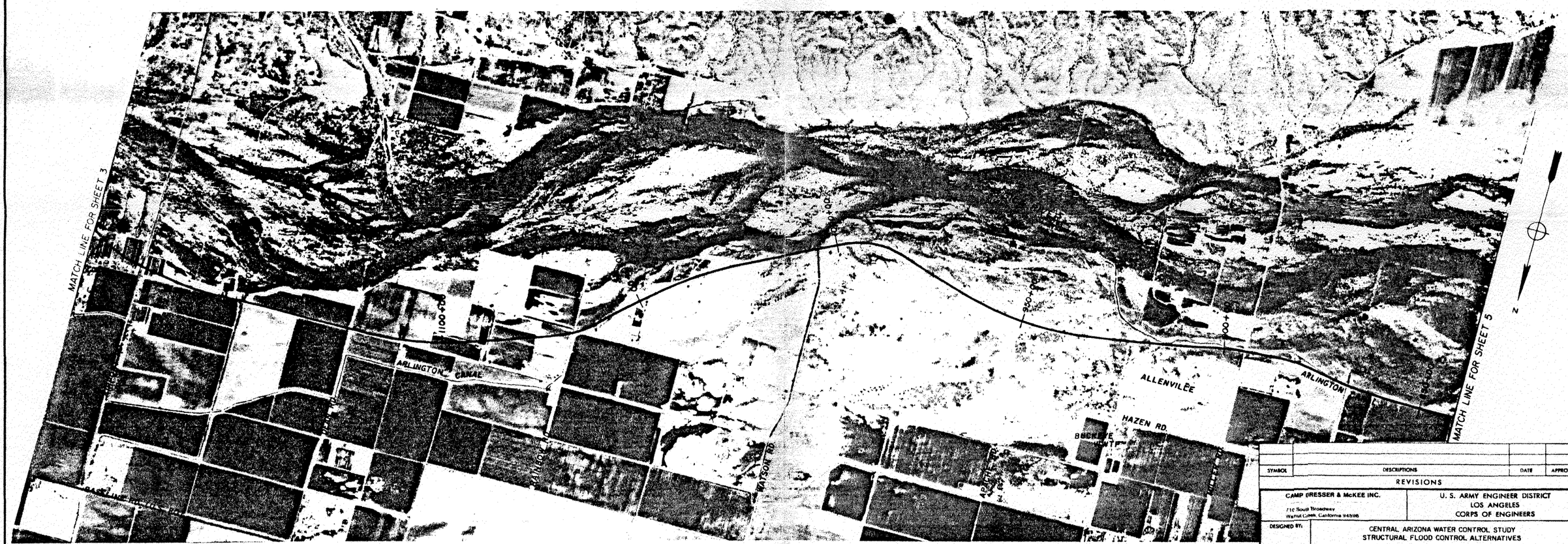

PLAN

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PRELIMINARY

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

CAMP DRESSER & McKEE INC.
716 South Broadway
Walnut Creek, California 94596

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

CENTRAL ARIZONA WATER CONTROL STUDY
STRUCTURAL FLOOD CONTROL ALTERNATIVES

GILA RIVER AND SALT RIVER CHANNELIZATION

LEVEES
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
91 st AVENUE TO GILLESPIE DAM

SUBMITTED BY:
CHIEF, BRANCH

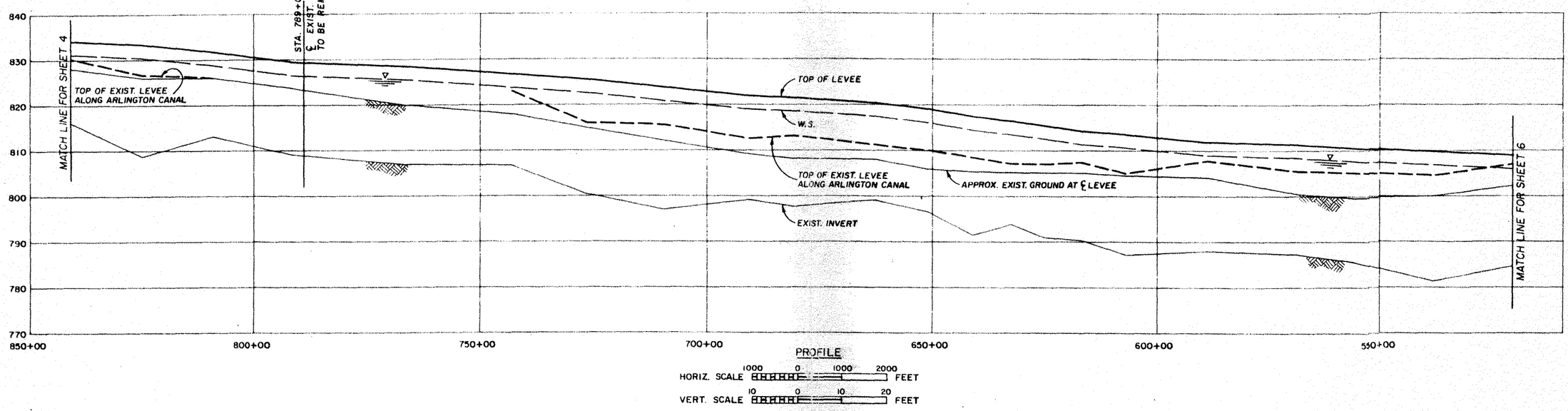
DATE APPROVED:

SPEC. NO. DACW09- ____ B- ____

DISTRICT FILE NO.

SHEET 4 OF 7

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



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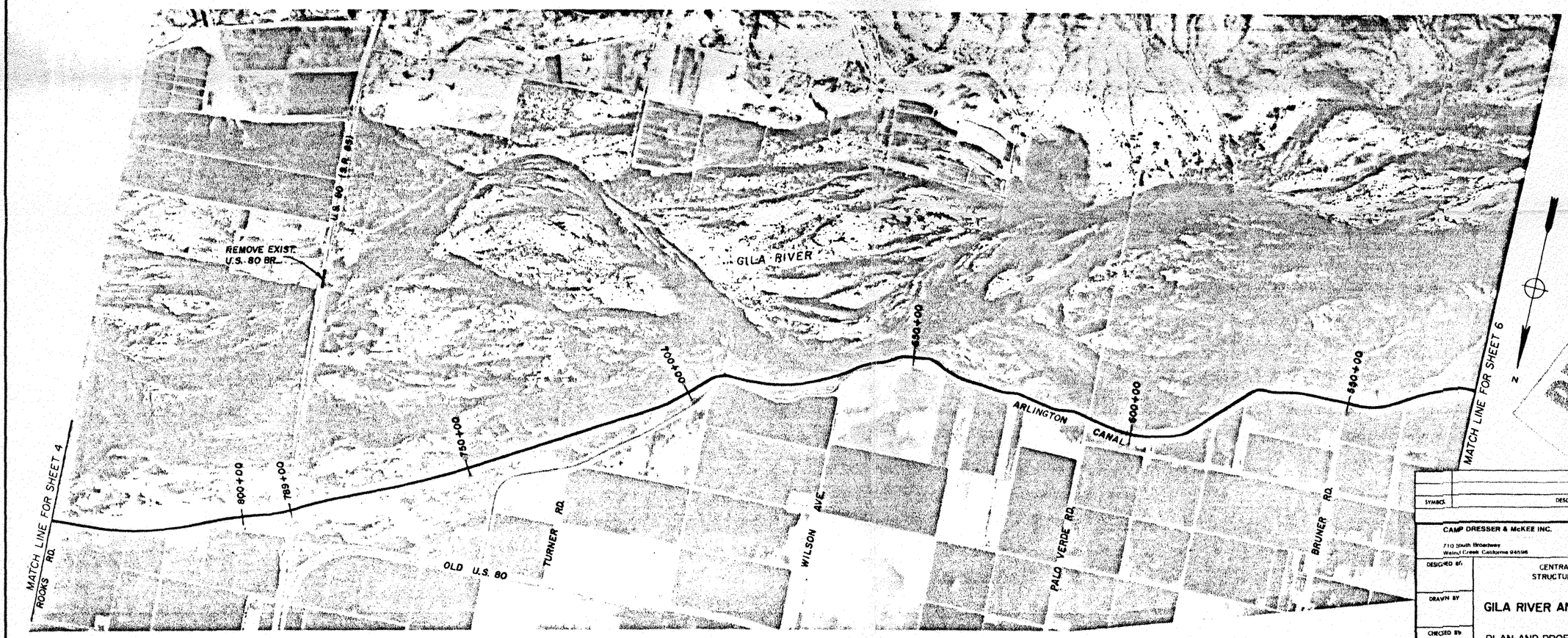

PRELIMINARY

PLAN

SCALE 1000 0 1000 2000 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

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CAMP DRESSER & McKEE INC.
710 South Broadway
Walnut Creek California 94596

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

CENTRAL ARIZONA WATER CONTROL STUDY
STRUCTURAL FLOOD CONTROL ALTERNATIVES

GILA RIVER AND SALT RIVER CHANNELIZATION

LEVEES
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
91 st AVENUE TO GILLESPIE DAM

SUBMITTED BY: DATE APPROVED:

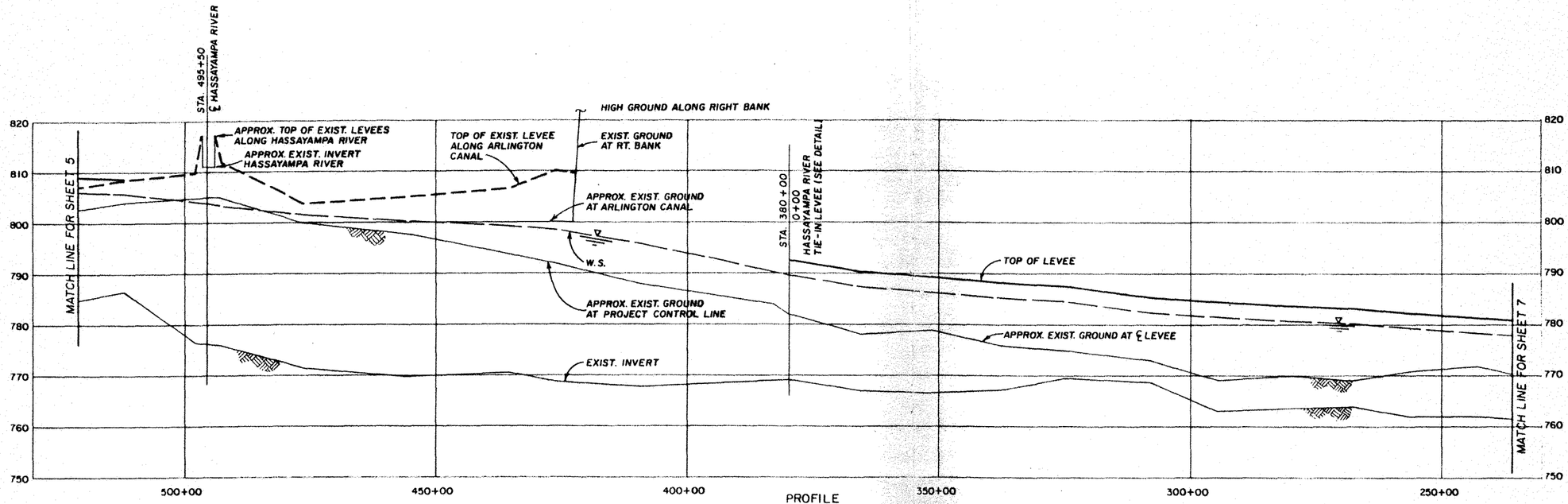
SPEC. NO. DACW09- ____ B- ____

DISTRICT FILE NO.

SHEET 5 OF 7

CHIEF, DRAFTER

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE

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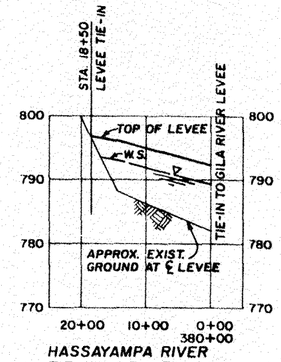
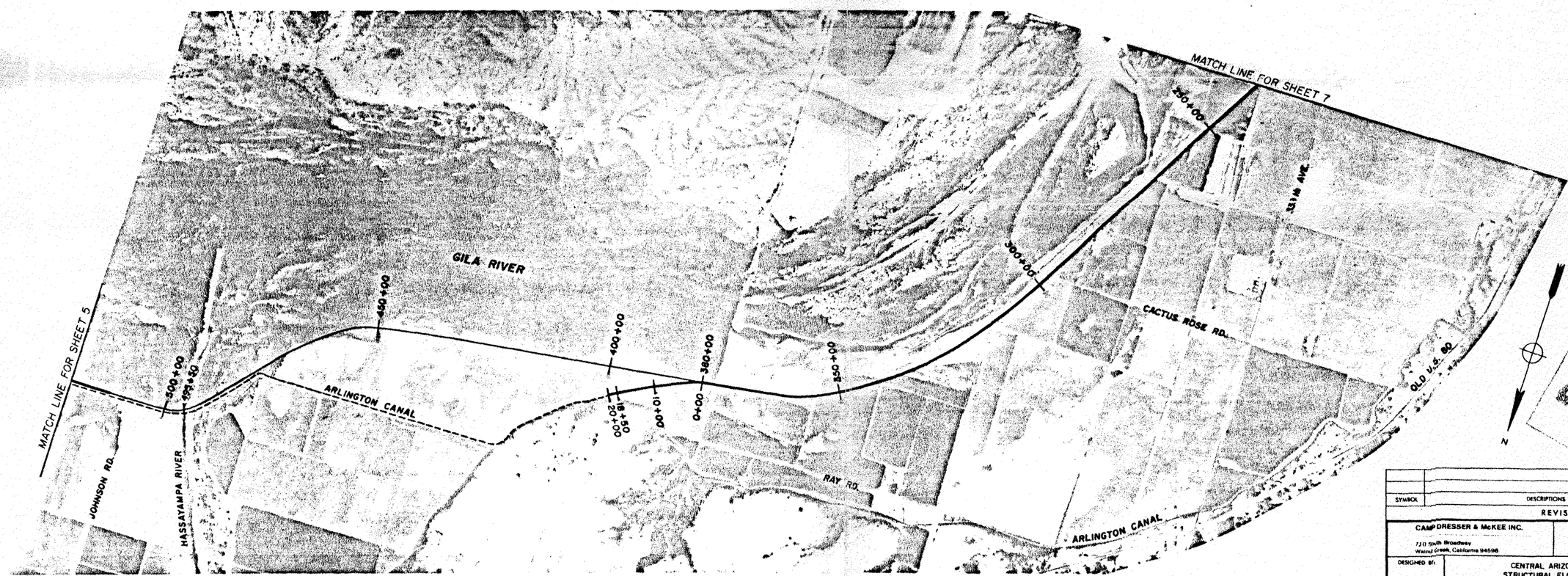


HASSAYAMPA RIVER TIE - IN LEVEE



PLAN

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SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

CAMP DRESSER & McKEE INC.
720 South Broadway
Walnut Creek, California 94596

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY
CHECKED BY

CENTRAL ARIZONA WATER CONTROL STUDY
STRUCTURAL FLOOD CONTROL ALTERNATIVES

GILA RIVER AND SALT RIVER CHANNELIZATION

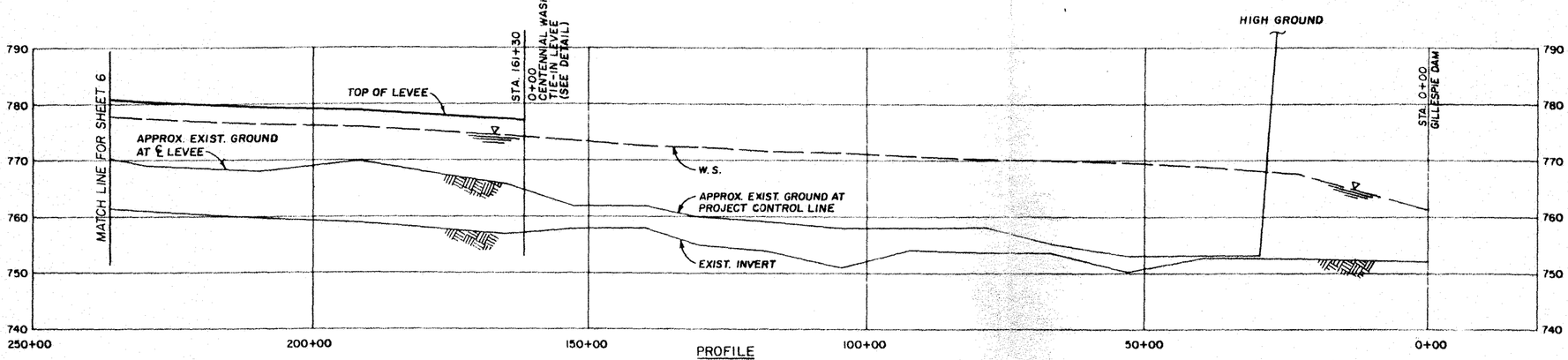
LEVEES
PLAN AND PROFILE - HYDRAULIC DESIGN - 300,000 cfs
91 st AVENUE TO GILLESPIE DAM

SUBMITTED BY: DATE APPROVED:
CHIEF MANAGER

SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO.

SHEET 6 OF 7

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE

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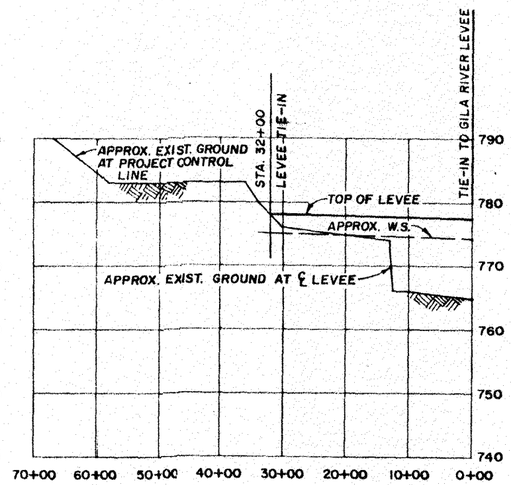
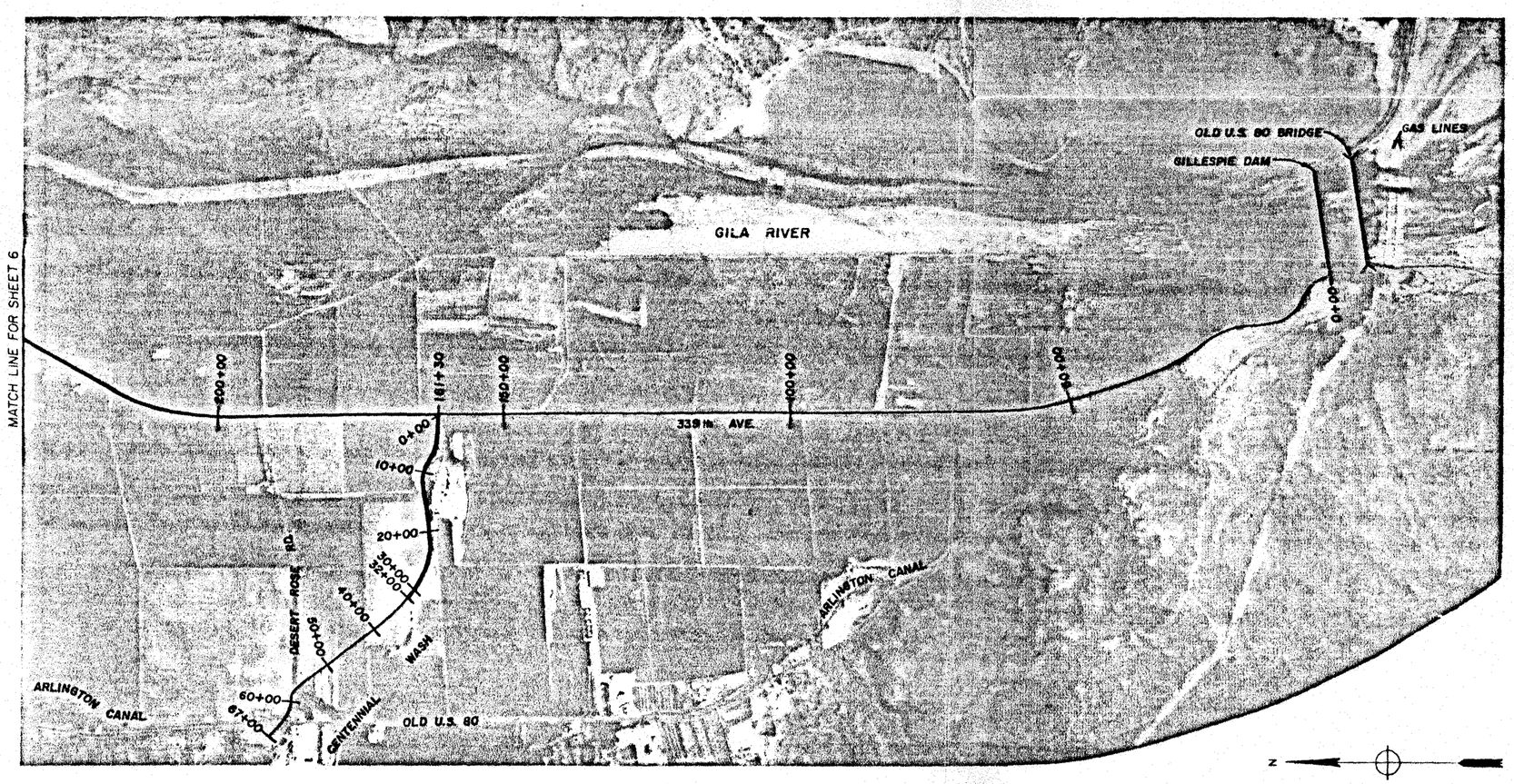


CENTENNIAL WASH TIE-IN LEVEES



PLAN

SCALE 1000 0 1000 2000 FEET

PRELIMINARY

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

CAMP DRESSER & McKEE INC.
110 South Broadway
Walnut Creek, California 94596

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

CENTRAL ARIZONA WATER CONTROL STUDY
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GILA RIVER AND SALT RIVER CHANNELIZATION

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SUBMITTED BY:
DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW 09- ____ B- ____
DISTRICT FILE NO.

SHEET 7 OF 7

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING