NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE ENHANCEMENT PROJECT
PHASE IVA

FINAL REPORT

Prepared for

Edwards Underground Water District

by

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NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE ENHANCEMENT PROJECT
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EXECUTIVE SUMMARY

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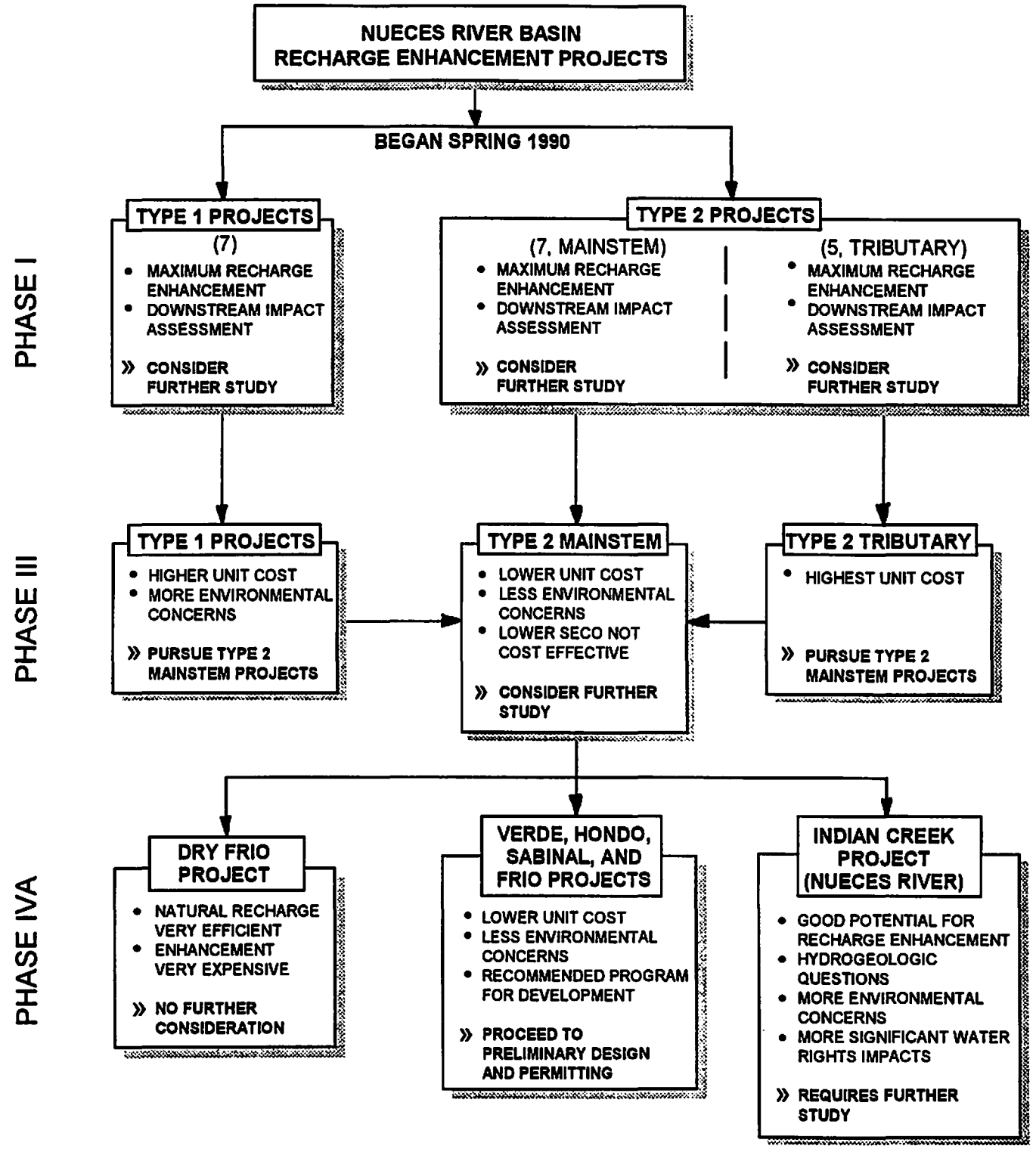
1.0 Background and Objectives

The Edwards Aquifer is the principal source of water supply for the City of San Antonio, as well as numerous other communities and agricultural interests throughout south-central Texas. The aquifer also supplies Leona, San Pedro, San Antonio, Comal, and San Marcos Springs, creating unique environments and recreational opportunities while providing base flow to the Leona, San Antonio, Guadalupe, and San Marcos Rivers. Over the past several decades, the increasing water demands on the Edwards Aquifer have raised concerns about the ability of the aquifer to meet these demands without causing social, economic, and environmental problems.

The headwaters of the Nueces River Basin contribute about 51 percent of the total volume of surface water recharged to the San Antonio portion of the Edwards Aquifer[1]. Streams crossing the aquifer recharge zone lose a significant portion of their flow through faults and cavities in the limestone formations. During many storms, however, runoff from the headwater area exceeds the natural recharge capability of the recharge zone and travels on downstream. This study evaluates the feasibility of constructing several recharge dams on selected streams that would capture this runoff and recharge it to the Edwards Aquifer.

A series of studies sponsored by the Edwards Underground Water District (EUWD), Texas Water Development Board, Nueces River Authority, City of Corpus Christi, and South Texas Water Authority have been undertaken to evaluate the potential for enhancement of aquifer recharge, as well as determine the potential impact of such enhancement to other water users in the Nueces River Basin. These studies, which began in the Spring of 1990, are summarized in Figure ES-1. During the Phase I Study, a hydrologic model of the Nueces River Basin was developed and the findings indicated that a significant potential exists for enhancing recharge through the construction of dams on

[1]HDR Engineering, Inc., "Guadalupe-San Antonio River Basin, Recharge Enhancement Study, Phase I," Edwards Underground Water District, September, 1993.



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RECHARGE ENHANCEMENT PROGRAM DEVELOPMENT

FIGURE ES-1

the major streams crossing the aquifer recharge zone. In Phase I, no direct consideration was given to cost or environmental concerns, however, the potential impacts of the dams on water availability to the City of Corpus Christi and the Nueces Estuary were estimated. (Note: Phase II studies did not include any work on the recharge projects). The Phase III Study was completed in late 1991, and was the first attempt to optimize the size of each recharge project considering not only recharge enhancement, but also capital and annual costs, and potential environmental impacts. The Phase III Study concluded that six potential recharge projects constructed within the aquifer recharge zone could be sized to provide significant recharge enhancement at an attractive unit cost. The viability of these projects, however, was found to be subject to mitigation of relatively minor impacts to the yield of the Choke Canyon/Lake Corpus Christi (CC/LCC) Reservoir System and small changes in inflows to the Nueces Estuary. The primary objective of this Phase IVA Study, which was sponsored solely by the EUWD, was to perform more in-depth analyses of the six projects selected from Phase III and to develop an appropriate program of recharge enhancement projects in order to proceed with permitting of those projects proving to be most economical.

2.0 Recharge Enhancement Projects Considered

The six projects considered in this study were recommended for further investigation in the Phase III Study. These projects include, from east to west, the Lower Verde, Lower Hondo, Lower Sabinal, Lower Frio, Lower Dry Frio, and Indian Creek Projects. Each of these potential recharge enhancement projects is located within and near the downstream boundary of the recharge zone, as shown in Figure 2.0-1 in Section 2 of this report.

3.0 Basis for Recharge Enhancement and Project Cost Estimates

Key components of this study include site-specific evaluations of recharge characteristics, development of comprehensive flood hydrology, and a surficial assessment of the site geology and construction material availability for each of the projects. A site reconnaissance of each project was conducted by members of the project team to gather key data for the study. Although the geology at certain sites is complex, no fatal geologic or

geotechnical flaws were evident during the site reconnaissance that would prohibit development of the projects. Additionally, no fatal environmental flaws are believed to exist at the project sites.

Significant improvements were made to the Nueces River Basin model to obtain a more accurate estimate of recharge enhancement to the aquifer and potential downstream impacts resulting from construction of the projects. Revisions to the model were made to employ a daily, rather than a monthly, time step for simulating the performance of the recharge reservoirs. Site specific evaluations of recharge potential were incorporated by: 1) modelling the proposed projects at their actual locations within the recharge zone; and 2) developing a stage versus recharge rate relationship for each project. Modifications to the model were also made to improve accounting for the existing Middle Verde Creek Project and the proportioning of recharge enhancement between the Lower Frio and Lower Dry Frio Projects based on instream loss rates and upstream gage flows.

Recharge enhancement volumes were calculated subject to average and drought conditions. Average conditions are represented by the average annual recharge rate for the entire 56-year period (1934 through 1989) analyzed. Drought conditions are represented by the average annual recharge rate for the 10-year period from 1947 through 1956, which is when the most severe drought of record occurred.

Comprehensive flood hydrology modelling was performed, as the magnitude of the design flood event is a significant factor affecting the size and cost of each recharge enhancement project. Procedures included the calibration of flood hydrology model parameters using gaged data from historical flood events. These model calibrations offer increased confidence that the parameters used in the flood hydrology models accurately represent the conditions in each specific watershed. This approach has been discussed with Dam Safety staff of the Texas Natural Resource Conservation Commission (TNRCC).

Feasibility designs for the dam and spillway types considered in the Phase III Study were modified based on pertinent data gathered during the site reconnaissance and the results of the flood hydrology modelling. Unit cost data were updated to 1994 levels based on the apparent availability of construction materials and recent bids for similar type projects. Annual debt service requirements on capital costs were based on 25-year financing

at an interest rate of 7.5 percent. Annual costs were also computed for operation and maintenance of the projects, and for estimated payments to Corpus Christi to mitigate reduction in yield of the CC/LCC Reservoir System.

4.0 Project Evaluations

In order to determine the optimum size reservoir for each project, a range of storage capacities was examined. The program optimum reservoir capacity determined in the Phase III Study was used as a starting point, and sizes smaller and larger were selected to establish a range. In determining the upper range of storage capacities to evaluate, consideration was given to known site constraints that would increase project costs, such as major road relocations. Generally, three different storage capacities were evaluated for each recharge project. In some instances, this did not reveal a clear optimum and it was necessary to evaluate a fourth size. The optimum size reservoir capacity for each project was selected on the basis of the minimum unit cost of recharge enhancement under average conditions.

During the course of this study, two major findings became apparent following the individual project evaluations. These findings include:

1) The high natural capability of the Dry Frio River to recharge the Edwards Aquifer results in little water being available for recharge enhancement. Since there is minimal recharge enhancement, the unit cost is very expensive. Therefore, no further consideration was given to the Lower Dry Frio Project in the recharge enhancement program development, and it is not recommended for development at this time.

2) Uncertainties exist regarding the amount and subsequent cost of recharge enhancement at the Indian Creek Project because of questions about the natural infiltration rate and movement of ground water in the vicinity of the site. These questions prohibit the inclusion of the Indian Creek Project in the recharge enhancement program recommended for development at this time. However, the Indian Creek Project holds tremendous potential for recharge and management of the Edwards Aquifer, especially during drought, and recommendations are provided for additional studies to determine the most cost effective means of developing this significant potential water source.

On the basis of these findings, four of the six projects under consideration were determined to be ready to move forward into the permitting phase at this time. They include the Lower Verde, Lower Hondo, Lower Sabinal, and Lower Frio Projects.

5.0 Sizing of Recommended Projects

The recommended size of each project was determined by examining the unit cost of recharge enhancement under average conditions for several storage capacities at each of the four sites. The sizing procedure began by selecting the storage capacity of each project having the lowest unit cost (i.e., optimum size) and continued by enlarging the project up to the maximum storage capacity considered.

Table ES-1 illustrates this process. The Lower Sabinal Project at its optimum size represents the lowest unit cost of recharge enhancement of the four projects considered. The next most cost effective quantity of recharge enhancement is obtained by developing the Lower Verde Project at its optimum size. As shown in the upper portion of the table, each project developed at its optimum size represents the first four steps of the program development. The program development continues by evaluating the incremental cost to enlarge each project above its optimum size, up to the maximum storage capacity considered for each of the projects. This process is shown in the lower portion of Table ES-1. For example, enlarging the storage capacity of the Lower Sabinal Project by 3,500 acft above the optimum size storage capacity of 8,750 acft results in 1,099 acft of additional recharge enhancement under average conditions and no additional recharge enhancement during the drought. The incremental cost to enlarge the project and obtain the additional average recharge enhancement is $272/acft/yr.

Graphical presentations of recharge enhancement program development are shown in Figures ES-2 and ES-3. The points on the graphs correspond to either the optimum size unit cost (Nos. 1 - 4) or incremental cost (Nos. 5-10) as presented in Table ES-1. A well-defined break point occurs in the program development process at point No. 4, beyond which significantly greater investment would yield little additional recharge enhancement. This point represents the four projects developed to their optimum size, which provides the most water for each dollar spent on the program. No additional recharge enhancement is obtained for the 10-year drought period by enlarging the projects above their optimum size, as indicated by the far right column in Table ES-1 and the lower curve in Figure ES-3.

Table ES-1a
Recharge Enhancement Program Development

RECOMMENDED OPTIMUM SIZE PROJECTS

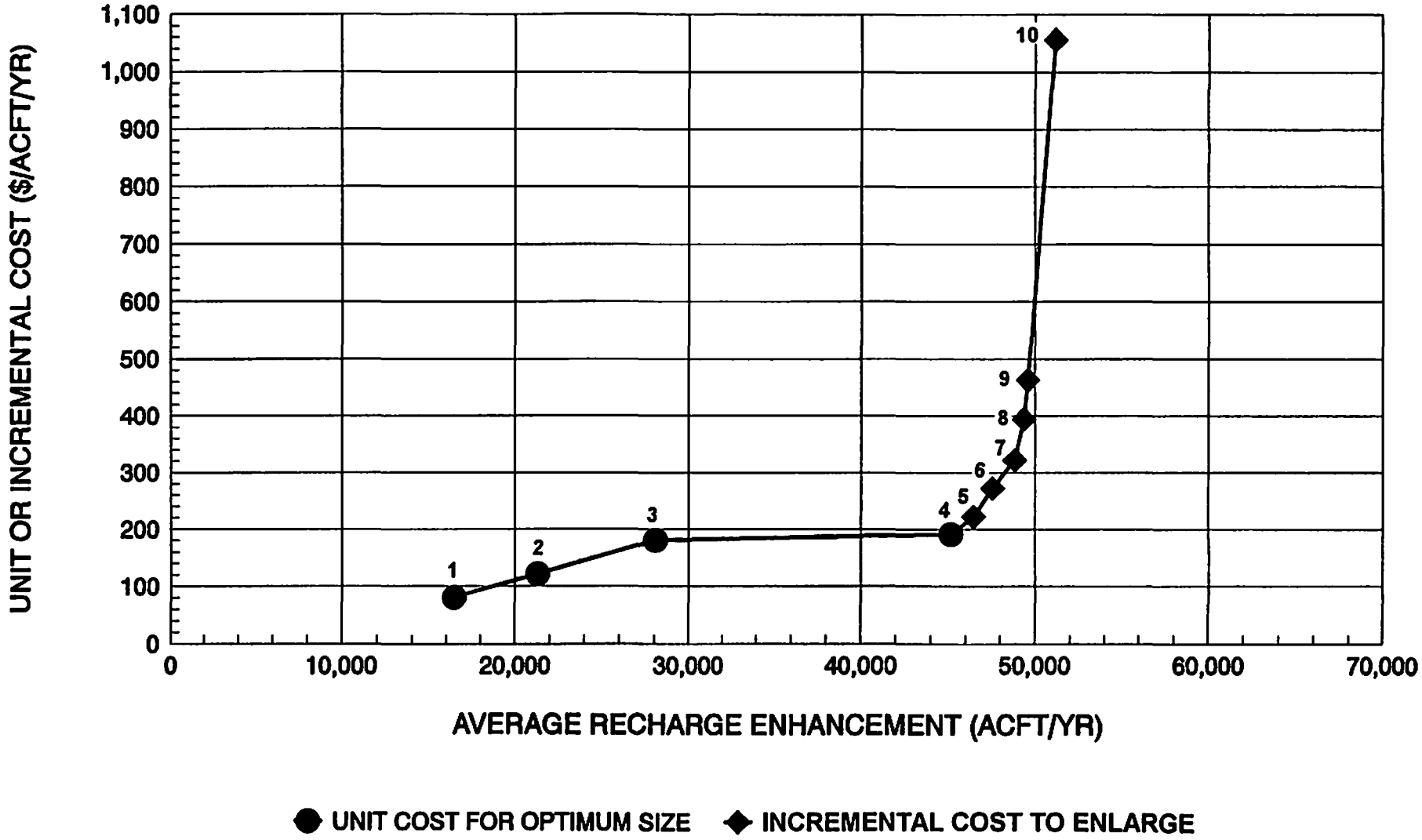
Unit Cost Ranking*	Unit Cost ($/acft/yr)	Optimum Size Projects	Optimum Storage Capacity (acft)	Annual Cost ($)	Recharge Enhancement (acft/yr)	
					Average Conditions	Drought Conditions
1	81	Lower Sabinal	8,750	1,335,379	16,442	2,358
2	122	Lower Verde	3,600	590,084	4,850	1,719
3	180	Lower Hondo	2,800	1,218,701	6,779	1,193
4	191	Lower Frio	17,500	3,258,211	17,064	3,980
		Subtotals	32,650	6,402,375	45,135	9,250

Table ES-1b
Recharge Enhancement Program Development

EVALUATION OF ENLARGED PROJECTS

Incremental Cost Ranking*	Incremental Cost to Enlarge ($/acft/yr)	Enlarged Projects	Enlarged Storage Capacity (acft)	Annual Cost ($)	Additional Recharge Enhancement (acft/yr)	
					Average Conditions	Drought Conditions
5	222	Lower Hondo	5,600	1,509,637	1,310	0
		Subtotals	35,450	6,693,311	46,445	9,250
6	272	Lower Sabinal	12,250	1,634,288	1,099	0
		Subtotals	38,950	6,992,220	47,544	9,250
7	322	Lower Sabinal	17,500	2,046,492	1,281	0
		Subtotals	44,200	7,404,424	48,825	9,250
8	394	Lower Verde	6,000	803,444	542	39
		Subtotals	46,600	7,617,784	49,367	9,289
9	463	Lower Verde	8,400	906,592	223	0
		Subtotals	49,000	7,720,932	49,590	9,289
10	1,056	Lower Frio	25,000	4,923,634	1,577	6
		Subtotals	56,500	9,386,355	51,167	9,295

*Ranking is based on unit or incremental cost of recharge enhancement for average conditions.



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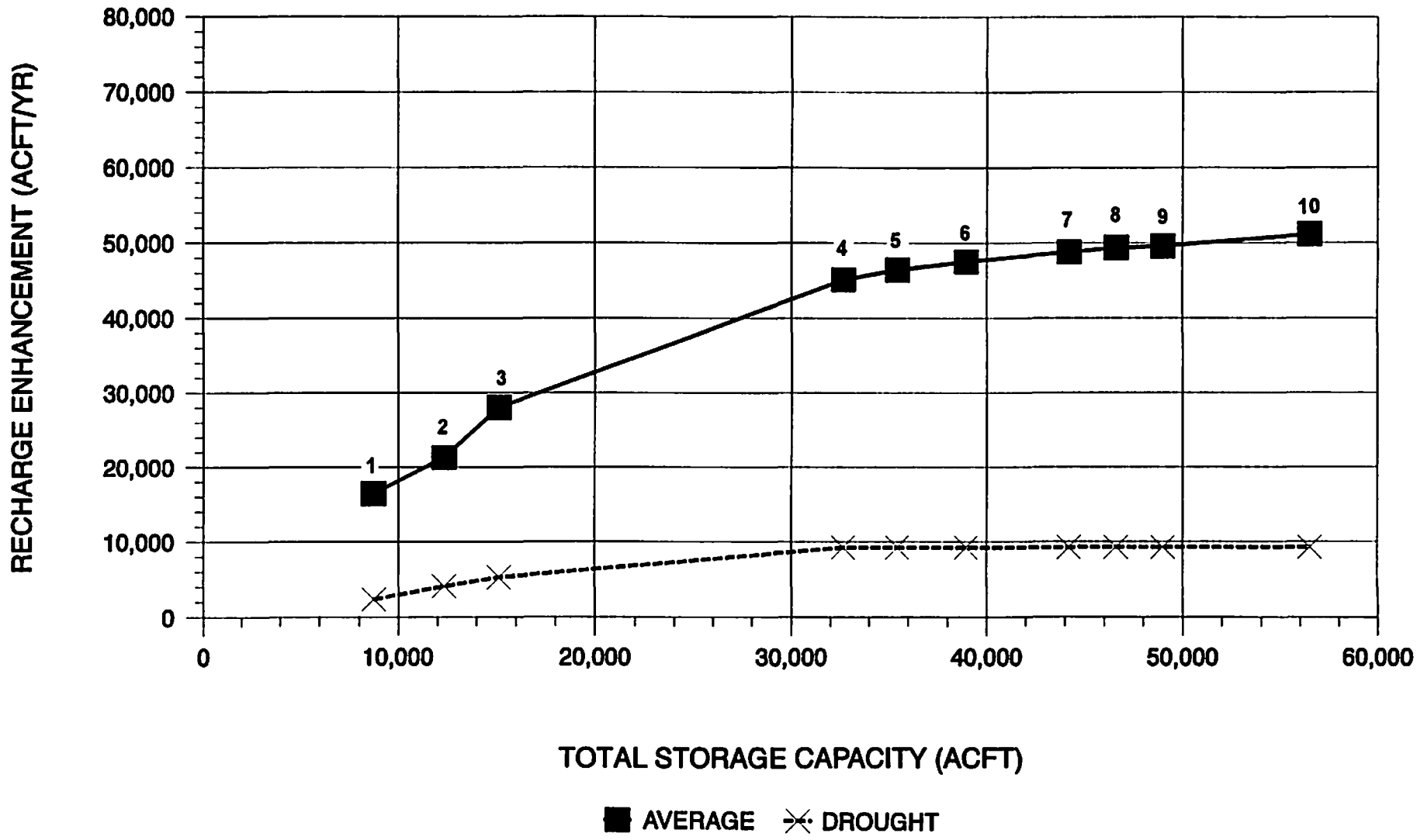
RECHARGE ENHANCEMENT PROGRAM DEVELOPMENT - COST SUMMARY

FIGURE ES-2



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RECHARGE ENHANCEMENT PROGRAM DEVELOPMENT - STORAGE SUMMARY

FIGURE ES-3

6.0 Summary of Recommended Recharge Enhancement Program

The recommended recharge enhancement program is comprised of the Lower Verde, Hondo, Sabinal, and Frio Projects, each constructed at their optimum size. A summary of this program is presented in Table ES-2. Development of the program would provide 45,135 acre-feet/year (acft/yr) of recharge enhancement under average conditions at an average unit cost of $142/acft/yr ($0.44 per 1,000 gallons). Recharge enhancement under drought conditions would be 9,250 acft/yr at an average unit cost of $692/acft/yr ($2.12 per 1,000 gallons).

A graph showing how the annual recharge to the Edwards Aquifer occurring in the Nueces River Basin would be affected by implementation of the recommended program is presented in Figure ES-4. This figure illustrates: 1) natural recharge to the Edwards Aquifer; 2) recharge enhancement resulting from development of the recommended program of the four optimum size projects; and 3) an estimate of the recharge enhancement that could be provided by the Indian Creek Project. Recharge to the Nueces River Basin portion of the aquifer would be increased by approximately 14 percent under average conditions and 6 percent under drought conditions with the implementation of the recommended recharge enhancement program. Including the estimates for the Indian Creek Project, recharge could be increased under average and drought conditions by 23 percent and 18 percent, respectively.

Impacts to the CC/LCC Reservoir System yield and reductions in inflows to the Nueces Estuary are shown for each project in Table ES-2. Reductions in the CC/LCC Reservoir System yield range from 170 to 1,229 acft/yr for each project, and represent 0.1 to 0.7 percent of the yield based on the current operating policy of the CC/LCC Reservoir System. The estimated cumulative impact on firm yield is about 2,954 acft/yr, or 1.8 percent. Reductions in the average estuarine inflow range from 728 to 2,594 acft/yr for each project, and represent 0.1 to 0.5 percent of the average annual inflow to the Nueces Estuary. The estimated cumulative impact on inflows to the Nueces Estuary is about 7,022 acft/yr, or 1.4 percent.

Table ES-2
Summary of Recommended Recharge Enhancement Program

Rank*	Project	Capacity (acft)	Surface Area (ac)	Annual Cost ($)	Average Conditions: Recharge Enhance-ment (acft/yr)	Average Conditions: Cost/Unit Recharge Enhance-ment ($/acft/yr)	Drought Conditions: Recharge Enhance-ment (acft/yr)	Drought Conditions: Cost/Unit Recharge Enhance-ment ($/acft/yr)	Reduction in Average Estuarine Inflow (acft/yr)	Reduction in CC/LCC System Yield (acft/yr)
1	Lower Sabinal	8,750	454	1,335,379	16,442	81	2,358	566	2,566	1,229
2	Lower Verde	3,600	334	590,084	4,850	122	1,719	343	728	170
3	Lower Hondo	2,800	232	1,218,701	6,779	180	1,193	1,022	1,134	403
4	Lower Frio	17,500	1,099	3,258,211	17,064	191	3,980	819	2,594	1,152
	Total	32,650	2,119	6,402,375	45,135		9,250		7,022[a]	2,954[a]
	Average					142		692		

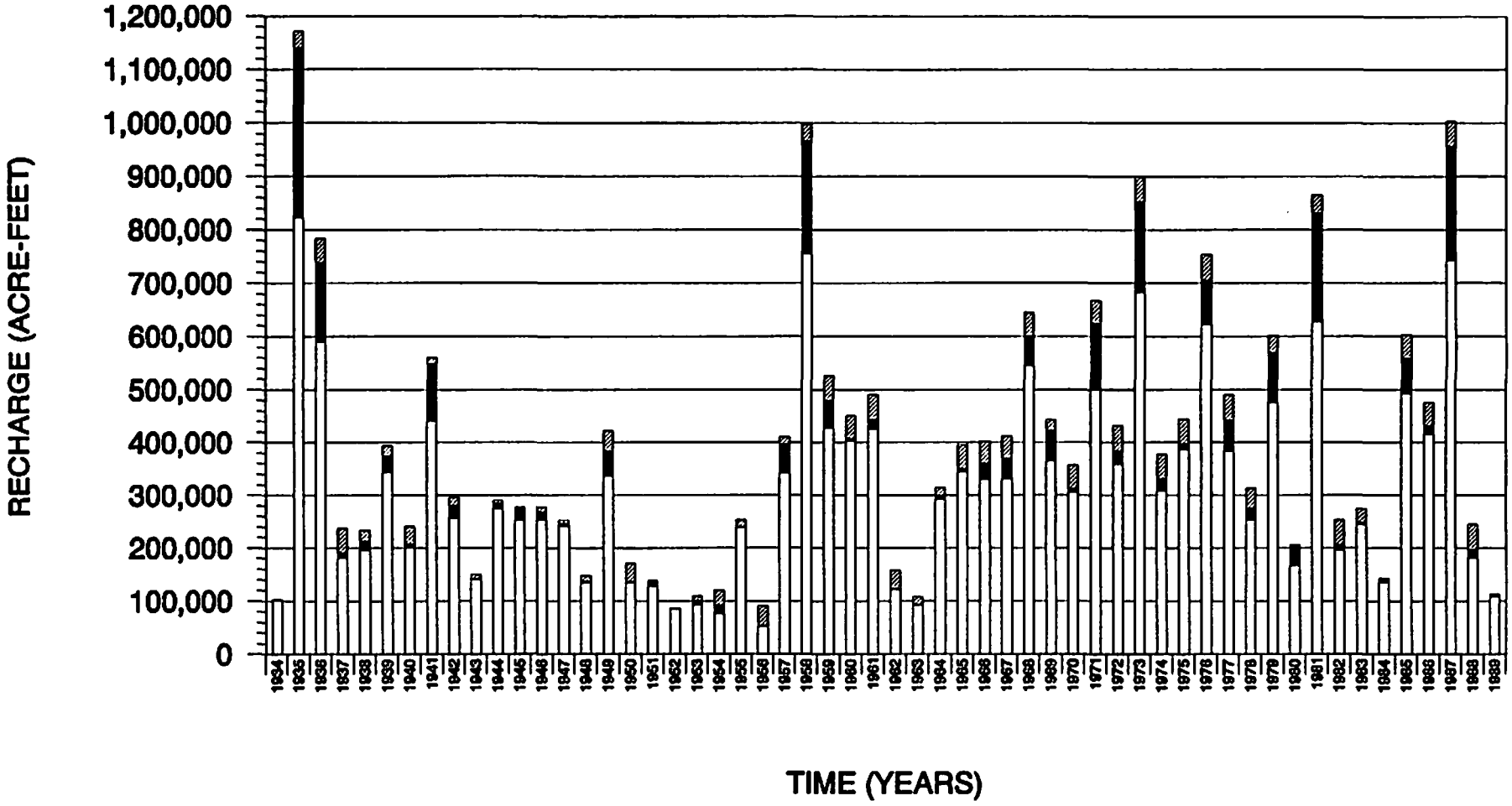
*Rank is based on Cost/Unit Recharge Enhancement for Average Conditions.
**Program includes projects with a Cost/Unit Recharge Enhancement for Average Conditions less than $192/acft/yr ($0.59/1,000 gallons).
[a]Estimated cumulative impact. Actual impact of constructing all four projects as a program could be less than that indicated.

The total capital cost of the recommended program is estimated to be $55.8 million and the total annual cost for this program would be about $6.4 million. A summary of capital and annual costs for each project is presented in Table ES-3.

7.0 Importation of Water for Recharge

The recharge reservoirs considered in this study could also provide terminal points for imported water from other existing or potential projects. The Lower Verde Creek Project, for example, is the most eastern project in the recommended program and could be a candidate location for discharging imported water. Based on the frequency of water being impounded in the proposed Lower Verde Project, sufficient reservoir capacity would exist 95 percent of the time to accept significant quantities of water.

To maximize recharge from natural flows, importation would need to be suspended during significant flood events in the Verde Creek watershed. It is not anticipated that significant additional reservoir development costs would be required to import water to the Lower Verde Project, unless very high importation rates are considered. However, if importation is considered, several additional issues may need to be investigated. These could include: 1) water quality of imported water, considering both biological and chemical compatibility; 2) environmental impacts of more permanent reservoir pools; 3) long-term changes in recharge acceptance rates; and 4) potential design changes necessitated by more permanent reservoir pools.



NATURAL RECHARGE IN NUECES RIVER BASIN (INCLUDES EXISTING RECHARGE PROJECTS)

RECHARGE ENHANCEMENT FROM RECOMMENDED PROGRAM OF FOUR OPTIMUM PROJECTS

RECHARGE ENHANCEMENT FROM INDIAN CREEK PROJECT

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ANNUAL RECHARGE TO NUECES RIVER BASIN PORTION OF EDWARDS AQUIFER

FIGURE ES-4

Table ES-3 Cost Summary of Recommended Recharge Enhancement Program					
	Lower Verde	Lower Hondo	Lower Sabinal	Lower Frio	Total
Storage Capacity (acre-feet)	3,600	2,800	8,750	17,500	32,650
Dam, Spillway, and Appurtenant Works	$2,857,013	$5,358,732	$6,724,748	$23,589,011	$38,529,504
Road Relocations	$510,000	$3,750,000	$0	$0	$4,260,000
Land Acquisition	$1,266,500	$397,000	$672,000	$1,266,100	$3,601,600
Environmental Mitigation	$23,161	$16,735	$36,172	$77,264	$153,332
Engineering, Legal, Permitting, Financial, and Misc.	$931,335	$1,904,493	$1,486,584	$4,986,475	$9,308,887
Total Capital Cost	$5,588,009	$11,426,960	$8,919,504	$29,918,850	$55,853,323
Annual Capital Cost (25 years @ 7.5% interest)	$501,244	$1,024,998	$800,080	$2,683,721	$5,010,043
Operations and Maintenance	$11,428	$21,435	$26,899	$94,356	$154,118
Water Rights Mitigation	$77,412	$172,268	$508,400	$480,134	$1,238,214
Total Annual Cost	$590,084	$1,218,701	$1,335,379	$3,258,211	$6,402,375

8.0 Major Recommendations

1) The EUWD should initiate discussions as soon as possible with the City of Corpus Christi with the intent of negotiating an agreement to mitigate the relatively small impacts on the yield of the CC/LCC Reservoir System.

2) Permission from landowners should be secured to perform various field investigations that will be required for preliminary design and permitting of the projects.

3) Preliminary Design and Permitting Phases should be initiated for the recommended program to prepare supporting documents for the permit applications that will be required. (Note: An accelerated schedule for development of the recommended recharge enhancement program is provided in Section 5.3 of this report).

4) Additional studies should be performed to determine the most beneficial and cost effective means of developing the Indian Creek Project. These should include comprehensive geologic and hydrogeologic investigations and hydrologic modelling to examine other potential project configurations.

5) Additional detailed recommendations are provided in Section 5.2 of this report.

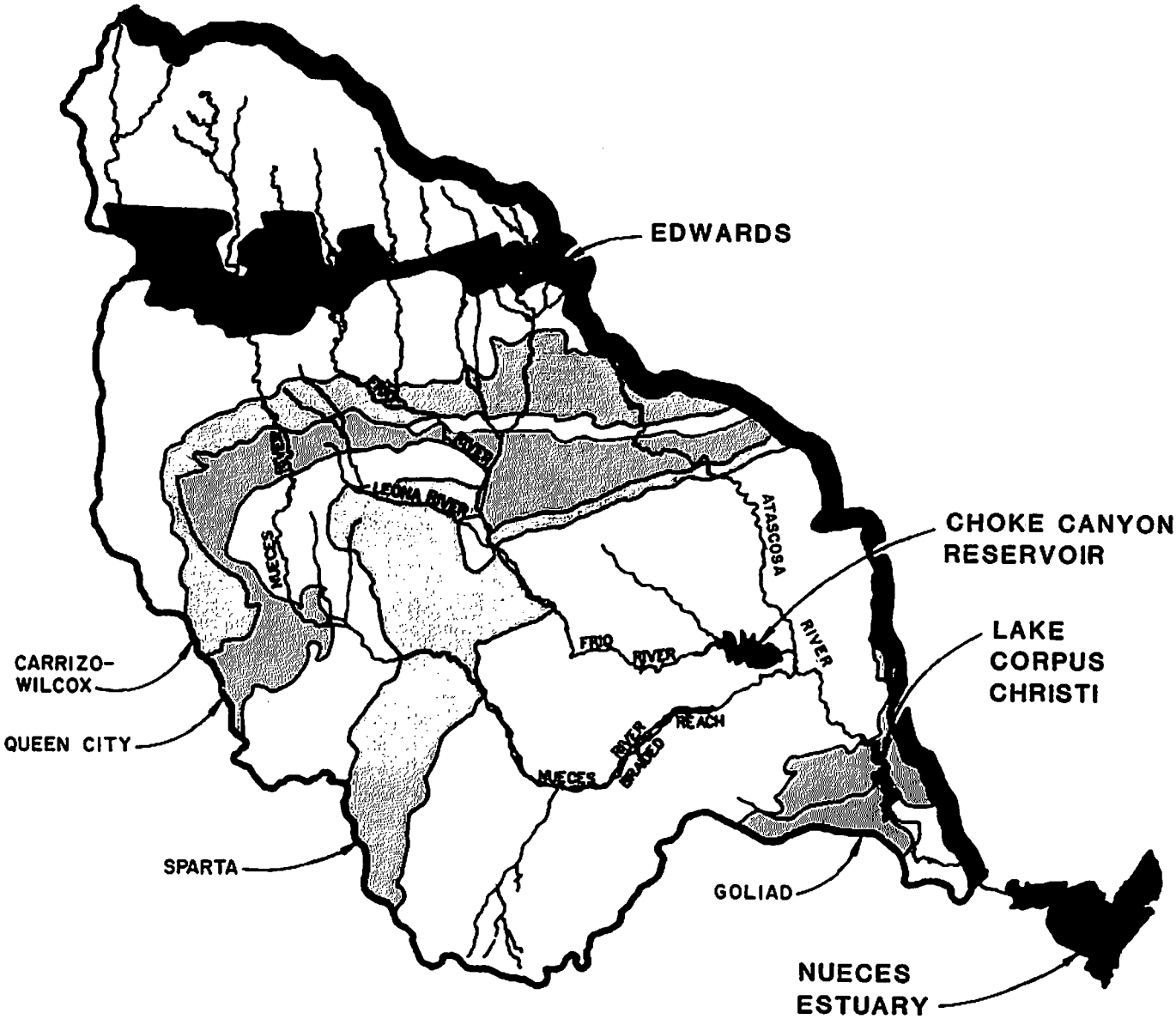
SECTION 1

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE ENHANCEMENT PROJECT
PHASE IVA

1.0 INTRODUCTION

The Nueces River Basin encompasses almost 17,000 square miles, extending from the headwaters on the Edwards Plateau north of Uvalde through the Rio Grande Plains and Gulf Coast Prairies to the outlet at Nueces Bay near Corpus Christi. As shown in Figure 1.0-1, the Nueces River Basin is crossed by five major aquifer recharge zones including the Edwards, Carrizo-Wilcox, Queen City, Sparta, and Goliad. The most transmissive of these zones is the Edwards, which lies at the base of the Balcones Escarpment in the headwaters of the Nueces and Frio Rivers. Approximately 20 percent of the Basin drainage area contributes recharge to the Edwards Aquifer. The Edwards, a limestone aquifer, is the principal source of water supply for the City of San Antonio, as well as numerous other communities and agricultural interests throughout south-central Texas. The Edwards Aquifer also supplies Leona, San Pedro, San Antonio, Comal, and San Marcos Springs, creating unique environments and recreational opportunities while providing base flow to the Leona, San Antonio, Guadalupe, and San Marcos Rivers.

A series of studies with the objectives of evaluating the potential for artificial enhancement of aquifer recharge, as well as the potential impacts of such enhancement to other interests in the Nueces River Basin, began in the Spring of 1990. The Edwards Underground Water District (EUWD), Texas Water Development Board, Nueces River Authority, City of Corpus Christi, and South Texas Water Authority were sponsors in this multi-phased Regional Water Supply Planning Study to accomplish these objectives. In the Phase I Study (Ref. 13), a hydrologic model of the basin was developed and findings indicated that the potential exists for significantly enhancing recharge to the Edwards Aquifer through the construction of dams on each of the major streams crossing the aquifer recharge zone within the basin. No direct consideration was given to cost or environmental factors in estimating recharge enhancement in the Phase I Study. Phase II was an independent study unrelated to recharge enhancement conducted by another consultant for



NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

RIVER BASIN MAP WITH AQUIFER RECHARGE ZONES

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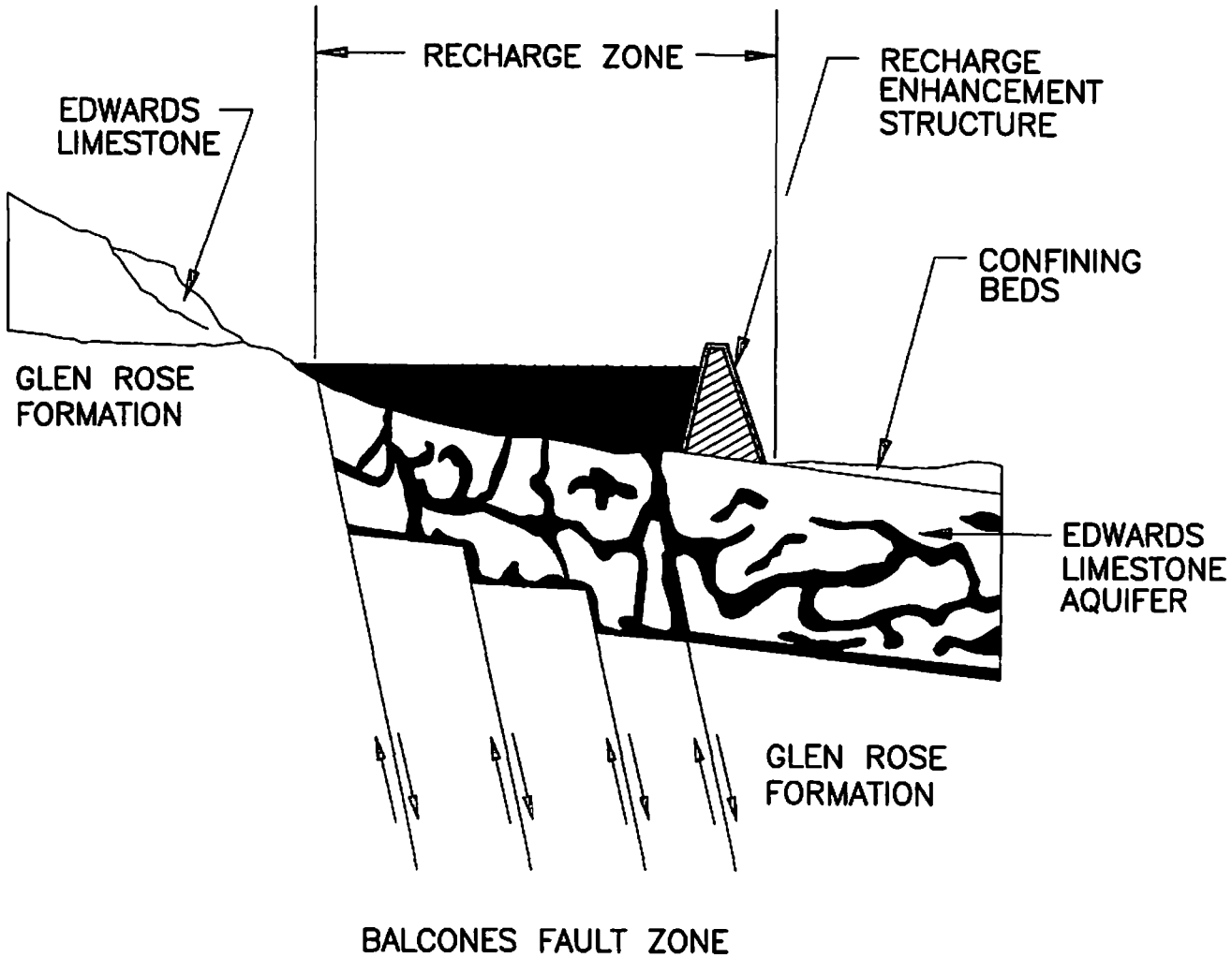
FIGURE 1.0 - 1

the City of Corpus Christi. The Phase III Study (Ref. 14), completed in late 1991, was the first attempt to optimize the size of the 19 previously identified recharge projects on the basis of recharge enhancement, capital and annual costs, and potentially significant environmental impacts. The results of the Phase III Study indicated that six direct percolation (Type 2) recharge projects (see Figure 1.0-2) constructed within the Edwards Aquifer recharge zone could provide significant enhancement of recharge at an attractive unit cost, subject to mitigation of relatively minor impacts to the yield of the Choke Canyon/Lake Corpus Christi (CC/LCC) System and inflows to the Nueces Estuary.

The objective of this study (Phase IVA), sponsored solely by the EUWD, is to develop an appropriate program of recharge enhancement projects in order to proceed with permitting by: 1) more accurately computing recharge enhancement to the Edwards Aquifer through site specific evaluations of recharge potential and revisions to the Nueces River Basin model to employ a daily, rather than a monthly, time step at five of the sites; 2) minimizing costs of project development through comprehensive flood hydrology modelling; and 3) optimizing the size of selected individual recharge projects. The following sections of this report summarize the various methodologies and site-specific considerations involved. Section 2 describes the methodologies applied to calculate recharge enhancement, develop project design floods, and determine various project costs. Section 3 presents the unique characteristics of each individual recharge enhancement project and the process involved in determining the optimum size. Section 4 describes the development of recharge enhancement programs comprised of the individual projects, and Section 5 presents the conclusions and recommendations, along with a schedule for development of the recommended recharge enhancement program.



SOURCE: USGS AND EUWD

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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SCHEMATIC OF RECHARGE ENHANCEMENT STRUCTURE

FIGURE 1.0 - 2

SECTION 2

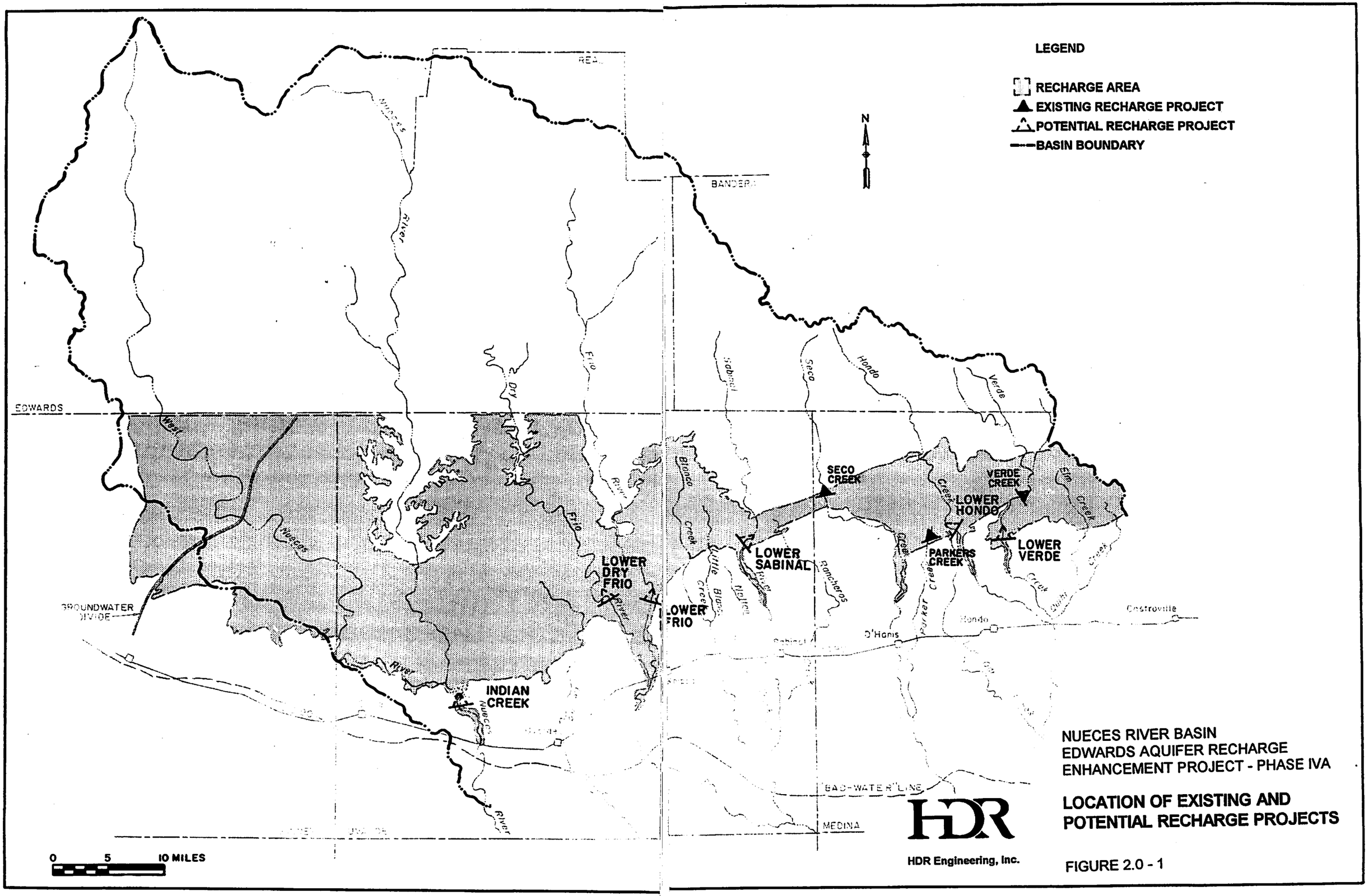
2.0 RECHARGE PROJECT EVALUATION METHODOLOGY

Key components of this study include site-specific evaluations of recharge characteristics, development of comprehensive flood hydrology, and a surficial assessment of the site geology and construction material availability for the six potential recharge projects identified in the Phase III Study. These six sites include, from east to west, the Lower Verde, Lower Hondo, Lower Sabinal (location changed since Phase III Study), Lower Frio, Lower Dry Frio, and Indian Creek Projects. The locations of these projects are shown in Figure 2.0-1, along with three existing recharge projects developed by the EUWD in the Nueces River Basin. The following subsections summarize the physical considerations and the technical methodologies applied to estimate recharge enhancement, develop flood hydrology models, and determine the related costs of dam and spillway construction, road relocations, land acquisition, water rights mitigation, environmental mitigation, permitting, and engineering.

2.1 Site Reconnaissance

A site reconnaissance was conducted in June, 1993 at each of the projects to gather key data for the study. Approximately a half day was spent at each site, except for Indian Creek at which a full day was spent. Following the June site reconnaissance, the location of the Lower Sabinal Project proposed in the Phase III Study was abandoned because of uncertainties related to recharge being able to reach the Edwards Aquifer (Ref. 19). An alternate site was selected upstream within the recharge zone based on an assessment of the topography and fault locations. A full day site reconnaissance was conducted at the alternate site in August, 1993.

Participants in the site reconnaissance included various HDR staff, Robert Bader (EUWD), Greg Rothe (Project Coordinator for the EUWD), and subconsultants to HDR, including Freese & Nichols (F&N), Fugro-McClelland (Southwest), Inc. (F-M), LBG-Guyton Associates (LBG), International Aerial Mapping (IAM), and Paul Price Associates (PPA). The primary role of each project team member, both during the site reconnaissance and throughout the study, is identified as follows:



NUECES RIVER BASIN
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LOCATION OF EXISTING AND POTENTIAL RECHARGE PROJECTS

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FIGURE 2.0 - 1

Team Member	Role
HDR Engineering, Inc.	Hydrology and Dam Design
Freese & Nichols, Inc.	Flood Hydrology and Dam Design (primarily for Indian Creek Project)
Fugro-McClelland (SW), Inc.	Site Geology and Geotechnical Engineering
LBG-Guyton Associates	Geohydrology
International Aerial Mapping	Surveying
Paul Price Associates	Environmental Assessment

HDR and F&N's primary objectives during the site reconnaissance were to gather information concerning the dam site and upstream watershed for each project. Working in conjunction with F-M, HDR and F&N reviewed and made adjustments (as necessary) to the Phase III dam and spillway alignments, assisted with the development of geotechnical considerations for design, and scouted potential sources of locally available construction materials at each project. Additionally, HDR and F&N staff examined the upstream watershed characteristics to facilitate development of parameters for flood hydrology modelling.

The primary objectives of F-M during the site visits were to conduct a geologic "fatal flaw" assessment for construction of a major dam and spillway, develop a geologic and geotechnical field investigation program for preliminary design of the project, develop geotechnical considerations for project design, assist with review of the Phase III dam and spillway alignments, and delineate locally available construction materials. Although the geology at certain sites is complex, no fatal geologic or geotechnical flaws were evident during the site reconnaissance that would prohibit development of the proposed recharge projects (Ref. 6).

During the site reconnaissance, LBG staff examined the streambed and reservoir areas of each project. The objectives of this work were to develop: 1) an understanding of geohydrologic conditions which affect and control ground water movement at each site; 2) a basis for comparative evaluation of sites with respect to potential for recharge; and 3) a ranking of the sites in terms of their relative recharge potential (Ref. 19). LBG developed

a numerical rating system, called the Hydrogeologic Setting Index (HSI), to compare the relative recharge potential of each site. The HSI is used as a composite description of eight key geologic and hydrogeologic factors which are believed to affect and control recharge to the Edwards Aquifer. A matrix of these factors and the computed HSI for each project is provided in Appendix A. The resulting HSI values indicate that the potential for recharge at each of the projects is essentially equal, with the exception of the Indian Creek Project. The potential for recharge at Indian Creek was downgraded because of lower measured stream losses within the recharge zone, a relatively high water table with the possibility of hydraulic connection between the Edwards Aquifer and the overlying Leona (Gravel) Formation, and a different geologic setting (Maverick basin versus the Devils River trend) as compared to the other sites.

IAM participated in the site reconnaissance to survey the proposed dam centerline and gain familiarity with property restrictions, access locations, and the physical conditions at each site. Following the site visits, IAM performed ground surveying to obtain a dam centerline profile for each site. This information was later used to more accurately compute dam and spillway construction quantities.

PPA participated in the site reconnaissance to identify any "fatal" (or very expensive) environmental concerns and provide confirmation of the vegetation and habitat mapping that was conducted for the Phase III Study. Environmental concerns that may constitute a fatal flaw and prohibit development of the proposed recharge projects were not evident during the site visits.

Research on site specific information concerning the presence, or potential presence, of threatened and endangered species was also conducted by PPA. Table 2.1-1 lists protected species and resources of concern that have been reported in the literature in the vicinity of the project sites, generally from the same, or an adjacent, U.S. Geological Survey (USGS) 7.5 minute topographic map. Specific potential environmental impacts and mitigation requirements are discussed in a report prepared by PPA which was included in Appendix A of the Phase III Study report (Ref. 14).

Table 2.1-1 Environmental Evaluation			
Common Name	**Federal Status**	**State Status**	**Project Site**
Black-capped vireo	Endangered	Endangered	Lower Frio Lower Dry Frio Lower Sabinal[1] Indian Creek[1]
Tobusch fishhook cactus	Endangered	Endangered	Lower Dry Frio
Guadalupe bass	Candidate	None	Indian Creek
Comal snakewood	Candidate	None	Indian Creek Lower Dry Frio
Sabinal prairie-clover	Candidate	None	Indian Creek
Texas grease bush	Candidate	None	Indian Creek

[1]Potential vireo nesting habitat observed during site reconnaissance, but not reported in the literature.

No endangered or candidate species listed in Table 2.1-1 were observed during the site visits. Habitat areas that would be potentially useful for black-capped vireo nesting were not observed at the Lower Frio site; however, potential vireo nesting habitat was observed at the Lower Dry Frio, Indian Creek, and Lower Sabinal sites. Spring site surveys will be required to determine if the habitat is being used by the vireos, and an evaluation will need to be made of how occasional inundation would affect use of the habitat. No fishhook cacti were observed at the Lower Dry Frio site, but additional botanical survey work would be required if this site were to be developed. All sites exhibited evidence of Native American habitation, which will necessitate performing archaeological surveys prior to development.

2.2 Recharge Enhancement Hydrology

2.2.1 Nueces River Basin Models

The original computer model of the Nueces River Basin and the associated input databases were developed as parts of the Phase I Study (Ref. 13) completed in early 1991 and sponsored by the Edwards Underground Water District (EUWD), Nueces River Authority (NRA), City of Corpus Christi, South Texas Water Authority (STWA), and Texas Water Development Board (TWDB). The Phase I model was created specifically to evaluate recharge enhancement associated with potential projects and assess potential impacts on water availability downstream. It employs a monthly time step proceeding with flow calculations in an upstream to downstream order simulating recharge, channel losses, water rights, and reservoir operations at 30 control points for a 56-year (1934-89) period of record. The original basin model was capable of simulating the operations of the Choke Canyon / Lake Corpus Christi (CC/LCC) System subject to a maximum firm yield policy with very limited consideration of release programs for maintenance of estuarine inflows.

As a portion of an independent series of studies (Refs. 9, 11, and 12) with sponsors including the City of Corpus Christi, NRA, STWA, Corpus Christi Board of Trade, Port of Corpus Christi, and TWDB, the Lower Nueces River Basin Model was developed with a focus on the CC/LCC System and the Nueces Estuary. As a result of these studies, the Lower Nueces River Basin Model is presently capable of simulating CC/LCC System operations under a variety of operation policies, alternative bay and estuary release programs, and effluent diversion schemes, and capable of computing firm yield and estuarine inflow statistics.

In the performance of the Phase III Study (Ref. 14), the Nueces River Basin Model and an early version of the Lower Nueces River Basin Model were used in tandem to determine recharge enhancement under average and drought conditions, reductions in CC/LCC System firm yield, and reductions in average estuarine inflow associated with the implementation of each of 19 potential projects. Although the model versions used in the Phase III studies were adequate for comparison of the relative merits of potential projects over a range of recharge pool capacities, the accuracy of recharge enhancement and downstream impact estimates was limited by the following assumptions:

1) Projects were located at the nearest model control point and/or streamflow gage site rather than at their actual locations.

2) Project inflow and storage were evaluated on a monthly timestep.

3) Streamflows impounded in direct percolation (Type 2) projects, with the exception of the Indian Creek Project, were assumed to recharge within one month.

4) Net evaporation from Type 2 recharge reservoirs was neglected.

5) Outlet conduits at recharge enhancement projects were sized to be capable of passing any amount of water theoretically required.

6) Impacts of recharge enhancement on the CC/LCC System were based on a maximum firm yield policy with no specified monthly estuarine inflow requirements.

Accuracy of recharge enhancement and downstream impact estimates has improved in the current study as a result of the synthesis of new methodologies and incorporation of the following modifications to the river basin models:

1) Projects are located at their actual sites.

2) A daily computational timestep is employed to more accurately simulate recharge at and below the proposed projects. Using a daily timestep, the simultaneous occurrence of inflow and recharge at the proposed projects can be simulated, and the incremental recharge accounted for. In the previous version of the basin model, any monthly inflow in excess of the recharge pool volume would have been spilled without having an opportunity to contribute to recharge. The Indian Creek Project, however, is still simulated on a monthly timestep due to its large storage and slow recharge rate compared to other potential projects.

3) Measured channel loss rates across the Edwards Aquifer recharge zone (Ref. 35) are used in the computation of natural and enhanced recharge. Incorporation of these loss rates significantly improved estimates of recharge enhancement for the Lower Frio and Lower Dry Frio Projects.

4) Recharge rate curves based on measured channel loss rates and soil permeability characteristics (Refs. 26 and 27), and calibrated to observations at Parkers Creek and Middle Verde are used to evaluate daily recharge as a function of average storage.

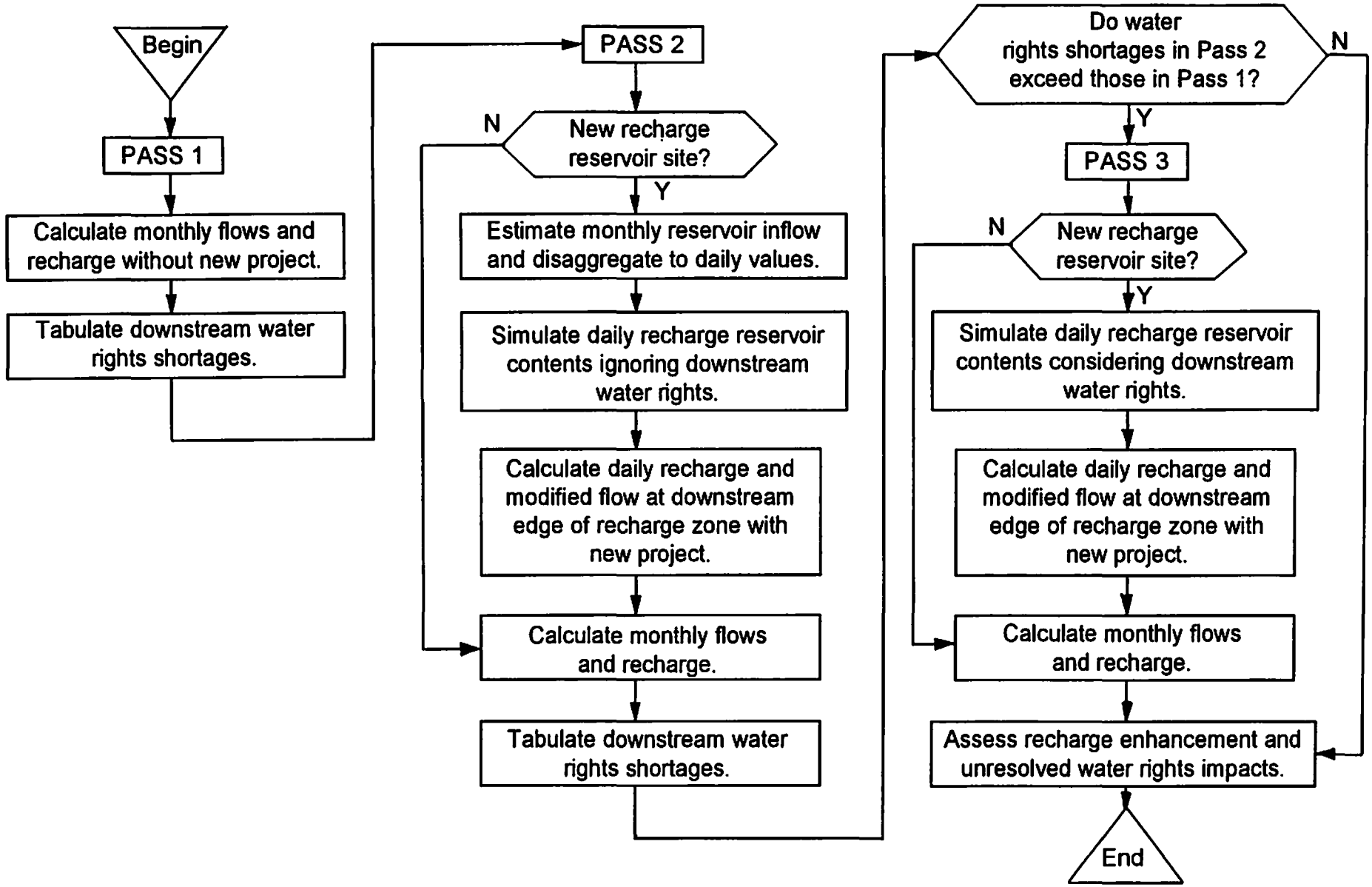
5) Daily net evaporation from each recharge reservoir is computed as a function of daily average storage.

6) Passage of water for mitigation of impacts to downstream water rights is based on proposed outlet sizes and daily average storage.

7) Impacts of recharge enhancement on the CC/LCC System are based on the current operating policy adopted by the City of Corpus Christi and the March 9, 1992 Interim Order of the Texas Natural Resource Conservation Commission (TNRCC) governing release requirements for Nueces Bay.

The derivation and application of these methodologies and model modifications is described in greater detail in following subsections.

Computation of daily recharge at each of the proposed projects while minimizing adverse impacts on downstream water availability is accomplished in the Nueces River Basin Model using the three-pass process presented in Figure 2.2-1. In the first pass, recharge without the new project is computed, monthly flows are simulated at all control points, and any shortages or failures to satisfy downstream diversion rights are tabulated. In the second pass, the new project is included and any downstream shortages are tabulated assuming full impoundment and/or diversion of inflows considering recharge and evaporation on a daily timestep at the new project. If shortages in the second pass exceed those in the first pass, the monthly flow volume required to eliminate the additional shortages is computed for the next control point below the new project. In the third and final pass, recharge, evaporation, and water rights releases at the new project are computed on a daily basis and modified monthly flows are simulated at all control points. Modified flows for the Nueces River near Tilden and modified inflows to Choke Canyon Reservoir are then used as input to the Lower Nueces River Basin Model for computation of any remaining impacts to the CC/LCC System firm yield and inflows to the Nueces Estuary.



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RIVER BASIN MODEL FLOWCHART

FIGURE 2.2- 1

2.2.2 Recharge Enhancement Computation Methodology

An improved methodology employing a daily computational timestep for the estimation of monthly Edwards Aquifer recharge enhancement associated with proposed projects was developed for this study. The daily timestep was applied in the simulation of both recharge reservoir contents and delivery of spills and releases to the next downstream control point located near the downstream edge of the recharge zone. The procedure applied for recharge enhancement computation using the Nueces River Basin Model is outlined in the following paragraphs. A typical gaged watershed, including a proposed project is shown in Figure 2.2-2.

Recharge enhancement is defined as the difference between recharge with and without a new project. Hence, the first step in the computation of enhanced recharge is the estimation of baseline monthly recharge without the proposed project. As described in previous reports (Refs. 10 and 13), monthly recharge in a typical gaged watershed traversing the recharge zone may be estimated using the following equation:

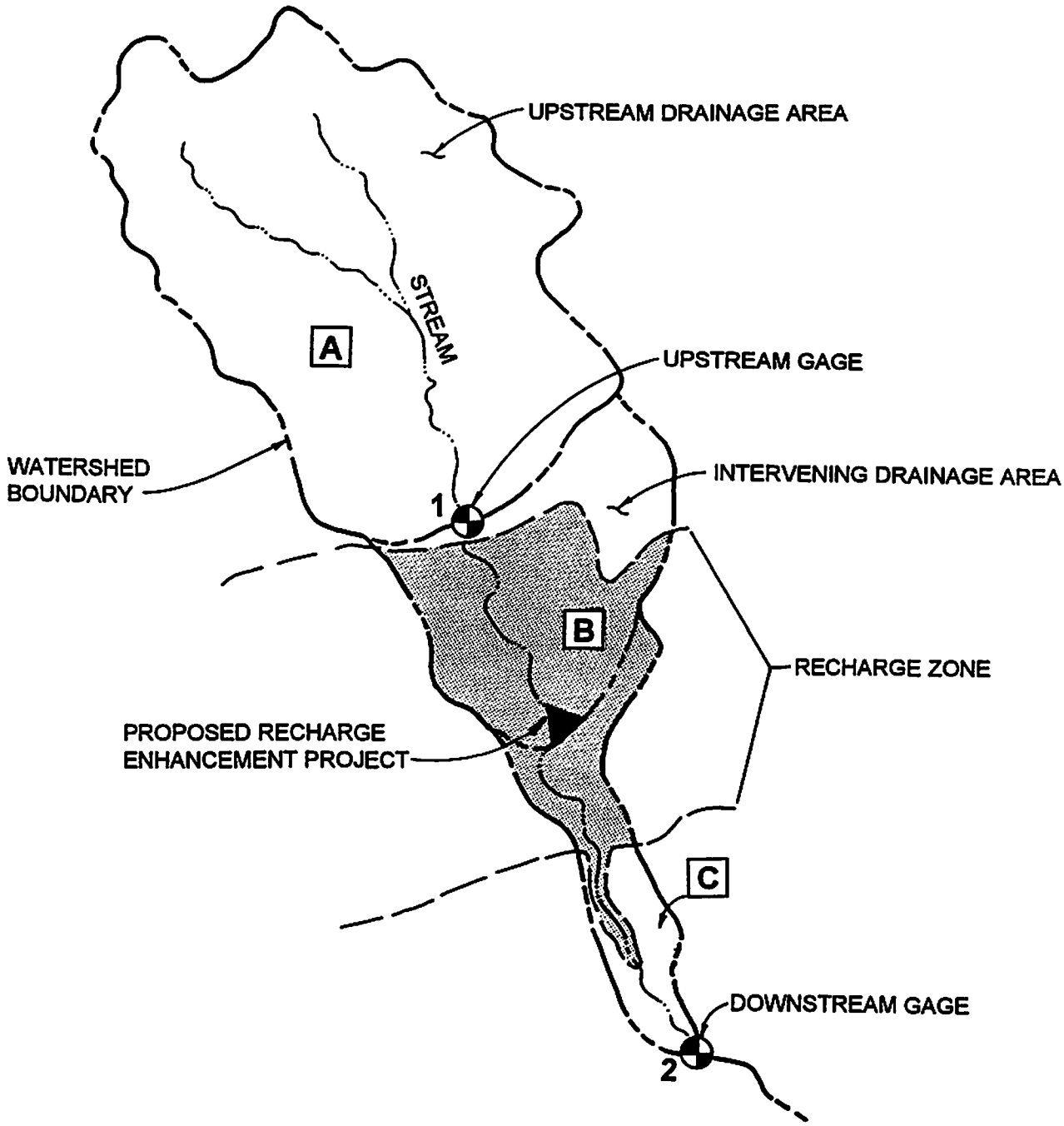
$$R_O = Q_1 + QI - Q_2$$

where:

R_0 = Recharge without project;
Q_1 = Flow at upstream control point;
QI = Potential intervening runoff; and
Q_2 = Flow at downstream control point.

Flows at the upstream and downstream control points reflect adjustments for the monthly diversion of existing water rights. With knowledge of the baseline recharge as well as the portions of the intervening area and the typical instream loss rates both above and below the project shown in Figure 2.2-3, monthly inflow to the project was estimated using the following equation:

$$QD = Q_2 - QI\left(\frac{A_C}{(A_B + A_C)}\right) + R_O\left(\frac{L_C}{(L_B + L_C)}\right)$$



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TYPICAL GAGED WATERSHED WITH RECHARGE ENHANCEMENT PROJECT

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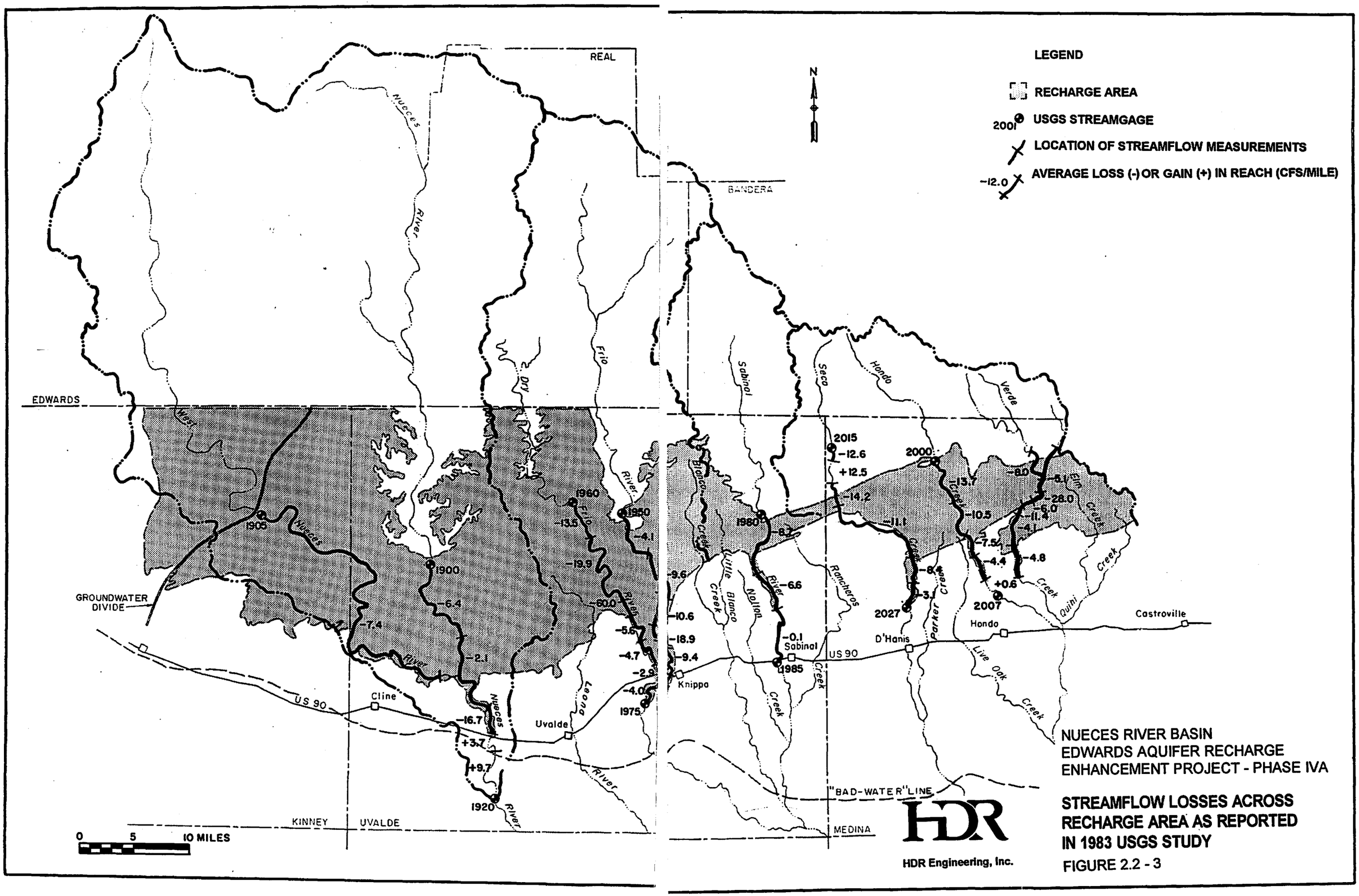
FIGURE 2.2 - 2



NUECES RIVER BASIN
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STREAMFLOW LOSSES ACROSS RECHARGE AREA AS REPORTED IN 1983 USGS STUDY

FIGURE 2.2 - 3

where:

QD = Monthly project inflow;
A_C = Intervening area downstream of project;
A_B = Intervening area upstream of project;
L_C = Loss rate for reach downstream of project; and
L_B = Loss rate for reach upstream of project.

As is apparent in this equation, potential runoff is prorated above and below the project based on subwatershed area, while baseline recharge is prorated based on measured instream loss rates.

Monthly estimates of project inflow were disaggregated to daily values using gaged streamflow records when available upstream or downstream of the project. If daily streamflow records were not available upstream or downstream of the project, then monthly estimates of project inflow were disaggregated using available daily records from an adjacent watershed. Procedures applied are listed in order of preference as follows:

1) Daily project inflows based on the daily percentage of gaged monthly streamflow as recorded at the next downstream control point identified with the number 2 in Figure 2.2-2.

2) Daily project inflows based on the daily percentage of the sum of gaged daily streamflows as recorded at the next upstream control point identified with the number 1 in Figure 2.2-2, which are in excess of the loss rate for the reach upstream of the project.

3) Obtain an estimate of daily streamflow at the next downstream or upstream control point based on the daily percentage of gaged monthly streamflow in the nearest adjacent watershed, and estimate daily project inflows using procedure 1 or 2, respectively.

Importation of water to a recharge reservoir can be considered simply by adding imported flows to the daily inflows originating in the local watershed.

Using the daily project inflow estimates, recharge reservoir contents are simulated in accordance with the methodology detailed in Section 2.2.3. Daily recharge through direct percolation is based on project-specific relationships between daily recharge rate and average reservoir storage (expressed as surface area) presented in Section 2.2.5. Diversions from the reservoir for recharge, such as those from the Indian Creek Project, are user-specified.

Total monthly recharge with the project in place is computed using the following equation:

$$R = \left[Q_1 + QI\left(\frac{A_B}{(A_B + A_C)}\right) - QD\right] + \sum RD_t + \sum D_t + \sum RC_t$$

where:

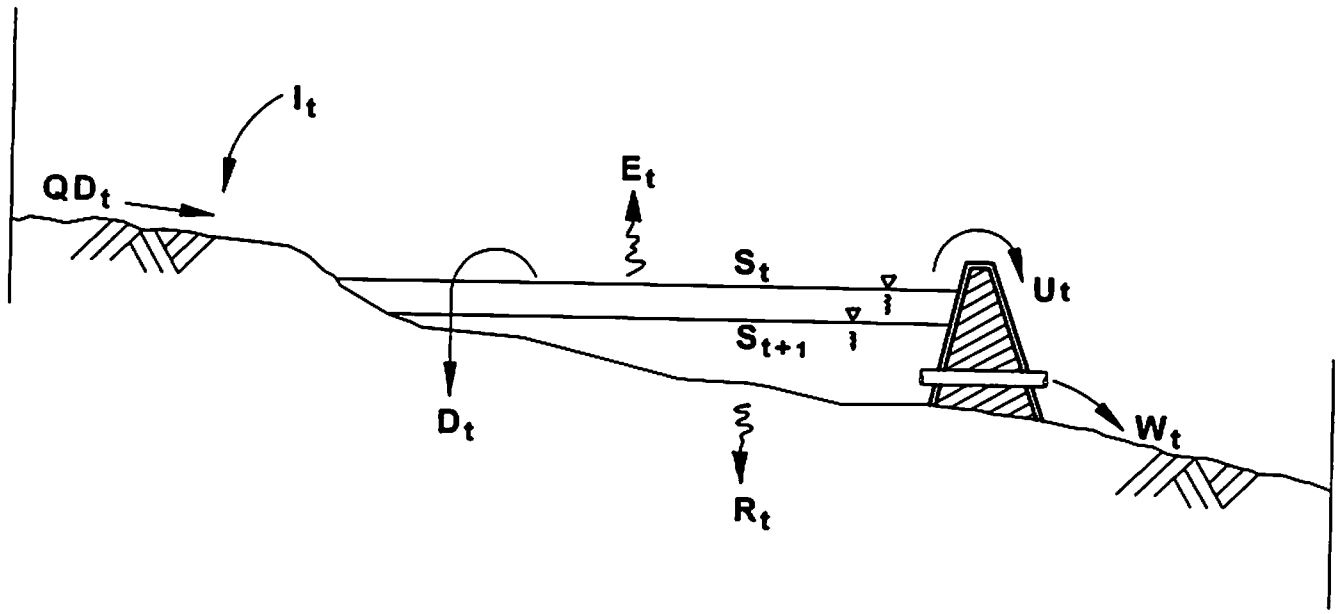
R = Monthly recharge with project;
ΣRD_t = Sum of daily recharge estimates of direct percolation from project;
ΣD_t = Sum of daily recharge estimates of diversion from project; and
ΣRC_t = Sum of daily recharge estimates downstream of project.

Note that the first term in this equation is essentially the natural monthly recharge occurring upstream of the project while the remaining terms are affected either directly (ΣRD_t, ΣD_t) or indirectly (ΣRC_t) by reservoir storage.

The recharge computation methodology and its incorporation in the Nueces River Basin Model was verified in part by performance of simulations assuming zero project storage capacity in which case ΣRD_t and ΣD_t became zero and recharge (R) was essentially equal to recharge without the project (R_0). Further verification of all model simulation capabilities was accomplished through extensive manual checking of intermediate computations and final output summaries.

2.2.3 Recharge Reservoir Operations

Simulation of recharge reservoir operations in the Nueces River Basin Model is governed by the integral equation of continuity (Ref. 4) as expressed in Figure 2.2-4 in which the various volume fluxes affecting storage are identified. A simultaneous solution for these fluxes is necessary to obtain an accurate estimate of end-of-day storage as recharge, net evaporation, and water rights releases are dependent upon the water surface area or elevation associated with the average storage ($\overline{S}$) for a given day, . This solution is obtained in the basin model using the Half-Interval Method (Ref. 3) the application of which to reservoir contents simulation is described in detail in Reference 10. Simulated end-of-day water surface elevations for each recharge enhancement project were tabulated



$$S_{t+1} = S_t + QD_t + I_t - E_t - R_t - D_t - W_t - U_t$$

WHERE:

S_{t+1} = END-OF-DAY STORAGE
S_t = BEGINNING-OF-DAY STORAGE

$$\bar{S} = \frac{S_t + S_{t+1}}{2}$$

QD_t = RESERVOIR INFLOW ⟶ COMPUTED
I_t = IMPORTED INFLOW ⟶ USER SPECIFIED
E_t = NET EVAPORATION ⟶ $f(\bar{S}$, TWDB DATA)
R_t = DIRECT RECHARGE ⟶ $f(\bar{S}$, RECHARGE CURVE)
D_t = DIVERSION FOR RECHARGE ⟶ USER SPECIFIED
W_t = WATER RIGHTS RELEASE ⟶ $f(\bar{S}$, OUTLET CHARACTERISTICS)
U_t = SPILLS ⟶ COMPUTED

* THE HALF-INTERVAL METHOD WAS USED TO SOLVE FOR S_{t+1} USING E_t, R_t, AND W_t BASED ON $\bar{S}$.

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RECHARGE RESERVOIR CONTENTS SIMULATION

FIGURE 2.2 - 4

and used to develop graphical relationships illustrating the percentage of time that reservoir storage was less than various percentages of recharge pool capacity.

Monthly net evaporation rates used in this study for the 1940-89 period were calculated from TWDB quadrangle data (Ref. 29) using a standard inverse distance ratio procedure to convert values typical of the centroids of adjacent quadrangles to values representative of a specific reservoir site. Net evaporation rates for the 1934-39 period were computed from available pan evaporation records (Ref. 28) adjusted by pan coefficients recommended by the TWDB (Ref. 29) and by coincident measured precipitation. Daily estimates of net evaporation were obtained by dividing the monthly rate by the number of days in the month, and multiplying by the surface area associated with average daily storage.

The relationship between water surface elevation, surface area, and storage capacity (E-A-C) was established for each project using a polar planimeter to measure surface area from successive elevation contours on available USGS 7.5-minute topographic maps. Storage volume calculations were generally performed using the average end area method.

2.2.4 Water Rights Considerations

The most significant water rights consideration in this study is the general assumption that impacts to downstream diversion and storage rights in the CC/LCC System due to the operations of recharge enhancement projects can be mitigated through some form of compensation other than passage of impounded flows. It is important to note that these impacts cannot be completely avoided solely by the release of water because even temporary impoundment of flood flows would result in percolation rates in excess of those which would have occurred naturally. Therefore, some form of financial mitigation of impacts to the CC/LCC System will likely be a component of the annual cost of each recharge enhancement project.

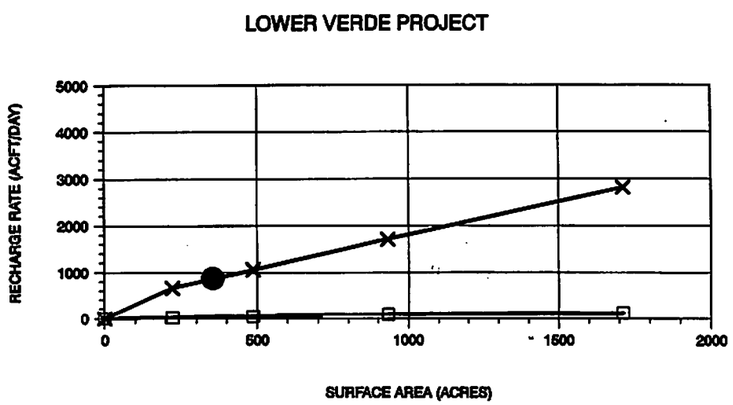
All simulations of CC/LCC System operations in this study were based on Phase II of the City of Corpus Christi reservoir system operation plan and reflect monthly Nueces Bay freshwater inflow requirements and operational constraints summarized in the TNRCC Interim Order of March 9, 1992. The impact of each potential recharge enhancement project on the firm yield of the CC/LCC System and on average estuarine inflows during

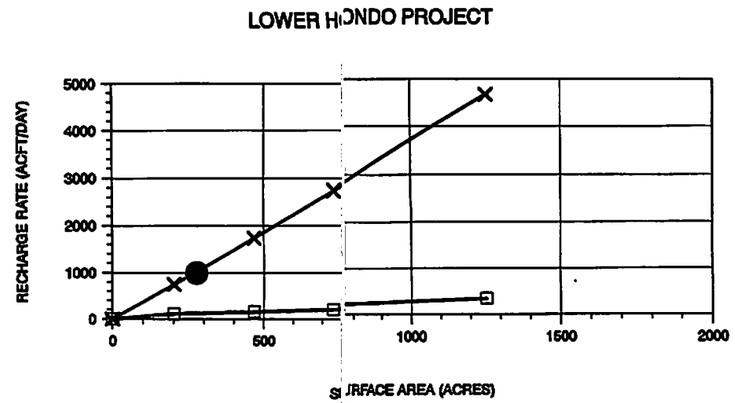
the 1934-89 period were evaluated using the Lower Nueces River Basin Model. The impact on firm yield represents a reduction in the dependable water supply of the City of Corpus Christi while the impact on average estuarine inflows is representative of less tangible, long-term economic and environmental effects associated with minor reductions in lake levels and/or fishery harvests.

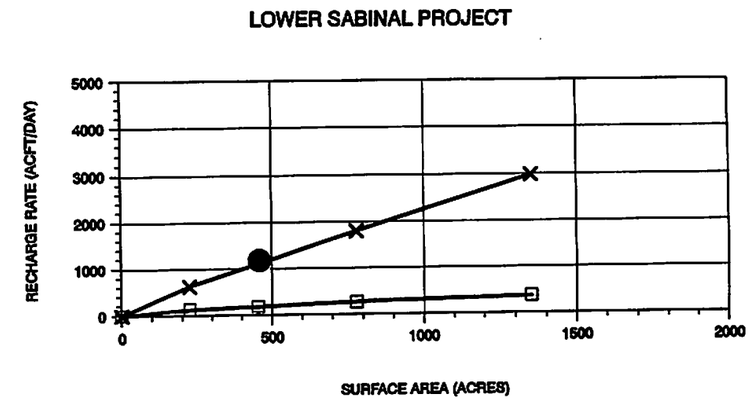
The outlet conduit at each recharge enhancement project was sized to pass sufficient flow to traverse the remainder of the recharge zone, suffer downstream channel losses, and deliver peak monthly demand under water rights on the mainstem in seven days with an average of 10 feet of head on the conduit. Selected conduit sizes in this study ranged from a minimum of 48 inches in diameter at several of the smaller projects up to 108 inches in diameter at the Indian Creek Project. The Nueces River Basin Model attempts to satisfy all of these run-of-the-river diversion rights to the extent they would have been satisfied without the proposed recharge enhancement project. In each month when additional shortages occur, a desired monthly flow volume is established for the next control point downstream of the project and daily releases dependent on reservoir stage and conduit size continue until the desired volume has been delivered, the reservoir drains completely, or the end of the month arrives. Temporal water availability to some downstream rights may well be enhanced by extended releases from recharge projects at controlled rates as opposed to the relatively swift passage of uncontrolled flood flows.

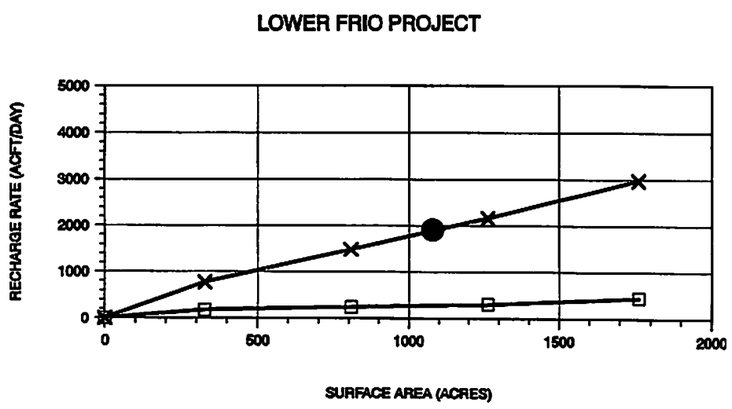
2.2.5 Recharge Rate Curves

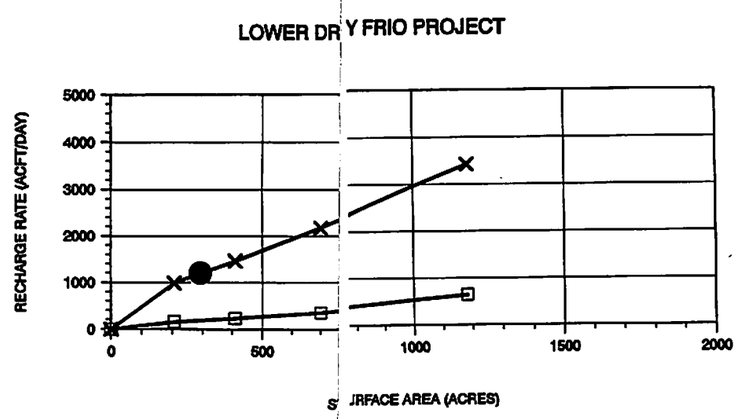
Unique recharge rate curves based on site-specific geologic characteristics were developed for each of the Lower Verde, Hondo, Sabinal, Frio, and Dry Frio Projects. These curves relate an estimated direct percolation rate to the surface area associated with average daily storage in each recharge reservoir and are presented in Figure 2.2-5. The recharge rate curves provide the basis for computation of the daily recharge flux which generally comprises the greatest portion of the water leaving the reservoir. The methodologies applied in the development and verification of these curves are described in the following paragraphs.











LEGEND

- CHANNEL RECHARGE
- TOTAL RECHARGE
- RECOMMENDED OPTIMUM SIZE

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HDR

HDR Engineering, Inc.

RECHARGE RATE CURVES FOR PROPOSED PROJECTS

FIGURE 2.2 - 5

The recharge rate curves are based on the sum of two assumed components of recharge. These include that occurring in the main channel and that occurring in the overbank areas. As is apparent in Figure 2.2-5, the overbank component dominates the estimated total daily recharge rate. The overbank recharge component for each project was derived from soil mapping and permeability rates published by the Soil Conservation Service (Refs. 26 and 27). Weighted average permeability rates for the range of reservoir capacities considered at each project were calculated based on the average of the high and low published permeabilities and on the areal concentration of mapped soil types.

The main channel component of the daily recharge rate was based on the assumption that the hydraulic characteristics of the fissures and solution cavities in the bed of the channel could be approximated by an orifice equation of the theoretical form:

$$Q = A\sqrt{2gH}$$

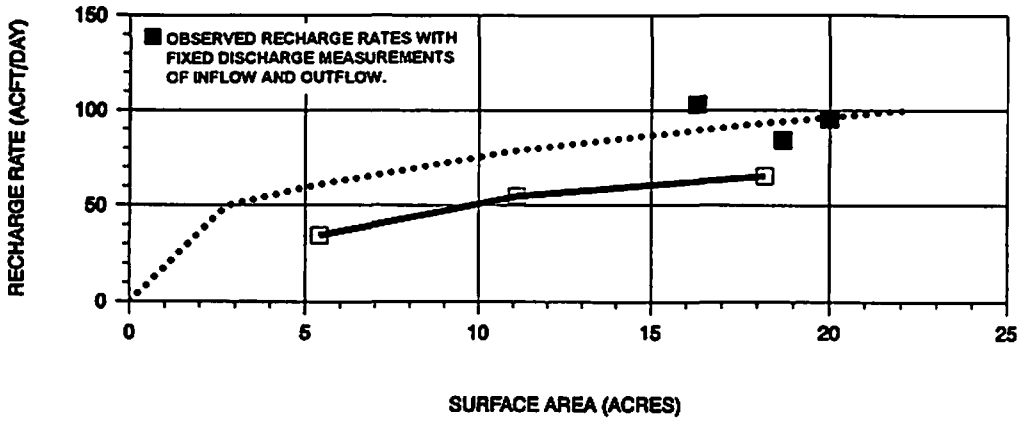
where:

Q = Flow (cubic feet per second);
A = Cross-sectional area of openings (square feet);
g = Acceleration of gravity (32.2 feet per second squared); and
H = Depth of water over the openings, or head (feet).

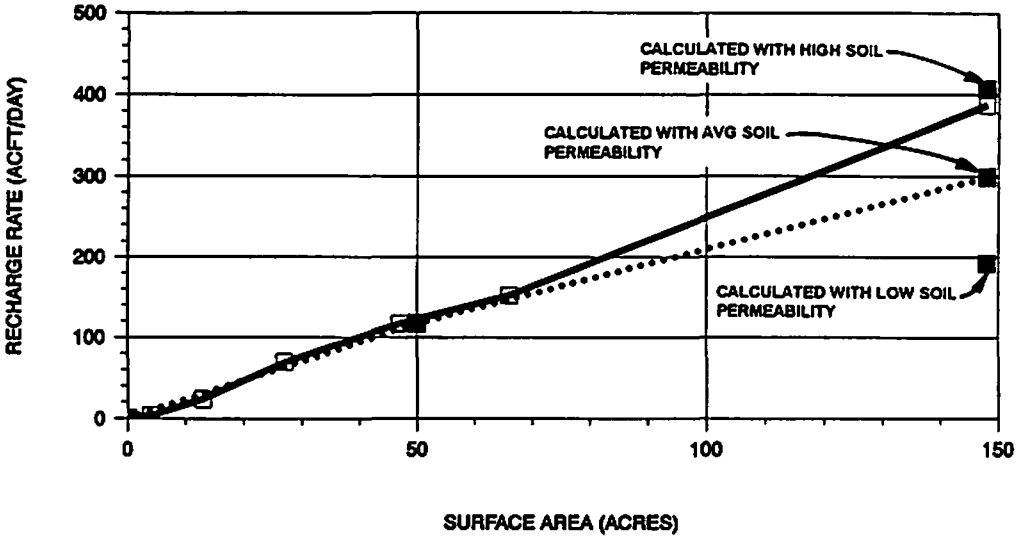
Using this equation, an approximate area of openings in the channel bed (A) was computed based on average measured loss rates (Ref. 35) for the stream reaches potentially inundated by the recharge reservoir, along with an assumed depth of flow coincident with these measurements. The main channel recharge rate was then computed for the range of reservoir capacities considered using the area of openings and the average depth of water in the reservoir.

Calibration and/or verification of the overbank and main channel components of the recharge curves was accomplished by preparing theoretical recharge rate curves for the existing Parkers Creek and Middle Verde Recharge Projects and comparing them to observed recharge rates at these projects. These comparisons are presented in Figure 2.2-6. The calculated recharge rate seems to correlate well with the observed recharge rate at the Parkers Creek Project, which lacks a well-defined channel and is assumed typical of

MIDDLE VERDE RECHARGE PROJECT



PARKERS CREEK RECHARGE PROJECT



LEGEND

- OBSERVED RECHARGE RATE (ASSUMING NO INFLOW)
- CALCULATED RECHARGE RATE

HDR Engineering, Inc.

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

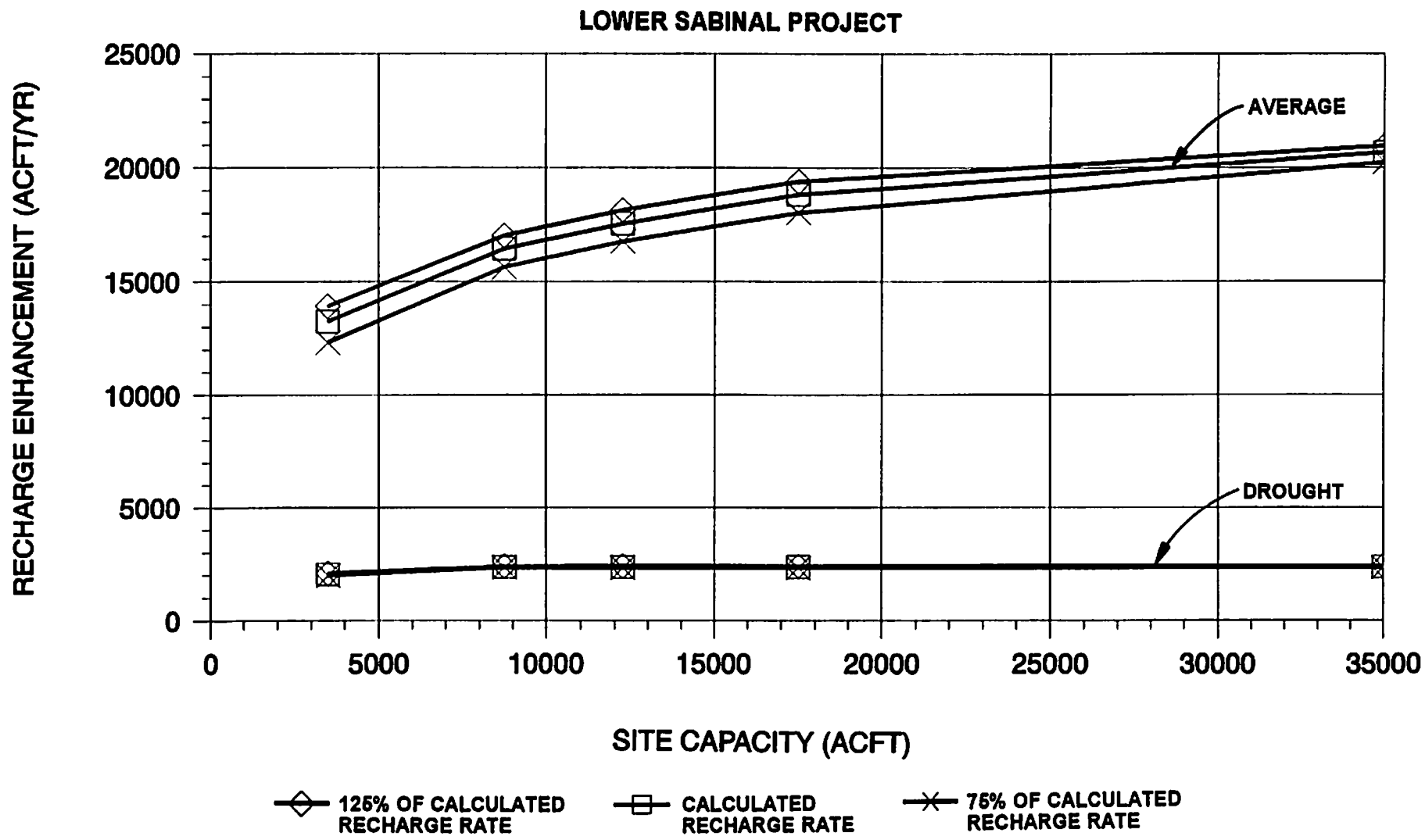
COMPARISON OF COMPUTED AND OBSERVED RECHARGE RATES AT EXISTING RECHARGE PROJECTS

FIGURE 2.2 - 6

overbank areas near the major streams on which the proposed recharge enhancement projects will be located. Due to variability in the soil permeability data, it was decided that average, rather than high, soil permeabilities would be used to develop the overbank component of the recharge rate curves. Calculated and observed recharge rates at the Middle Verde Project, the recharge pool of which is essentially confined to the main channel of Verde Creek, also correlate well and validate the application of a theoretical orifice equation. While comparisons with observed recharge rates tend to support the adopted recharge rate curve methodology, it is important to remember that the existing recharge projects are much smaller than the proposed projects.

The recharge rate curves were reviewed by geohydrologists with LBG-Guyton Associates (LBG) who supported their applicability at all sites with the exception of the Indian Creek Project. LBG's support was based, in part, on their assessment of hydraulic conductivity within the Edwards Aquifer near the existing and proposed recharge projects. This assessment concluded that recharge rates would more likely be controlled by soil cover and surface openings than by the ability of the Edwards Aquifer to transmit water away from the point of recharge. The recharge rate curve developed for the Indian Creek Project was not used in this study because of the respective opinions of LBG, the USGS (Ref. 35), and Dr. Wanakule of the Edwards Aquifer Research and Data Center (Ref. 37) that recharge rates in the Nueces River watershed are at times limited by water levels in the Edwards Aquifer. Hence, the average recharge rate of 2,000 acre-feet per month used in previous studies (Refs. 13 and 14) was adopted for Indian Creek pending further study.

A sensitivity analysis using the Lower Sabinal Project was conducted in order to assess the effect of uncertainty associated with the calculated recharge rate curves on long-term and drought average recharge enhancement. Using recharge rates at 75, 100, and 125 percent of the rates calculated using the methods described herein, monthly recharge enhancement for the 1934-89 period was computed using the Nueces River Basin Model. The results of this sensitivity analysis are presented in Figure 2.2-7. While assumed recharge rates were varied by plus or minus 25 percent, average annual recharge enhancement at the optimum site capacity (8,750 acre-feet) for the Lower Sabinal Project varied by less than plus or minus 5 percent. Average annual recharge enhancement during the 1947-56 drought



NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HDR

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RECHARGE RATE CURVE SENSITIVITY ANALYSIS

FIGURE 2.2 - 7

period remained unaffected. On the basis of this analysis, it might be concluded that long-term average recharge enhancement is no more sensitive to uncertainty in the assumed recharge rate curve than to uncertainty in measurements of areal precipitation and/or streamflow if, in fact, the computed recharge rates are within 25 percent of the actual rates.

2.3 Flood Hydrology

Flood hydrology is the primary factor affecting the cost of many of the recharge enhancement projects. This is because the size of the floods determine dam heights and spillway sizes, along with land acquisition and road relocation requirements. The TNRCC has promulgated design flood criteria for dams, summarized in Table 2.3-1, specifying the applicable percentage of the probable maximum flood (PMF) a structure must pass based on hazard and size classifications. The PMF is defined as the flood that can be expected from the most severe combination of meteorological and hydrologic conditions that are reasonably possible in a region and was assumed to be the design flood event for the structures considered in this study. The PMF is commonly used in the design of projects such as dams and spillways for which virtually complete security from a flood-induced failure is required.

The PMF is an extreme event. The magnitude of the PMF was computed for the six recharge projects using storm events with 24-hour rainfall totals as high as 35 inches, producing peak discharges that range from two to seven times greater than any previously known event. The TNRCC criteria requiring the use of the PMF in the design of dams is principally based on risk. The potential for severe damage and loss of life due to a dam failure, along with the economic loss of the structure itself, dictate using a low level of risk in the design. For structures with a design life of 100 years which are sized to safely pass up to the 100-year return interval flood event, the risk of failure during the design life would be 63 percent. This is a rather high risk for a multi-million dollar structure with the potential for devastating impacts downstream if a failure were to occur. In order to achieve a risk of failure of one percent during the design life, the structure would need to be designed for the 10,000-year return interval flood event. This highlights the fact that a low level of risk requires designing for a very rare and extreme event. Significant uncertainty exists in the estimation of even the 100-year flood using a gaged record of 40 to 50 years, thus any analysis of extreme events such as a 10,000-year flood would be extremely unreliable. Therefore, the PMF is required as the design flood event for dams in order to represent the physical upper limit of flood severity.

Table 2.3-1
Texas Natural Resource Conservation Commission
Hydrologic Criteria For Dams

Hazard Classification	Size Classification	Design Flood Event
Low Hazard	Small Intermediate Large	¼ PMF ¼ PMF to ½ PMF PMF
Significant Hazard	Small Intermediate Large	¼ PMF to ½ PMF ½ PMF to PMF PMF
High Hazard	Small Intermediate Large	PMF PMF PMF

Notes:

Hazard Classification:

- Low hazard dams are defined as those dams where failure may damage farm buildings, limited agricultural improvements, and county roads. For low hazard dams, no loss of human life would be expected.
- Significant hazard dams are defined as those dams where failure would not be expected to cause loss of human life, but may cause damage to isolated homes, secondary highways, minor railroads, or cause interruption of service or use of relatively important public utilities.
- High hazard dams are defined as those dams where failure would be expected to cause loss of human life, extensive damage to agricultural, industrial, or commercial facilities, important public utilities, main highways, or railroads.

Size Classification:

- Small size dams are classified as those dams which have a total height less than 40 feet and have a total reservoir storage at top of dam of less than 1,000 acre-feet.
- Intermediate size dams are classified as those dams which have a total height between 40 feet and 100 feet and a total reservoir storage at top of dam between 1,000 acre-feet and 50,000 acre-feet.
- Large dams are classified as those dams which have a total height in excess of 100 feet and have a total reservoir storage at top of dam greater than 50,000 acre-feet.

2.3.1 History of Flooding

Several major storm events have occurred in the region which have resulted in severe flooding on each of the streams considered in this study. Climate and physiography are the two primary contributing factors to the chronic floods that occur in the region.

The dominant physiographic element of the region is the Balcones Escarpment which separates the deeply dissected limestone terrain of the Edwards Plateau from the gently sloping, undulating clay and sand terrain of the Coastal Plain. Studies have shown that significant rainfall events occur as a result of convective thunderstorm activity and the movement of moisture-laden air along the established tropical Gulf storm tract (Ref. 1). These storms have produced some astonishing amounts of rainfall, including both national and world records for a given storm duration. The western edge of the Balcones fault zone is characterized by a relative steep, high escarpment at generally right angles to the direction of storm winds. The situation is ideal for moisture-laden air to be lifted as it moves northward from the Gulf, resulting in thunderstorms being initiated where moist air is forced to rise (Ref. 34). These lift convective storms produce extremely heavy rainfall. One of the most spectacular cloudburst-type thunderstorms on record occurred on May 31, 1935, when a tongue of moist air protruded from the Gulf of Mexico to the vicinity of D'Hanis, Texas. The lift effect of this convectively unstable air at the Balcones Escarpment resulted in the production of 22 inches of rainfall in two hours and 45 minutes (Ref. 1).

Weather disturbances of tropical origin have generated some of the greatest storms in Texas. The meteorology of such storms is characterized by easterly waves which pick up large quantities of moisture while passing over thousands of miles of warm tropical seas. As a result of weather conditions in the Caribbean, stable easterly waves are most likely to occur in the month of September. If an especially vigorous wave reaches the orographic barrier of the Balcones Escarpment, long-duration, heavy rains may result. This happened in the great Thrall, Texas storm (located northeast of the study area) of September 9-10, 1921, which produced locally 36.4 inches of rainfall in 18 hours and 38.2 inches of rainfall in 24 hours. This storm is considered to be one of the greatest of all continental United States rainstorms. Another example is the storm of September 9-10, 1952, which was the result of the near simultaneous arrival over Texas of a pressure surge from the northeast

and an easterly wave trough. The warm easterly tropical air current decreased in stability while lifting over the Balcones Escarpment and ascended rain-cooled air that developed over the Edwards Plateau region. Storm totals of 20 to 26 inches were concentrated in small centers over the upper Pedernales and Guadalupe Rivers.

Flooding along the Edwards Aquifer recharge zone originating from the Edwards Plateau area is caused in part by the extreme storm events that occur in the area and also by physical characteristics of the drainage basins and stream channels. Very rapid runoff in the Edwards Plateau area is promoted by sparse scrub vegetation and bare limestone slopes. Steep slopes dominate the headwaters of the major streams which generate rapidly moving flood waves, producing significant flow depths. Some of the largest floods that have occurred in the streams in the study area have produced stages in excess of 30 feet to 40 feet.

Table 2.3-2 summarizes the largest floods that have occurred for each of the streams considered in this study with published records. The Nueces River and West Nueces River have experienced some of the larger floods in the region, partly due to their large drainage areas and sparsely vegetated, steep slopes. The largest flood of record measured on the Nueces River occurred on September 24, 1955; interestingly, this was during the worst drought of record in the region. Storm total rainfall amounts were recorded in the headwaters of the Nueces River watershed ranging from five inches to 24 inches over an 18-hour period. This storm produced a peak discharge of 307,000 cubic feet per second (cfs) causing the river to rise to a stage of 32.7 feet at Laguna. The same storm event produced a peak flow of 189,000 cfs and caused the river to rise to a peak stage of 24.6 feet near Uvalde. Outside of the period of record, the largest known flood event on the Nueces River since 1836 occurred on June 14, 1935. This flood occurred in response to a storm event that produced rainfall depths ranging from six inches to 17 inches over a 3-day period. The most intense portion of the storm occurred during the later hours of June 13 causing the Nueces River to rise to a stage of 40.4 feet near Uvalde, corresponding to a peak discharge of 616,000 cfs. The same storm event produced the largest known flood event on the West Nueces River since 1879, causing the river to rise to a stage of 40.0 feet (550,000 cfs) near Bracketville.

Table 2.3-2
Flood History Summary

Stream	Gage Location	Gaged Records Since[1]	Largest Flood for Period of Record[2]			Largest Flood Outside Period of Record[3]			Largest Flood Since[4]
			Peak Flow (cfs)	Peak Stage (feet)	Date	Peak Flow (cfs)	Peak Stage (feet)	Date	
Nueces River	at Laguna, 737 sq.mi	1923	307,000	32.7	9/24/1955	210,000	29.0	6/1913	1866
	near Uvalde, 1,861 sq.mi.	1939	189,000	24.6	9/24/1955	616,000	40.4	6/14/1935	1836[5]
West Nueces River	near Bracketville, 694 sq.mi.	1939	246,000	31.3	9/20/1964	550,000	40.0	6/14/1935	1879
Dry Frio River	near Reagan Wells, 126 sq.mi.	1952	123,000	27.6	8/13/1966	N/A	33.0	1880	1875
Frio River	at Concan, 389 sq.mi.	1923	162,000	34.4	7/01/1932	N/A	28.8	1913	1869
	near Uvalde, 631 sq.mi.	1952	99,500	25.1	5/29/1987	N/A	35.0	1894	1887[6]
Sabinal River	near Sabinal, 206 sq.mi.	1942	55,200	28.3	6/17/1958	N/A	33.0	7/02/1932	1892
	at Sabinal, 241 sq.mi.	1952	73,300	33.3	6/17/1958	N/A	40.0	8/23/1919	1890[7]
Hondo Creek	near Tarpley, 95.6 sq.mi.	1952	69,800	28.2	5/17/1958	58,500	26.0	7/1932	1907
	near Hondo, 149 sq.mi.	1960	51,800	17.2	5/17/1958	N/A	21.0	9/1919	1875

Notes:
1) Published records based on an established U.S. Geological Survey streamflow gaging station.
2) Largest flood since published records were available.
3) Largest flood known to have occurred outside of the period of published record. Usually based on information from local residents.
4) Indicates that the largest flood event known, either during or outside of the period of record, is the largest flood known to have occurred since at least this time.
5) Large floods also occurred in 1901 and 1913 (stages unknown).
6) A flood that was several feet higher than the 1894 flood occurred prior to 1887.
7) A legend exists that a flood in 1858 covered part of the townsite of Sabinal and was higher than any flood since 1850.

The Dry Frio River was subjected to a severe storm on August 13, 1966. A storm cell which produced 10.4 inches of rain in six hours was responsible for a flood event which peaked at a stage of 27.6 feet, corresponding to a discharge of 123,000 cfs. The Frio River has been subjected to several significant flood events over the period of record, the largest of which occurred on July 1, 1932. This flood caused a peak stage of 34.4 feet (162,000 cfs) at Concan in response to storm total rainfall depths ranging from eight inches to 25 inches, most of which occurred over a 18-hour period. The Sabinal River experienced its largest flood event of record on June 17, 1958. This event produced a peak stage of 33.3 feet (73,300 cfs) at Sabinal. Larger flood events have been reported to have occurred in 1919 and 1932. Hondo Creek experienced its largest recorded flood event on May 17, 1958. The flood stage on Hondo Creek rose to 28.2 feet (69,800 cfs) at Tarpley. Other large flood events occurred on Hondo Creek in September, 1919 and July, 1932.

2.3.2 Flood Hydrology Model

Dam height and spillway requirements are principally based on the volume and magnitude of the design flood event. The design flood event, which is the PMF event for large, high hazard dams, is determined using a computer model that simulates a watershed's response to precipitation. The HEC-1 Flood Hydrograph Package (Ref. 16), developed by the U.S. Army Corps of Engineers, was utilized to compute the design flood event at each proposed dam site. The HEC-1 model is designed to simulate the surface runoff response of a watershed to precipitation by representing the watershed as a system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process. Representation of each component involves the specification of a set of parameters which describe the characteristics of the component and the mathematical relations which describe the physical process. The result is the computation of a streamflow hydrograph at each dam site.

Surface runoff is computed for the design flood event with the primary component being a precipitation hyetograph. Precipitation excess is computed by subtracting infiltration and surface detention losses based on a particular soil water infiltration rate function. Rainfall and infiltration are assumed to be uniform over the entire watershed being

modelled. The resulting rainfall excesses are then routed using the unit hydrograph method to the downstream outlet of the watershed. A HEC-1 model for a single watershed can therefore be defined by four basic components. These are:

1) watershed area;
2) precipitation hyetograph;
3) precipitation losses; and
4) unit hydrograph routing parameters.

The watershed area is a known parameter that is determined based on available topographic mapping. The precipitation hyetograph, which is the primary component of the model, describes the volume and temporal pattern of rainfall that occurs across the watershed for a particular storm event. The last two components, precipitation losses and unit hydrograph routing parameters, present the primary unknowns in the development of the rainfall-runoff model. Precipitation losses are calculated in HEC-1 using a loss rate function. The loss rate function selected as the most appropriate for the watersheds considered in this study was the initial and uniform loss rate function, which is commonly used to represent the average precipitation losses for large watersheds. Precipitation losses are defined by two parameters in the initial and uniform loss rate function. The first parameter, the initial loss, represents the amount of rainfall that occurs before any runoff will begin. This term generally reflects the land surface interception of precipitation by vegetation, both trees and grass, initial saturation of the surface soil matrix, and depression storage on the ground surface as water accumulates in hollows, cracks, and crevices or in any area where water is not free to move as overland flow. The second term, uniform loss rate, describes the infiltration of precipitation into the soil which is assumed to occur at a uniform rate over the duration of the storm event. In HEC-1, precipitation losses are assumed to be lost from the system and do not contribute to the runoff process.

The unit hydrograph method is the component in the rainfall-runoff model that transforms the rainfall excess into a surface runoff hydrograph. The unit hydrograph is a typical hydrograph for a watershed. Since the physical characteristics of a watershed (i.e., shape, size, slope, etc.) generally remain constant, it is expected that considerable similarity in the shape of runoff hydrographs from storms of similar rainfall characteristics will result.

The unit hydrograph for a watershed is defined as the direct runoff hydrograph resulting from one inch of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration (Ref 4). Snyder's unit hydrograph method was utilized in the HEC-1 model to develop a unit hydrograph for each watershed at the proposed dam locations. Snyder's method relates hydrograph characteristics to the physical characteristics of the watershed. Two basic parameters, basin lag time and Snyder's peaking coefficient, are required to define the unit hydrograph using Snyder's method.

The basin lag time is defined as the time between the centroid of the rainfall excess for a specified storm to the time of the peak rate of runoff. Snyder's method estimates the basin lag time to be a function of basin size and shape expressed by:

$$t_p = C_t \, (L \, L_c)^{0.3}$$

where:

t_p =	basin lag time (hours);
C_t =	coefficient depending on the basin properties;
L =	the main stream distance from the outlet to the divide (miles); and
L_c =	the main stream distance from the outlet to a point opposite the basin centroid (miles).

The use of L and L_c accounts for the watershed shape and size and C_t is considered to account for wide variations in topography, from plains to mountainous regions. Values of C_t have been found to range from 0.4 for the steep regions of Southern California to 8.0 along the Gulf of Mexico. Linsley (Ref. 20) proposed a modified form of Snyder's equation:

$$t_p = C_L \left(\frac{L \, L_c}{\sqrt{S}} \right)^{0.3}$$

where s is the average watershed slope (ft/ft) and C_L is a coefficient dependent on basin properties reflecting the inclusion of slope in the equation. Known values of basin lag time can be correlated to the watershed characteristics ($L\ L_c/S^{1/2}$) for watersheds with similar hydrologic characteristics in order to define a regional relationship for C_L.

Snyder's peaking coefficient is used to compute the peak discharge of the unit hydrograph. The peak discharge in Snyder's unit hydrograph is expressed by the following equation:

$$Q_p = \frac{640\ C_p\ A}{t_p}$$

where:

Q_p = peak discharge of the unit hydrograph (cfs);
C_p = Snyder's peaking coefficient;
A = watershed size (sq.mi.); and
t_p = basin lag time (hours).

Snyder's peaking coefficient accounts for flood wave and storage conditions. It is a function of the lag time, duration of storm producing runoff, configuration of the effective drainage area contributing to the peak flow, and watershed size. Values of C_p range from 0.4 to 0.8 and generally indicate the retention or storage capacity of the watershed. Larger values of C_p are generally associated with smaller values of C_L (Ref. 36).

2.3.3 Historical Flood Calibrations

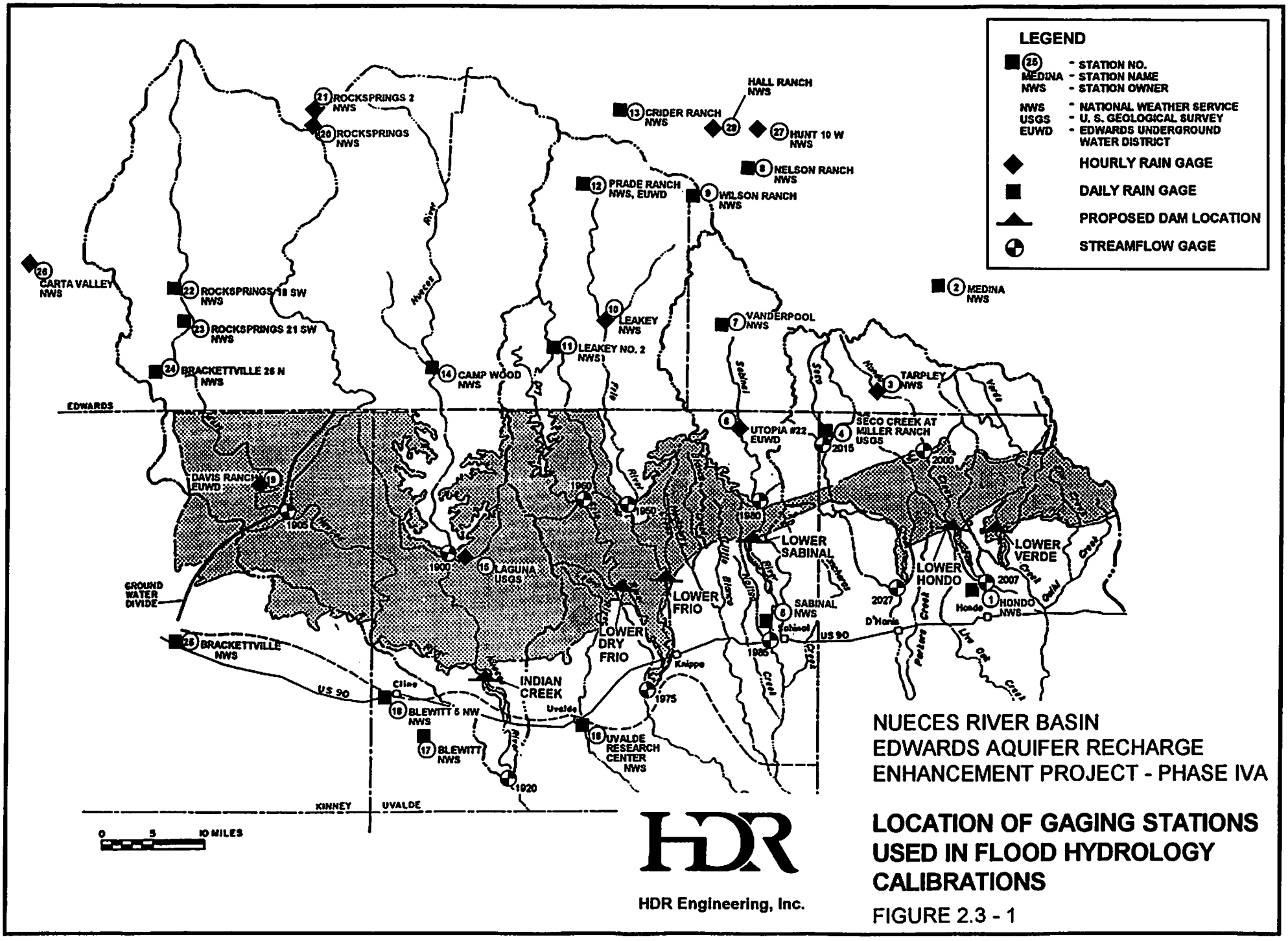
The parameters, t_p and C_p, which are required to define the unit hydrograph using Snyder's method are specific to a given watershed and can be derived by an evaluation of these parameters for the study area. This is accomplished by calibrating the unit hydrograph parameters for flood events measured at gaged locations in the region. Model calibration is accomplished by simulating historical storm events and comparing the computed runoff hydrograph to the measured runoff hydrograph at a streamflow gaging station. The individual parameters are optimized in order to compute a runoff hydrograph that is comparable to the measured runoff hydrograph from the historical storm event.

Streamflow gaging stations with a minimum period of record of 34 years are located throughout the study area in the immediate vicinity of each of the proposed recharge dams, with the exception of the Verde Creek Project. A streamflow gaging station was established in the upper part of the Verde Creek watershed in 1991, but no records were available for major storm events. Data from nearby watersheds was used to select model parameters for the Verde Creek watershed.

A review of gage records revealed several major flood events where adequate data was available for model calibration. Data required for model calibration includes both precipitation to describe the storm event and streamflow to describe the runoff hydrograph. The flood events used in the model calibrations were usually some of the larger flood events on record. A total of 22 flood events were calibrated. Data from 28 rainfall gaging stations and nine streamflow gaging stations was used to perform the model calibrations. The locations of the gaging stations are identified in Figure 2.3-1.

For each flood event, daily, hourly, and 15-minute interval rainfall gages were identified and plotted on a watershed map. Rainfall gage data was researched from a variety of sources, including the National Weather Service, USGS, EUWD, and the TWDB. In general, rainfall data recorded every 15 minutes was only available at a few select gages which began operation in 1990, hence, hourly gages were relied upon heavily to obtain the temporal distribution of rainfall for each storm event. Obtaining rainfall data that could be used to accurately describe the storm event, especially those storm events prior to the 1980's, proved to be the primary constraint in calibration of the flood hydrology models.

Once the rainfall gages were identified for a storm event, the Thiessen polygon procedure was employed to compute the basin average storm total rainfall. This procedure provides a method to determine the weight of each rainfall gaging station that should be applied relative to its location to the watershed area. Once the storm total rainfall was computed, the rainfall gages which could be used to describe the temporal rainfall pattern were selected. For several of the storm events, this was based on the closest hourly or 15-minute gaging station. However, for some storm events where information was available at more than one hourly or 15-minute gaging station, the data at each of the gaging stations was used to describe the pattern of rainfall.


LEGEND
25 - STATION NO.
MEDINA - STATION NAME
NWS - STATION OWNER
NWS - NATIONAL WEATHER SERVICE
USGS - U. S. GEOLOGICAL SURVEY
EUWD - EDWARDS UNDERGROUND WATER DISTRICT
HOURLY RAIN GAGE
DAILY RAIN GAGE
PROPOSED DAM LOCATION
STREAMFLOW GAGE
21 ROCKSPRINGS 2 NWS
20 ROCKSPRINGS NWS
13 CRIDER RANCH NWS
HALL RANCH NWS
28
27 HUNT 10 W NWS
8 NELSON RANCH NWS
12 PRADE RANCH NWS, EUWD
9 WILSON RANCH NWS
26 CARTA VALLEY NWS
22 ROCKSPRINGS 18 SW NWS
23 ROCKSPRINGS 21 SW NWS
24 BRACKETTVILLE 26 N NWS
14 CAMP WOOD NWS
10 LEAKEY NWS
11 LEAKEY NO. 2 NWS1
7 VANDERPOOL NWS
2 MEDINA NWS
3 TARPLEY NWS
6 UTOPIA #22 EUWD
4 SECO CREEK AT MILLER RANCH USGS
2015
2000
EDWARDS
DAVIS RANCH EUWD
19
1905
1960
1950
1980
LOWER SABINAL
LOWER VERDE
LOWER HONDO
2007
1 HONDO NWS
Hondo
2027
D'Hondo
1900
15 LAGUNA USGS
LOWER FRIO
LOWER DRY FRIO
SABINAL NWS
8
Sabinal
1985
US 90
GROUND WATER DIVIDE
25 BRACKETTVILLE NWS
US 90
Cline
18 BLEWITT 5 NW NWS
17 BLEWITT NWS
INDIAN CREEK
Uvalde
Knippa
1975
16 UVALDE RESEARCH CENTER NWS
1920
KINNEY
UVALDE
0 5 10 MILES
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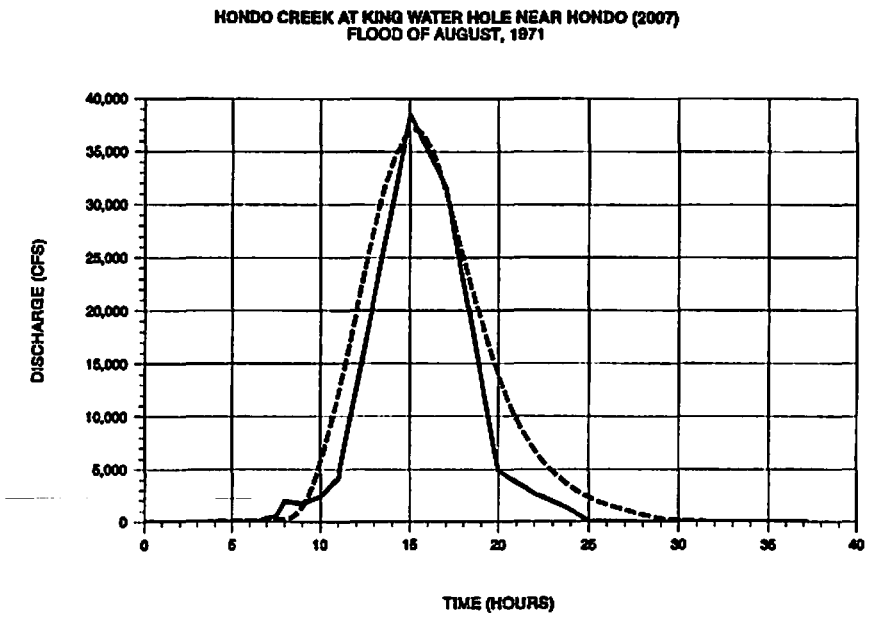
NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA
LOCATION OF GAGING STATIONS
USED IN FLOOD HYDROLOGY
CALIBRATIONS
FIGURE 2.3 - 1

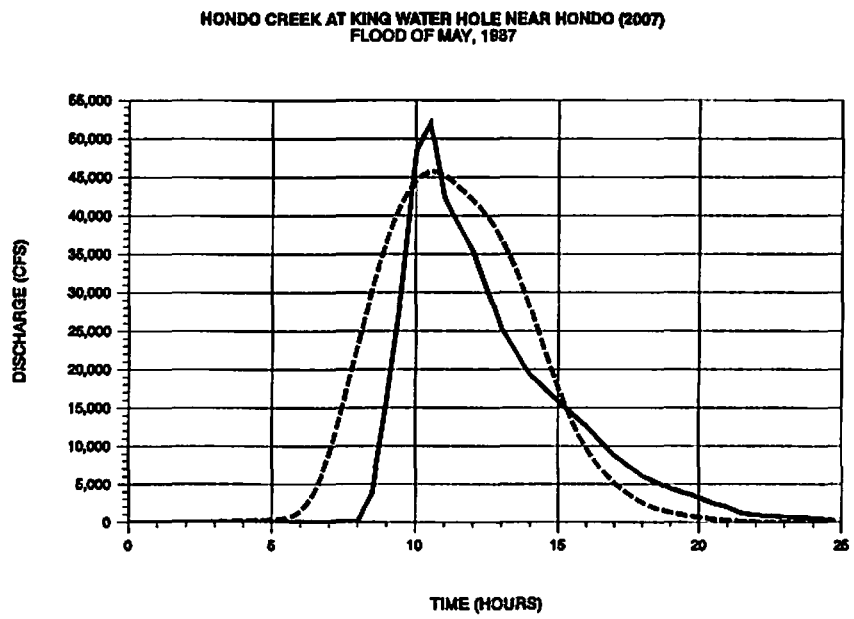
The runoff hydrograph at the streamflow gaging station used in the calibration was determined from USGS records. Data for historical flood events were usually provided in the form of a time-stage series. The discharge for each time interval, usually one or two hours, was determined using the stage-discharge rating table in use at that time. The baseflow component of the streamflow hydrograph was separated from the runoff component of the flood event, although it was generally found to be a relatively minor component in comparison to the volume and magnitude of the flood.

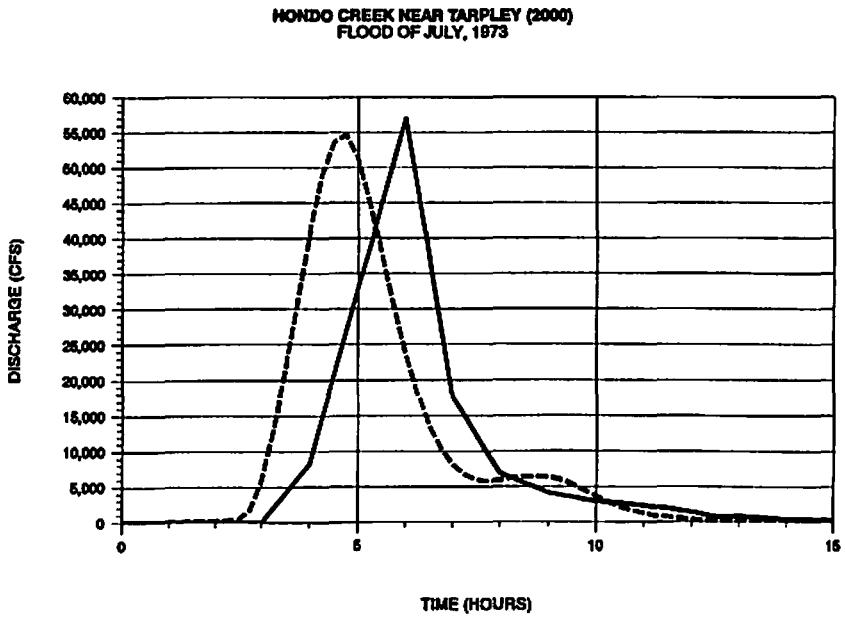
Calibration to the flood events was accomplished by optimizing the unit hydrograph parameters and loss rate parameters until, after a number of iterations, the computed peak flow, runoff volume, and hydrograph shape closely matched the observed runoff event. The calibrations involved varying the basin lag time (t_p), peaking coefficient (C_p), initial loss (L_I), and the uniform loss rate (L_U). The steep rise in the observed hydrographs, which is indicative of the region, resulted in the adoption of a peaking coefficient of 0.80, the largest value which HEC-1 will effectively accept. Thus, only the remaining three parameters required optimization. Since the peak flow of the design inflow hydrograph was of principal concern in the dam and spillway designs, calibration of the peak flow for the historical flood events was given the most priority. In addition, the parameters were also calibrated to correlate the runoff volume and shape of the runoff hydrograph. The basin lag time is the primary parameter contributing to the computed peak flow of the calibrated runoff hydrograph. Although the initial loss and uniform loss rate parameters also contribute to the computation of the peak flow, they are primarily used to correlate the runoff volume.

The calibration results of each flood event showed that the peak discharge, runoff volume, and shape of the runoff hydrograph could be simulated well. Figures 2.3-2 through 2.3-6 show comparisons of observed runoff hydrographs and computed runoff hydrographs using calibrated model parameters for selected flood events. A complete summary of the model calibrations is provided in Table 2.3-3.

Because of the many variables and components involved in the flood hydrograph calibrations, a range in the results of each parameter will typically result. In order to derive the parameters to be used in computing the design inflow hydrographs for each project, the calibrated parameters for each individual watershed were considered on a regional basis.







LEGEND

OBSERVED DISCHARGE
COMPUTED DISCHARGE

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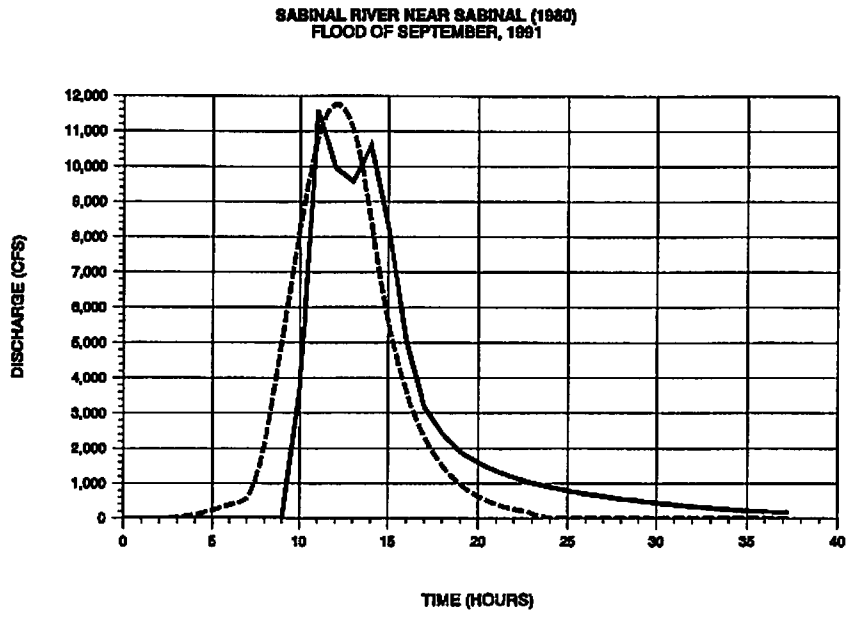
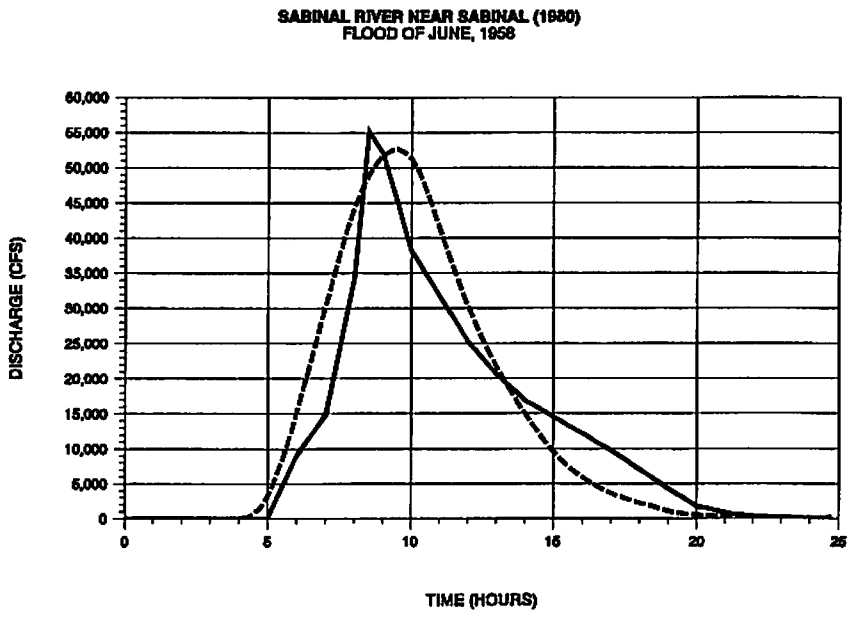
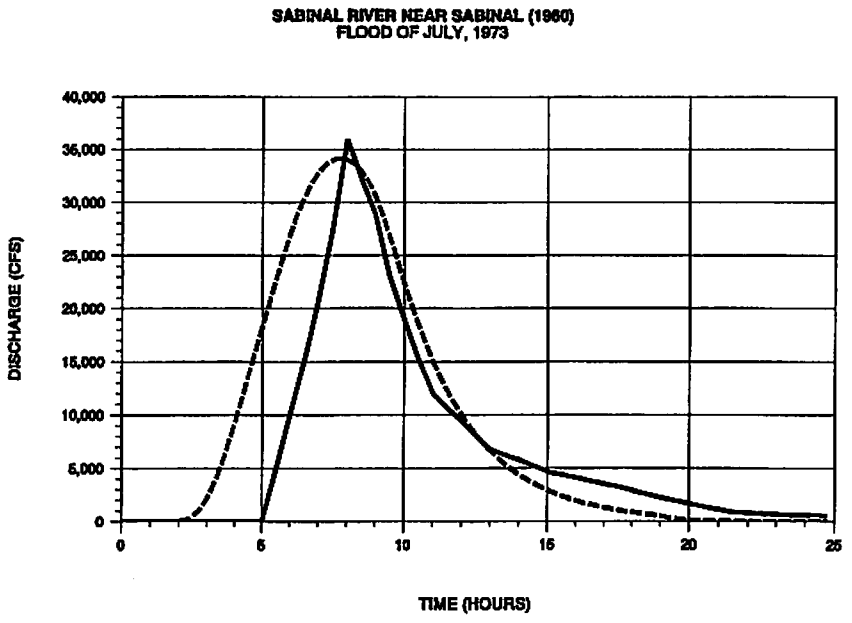
ENHANCEMENT PROJECT - PHASE IVA

HISTORICAL FLOOD CALIBRATIONS
HONDO CREEK

FIGURE 2.3 - 2







LEGEND

- OBSERVED DISCHARGE
- COMPUTED DISCHARGE

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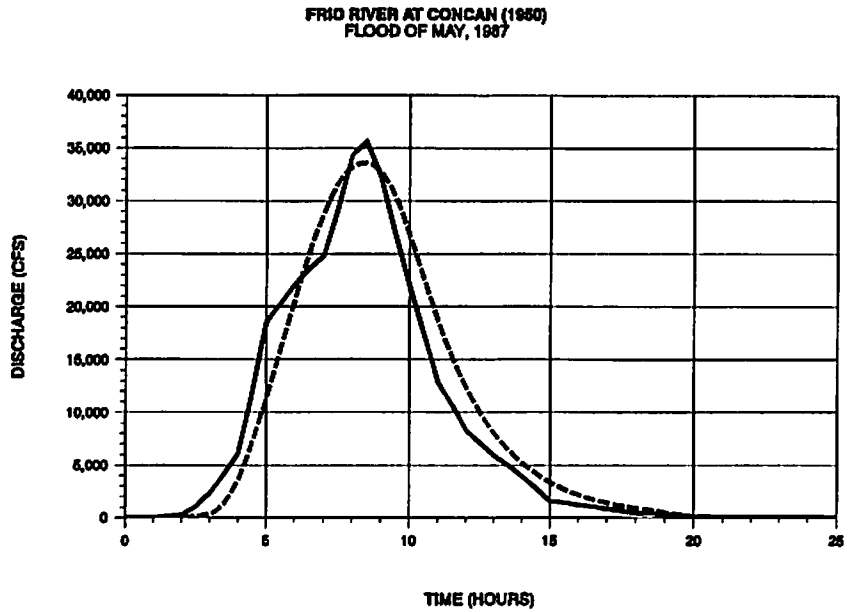
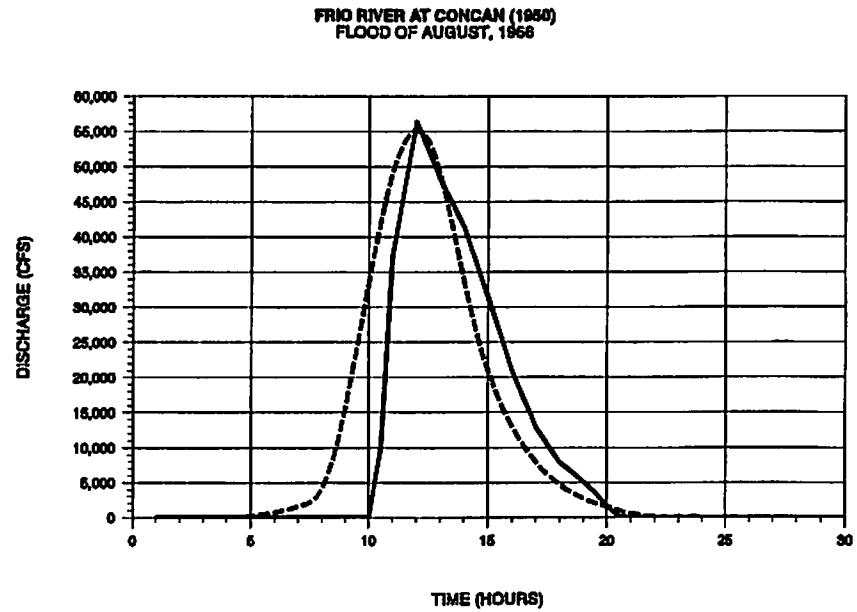
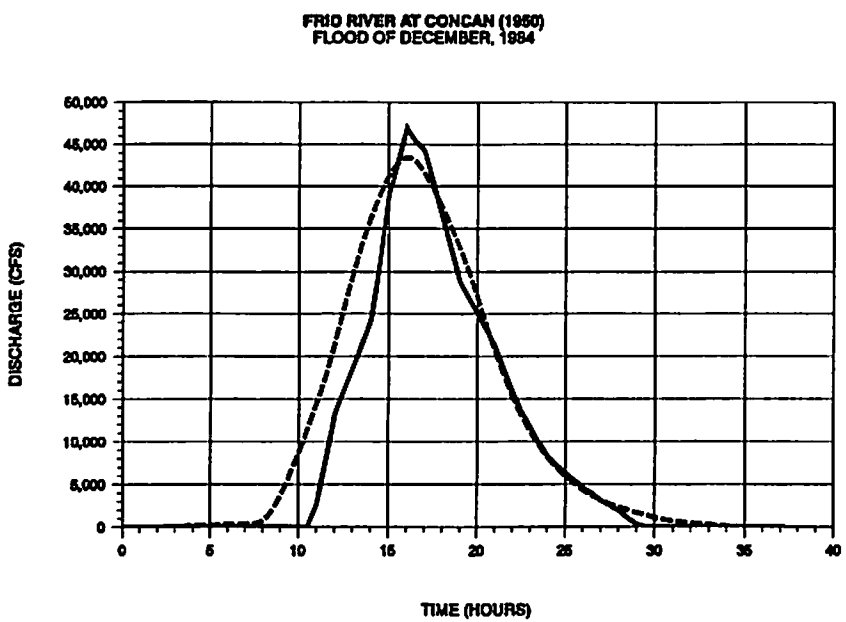
NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HISTORICAL FLOOD CALIBRATIONS
SABINAL RIVER

FIGURE 2.3 - 3







LEGEND

- OBSERVED DISCHARGE
- COMPUTED DISCHARGE

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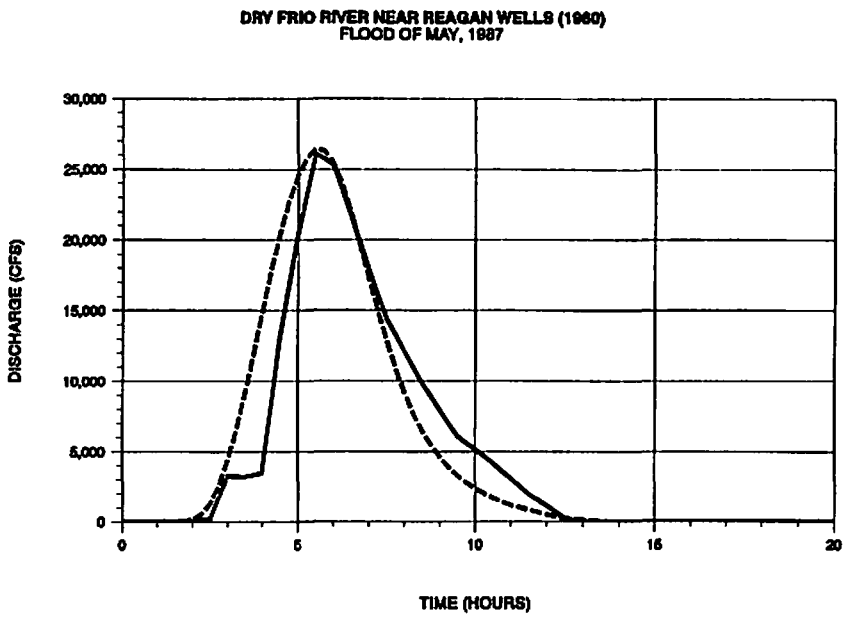
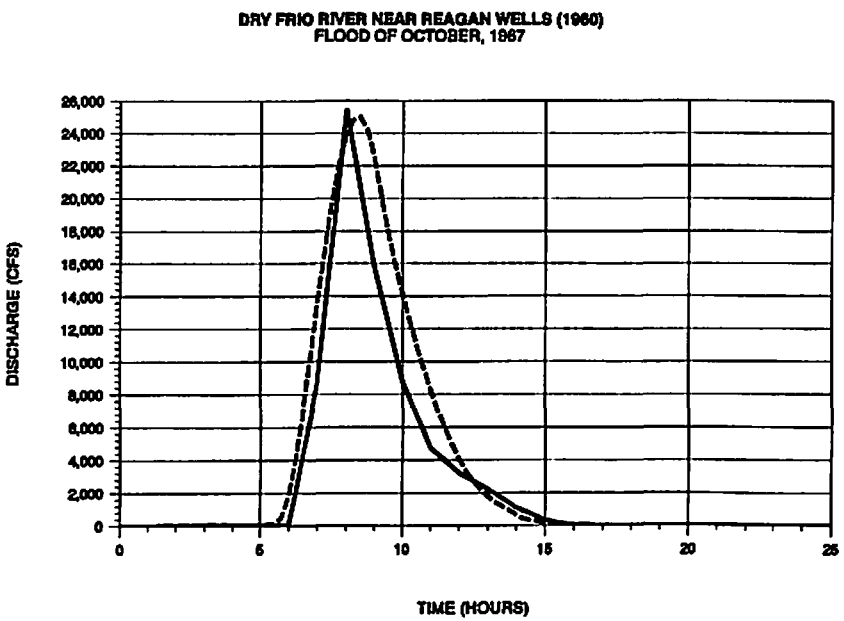
NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HISTORICAL FLOOD CALIBRATIONS
FRIO RIVER

FIGURE 2.3 - 4





LEGEND

OBSERVED DISCHARGE
COMPUTED DISCHARGE

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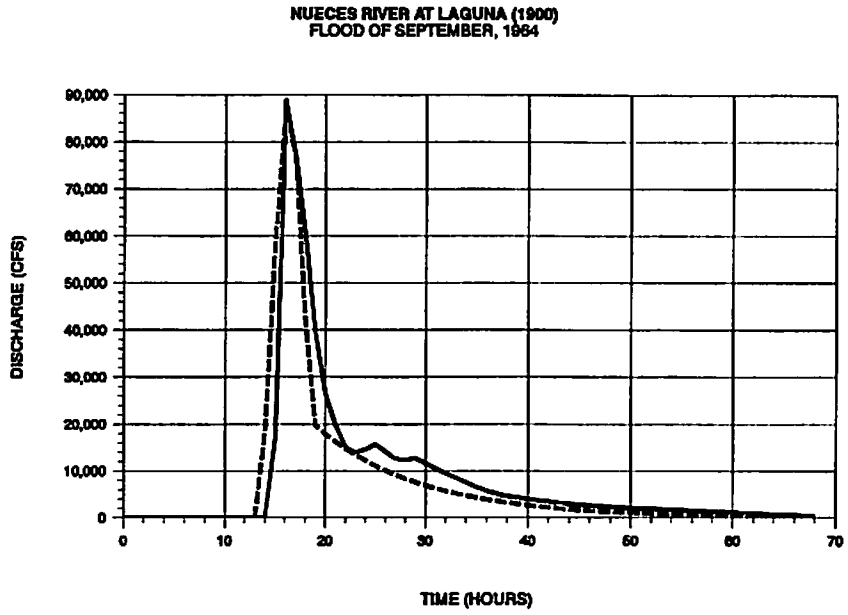
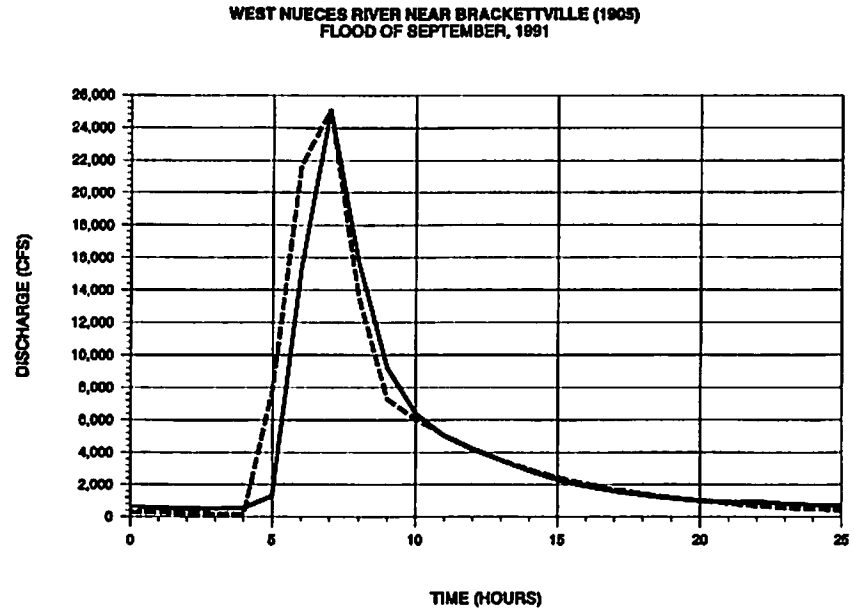
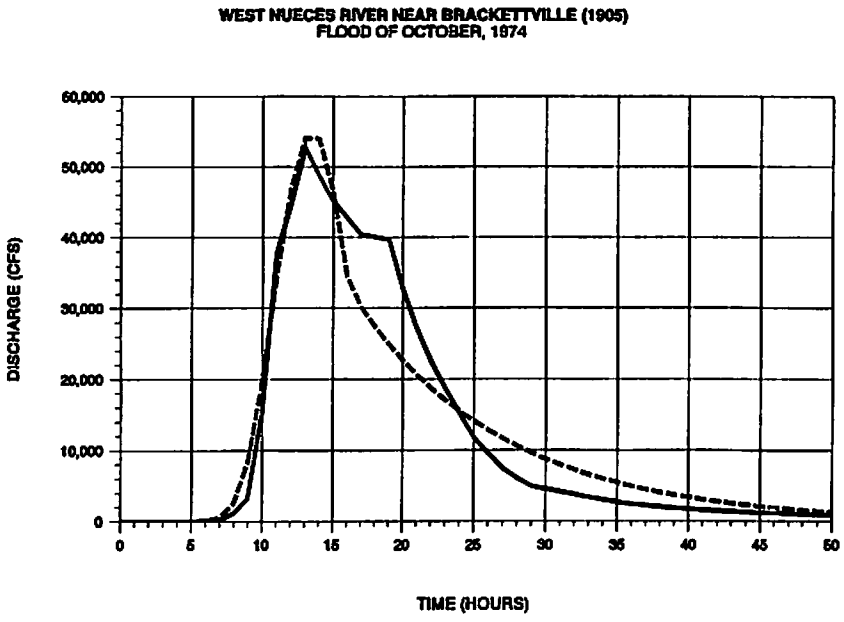
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HISTORICAL FLOOD CALIBRATIONS
DRY FRIO RIVER

FIGURE 2.3 - 5







LEGEND

OBSERVED DISCHARGE
COMPUTED DISCHARGE

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EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HISTORICAL FLOOD CALIBRATIONS
NUECES AND WEST NUECES RIVERS

FIGURE 2.3 - 6

Table 2.3-3
Summary of Historic Flood Calibrations

Stream		Watershed Characteristics					Observed Data			Calibrated Parameters			Computed Results			
Name	Gage I.D.	Area sq.mi.	L miles	L_c miles	s ft/ft	Flood Event	P inches	Q_p cfs	V inches	t_p hours	L_I inches	L_U in/hr	Q_p cfs	V inches	Q_p Error	V Error
Dry Frio River	1960	126	35.6	15.3	0.0044	Oct, 1967	4.00	25,470	0.87	2.50	1.80	0.40	20,477	0.81	-20%	-7%
						May, 1987	5.11	26,108	1.24	3.00	1.50	0.60	26,427	1.29	1%	4%
Frio River	1950	389	49.1	29.9	0.0035	Aug, 1966	8.20	56,462	1.03	4.00	6.80	0.05	55,163	1.18	-2%	14%
						Dec, 1984	4.85	46,920	1.40	6.75	2.90	0.05	43,454	1.65	-7%	18%
						May, 1987	4.25	35,620	0.79	5.00	2.40	0.25	33,601	0.84	-6%	6%
Hondo Creek	2000	95.6	16.1	8.9	0.0061	Aug, 1971	7.00	24,359	1.47	2.50	3.60	0.40	23,385	1.52	-4%	3%
						Jul, 1973	7.86	57,057	2.19	1.75	3.00	0.40	54,602	2.40	-4%	10%
Hondo Creek	2007	149	35.0	22.3	0.0034	Aug, 1971	6.63	38,490	2.40	6.00	2.35	0.20	37,029	3.07	-4%	28%
						Jul, 1973	7.32	46,900	1.65	4.00	2.50	0.50	39,696	2.09	-15%	27%
						May, 1987	7.21	51,900	2.51	3.50	3.50	0.10	45,748	3.16	-12%	26%
Sabinal River	1980	206	35.6	10.7	0.0044	Jun, 1958	5.87	55,137	2.26	4.50	0.75	0.25	52,638	2.47	-5%	9%
						Jul, 1973	4.69	35,924	1.28	5.25	1.00	0.40	34,170	1.66	-5%	30%
						Sep, 1991	5.42	11,577	0.58	5.00	3.60	0.60	11,758	0.56	2%	4%
Sabinal River	1985	241	54.4	35.2	0.0035	Jun, 1958	5.98	73,296	2.57	6.75	1.00	0.10	55,052	3.32	-25%	29%
						May, 1987	6.32	34,092	1.72	6.75	3.25	0.20	30,935	1.84	-9%	7%
West Nueces River	1905	682	62.5	34.7	0.0032	Sep, 1964	7.40	246,000	2.74	1.50	2.30	0.25	242,105	2.84	-2%	4%
						Oct, 1974	5.89	52,700	1.42	5.00	3.15	0.28	54,075	1.42	3%	0%
						Sep, 1991	5.28	25,100	0.36	2.00	3.21	0.16	25,113	0.37	0%	3%
Nueces River	1900	756	58.2	28.7	0.0034	Sep, 1955	7.85	307,000	3.92	5.90	2.34	0.17	205,425	4.31	-33%	10%
						Sep, 1964	7.07	89,000	1.19	2.70	3.70	0.33	87,230	1.05	-2%	-12%
						Oct, 1974	7.14	150,000	3.06	8.00	2.93	0.16	112,846	3.07	-25%	0%
						Sep, 1991	7.53	36,700	1.31	4.00	4.67	0.29	37,135	0.83	3%	-37%

Notes: L watershed length; L_c watershed length to centroid; s average watershed slope; P average storm rainfall; Q_p peak discharge; V runoff volume; t_p basin lag time; L_I initial loss; L_U uniform loss rate

In addition to the calibrations performed in this study, the U.S. Army Corps of Engineers performed calibrations of basin lag times based on individual flood events for watersheds in the study area and for nearby watersheds in the Guadalupe River Basin and Blanco River Basin (Ref. 33). The Corps of Engineers calibration data, summarized in Table 2.3-4, was included in the development of a regional relationship for basin lag time.

As discussed earlier, the basin lag time can be correlated to the physical parameters of the watershed using the relationship:

$$t_p = C_L \left(\frac{L\, L_c}{\sqrt{s}} \right)^{0.3}$$

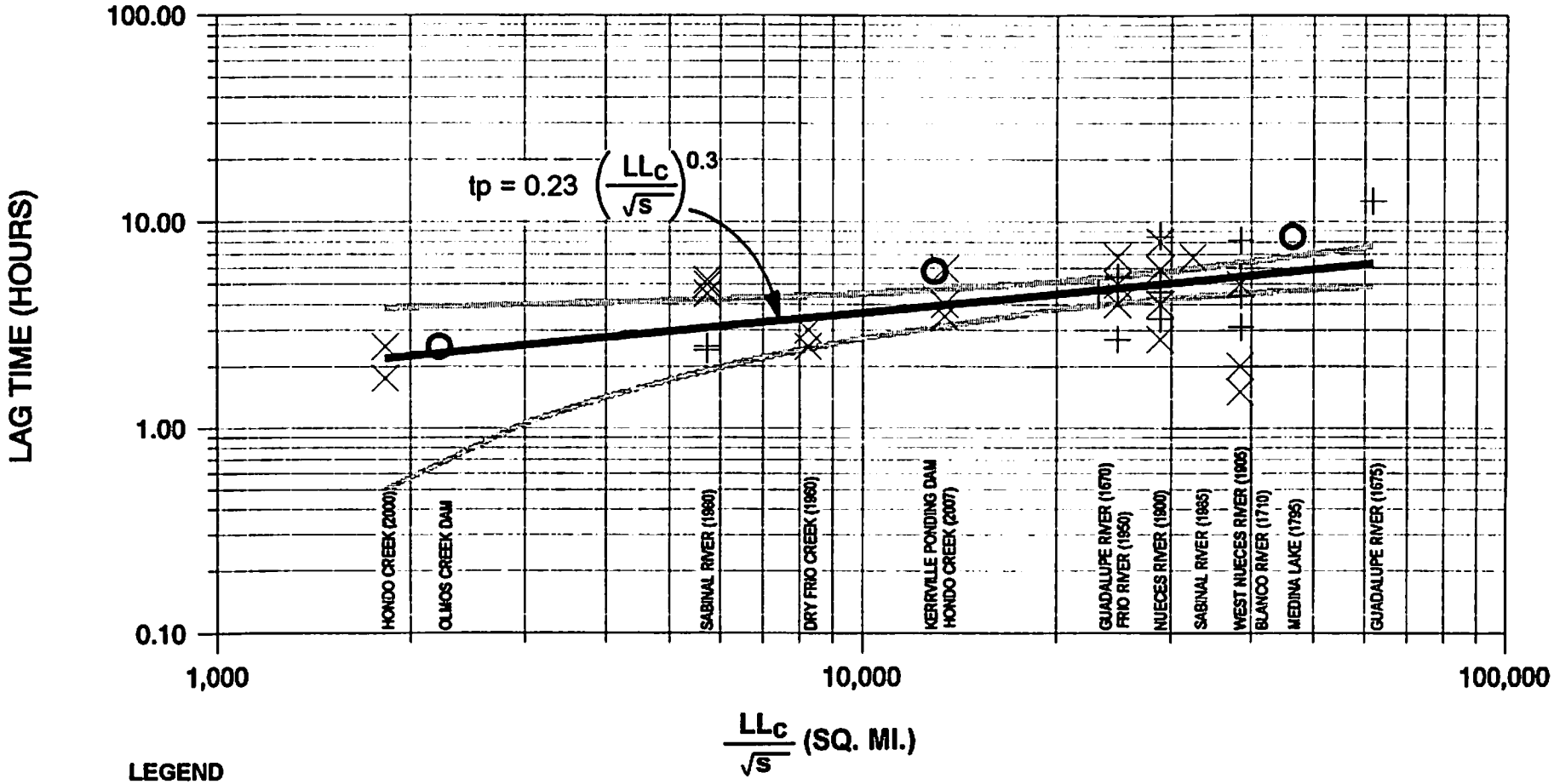
The length (L), length to centroid (L_c), and average watershed slope (s) were computed for each of the gaged watersheds used in the calibrations. A total of 39 flood events (22 for Phase IVA Study and 17 from Corps of Engineers) were used to develop the regional relationship for basin lag time. Using standard linear regression techniques, C_L was found to be 0.23 (r^2=0.19). A plot of the regional relationship developed for basin lag time and the individual calibrations is shown in Figure 2.3-7. Past flood hydrology studies for dams in the region were researched. A comparison was made of the basin lag time used in the past studies with those resulting from the regional relationship. The past studies were from dam safety inspection reports conducted under the Phase I National Dam Safety Program during the period 1978 to 1981 (Refs. 30, 31, 32). Figure 2.3-7 includes a comparison of the basin lag times used in the dam safety inspection reports with the regional relationship developed. In general, the basin lag times from the regional relationship were found to be slightly lower than those used in the past dam safety investigations, which would result in the computation of higher peak discharges for design flood events.

Initial loss and uniform loss rate parameters calibrated for the individual floods were highly variable. These parameters are very sensitive to the moisture condition of the watershed antecedent to the storm event and to the volume and pattern of the storm. Large values of initial loss and uniform loss rates were found for many of the storms analyzed.

Table 2.3-4
Summary of Historic Flood Calibrations by the U.S. Army Corps of Engineers (1964)

Stream		Watershed Characteristics					Observed Data			Calibrated Parameters		
Name	Gage I.D.	Area sq.mi.	L miles	L_c miles	s ft/ft	Flood Event	P inches	Q_p cfs	V inches	t_p hours	L_I inches	L_U in/hr
Sabinal River	1980	206	35.6	10.7	0.0044	May, 1951	1.46	N/A	0.27	2.5	0.50	0.50
						May, 1954	2.07	15,800	0.44	2.4	1.20	0.30
						Jun, 1958	10.50	55,200	2.46	2.4	3.50	0.48
Frio River	1950	389	49.1	29.9	0.0035	Jun, 1932	12.59	162,000	5.75	4.8	2.80	0.45
						Jun, 1935	5.95	106,000	4.79	5.4	0.25	0.09
						Sep, 1936	4.31	119,000	1.71	2.7	0.90	0.42
Nueces River	1900	756	58.2	28.7	0.0034	Sep, 1932	4.60	N/A	1.65	8.5	1.50	0.18
						Sep, 1936	3.03	114,000	1.40	5.0	0.50	0.17
						Jul, 1939	4.72	222,000	2.18	3.4	1.30	0.17
						Sep, 1955	8.47	307,000	3.78	4.6	2.00	0.36
Blanco River	1710	355	58.6	32.0	0.0024	May, 1929	8.95	113,000	3.69	4.4	1.80	0.44
						Jun, 1936	3.92	N/A	1.01	5.3	1.00	0.46
						Sep, 1952	13.75	95,000	4.02	8.2	1.50	0.50
						Apr, 1957	4.47	62,600	1.22	3.1	2.80	0.25
						May, 1958	3.15	96,400	1.68	3.1	0.80	0.42
Guadalupe River	1670	839	52.5	25.0	0.0032	May, 1944	N/A	59,400	1.10	4.5	N/A	N/A
Guadalupe River	1675	1315	80.0	42.1	0.0030	Sep, 1936	N/A	N/A	N/A	12.6	N/A	N/A

Notes:
- L watershed length
- L_c watershed length to centroid
- s average watershed slope
- P average storm rainfall
- Q_p peak discharge
- V runoff volume
- t_p basin lag time
- L_I initial loss
- L_U uniform loss rate



LEGEND

- PHASE IVA CALIBRATIONS
- CORPS OF ENGINEERS CALIBRATIONS
- REGIONAL LAG TIME CURVE
- 90% CONFIDENCE LIMITS
- PHASE I DAM SAFETY INSPECTIONS

tp - LAGTIME (HOURS)
L - WATERSHED LENGTH (MI.)
L_c - WATERSHED LENGTH TO CENTROID (MI.)
S - AVERAGE WATERSHED SLOPE (FT/FT)

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ENHANCEMENT PROJECT - PHASE IVA

REGIONAL LAG TIME RELATIONSHIP

FIGURE 2.3 - 7

Due to the precipitation data being the least accurate element in the historical flood calibrations, the initial loss and uniform loss rate parameters provide an adjustment to the basin average rainfall data in addition to representing interception, storage, and infiltration losses. Selection of appropriate parameters for use in the computation of design flood events involves engineering judgement, considering both the calibrated parameters and design parameters typically used in the region.

2.3.4 Model Development

An HEC-1 flood hydrology model was developed for the watershed above each proposed recharge project. The individual models were developed to compute the runoff hydrographs for various design flood events including the 25-year, 50-year, 100-year, and probable maximum flood events.

Design storm events were used in the HEC-1 model to generate the corresponding runoff hydrograph for each flood event. The probable maximum storm (PMS) is used in the HEC-1 model to compute the probable maximum flood. Probable maximum precipitation (PMP), which is the basis for deriving a PMS, is defined as the greatest depth of precipitation physically possible for a given set of conditions (Ref. 15). The conditions include a given duration, area, and season. In the study area, PMP estimates are furnished by the National Oceanic and Atmospheric Administration (NOAA) in Hydrometeorological Report No. 51 (HMR No. 51, Ref. 21). This publication provides PMP estimates for various combinations of storm areas and durations which are applicable to all seasons. National Weather Service criteria for developing a PMS from PMP estimates in HMR No. 51 are specified in Hydrometeorological Report No. 52 (Ref. 18). The criteria require determination of four conditions that will produce the maximum peak discharge at a given location. These conditions are the location of the storm center, the size of the storm area, storm orientation, and the temporal arrangement of precipitation amounts. These four condition are determined using a trial-and-error procedure that has been incorporated into the computer program HMR52. The PMS, with a total duration of 72 hours, was computed for each watershed using HMR No. 51 and HMR52 and used as input to the HEC-1 model to compute the PMF for each recharge project.

In order to compute runoff hydrographs for various return interval events (i.e., 25-, 50-, and 100-year floods), rainfall amounts that correspond to each of these return interval events were modelled using HEC-1. Rainfall amounts for each storm event were obtained from National Weather Service TP-40 (Ref. 24) and HYDRO-35 (Ref. 23). These values were used in HEC-1 to develop 24-hour duration design storms for determining runoff hydrographs for the corresponding return interval flood events. The storm rainfall was distributed using the "balanced storm" procedure in HEC-1, which creates a triangular shaped hyetograph from the given rainfall depths. Areal rainfall reduction factors were used in the model to reduce the point rainfall amounts from TP-40 and HYDRO-35 to an average depth for the larger watersheds. HEC-1 reduces the point rainfall amounts according to recommendations in TP-40. A 24-hour rainfall depth summary for each recharge project is provided in Table 2.3-5.

The unit hydrograph parameters required by the HEC-1 model for Snyder's method include the basin lag time (t_p) and peaking coefficient (C_p). The peaking coefficient was set to 0.80, the maximum value effectively allowed in HEC-1, in order to simulate the rapid rise of the runoff hydrographs typical of the region. The basin lag time for the watershed of each recharge project was determined using the regional relationship (Figure 2.3-7) derived from the historical flood calibrations expressed as:

$$t_p = 0.23 \left(\frac{L\ L_c}{\sqrt{s}} \right)^{0.3}$$

The watershed length (L), length to centroid (L_c), and average slope (s) were computed for each project to determine the lag time using the above equation.

Table 2.3-5 Design Storm Summary				
		24-Hour Storm Totals[1]		
Recharge Project	Watershed Area (sq.mi.)	25-year Storm Rainfall (inches)	100-year Storm Rainfall (inches)	Probable Maximum Storm Rainfall (inches)[2]
Lower Verde	96.0	7.02	8.94	35.35
Lower Hondo	138.5	6.85	8.72	34.45
Lower Sabinal	215.7	6.68	8.52	32.03
Lower Frio	425.8	6.56	8.35	28.90
Lower Dry Frio	173.5	6.61	8.41	31.65
Indian Creek	1801.7	6.45	8.18	15.17

Notes:
[1]24-hour storm totals include the application of areal rainfall reduction factors.
[2]72-hour storm used to compute the PMF. Maximum basin average 24-hour storm total listed for comparison purposes.

The initial and uniform loss rate function was used in HEC-1 to represent precipitation losses. The initial loss and uniform loss rate parameters were selected based on engineering judgement considering the results of the historical flood calibrations and values typically used for design storms in the region. Selection of these parameters depends on the flood event being analyzed. For the probable maximum flood, hydrologic parameters are used which would maximize runoff in the watershed. Saturated watershed conditions are typically assumed to simulate the PMF. For lesser flood events, parameters are generally selected which represent average or normal runoff conditions in the watershed. For the Verde Creek, Hondo Creek, Sabinal River, Frio River, and Dry Frio River watersheds, the values selected for the initial loss and uniform loss rate for use in computing runoff hydrographs for flood events less than the PMF (i.e. 25-year through 100-year flood events) were 2.0 inches and 0.20 inches per hour, respectively. For the PMF, the initial loss used in the model was 0.00 inches and the uniform loss rate was 0.15 inches per hour in order to represent saturated conditions. For the Nueces

River and West Nueces River watersheds, the values of initial loss and uniform loss rate selected for flood events less than the PMF were 2.5 inches and 0.20 inches per hour, respectively. The values selected to simulate the PMF were an initial loss of 0.00 inches and a uniform loss rate of 0.15 inches per hour. The slightly higher initial loss for flood events less than the PMF for the Nueces River and West Nueces River watersheds reflects the significant coverage of the Edwards Aquifer recharge zone in these watersheds and a normally drier antecedent moisture condition in the western watersheds. Table 2.3-6 provides a summary of the unit hydrograph and initial and uniform loss rate parameters used in the flood hydrology models for each recharge project.

Two USGS streamflow gaging stations exist in the watershed for the Indian Creek Project, with another gaging station located just below the project near Uvalde. The total watershed area (1801.7 sq.mi.) for the project was subdivided using the location of the gaging stations, where historical flood calibrations were performed, as a reference. Subwatershed 1 represents the West Nueces River watershed (682.1 sq.mi.) at the Bracketville gaging station (Station 1905). Subwatershed 2 encompasses the intervening drainage area (218.3 sq.mi.) for the West Nueces River between the Bracketville gaging station and the Indian Creek Project site. Subwatershed 3 represents the Nueces River watershed (756.1 sq.mi.) at the Laguna gaging station (Station 1900). Subwatershed 4 includes the intervening drainage area (145.2 sq.mi.) for the Nueces River and Indian Creek between the Laguna gage and the Indian Creek Project site. Channel routing was performed in the HEC-1 model to translate the runoff hydrographs at the Bracketville and Laguna gages to the Indian Creek site. The channel routing parameters for the Muskingum method were calibrated utilizing historical hydrographs at each of the gaging stations.

Table 2.3-6
Summary of Flood Hydrology Model Parameters

Recharge Project	Watershed Characteristics				Unit Hydrograph Parameters		Initial and Uniform Loss Rate Parameters			
							Flood Events Less than PMF		PMF	
	A sq.mi.	L miles	L_c miles	s ft/ft	t_p hours	C_p	L_I inches	L_U in/hr	L_I inches	L_U in/hr
Lower Verde	96.2	25.2	15.5	0.0049	3.1	0.8	2.0	0.20	0.0	0.15
Lower Hondo	138.5	25.4	14.2	0.0034	3.2	0.8	2.0	0.20	0.0	0.15
Lower Sabinal	215.7	38.9	22.0	0.0041	4.0	0.8	2.0	0.20	0.0	0.15
Lower Frio	425.8	61.9	39.7	0.0032	5.7	0.8	2.0	0.20	0.0	0.15
Lower Dry Frio	173.5	45.4	26.7	0.0043	4.4	0.8	2.0	0.20	0.0	0.15
Indian Creek										
Subwatershed 1	682.1	62.5	34.7	0.0032	5.5	0.8	2.5	0.20	0.0	0.15
Subwatershed 2	218.3	37.9	23.3	0.0025	4.3	0.8	2.5	0.20	0.0	0.15
Subwatershed 3	756.1	58.2	28.7	0.0034	5.0	0.8	2.5	0.20	0.0	0.15
Subwatershed 4	145.2	21.6	9.8	0.0042	2.6	0.8	2.5	0.20	0.0	0.15

Notes:

A	watershed area	t_p	basin lag time
L	watershed length	C_p	peaking coefficient
L_c	watershed length to centroid	L_I	initial loss
s	average watershed slope	L_U	uniform loss rate

2.3.5 Model Results

Execution of the HEC-1 flood hydrology models provided the necessary data to determine the dam height and spillway requirements for each recharge project. The results are in the form of a runoff hydrograph for each simulated storm event which serves as inflow to the reservoir. A summary of the peak discharge and total runoff volume for the 25-year, 100-year, and probable maximum flood events is provided in Table 2.3-7, along with the maximum recorded historical flood event for each stream. Based on the HEC-1 model results, the Lower Dry Frio, Lower Frio, and Indian Creek project sites have experienced flood events (during the period of record) in excess of the 100-year return interval event, while the Lower Hondo and Lower Sabinal projects have experienced flood events of about a 25-year return interval magnitude.

2.3.6 Flood Frequency Comparison

A comparison was made of the HEC-1 model results for flood frequency with the results that would be obtained by performing a statistical analysis of the annual flood peaks for the streamflow gaging stations. The Corps of Engineers computer program "HEC-FFA" (Ref. 17) was utilized to compute a flood frequency curve using the recorded annual flood peaks at the gaging station. HEC-FFA employs methods outlined by the Water Resources Council - Bulletin 17B, "Guidelines for Determining Flood Flow Frequencies" (Ref. 38). An HEC-1 model was developed for the watershed above the gaging stations in order to compute peak flows at the gage location and compare them with the HEC-FFA results.

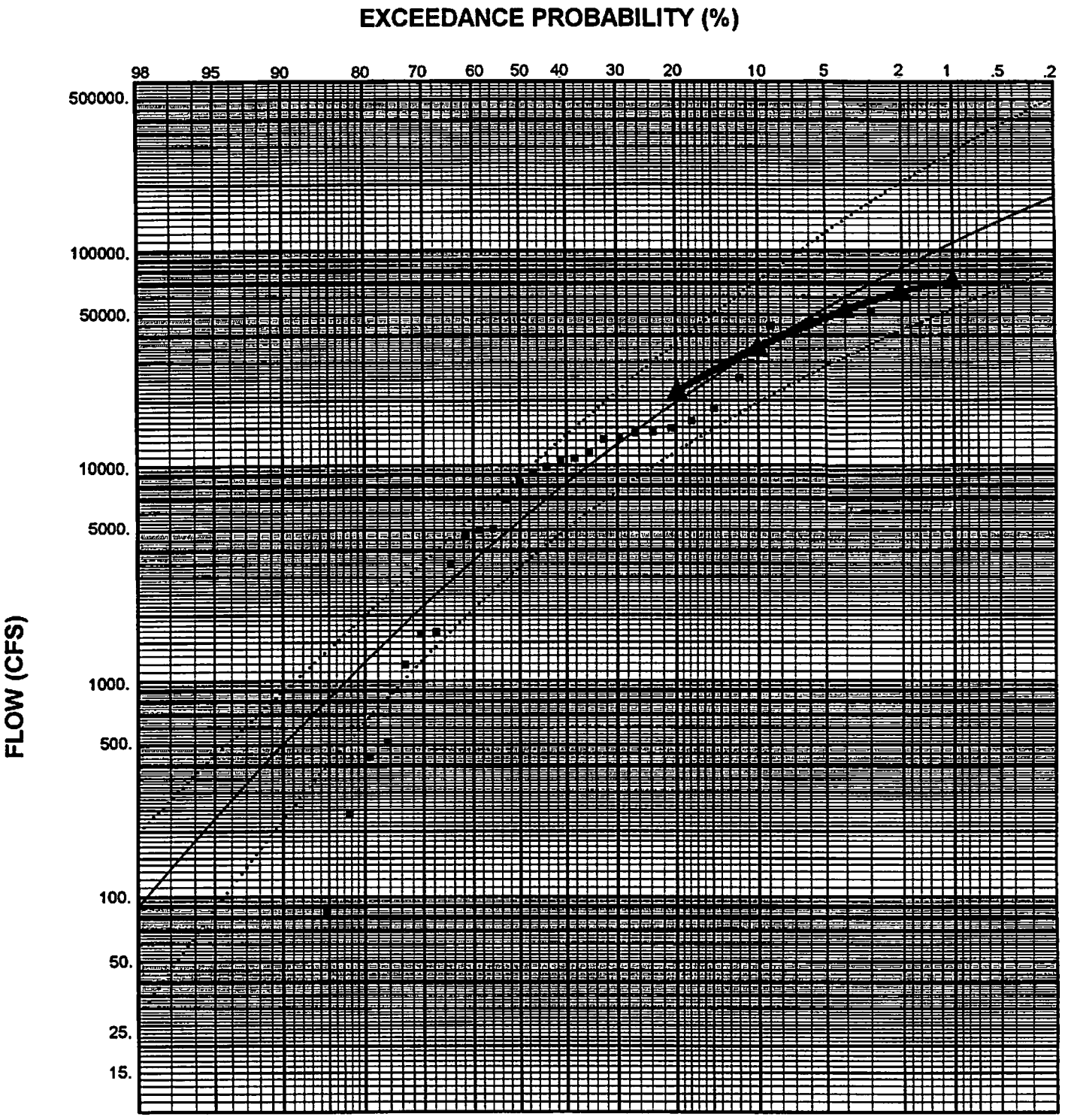
Figure 2.3-8 shows a comparison of the HEC-1 model and HEC-FFA results for Hondo Creek streamflow gaging station near Hondo (USGS Gaging Station 2007). The HEC-1 model results compare well with the computed flow frequency based on the annual flood peaks. This streamflow gage has a period of record of 33 years which results in considerable uncertainty for the larger flood events. This uncertainty is reflected in the rather wide confidence limits for this analysis.

Figure 2.3-9 shows a comparison of the HEC-1 model and HEC-FFA results for the Sabinal River streamflow gaging station at Sabinal (USGS Gaging Station 1985). The HEC-1 model results are lower than the computed flow frequency based on the annual flood peaks, but

Table 2.3-7
Flood Hydrology Summary

Recharge Enhancement Project	Watershed Area (sq.mi)	25-Year Flood		100-Year Flood		PMF		Historical Records			
		24-hr Rainfall (inches)	Peak Flow (cfs)	24-hr Rainfall (inches)	Peak Flow (cfs)	24-hr Rainfall (inches)	Peak Flow (cfs)	Maximum Peak Flow (cfs)	Year	Station and Watershed Area	Period of Record (years)
Lower Verde	96.0	7.02	46,800	8.94	65,600	35.35	291,700	N/A	N/A	N/A	N/A
Lower Hondo	138.5	6.85	62,000	8.72	87,000	34.45	396,400	69,800	1958	Sta 2000 96 sq.mi	37
Lower Dry Frio	173.5	6.61	54,300	8.41	74,200	31.65	370,900	123,000	1966	Sta 1960 126 sq.mi	37
Lower Sabinal	215.7	6.68	74,000	8.52	107,300	32.03	486,300	73,300	1958	Sta 1985 241 sq.mi	37
Lower Frio	425.8	6.56	104,000	8.35	153,100	28.90	671,600	162,000	1932	Sta 1950 389 sq.mi	67
Indian Creek	1801.7	6.45	205,100	8.18	320,900	15.17	1,353,200	616,000	1935	Sta 1920 1861 sq. mi	62

Note: 1) 72-hour storm used to compute the PMF. Maximum basin 24-hour storm total listed for comparison purposes.



NOTE: FLOOD FREQUENCY COMPARISON MADE FOR HONDO CREEK AT KING WATERHOLE NEAR HONDO, USGS STREAMFLOW GAGING STATION 08200700, WATERSHED AREA = 149 SQ. MI.

LEGEND

- HEC-1 MODEL RESULTS
- FLOW FREQUENCY (HECFFA)
- 5% AND 95% CONFIDENCE LIMITS (HECFFA)
- WEIBULL PLOTTING POSITIONS

HDR

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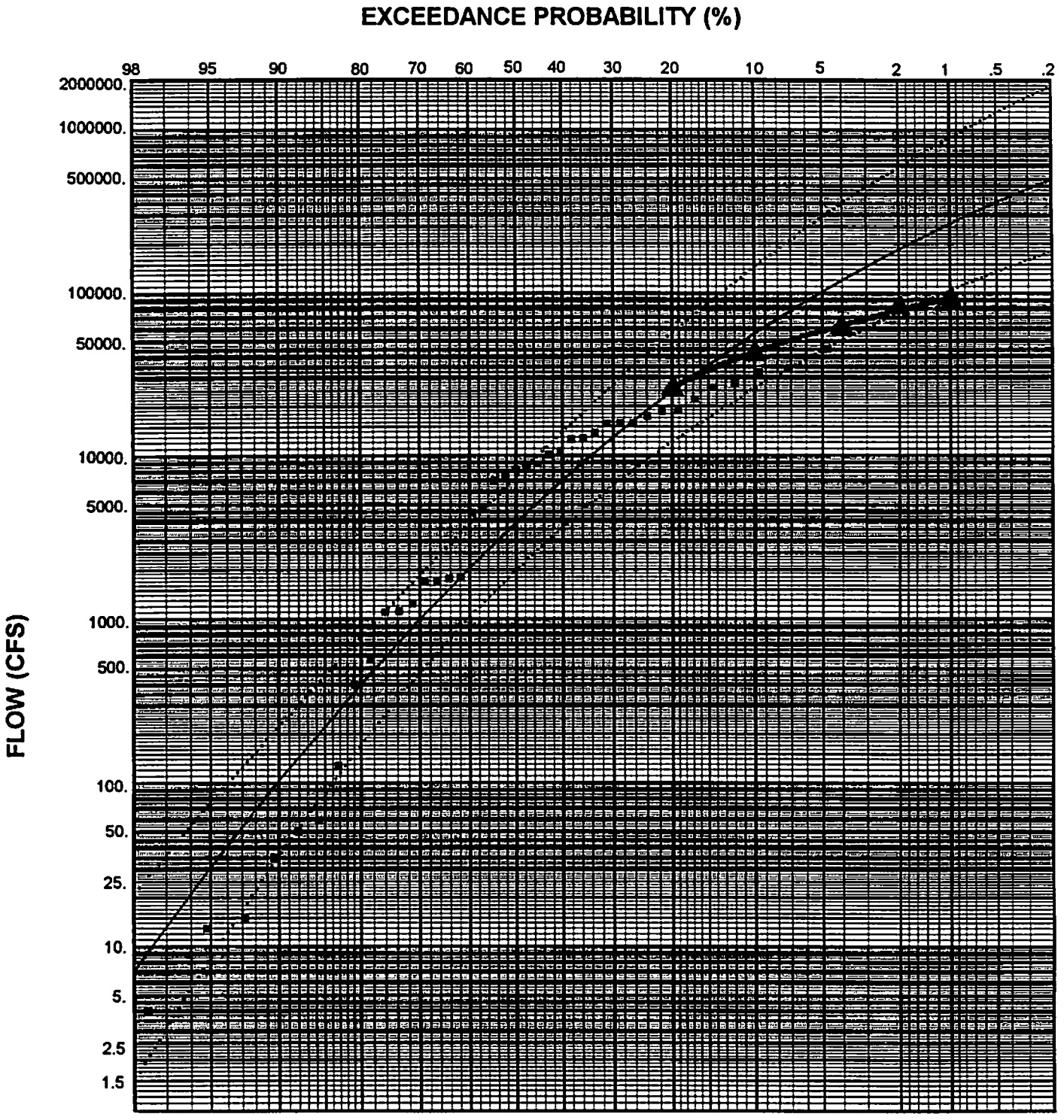
NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

FLOOD FREQUENCY COMPARISON
HONDO CREEK

FIGURE 2.3 - 8



NOTE: FLOOD FREQUENCY COMPARISON MADE FOR SABINAL RIVER AT SABINAL, USGS STREAMFLOW GAGING STATION 08198500, WATERSHED AREA = 241 SQ. MI.

LEGEND

- HEC-1 MODEL RESULTS
- FLOW FREQUENCY (HECFFA)
- 5% AND 95% CONFIDENCE LIMITS (HECFFA)
- WEIBULL PLOTTING POSITIONS

HDR

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NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

FLOOD FREQUENCY COMPARISON
SABINAL RIVER

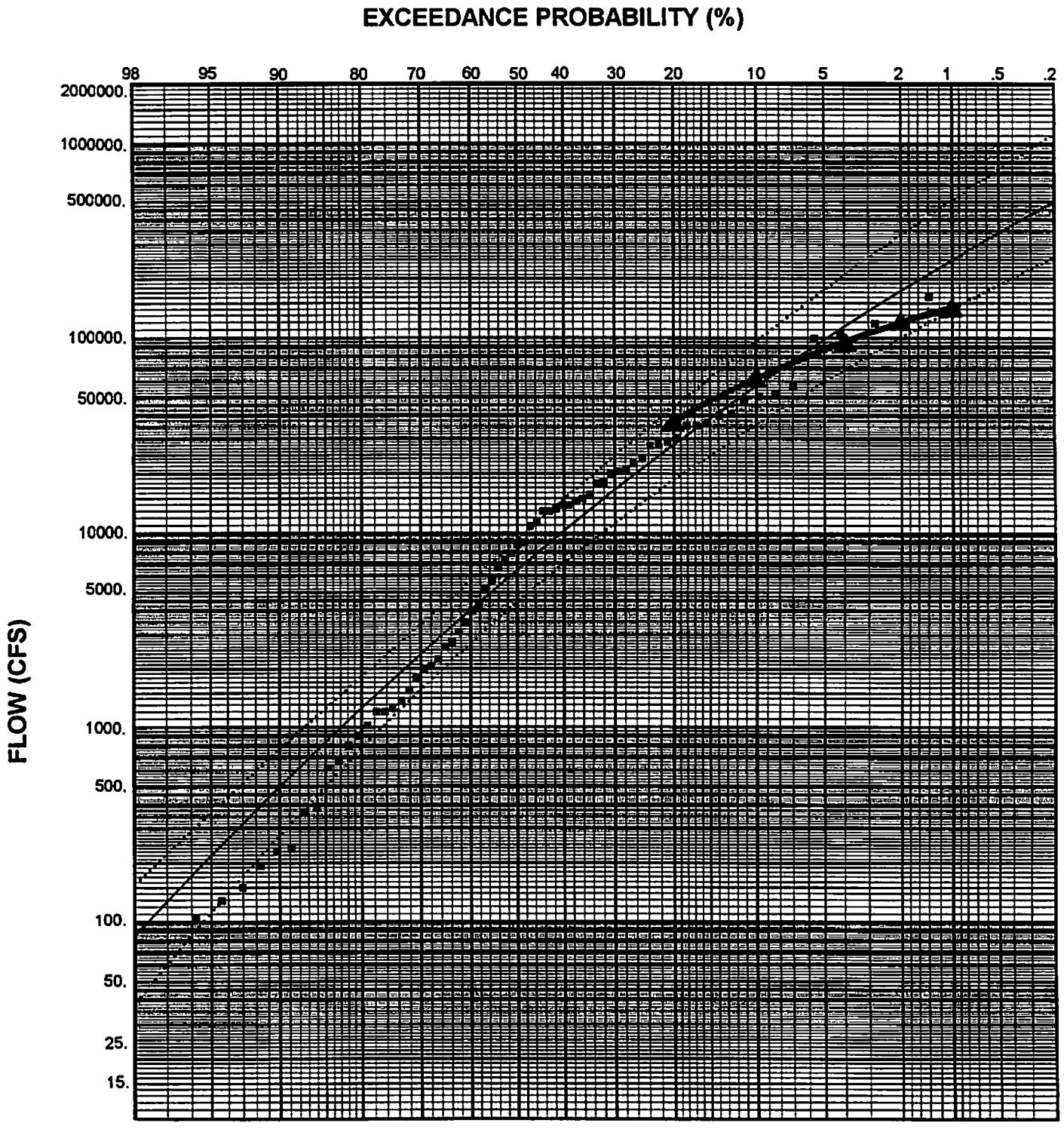
FIGURE 2.3 - 9

within the 90 percent confidence limits shown. However, the HEC-1 model results do correlate well with the data trend shown by the Weibull plotting position points.

Figure 2.3-10 provides a comparison of the HEC-1 model and HEC-FFA results for the Frio River streamflow gaging station at Concan (USGS Gaging Station 1950). The HEC-1 model results compare well with the computed flow frequency curve based on the annual flood peaks, especially for the more frequent flood events, and within the 90 percent confidence limits for the less frequent flood events. Similar to the Sabinal River analysis, the HEC-1 model results correlate well with the data trend shown by the Weibull plotting position points.

Figure 2.3-11 shows the comparison of the HEC-1 model and HEC-FFA results for the Dry Frio River streamflow gaging station near Reagan Wells (USGS Gaging Station 1960). The HEC-1 model results generally track lower than the computed flow frequency curve based on the annual flood peaks, but still within the 90 percent confidence limits of the computed curve. The computed flow frequency curve is heavily influenced by the single extreme flood event that occurred in 1966 producing a peak discharge of 123,000 cfs, almost five times greater than any other recorded flood event. With the exception of the 1966 flood event, the HEC-1 model results do appear to correlate well with the data trend, although slightly higher, as shown by the Weibull plotting position points.

Figure 2.3-12 provides a comparison of the HEC-1 model and HEC-FFA results for the Nueces River streamflow gaging station below Uvalde (USGS Gaging Station 1920). This gaging station is representative of the flow conditions at the proposed Indian Creek Project. The HEC-1 model results generally track lower than the computed flow frequency curve. The computed HEC-1 model results are within the 90 percent confidence limits, except for the 100-year return interval event (1 percent exceedance frequency) which is slightly lower. Similar to the Dry Frio River frequency analysis, the computed flow frequency curve using the annual flood peaks is heavily influenced by a single extreme flood event (616,000 cfs) that occurred in 1935. This event is three times larger than any other known event at this location. With the exception of the 1935 flood event, the HEC-1 model results compare very well with the data trend as shown by the Weibull plotting position points for the 66 year period of record at this gage.



NOTE: FLOOD FREQUENCY COMPARISON MADE FOR FRIO RIVER AT CONCAN, USGS STREAMFLOW GAGING STATION 08195000, WATERSHED AREA = 389 SQ. MI.

LEGEND

- HEC-1 MODEL RESULTS
- FLOW FREQUENCY (HECFFA)
- 5% AND 95% CONFIDENCE LIMITS (HECFFA)
- WEIBULL PLOTTING POSITIONS

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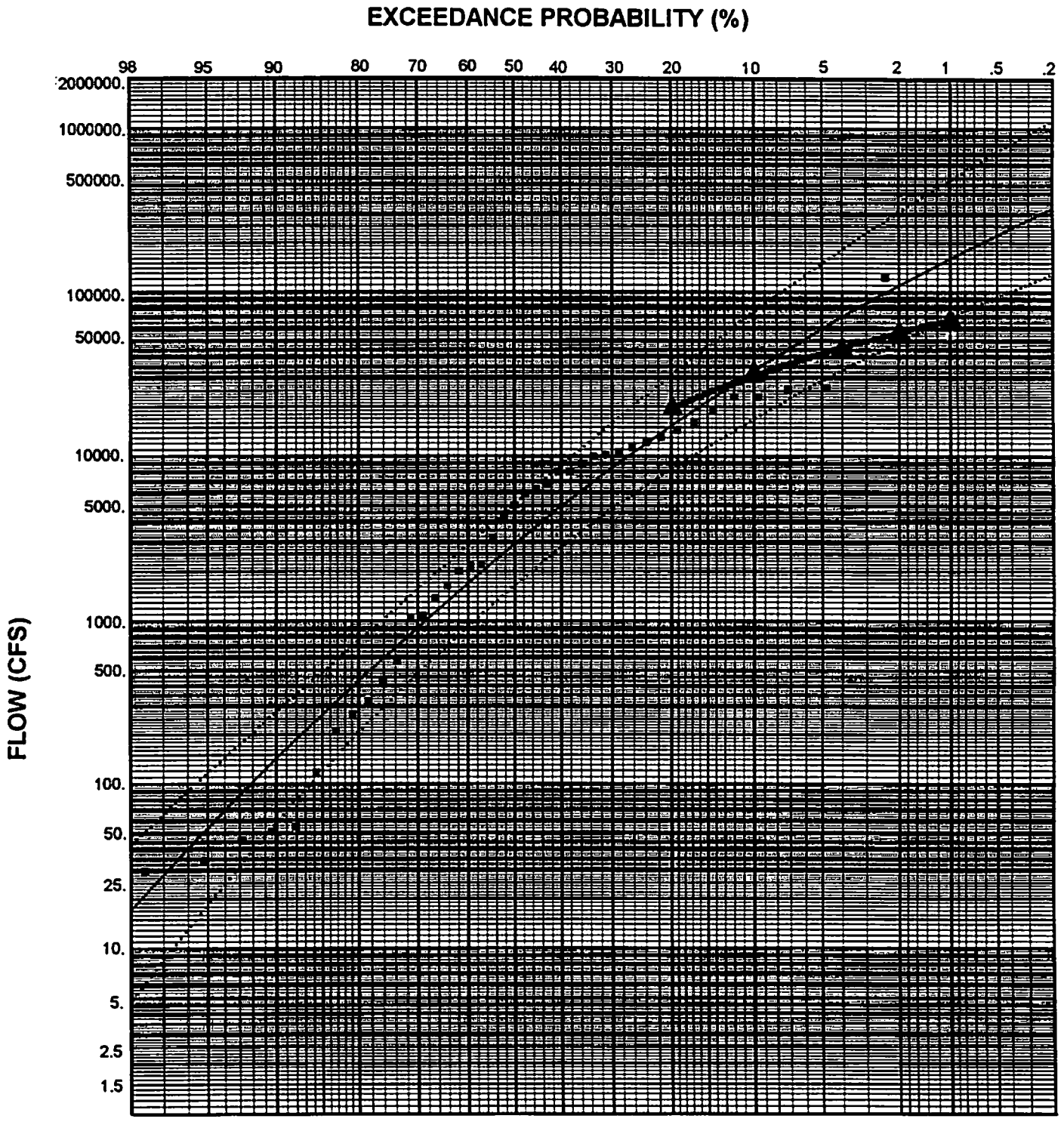
NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

FLOOD FREQUENCY COMPARISON
FRIO RIVER

FIGURE 2.3 - 10



NOTE: FLOOD FREQUENCY COMPARISON MADE FOR DRY FRIO RIVER NEAR REAGEN WELLS, USGS STREAMFLOW GAGING STATION 08196000, WATERSHED AREA = 126 SQ. MI.

LEGEND

- HEC-1 MODEL RESULTS
- FLOW FREQUENCY (HECFFA)
- 5% AND 95% CONFIDENCE LIMITS (HECFFA)
- WEIBULL PLOTTING POSITIONS

HDR

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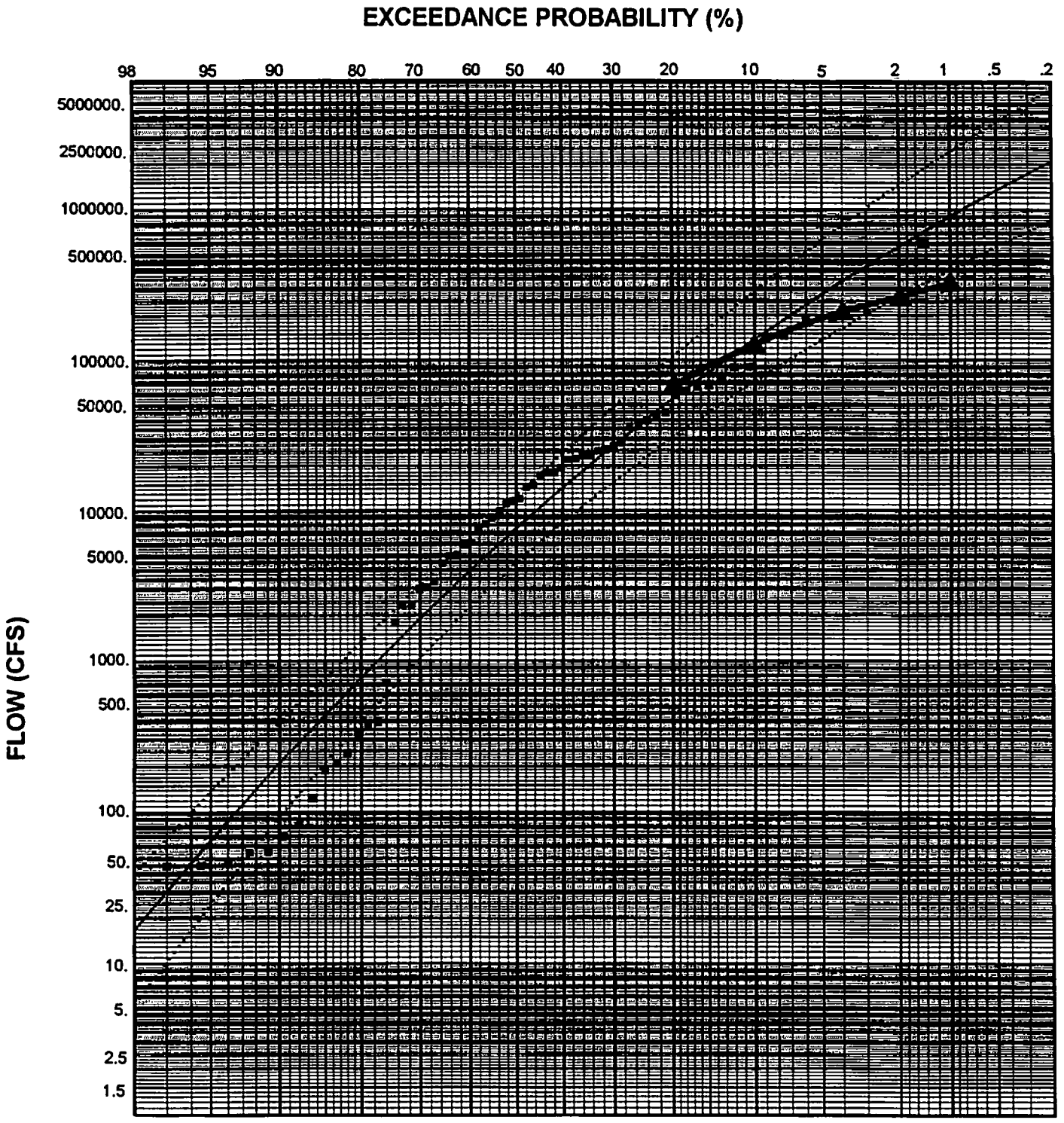
NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

FLOOD FREQUENCY COMPARISON
DRY FRIO RIVER

FIGURE 2.3 - 11



NOTE: FLOOD FREQUENCY COMPARISON MADE FOR NUECES RIVER BELOW UVALDE, USGS STREAMFLOW GAGING STATION 08192000, WATERSHED AREA = 1,861 SQ. MI.

LEGEND

- HEC-1 MODEL RESULTS
- FLOW FREQUENCY (HECFFA)
- 5% AND 95% CONFIDENCE LIMITS (HECFFA)
- WEIBULL PLOTTING POSITIONS

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HDR

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FLOOD FREQUENCY COMPARISON
NUECES RIVER

FIGURE 2.3 - 12

In summary, the peak discharges for the various return interval flood events computed using the HEC-1 models appear to correlate well with the available historical data at each of the streamflow gaging stations. This trend provides confidence that the HEC-1 models produce reasonable results for computing the magnitude of the design flood events for use in determining dam and spillway configurations, as well as land acquisition and road relocation requirements.

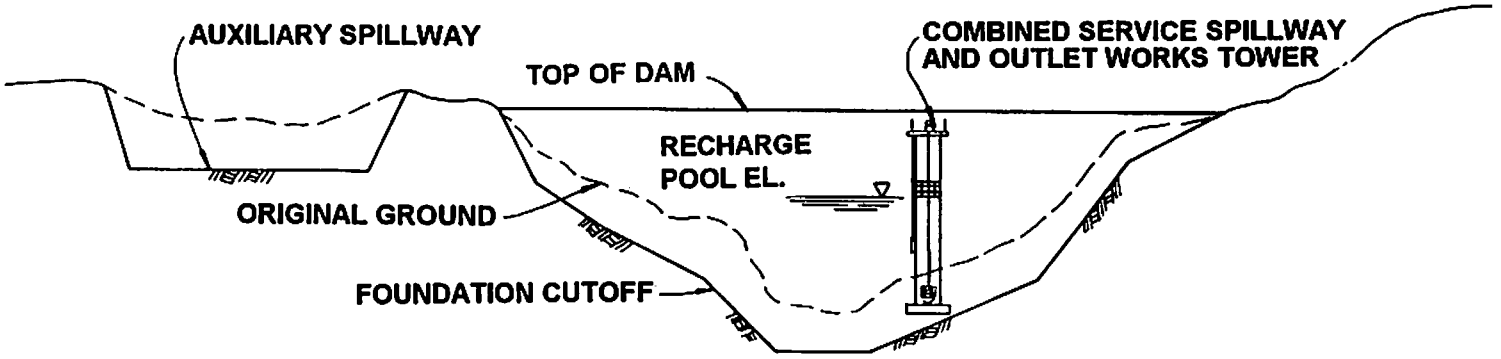
2.4 Project Feasibility Designs and Cost Estimates

2.4.1 Dam, Spillway, and Appurtenant Works

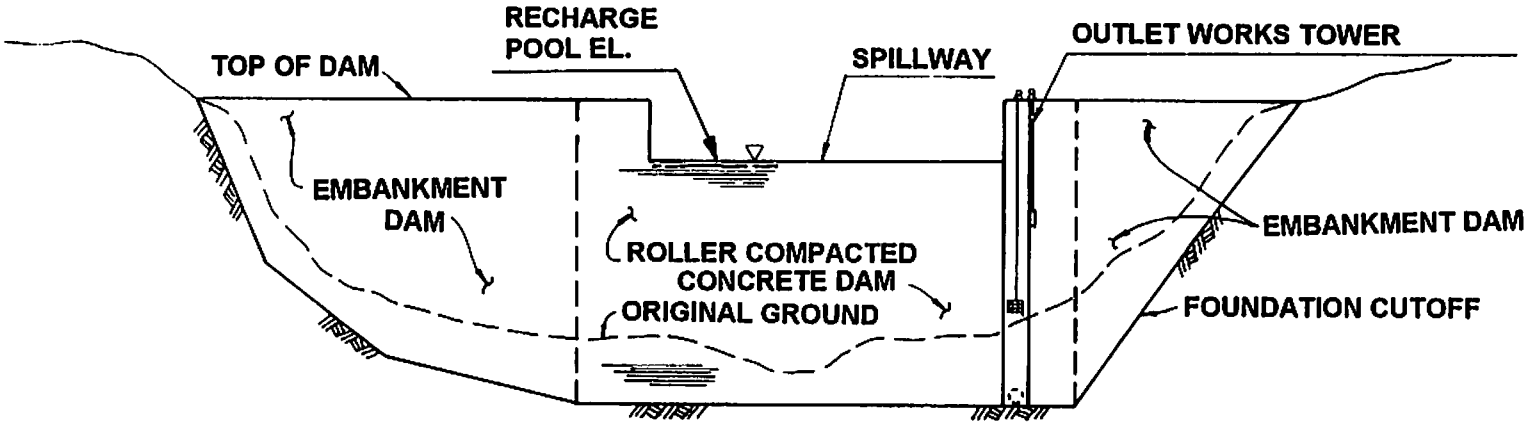
The dam and spillway configuration options considered for the recharge projects in the Phase III Study were determined to be appropriate for this study based on information gathered during the site visits. These options include: 1) an embankment dam with a relatively thin, central-clay core, rockfill shells, and a side-channel rock cut auxiliary spillway (see Figures 2.4-1 and 2.4-2); and 2) a composite dam consisting of a roller compacted concrete (RCC) overflow section connected to each abutment with embankment dams as previously described (see Figures 2.4-1 and 2.4-3).

The selection and conceptual design of these options are based on three key observations/assumptions regarding the project sites: 1) availability of clayey material for use in a dam core appears to be limited and of marginal quality; 2) an apparent abundance of material suitable for use in constructing the shells of the embankment dam; and 3) foundation strengths appear to be adequate to support an RCC gravity dam and/or the relatively steep slopes of a rockfill dam.

Several observations made during the site reconnaissance necessitated changes to certain dam design criteria and unit costs used in the Phase III Study. Sand size material for producing filter zones, RCC, and conventional concrete, appear to be limited at each site, with the exception of the Lower Hondo site. Sand may need to be crushed from limestones, processed by screening and washing local terrace deposits, or purchased from an off-site commercial source. The unit cost for the filter zone in the embankment dam and for RCC were increased to account for the apparent limited availability of sand. Terrace deposits at the dam sites were noted to be more extensive than previously assumed in the Phase III Study; therefore, additional excavation will be required to reach competent rock for the RCC gravity dam foundation and for the cutoff trench in the embankment dam. Highly fractured limestone with dissolutioning along joints and fractures was observed at each of the sites. This translates to the potential for high foundation treatment costs and the need for extensive blanket grouting in the dam foundation. Costs for these items have been included in this study.



EMBANKMENT DAM WITH SIDE CHANNEL AUXILIARY SPILLWAY



COMPOSITE EMBANKKMENT/ROLLER COMPACTED CONCRETE DAM WITH OVERFLOW SPILLWAY

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

DAM PROFILES

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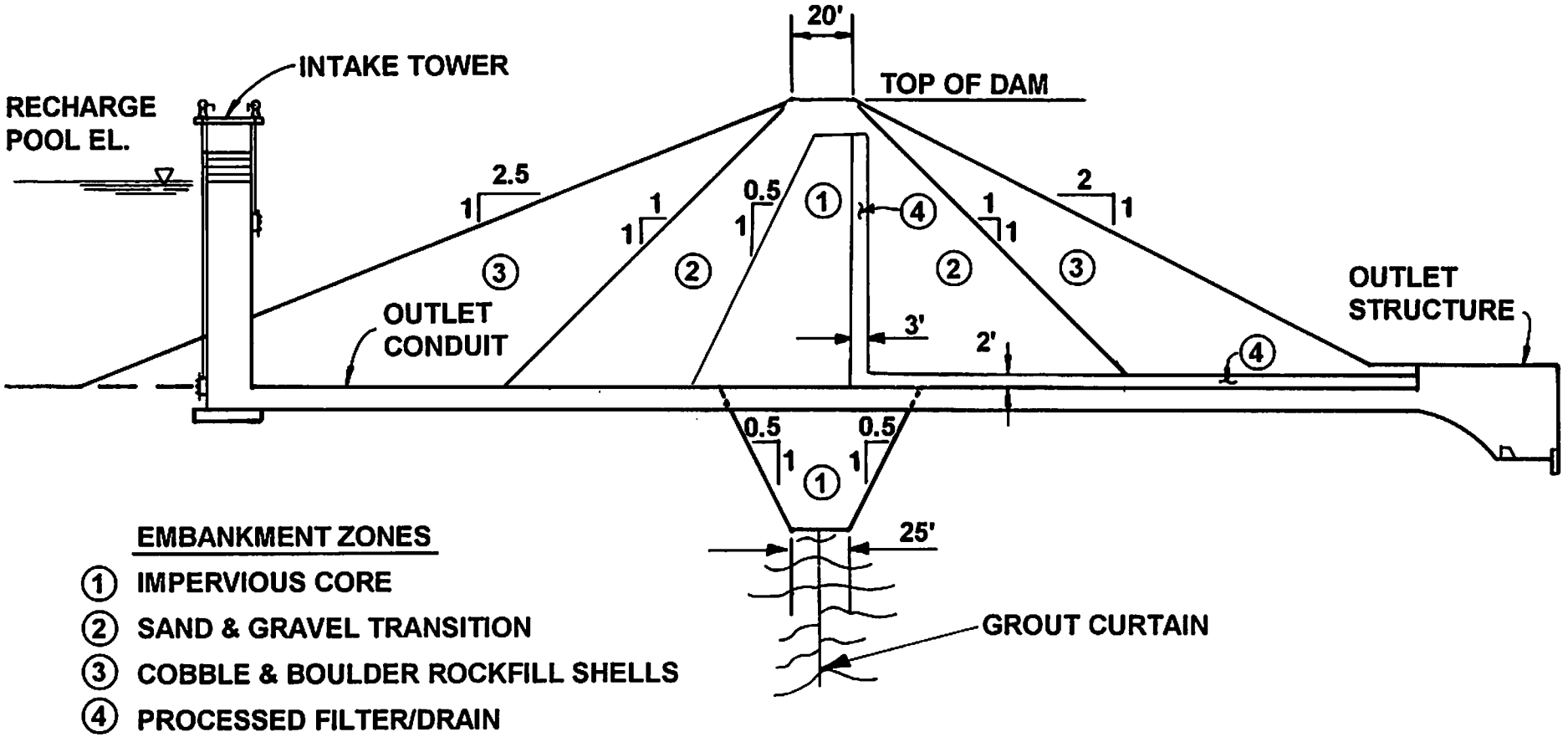
FIGURE 2.4 - 1



NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HDR

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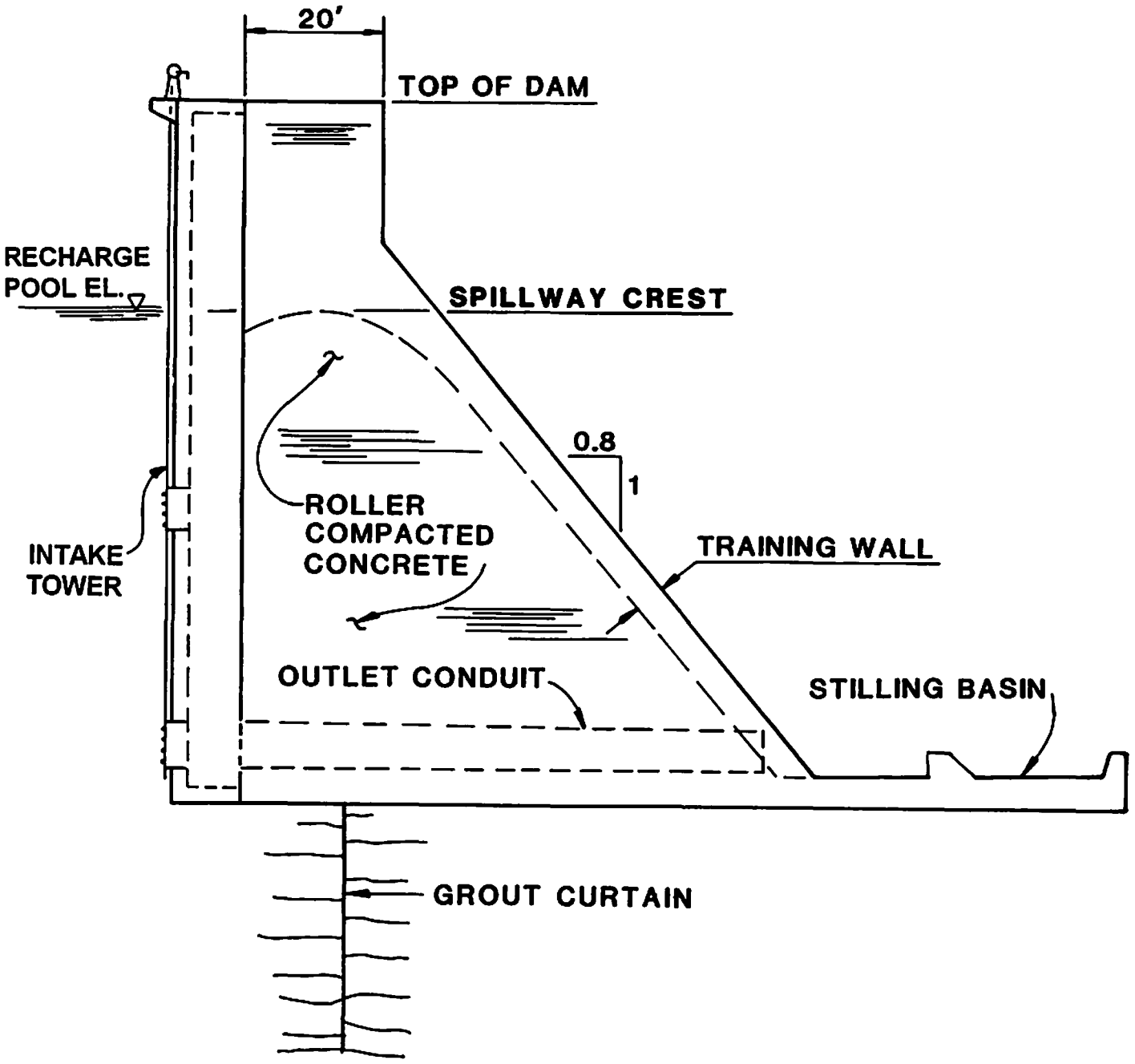
TYPICAL EMBANKMENT DAM SECTION

FIGURE 2.4 - 2



NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

HDR

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TYPICAL ROLLER COMPACTED CONCRETE DAM SECTION

FIGURE 2.4 - 3

For the composite dam alternative, the crest elevation of the RCC overflow spillway was set at the recharge pool elevation. Properly designed and constructed RCC can withstand frequent overtopping flows without jeopardizing the structural integrity of the spillway. For the embankment dam alternative, the side- channel rock cut auxiliary spillway was generally set five feet above the recharge pool elevation. In some instances, the elevation was raised to provide a better balance between the required spillway excavation quantity and the granular materials needed for the embankment dam shells. Depending on the integrity of the natural materials in which this type of spillway is excavated, it is typically desirable to minimize the frequency of flows through the spillway to reduce the potential for erosion damage. Because of the higher crest elevation and hydraulic inefficiencies relative to the RCC overflow section, a higher dam crest elevation is needed for the embankment option to safely pass the probable maximum flood (PMF) without overtopping.

Spillway widths were generally selected to limit the depth of flow to less than 30 feet during the PMF. For the embankment dam, the spillway width was also adjusted to provide a reasonable balance between the required spillway excavation and the amount of material required to construct the dam shells. The dam crest elevation was established, using the PMF, assuming the reservoir would be at the recharge pool elevation at the beginning of the flood.

A combined service spillway and low-flow outlet works was incorporated into each dam design. For the embankment dam alternative, the outlet works would consist of a concrete intake tower near the upstream toe of the dam, a conduit passing through the base of the dam, and an energy dissipation structure at the discharge end of the conduit as shown in Figure 2.4-2. For the composite dams, the concrete intake tower would be cast into the vertical upstream face of the RCC section as indicated in Figure 2.4-3. Flow would discharge from the conduit directly onto the spillway stilling basin, eliminating the need for a separate energy dissipation structure. The intake tower in either case would include a low-flow gate and an uncontrolled overflow crest at the recharge pool elevation. The top of the intake tower was assumed to be at approximately the 100-year flood level for the embankment dam option and at the top of the RCC gravity dam for the composite dam option. Outlet conduits were sized to pass downstream water rights releases as described in Section 2.2.4. Conduit diameters ranged from 48 inches to 108 inches.

Computer spreadsheets were developed for both dam types to rapidly calculate material quantities and construction costs for different reservoir sizes and auxiliary spillway widths. The spreadsheets utilize the average end area method to calculate construction material quantities, given the surveyed dam centerline profile and a top of dam elevation determined from the PMF routing analyses for each reservoir size and spillway width. Unit cost data used in this study are presented in Table 2.4-1.

Table 2.4-1
Unit Cost Data

Item	Unit	Unit Cost ($)
Impervious Clay Core/Cutoff	CY	3.00
Sand & Gravel Transitions (Fine Random)	CY	2.00
Rockfill Shells (Coarse Random)	CY	4.00
Processed Filter/Drain	CY	20.00
Foundation Excavation[1]	CY	2.00 to 3.00
Reinforced Concrete - Towers	CY	400.00
Reinforced Concrete - Walls	CY	300.00
Reinforced Concrete - Slabs	CY	160.00
Roller Compacted Concrete	CY	50.00
Grouting	LF	30.00
Intake Tower Gates	LS	52,500
Highway Relocations		
Flat Terrain	LF	125.00
Rolling Terrain	LF	175.00
Mountainous Terrain	LF	225.00
Bridge Deck (40' Wide)	LF	1,600.00
County/Private Road Relocations		
Paved	LF	50.00
Gravel	LF	50.00

[1]Unit cost varies depending on relative proportions of soil versus rock excavation.

During the course of this study, it became apparent that the Indian Creek Project needed to be evaluated considering slightly different design and cost criteria. Several factors led to this conclusion: 1) the limited natural infiltration rate assumed for the Indian Creek site and the large upstream watershed results in water being impounded by the dam much more frequently than the other five projects; 2) the magnitude of the probable maximum flood is substantially greater than the other projects; and 3) construction material quantity requirements are much greater than the other projects, which would likely result in reduced unit costs for certain items. Design and cost criteria applicable to this site are discussed in Section 3.6.

2.4.2 Road Relocations

Road relocations necessitated by the development of each recharge enhancement project were determined using USGS 7.5-minute topographic maps. State and U.S. Highways were relocated above the 50-year flood level, in accordance with current Texas Department of Transportation (TxDOT) criteria. The 50-year flood pool elevation was established assuming the reservoir would be empty at the beginning of the flood, with the exception of the Indian Creek Project which was assumed to be at 50 percent of capacity. Private gravel and paved roads providing access to houses or other structural improvements were generally relocated above the recharge pool level. Road relocation costs were estimated, as necessary, for each reservoir size evaluated at a site.

Relocated highway alignments were selected to minimize cost by avoiding mountainous terrain and stream crossings whenever possible. Both highway and private road relocation costs were calculated using unit prices per linear foot based on consultation with the local offices of the TxDOT in Uvalde and Medina Counties and on recent bid tabulations for comparable work in Texas. Highway relocation costs were calculated by classifying segments of the revised alignment according to terrain. Terrain classifications and associated unit costs in dollars per linear foot are shown in Table 2.4-1. Highway bridge replacements were based on utilizing a 40-foot wide bridge deck at a cost of $40/square foot, resulting in the cost per linear foot shown in Table 2.4-1. Private road relocation costs were calculated for paved and gravel roads using the unit costs shown in Table 2.4-1.

2.4.3 Land Acquisition

Land acquisition is a significant component of capital cost for the recharge enhancement projects. For the purposes of this study, it was assumed that all periodically inundated land up to the 25-year flood level would be purchased outright and that a flood easement would be obtained for land between the 25-year and 100-year flood levels. Review of rural land prices (Refs. 7 and 8) for the area resulted in the selection of purchase and easement costs of $800 per acre and $500 per acre, respectively. An additional cost of $50,000 per unit was included for purchase of structural improvements noted on the topographic maps as being within the 100-year flood pool. The 25- and 100-year flood pool elevations were established assuming the reservoir would be empty at the beginning of the flood, with the exception of the Indian Creek Project which was assumed to be at 50 percent of capacity.

2.4.4 Environmental Mitigation

Estimated environmental mitigation costs were developed during the Phase III Study by Paul Price Associates, Inc. (PPA) for a maximum (100 percent) potential size of each project site. These costs include environmental studies and reports, archaeological work, and, if necessary, costs for habitat evaluations and acquisition and management of mitigation lands. Environmental mitigation costs for smaller storage capacities at each project were estimated by reducing the cost for the 100 percent capacity based on a ratio of recharge pool acreage at the smaller capacity to that at the 100 percent capacity. These costs were inflated 2.5 percent per year for three years to estimate the environmental mitigation costs for this study (Phase IVA). For a detailed summary of pertinent environmental considerations and a more thorough explanation of environmental mitigation costs, refer to Appendix A in the Phase III Study report (Ref. 14).

2.4.5 Water Rights Mitigation

Costs for water rights mitigation to offset downstream impacts to the CC/LCC Reservoir System, by a means other than flow releases from the projects, have been included in the project cost estimates. These costs were calculated on the basis of two components.

The first component includes payment of the cost of replacement water for the reduced yield of the CC/LCC Reservoir System. For the purposes of this study, an annual cost of $374.00 per acre-foot per year was used as compensation for any reduction in the system yield. This amount is equivalent to about $1.15 per 1,000 gallons, and is based on a 1993 estimated cost for the City of Corpus Christi to develop a comparable source of water to replace the reduced firm yield (Ref. 9). The second cost component addresses the long-term average impacts on reservoir inflows, lake levels, and inflows to the Nueces Estuary. It was assumed that all of these impacts are reflected in the change in average annual inflows to the Nueces Estuary. For each recharge project evaluated, the resulting average annual reduction in estuarine inflow was multiplied by a unit cost of $19 per acre-foot per year. This unit cost is approximately 5 percent of the unit cost of firm-yield water which is consistent with the concept of "interruptible" supply as implemented by the Lower Colorado River Authority, City of Austin, and Texas Natural Resource Conservation Commission. It is acknowledged that the selection of these cost values for mitigation of water rights impacts is arbitrary, as a mutually acceptable cost for mitigation of water rights impacts will ultimately need to be negotiated by the parties involved.

2.4.6 Miscellaneous Project Costs

Engineering, permitting, legal, financial, and other miscellaneous costs associated with project development were assumed to total 20 percent of related capital costs based on comparable reservoir projects. Project capital costs were annualized based on a 25-year finance period and an annual interest rate of 7.5 percent. Annual operations and maintenance (O&M) costs were assumed to be approximately 0.4 percent of the total capital cost of each project.

SECTION 3

LOWER VERDE

3.0 RECHARGE ENHANCEMENT PROJECT EVALUATIONS

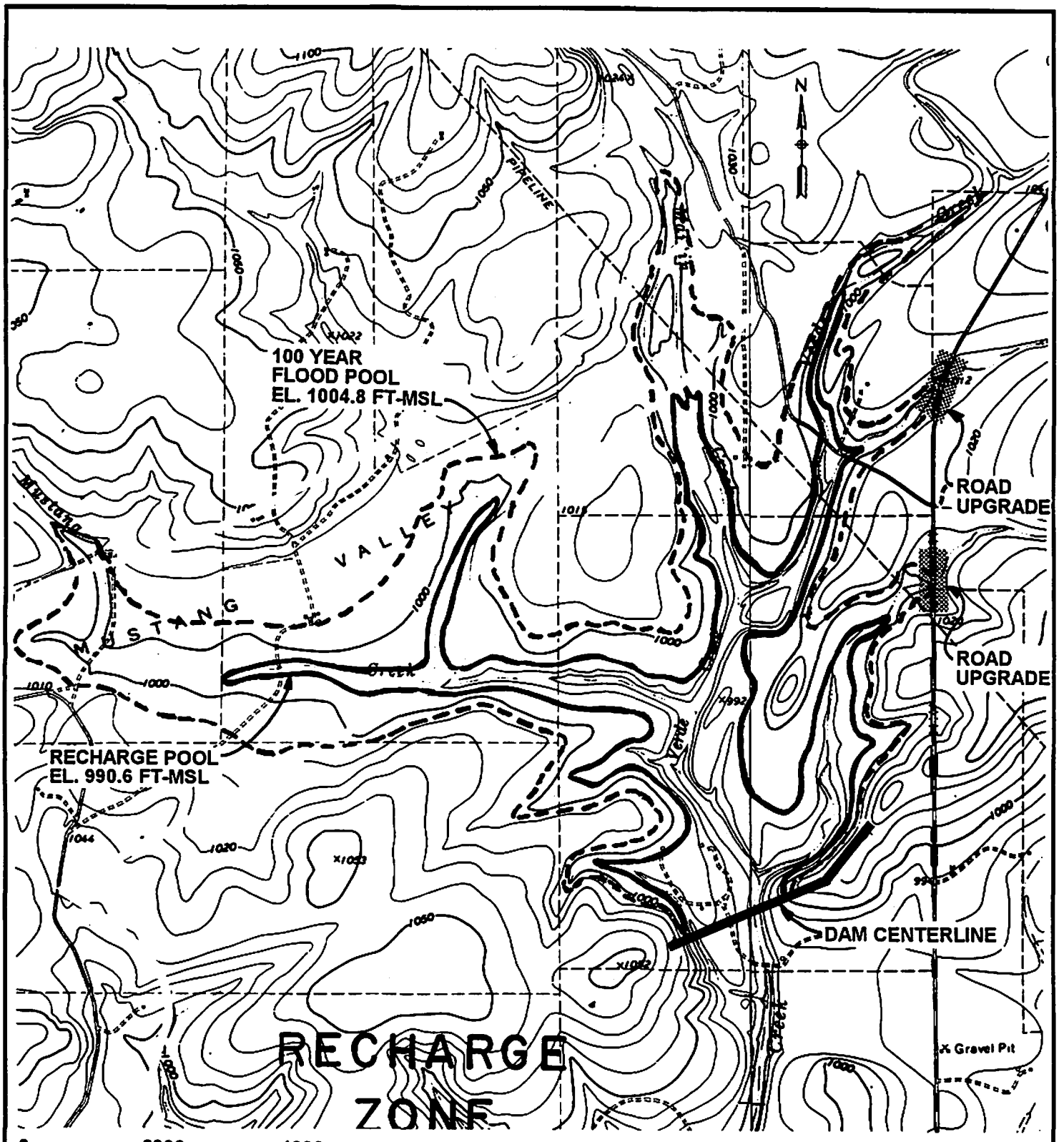
An evaluation of each of the proposed recharge enhancement projects is presented in this section. A brief description of the site characteristics gleaned from the site reconnaissance is followed by a discussion of the optimum dam and spillway configuration for each site. A project cost and data summary table delineating the reservoir sizes examined, along with a graphical evaluation summary, are included for each project. A site plan showing the dam centerline, recharge pool elevation, and limits of the 100-year flood pool is provided for the optimum size of each project. Road relocations, where necessitated by construction of the optimum size reservoir, are also shown on the site plan. A dam centerline profile is included for each project at its optimum size to illustrate the selected dam type, pertinent elevations, and the location and width of the spillway required to pass the probable maximum flood (PMF). A graph is provided for each project which summarizes annual recharge to the Edwards Aquifer, both under natural conditions (i.e., without the project) and recharge enhancement resulting from development of the optimum size reservoir. Graphs are also included for each project to illustrate the frequency of various storage levels in the reservoir relative to the optimum capacity. These graphs will be particularly beneficial when evaluating the potential for recharging imported water, assessing environmental impacts, and determining reservoir easement requirements.

3.1 Lower Verde Project

3.1.1 Site Characteristics

The Lower Verde Project is located on Verde Creek approximately nine miles north of Hondo in Medina County. The project is located in the Edwards Aquifer recharge zone downstream of the existing Middle Verde Creek recharge project, and as shown in Figure 3.1-1, it is immediately west of State Highway (SH) 173. During the site reconnaissance, the west abutment for the dam was shifted downstream to shorten the length of the dam.

The dam site is situated immediately below an area of faulted Edwards Limestone. Observations during the site reconnaissance revealed that Edwards bedrock is exposed continuously where the proposed dam crosses the creek and no surficial evidence of faulting



0 2000 4000

SCALE IN FEET

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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HDR Engineering, Inc.

LOWER VERDE PROJECT
SITE MAP

FIGURE 3.1 - 1

was noted at the dam site. No geologic or geotechnical fatal flaws were evident at the site that would prohibit development of a recharge dam (Ref. 6).

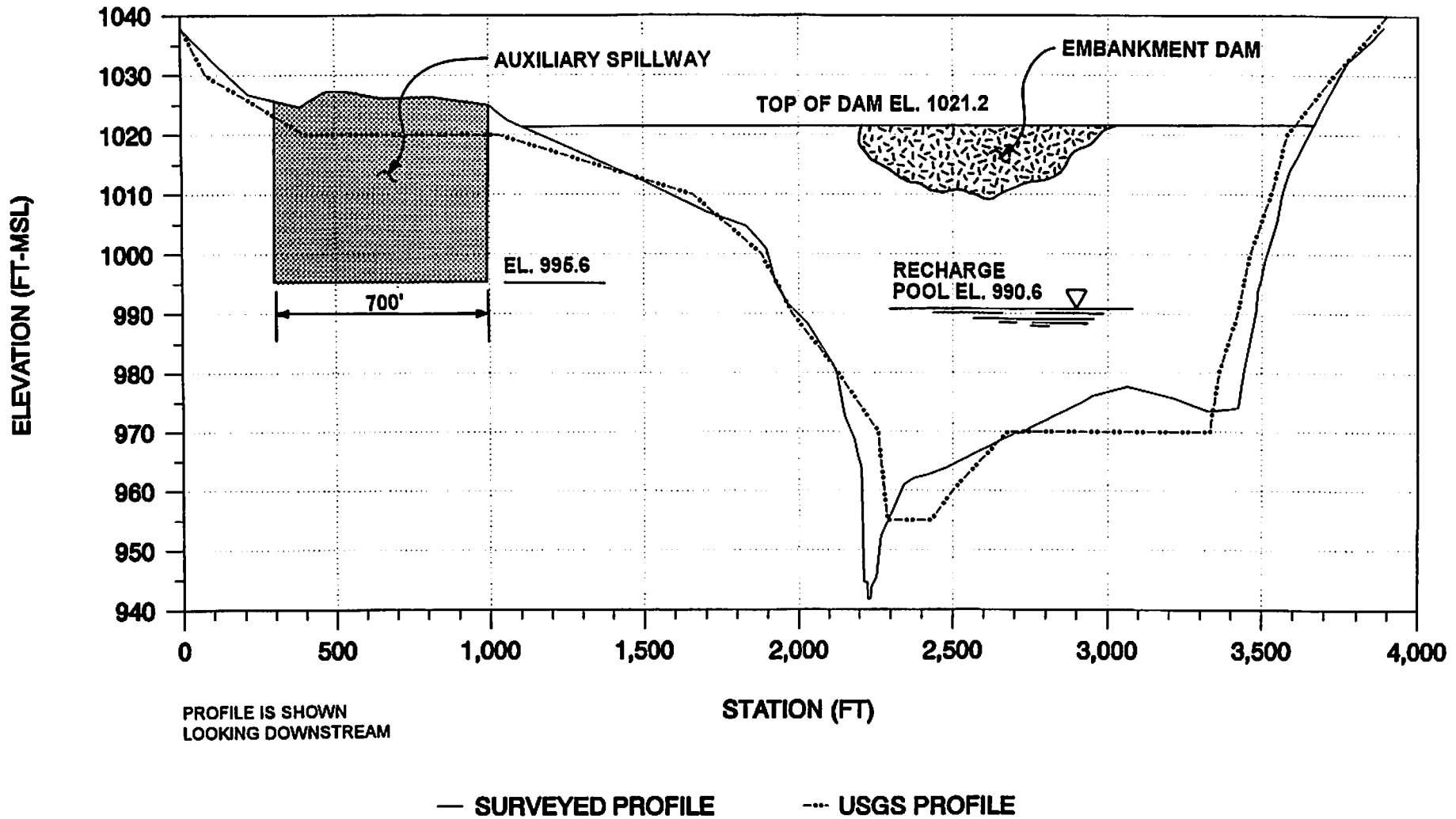
The left (east) abutment of the dam site consists of exposed Edwards with a thin, rocky soil cover. A terrace deposit consisting of clayey gravel with cobbles and boulders extends about 1,000 feet from the creek channel to the right (west) abutment. The right abutment above the terrace deposit consists of exposed Edwards with a very thin, rocky soil cover. A large terrace deposit was observed upstream of the dam along the east bank of the creek. The terrace appears to consist of at least three feet of lean clay overlying an unknown depth of clayey gravel with cobbles and boulders.

Sufficient materials exist within the immediate vicinity of the site to construct either dam type considered in this study (see Section 2.4.1). Clay of marginal quality for an impervious dam core and cutoff appears to be present in sufficient quantities in the upper portion of the terrace deposits within the dam and reservoir area. An abundant quantity of material suitable for constructing the embankment dam shells exists in the terrace deposit or in the spillway excavation required for the embankment dam option. Sand size material for producing concrete and filters is limited at the site and would need to be obtained either from processing the terrace deposits, crushing limestone, or an off-site commercial source. Rock for slope protection is plentiful and would likely be obtained from quarrying Edwards limestone or from the spillway excavation required for the embankment dam option.

3.1.2 Dam/Spillway Selection

A range of recharge pool capacities from 2,400 to 8,400 acre-feet (acft) was evaluated for the Lower Verde Project. This represents approximately 10 to 35 percent of the maximum potential storage capacity considered in the Phase III Study. A recharge reservoir capacity of 3,600 acft was determined to be the optimum size, based on the minimum unit cost of recharge enhancement under average conditions.

The embankment dam with side channel auxiliary spillway is the most cost effective dam/spillway type for this site. A 700-foot wide rock cut auxiliary spillway located in a ridge of Edwards bedrock on the left abutment would be required to safely pass the PMF, as shown in Figure 3.1-2. The crest elevation of this spillway was set five feet above the



NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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LOWER VERDE PROJECT
CENTERLINE PROFILE

FIGURE 3.1 - 2

recharge pool elevation to minimize the frequency of operation and to provide a reasonable balance between the spillway excavation quantity and the amount of shell material needed for the embankment dam. This configuration results in a dam height of about 80 feet measured from the surveyed low point of the creek. The maximum flow depth through the auxiliary spillway would be approximately nine feet during the 100-year flood and 25.5 feet during the PMF.

3.1.3 Road Relocations

As shown in Figure 3.1-1, the optimum size reservoir at the Lower Verde site would not require a major relocation of SH 173. However, drainage improvements at two tributaries that pass beneath the highway would be required to provide for uninterrupted travel during the 50-year flood event. Larger reservoirs considered at this site would necessitate a major relocation of the highway to the east of its present route. This site constraint, combined with diminishing recharge enhancement, results in higher unit costs of recharge enhancement for reservoir sizes above the optimum.

3.1.4 Project Evaluation

A summary of the project costs and physical data for the Lower Verde Project is provided in Table 3.1-1. Much of this data is also presented graphically in Figure 3.1-3. As indicated in the table and figure, the optimal site development is a reservoir capacity of 3,600 acft, which results in 4,850 acft/yr of recharge enhancement under average conditions at a unit cost of $122/acft/yr. Recharge under drought conditions would be increased by 1,719 acft/yr at a unit cost of $343/acft/yr.

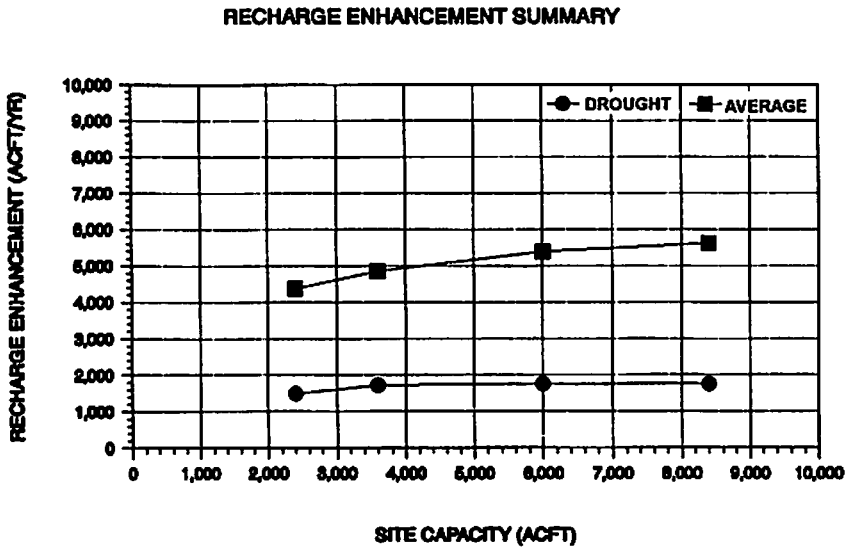
A graph illustrating the natural recharge and the recharge enhancement resulting from development of the optimum size Lower Verde Project is shown in Figure 3.1-4 for the 56-year period of record from 1934 through 1989.

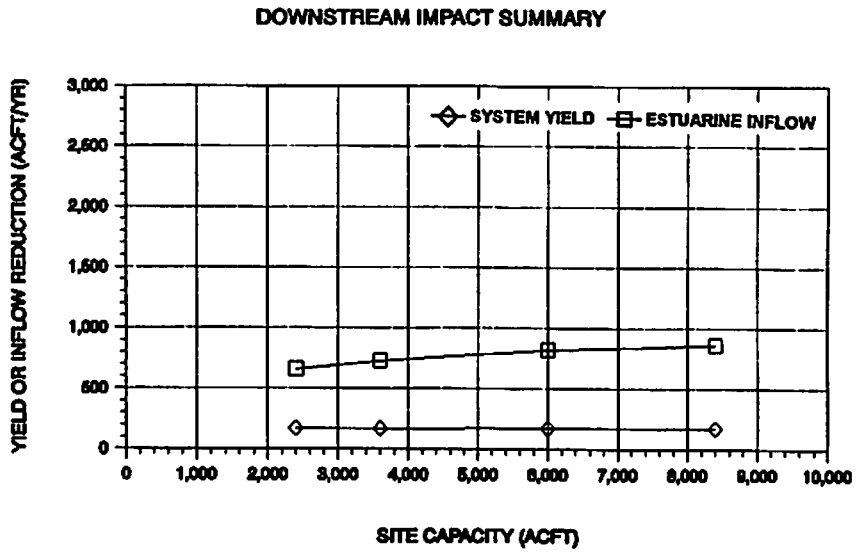
Figure 3.1-5 illustrates the typical performance of direct percolation recharge projects located within the Edwards Aquifer recharge zone. The primary purpose of these recharge projects is to store flood flows and allow the water to percolate over time through cracks and fissures into the aquifer. The figure indicates that, on the average, the reservoir would

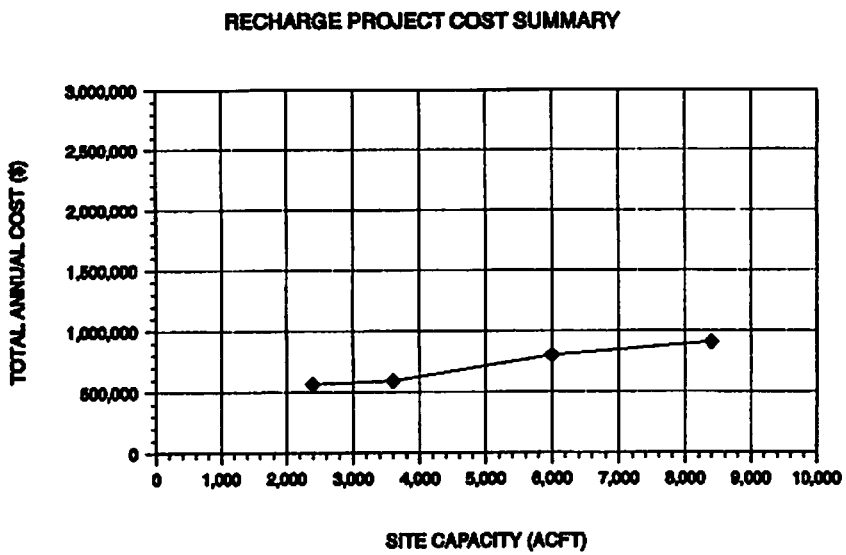
Table 3.1-1
Lower Verde Project Cost and Data Summary

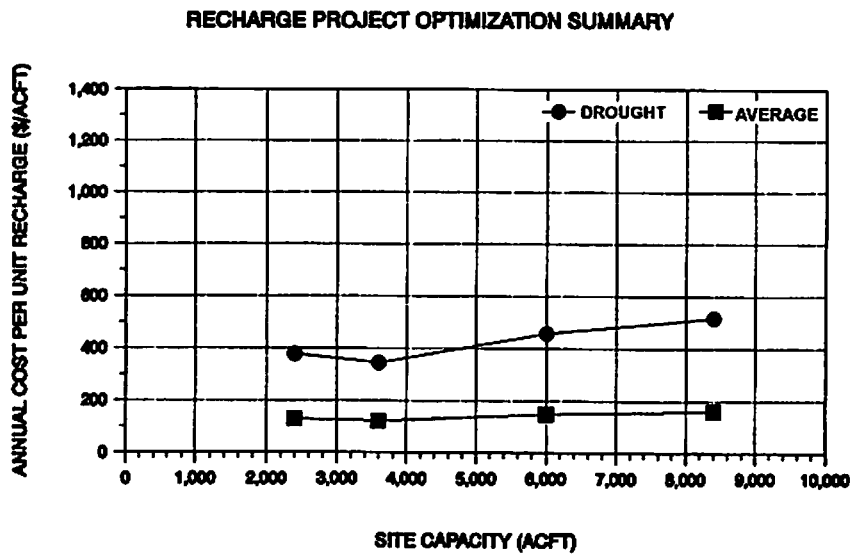
Physical Data				
Recharge Pool:				
Capacity (acft)	2,400	3,600	6,000	8,400
Surface Area (ac)	237	334	514	683
Elevation (ft-msl)	986.4	990.6	996.4	1000.4
Spillway Elevation (ft-msl)	991.4	995.6	1001.4	1005.4
Spillway Width (ft)	500	700	800	900
25-Year Flood Pool[1]:				
Elevation (ft-msl)	1000.5	1002.8	1007.1	1009.9
Surface Area (ac)	693	865	1,184	1457
50-Year Flood Pool[1]:				
Elevation (ft-msl)	1001.7	1003.9	1008.1	1010.9
100-Year Flood Pool[1]:				
Elevation (ft-msl)	1003.0	1004.8	1009.0	1011.9
Surface Area (ac)	880	1,014	1,359	1,674
Dam Type	Embankment	Embankment	Embankment	Embankment
Top of Dam Elevation (ft-msl)	1022.1	1021.2	1024.6	1026.8
Streambed Elevation (ft-msl)	941.8	941.8	941.8	941.8
Hydrologic Data				
Recharge Enhancement (acft/yr):				
Drought Conditions	1,498	1,719	1,758	1,758
Average Conditions	4,379	4,850	5,392	5,615
Median Conditions	1,804	1,813	1,825	1,831
CC/LCC System Yield Reduction (acft/yr)	170	170	171	172
Median CC/LCC System Storage Reduction (%)	0.19	0.32	0.32	0.32
Estuarine Inflow Reduction (acft/yr)	658	728	820	861
Summary of Project Costs				
Dam, Spillway, and Appurtenant Works	$2,936,225	$2,857,013	$3,184,120	$3,467,527
Road Relocations	$322,500	$510,000	$1,752,500	$2,005,000
Land Acquisition	$1,147,900	$1,266,500	$1,634,700	$2,024,100
Environmental Mitigation	$16,435	$23,161	$35,643	$47,362
Engineering, Legal, Financial, and Misc.	$884,612	$931,335	$1,321,393	$1,508,798
Total Capital Cost	$5,307,672	$5,588,009	$7,928,356	$9,052,787
Annual Capital Cost (25 years @ 7.5% interest)	$476,098	$501,244	$711,174	$812,035
Operations and Maintenance (annual)	$11,745	$11,428	$12,736	$13,870
Water Rights Mitigation (annual)	$76,082	$77,412	$79,534	$80,687
Total Annual Cost	$563,925	$590,084	$803,444	$906,592
Annual Cost / Unit Recharge Enhancement:				
Drought Conditions ($/acft/yr)	$376	$343	$457	$516
Average Conditions ($/acft/yr)	$129	$122	$149	$161

Shaded column represents optimum size of project based on lowest unit cost for average conditions.
[1]Flood pools based on reservoirs being empty at beginning of flood.
Refer to Appendix B for general notes on which project costs and data are based.









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ENHANCEMENT PROJECT - PHASE IVA

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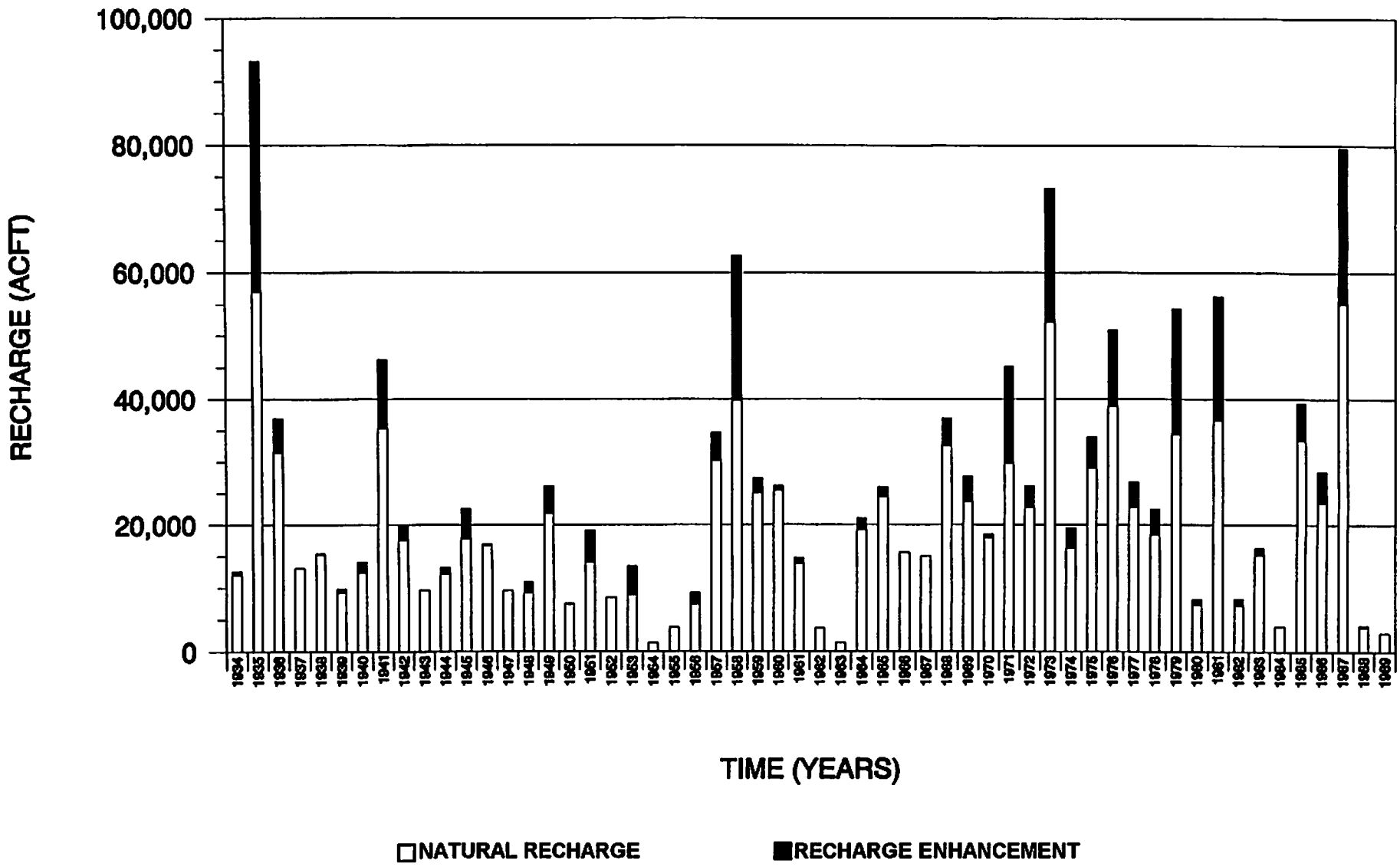
LOWER VERDE PROJECT
EVALUATION SUMMARY

FIGURE 3.1 - 3



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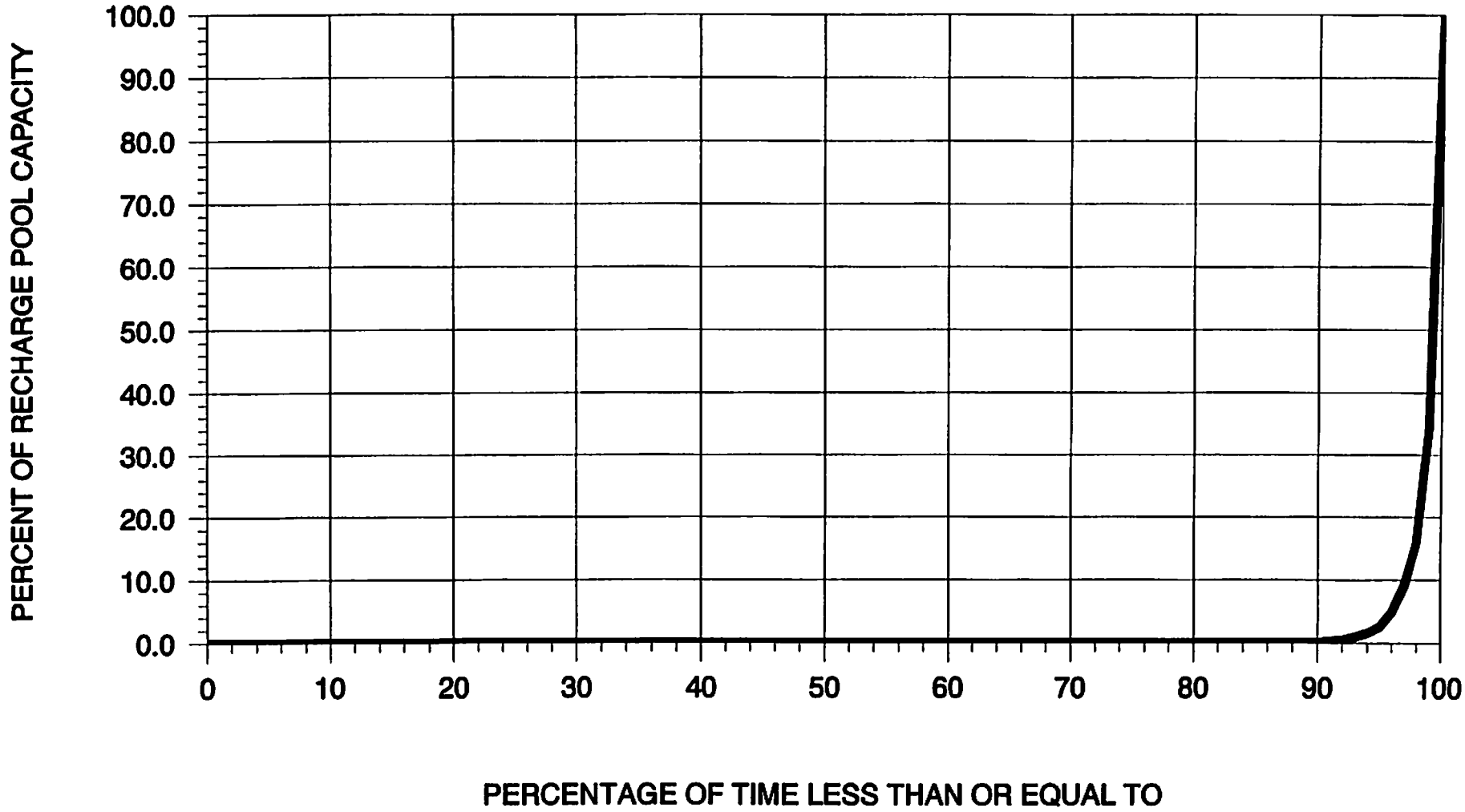
LOWER VERDE PROJECT
ANNUAL RECHARGE

FIGURE 3.1 - 4



RECHARGE POOL CAPACITY = 3,600 ACFT

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LOWER VERDE PROJECT
STORAGE FREQUENCY

FIGURE 3.1 - 5

be empty 90 percent of the time. Approximately 98 percent of the time, storage in the reservoir would be less than 50 percent of optimum capacity.

3.1.5 Importation of Water for Recharge

The recharge reservoirs considered in this study could provide terminal points for imported water from existing or potential projects. The Lower Verde Project may be a candidate site for receipt of imported water. This project, being the most eastern, makes it the closest site to potential sources of imported water from the north and east. Water discharged in the headwaters of the Verde Creek basin would have several opportunities to enter the Edwards Aquifer: 1) upstream of the existing Middle Verde Project; 2) at the existing Middle Verde Project; 3) in the recharge zone between the existing and proposed projects; 4) at the new Lower Verde Project; and 5) in the recharge zone downstream of the Lower Verde Project.

Based on the frequency of water being impounded in the Lower Verde Project, as shown in Figure 3.1-5, sufficient reservoir capacity exists 95 percent of the time to accept significant quantities of water. Based on the calculated recharge rate curve for the Lower Verde Project (see Section 2.2.5), a diversion rate of 150 cubic feet per second (300 acft/day or 100,000 acft/yr) could be recharged to the aquifer with a resulting storage volume equal to approximately 900 acft. This volume equals one-fourth of the proposed 3,600 acft capacity. A diversion rate of 150 cfs represents roughly the amount which could be delivered by a 6-foot diameter pipeline.

To maximize recharge from natural flows, importation would need to be suspended during significant flood events in the Verde Creek watershed. It is not anticipated that significant additional reservoir development costs would be required to import water to the Lower Verde Project, unless very high importation rates are considered. However, if importation is considered, several additional issues may need to be investigated. These could include: 1) water quality of imported water, considering both biological and chemical compatibility; 2) environmental impacts of more permanent reservoir pools; 3) long-term changes in recharge acceptance rates; and 4) potential design changes necessitated by more permanent reservoir pools.

LOWER HONDO

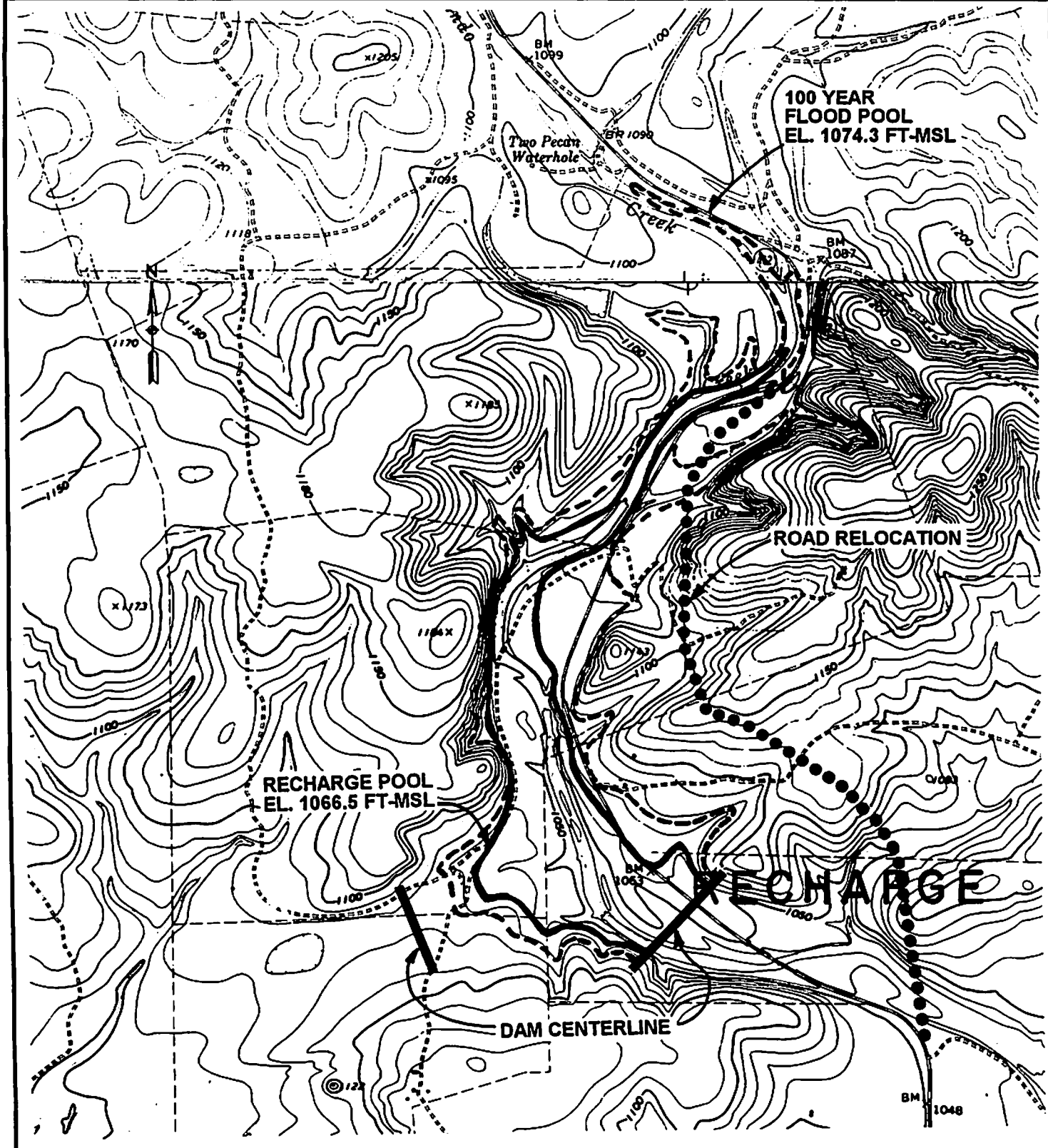
3.2 Lower Hondo Project

3.2.1 Site Characteristics

The Lower Hondo Project is located on Hondo Creek approximately 10 miles north by northwest of Hondo in Medina County. The project is located in the Edwards Aquifer recharge zone, and as shown in Figure 3.2-1, it is immediately west of State Highway (SH) 462. Access during the site reconnaissance was limited to the area east of the highway. Although the right (west) abutment was not examined during the site visit, it is speculated that Edwards Limestone underlies the entire dam site, based on geologic maps and observations of the left (east) abutment. The geology at the saddle area west of the dam is believed to consist of clayey soils underlain by the Del Rio Clay Formation. Although additional geologic mapping work will be required to completely characterize the Lower Hondo site, it is anticipated that geologic constraints related to design would be minimal at the proposed main dam and saddle dike sites (Ref. 6).

A major terrace deposit, on which SH 462 is located, extends from the creek to a point about 1,200 feet east at the base of the left abutment. The abutment above this point consists of exposed Edwards with a thin, rocky clay soil cover. What is believed to be an abandoned channel of Hondo Creek was observed near the toe of the left abutment. Exposed materials in the terrace deposit consist of a sandy silt to sandy clay overlying gravels of unknown depth. Based on observations of the creek channel at the highway crossing about 1,500 feet downstream of the dam site, the channel deposits appear to be comprised of a mixture of fractured limestone (Edwards) cobbles and boulders and sandy gravel with layers and bars of medium sand. It does not appear from the geologic and topographic maps that a major terrace deposit exists at the right abutment. It is speculated that the right abutment consists of exposed Edwards with a thin, rocky soil cover.

Sufficient materials appear to exist within the immediate vicinity of the site to construct either dam type considered in this study (see Section 2.4.1). Clay of marginal quality for an impervious dam core and cutoff appears to be present in sufficient quantities in the upper portion of the extensive terrace deposits at the dam site and in the reservoir area. An abundant quantity of material suitable for constructing the embankment dam shells exists in the terrace deposits or in the spillway excavation required for the



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SCALE IN FEET

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LOWER HONDO PROJECT
SITE MAP

FIGURE 3.2 - 1

embankment dam option. Sand and gravel materials for the production of concrete and filters would likely be obtained by washing and screening the terrace deposits along the creek channel. Rock for slope protection would likely be obtained by quarrying Edwards limestone or from the spillway excavation required for the embankment dam option.

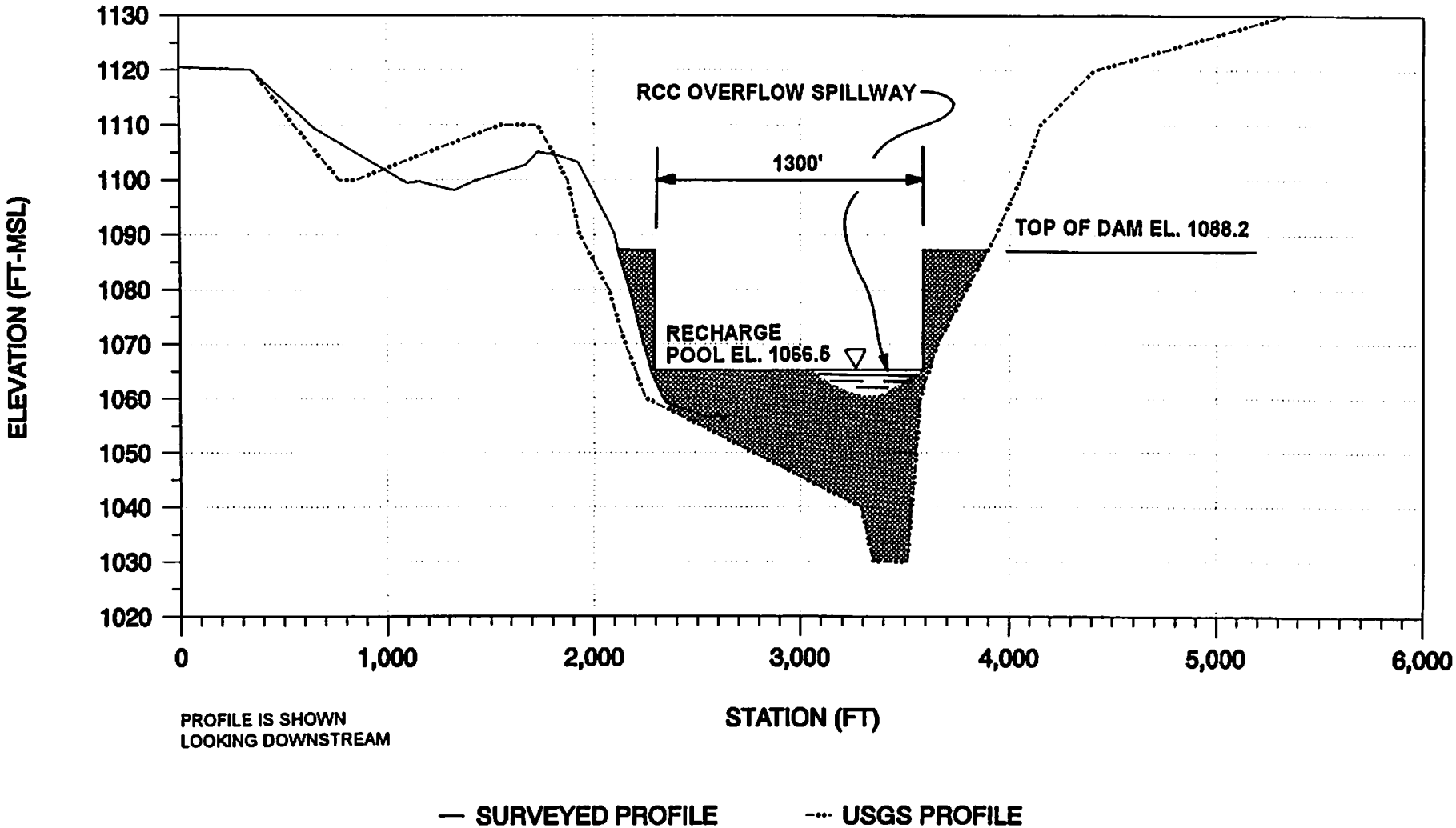
3.2.2 Dam/Spillway Selection

A range of recharge pool capacities from 1,400 to 5,600 acre-feet (acft) was evaluated for the Lower Hondo Project. This represents approximately 5 to 20 percent of the maximum potential storage capacity considered in the Phase III Study. A recharge reservoir capacity of 2,800 acft was determined to be the optimum size, based on the minimum unit cost of recharge enhancement under average conditions.

The composite dam with an RCC overflow spillway is the most cost effective dam/spillway type for this site. The main dam consists entirely of an RCC gravity overflow section, as shown in Figure 3.2-2. An embankment dam would be constructed in the west saddle area. A spillway width of 1,300 feet would be required to safely pass the PMF. This configuration results in the RCC overflow section being about 36.5 feet high measured from the low point in the creek (estimated from USGS topography as no surveying was performed west of the highway because of access restrictions). The height to the top of the dam would be about 58 feet. The maximum flow depth through the spillway would be approximately eight feet during the 100-year flood and 22 feet during the PMF.

3.2.3 Road Relocations

Because of the close proximity of SH 462 to the creek channel, virtually any size project considered for Hondo Creek necessitates a major road relocation. As shown in Figure 3.2-1, the optimum size reservoir would require relocating SH 462 to the east of the site to provide uninterrupted travel during the 50-year flood event. For larger reservoirs considered at this site, the embankment dam option was more economical than the composite dam. However, the higher 50-year flood level that results from a rock cut spillway located in the east abutment would necessitate relocating SH 462 around the west side of the reservoir. Although the embankment dam is more economical for the larger



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LOWER HONDO PROJECT
CENTERLINE PROFILE

FIGURE 3.2 - 2

sizes, the cost of relocating the road on the west side is substantially higher and more than offsets the cost savings of the dam. This site constraint, combined with diminishing recharge enhancement, results in higher unit costs of recharge enhancement for reservoir sizes above the optimum.

3.2.4 Project Evaluation

A summary of the project costs and physical data for the Lower Hondo Project is provided in Table 3.2-1. Much of this data is also presented graphically in Figure 3.2-3. As indicated in the table and figure, the optimal site development is a reservoir capacity of 2,800 acft, which results in 6,779 acft/yr of recharge enhancement under average conditions at a unit cost of $180/acft/yr. Recharge under drought conditions would be increased by 1,193 acft/yr at a unit cost of $1,022/acft/yr.

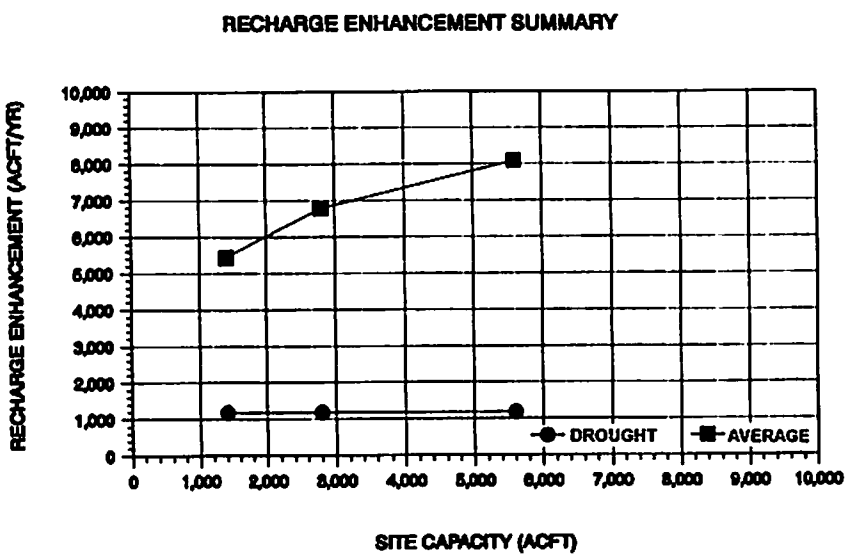
A graph illustrating the natural recharge and the recharge enhancement resulting from development of the optimum size Lower Hondo Project is shown in Figure 3.2-4 for the 56-year period of record from 1934 through 1989.

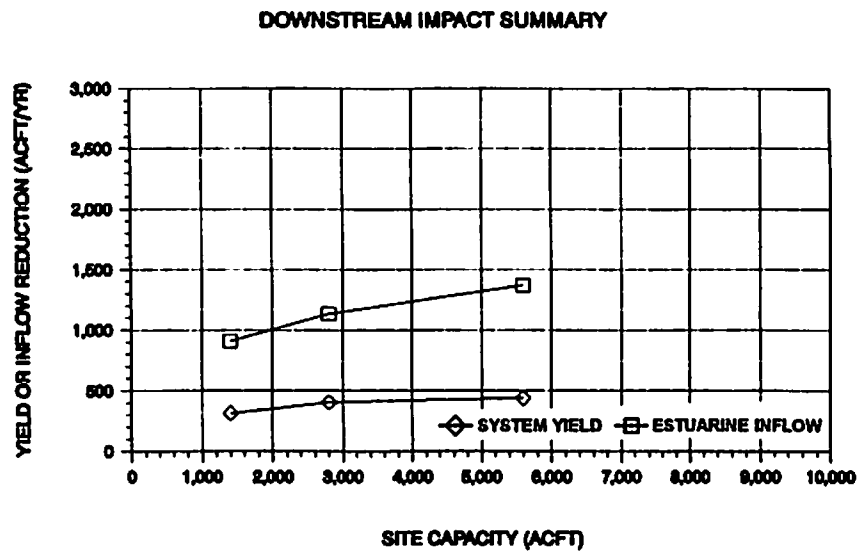
Figure 3.2-5 shows the frequency of various storage levels for the optimum size project. It indicates that, on the average, the reservoir would be empty 80 percent of the time, and approximately 98 percent of the time the reservoir storage would be less than 50 percent of its optimum capacity.

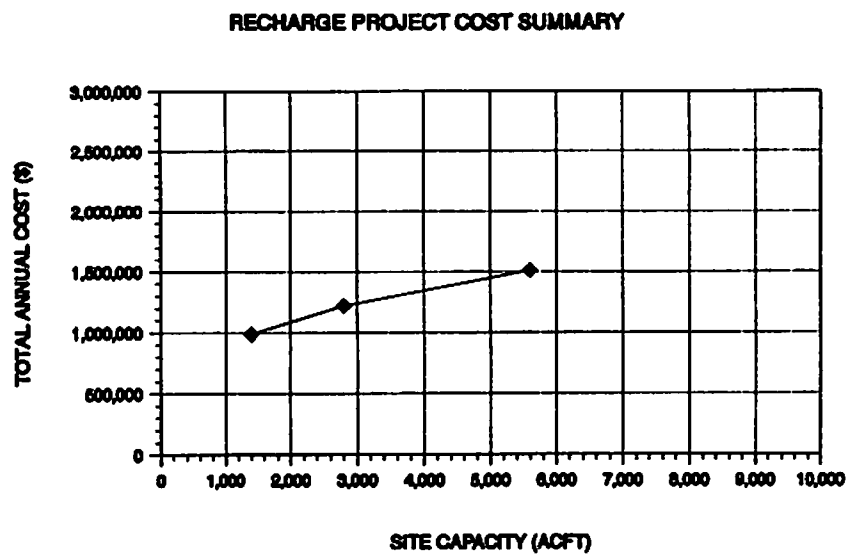
Table 3.2-1
Lower Hondo Project Cost and Data Summary

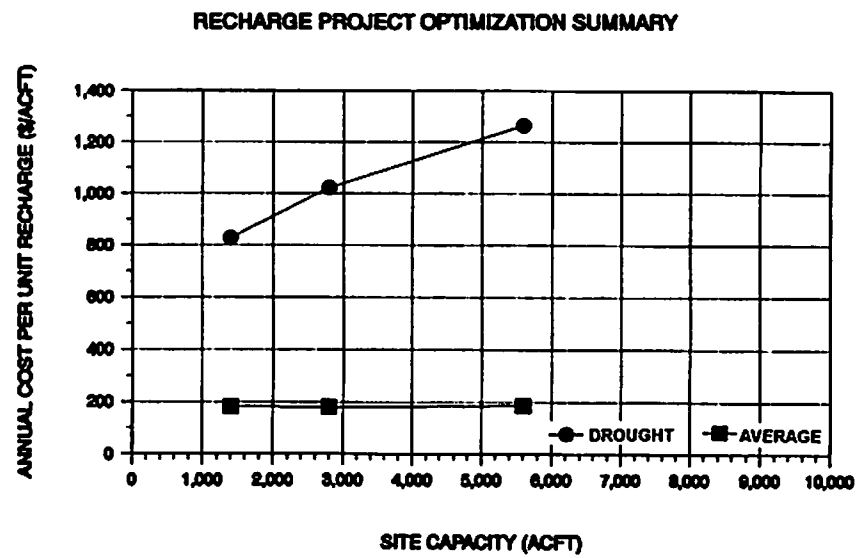
Physical Data			
Recharge Pool:			
Capacity (acft)	1,400	2,800	5,600
Surface Area (ac)	132	232	407
Elevation (ft-msl)	1058.6	1066.5	1075.4
Spillway Elevation (ft-msl)	1058.6	1066.5	1080.4
Spillway Width (ft)	1,200	1,300	1,000
25-Year Flood Pool[1]:			
Elevation (ft-msl)	1065.2	1072.7	1088.0
Surface Area (ac)	213	350	750
50-Year Flood Pool[1]:			
Elevation (ft-msl)	1066.0	1073.5	1089.0
100-Year Flood Pool[1]:			
Elevation (ft-msl)	1066.9	1074.3	1090.0
Surface Area (ac)	239	384	815
Dam Type	RCC Overflow	RCC Overflow	Embankment
Top of Dam Elevation (ft-msl)	1081.5	1088.2	1107.8
Streambed Elevation (ft-msl)	1030.0	1030.0	1030.0
Hydrologic Data			
Recharge Enhancement (acft/yr):			
Drought Conditions	1,194	1,193	1,193
Average Conditions	5,436	6,779	8,089
Median Conditions	2,306	2,451	2,739
CC/LCC System Yield Reduction (acft/yr)	315	403	440
Median CC/LCC System Storage Reduction (%)	0.00	0.23	0.49
Estuarine Inflow Reduction (acft/yr)	908	1,134	1,370
Summary of Project Costs			
Dam, Spillway, and Appurtenant Works	$4,076,022	$5,358,732	$5,300,418
Road Relocations	$3,500,000	$3,750,000	$5,945,000
Land Acquisition	$183,400	$397,000	$782,500
Environmental Mitigation	$9,522	$16,735	$29,359
Engineering, Legal, Financial, and Misc.	$1,553,789	$1,904,493	$2,411,455
Total Capital Cost	$9,322,733	$11,426,961	$14,468,732
Annual Capital Cost (25 years @ 7.5% interest)	$836,249	$1,024,998	$1,297,845
Operations and Maintenance (annual)	$16,304	$21,435	$21,202
Water Rights Mitigation (annual)	$135,062	$172,268	$190,590
Total Annual Cost	$987,615	$1,218,701	$1,509,637
Annual Cost / Unit Recharge Enhancement:			
Drought Conditions ($/acft/yr)	$827	$1,022	$1,265
Average Conditions ($/acft/yr)	$182	$180	$187

Shaded column represents optimum size of project based on lowest unit cost for average conditions.
[1]Flood pools based on reservoirs being empty at beginning of flood.
Refer to Appendix B for general notes on which project costs and data are based.









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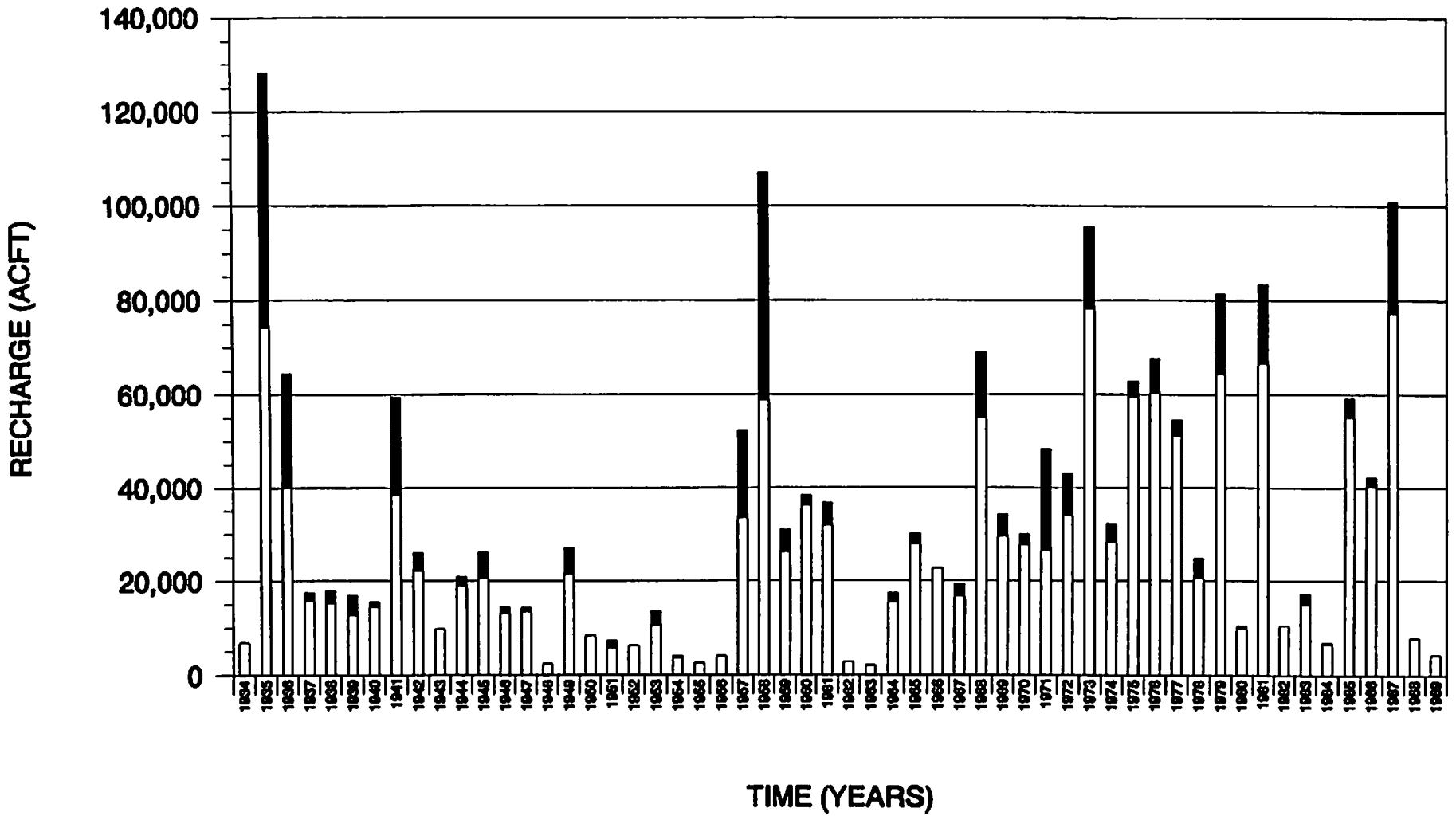
LOWER HONDO PROJECT
EVALUATION SUMMARY

FIGURE 3.2 - 3



□NATURAL RECHARGE ■RECHARGE ENHANCEMENT

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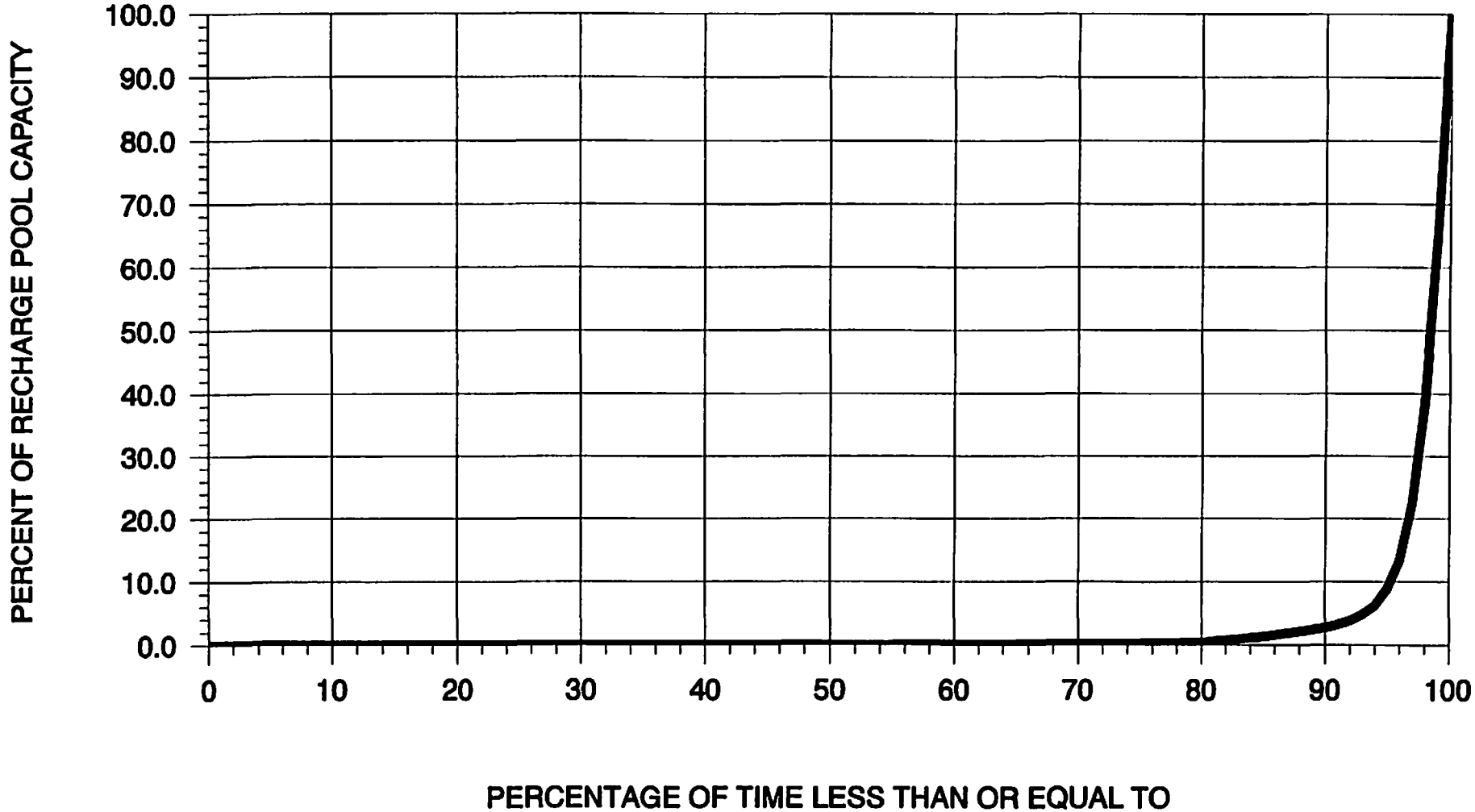
LOWER HONDO PROJECT
ANNUAL RECHARGE

FIGURE 3.2 - 4



RECHARGE POOL CAPACITY = 2,800 ACFT

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ENHANCEMENT PROJECT - PHASE IVA

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LOWER HONDO PROJECT
STORAGE FREQUENCY

FIGURE 3.2 - 5

3.2.5 Design Flood Reduction Considerations

Given the relatively small storage volume and dam height of the optimum size Lower Hondo Project, an opportunity may exist to reduce the capital cost by designing the dam for a flood event less than the PMF. For the Texas Natural Resource Conservation Commission (TNRCC) to accept a lesser design flood, the owner must demonstrate through technical analyses and field surveys that the dam only constitutes a "significant" hazard instead of a "high" hazard to downstream property and potential loss of life (see Table 2.3-1). Various hypothetical dam failure scenarios can be simulated and combined with downstream channel cross section surveys and flood routing analyses to identify potential areas of impact and determine the critical design flood event for the project.

For comparison purposes, a cost estimate for the composite embankment/RCC dam was prepared assuming the critical design flood is 75 percent rather than 100 percent of the PMF. The construction cost savings which could be realized by reducing the design flood are estimated to be $150,000, which equates to about 1.5 percent of the total capital cost. Designing the dam for this reduced PMF would not, however, change the elevations for the 25-, 50-, and 100-year flood events on which land acquisition and road relocation costs are based. The cost savings are solely a result of constructing the top of the dam to a lower elevation, in this case by about 3.8 feet.

Although a potential cost savings may be realized, it is important to note that the TNRCC may require that the dam be upgraded to pass the full PMF at any point in the future if they judge that the dam poses a "high" hazard to downstream interests. This means that if development occurs downstream of the dam within a potential impact area (which often occurs after a dam is built because of improved flood control), the TNRCC could reclassify the dam and require the owner upgrade it to pass the full PMF.

The costs presented in this report assume that the Hondo Creek Dam would be designed for the full PMF. However, further discussions regarding this matter should be reserved for the preliminary design phase after detailed topographic mapping and preliminary geologic and geotechnical data are obtained for the project.

LOWER SABINAL

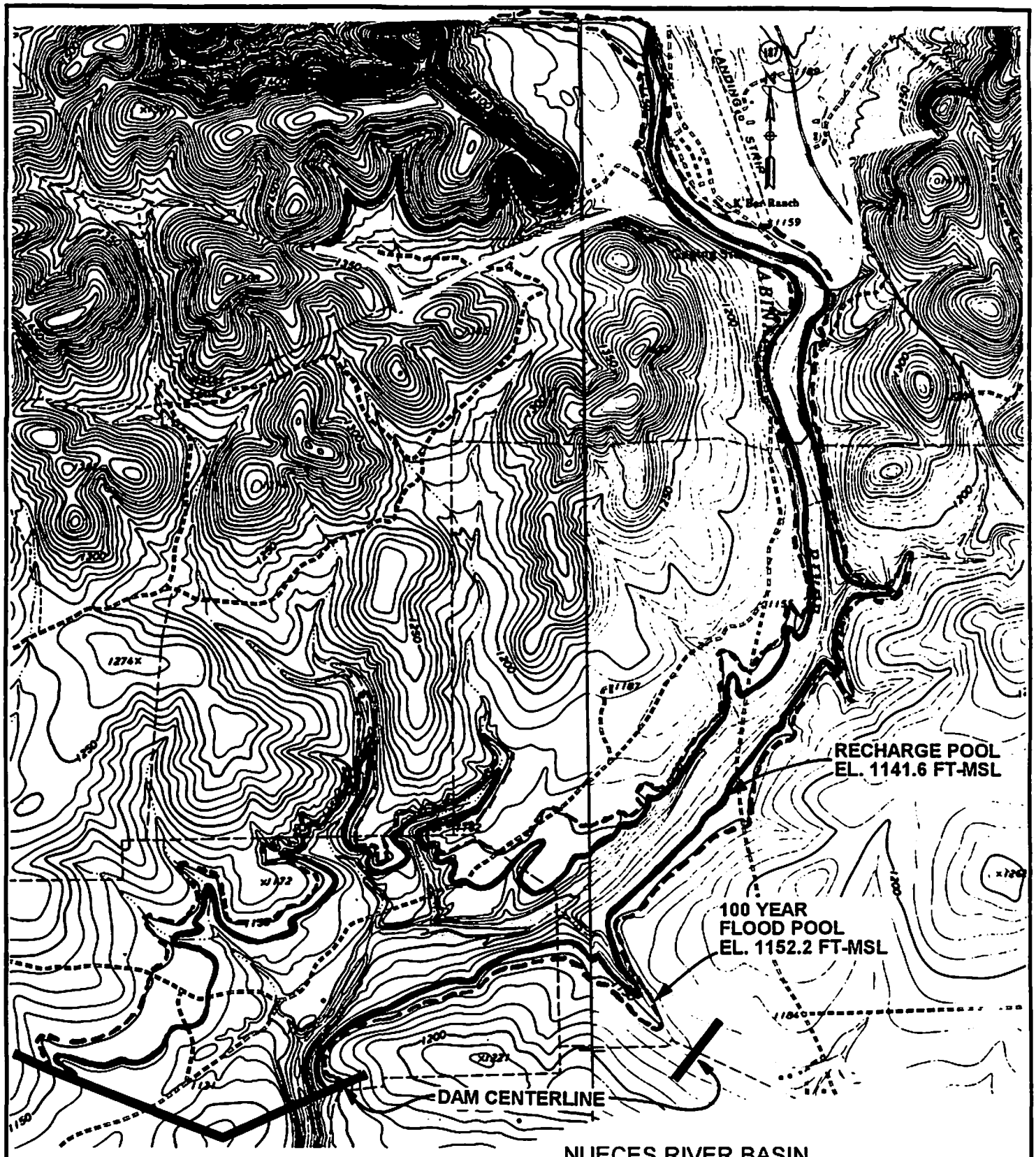
3.3 Lower Sabinal Project

3.3.1 Site Characteristics

The Lower Sabinal Project is located on the Sabinal River approximately 10 miles north by northwest of Sabinal in Uvalde County. The project is located within the Edwards Aquifer recharge zone approximately two miles west of State Highway (SH) 187. As shown in Figure 3.3-1, the upper end of the reservoir is adjacent to SH 187. The site consists of a main dam section across the Sabinal River, an adjacent west saddle area, and a second saddle area about one mile east of the river.

Observations during the site reconnaissance revealed that Edwards bedrock is exposed continuously across the main dam section and no surficial evidence of faulting was noted at the main dam site. However, significant karst (cavern) features are evident in the high bluff along the west side of the river. Some evidence of vertical solution cavities was also noted on the left (east) abutment. The geology of the west saddle area is complex and is believed to consist of an ancient alluvial-filled channel of the Sabinal River. This area is shown on geologic maps as a graben in which Buda Limestone is down-faulted against Edwards Limestone. It is speculated that this graben provided an erodible surface across which the Sabinal River once flowed and that Del Rio Clay probably exists beneath the alluvial deposits. The east saddle area consists of Edwards Limestone with local alluvial valley fill material. Although the geology of the Lower Sabinal Project is complex and additional geologic mapping will be required to completely characterize the site, it does not appear that any geologic or geotechnical fatal flaws exist that would prohibit development of the project (Ref. 6).

Sufficient materials appear to exist within the immediate vicinity of the dam and reservoir site to construct either dam type considered in this study (see Section 2.4.1). Clay derived from the Del Rio Clay Formation beneath the alluvium in the west saddle area and from exposures upstream along the west side of the river would be suitable for an impervious dam core and cutoff. An abundant quantity of material suitable for constructing the embankment dam shells exist in the alluvial deposits in areas west of the main dam and river, or in the spillway excavation required for the embankment dam option. Sand and gravel materials for producing concrete and filters would need to be obtained by washing



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ENHANCEMENT PROJECT - PHASE IVA

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LOWER SABINAL PROJECT
SITE MAP

FIGURE 3.3 - 1

and screening the alluvial deposits. Sand size material in these deposits may be limited. If geotechnical investigations confirm this, it may be necessary to produce sand by crushing Edwards limestone or purchase it from an off-site commercial source. Rock for slope protection would likely be obtained by quarrying Edwards limestone or from the spillway excavation required for the embankment dam option.

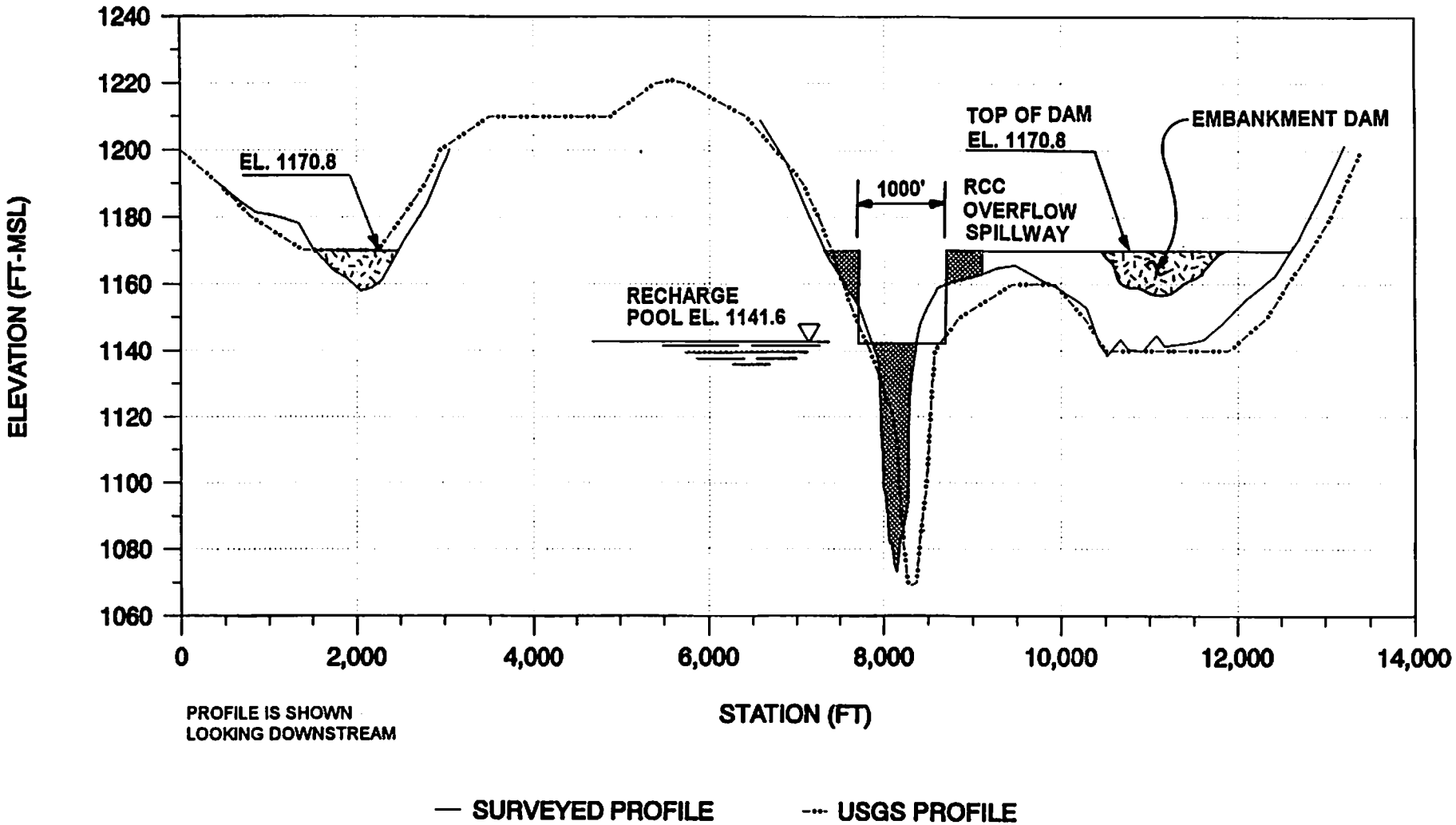
3.3.2 Dam/Spillway Selection

A range of recharge pool capacities from 8,750 to 17,500 acre-feet (acft) was evaluated for the Lower Sabinal Project. This represents approximately 25 to 50 percent of the maximum potential storage capacity considered in the Phase III Study. A recharge reservoir capacity of 8,750 acft was determined to be the optimum size, based on the minimum unit cost of recharge enhancement under average conditions.

The composite dam with an RCC overflow spillway is the most cost effective dam/spillway type for this site. The main dam consists entirely of an RCC gravity overflow section, as shown in Figure 3.3-2. Embankment dams would be constructed in the east and west saddle areas. A spillway width of 1,000 feet would be required to safely pass the PMF. This configuration results in the RCC overflow section being about 69 feet high measured from the surveyed low point of the river. The height to the top of the dam would be about 98 feet. The maximum flow depth through the spillway would be approximately 10.5 feet for the 100-year flood and 29 feet during the PMF.

3.3.3 Road Relocations

As shown in Figure 3.3-1, the optimum size reservoir at the Lower Sabinal site would not require any road relocations. Larger recharge reservoirs considered at this site necessitate relocation of SH 187 near the upper end of the reservoir. Additionally, reservoir sizes larger than the optimum begin to inundate significantly more land near the upper end of the reservoir in the vicinity of what is identified on the USGS topographic map as the "K Bar Ranch". These site constraints, combined with diminishing recharge enhancement, results in higher unit costs of recharge enhancement for reservoir sizes above the optimum.



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ENHANCEMENT PROJECT - PHASE IVA

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LOWER SABINAL PROJECT
CENTERLINE PROFILE

FIGURE 3.3 - 2

3.3.4 Project Evaluation

A summary of the project costs and physical data for the Lower Sabinal Project is provided in Table 3.3-1. Much of this data is also presented graphically in Figure 3.3-3. As indicated in the table and figure, the optimal site development is a reservoir capacity of 8,750 acft, which results in 16,442 acft/yr of recharge enhancement under average conditions at a unit cost of $81/acft/yr. Recharge under drought conditions would be increased by 2,358 acft/yr at a unit cost of $566/acft/yr.

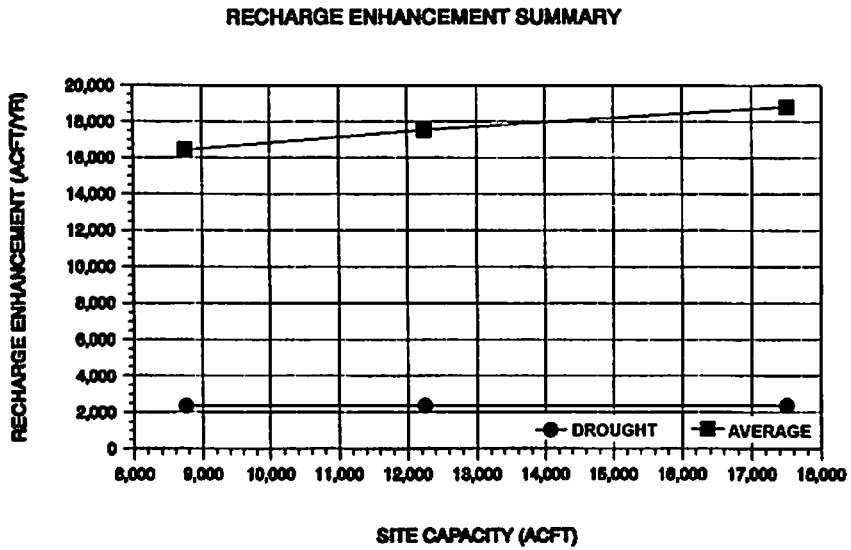
A graph illustrating the natural recharge and the recharge enhancement resulting from development of the optimum size Lower Sabinal Project is shown in Figure 3.3-4 for the 56-year period of record from 1934 through 1989.

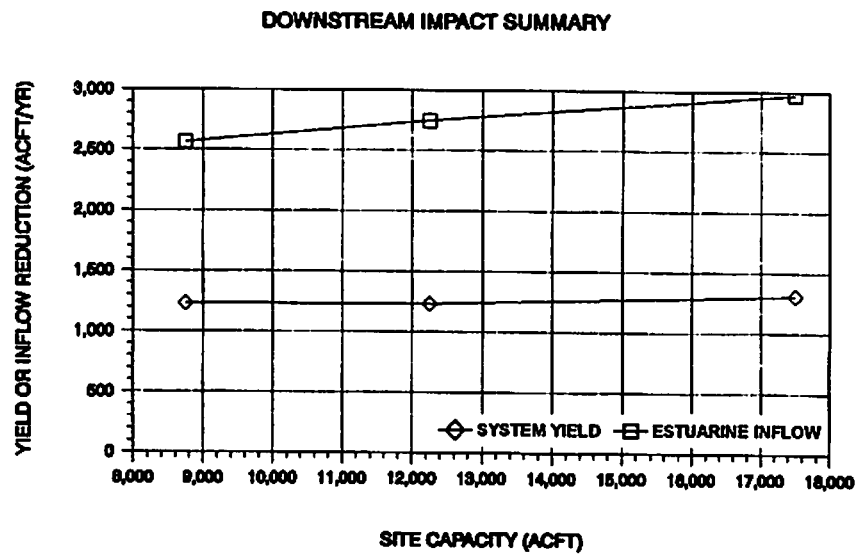
Figure 3.3-5 shows the frequency of various storage levels for the optimum size project. It indicates that, on the average, the reservoir would be empty about 50 percent of the time. Approximately 90 percent of the time it would be less than 10 percent full, and 97 percent of the time the reservoir storage would be less than 50 percent of its optimum capacity.

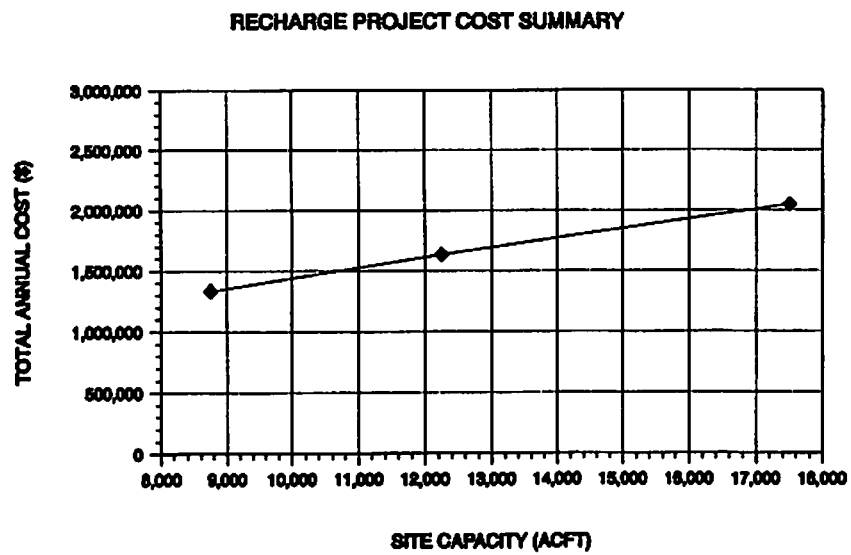
Table 3.3-1
Lower Sabinal Project Cost and Data Summary

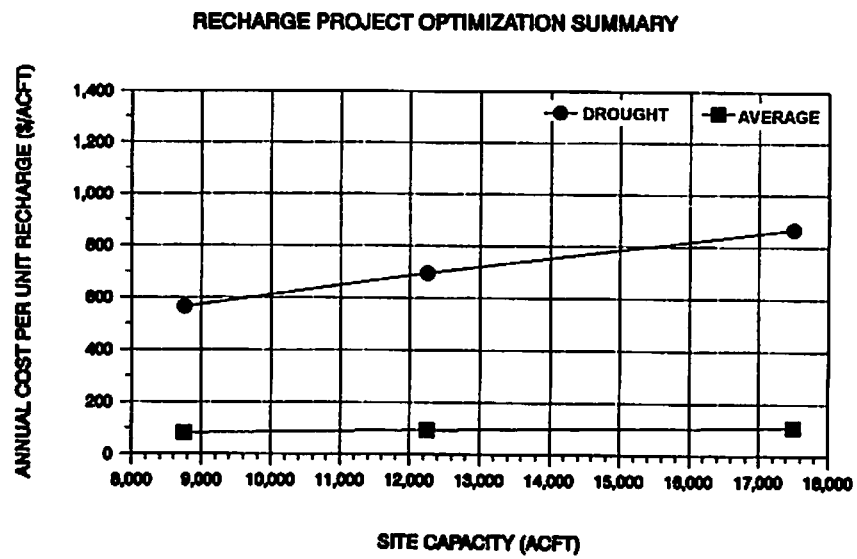
Physical Data			
Recharge Pool:			
Capacity (acft)	8,750	12,250	17,500
Surface Area (ac)	454	587	781
Elevation (ft-msl)	1141.6	1148.4	1155.8
Spillway Elevation (ft-msl)	1141.6	1148.4	1155.8
Spillway Width (ft)	1,000	1,000	1,000
25-Year Flood Pool[1]:			
Elevation (ft-msl)	1149.8	1156.5	1163.7
Surface Area (ac)	615	784	991
50-Year Flood Pool[1]:			
Elevation (ft-msl)	1151.0	1157.6	1164.8
100-Year Flood Pool[1]:			
Elevation (ft-msl)	1152.2	1158.8	1166.0
Surface Area (ac)	675	847	1070
Dam Type	RCC/Earth Composite	RCC/Earth Composite	RCC/Earth Composite
Top of Dam Elevation (ft-msl)	1170.8	1177.3	1184.3
Streambed Elevation (ft-msl)	1073.0	1073.0	1073.0
Hydrologic Data			
Recharge Enhancement (acft/yr):			
Drought Conditions	2,358	2,356	2,359
Average Conditions	16,442	17,541	18,822
Median Conditions	5,388	5,388	5,388
CC/LCC System Yield Reduction (acft/yr)	1,229	1,231	1,308
Median CC/LCC System Storage Reduction (%)	0.65	0.72	0.70
Estuarine Inflow Reduction (acft/yr)	2,566	2,746	2,975
Summary of Project Costs			
Dam, Spillway, and Appurtenant Works	$6,724,748	$8,451,496	$10,738,945
Road Relocations	$0	$700,000	$1,110,000
Land Acquisition	$672,000	$908,700	$1,632,300
Environmental Mitigation	$36,172	$46,769	$62,226
Engineering, Legal, Financial, and Misc.	$1,486,584	$2,021,393	$2,708,694
Total Capital Cost	$8,919,504	$12,128,358	$16,252,165
Annual Capital Cost (25 years @ 7.5% interest)	$800,080	$1,087,914	$1,457,819
Operations and Maintenance (annual)	$26,899	$33,806	$42,956
Water Rights Mitigation (annual)	$508,400	$512,568	$545,717
Total Annual Cost	$1,335,379	$1,634,288	$2,046,492
Annual Cost / Unit Recharge Enhancement:			
Drought Conditions ($/acft/yr)	$566	$694	$868
Average Conditions ($/acft/yr)	$81	$93	$109

Shaded column represents optimum size of project based on lowest unit cost for average conditions.
[1]Flood pools based on reservoirs being empty at beginning of flood.
Refer to Appendix B for general notes on which project costs and data are based.









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ENHANCEMENT PROJECT - PHASE IVA

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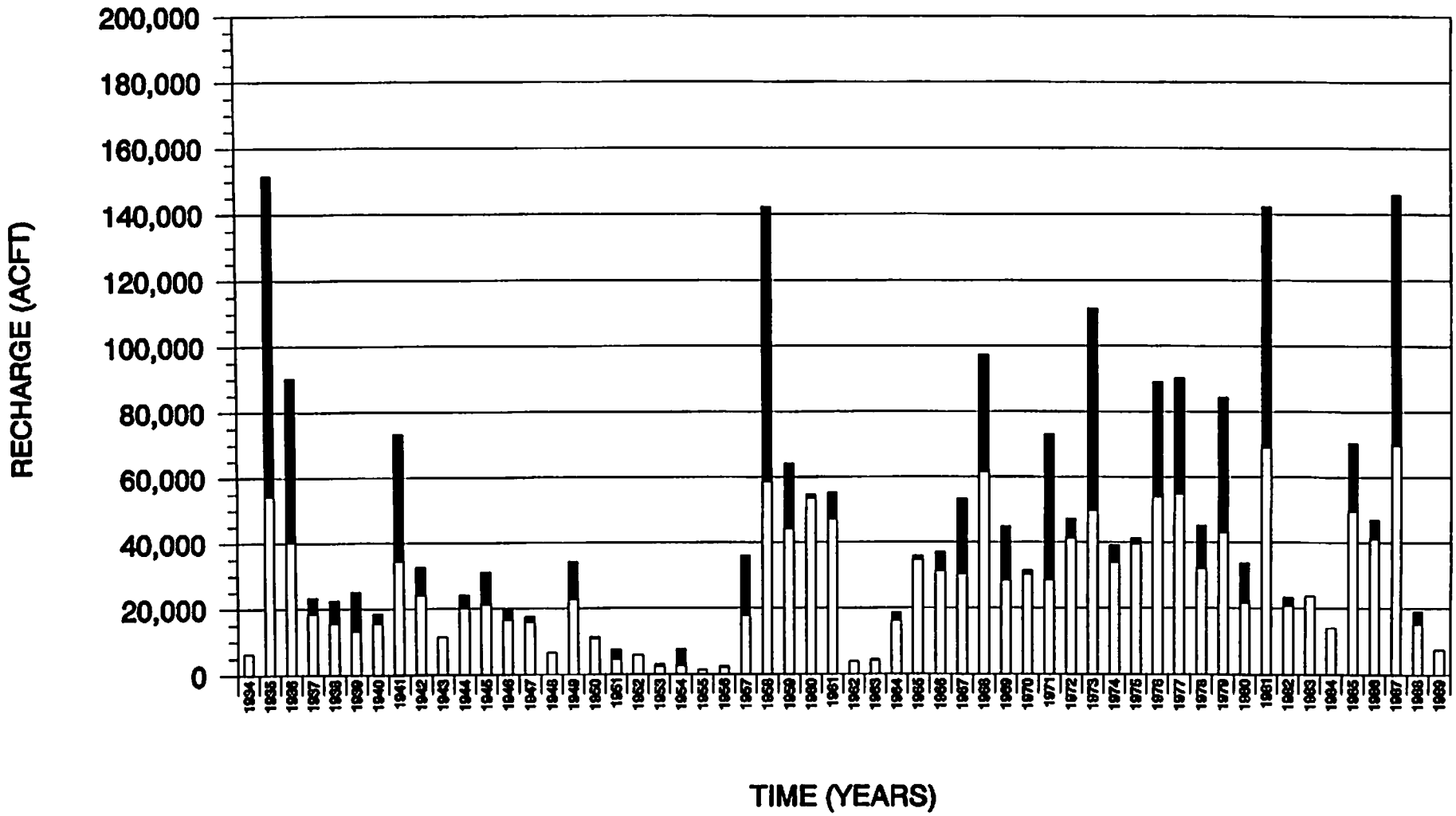
LOWER SABINAL PROJECT EVALUATION SUMMARY

FIGURE 3.3 - 3



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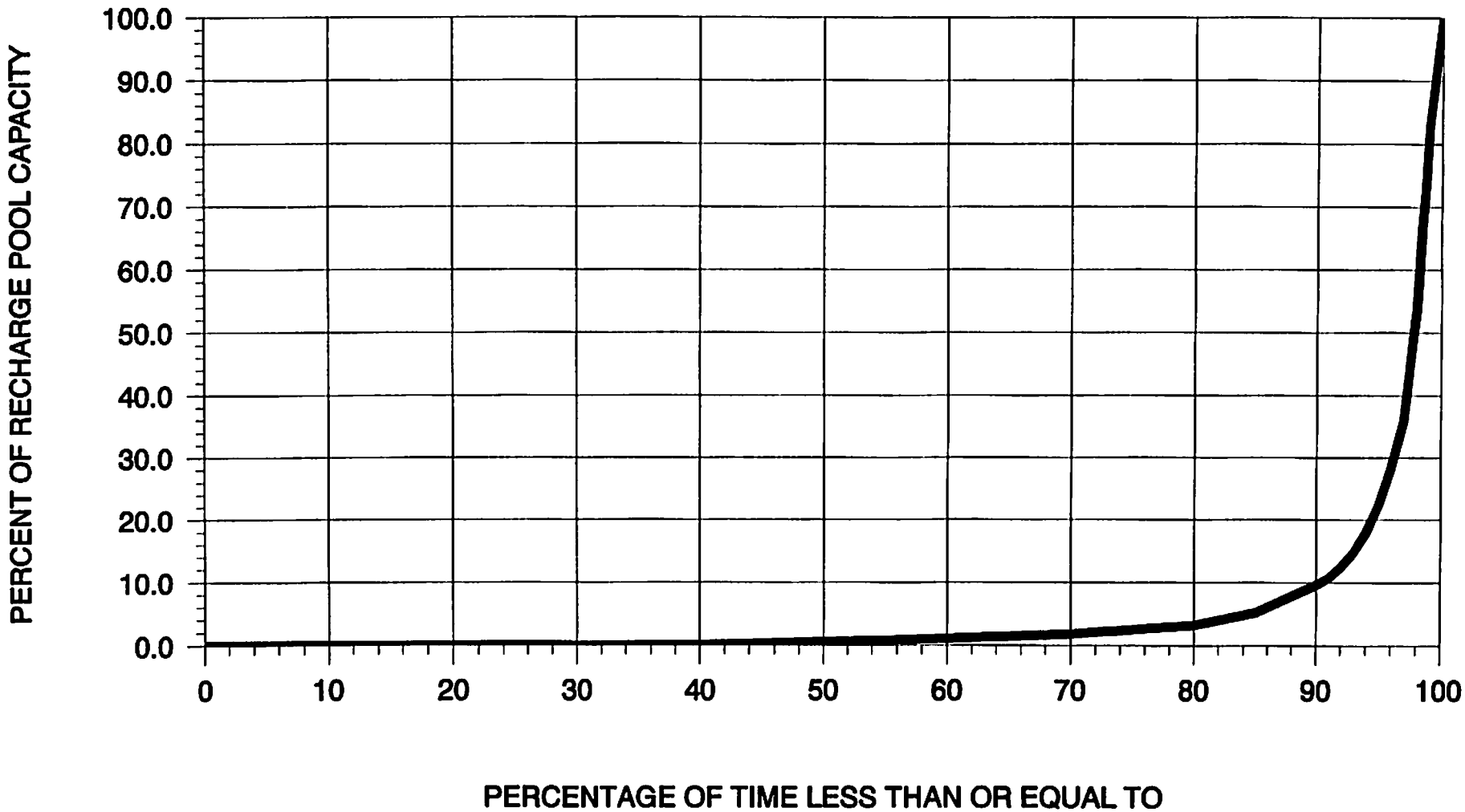
LOWER SABINAL PROJECT
ANNUAL RECHARGE

FIGURE 3.3 - 4



RECHARGE POOL CAPACITY = 8,750 ACFT

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LOWER SABINAL PROJECT
STORAGE FREQUENCY

FIGURE 3.3 - 5

LOWER FRIO

3.4 Lower Frio Project

3.4.1 Site Characteristics

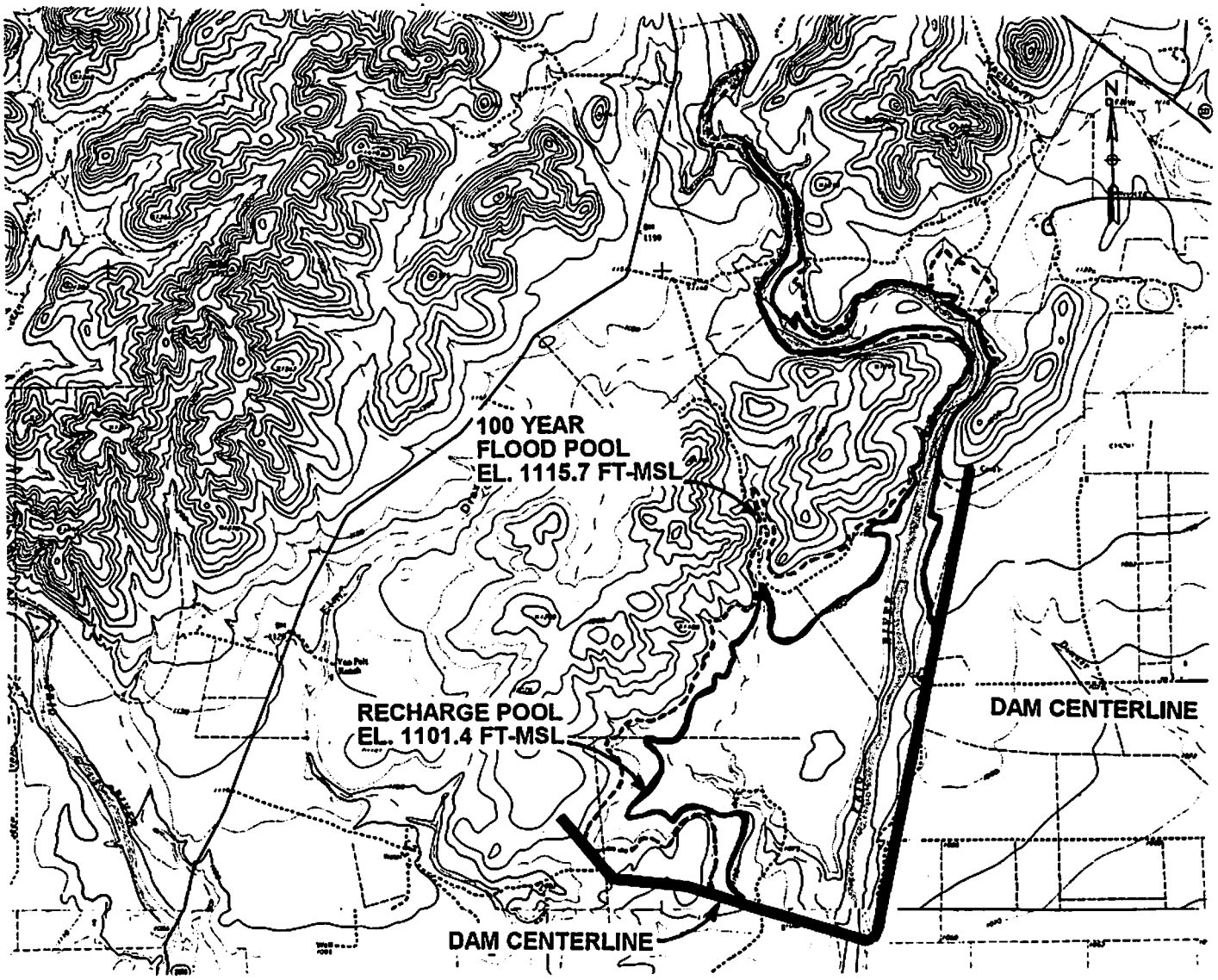
The Lower Frio Project is located on the Frio River approximately seven miles north of Knippa in Uvalde County. Access to the project is off State Highway (SH) 1049 approximately two miles east of the dam site. A site plan of the project, which is located within the Edwards Aquifer recharge zone, is shown in Figure 3.4-1.

The Lower Frio site geology is the most complex of the six recharge projects considered in this study. The geology at the dam site consists of the Edwards Limestone, Buda Limestone, and Del Rio Clay Formations, locally covered by residual and alluvial soils. The half day allotted to the site survey was insufficient for a complete geologic assessment, even at a reconnaissance level. However, from the office and field studies to date, there is no obvious evidence of a geologic or geotechnical fatal flaw that would prohibit development of the project (Ref. 6).

Much of the left (east) side of the dam site traverses a high alluvial terrace. This terrace, which is estimated to be 25 to 50 feet thick, consists of a sandy silt topsoil which overlies clayey to silty gravel with cobbles and boulders. A fault block of Edwards Limestone is exposed near the northeastern end of the dam alignment. A cave exists about 600 feet beyond the east dam abutment. Exploration of this cave will be required during the preliminary geologic investigations.

The river channel and banks at the proposed dam centerline consist of exposed bedrock (probably Edwards). The rock is heavily jointed and has been eroded into rills 0.5 to 1.5 feet deep over distances of five to 10 feet. Approximately 1,500 feet upstream (north) of the dam, the river crosses what appears to be a major fault which has displaced the bedrock below channel level.

An alluvial terrace extends about 500 feet towards the abutment immediately to the right (west) of the channel Materials in this terrace are similar to those observed in the terrace on the east side of the channel. Aerial photographs suggest a faulted section of Edwards Limestone, Del Rio Clay, and Buda Limestone along the west side of the proposed dam site. The right abutment beyond the terrace is believed to be comprised of these three



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SCALE IN FEET

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ENHANCEMENT PROJECT - PHASE IVA

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LOWER FRIO PROJECT
SITE MAP

FIGURE 3.4 - 1

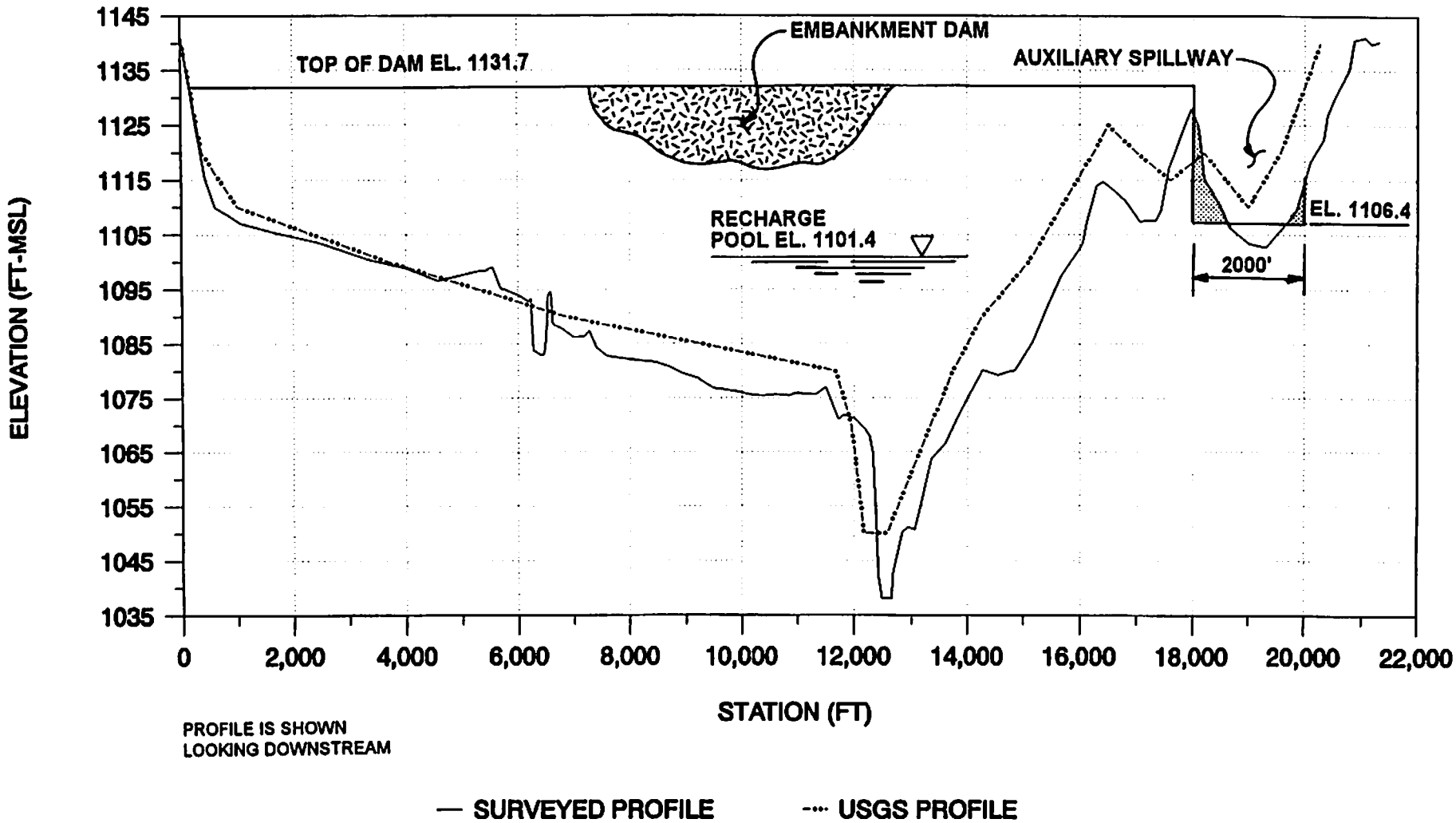
geologic units proceeding from the terrace to the west end of the dam, respectively. Local alluvial deposits mask these units, making surficial observations speculative at best.

Sufficient materials appear to exist within the immediate vicinity of the site to construct either dam type considered in this study (see Section 2.4.1). Clay of marginal quality for an impervious dam core and cutoff appears to be present in sufficient quantities in the upper portion of the terrace deposits within the reservoir and along the eastern reaches of the dam. An abundant quantity of material suitable for constructing the embankment dam shells exists in the terrace deposits or in the spillway excavation required for the embankment dam option. Sand size material for producing concrete and filters may be limited at the site and would need to be obtained from processing the terrace deposits, crushing limestone (Edwards or Buda), or purchased from an off-site commercial source. Rock for slope protection would likely be obtained by quarrying limestone or from the spillway excavation required for the embankment dam option.

3.4.2 Dam/Spillway Selection

A range of recharge pool capacities from 7,500 to 25,000 acre-feet (acft) was evaluated for the Lower Frio Project. This represents approximately 15 to 50 percent of the maximum potential storage capacity considered in the Phase III Study. A recharge reservoir capacity of 17,500 acft was determined to be the optimum size, based on the minimum unit cost of recharge enhancement under average conditions.

The embankment dam with side channel spillway was determined to be the most cost effective dam/spillway type for this site. A 2,000-foot wide auxiliary spillway cut into the saddle area at the far west end of the dam, as shown in Figure 3.4-2, would be required to safely pass the PMF. The crest elevation of this spillway would be set five feet above the recharge pool elevation to reduce the frequency of operation and to provide a reasonable balance between the spillway excavation quantity and the amount of shell material needed for the embankment dam. This configuration results in a dam height of about 94 feet measured from the surveyed low point of the river. The maximum flow depth through the auxiliary spillway would be approximately nine feet during the 100-year flood and 25 feet during the PMF.



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LOWER FRIO PROJECT
CENTERLINE PROFILE

FIGURE 3.4 - 2

3.4.3 Road Relocations

Development of the optimum size reservoir at the Lower Frio site would not require any road relocations.

3.4.4 Project Evaluation

A summary of the project costs and physical data for the Lower Frio Project is provided in Table 3.4-1. Much of this data is also presented graphically in Figure 3.4-3. As indicated in the table and figure, the optimal site development is a reservoir capacity of 17,500 acft, which results in 17,064 acft/yr of recharge enhancement under average conditions at a unit cost of $191/acft/yr. Recharge under drought conditions would be increased by 3,980 acft/yr at a unit cost of $819/acft/yr.

Larger recharge reservoirs considered at this site necessitate use of the composite dam option because of limited topography for a side-channel spillway. The high cost of the composite dam, combined with diminishing recharge enhancement, results in higher unit costs of recharge enhancement for reservoir sizes above the optimum.

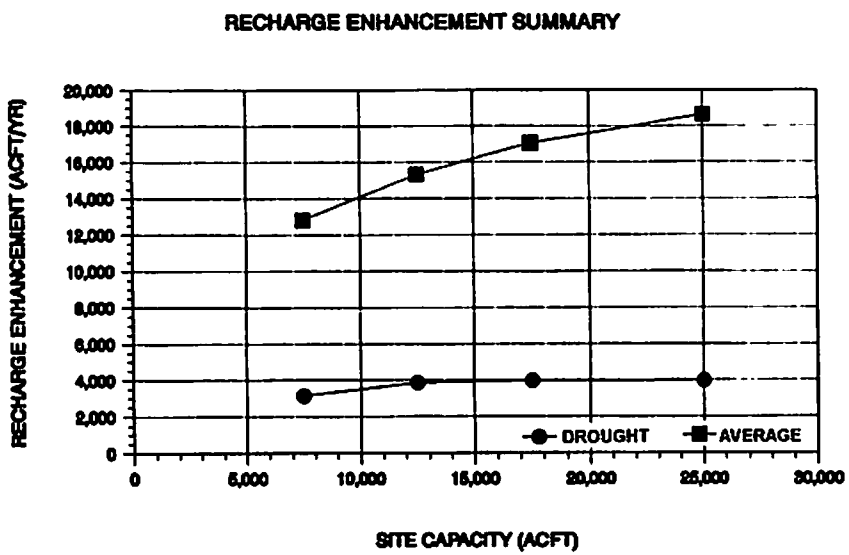
A graph illustrating the natural recharge and the recharge enhancement resulting from development of the optimum size Lower Frio Project is shown in Figure 3.4-4 for the 56-year period of record from 1934 through 1989.

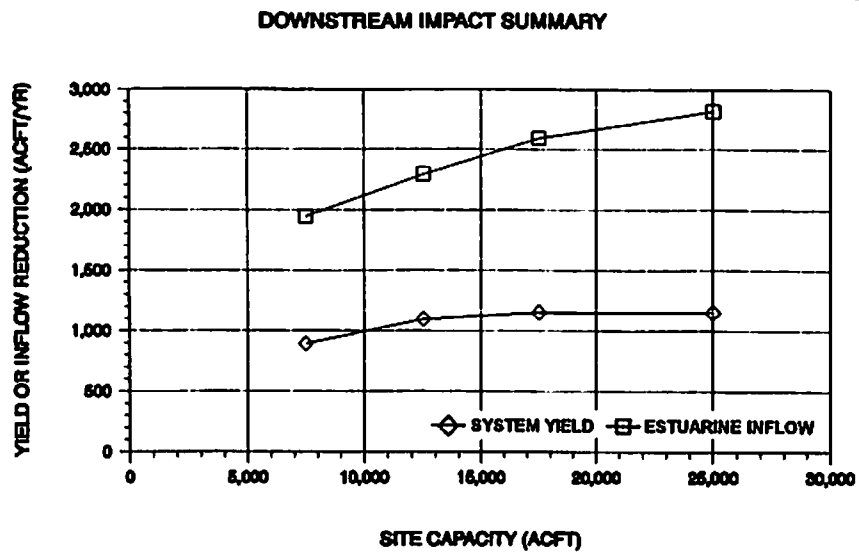
Figure 3.4-5 shows the frequency of various storage levels for the optimum size project. It indicates that, on the average, the reservoir would be empty about 60 percent of the time. Approximately 92 percent of the time it would be less than 10 percent full, and 98 percent of the time the reservoir storage would be less than 50 percent of its optimum capacity.

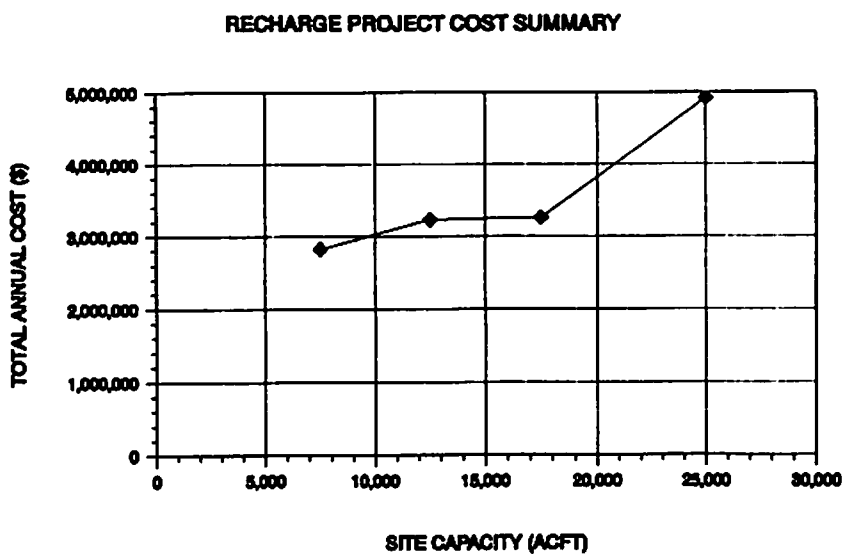
Table 3.4-1
Lower Frio Project Cost and Data Summary

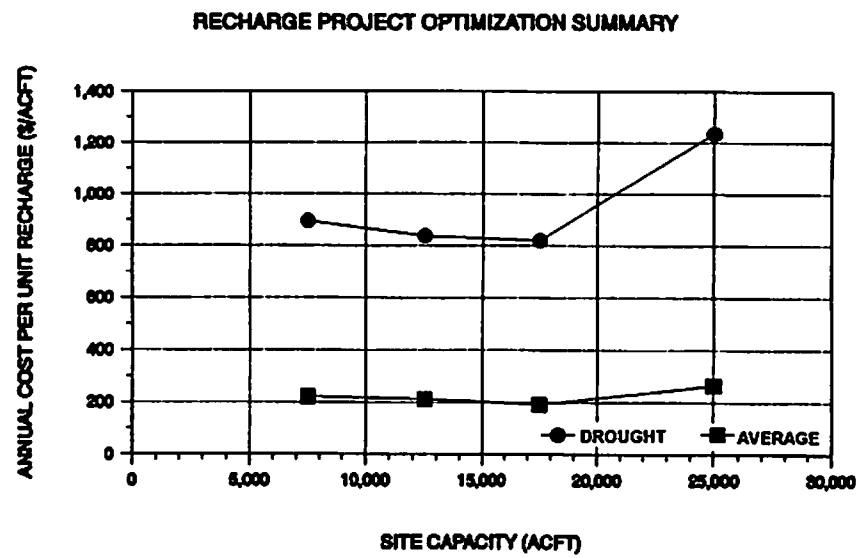
Physical Data				
Recharge Pool:				
Capacity (acft)	7,500	12,500	17,500	25,000
Surface Area (ac)	583	874	1,099	1,291
Elevation (ft-msl)	1089.4	1096.4	1101.4	1107.7
Spillway Elevation (ft-msl)	1089.4	1106.4	1106.4	1107.7
Spillway Width (ft)	2,000	2,000	2,000	1,500
25-Year Flood Pool[1]:				
Elevation (ft-msl)	1096.0	1113.3	1113.4	1115.5
Surface Area (ac)	855	1,474	1,477	1,540
50-Year Flood Pool[1]:				
Elevation (ft-msl)	1097.0	1114.4	1114.5	
100-Year Flood Pool[1]:				
Elevation (ft-msl)	1098.0	1115.6	1115.7	1117.9
Surface Area (ac)	954	1,543	1,546	1,619
Dam Type	RCC/Earth Composite	Embankment	Embankment	RCC/Earth Composite
Top of Dam Elevation (ft-msl)	1112.5	1131.7	1131.7	1135.4
Streambed Elevation (ft-msl)	1038.0	1038.0	1038.0	1038.0
Hydrologic Data				
Recharge Enhancement (acft/yr):				
Drought Conditions	3,158	3,856	3,980	3,986
Average Conditions	12,817	15,315	17,064	18,641
Median Conditions	4,354	4,363	4,398	4,398
CC/LCC System Yield Reduction (acft/yr)	892	1,095	1,152	1,152
Median CC/LCC System Storage Reduction (%)	0.45	0.43	0.46	0.46
Estuarine Inflow Reduction (acft/yr)	1,945	2,294	2,594	2,821
Summary of Project Costs				
Dam, Spillway, and Appurtenant Works	$21,184,825	$23,589,011	$23,589,011	$38,401,743
Road Relocations	$0	$0	$0	$0
Land Acquisition	$783,500	$1,263,700	$1,266,100	$1,321,500
Environmental Mitigation	$40,987	$61,446	$77,264	$90,762
Engineering, Legal, Financial, and Misc.	$4,401,862	$4,982,831	$4,986,475	$7,962,801
Total Capital Cost	$26,411,175	$29,896,988	$29,918,850	$47,776,807
Annual Capital Cost (25 years @ 7.5% interest)	$2,369,082	$2,681,760	$2,683,721	$4,285,580
Operations and Maintenance (annual)	$84,739	$94,356	$94,356	$153,607
Water Rights Mitigation (annual)	$370,563	$453,116	$480,134	$484,447
Total Annual Cost	$2,824,385	$3,229,232	$3,258,211	$4,923,634
Annual Cost / Unit Recharge Enhancement:				
Drought Conditions ($/acft/yr)	$894	$837	$819	$1,235
Average Conditions ($/acft/yr)	$220	$211	$191	$264

Shaded column represents optimum size of project based on lowest unit cost for average conditions.
[1]Flood pools based on reservoirs being empty at beginning of flood.
Refer to Appendix B for general notes on which project costs and data are based.









NUECES RIVER BASIN
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ENHANCEMENT PROJECT - PHASE IVA

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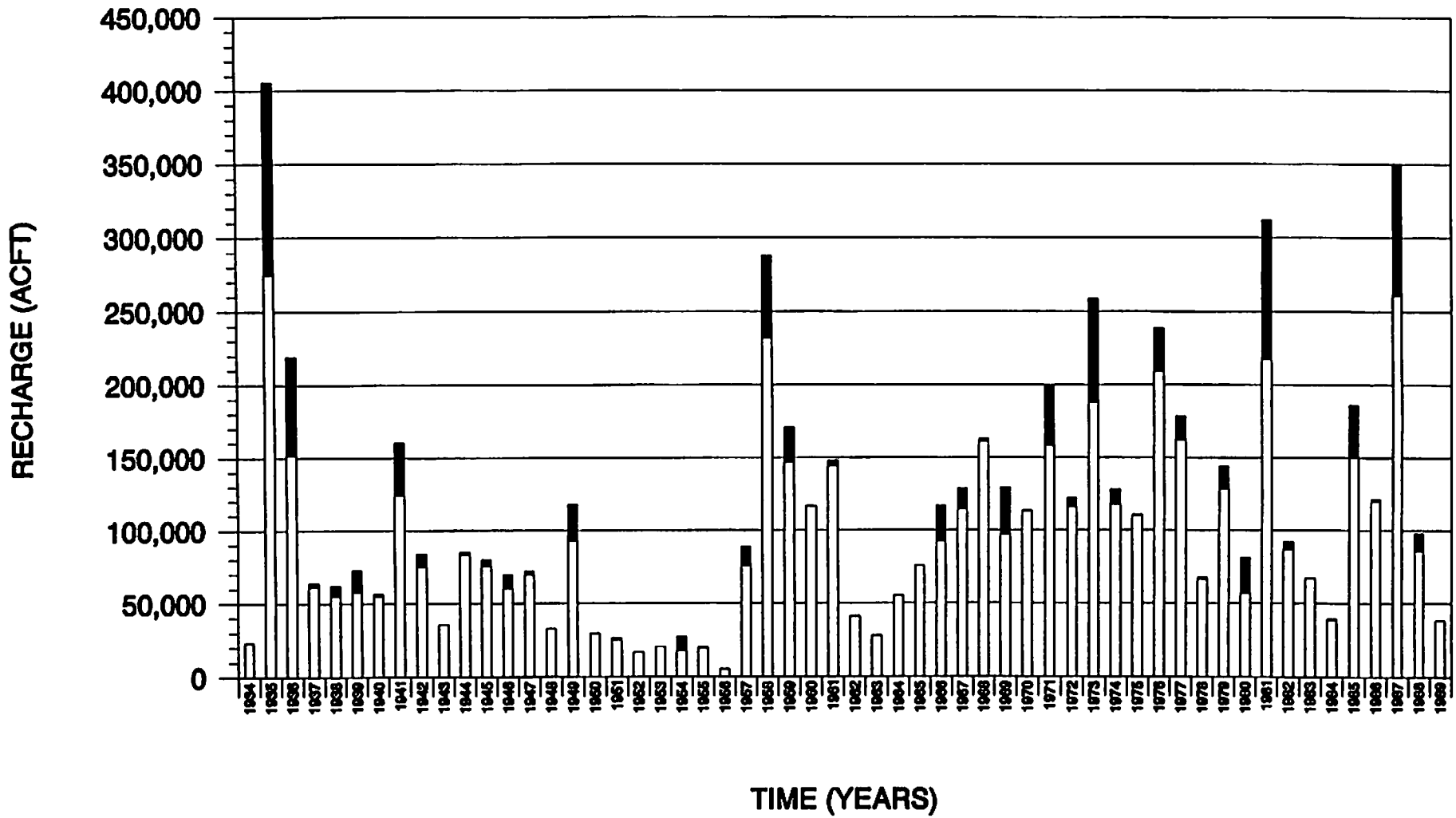
LOWER FRIO
EVALUATION SUMMARY

FIGURE 3.4 - 3



□NATURAL RECHARGE ■RECHARGE ENHANCEMENT

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ENHANCEMENT PROJECT - PHASE IVA

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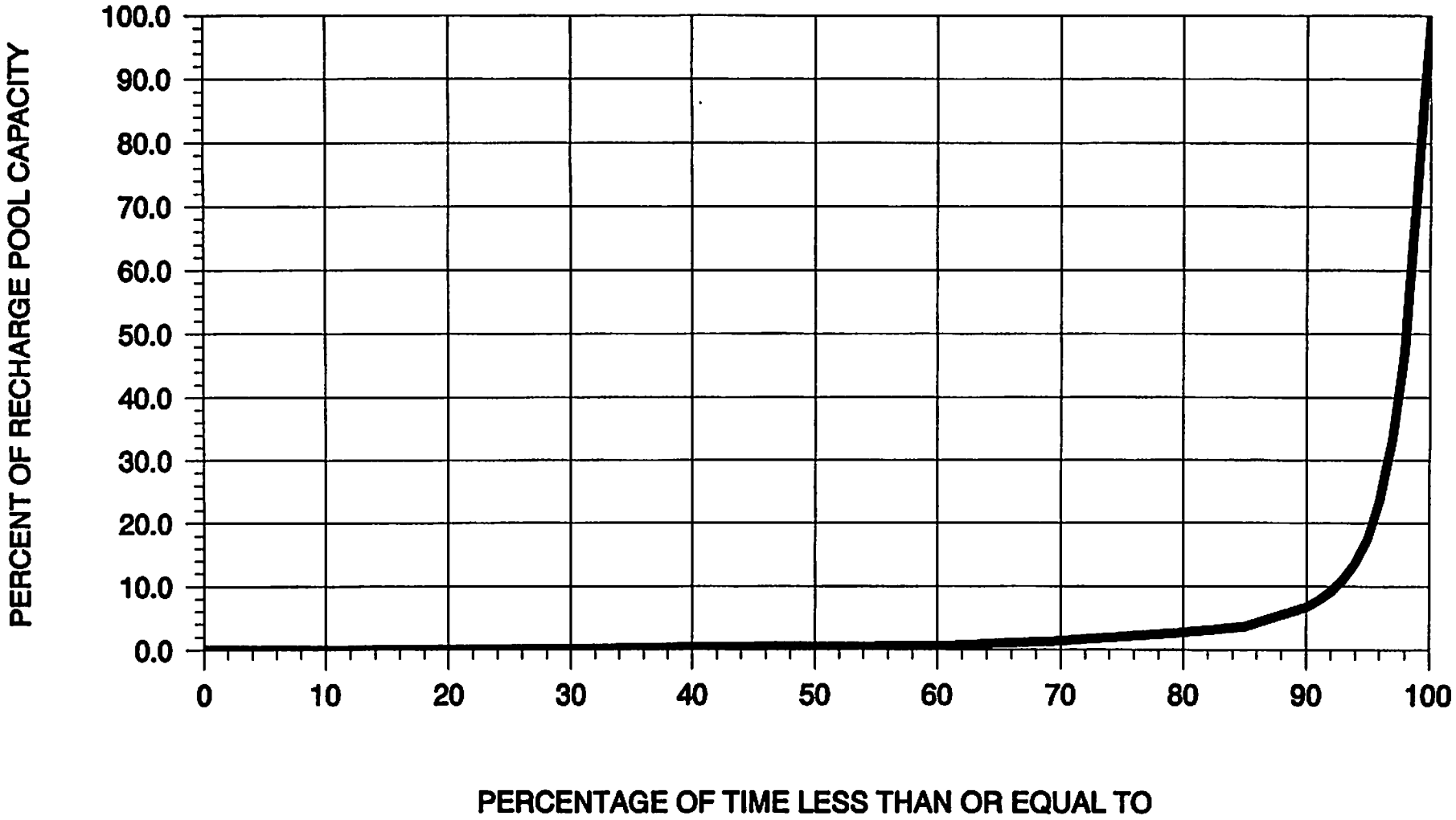
LOWER FRIO PROJECT
ANNUAL RECHARGE

FIGURE 3.4 - 4



RECHARGE POOL CAPACITY = 17,500 ACFT

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ENHANCEMENT PROJECT - PHASE IVA

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LOWER FRIO PROJECT
STORAGE FREQUENCY

FIGURE 3.4 - 5

LOWER DRY FRIO

3.5 Lower Dry Frio Project

3.5.1 Site Characteristics

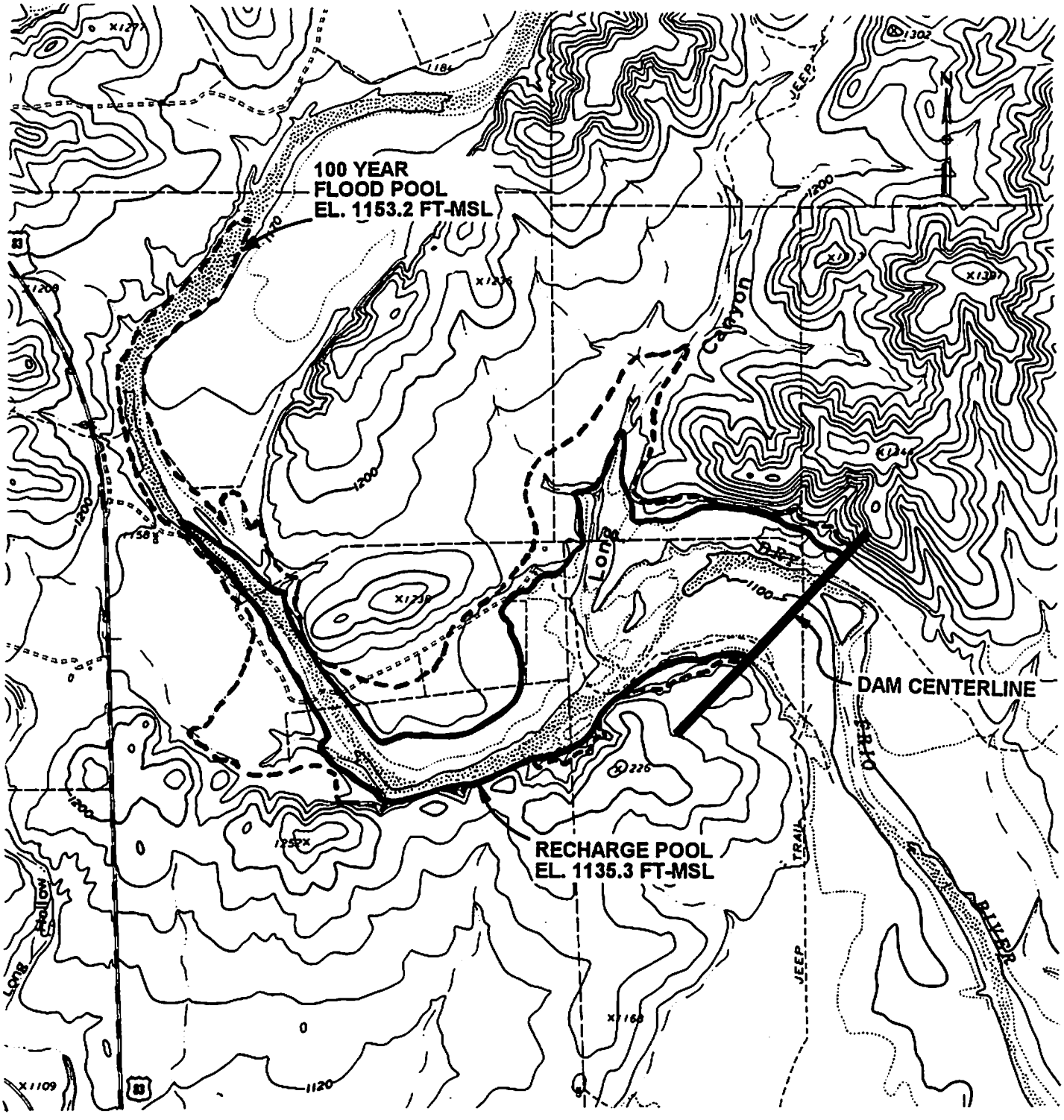
The Lower Dry Frio Project is located on the Dry Frio River approximately 12 miles north of Uvalde in Uvalde County. Access to the project is off U.S. Highway (US) 83 approximately 1.5 miles west of the dam site. A site plan of the project, which is located in the Edwards Aquifer recharge zone, is shown in Figure 3.5-1.

The Lower Dry Frio site geology is the least complex of the six recharge projects considered in this study. No geologic or geotechnical fatal flaws were evident at the site that would prohibit development of the project (Ref. 6).

Edwards Limestone is exposed in both abutments above terrace deposits on each side of the river. The channel at the dam site is filled with alluvium consisting of silty gravel with cobbles and boulders; however, Edwards is exposed in the channel bottom within a distance of about 2,000 feet upstream of the proposed dam alignment.

The terrace on the left (east) side of the river contains a conglomerate bed a few feet below a layer of sandy silt topsoil. Below these materials, the terrace appears to consist of clayey to silty gravel with cobbles and boulders. The terrace on the right (west) side of the river is evident at three levels proceeding about 1,000 feet right of the river channel. Materials in this terrace are similar to those on the left side.

Sufficient materials exist within the vicinity of the reservoir to construct either dam type considered in this study (see Section 2.4.1). Clay of marginal quality for an impervious clay core and cutoff appears to be present in sufficient quantities in the upper portion of the terrace deposits within the dam and reservoir area. An abundant quantity of material suitable for constructing the embankment dam shells exists in the terrace deposits or in the spillway excavation required for the embankment dam option. Sand size material for producing concrete and filters are limited at the site, and would need to be obtained from processing the terrace deposits, crushing Edwards limestone, or an off-site commercial source. Rock for slope protection is plentiful and would likely be obtained from quarrying Edwards limestone or from the spillway excavation required for the embankment dam option.



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SCALE IN FEET

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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LOWER DRY FRIO PROJECT
SITE MAP

FIGURE 3.5 - 1

3.5.2 Dam/Spillway Selection

A range of recharge pool capacities from 4,500 to 10,500 acre-feet (acft) was evaluated for the Lower Dry Frio Project. This represents approximately 15 to 35 percent of the maximum potential storage capacity considered in the Phase III Study. A recharge reservoir capacity of 4,500 acft was determined to be the optimum size, based on the minimum unit cost of recharge enhancement under average conditions.

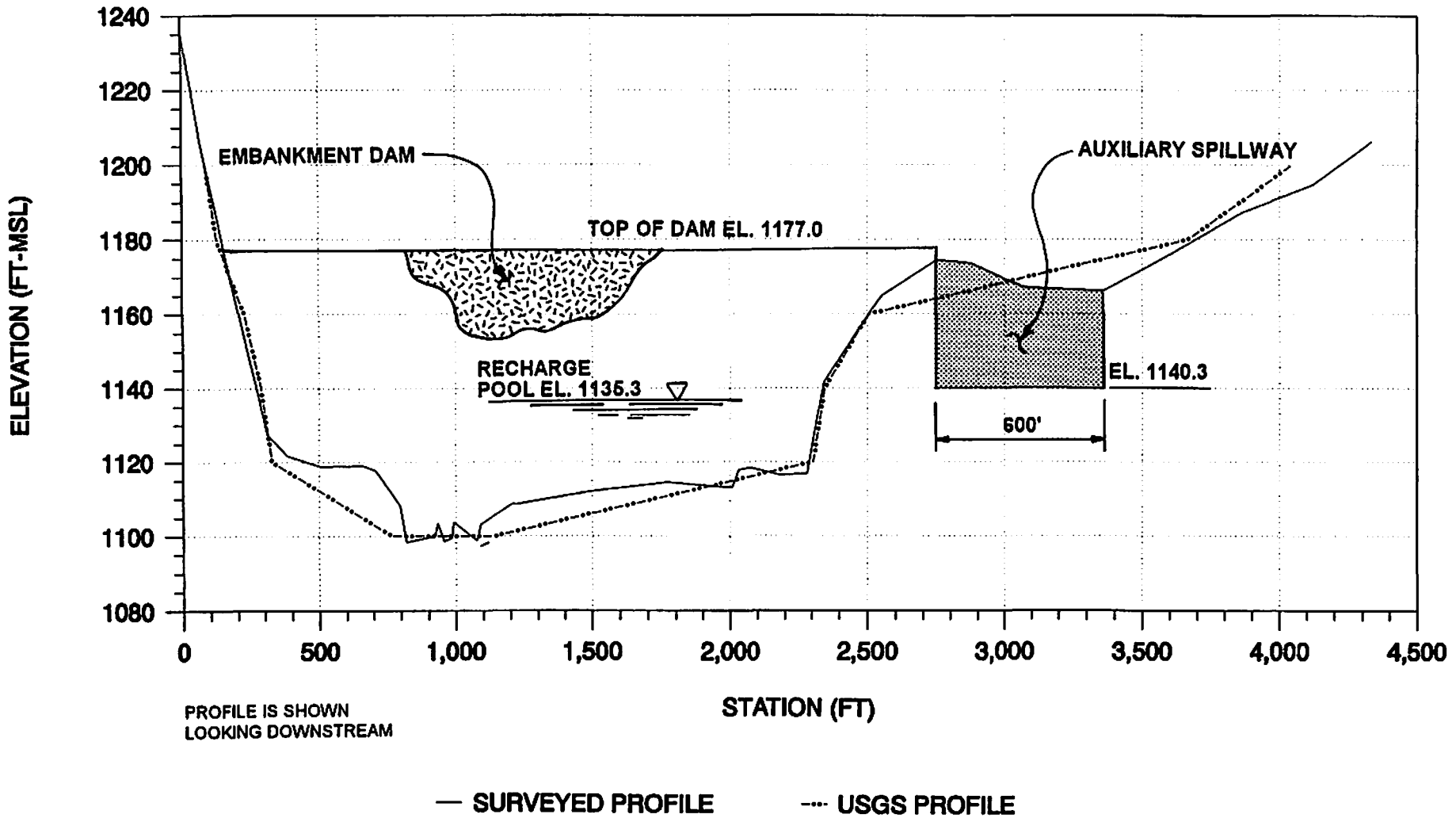
The embankment dam with side channel auxiliary spillway is the most cost effective dam/spillway type for this site. A 600-foot wide rock cut auxiliary spillway located in a shallow saddle of Edwards bedrock on the right abutment, as shown in Figure 3.5-2, is required to safely pass the PMF. The crest elevation of this spillway would be set five feet above the recharge pool elevation to reduce the frequency of operation and to provide a reasonable balance between the spillway excavation quantity and the amount of shell material needed for the embankment dam. This configuration results in a dam height of about 79 feet measured from the surveyed low point of the river. The maximum flow depth through the auxiliary spillway would be approximately 13 feet during the 100-year flood and 37 feet during the PMF.

3.5.3 Road Relocations

Development of the optimum size recharge reservoir at the Lower Dry Frio site would not require any road relocations.

3.5.4 Project Evaluation

A summary of the project costs and physical data for the Lower Dry Frio Project is provided in Table 3.5-1. Much of this data is also presented graphically in Figure 3.5-3. As indicated in the table and figure, the optimal site development is a reservoir capacity of 4,500 acft, which results in 760 acft/yr of recharge enhancement under average conditions at a unit cost of $868/acft/yr. Recharge under drought conditions would be increased by 708 acft/yr at a unit cost of $932/acft/yr. Because the natural capability of the Dry Frio River to recharge the Edwards Aquifer is very high, the recharge enhancement obtained by constructing a reservoir is minimal. This results in a very high unit cost of recharge enhancement.



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ENHANCEMENT PROJECT - PHASE IVA

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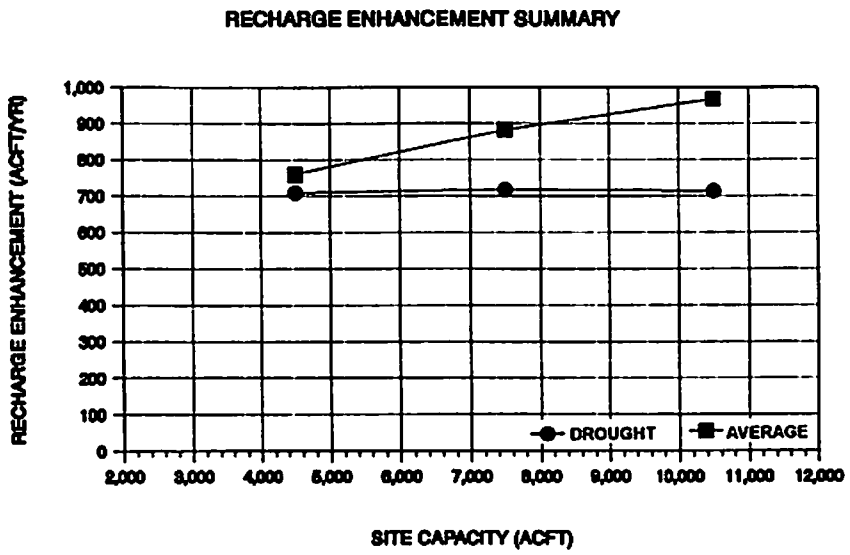
LOWER DRY FRIO PROJECT
CENTERLINE PROFILE

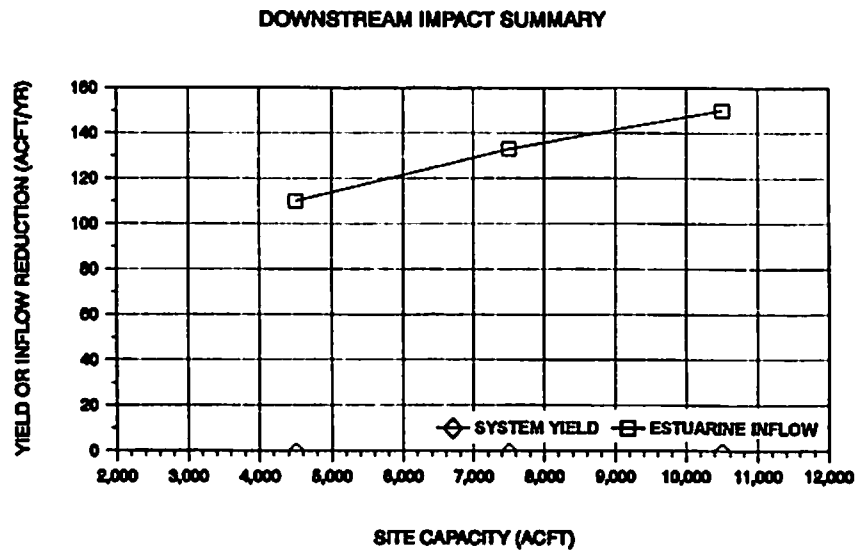
FIGURE 3.5 - 2

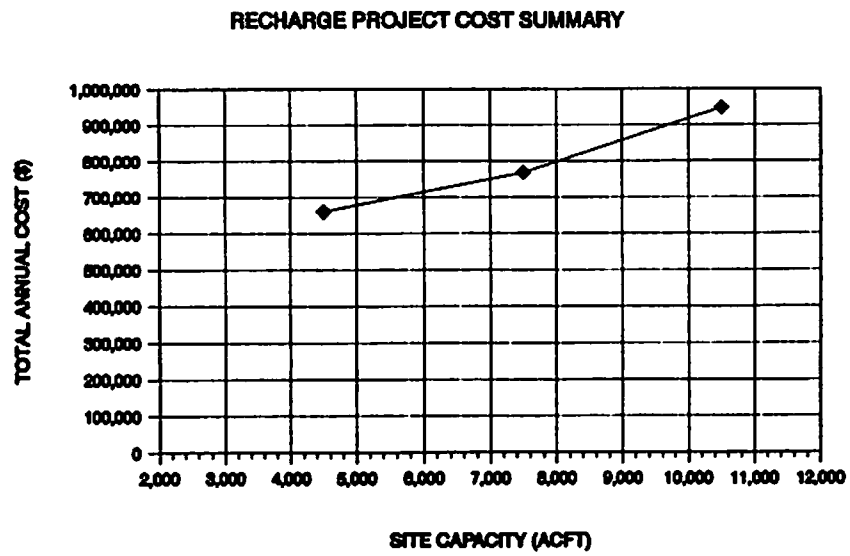
Table 3.5-1
Lower Dry Frio Project Cost and Data Summary

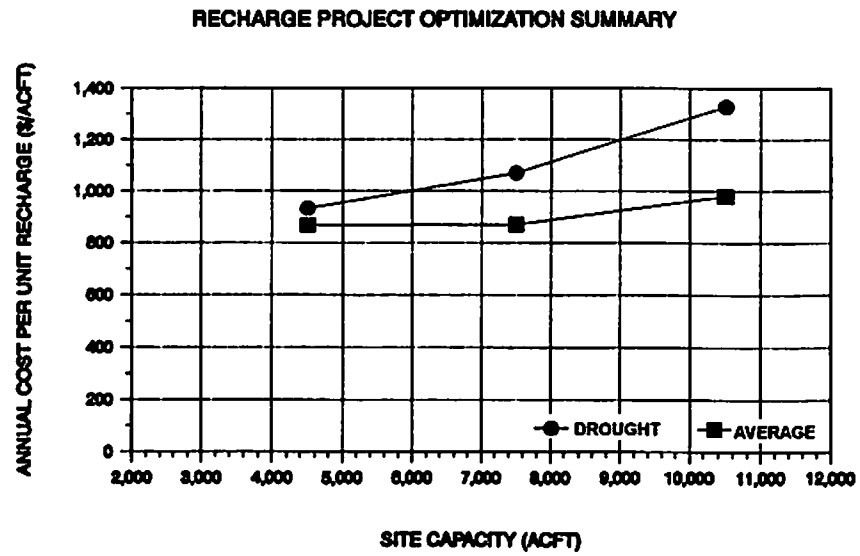
Physical Data			
Recharge Pool:			
Capacity (acft)	4,500	7,500	10,500
Surface Area (ac)	300	436	559
Elevation (ft-msl)	1135.3	1143.5	1149.5
Spillway Elevation (ft-msl)	1140.3	1148.5	1154.5
Spillway Width (ft)	600	700	800
25-Year Flood Pool[1]:			
Elevation (ft-msl)	1150.3	1157.2	1162.1
Surface Area (ac)	577	772	917
50-Year Flood Pool[1]:			
Elevation (ft-msl)	1151.8	1158.6	1163.4
100-Year Flood Pool[1]:			
Elevation (ft-msl)	1153.2	1160.0	1164.6
Surface Area (ac)	650	870	976
Dam Type	Embankment	Embankment	Embankment
Top of Dam Elevation (ft-msl)	1177.0	1181.5	1184.6
Streambed Elevation (ft-msl)	1098.3	1098.3	1098.3
Hydrologic Data			
Recharge Enhancement (acft/yr):			
Drought Conditions	708	717	715
Average Conditions	760	883	968
Median Conditions	0	0	0
CC/LCC System Yield Reduction (acft/yr)	0.00	0.00	0.00
Median CC/LCC System Storage Reduction (%)	0.00	0.00	0.00
Estuarine Inflow Reduction (acft/yr)	110	133	150
Summary of Project Costs			
Dam, Spillway, and Appurtenant Works	$5,243,866	$5,917,803	$6,414,862
Road Relocations	$0	$125,000	$1,130,000
Land Acquisition	$648,100	$816,600	$963,100
Environmental Mitigation	$22,805	$33,143	$42,493
Engineering, Legal, Financial, and Misc.	$1,182,954	$1,378,509	$1,710,091
Total Capital Cost	$7,097,725	$8,271,055	$10,260,546
Annual Capital Cost (25 years @ 7.5% interest)	$636,666	$741,914	$920,371
Operations and Maintenance (annual)	$20,975	$23,671	$25,659
Water Rights Mitigation (annual)	$2,090	$2,527	$2,850
Total Annual Cost	$659,731	$768,112	$948,880
Annual Cost / Unit Recharge Enhancement:			
Drought Conditions ($/acft/yr)	$932	$1,071	$1,327
Average Conditions ($/acft/yr)	$868	$870	$980

Shaded column represents optimum size of project based on lowest unit cost for average conditions.
[1]Flood pools based on reservoirs being empty at beginning of flood.
Refer to Appendix B for general notes on which project costs and data are based.









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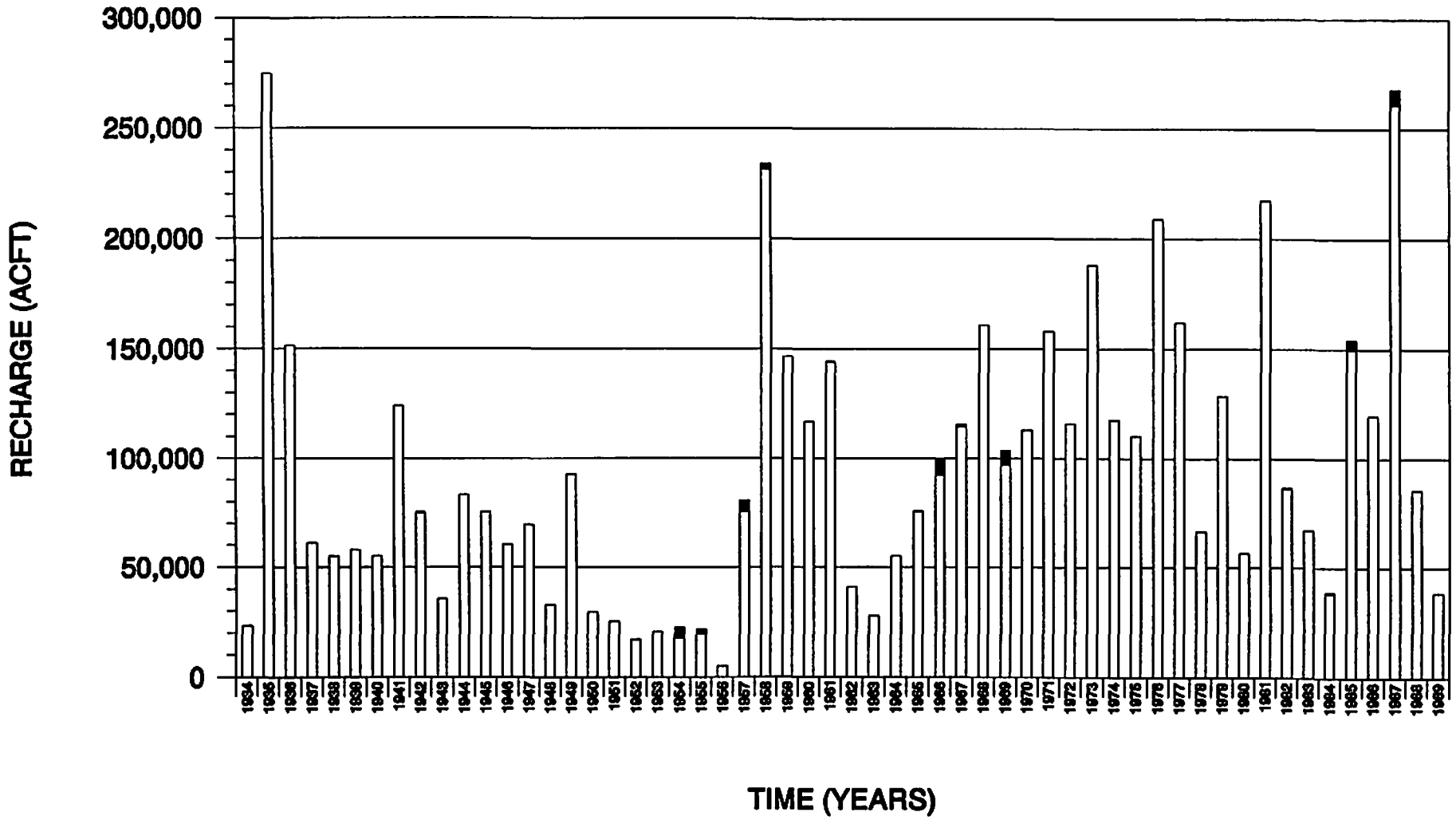
LOWER DRY FRIO RPOJECT EVALUATION SUMMARY

FIGURE 3.5 - 3

A graph illustrating the natural recharge and the recharge enhancement resulting from development of the optimum size Lower Dry Frio Project is shown in Figure 3.5-4 for the 56-year period of record from 1934 through 1989. This graph dramatically illustrates the minimal recharge enhancement potentially obtained from a storage reservoir at this project because of the high natural capability of the Dry Frio River to recharge the aquifer. The high natural capacity for recharge within the Dry Frio River may make it suitable for importation of water from other sources. A series of small channel dams on the Dry Frio River, in conjunction with diversion from the Indian Creek Project (Nueces River Basin), could be investigated as part of the further studies recommended for the Indian Creek Project.

Figure 3.5-5 shows the frequency of various storage levels for the optimum size project. It indicates that, on the average, the reservoir would be empty about 95 percent of the time. Approximately 99 percent of the time the reservoir storage would be less than 50 percent of its optimum capacity. This figure further demonstrates that a storage reservoir on the Dry Frio River would not significantly enhance recharge to the aquifer.



□ NATURAL RECHARGE
(SHOWN FOR ENTIRE FRIO RIVER BASIN ABOVE USGS STREAMFLOW GAGE (1975) AT UVALDE)

■ RECHARGE ENHANCEMENT

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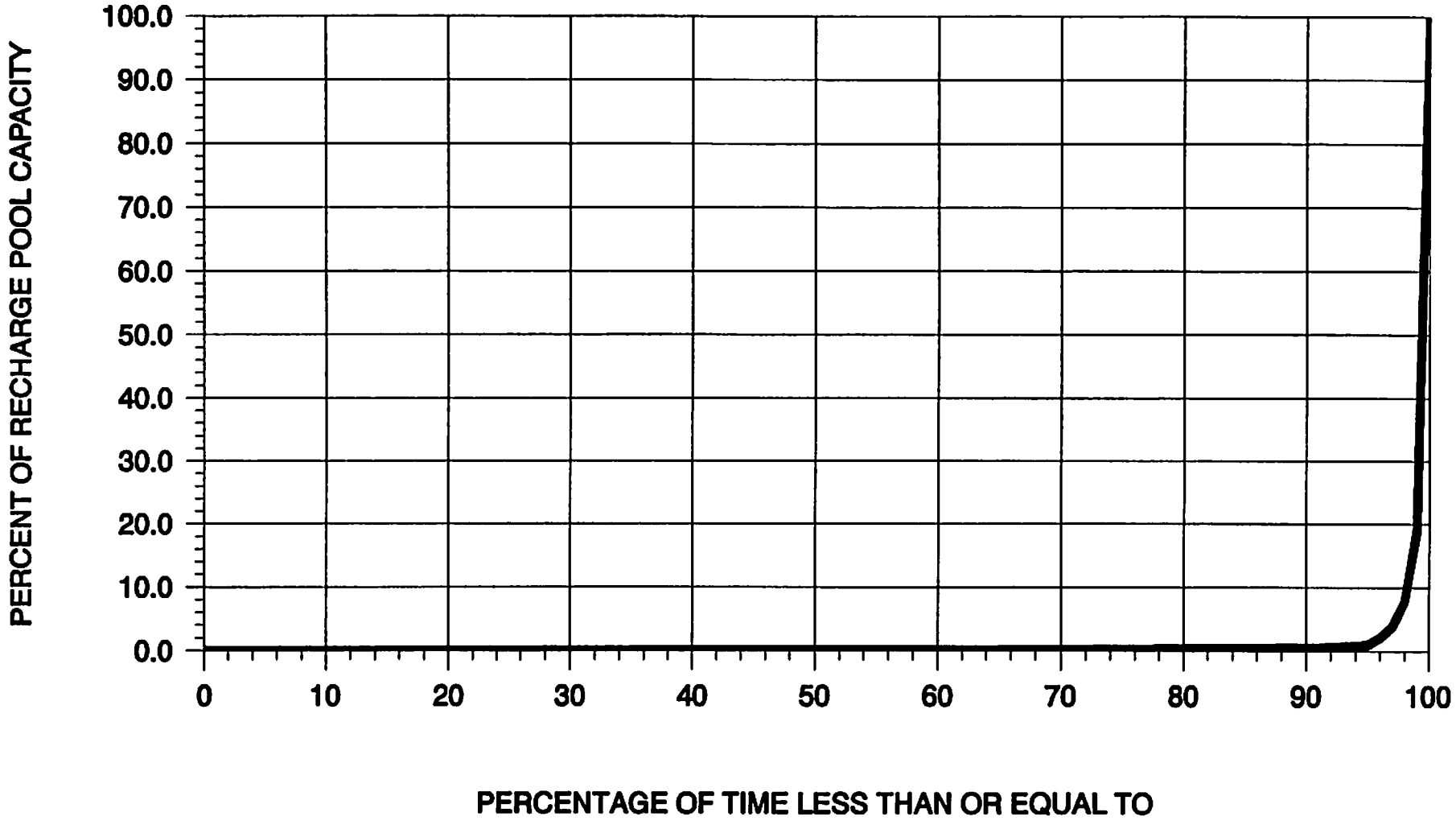
LOWER DRY FRIO PROJECT
ANNUAL RECHARGE

FIGURE 3.5 - 4



RECHARGE POOL CAPACITY = 4,500 ACFT

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LOWER DRY FRIO PROJECT
STORAGE FREQUENCY

FIGURE 3.5 - 5

INDIAN CREEK

3.6 Indian Creek Project

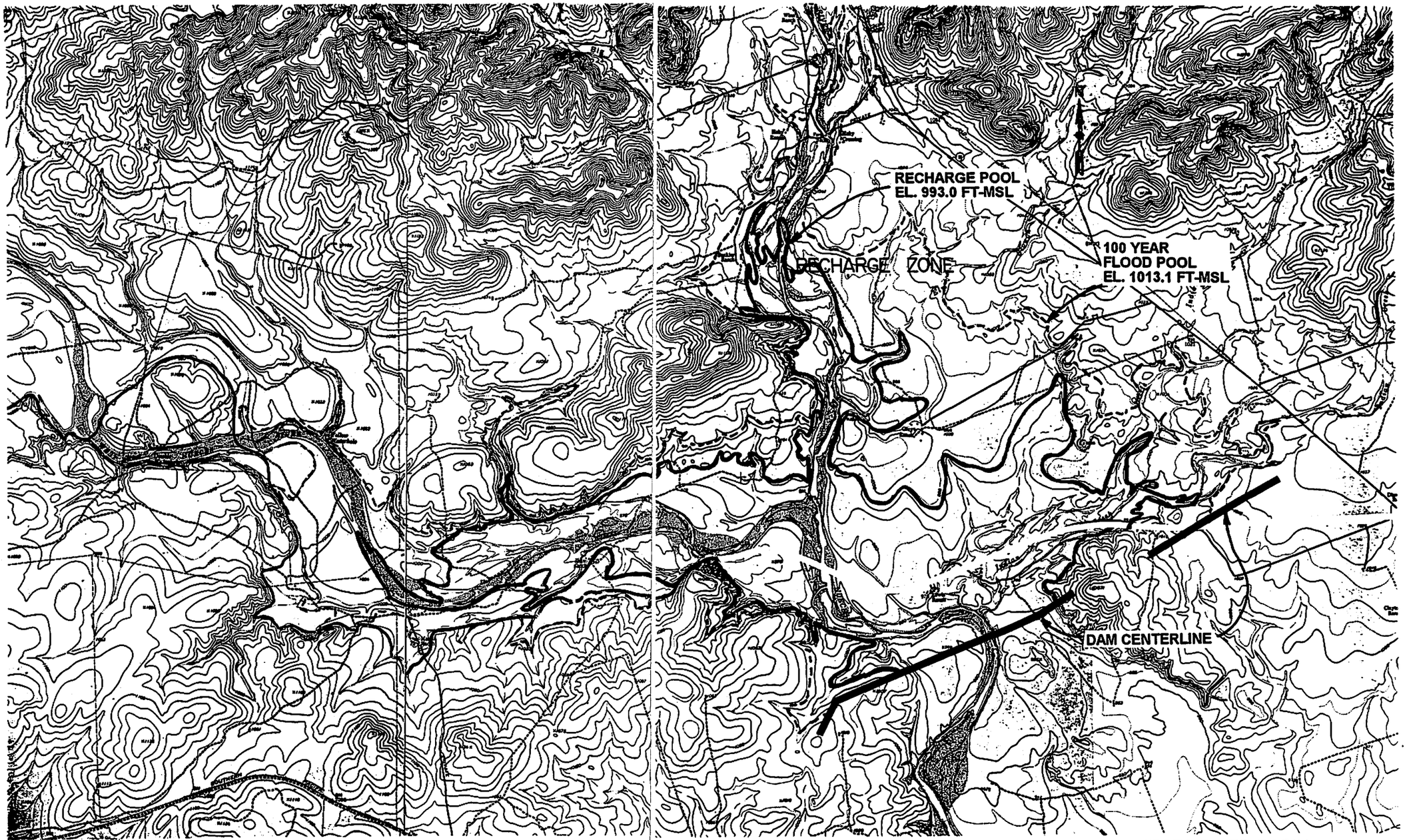
3.6.1 Site Characteristics

The Indian Creek Project is located on the Nueces River approximately two miles downstream of the West Nueces River confluence and immediately downstream of the Indian Creek confluence. The site, which extends nearly four miles in length as shown in Figure 3.6-1, is immediately west of State Highway (SH) 55 approximately seven miles west of Uvalde in Uvalde County.

The site geology at Indian Creek is very complex, and the full day allotted to the field survey was insufficient for a complete geologic assessment, even at a reconnaissance level. However, from the office and field studies to date, there is no obvious evidence of a geologic or geotechnical fatal flaw that would prohibit development of the project. Bedrock at the site consists of the Edwards Limestone, Buda Limestone, Del Rio Clay, and Eagle Ford Formations, locally covered by residual and alluvial soils. The dam centerline is characterized by terrace deposits and bedrock abutments on either side of the main channel and a saddle area approximately one mile east of the left (east) abutment. Bedrock relations are not readily apparent because of pervasive alluvial soil cover (Ref. 6).

Limestone bedrock (possibly Edwards) is exposed along the east side of the river channel at the proposed dam centerline. It was noted to be fractured with dissolutioning along the fractures. A major terrace deposit extends about 2,700 feet from the river to the left abutment bedrock. The surface soils appeared to be clayey, but otherwise the composition and thickness of this deposit are unknown. It is presumed that the left abutment above the terrace is comprised of Buda Limestone. A major terrace deposit also extends about 3,000 feet from the river to the right (west) abutment. Downstream of the proposed dam centerline, the terrace was noted to consist of gravel, cobbles, and boulders beneath thin surficial clayey soils. The terrace deposit was estimated to be up to 40 feet thick at this location. Local bedrock exposures in the right abutment beyond the terrace suggest that it consists of the Eagle Ford Formation with limestone interbeds similar to the Boquillas Flags Formation of west Texas.

Gravel size aggregate for the concrete and RCC appear to be plentiful in the alluvial terrace deposits. However, apparently because of the steep gradient of the Nueces River,



0 4000 8000

SCALE IN FEET

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INDIAN CREEK PROJECT
SITE MAP

FIGURE 3.6 - 1

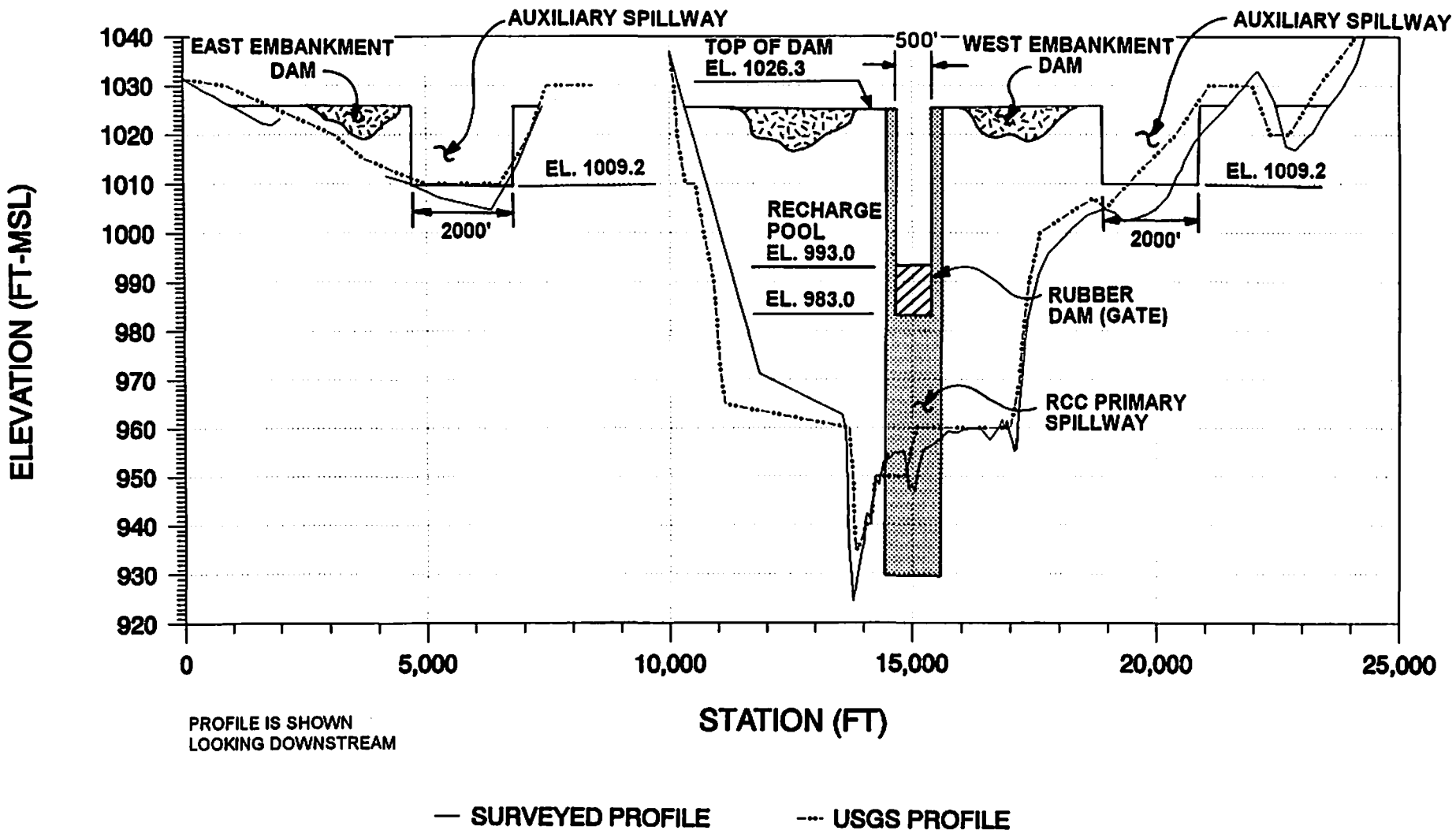
very little sand size material is evident at the site. Fine aggregates would either need to be crushed from on-site materials or purchased from an off-site commercial source. Rocky surface soils observed in the right abutment saddle area and clayey soils in the left saddle area would provide an acceptable random fill material for the embankments. Rockfill materials would likely be obtained from core trench excavation in the main channel and terrace areas and/or from quarrying rock outcrops just upstream of the dam site. Sufficient quality clay for the dam core and cutoff appears to be available in the upper portion of the terraces and in the left saddle area.

3.6.2 Dam/Spillway Selection

A range of recharge pool capacities from 25,700 to 61,750 acre-feet (acft) was evaluated for the Indian Creek Project. This represents approximately 15 to 35 percent of the maximum potential storage capacity considered in the Phase III Study. A recharge reservoir capacity of 61,750 acft appears to be the optimum size, based on the minimum unit cost of recharge enhancement under average conditions.

Twelve alternative dam and spillway configurations were developed, primarily to determine the most efficient way to safely pass the large PMF computed for the Nueces River at the project location. Construction material quantities for each alternative were calculated using the dam centerline survey. The survey was incomplete from approximately Station 20+00 to 40+00 and Station 75+00 to 135+00 because of access restrictions. The USGS 7.5 minute topography was used to obtain centerline elevations through these areas. Various aspects of the alternative configurations are discussed in detail in the following subsections.

Alternative 6, with an estimated construction cost of $50.3 million, was chosen for the project. This configuration, shown in Figure 3.6-2, consists of the embankment dam, a 500-foot wide service spillway with a 10-foot high rubber dam (gate), and two 2,000-foot wide emergency spillways with 6-foot fuse plugs. This alternative has several advantages. It makes use of available emergency spillway excavation in the embankment fill. In comparison to the RCC overtopping alternative, discharges from the spillways enter the river further downstream from the dam, thereby eliminating the need for erosion protection along



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INDIAN CREEK PROJECT
CENTERLINE PROFILE

FIGURE 3.6 - 2

the downstream toe. The rubber dam placed across the service spillway reduces flood levels in the reservoir, which in turn reduces the cost of land acquisition.

3.6.2.1 Alternatives 1-6 - Concrete Service Spillway with Earth Cut Emergency Spillways

The first six alternative dam and spillway arrangements examined consisted of an embankment dam with an RCC/reinforced concrete service spillway and two 2,000-foot wide emergency spillways. The service spillway was located in the right terrace approximately 1,000 feet west of the main channel, which provides the shortest length of flow back to the river. The bulk of the spillway is constructed using mass RCC extending down to bedrock. A reinforced concrete facing would then be formed on the upstream face, crest, and downstream chute of the RCC. The stilling basin would be a 4-foot concrete slab, 160 feet in length. A 200-foot long RCC gravity dam section with reinforced concrete facing would be provided on each side of the spillway to tie into the embankment dam.

In an effort to minimize the dam height, a 10-foot high rubber dam (gate) was assumed to span the entire service spillway crest for Alternatives 4 - 6. The rubber dam, assumed to be in 100-foot sections, would provide an economical gated structure that could be designed to operate automatically. When fully inflated, the rubber dam allows the entire recharge pool to be impounded. During a major flood, the rubber dam would be deflated to provide additional spillway capacity. Rubber dams typically require very little maintenance and are well-suited for remote locations.

The two 2,000-foot wide earth cut emergency spillways were located adjacent to each abutment in order to return discharges to the river as quickly as possible, thereby minimizing flooding of downstream properties. The locations of the spillways are shown in Figure 3.6-2. A fuse plug would be constructed across each emergency spillway to reduce the frequency of operation to floods in excess of the 100-year event. Fuse plugs are low (less than 10 feet) embankments, comprised of erodible granular fill, placed across the emergency spillway to temporarily provide a higher reservoir level during flooding. This higher level results in more water passing through the service spillway at the river channel. However, once the reservoir level exceeds the top of the fuse plug, it begins to erode and

breaches, thereby increasing the total spillway capacity to pass a major flood through the project site.

The embankment dam height for each option was established to safely pass the PMF through the project site. For Alternatives 1-6, the reservoir elevation resulting from routing the 100-year flood through the service spillway was used to set the top of the fuse plug. Following this, an emergency spillway crest elevation was selected and the PMF was routed to determine the top of the dam. The amount of required excavation for the emergency spillway was then compared with the embankment fill requirements. The emergency spillway crest elevation was varied until excavation and fill quantities were reasonably balanced.

The embankment dam design was similar to that considered for the other five projects. However, since the dam will impound water more frequently, two design items were changed: 1) a two-foot layer of soil cement was included on the upstream face for slope protection, and 2) the dam slopes were flattened to 3H:1V upstream and 2.5H:1V downstream.

3.6.2.2 Alternatives 7-12 - Service Spillway with RCC Overtopping Emergency Spillway

Alternatives 7-12 consisted of a service spillway, as previously described, and RCC overtopping protection on the embankment dam which would span approximately 6,000 feet across the main channel and terrace deposits. The overtopping protection would consist of RCC armor plating on the downstream slope of the embankment dam. The RCC would extend down to bedrock, where it would then transition to a 70-foot long stilling basin to return flows to the river. The crest elevation of the RCC overtopping section was set at the peak reservoir elevation resulting from the 10-year flood event routed through the service spillway. Gates consisting of inflatable rubber dams, as previously described, were also evaluated in conjunction with the RCC overflow concept.

The dam/spillway configuration selected for each alternative was based upon assuming different service spillway widths and calculating the corresponding emergency spillway and top of dam elevations. The method of balancing cut and fill used for Alternatives 1-6 was not used for Alternatives 7-12, since the emergency spillway consisted

of the RCC overtopping section (i.e., there is no emergency spillway excavation). The top of dam elevation was determined by routing the PMF through reasonable widths of the overflow section, with the crest set at the 10-year flood elevation.

Certain unit costs developed for the Indian Creek Project varied slightly from those for the other five projects. Random fill was assumed to cost $2.50 per cubic yard (cy), while at the other sites it was assumed to be $2.00/cy. This difference is primarily due to the longer haul distances from the emergency spillway excavations to the dam. The cost of reinforced concrete was assumed to be $280/cy (instead of $300/cy), because much larger concrete volumes would be required at the Indian Creek Project. The increased amount of foundation excavation for the Indian Creek Project dictated using a lower unit cost for this item, $2.50/cy compared to as much as $3.00/cy for foundation excavation at the other sites.

3.6.3 Road Relocations

Development of the optimum size reservoir at the Indian Creek site would not require any road relocations. Larger reservoirs considered at the site would likely necessitate drainage improvements to SH 55 or relocation of the highway to the east.

3.6.4 Diversion of Water to Dry Frio River

Although the Indian Creek Project is located near the downstream edge of the Edwards Aquifer recharge zone, flows may be stored in the reservoir for extended periods because of the apparently limited natural infiltration rate. Recharge enhancement was calculated assuming a natural percolation rate from the reservoir of 2,000 acft per month. In order to efficiently utilize the water stored in the reservoir for recharge, it was assumed that up to 2,000 acft per month would be diverted to the Dry Frio River. The Dry Frio River has a very high natural capability to recharge the aquifer, as discussed in Section 3.5.4. An optimization analysis was performed which determined that the 2,000 acft per month (about 33 cfs through a 36-inch diameter pipeline) was the most cost effective maximum diversion rate. Capital costs are included in the Indian Creek Project for a diversion dam, pump station, and pipeline to convey the water approximately 19 miles east to the Dry Frio River.

3.6.5 Project Evaluation

A summary of the project costs and physical data for the Indian Creek Project is provided in Table 3.6-1. Much of this data is also presented graphically in Figure 3.6-3. As indicated in the table and figure, the optimal site development is a reservoir capacity of 61,750 acft, which could result in 29,307 acft/yr of recharge enhancement under average conditions at a unit cost of $343/acft/yr. Recharge under drought conditions could be enhanced by 18,596 acft/yr at a unit cost of $540/acft/yr.

A graph illustrating the natural recharge and the recharge enhancement that could result from development of the optimum size Indian Creek Project is shown in Figure 3.6-4 for the 56-year period of record from 1934 through 1989.

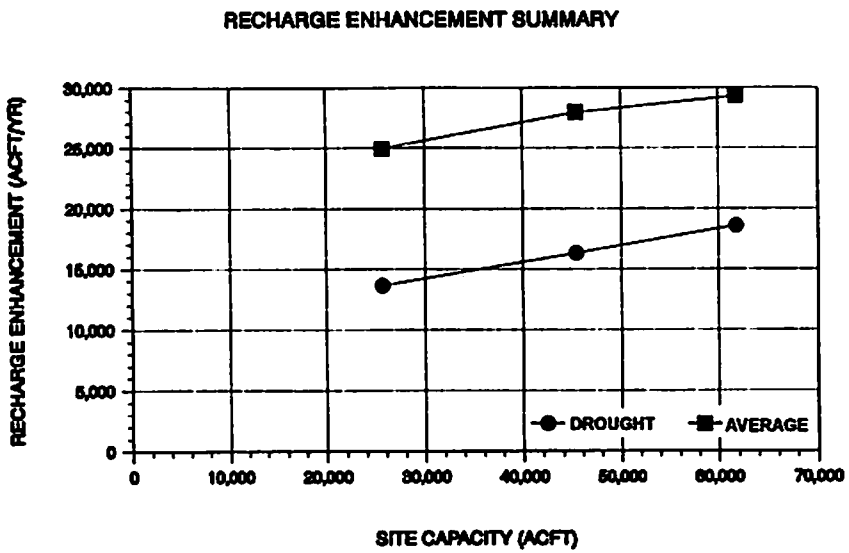
Figure 3.6-5 shows the frequency of various storage levels for the optimum size project. It indicates that, on the average, the reservoir would be empty only 30 percent of the time. It also shows that 10 percent of the time the reservoir would be full. This graph helps to illustrate the tremendous potential this project has for recharging the Edwards Aquifer through the storage and diversion of water captured in the Nueces River Basin.

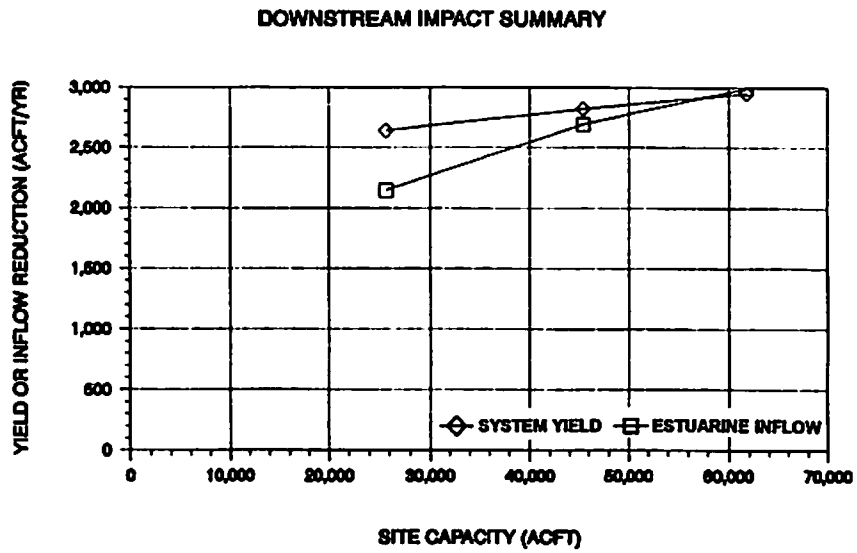
The calculation of potential recharge enhancement and, therefore, the unit cost of enhancement is a function of the natural percolation rate used for the reservoir in the model. Uncertainties exist regarding the natural percolation rate and subsequent movement of the ground water at the Indian Creek site. Work required to address these uncertainties is beyond the scope of this study. Further geologic and hydrogeologic investigations are recommended to obtain a better understanding of these issues at the Indian Creek site and determine the most beneficial and cost effective means of developing this significant potential water source.

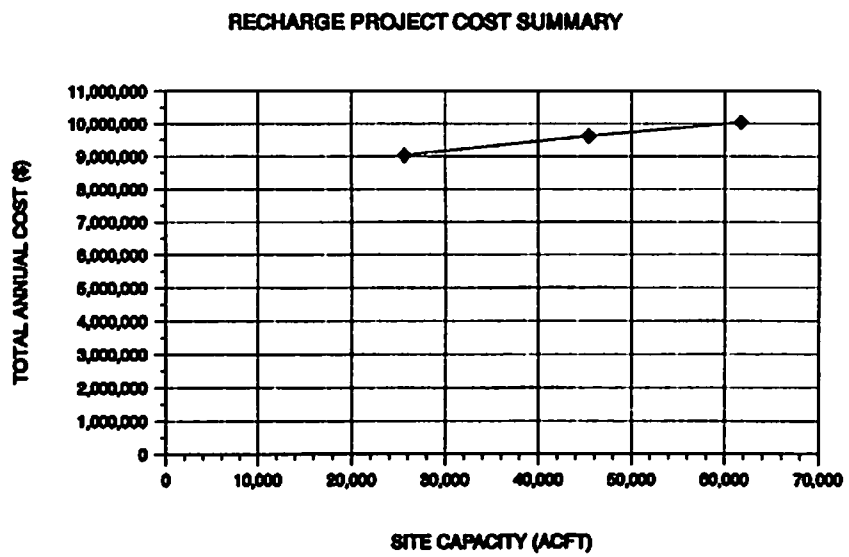
Table 3.6-1
Indian Creek Project Cost and Data Summary

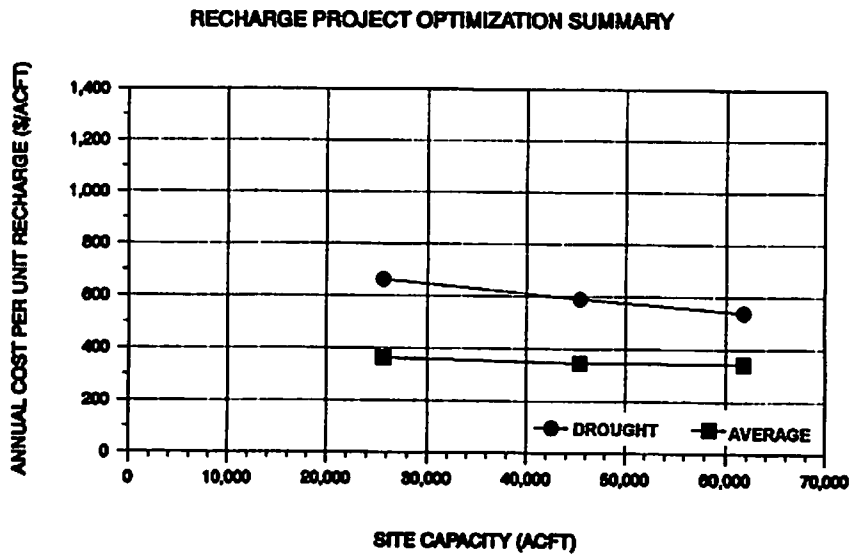
Physical Data			
Recharge Pool:			
Capacity (acft)	25,668	45,385	61,750
Surface Area (ac)	1,982	2,890	3657
Elevation (ft-msl)	980.0	988.0	993.0
Spillway Elevation (ft-msl)	970/1008	978/1008.5	983/1009.2
Spillway Width (ft)	300/4000	400/4000	500/4000
25-Year Flood Pool[1]:			
Elevation (ft-msl)	997.5	1000.7	1002.6
Surface Area (ac)	4,485	5,133	5,541
50-Year Flood Pool[1]:			
Elevation (ft-msl)	1002.0	1004.6	1006.1
100-Year Flood Pool[1]:			
Elevation (ft-msl)	1010.6	1012.1	1013.1
Surface Area (ac)	7,279	7,584	7,772
Dam Type	Embankment	Embankment	Embankment
Top of Dam Elevation (ft-msl)	1026.0	1026.2	1026.3
Streambed Elevation (ft-msl)	924.4	924.4	924.4
Hydrologic Data			
Recharge Enhancement (acft/yr):			
Drought Conditions	13,649	16,333	18,596
Average Conditions	24,946	27,984	29,307
Median Conditions	21,619	31,434	33,462
CC/LCC System Yield Reduction (acft/yr)	2,639	2,824	2,953
Median CC/LCC System Storage Reduction (%)	0.00	0.00	0.00
Estuarine Inflow Reduction (acft/yr)	2,146	2,693	2,998
Summary of Project Costs			
Dam, Spillway, and Appurtenant Works	$44,726,980	$47,697,967	$50,335,771
Diversion Dam, Pump Station, & Pipeline	$15,200,000	$15,200,000	$15,200,000
Road Relocations	$0	$0	$0
Land Acquisition	$5,585,000	$5,931,900	$6,148,300
Environmental Mitigation	$623,997	$909,865	$1,151,341
Engineering, Legal, Financial, and Misc.	$13,227,195	$13,947,946	$14,567,082
Total Capital Cost	$79,363,173	$83,687,678	$87,402,495
Annual Capital Cost (25 years @ 7.5% interest)	$7,118,877	$7,506,785	$7,840,004
Operations and Maintenance (annual)	$882,658	$1,007,312	$1,042,713
Water Rights Mitigation (annual)	$1,027,760	$1,107,343	$1,161,384
Total Annual Cost	$9,029,295	$9,621,440	$10,044,101
Annual Cost / Unit Recharge Enhancement:			
Drought Conditions ($/acft/yr)	$662	$589	$540
Average Conditions ($/acft/yr)	$362	$344	$343

Shaded column represents optimum size of project based on lowest unit cost for average conditions.
[1]Flood pools are based on reservoirs being empty at beginning of flood.
Refer to Appendix B for general notes on which project costs and data are based.









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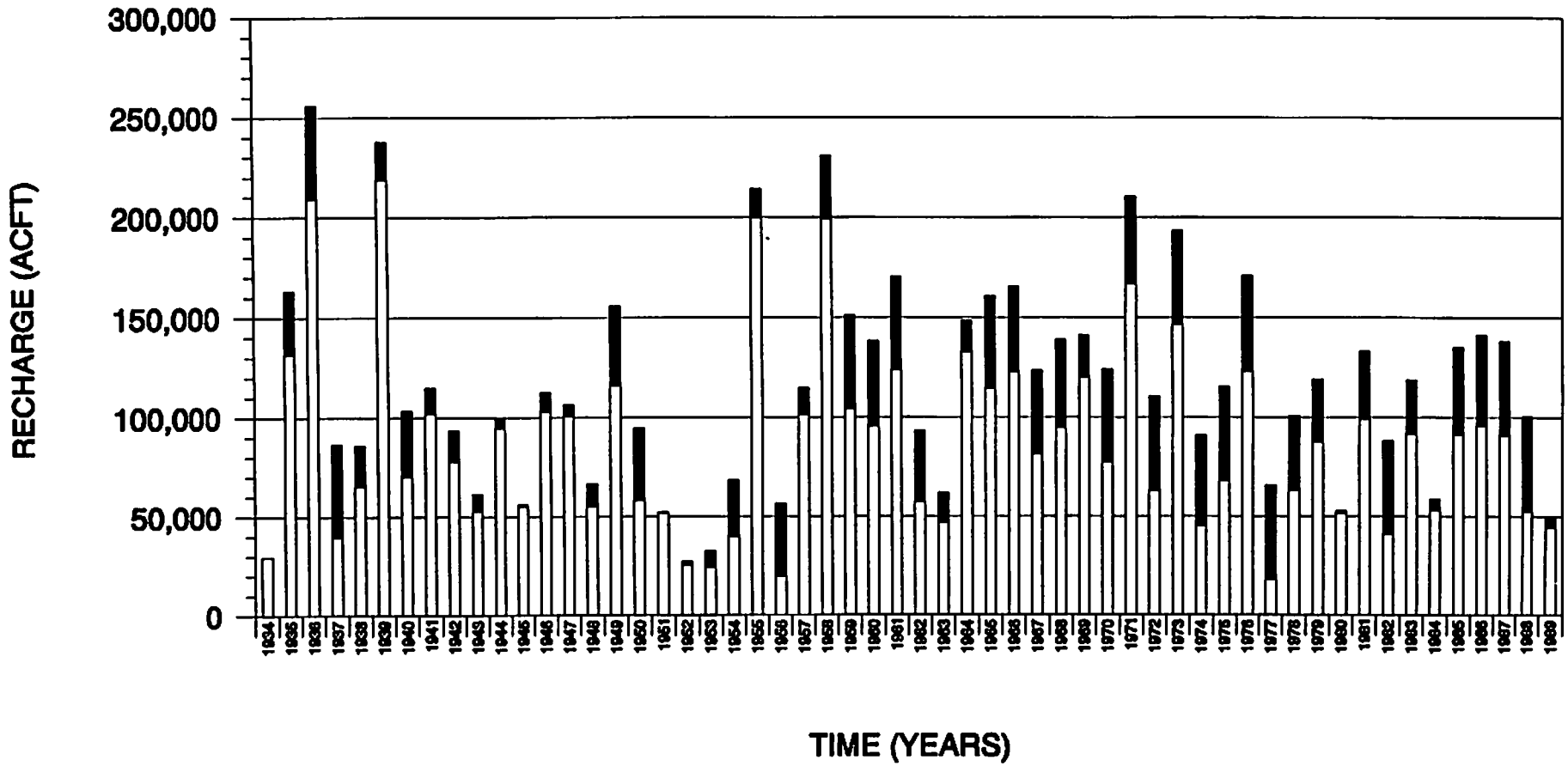
INDIAN CREEK
EVALUATION SUMMARY

FIGURE 3.6 - 3



INCLUDES MAX DIVERSION OF 2,000 ACFT/MO TO DRY FRIO RIVER

□NATURAL RECHARGE ■RECHARGE ENHANCEMENT

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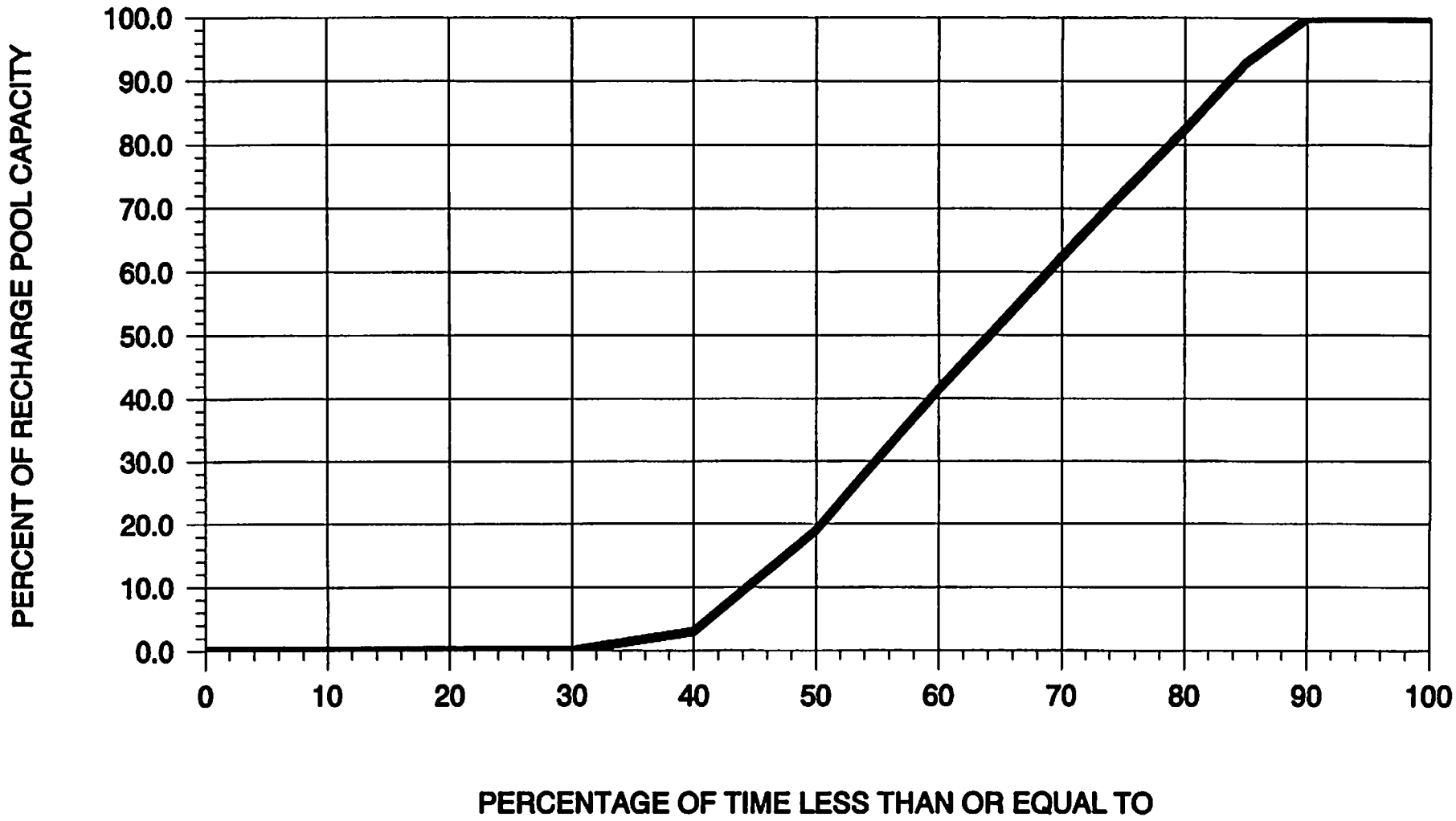
INDIAN CREEK PROJECT
ANNUAL RECHARGE

FIGURE 3.6 - 4



RECHARGE POOL CAPACITY = 61,750 ACFT

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ENHANCEMENT PROJECT - PHASE IVA

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INDIAN CREEK PROJECT
STORAGE FREQUENCY

FIGURE 3.6 - 5

SECTION 4

4.0 DEVELOPMENT OF RECHARGE ENHANCEMENT PROGRAM

A range of storage capacities was examined for each project in order to determine the optimum size reservoir. The optimum reservoir capacity selected during the Phase III Study program development was used as a starting point, and sizes smaller and larger were selected to establish a range. In determining the range of storage capacities to evaluate, consideration was given to known site constraints that would increase project costs, such as major road relocations. Generally, three different storage capacities were evaluated for each recharge project. In some instances this did not reveal a clear optimum and it was necessary to evaluate a fourth size. The optimum size reservoir capacity for each project was selected on the basis of the minimum unit cost of recharge enhancement under average conditions.

Two major findings became apparent during the individual project evaluations. These findings include:

1) The high natural capability of the Dry Frio River to recharge the Edwards Aquifer results in little water being available for recharge enhancement. Since there is minimal recharge enhancement, the unit cost is very expensive. Therefore, no further consideration was given to the Lower Dry Frio Project in the recharge enhancement program development, and it is not recommended for development at this time.

2) Uncertainties exist regarding the amount and subsequent cost of recharge enhancement at the Indian Creek Project because of questions about the natural infiltration rate and movement of ground water in the vicinity of the site. These questions prohibit the inclusion of the Indian Creek Project in the recharge enhancement program recommended for development at this time. However, the Indian Creek Project holds tremendous potential for recharge and management of the Edwards Aquifer, especially during drought, and recommendations are provided for additional studies to determine the most cost effective means of developing this significant potential water source.

4.1 Sizing of Recommended Projects

On the basis of this study, the Lower Verde, Lower Hondo, Lower Sabinal, and Lower Frio Projects are believed to be ready to move forward with the preliminary design and permitting phase at this time. The recommended size of each project was determined by examining the unit cost of recharge enhancement under average conditions for several storage capacities at each of the four sites. The sizing procedure began by selecting the storage capacity of each project having the lowest unit cost (i.e., optimum size) and continued by enlarging the project up to the maximum storage capacity considered.

Table 4.1-1 illustrates this process. The Lower Sabinal Project at its optimum size represents the lowest unit cost of recharge enhancement of the four projects considered. The next most cost effective quantity of recharge enhancement is obtained by developing the Lower Verde Project at its optimum size. As shown in the upper portion of the table, each project developed at its optimum size represents the first four steps of the program development. The program development continues by evaluating the incremental cost to enlarge each project above its optimum size, up to the maximum storage capacity considered for each of the projects. This process is shown in the lower portion of Table 4.1-1. For example, enlarging the storage capacity of the Lower Sabinal Project by 3,500 acft above the optimum size storage capacity of 8,750 acft results in 1,099 acft of additional recharge enhancement under average conditions and no additional recharge enhancement during the drought. The incremental cost to enlarge the project and obtain the additional average recharge enhancement is $272/acft/yr.

Graphical presentations of recharge enhancement program development are shown in Figures 4.1-1 and 4.1-2. The points on the graphs correspond to either the optimum size unit cost (No. 1-4) or incremental cost (No. 5-10) as presented in Table 4.1-1. A well-defined break point occurs in the program development process at point No. 4, beyond which significantly greater investment would yield little additional recharge enhancement. This point represents the four projects developed to their optimum size, and provides the most water for each dollar spent on the program. No additional recharge enhancement is obtained for the 10-year drought period by enlarging the projects above their optimum size, as indicated by the far right column in Table 4.1-1 and the lower curve in Figure 4.1-2.

Table 4.1-1a
Recharge Enhancement Program Development

RECOMMENDED OPTIMUM SIZE PROJECTS

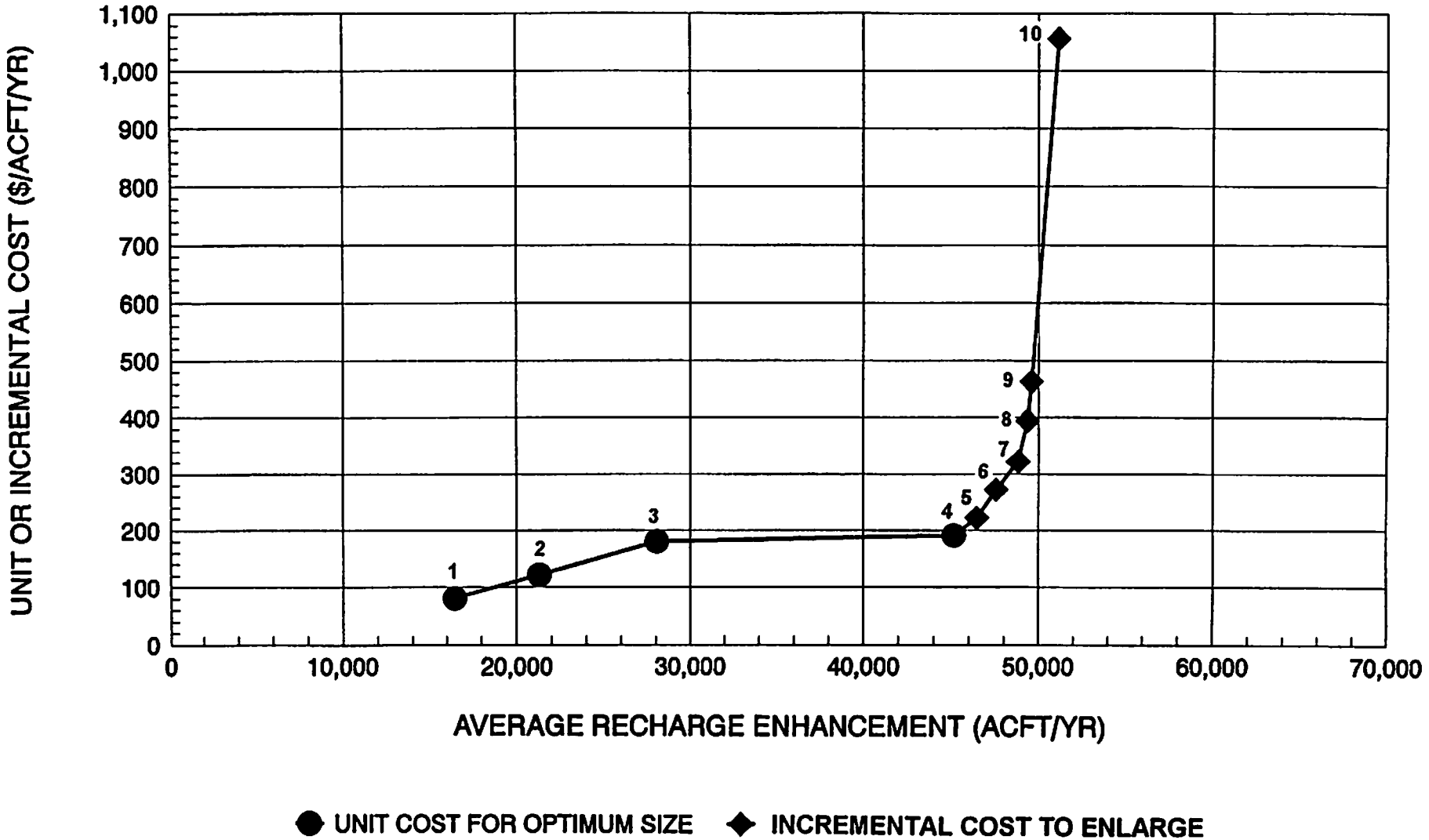
Unit Cost Ranking*	Unit Cost ($/acft/yr)	Optimum Size Projects	Optimum Storage Capacity (acft)	Annual Cost ($)	Recharge Enhancement (acft/yr)	
					Average Conditions	Drought Conditions
1	81	Lower Sabinal	8,750	1,335,379	16,442	2,358
2	122	Lower Verde	3,600	590,084	4,850	1,719
3	180	Lower Hondo	2,800	1,218,701	6,779	1,193
4	191	Lower Frio	17,500	3,258,211	17,064	3,980
		Subtotals	32,650	6,402,375	45,135	9,250

Table 4.1-1b
Recharge Enhancement Program Development

EVALUATION OF ENLARGED PROJECTS

Incremental Cost Ranking*	Incremental Cost to Enlarge ($/acft/yr)	Enlarged Projects	Enlarged Storage Capacity (acft)	Annual Cost ($)	Additional Recharge Enhancement (acft/yr)	
					Average Conditions	Drought Conditions
5	222	Lower Hondo	5,600	1,509,637	1,310	0
		Subtotals	35,450	6,693,311	46,445	9,250
6	272	Lower Sabinal	12,250	1,634,288	1,099	0
		Subtotals	38,950	6,992,220	47,544	9,250
7	322	Lower Sabinal	17,500	2,046,492	1,281	0
		Subtotals	44,200	7,404,424	48,825	9,250
8	394	Lower Verde	6,000	803,444	542	39
		Subtotals	46,600	7,617,784	49,367	9,289
9	463	Lower Verde	8,400	906,592	223	0
		Subtotals	49,000	7,720,932	49,590	9,289
10	1,056	Lower Frio	25,000	4,923,634	1,577	6
		Subtotals	56,500	9,386,355	51,167	9,295

*Ranking is based on unit or incremental cost of recharge enhancement for average conditions.



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EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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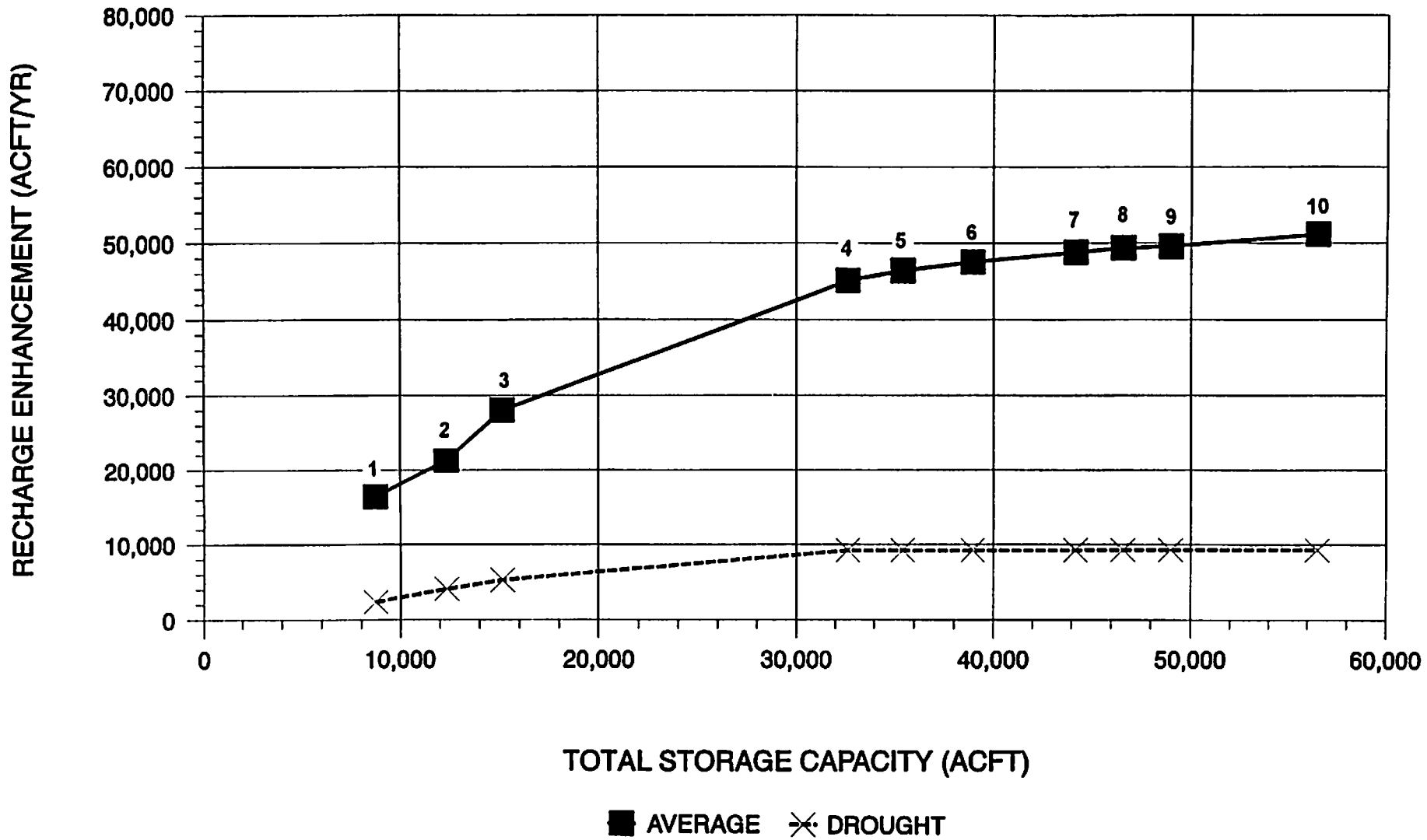
RECHARGE ENHANCEMENT PROGRAM DEVELOPMENT - COST SUMMARY

FIGURE 4.1 - 1



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RECHARGE ENHANCEMENT PROGRAM DEVELOPMENT - STORAGE SUMMARY

FIGURE 4.1 - 2

4.2 Summary of Recommended Recharge Enhancement Program

The recommended recharge enhancement program is comprised of the Lower Verde, Hondo, Sabinal, and Frio Projects, each constructed at their optimum size. A summary of this program is presented in Table 4.2-1. Development of the program would provide 45,135 acre-feet/year (acft/yr) of recharge enhancement under average conditions at an average unit cost of $142/acft/yr ($0.44 per 1,000 gallons). Recharge enhancement under drought conditions would be 9,250 acft/yr at an average unit cost of $692/acft/yr ($2.12 per 1,000 gallons).

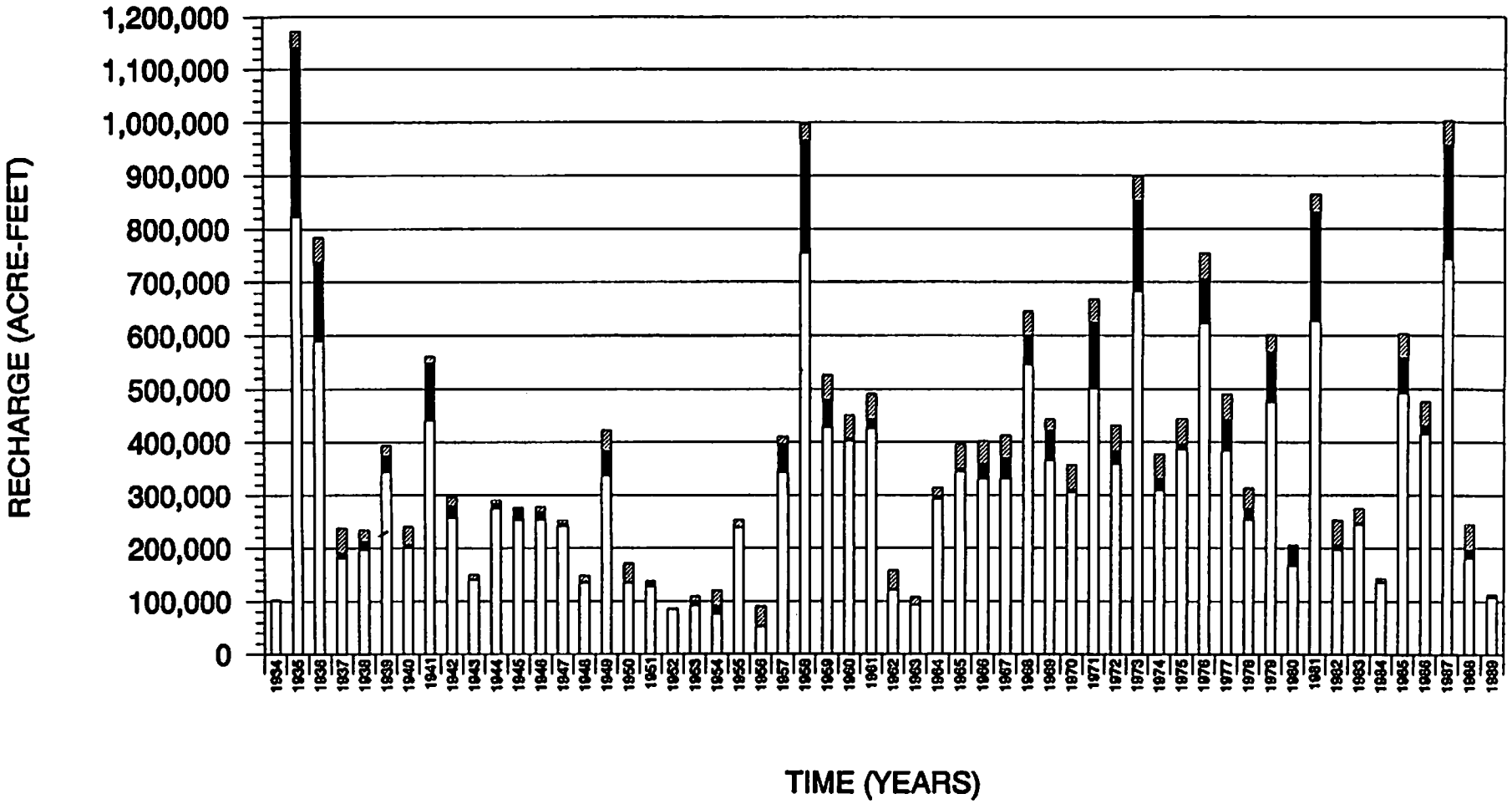
A graph showing how the annual recharge to the Edwards Aquifer occurring in the Nueces River Basin would be affected by implementation of the recommended program is presented in Figure 4.2-1. This figure illustrates: 1) natural recharge to the Edwards Aquifer; 2) recharge enhancement resulting from development of the recommended program of the four optimum size projects; and 3) an estimate of the recharge enhancement that could be provided by the Indian Creek Project. Recharge to the Nueces River Basin portion of the Edwards Aquifer would be increased by approximately 14 percent under average conditions and 6 percent under drought conditions with the implementation of the recommended recharge enhancement program. Including the estimates for the Indian Creek Project, recharge could be increased under average and drought conditions by 23 and 18 percent, respectively.

Impacts to the CC/LCC Reservoir System yield and reductions in inflows to the Nueces Estuary are shown for each project in Table 4.2-1. Reductions in the CC/LCC Reservoir System yield range from 170 to 1,229 acft/yr for each project, and represent 0.1 to 0.7 percent of the yield based on the current operating policy of the CC/LCC Reservoir System. The estimated cumulative impact on firm yield is about 2,954 acft/yr, or 1.8 percent. Reductions in the average Estuary inflow range from 728 to 2,594 acft/yr for each project, and represent 0.1 to 0.5 percent of the average annual inflow to the Nueces Estuary. The estimated cumulative impact on inflows to the Nueces Estuary is about 7,022 acft/yr, or 1.4 percent.

Table 4.2-1
Summary of Recommended Recharge Enhancement Program

Rank*	Project	Capacity (acft)	Surface Area (ac)	Annual Cost ($)	Average Conditions		Drought Conditions		Reduction in Average Estuarine Inflow (acft/yr)	Reduction in CC/LCC System Yield (acft/yr)
					Recharge Enhance-ment (acft/yr)	Cost/Unit Recharge Enhance-ment ($/acft/yr)	Recharge Enhance-ment (acft/yr)	Cost/Unit Recharge Enhance-ment ($/acft/yr)		
1	Lower Sabinal	8,750	454	1,335,379	16,442	81	2,358	566	2,566	1,229
2	Lower Verde	3,600	334	590,084	4,850	122	1,719	343	728	170
3	Lower Hondo	2,800	232	1,218,701	6,779	180	1,193	1,022	1,134	403
4	Lower Frio	17,500	1,099	3,258,211	17,064	191	3,980	819	2,594	1,152
	Total	32,650	2,119	6,402,375	45,135		9,250		7,022[E]	2,954[E]
	Average					142		692		

*Rank is based on Cost/Unit Recharge Enhancement for Average Conditions.
**Program includes projects with a Cost/Unit Recharge Enhancement for Average Conditions less than $192/acft/yr ($0.59/1,000 gallons).
[E]Estimated cumulative impact. Actual impact of constructing all four projects as a program could be less than that indicated.



NATURAL RECHARGE IN NUECES RIVER BASIN (INCLUDES EXISTING RECHARGE PROJECTS)

RECHARGE ENHANCEMENT FROM RECOMMENDED PROGRAM OF FOUR OPTIMUM PROJECTS

RECHARGE ENHANCEMENT FROM INDIAN CREEK PROJECT

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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ANNUAL RECHARGE TO NUECES RIVER BASIN PORTION OF EDWARDS AQUIFER

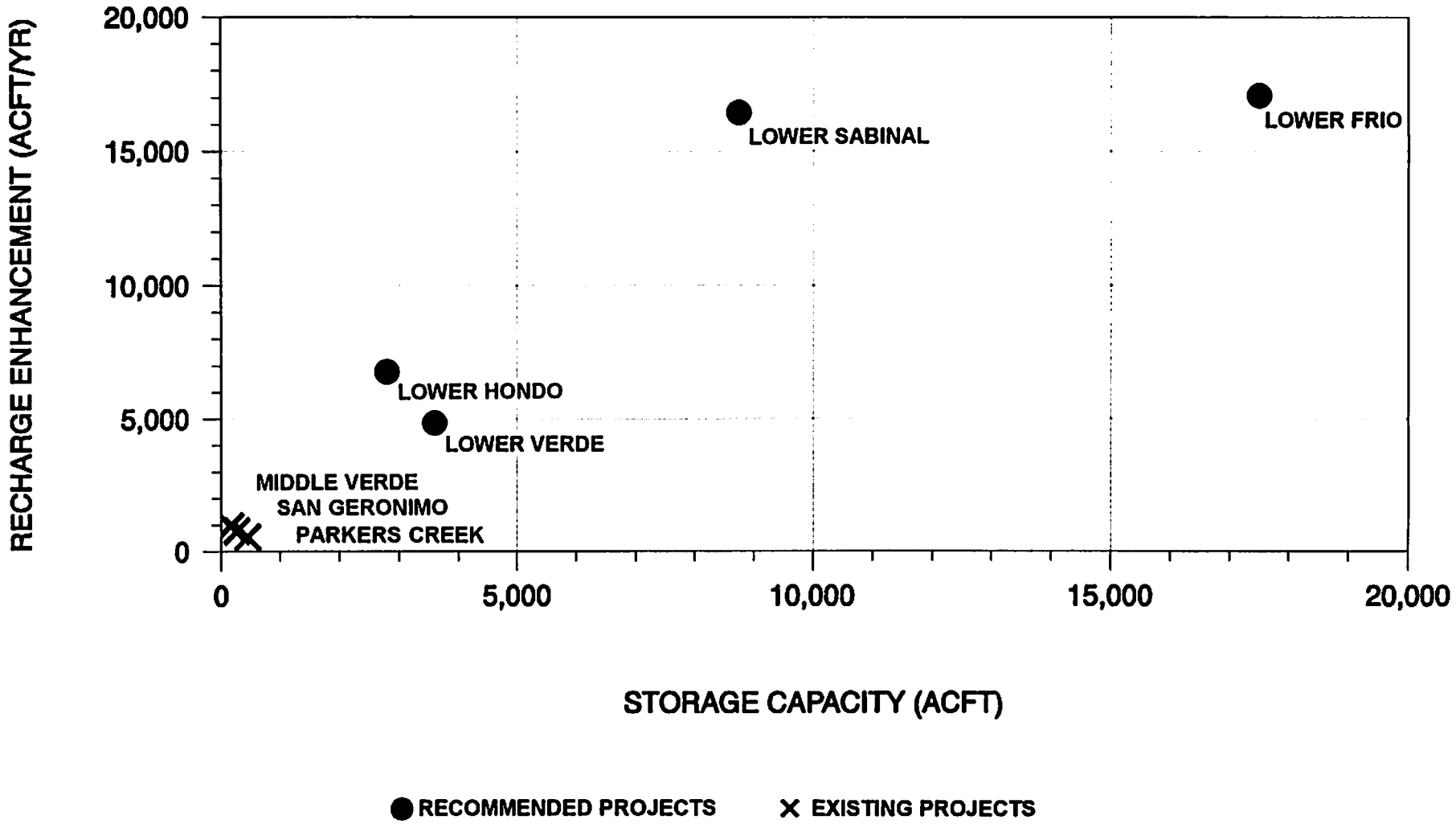
FIGURE 4.2 - 1

The total capital cost of the recommended recharge enhancement program is estimated to be $55.8 million and the total annual cost for this program would be about $6.4 million. A summary of capital and annual costs for each project is presented in Table 4.2-2.

Table 4.2-2
Cost Summary of Recommended Recharge Enhancement Program

	Lower Verde	Lower Hondo	Lower Sabinal	Lower Frio	Total
Storage Capacity (acre-feet)	3,600	2,800	8,750	17,500	32,650
Dam, Spillway, and Appurtenant Works	$2,857,013	$5,358,732	$6,724,748	$23,589,011	$38,529,504
Road Relocations	$510,000	$3,750,000	$0	$0	$4,260,000
Land Acquisition	$1,266,500	$397,000	$672,000	$1,266,100	$3,601,600
Environmental Mitigation	$23,161	$16,735	$36,172	$77,264	$153,332
Engineering, Legal, Permitting, Financial, and Misc.	$931,335	$1,904,493	$1,486,584	$4,986,475	$9,308,887
Total Capital Cost	$5,588,009	$11,426,960	$8,919,504	$29,918,850	$55,853,323
Annual Capital Cost (25 years @ 7.5% interest)	$501,244	$1,024,998	$800,080	$2,683,721	$5,010,043
Operations and Maintenance	$11,428	$21,435	$26,899	$94,356	$154,118
Water Rights Mitigation	$77,412	$172,268	$508,400	$480,134	$1,238,214
Total Annual Cost	$590,084	$1,218,701	$1,335,379	$3,258,211	$6,402,375

To better understand the magnitude of the proposed recharge enhancement projects, a comparison between three of the existing recharge projects and the four recommended projects is illustrated in Figure 4.2-2. The figure illustrates that the proposed projects are at least an order of magnitude larger in terms of their storage capacity and the recharge enhancement that could potentially be provided to the Edwards Aquifer.



RECOMMENDED PROJECTS ARE AT OPTIMUM SIZE

NUECES RIVER BASIN
EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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COMPARISON OF EXISTING AND RECOMMENDED RECHARGE PROJECTS

FIGURE 4.2 - 2

SECTION 5

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

1) Substantial enhancement of recharge to the Edwards Aquifer within the Nueces River Basin can be achieved through the development of a program of four optimum size recharge projects located near the downstream boundary of the recharge zone. These include the Lower Sabinal, Lower Verde, Lower Hondo, and Lower Frio Projects, each constructed at their optimum size, as determined in this study. The total combined storage capacity of these four projects would be 32,650 acft. Implementation of this program would result in average recharge enhancement of 45,135 acft/yr at an average unit cost of $142/acft/yr ($0.44 per 1,000 gallons). Recharge under drought conditions would be increased by 9,250 acft/yr at an average unit cost of $692/acft/yr ($2.12 per 1,000 gallons).

2) Reductions in the CC/LCC Reservoir System yield range from 170 to 1,229 acft/yr for the four projects comprising the recommended recharge enhancement program. This represents 0.1 to 0.7 percent of the yield based on the current operating policy of the CC/LCC Reservoir System. The estimated cumulative impact on firm yield is about 2,954 acft/yr, or 1.8 percent. Reductions in the average estuarine inflow range from 728 to 2,594 acft/yr for the four projects, which represents 0.1 to 0.5 percent of the average annual inflow to the Nueces Estuary. The estimated cumulative impact on inflows to the Nueces Estuary is about 7,022 acft/yr, or 1.4 percent.

3) The Indian Creek Project at its optimum size (61,750 acft), with diversion to the Dry Frio River (maximum of 2,000 acft/month), could enhance recharge to the Edwards Aquifer by 29,307 acft/yr at an average unit cost of $343/acft/yr. Recharge under drought conditions could be increased by 18,596 acft/yr at an average unit cost of $540/acft/yr. However, questions about the natural infiltration rate and movement of ground water in the vicinity of the Indian Creek site create uncertainties regarding the amount and unit cost of recharge enhancement. These questions prohibit the inclusion of the Indian Creek Project in the recharge program recommended for development at this time.

4) The natural capability of the Dry Frio River to recharge the Edwards Aquifer is very high. The additional recharge obtained by constructing a storage reservoir would be minimal, resulting in a very expensive unit cost of recharge enhancement.

5) Although the site geology is complex at certain projects, no fatal geologic or geotechnical flaws are evident that would prohibit development of the proposed recharge projects.

6) No fatal environmental flaws are believed to exist at the project sites.

5.2 Recommendations

1) The EUWD should proceed with development of the four optimum size projects which comprise the recommended recharge enhancement program. A schedule for development of the recommended program is provided in Section 5.3.

2) Additional studies should be performed to determine the most beneficial and cost effective means of developing the Indian Creek Project. These studies should include: 1) comprehensive geologic and hydrogeologic investigations at both the reservoir site and along the Dry Frio River to address natural infiltration rates and ground water movement in the various geologic units; and 2) hydrologic modelling using daily simulation to examine other potential project configurations, including the possibility of constructing a small channel dam with a large capacity pump station and pipeline to the Dry Frio River. An estimated schedule for the ultimate development of the Indian Creek Project is provided in Section 5.4.

3) No further consideration should be given to locating a major recharge enhancement project on the Dry Frio River at this time. However, the possibility of using a series of small channel dams on the Dry Frio River in conjunction with diversion from the Indian Creek Project could be investigated as part of the further studies recommended for the Indian Creek Project.

The following recommendations are presented for the EUWD's consideration assuming a decision is made to proceed with development of the recommended recharge enhancement program:

4) Initiate discussion as soon as possible with the City of Corpus Christi with the intent of negotiating an agreement to mitigate the relatively small impacts on the yield of the CC/LCC Reservoir System. Evaluate the potential impact of the recommended recharge enhancement program (four projects considered as a group) on the yield of the CC/LCC Reservoir System subject to the current operational constraints and estuarine inflow requirements.

5) Secure permission from landowners to perform various field investigations that will be required for preliminary design and permitting of the recharge enhancement program. These investigations include geologic and geotechnical field work, ground control surveys for detailed topographic mapping, and environmental and archaeological field survey.

6) Initiate the Preliminary Design and Permitting Phase for the recommended program to prepare supporting documents for the permit applications that will be required. Refine the size and cost of each project in preliminary design for permitting based on detailed topographic maps, geologic and geotechnical investigations, environmental considerations, potential sites of archaeological significance, and relocations. Update estimates of recharge enhancement for permitting given refined sizing.

7) Install additional streamflow gages upstream of the proposed Lower Verde, Lower Hondo, and Lower Frio Projects to facilitate calculating recharge to the Edwards Aquifer following construction of the projects. If possible, gages should be installed as part of the EUWD's current streamflow and precipitation gage installation program to allow time prior to construction to properly calibrate the gages.

8) Perform intensive high baseflow stream surveys on streams across the recharge zone at each recommended project site to further define loss rates and baseline recharge conditions. Measurement points and methods should duplicate the 1983 USGS stream loss survey (Ref. 35) at a minimum, with possible additional interim measuring sites to better define losses and geologic features in the proposed recharge project pools.

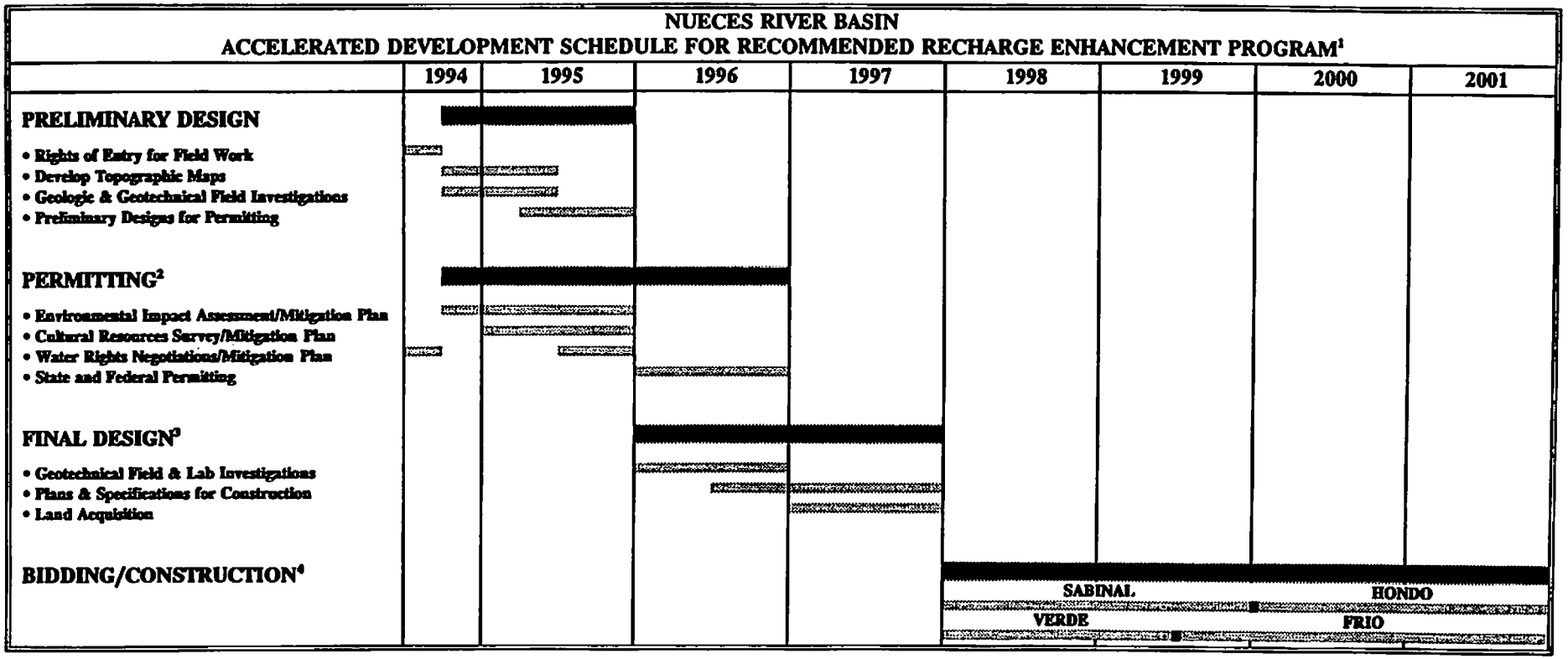
9) Incorporate the installation of well level recorders on selected borings used for geologic and geotechnical investigations at, and adjacent to, project sites to determine the response and movement of ground water to recharge, both prior to and after project construction.

5.3 Development Schedule for Recommended Recharge Enhancement Program

A schedule for development of the four projects that comprise the recommended recharge enhancement program is presented in Figure 5.3-1. Under this scenario, the first recharge enhancement project would be completed and "on-line" by mid to late 1999. All four projects would be completed by the end of the year 2001. The development schedule presented is considered to be accelerated for the following reasons:

1) It assumes permission from landowners to perform the various field investigations can be secured by October 1, 1994;

2) It assumes water rights negotiations with the City of Corpus Christi can commence by July 1, 1994, and an understanding between the parties regarding the intent to negotiate an agreement for mitigating impacts to the CC/LCC Reservoir System yield can be reached prior to proceeding with the Preliminary Design and Permitting Phase by October 1, 1994;



[1] Recommended recharge enhancement program consists of the Lower Sabinal, Verde, Hondo, and Frio projects constructed at their optimum size.
[2] Schedule based on conducting Fall '94 and Spring '95 environmental field surveys, and on minimal opposition to permit application.
[3] Schedule based on final design being concurrent with permitting. Add 1 to 1.5 years to schedule if final design is initiated after permits are obtained.
[4] Schedule based on overlapping construction of projects. Length of construction for each project would vary from 1.5 to 2.5 years, depending on complexity of site, size of dam, and required road relocations.

NUECES RIVER BASIN
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ENHANCEMENT PROJECT - PHASE IVA

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ACCELERATED PROGRAM DEVELOPMENT SCHEDULE

FIGURE 5.3 - 1

3) It is based on being able to perform the initial environmental field survey in the Fall of 1994, and on minimal opposition during permitting;

4) It is based on final designs proceeding prior to permits being obtained; and

5) It assumes that construction of the projects would overlap in order to complete all four projects within a four year period.

5.4 Development Schedule for the Indian Creek Project

An estimated development schedule for the Indian Creek Project is presented in Figure 5.4-1. Prior to proceeding with preliminary design and permitting work, additional studies, as recommended in Section 5.2, would need to be performed. The complexities associated with the development of this large project, including additional environmental concerns, downstream water rights impacts, and project investigations and designs necessitate additional time to conduct each major phase of the project. As shown in Figure 5.4-1, it is estimated that the Indian Creek Project could possibly be completed by about the year 2006.

NUECES RIVER BASIN
DEVELOPMENT SCHEDULE FOR INDIAN CREEK PROJECT

	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005
GEOHYDROLOGY & HYDROLOGY STUDIES	■	■									
PRELIMINARY DESIGN & PERMITTING[1]			■	■	■						
FINAL DESIGN & LAND ACQUISITION						■	■	■			
BIDDING & CONSTRUCTION								■	■	■	■

[1]Schedule based on minimal opposition to permit application. Significant opposition could cause unknown delays.

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EDWARDS AQUIFER RECHARGE
ENHANCEMENT PROJECT - PHASE IVA

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INDIAN CREEK PROJECT
DEVELOPMENT SCHEDULE

FIGURE 5.4 - 1

REFERENCES

1. Baker, Victor R., "Flood Hazards along the Balcones Escarpment in Central Texas, Alternative Approaches to their Recognition, Mapping, and Management", Bureau of Economic Geology, Geologic Circular 75-5, University of Texas at Austin, 1975.

2. Bureau of Economic Geology, "Geological Atlas of Texas, San Antonio Sheet," The University of Texas at Austin, 1977.

3. Carnahan, B. and Wilkes, J.O., Digital Computing and Numerical Methods, John Wiley and Sons, Inc., 1973.

4. Chow, Ven Te, D.R. Maidment, and L.W. Mays, Applied Hydrology, McGraw-Hill Book Co., 1988.

5. Freese and Nichols, Inc., "Report on Availability of Additional Surface Water Supply from the Nueces River Between Uvalde and Three Rivers," December, 1982.

6. Fugro-McClelland (Southwest), Inc., "Geotechnical Consultation - Nueces River Basin, Edwards Aquifer Recharge Enhancement Projects, Phase IVA," HDR Engineering, Inc., December 17, 1993.

7. Gilliland, C. E., Patrick, H.P., and Vanderburg, T.J., "Rural Land Values in the Southwest: First Half, 1993," Real Estate Center, A & M University, College Station, TX, August, 1993.

8. Gilliland, C. E., and Vanderburg, T.J., "Texas Rural Land Prices, 1992," Real Estate Center, A & M University, College Station, Texas, January, 1994.

9. HDR Engineering, Inc. (HDR), "Evaluation of Alternative Release Programs for the Choke Canyon / Lake Corpus Christi System, Phase I, Draft," City of Corpus Christi, March, 1993.

10. HDR, Guadalupe - San Antonio River Basin Recharge Enhancement Study," Vols. 1, 2, and 3, Edwards Underground Water District, September, 1993.

11. HDR, "Nueces Estuary Regional Wastewater Planning Study - Phase I," South Texas Water Authority, November, 1991.

12. HDR, "Nueces Estuary Regional Wastewater Planning Study - Phase II," South Texas Water Authority, June, 1993.

13. HDR Engineering, Inc., "Nueces River Basin Regional Water Supply Planning Study-Phase I," Vols. 1, 2, and 3, Nueces River Authority, May, 1991.

14. HDR, "Nueces River Basin Regional Water Supply Planning Study, Phase III - Recharge Enhancement," Nueces River Authority, November, 1991.

15. Hoggan, Daniel H., Computer-Assisted Floodplain Hydrology & Hydraulics, McGraw-Hill Book Co., 1989.

16. Hydrologic Engineering Center, "HEC-1 Flood Hydrograph Package", U.S. Army Corps of Engineers, Davis, Ca, September, 1990.

17. Hydrologic Engineering Center, "HEC-FFA Flood Frequency Analysis User's Manual," U.S. Army Corps of Engineers, May, 1992.

18. Hydrologic Engineering Center, "HMR52 Probable Maximum Storm (Eastern United States) Users Manual," U.S. Army Corps of Engineers, March, 1984.

19. LBG-Guyton Associates, "Reconnaissance-Level Geohydrologic Evaluation of the Nueces River Basin, Edwards Aquifer Recharge Enhancement Projects," HDR Engineering, Inc., August and November, 1993.

20. Linsley, Ray K., Jr., M.A. Kohler, and J.L.H. Paulhus, Hydrology for Engineers, McGraw-Hill Book Co., Third Edition, 1982.

21. National Oceanic and Atmospheric Administration, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 51, June, 1978.

22. National Weather Service, "Comparison of Generalized Estimates of Probable Maximum Precipitation with Greatest Observed Rainfall, Technical Report NWS 25, National Oceanic and Atmospheric Administration, 1987.

23. National Weather Service, "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States,", NOAA Technical Memorandum NWS Hydro-35, Office of Hydrology, Silver Spring, Md, June, 1977.

24. National Weather Service, "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years," Technical Paper No. 40, U.S. Department of Commerce, May, 1961.

25. Patterson, James L., "Floods in Texas, Magnitude and Frequency of Peak Flows", Texas Water Commission, Bulletin 6311, December, 1963.

26. Soil Conservation Service (SCS), "Soil Survey of Medina County, Texas," USDA, August, 1977.

27. SCS, "Soil Survey of Uvalde County, Texas," USDA, 1981.

28. Texas Water Development Board (TWDB), "Evaporation Data in Texas, Compilation Report, January 1907 - December 1970," Report 192, June, 1975.

29. TWDB, "Monthly Reservoir Evaporation Rates for Texas, 1940 through 1965," Report 64, October, 1967.

30. U.S. Army Corps of Engineers (USCE), "Kerrville Ponding Dam," Phase I Inspection Report, National Dam Safety Program, Kerr County, Texas, Inventory Number TX 04650, Report prepared by Texas Department of Water Resources, January, 1981.

31. USCE, "Medina Dam," Phase I Inspection Report, National Dam Safety Program, Medina County, Texas, Inventory Number TX 01787, Report prepared by Texas Department of Water Resources, January, 1979.

32. USCE, "Olmos Dam," Phase I Inspection Report, National Dam Safety Program, Bexar County, Texas, Inventory Number TX 04313, Report prepared by Texas Department of Water Resources, June, 1978.

33. USCE, "Survey Report on Edwards Underground Reservoir, Guadalupe, San Antonio, and Nueces Rivers and Tributaries, Texas," Appendix II, Hydrology and Hydraulic Design, 1965.

34. U.S. Department of Commerce, Weather Bureau, "The Climate of Central and Coastal Watersheds", Asheville, North Carolina, January, 1961.

35. U.S. Geological Survey, "Streamflow Losses Along the Balcones Fault Zone, Nueces River Basin, Texas," Water-Resources Investigations Report 83-4368, Austin, Texas, 1983.

36. Viessman, Warren, Jr., J.W. Knapp, G.L. Lewis, and T.E. Harbaugh, Introduction to Hydrology, Harper & Row, Second Edition, 1977.

37. Wanakule, N. and Anaya, R., "A Lumped Parameter Model for the Edwards Aquifer, Draft," Technical Report No. 163, Texas Water Resources Institute, Texas A&M University System, College Station, Texas, 1994.

38. Water Resources Council, "Guidelines for Determining Flood Flow Frequency," Bulletin 17B, U.S. Geological Survey, March, 1982.

APPENDIX A

HYDROGEOLOGICAL SETTINGS

Proposed Recharge Structure	Geologic and Hydrologic Factors								Hydro-geologic Setting Index
	Depth to Water	Water-Table Configuration	Streamflow Losses	Vadose Zone	Soils	Aquifer Media	Hydraulic Properties	Geologic Structure	
Lower Verde	6	5	3	6	5	6	5	6	42
Lower Hondo	6	4	5	6	6	6	5	4	42
Lower Sabinal	6	6	4	4	6	4	4	6	40
Lower Frio	6	6	4	6	6	6	5	5	44
Lower Dry Frio	6	6	6	5	6	5	4	4	42
Indian Creek	5	4	1	4	5	4	5	3	31

APPENDIX B

APPENDIX B

General Notes for Project Cost and Data Summary Tables

1) Calculated water surface elevations are presented to the nearest tenth of a foot for comparative purposes only, as they are based on USGS topographic mapping generally having a 10-foot or 20-foot contour interval.

2) Dam, Spillway, and Appurtenant Works Costs include 20 percent contingencies.

3) Road Relocation Costs are based on major state highways and associated bridges being raised or relocated above the 50-year flood pool elevation, in accordance with current Texas Department of Transportation criteria.

4) Land Acquisition Costs are based on purchase of land up to the 25-year flood level at $800/acre, easement purchase between the 25- and 100-year flood levels at $500/acre, and $50,000/unit for purchase of habitable structures below the 100-year flood pool level.

5) Environmental Mitigation Costs are calculated by inflating the costs in the Phase III Study by 2.5 percent per year from 1991 to 1994.

6) Engineering, Legal, Financial, and Misc. Costs are assumed equal to 20 percent of related capital costs.

7) Annual Capital Costs are based on a 25-year finance period at an annual interest rate of 7.5 percent.

8) Operations and Maintenance Costs are based on 0.4 percent of Total Capital Cost, and include power costs for the diversion works associated with the Indian Creek Project.

9) Water Rights Mitigation Costs are based on $374/acft of Choke Canyon/Lake Corpus Christi System Yield Reduction and $19/acft of Estuarine Inflow Reduction.

10) All estimated costs are in 1994 dollars.