FLOOD CONTROL STUDY
SAN PATRICIO COUNTY, TEXAS

SAN PATRICIO COUNTY DRAINAGE DISTRICT

JULY 1987

HDR Infrastructure, Inc.
A Centerra Company

NAISMITH ENGINEERS, INC.
Consulting Engineers

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July, 1987

Prepared by

HDR Infrastructure, Inc.
Austin, Texas

Naismith Engineers, Inc.
Corpus Christi, Texas

Funded by

Texas Water Development Board
and
San Patricio County Drainage District

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EXECUTIVE SUMMARY

San Patricio County is frequently subjected to intense precipitation associated with both tropical storms and frontal storm events due to its location in the Coastal Bend of Texas. This fact, coupled with the generally flat topography and clayey soils of the coastal plain, makes the County naturally susceptible to flooding. In addition, flooding problems resulting from naturally poor drainage may be compounded by manmade influences. These influences may include inadequate drainage channels, undersized structures at stream crossings, developments in the floodplain, and a lack of consistent drainage design criteria. Flood control can, however, be achieved on a county-wide basis by the systematic mitigation of these negative influences. The primary objectives of this Flood Control Study are to assess the magnitude and causes of specific flooding problems affecting incorporated communities and rural areas within San Patricio County and to evaluate alternative means of resolving these problems from both an engineering and an economic perspective.

The objectives of this study were met by a three phased approach. The first phase involved identification of specific flooding problems, data collection, and selection of appropriate hydrologic and hydraulic design criteria. These tasks were accomplished primarily through coordination with the San Patricio County Drainage District Staff and Board of Directors, County

Commissioners, City Councils and staffs, and various governmental agencies. In the second phase, various alternative flood control measures were developed and evaluated in order to resolve specific flooding problems identified in the first phase. Flood control alternatives considered include both structural and non-structural measures, and each alternative was evaluated and ranked on the basis of benefit-cost analyses. The third phase of the study involved the preparation of all deliverable products. Deliverables include a watershed boundary map, FEMA floodplain boundary map, aerial mapping of the entire County showing existing and proposed 100-year floodplain boundaries in the areas studied, channel and water surface profiles, computer models, a Drainage Criteria and Design Manual, and this comprehensive Flood Control Study report.

Hydrologic studies of selected drainage basins within the County resulted in estimates of peak discharge associated with given design return periods at key points along major drainageways. Detailed statistical analyses of annual maximum discharge records for Chiltipin Creek at Sinton indicated that the Region 2 Flood-Frequency equations developed by the U.S. Geological Survey would provide a reasonable means of estimating peak discharge for watersheds in San Patricio County. Stream hydraulics and water surface profiles were then computed using the U.S. Army Corps of Engineers HEC-2 computer program. The HEC-2 program is capable of computing water surface profiles for channels of any configuration and slope and can readily incorporate the impacts of bridges, culverts, weirs, levees, and dams. Once the magnitude of each flooding problem was assessed, alternative structural and non-structural flood control measures were considered to reduce or alleviate future flood damages. Structural alternatives include channelization, improvement of bridges, culverts, and other hydraulic structures, upstream detention storage, levee construction, stormwater pump station installation, and interbasin diversion. Non-structural alternatives include minimum building elevations, floodplain zoning, and flood forecasting and warning.

Alternative solutions to flooding problems affecting each specific area or community were evaluated by estimating annual flood damage and emergency cost reduction benefits as well as property value enhancement benefits and comparing them to annual costs of implementation and maintenance of flood control improvements. Improvement projects were prioritized on an area- and county-wide basis using computed benefit-cost ratios for existing and future development conditions. One flood control project for each area of the County was selected as having the greatest benefit-cost ratio for existing development conditions, and these are presented in the following table. The total capital cost of these alternative projects is estimated to be $38,064,400 and combined annual benefits of implementation ($3,204,000) do exceed annual costs ($3,093,200) over a 100-year project life. The reader is directed to Section 4 of this report for a more detailed presentation and evaluation of all flood control projects considered in this study, as all viable projects are not included in this summary table.

SUMMARY TABLE

Area	Drainage Channel(s)	Type of Improvements	Capital Cost ($)	Annual Cost ($)	Annual Benefit ($)	Benefit -Cost Ratio
Ingleside	Kinney Bayou	Channel & Structures	1,029,700	83,700	272,300	3.3
Taft	John Deere Ditch (Main AJ)	Channel & Structures	2,872,800	233,500	385,100	1.6
Aransas Pass	South Basin	Stormwater Pump Station & Detention Storage	2,137,900	173,700	182,900	1.1
Mathis	Sixmile Crk. & Extension	Channels & Structures	718,600	58,300	65,300	1.1
Sinton	Chiltipin Crk. & South Ditch	Channels & Structures & Rectification	21,944,900	1,783,300	1,752,300	1.0
Odem	Peters Swale	Channel & Structures	2,726,500	221,600	189,000	0.9
Portland /Gregory	Alt. 1*	Channels & Structures & Interbasin Diversion	6,634,000	539,100	357,100	0.7
			**********	*********	*********	
			$ 38,064,400	3,093,200	3,204,000	

* Alt. 1 includes Green Lake, Main BG-00 and tributaries, the Doyle Addition Ditch, and the proposed Airport Ditch and Gum Hollow tributary diversion.

Selection and implementation of specific flood control improvements are ultimately the privilege and responsibility of the County and City governments and the citizens of San Patricio County. The Flood Control Study report and other deliverables are intended to provide a reasonable and consistent basis for floodplain management and the evaluation and selection of flood control projects.

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SECTION 1.0 - INTRODUCTION

San Patricio County is, in general, located between the Nueces and Aransas Rivers in the central portion of the Coastal Bend of Texas. The County has a total land area of 693 square miles and an estimated population of 61,000 persons. There are eight incorporated communities, several unincorporated communities, and various industrial and agricultural developments located within the County. These incorporated communities include the City of Sinton, which is the county seat, and the Cities of Mathis, Odem, Taft, Portland, Gregory, Ingleside, and Aransas Pass. The relative locations of these communities are noted in Figure 1.0-1.

As a result of its proximity to the Gulf of Mexico and its relatively flat coastal topography, the County has historically suffered major flood damages from both hurricanes and frontal storm events. Flooding has been both frequent and severe, as evidenced by the inclusion of San Patricio County in Presidential Disaster Declarations eight times since 1960. Several areas within the County are expected to experience significant development and population growth in the near future. Many of these areas with high development potential, as well as existing developed areas are susceptible to severe flooding; however, no comprehensive flood control and management program is currently

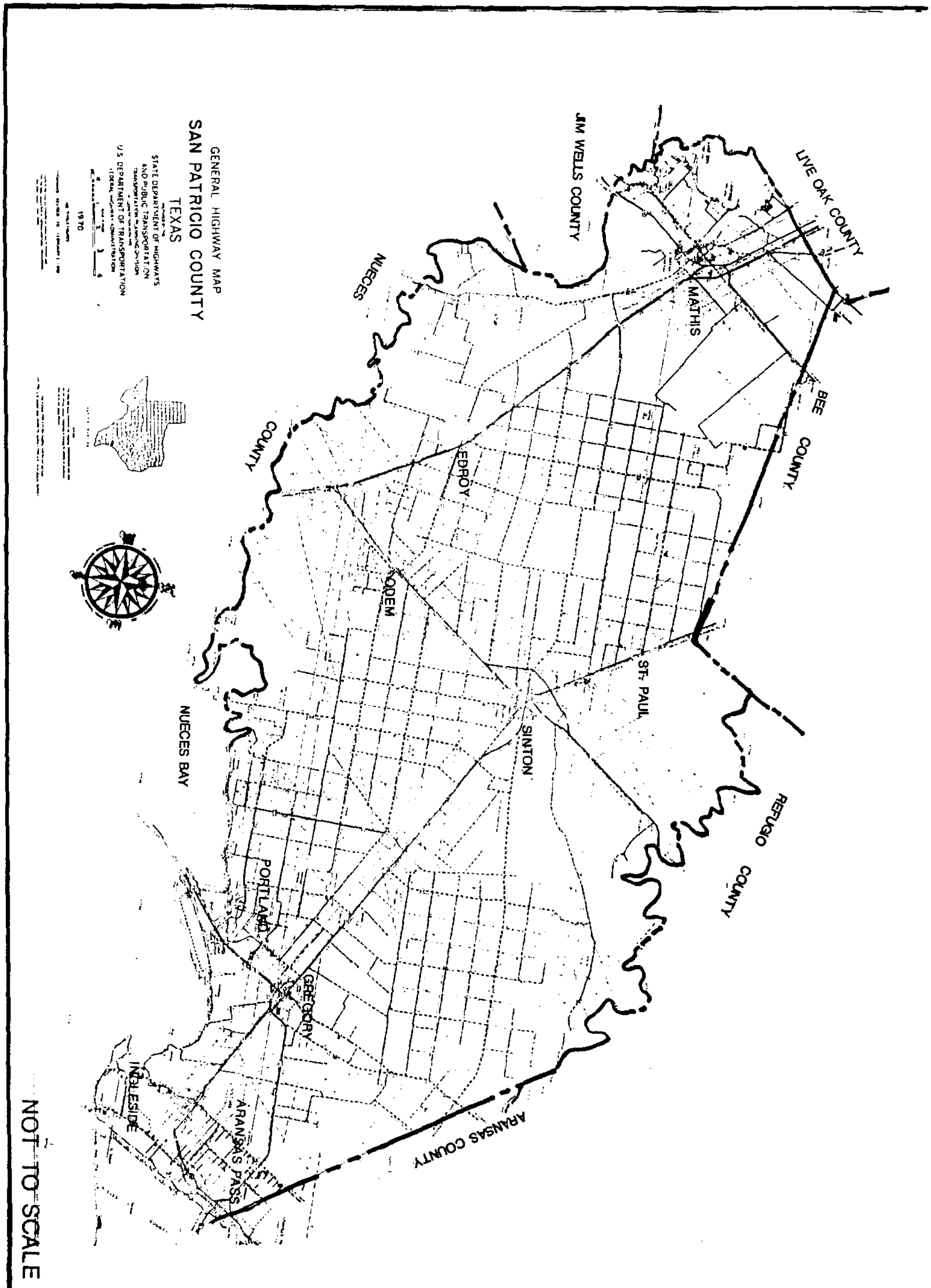


FIG.# 1.0-1	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	GENERAL VICINTY MAP
	NAISMITH ENGINEERS, INC. Consulting Engineers	

available. This Flood Control Study has been undertaken to assist the San Patricio County Drainage District (SPCDD) in the development of such a program.

HDR Infrastructure, Inc. (HDR) and Naismith Engineers, Inc. (NEI) were authorized July, 1986 to prepare a Flood Control Study for San Patricio County. The primary objectives of the study are to address specific flooding problems affecting each of the eight communities, other areas of potential development, and the County as a whole, and to evaluate various alternative solutions to these problems. The performance of the study was divided into three phases, as summarized below:

Phase 1 - The primary objectives of Phase I included the identification of flooding problems, data collection, and the development of design criteria and basin analysis methodologies. Flooding problems and concerns were identified primarily by coordination with the SPCDD Staff and Board of Directors, public information and workshop meetings with the County Commissioners and City Councils and staffs, and review of previous drainage and flood insurance studies. Summaries of the public information and workshop meetings are included herein as Appendix C. The data collection effort included acquisition of topographic maps, hydrologic data, aerial photography (provided by SPCDD), field surveys (provided by

SPCDD), and existing reports and drainage plans. Federal and State agencies including the U.S. Army Corps of Engineers (USCE), Soil Conservation Service (SCS), Federal Emergency Management Agency (FEMA), National Weather Service (NWS), U.S. Geological Survey (USGS), Texas Water Commission (TWC), and Texas Department of Highways and Public Transportation (SDHPT) were advised of the study and solicited for pertinent information.

Phase 2 - Phase 2 consisted of the development of flood control alternatives for the specific flood-prone areas identified in Phase 1 and evaluation and prioritization of these alternatives on the basis of benefit-cost analyses. Flood control alternatives included structural measures such as channelization, bridge replacement, storage, levee construction, and interbasin diversion. Non-structural measures such as floodplain zoning, minimum building elevations, and flood forecasting were also included.

Phase 3 - The tasks performed under Phase 3 included the preparation of the deliverable products of the study. Deliverables include a watershed boundary map, FEMA floodplain boundary map, aerial photographs showing existing and proposed 100-year floodplain boundaries, water surface profiles for areas where drainage improvements are evaluated, computer models, a Drainage Criteria and Design Manual, and a Flood Control Study Report.

This Flood Control Study report is divided into five Sections, with each Section summarizing a significant portion of the overall work effort. Section 2 presents general considerations with regard to types of flooding affecting San Patricio County, initial and major drainage systems, drainage criteria, and FEMA mapping. The basin analysis methodologies applied in this study pertaining to hydrology, hydraulics, benefit-cost analyses, and flood control alternatives are detailed in Section 3. The development, evaluation, and recommendation of flood control alternatives for each specific problem area are presented in Section 4 along with existing and proposed water surface profiles. At the conclusion of Section 4, a priority ranking of proposed flood control alternatives prepared in consultation with the San Patricio County Drainage District is presented.

SECTION 2.0 - GENERAL CONSIDERATIONS

2.1 Types of Flooding

San Patricio County is susceptible to three primary sources of flooding. One source is from the defined drainageways and creeks, wherein flooding occurs due to inadequate channel capacity, restrictions within the channels, and structures with inadequate capacity to pass the stormwater flows. During major storm events, these man-made barriers such as bridges, culverts, and railroad embankments can act as small dams that retard the flow. The second major source of flooding is from poor drainage due to the relatively flat topography within the County. Water tends to pond and then drain off, infiltrate, or evaporate very slowly during and after storm events. Therefore, a large area may have water slowly moving across it in a sheet flow pattern rather than in defined drainageways. This problem is aggravated by the fact that the soils have a low permeability rate allowing little water to percolate into the ground. The third cause of flooding is from tidal sources, but the detailed evaluation of flooding in tidal areas is beyond the scope of this study.

Due to the topography of the land and the expense of providing adequate drainage for major storm events, it is recognized that all areas of San Patricio County may not be suitable for development. One purpose of this study is to identify these areas and recommend limited further development of them, in recognition that the least expensive means of preventing flooding problems may be to limit development in floodplain or flood prone areas. These areas can then be maintained in their present land use and continue to provide the benefits of natural storage and unimpeded passage of flood waters. As some existing developed areas are located where frequent flooding occurs, alternatives were evaluated to provide relief to these areas and benefit-cost analyses were performed to determine whether the implementation of alternative flood control improvements could be economically justified.

Ponding and sheet flow are considered herein as related problems. If the ponding is due to man-made barriers, relief can be provided through construction or enlargement of drainageways or structures. If the problem is caused solely by flat topography, solutions such as specifying that floor slabs and roads be constructed above the anticipated flood levels can be examined. Other potential solutions may include floodplain zoning, floodproofing and flood warning systems.

2.2 Initial and Major Drainage Systems

Local and regional planning must take into consideration both the initial and the major stormwater drainage systems. The initial drainage system transports runoff from point of origin to the major drainage systems. The initial system consists of overland flow, flow in streets, gutters, and road ditches, and flow in small storm drains. The design frequency may vary according to development type, density, and valuation. A properly functioning initial drainage system is necessary to reduce street maintenance costs, provide protection against recurring damage from stormwater runoff, and reduce inconvenience to residents. Storm drainage systems consisting of swales, ditches, and underground conduits are parts of the initial storm drainage system, and generally offer protection from storm events with a return period of 25 years or less. The initial system must minimize future drainage problems within the ability of the community to afford drainage facilities.

The major drainage system is necessary to swiftly and efficiently evacuate runoff from extreme events. For the purpose of this study, major drainage systems have been designed to handle the runoff generated by storm events having a 100-year return period. Use of the 100-year runoff event for the preliminary design of drainage improvements is consistent with the National Flood Insurance Program requirement that new structures be elevated above the 100-year floodplain. A 100-year storm is expected to be equalled or exceeded once in a 100 year period and has a 1.0 percent chance of occurrence in any given year. The major drainage system may not prevent all overbank flooding, but must be designed to minimize hazards and damage to property and public facilities.

Both the initial and major systems should be planned, coordinated, and properly engineered to insure adequate drainage for every developed area. Future development proposals should require full site planning and engineering analyses, and should protect not only the property being developed, but upstream and downstream properties as well. In this regard, uniform design considerations must be applied to each site, and these considerations may be defined in a drainage design criteria manual for the area.

2.3 Drainage Criteria and Design Manual

Storm drainage design criteria and procedures are presented in the "Drainage Criteria and Design Manual, San Patricio County, Texas" (Ref. 14) in an effort to achieve a uniform method of assuring adequate storm drainage as the County develops. The manual lists the reference information used and gives design factors and graphs for use as engineering guides in the planning and design of drainage facilities for the initial and major storm systems. The manual can not be expected to address extraordinary situations, but should be adequate for most applications. It is not intended as a replacement for sound engineering judgment, but as a guide to providing adequate and uniform drainage. The manual is prepared in loose-leaf form and should be reviewed and modified periodically in order to maintain reliable and consistent design methods for the analysis of storm drainage practices.

2.4 FEMA Floodplain Management Program

The Federal Emergency Management Agency (FEMA) encourages state and local governments to adopt sound flooplain management programs through the National Flood Insurance Program. Each Flood Insurance Study includes a flood boundary and floodway map prepared to assist communities in developing floodplain management policies. These maps have been prepared for the Cities of Mathis, Sinton, Odem, Portland, Gregory, Ingleside, and Aransas Pass and for the unincorporated areas of San Patricio County. As an additional deliverable in the current study, a composite flood boundary and floodway map of the County has been prepared at a scale of 1" = 3000'. It is noted that floodplain boundaries on this map may differ significantly from those developed in this study due to the selection of different equations for the estimation of peak discharge for various return period events, the modified condition of some of the drainageways, and a more extensive use of field data in some areas. The salient features of the maps include:

Flood Boundaries - In order to provide a national standard, the 100-year flood has been adopted by FEMA as the base flood for purposes of floodplain management. The 500-year flood is employed to indicate additional areas of flood risk in a community. The 100-year and 500-year floods are defined as flood events expected to be equalled or exceeded only once in a 100-year or 500-year period, respectively. For each stream studied in detail, the boundaries of the 100- and the 500-year floods have been delineated using the flood elevations determined at each cross-section; between cross-sections, the boundaries were interpolated using topographic maps at a scale of 1:24,000 with a contour interval of 5.0 feet. In cases where the 100-year and 500-year boundaries are close together, only the 100-year boundary is shown. For inland flooding, FEMA uses three flood hazard zone designations: "A" or "numbered A" zones indicate areas inundated by the 100-year flood, "B" zones are areas between the 100-year and 500-year flood boundaries, and "C" zones are areas outside the 500-year flood boundary. Other map details involve coastline determinations and boundaries designating special flood hazards and areas not subject to flooding.

Floodways - Encroachment on floodplains, such as artificial fill, reduces the flood-carrying capacity and this increases the flood heights and flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the National Flood Insurance Program, the concept of a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 100-year flood is divided into a floodway and a floodway fringe. The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment in order that the 100-year flood may be carried without substantial increases in flood heights. Minimum standards of FEMA limit such increases in flood heights to 1.0 foot, provided that hazardous velocities are not produced. These floodways are presented to local agencies as minimum standards that can be adopted or used as a basis for additional studies. Floodways are not delineated in coastal high hazard areas.

Base Flood Elevations - Base flood elevations have been established in areas of special hazards ("A" and "V" zones) by detailed engineering methods. In coastal areas affected by wave action, base flood elevations are generally maximum at the normal open shoreline. These elevations generally decrease in a landward direction at a rate dependent on the presence of obstructions capable of dissipating the wave energy. Where possible, changes in base flood elevations have been shown in 1.0 foot increments on the Flood Insurance Rate Maps. Base flood elevations shown in the wave action areas represent the average elevation within the zone. Current program regulations generally require that all new construction be elevated such that the first floor, including basement, is above the base flood elevation in "A" and "V" zones.

Velocity Zones - The U.S. Army Corps of Engineers has established the 3.0 foot wave as the criterion for identifying coastal high hazard zones, and this has been adopted by FEMA for the determination of V zones. Because of the additional hazards associated with high-energy waves, the National Flood Insurance Program requires much more stringent floodplain management measures in these areas, such as elevating structures on piles or piers.

SECTION 3.0 - BASIN ANALYSIS METHODOLOGIES

In the process of developing and evaluating solutions to flooding and drainage problems, principles of hydrology, hydraulics, and economics are employed. These principles facilitate scientific evaluation of the applicability of various flood control alternatives to resolve a specific flooding problem. Recommended flood control alternatives for specific basins within San Patricio County are presented in Section 5 of this report.

3.1 HYDROLOGY

A significant component of the flood control study for San Patricio County involves the determination of instantaneous peak discharge values for key points located along the major outfall channels. Peak discharge values at each key point are determined for return periods ranging from 2 to 100 years utilizing standard flood flow frequency analysis methods. The estimated 100-year peak discharge is defined to be the discharge expected to be equalled or exceeded only once in a typical one hundred year period or having a 1.0 percent chance of occurrence in any given year. Peak discharge values are used in the computation of water surface profiles which, in turn, delineate the floodplain or inundated region surrounding the primary drainageway during a flood event.

Peak discharge at any given point in a watershed is a function of precipitation magnitude and intensity, drainage area, topography, general soil type, antecedent moisture conditions, channel conveyance, and numerous other factors. Due to the diversity of factors whose interrelationships determine the maximum runoff rate for a particular storm event, a number of methods have been developed to estimate instantaneous peak discharge for a given return period. One method for the estimation of flood flow frequency applicable to gaged watersheds (at the gage location) was developed by the U.S. Water Resources Council (WRC) and has been updated, extended, and re-published by the U.S. Geological Survey (USGS, Ref. 30). As all watersheds and sub-watersheds to be considered in this study with the exception of Chiltipin Creek at Sinton are ungaged, consideration of an alternative method is imperative. The primary method currently used in San Patricio County for the estimation of peak discharge for various return periods is a set of regionalized equations that were developed by the USGS (Ref. 31). This method has been utilized in the performance of Flood Insurance Studies for San Patricio County unincorporated areas and the Cities of Sinton, Odem, and Gregory (Refs. 4, 7, 9, and 10) and is currently employed by the State Department of Highways and Public Transportation in the hydrologic design of bridges and highway drainage.

Flood Flow Frequency Analysis Methods

Water Resources Council Method - The Water Resources Council Method is a statistically-based methodology for the estimation of peak discharge values for various return periods using the sample statistics of the logarithms of historical annual maximum instantaneous discharges at a selected gage location. The sample statistics utilized are the mean ($\bar{y}$), the standard deviation (S_y), and coefficent of skewness (C_s). As the coefficient of skewness is a particularly difficult parameter to estimate accurately for a small sample, the WRC has developed generalized map skew coefficients for the U.S. which are used along with the sample skew to compute a reasonable weighted skew coefficient. Outliers (data points which depart significantly from the trend of the remaining data) are tested for and removed if necessary. Assuming that the annual maximum flood peaks may be described by the Log Pearson Type III distribution, peak discharge values for various return periods are estimated using frequency factors (K_t) which vary with weighted skew and return period (t).

$$Y_t = \log Q_t \quad \text{and} \quad Y_t = \bar{y} + K_t S_t$$

where: Q_t = Peak discharge for return period t

Y_t = Log of peak discharge for return period t

K_t = Frequency factor for return period t immediately adjacent to Flood-Frequency Region 2. In both regions, the only independent variables found to be significant at the 95 percent confidence level were slope and drainage area. Slope is defined to be the average slope of the streambed between points 10 and 85 percent of the distance between the site and the basin divide. The regression equations developed for Regions 1 and 2 from which the 2- through 100-year peak discharge values can be determined, given watershed slope and drainage area, are presented in Table 3.1-1.

Evaluation of Flood Flow Frequency Analysis Methods

Each of the flood flow frequency analysis methods discussed in the preceding sections was applied to the Chiltipin Creek watershed upstream of Sinton. This is the only watershed in the area for which a sequence of unregulated historical gaged streamflow and annual peak discharge measurements (1971-85) could be obtained. In addition, this watershed is typical of the nearly level to gently sloping, very slowly permeable, clayey and loamy soils found in San Patricio County (Ref. 23). Peak discharge estimates for return periods ranging from 2 to 100 years computed using the WRC and the USGS (Regions 1 and 2) methods are presented in Table 3.1-2, as are the 95 percent confidence limits for the WRC method results.

TABLE 3.1-1

Regression Equations for

Flood-Frequency Regions 1 and 2*

Region 1	Region 2
$Q_2 = 89.9(A)^{0.629}(S)^{0.130}$	$Q_2 = 216(A)^{0.574}(S)^{0.125}$
$Q_5 = 117(A)^{0.685}(S)^{0.254}$	$Q_5 = 322(A)^{0.620}(S)^{0.184}$
$Q_{10} = 131(A)^{0.714}(S)^{0.317}$	$Q_{10} = 389(A)^{0.646}(S)^{0.214}$
$Q_{25} = 144(A)^{0.747}(S)^{0.386}$	$Q_{25} = 485(A)^{0.668}(S)^{0.236}$
$Q_{50} = 152(A)^{0.769}(S)^{0.431}$	$Q_{50} = 555(A)^{0.682}(S)^{0.250}$
$Q_{100} = 157(A)^{0.788}(S)^{0.469}$	$Q_{100} = 628(A)^{0.694}(S)^{0.261}$

Note: Q_t = Peak discharge in cfs for return period t in years

A = Drainage area in square miles

S = Average slope in feet per mile

* Ref. 31.

TABLE 3.1-2

Peak Discharge Estimates

Chiltipin Creek at Sinton, Texas

USGS Gage #08189800

Method	Peak Discharge For Return Period in Years (in Cubic feet per second, cfs)					
	2	5	10	25	50	100
WRC	4337	8707	12338	17693	22195	27116
USGS - Region 1	2435	5229	7620	11178	14272	17386
USGS - Region 2	4428	9220	13371	19333	24331	29768
WRC (Lower 95% C.L.)	3528	5967	8142	11062	13345	15709
WRC (Upper 95% C.L.)	5363	14826	23519	38466	52685	69675

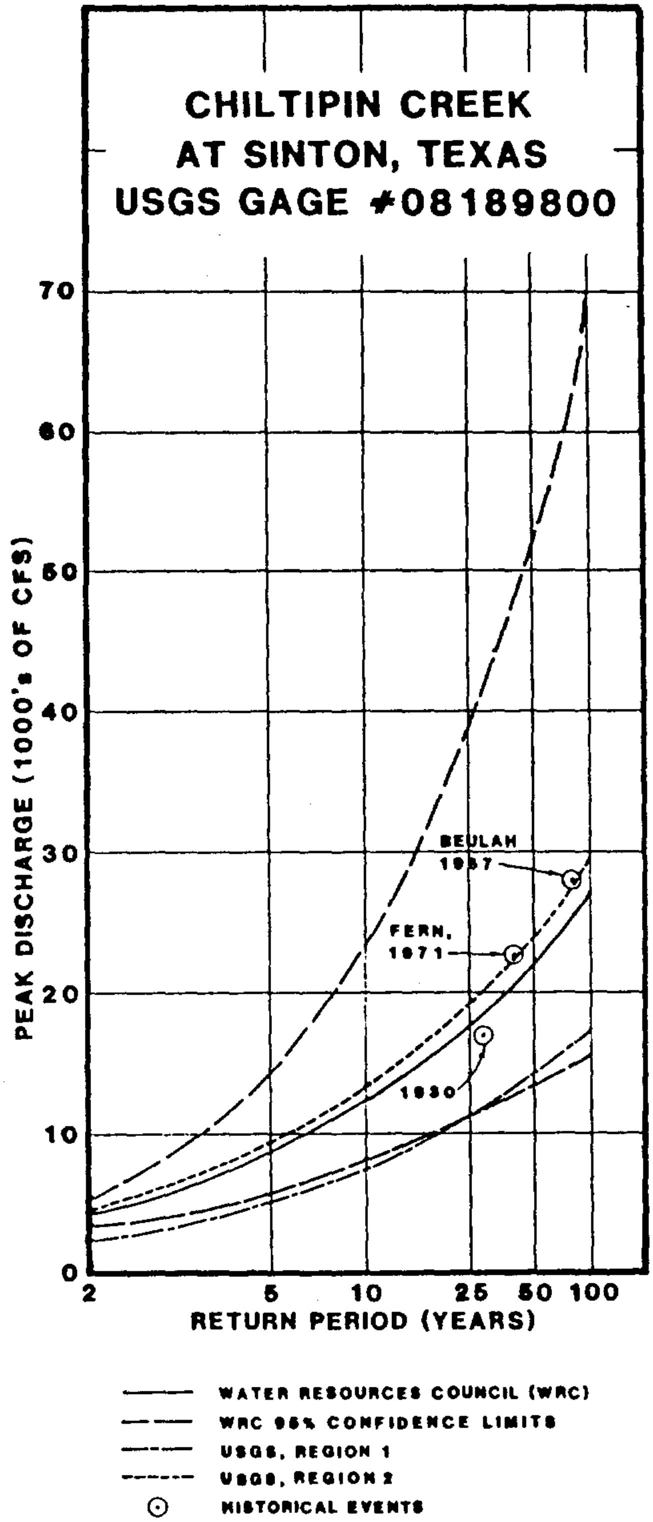
Note: WRC = Water Resources Council (Ref. 30)

USGS = U.S. Geological Survey (Ref. 31)

C.L. = Confidence limit

The values presented in Table 3.1-2 are also plotted in Figure 3.1-1 along with three additional points which represent observed historical maximum discharge values. These additional points correspond to flood events that occurred in 1930, 1967 (Hurricane Beulah), and 1971 (Hurricane Fern) which generated estimated peak discharges of 17,000 cubic feet per second (cfs), 28,000 cfs, and 22,300 cfs, respectively. Although streamflow records have been maintained continuously at the Chiltipin Creek gage only since 1970, USGS publications (Ref. 32) indicate that the maximum discharges which occurred in 1967 and 1930 were the greatest since 1910. Hence, the three historical peaks are plotted using the Weibull plotting position relationship (Ref. 12) assuming 76 years (1986 minus 1910) of record.

Review of Figure 3.1-1 indicates reasonably good agreement between the results obtained using the WRC flood flow frequency method and the historical maxima. The peak discharge estimates computed using the USGS method and the Region 1 equations, however, appear invalid as they lie partially outside the lower 95 percent confidence limit evaluated for the WRC curve. On the other hand, the curve based on the USGS method and Region 2 equations shows excellent agreement with the observed historical maxima. Given the proximity of the Chiltipin Creek watershed and the remainder of San Patricio County to Flood-Frequency Region 2 and the apparent agreement of peak discharges estimated by the



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	PEAK DISCHARGE ESTIMATES ALTERNATIVE METHODS	FIG.# 3.1-1
Consulting Engineers		DATE 7/87

USGS method using Region 2 equations with historical events, it was decided that this method and these equations be used in the performance of the hydraulic and economic analyses associated with the San Patricio County Flood Control Study.

Development of Runoff Hydrographs

Analyses of several drainage alternatives required the use of an approximate runoff hydrograph method. These alternatives included detention storage and storm runoff removal by pumping as well as the evaluation of existing situations in which runoff spills from the watershed of origin to an adjacent watershed. The peak discharge can be readily computed by methods described in the preceding paragraphs; however, the timing of the peak discharge and the total volume of runoff as described by a runoff hydrograph must also be considered in the above situations. Hence, procedures developed by the Soil Conservation Service (SCS) and described at length in the SCS National Engineering Handbook, Section 4, Hydrology (Ref. 22) were applied to generate approximate runoff hydrographs from the peak discharge estimates obtained from the USGS Region 2 equations.

The SCS method is based on a triangular approximation of the runoff hydrograph and the general form of the equation for estimating peak discharge is presented below:

$$Q_p = 645.33KAQ/T_p$$

where: Q_p = Peak discharge in cfs

A = Watershed area in sq. miles

Q = Depth of effective precipitation in inches

T_p = Time to peak discharge in hours

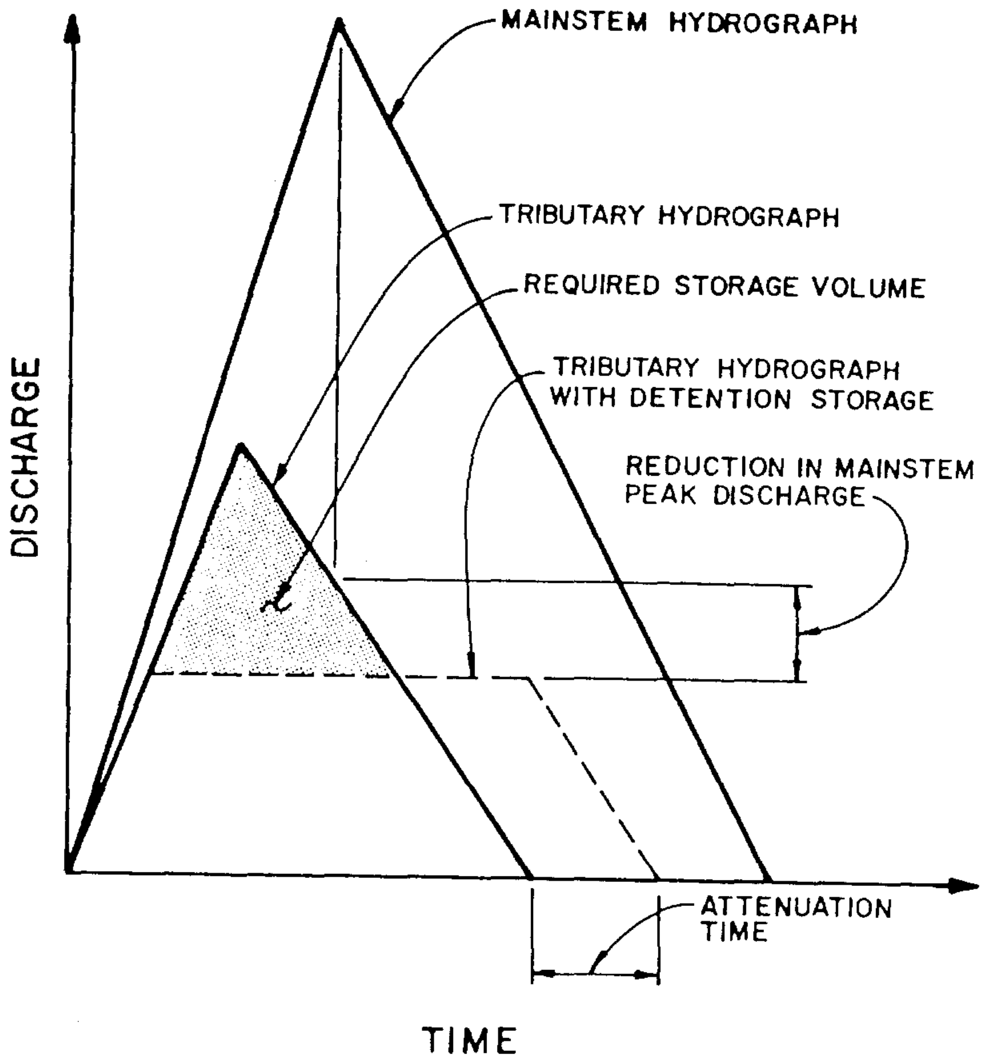
$K = 2/(1+T_r/T_p)$

T_r = Time of recession in hours

The depth of effective precipitation or direct runoff, Q, is a function of total depth of precipitation, watershed curve number, and the initial rainfall abstraction. By analysis of historical storm hydrographs in a gaged watershed, the peak discharge (Q_p), depth of effective precipitation (Q), average time of recession to time to peak ratio (T_r/T_p), and average time to peak (T_p) can be estimated for the watershed upstream of the gage. Utilizing the average T_r/T_p ratio and T_p in conjunction with peak discharge (Q_p) computed by the USGS Region 2 equations, the depth of effective precipitation (Q) for various return period events can be estimated. Assuming that Q and T_r/T_p are uniform over an entire watershed, T_p can be computed and a triangular hydrograph can be constructed for any location in the watershed by combined application of the above equation and a USGS Region 2 equation.

In order to apply the above procedure to the Chiltipin Creek watershed, runoff hydrographs from the gage on Chiltipin Creek at Sinton (USGS Gage #08189800) for four independent historical storm events were considered. These events included Hurricane Fern in 1971 and Hurricane Allen in 1980 as well as two less significant events which occurred in 1973 and 1982. Analysis of the observed storm runoff hydrographs using a triangular hydrograph approximation resulted in an average T_r/T_p ratio of 1.95, which is very comparable to that found for the Oso Creek basin in Nueces County (Ref. 15). Applying the SCS general equation to these events, the average T_p for the Chiltipin Creek watershed above the gage was estimated to be 28.7 hours. Inserting the 100-year peak discharge estimate for this location from the USGS Region 2 equation into the SCS general equation yielded an estimate of the 100-year depth of effective precipitation (Q) or runoff volume of about 15.2 inches. Given a 100-year runoff volume of 15.2 inches, a T_r/T_p ratio of 1.95, and a 100-year Q_p estimate from the USGS equation, a triangular hydrograph approximation could be developed for any location in the watershed by solving the SCS general equation for T_p.

The methodology used in quantifying the effect of upstream tributary detention storage on peak discharge estimates for the mainstem on the basis of triangular hydrograph approximations is illustrated in Figure 3.1-2. With the implementation of detention



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	RUNOFF HYDROGRAPH ANALYSIS METHODOLOGY	FIG.# 3.1-2
		DATE 7/87

storage, the shape of the tributary hydrograph is changed as outflow is limited so that available storage volume may be utilized. The required storage volume is dependent upon the selected outlet capacity of the detention storage basin. The reduction in mainstem peak discharge due to detention storage is assumed equal to the difference between the natural runoff rate and the modified (with storage) rate at the time of peak discharge on the mainstem. This methodology was also applied in evaluating the impact of Peters Swale overflows on peak discharge estimates for Chiltipin Creek and South Ditch near Sinton.

3.2 HYDRAULICS

Hydraulic modelling of various major outfall systems in San Patricio County has been performed using the HEC-2 Water Surface Profiles program (Ref. 27) developed at the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. This program is capable of rapidly computing the water surface profile for a given flowrate in a channel of any cross section subject to either subcritical or supercritical flow conditions. The effects of various hydraulic structures such as bridges, culverts, weirs, levees, and dams may be considered in the computation. A computer program of this type is well suited for the performance of flood control studies of this type, as large volumes of field data can be incorporated, multiple water surface profiles for varying frequency events can be evaluated simultaneously, and the impacts of channel and structural improvements may be readily assessed.

The computational procedure employed by the HEC-2 program applies Bernoulli's theorem for the total energy at each cross section and Manning's formula for the friction head loss between cross sections. The Manning Equation is defined as follows:

$$Q = (1.486/n)A(R_h)^{0.667}(S_f)^{0.5}$$

where: Q = Discharge in cfs

n = Roughness coefficient

A = Cross-sectional area of flow in sq. ft.

R_h = Hydraulic radius in ft. = A/P

P = Wetted perimeter in ft.

S_f = Friction slope in ft./ft.

Average friction slope for a reach between two cross sections is determined in terms of the average of the conveyances at the two ends of the reach. Other losses at transitions in channel geometry and bridge structures are computed using one of several methods discussed in the HEC-2 User's Manual.

Cross-section data, channel reach length, peak discharge rates, and values of the Manning roughness coefficient (n) are the basic required imputs to the HEC-2 program. Input n-values were selected on the basis of information published in Open Channel Hydraulics (Ref. 1) and verified by field reconnaissance and consultation with the San Patricio County Drainage District Staff. Typical n-values used included:

n	Channel Condition
0.015	Concrete-lined channel
0.045	Improved grass channel
0.06 - 0.45	Natural channels and overbank areas

The sensitivity of water surface elevations to discharge and channel geometry is low in many portions of San Patricio County due to flat overbank areas. As stormwater runoff exceeds the bank-full capacity, flow spills out into the flat overbank areas and establishes new flow patterns. In some instances the direction of overland flow is normal to the channel flow and would require a substantially more sophisticated model to accurately depict the flow patterns developed. The capacity of overbank floodplain areas to store water is generally so great that a large increase in discharge may result in only a small increase in flood elevations. When simulating these conditions in HEC-2, the effective flow area of a floodplain is determined and cross-sections are vertically extended at that point. When extensions at a particular section exceed one foot, the cross-section and/or hydrology may require modification. Existing channel geometry is fixed as per the field survey data. However, sections and overbank slopes are sometimes modified to provide for uniform flow regimes along a particular channel reach.

The results generated by any computer model are dependent on the quality of the input data and proper understanding of the assumptions incorporated in the model. Recently surveyed channel cross-sections and structural dimensions were compiled and reduced by the San Patricio County Drainage District staff. Additional data, including the results of HEC-2 modelling of many portions of the County, have been obtained from the Federal Emergency Management Agency (FEMA). This information has proven valuable in the development and assessment of drainage alternatives for specified flood-prone areas in San Patricio County.

In several areas within San Patricio County, drainage improvements including channelization and hydraulic structures were proposed for areas in which no defined drainageways currently exist or no surveyed cross-sectional information is available. Channel designs for these areas were developed by normal depth computations based on the Manning Equation and 100-year peak discharge estimates. The change in water surface upstream and downstream of proposed culvert crossings (head loss) was typically estimated using nomographs obtained from the Texas Department of Highways and Public Transportation (Ref. 24) and included in the Drainage Criteria and Design Manual (Ref. 14) for San Patricio County.

The water surface elevations generated by either the HEC-2 program or by normal depth computations represent base flood elevations from which flood events are defined and delineated. The water surface elevations computed for each of the areas studied are presented in profile view along with channel inverts and existing structures in Appendix A and in plan view on the aerial maps. Water surface profiles for existing conditions were not developed for areas in which no defined drainageway currently exists or insufficient cross-sectional data is available. Topwidths associated with computed water surface elevations at each cross-section were first plotted on USGS 7.5 minute topographic maps. The variation in topwidths between the cross-sections was interpolated between the 5-foot contours on the maps to estimate the floodplain boundaries. The cross-section locations and 100-year floodplain boundaries were then transferred to the aerial maps. From the water surface profiles and floodplain mapping, areas subject to significant areal flooding, excess backwater at structures, and overtopping of roads were identified. Existing flood profiles were compared with available high water marks for historical flood events to verify model adequacy and floodplain mapping.

3.3 BENEFIT-COST ANALYSES

Benefit-cost analyses have been incorporated in the San Patricio County Flood Control Study as a means of evaluating the relative merits of various alternative projects. These analyses facilitate the comparison of various individual projects throughout the County as well as alternative projects designed to reduce flooding in a specific area. General guidelines for the performance of benefit-cost analyses have been obtained from Economic and Environmental Principles and Guidelines for Water and Related

Land Resources Implementation Studies (Ref. 33) prepared by the U.S. Water Resources Council (WRC). The applied benefit-cost evaluation methodology and assumptions and estimation of average annual flood damages for the County are discussed and specific unit costs and unit benefits are assigned in the following sections.

Methodology and Assumptions

The WRC (Ref. 33) suggests that there are three basic types of benefits associated with the reduction of flood damages: 1) Inundation reduction benefits for which land use type and intensity remain the same with the project as without; 2) Intensification benefits for which land use type remains the same and intensity increases with the project; and 3) Location benefits for which a new land use type is allowed as a result of project implementation. All three types of benefits have been considered either directly or indirectly in the evaluation of alternative projects for the San Patricio County Flood Control Study. Project benefits are related to the reduction of physical damages including damages to property, structures, contents, crops, automobiles, utilities, and public amenities. Reductions in emergency costs related to evacuation, flood fighting, rescue, reoccupation, clean-up, and general public safety during flood events are also considered project benefits. Benefits may also be attributed to

2) Classify acreage removed from the floodplain by project implementation as urban or rural and compute flood damage reduction benefits.

3) Estimate average annual flood damage reduction benefits based on the ratio of average annual damages to estimated 100-year damages for the County.

4) Estimate reduction in emergency costs at 5.0 percent of the average annual flood damage reduction. The ratio of emergency costs to total flood damage was approximately five percent for Hurricane Beulah (Ref. 28), which caused severe flooding in Texas coastal areas in 1967.

5) Evaluate capital project costs including contingencies (10% of basic construction costs) and allowance for engineering, legal, administration, and finance fees (15% of total construction cost including contingencies).

6) Estimate annual operation and maintenance costs at 1.0 percent of the capital construction cost.

7) Compute annual project cost based on a 100-year project life and a 7.25 percent interest rate.

8) Compute benefit-cost ratio for existing conditions.

9) Classify rural acreage removed from the floodplain by project implementation as having urban development potential, enhanced development potential, or no significant development potential.

10) Estimate average annual potential development benefits and add to annual flood damage and emergency cost reduction benefits.

11) Compute benefit-cost ratio for future conditions.

Benefits and cost estimates presented in this report are prepared for conceptual and comparative purposes only. Consistent unit benefits and costs have been applied throughout the County so as to provide an unbiased baseline in assessing improvement priorities and budgetting long-range flood control improvements.

Specific channel improvements and structural dimensions are provided herein to define the hydraulic characteristics required to assess flood damage reduction benefits and estimate the costs of improvements. Final determination of these features and associated costs should not be made until a detailed engineering design has been completed.

Average Annual Flood Damages

Average annual flood damages for San Patricio County were estimated based on historical flood damage estimates for significant storm events. Key references containing information regarding historical flood damages in the County included the "Report on Hurricane Beulah" issued by the U.S. Army Corps of Engineers (Ref. 28), an Interagency Hazard Mitigation Report issued by the Federal Emergency Management Agency (Ref. 11), and a flood history of the County prepared by Roy Sedwick of the Texas Water Commission (Ref. 21). Limited damage estimates are available for four major events including Hurricanes Beulah, Fern, and Allen and the so-called "October Storm" of 1984, and these were considered in the estimation of average annual damages. The return periods of major flood events were based on the frequency distribution of annual maximum discharges observed at the USGS gage on Chiltipin Creek at Sinton. Table 3.3-1 summarizes the events considered and the associated damage estimates. Damage estimates were converted to 1985 dollars based on historical per capita income in San Patricio County (Ref. 26). Plotting damages versus return period, county-wide flood damages for the 100-year event were estimated to be approximately 73.0 million dollars. Average annual flood damages for San Patricio County were estimated at 3.937 million dollars by plotting damages versus frequency and computing the area under the curve. These analyses are summarized in Figure 3.3-1.

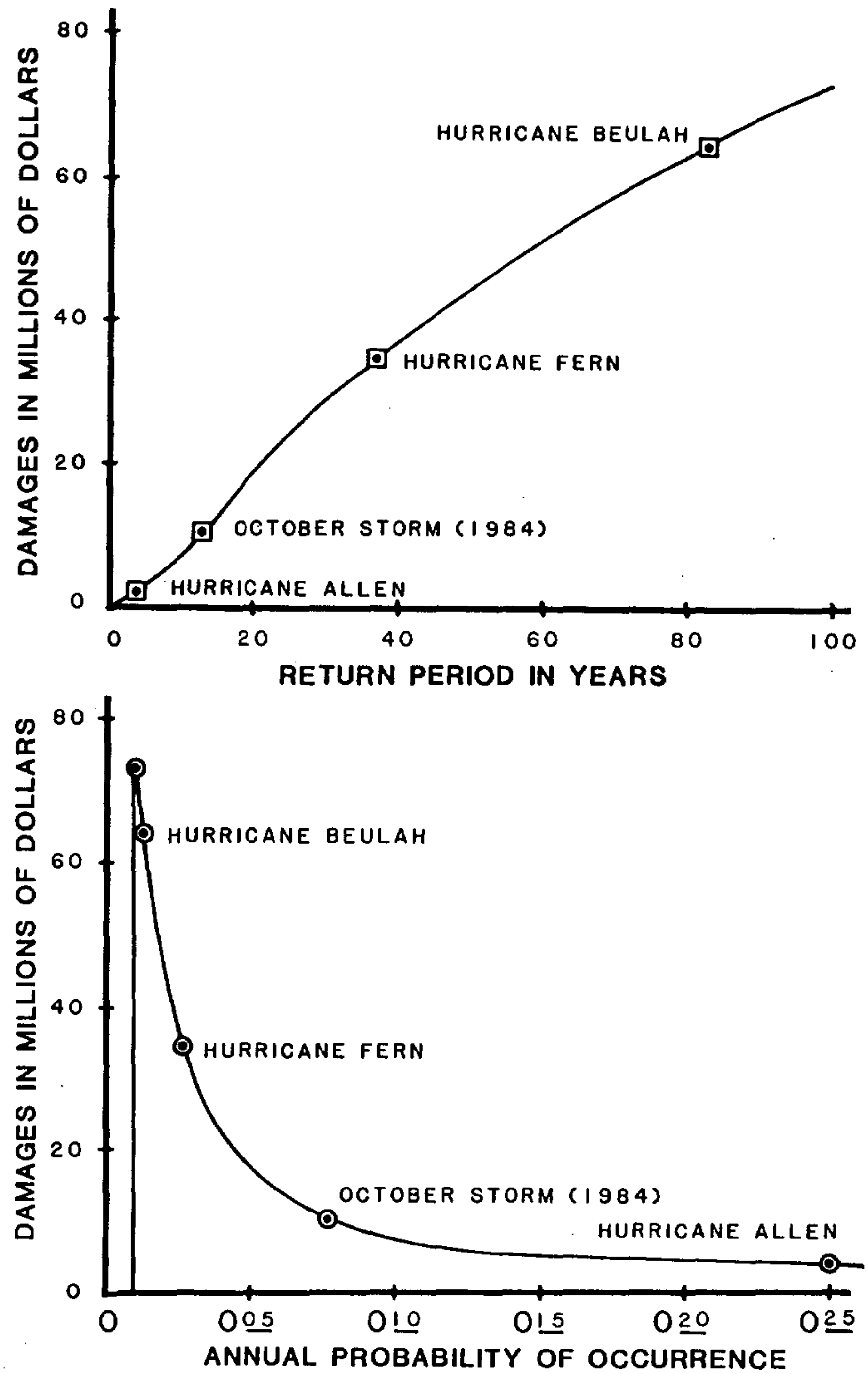
Unit Costs and Benefits

The unit costs and the benefits applied on a per acre basis in the performance of benefit-cost analyses for the San Patricio County Flood Control Study are presented in Table 3.3-2 and Table 3.3-3, respectively.

TABLE 3.3-1

Flood Damage Evaluation

Month /Year	Storm	Chiltipin Creek Peak Discharge (cfs)	Return Period (yrs)	Flood Damages ($)	Flood Damages (1985$)
9/67	Beulah	28,000	83	11,000,000	64,200,000
9/71	Fern	22,300	37	8,794,000	34,800,000
8/80	Allen	8,460	4	1,400,000	2,200,000
10/84	Frontal Event	14,600	13	10,000,000	10,500,000



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	PROBABILITY DISTRIBUTION OF FLOOD DAMAGES	FIG.# 3.3-1
NAISMITH ENGINEERS, INC. Consulting Engineers		DATE 7/87

TABLE 3.3-2

Applicable Unit Costs*

Item	Unit Cost	Unit	Explanation
1) Channel Excavation & Disposal	1.00-1.50	yd^3	Unit cost dependent on excavation quantity.
2) Concrete Channel Lining	4.00	ft^2	Installed.
3) Clear & Grub	350.00	acre	
4) Vegetation Establishment	1200.00	acre	R.O.W. inclusive.
5) County Road Bridges	40.00	ft^2	Bridge deck, typical 26' width, includes approach work.
6) Farm to Market Road Bridges	50.00	ft^2	Bridge deck, typical 44' width, includes approach work.
7) State & U.S. Highway Bridges	50.00	ft^2	Bridge deck, width variable, includes approach work.
8) Railroad Bridges	1000.00	lf	Bridge deck, includes approach work.
9) Bridge Demolition & Removal	10.00	ft^2	Bridge deck, included in above unit costs.
10) Concrete Box Culverts	350.00	yd^3	Installed.
11) Misc. Concrete Structures	350.00	yd^3	Installed.
12) Drop Structure Removal or Modification	200.00	yd^3	
13) Embankment Construction	3.00	yd^3	

TABLE 3.3-2
(Continued)

14) Road Replacement	40.00	yd^2	Subgrade and surface.
15) Pipeline Relocation	100,000.00	ls	Per crossing, includes: multiple lines, labor, materials, down time.
16) Right-of-Way (R.O.W.)			
Undeveloped	1000.00	acre	Includes farm land.
Future Development	2000.00	acre	Growth potential.
Developed	4000.00	acre	Existing urban area.
17) Spoil Disposal Sites	1000.00-4000.00	acre	Per R.O.W. unit cost, 25' max. ht., 3:1 SS.
18) Relocations	75,000.00	home	Includes house, lot, moving costs.

Note: yd^3 = Cubic yards;
ft^2 = Square feet;
lf = Linear feet;
yd^2 = Square yards; and
ls = Lump sum.

* All unit costs in dollars.

TABLE 3.3-3

Applicable Unit Benefits*

Item	Unit Benefit	Explanation
1) Urban Flood Damage Reduction	34,000.00	Based on historical urban flood damages and 2.75 structures per acre.
2) Rural Flood Damage Reduction	200.00	Based on historical rural flood damages to agricultural property.
3) Potential Urban Development Benefit	2000.00	Location/intensification benefit assigned to area credited with development potential in estimation of project cost.
4) Potential Development Benefit	1000.00	Location/intensification benefit assigned to currently undeveloped (or agricultural) area.

* Unit benefits are in dollars per acre removed from the 100-year floodplain or provided major outfall drainage as a result of project implementation.

3.4 FLOOD CONTROL ALTERNATIVES

Recommended flood control measures for each of the basins considered in San Patricio County were selected from a broad spectrum of alternatives available. These alternatives involve various means of reducing flood problems, including structural improvements, channelization, storage, levee construction, interbasin diversion, floodplain zoning, and minimum building elevations. Each of these alternatives is summarized below:

Structural Improvements - The objectives of this study suggest design criteria which will retain the 100-year flood within the channel banks, prevent overtopping of major roads and bridges, and prevent inundation of homes and businesses, where possible. Each of the existing and proposed drainage structures listed in the structure inventory for each basin was evaluated for hydraulic capacity and allowable head loss. Structures in areas with development potential that could not pass the 100-year flood without causing significant backwater or overtopping were recommended for replacement. Emphasis was placed on keeping Farm to Market roads, State highways, and U.S. highways free from overtopping. Structural improvements generally included replacement or modification of existing bridges and culvert crossings and replacement of low water crossings with bridges or culverts.

Channelization - Significant existing development has encroached upon the natural floodplains, creating flooding problems along primary drainageways. In these locations, it is necessary to provide additional channel capacity in order to reduce flooding potential. Whenever possible, natural drainage patterns based on topography were preserved and natural stream channel courses were retained. Some areas are extremely flat, and overflowing floodwaters may pass into adjacent drainage basins, aggravating the flood problems in these areas. Channelization alternatives generally consisted of widening and/or deepening existing channels, providing stable sideslopes, and providing maintenance and spoil easements on both sides of the channel. In severely meandering streams, channel rectification or straightening was also considered as a means of facilitating more rapid drainage and reducing upstream flood levels.

Storage - Various forms of storage including detention and natural storage were considered in the performance of this study. The primary purpose of storage is to reduce the peak flow rate by detaining a portion of the flow during high runoff periods and slowly releasing it as floodwaters recede. When it is desirable to preserve the ecology of environmentally sensitive low-lying areas, natural storage areas may be designated. These areas do not require any excavation, yet are effective in their natural state for improving stormwater quality, providing aquifer recharge, and reducing downstream flooding.

Levee Construction - The construction of levees to protect existing urban development from inundation due to overbank flooding was considered as a potential flood control alternative in this study. Care must be exercised in the evaluation of this alternative, as levee construction can significantly increase flood levels outside the levee and upstream of the protected area. In addition, high water surface elevations outside the levee can have an adverse effect on or even prevent drainage of the protected area without stormwater pump installation.

Interbasin Diversion - As the natural topography of San Patricio County is mostly flat, existing watershed boundaries are not always clearly defined and may be changed in the course of land development or by the construction of major drainage facilities. The diversion of stormwater runoff from developed watersheds suffering frequent flood damage to relatively undeveloped watersheds has been included in this study as a possible flood control alternative.

Floodplain Zoning - Floodplain zoning is a non-structural alternative which can reduce future flood damages. Zoning regulations usually restrict development in the 100-year floodplain. In areas that are not currently developed, zoning is an effective way to prevent development in the floodplains and avoid future flooding problems.

Minimum Building Elevations - Some portions of the County have historically had flood problems due to flat topography or inadequate internal drainage. While these areas may not be near defined drainageways, they may be subject to ponded water. Development can be allowed in these areas if floor slabs and roads are raised to minimum elevations and drainage within a reasonable time is provided. While fields, yards, and open areas may experience shallow flooding, houses and roads should remain above the anticipated flood levels.

Other alternatives that were considered included floodproofing, flood forecasting and warning, and a buyout of existing flood-prone buildings.

There may be limited opportunities to floodproof some structures located in the floodplains of drainageways within the County, however, due to the general types of structures and implementation cost to the owner, such measures are frequently unsuitable. Buyout or permanent evacuation of structures located in the floodplain were not generally felt to be acceptable means of reducing flood damages in San Patricio County unless these measures are required to facilitate channelization or structural improvements. Flood forecasting and warning systems have been installed on Chiltipin Creek and Peters Swale which provide a valuable service to the Cities of Sinton and Odem. Flood warning systems may have limited applicability in other areas within the county.

Recommended flood control improvements in many of the basins in San Patricio County involved integration of structural and non-structural solutions. The process utilized to select and prioritize recommended alternatives involved a set of criteria capable of comparing the various alternatives on an equitable basis.

SECTION 4.0 - BASIN ANALYSES AND ALTERNATIVE EVALUATION

Flood control and alternative analyses performed as a portion of the Flood Control Study for San Patricio County are summarized in the following sub-sections preceding by area from the western portion of the County near the City of Mathis to the extreme eastern end at the City of Aransas Pass. The relative locations within San Patricio County of each of the eight incorporated communities included in this study are noted in Figure 1.0-1. A brief general description of each watershed and summary of specific flood control problems is provided for each area considered. These are followed by a description of the analyses performed for each basin area including peak discharge estimates for various return periods, tables and maps indicating recommended channel and structural improvements, and an evaluation of associated benefits and costs. Tables referenced in each sub-section are grouped at the end of the sub-section for easy reference and to preserve continuity. At the conclusion of this section, a priority ranking of alternative projects and improvements prepared on the basis of computed benefit-cost ratios is presented for San Patricio County.

4.1 Mathis Area

Description of Flood Control Considerations

The City of Mathis is located in the northwestern portion of San Patricio County approximately two miles east of Lake Corpus Christi. Primary outfall drainage of much of the City is provided by Sixmile Creek, which originates on the west side of town and flows southeast to the Nueces River. The drainage area of Sixmile Creek upstream of Highway 359 is 3.35 square miles.

Historical flooding problems have been experienced in Mathis primarily in the extreme western portion of the City and in the eastern portion of the City near the intersection of Highways 359 and 666 (See Appendix C, page 1). In fact, a small stormwater lift station has been installed in an effort to improve drainage of the latter area. As Mathis is located on somewhat of a high area or knoll, it is felt that flooding problems are primarily due to a lack of major outfall channels which limits the effectiveness of internal drainage facilities.

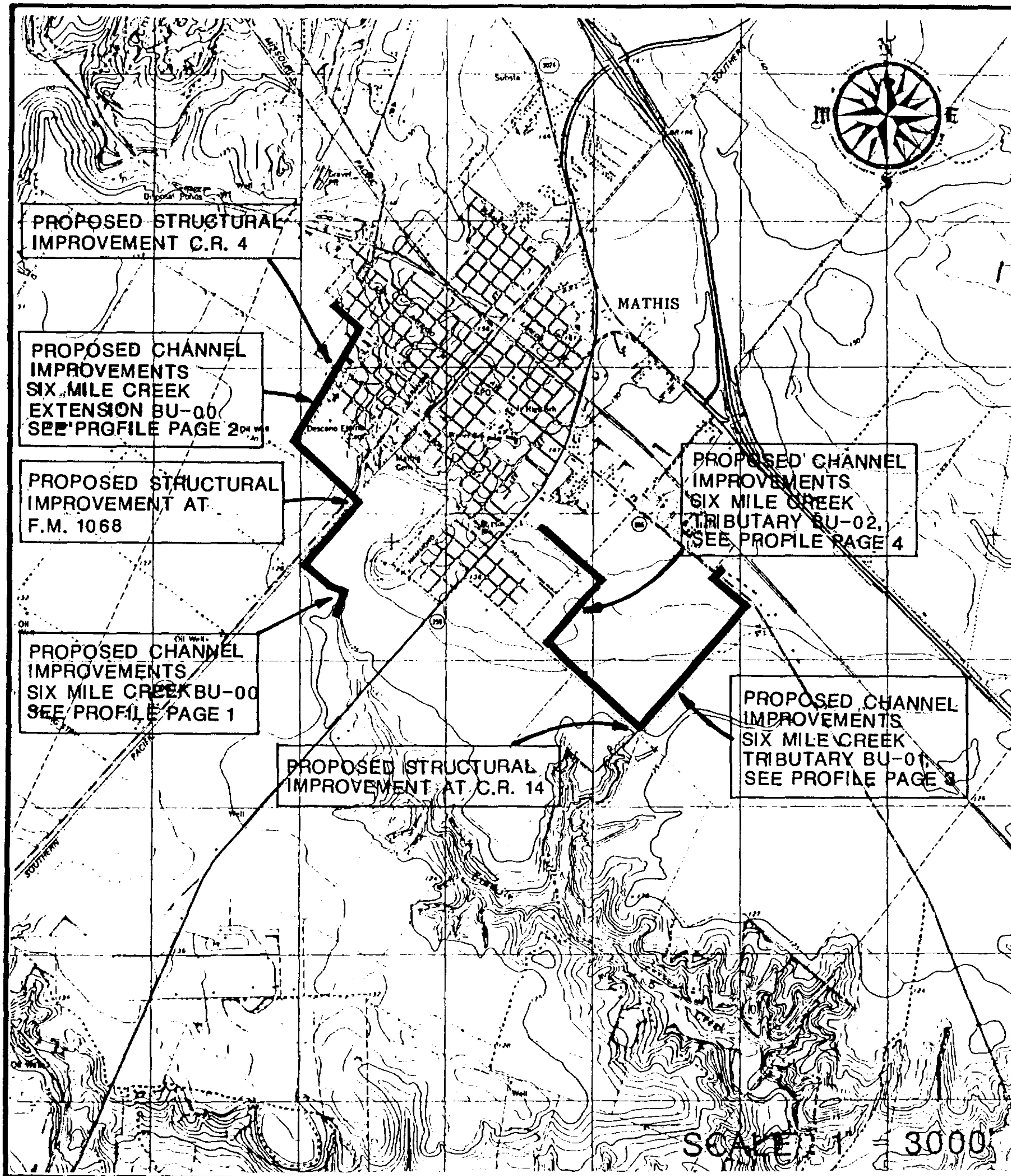
Basin Analyses and Recommended Improvements

In an effort to assess the magnitude of flooding problems in the western portion of the City, Sixmile Creek was modelled using the HEC-2 computer program (Ref. 27) from a point approximately

3500 feet downstream of Highway 359 upstream to the abandoned Southern Pacific Railroad (SPRR) line along FM 1068. Peak discharges were computed for various points along this stream using the USGS Region 2 equations (Ref. 31), and these are summarized in Table 4.1-1. The computed 100-year water surface profile for Sixmile Creek is presented on Page 1 of Appendix A. The computed water surface elevation at the SPRR line indicates that an expansive area west of Mathis including developed areas within the city limits would be flooded in the 100-year storm event.

As no major outfall channel exists to rapidly transport the storm runoff to Sixmile Creek beyond FM 1068, improvements to and extension of Sixmile Creek to serve this area have been proposed. Recommended improvements to existing Sixmile Creek downstream of SPRR consist of excavating a 100' bottom width channel at the existing slope and are noted in the profile (App. A, p. 2). Peak discharge estimates along the proposed alignment of the Sixmile Creek Extension are presented in Table 4.1-2. Using the design flow associated with the 100-year event, proposed channel and structural dimensions were developed (Tables 4.1-3 and 4.1-4) and a proposed water surface profile including right-of-way requirements was prepared (App. A, p. 2). The proposed alignment and improvement locations are also noted in Figure 4.1-1.

Two outfall channels (BU-01 and BU-02, Fig. 4.1-1) are proposed to serve the eastern portion of Mathis. These channels are intended to provide relief to the area served by the existing stormwater pump station and to establish primary drainage for the



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	LOCATION MAP FOR PROPOSED IMPROVEMENTS	FIG.# 4.1-1
NAISMITH ENGINEERS, INC. Consulting Engineers HDR Infrastructure, Inc. A Centerra Company	THE CITY OF MATHIS, TEXAS	DATE 7/87

area east of the city, which is felt to have a potential for future development. The channels and structures have been sized to contain the 100-year peak discharges of 440 cfs (BU-01) and 380 cfs (BU-02) within the banks. Ten foot bottom width earthen channels with slopes of 0.30 percent are proposed for both channels. Specific improvements and right-of-way requirements are noted in the profiles (App. A, p. 3 and 4).

Economic Evaluation

An economic evaluation in the form of a benefit-cost analysis has been performed for both the improvements to and extension of Sixmile Creek and for the proposed outfall channels east of Mathis. Respective detailed cost estimates for these recommended improvements are included in Appendix B (p. 1 and 2). Average annual benefits and costs and computed benefit-cost ratios are presented for the two recommended flood control projects in Tables 4.1-5 and 4.1-6.

TABLE 4.1-1

Sixmile Creek - Peak Discharge
Mathis Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
3800' Downstream of Hwy 359	0+00	4.35	1680	2290	2760	3260
Hwy 359	38+00	3.35	1290	1720	2060	2410
SPRR	67+50	1.96	790	1030	1210	1400

TABLE 4.1-2

Sixmile Creek Extension - Peak Discharge
Mathis Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Confluence Sixmile Creek	0+00	2.06			1720	1960
FM 1068	20+50	1.42			1020	1180
CR 4	58+50	0.57			570	650

TABLE 4.1-3

Proposed Sixmile Creek Extension - Channel Improvements
Mathis Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
Along SPRR	0+00 to 18+50	1960	120	0.04
FM 1068 to CR 4	18+50 to 58+50	1180	90	0.04
Upstream CR 4	58+50 to 76+00	650	25	0.10

TABLE 4.1-4

Proposed Sixmile Creek Extension - Structural Improvements
Mathis Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
FM 1068	18+50	1180	0.5	4-10'X7' Conc. Boxes
CR 4	58+50	650	0.4	6-5'X5' Conc. Boxes

TABLE 4.1-5

Economic Evaluation
Sixmile Creek Improvements and Extension
Mathis Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	62,000.00
Emergency Cost Reduction	3,100.00
Total Benefits - Existing Conditions	$ 65,300.00
Potential Development Benefits	28,700.00
Total Benefits - Future Conditions	$ 94,000.00
Costs:	
Proposed Improvements	52,000.00
Operations and Maintenance	6,200.00
Total Costs	$ 58,300.00

Benefit-Cost Ratios:

Existing Conditions = 1.1
Future Conditions = 1.6

TABLE 4.1-6

Economic Evaluation
Proposed Sixmile Creek Tributaries
Mathis Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	22,400.00
Emergency Cost Reduction	1,100.00
Total Benefits - Existing Conditions	$ 23,500.00
Potential Development Benefits	50,400.00
Total Benefits - Future Conditions	$ 73,900.00
Costs:	
Proposed Improvements	26,900.00
Operations and Maintenance	3,200.00
Total Costs	$ 30,100.00

Benefit-Cost Ratios:

Existing Conditions = 0.8
Future Conditions = 2.4

4.2 Sinton Area

Description of Flood Control Considerations

The City of Sinton is the county seat of San Patricio County and is centrally located at the crossroads of U.S. Highways 77 and 181 and the Missouri Pacific (MPRR) and Southern Pacific (SPRR) Railroads. Topography within the City is very flat with natural ground elevations ranging from 45.0 to 52.0 ft-msl. Chiltipin Creek and a major tributary, the Sinton South Ditch, are the primary drainageways serving the City and surrounding areas. The Chiltipin Creek watershed includes much of the central portion of the County and has a drainage area of approximately 141 square miles upstream of the South Ditch confluence. The drainage area of the South Ditch at its outfall to Chiltipin Creek is approximately 34.6 square miles.

Historical flooding in the northern and western portions of the City of Sinton has been caused by overflow from Chiltipin Creek, which may be attributed to a combination of inadequate channel and structural capacity. Flood waters have also inundated the southern portion of Sinton (October, 1984) as a result of overflow from Peters Swale (a major tributary of Chiltipin Creek) and the inability of the South Ditch to evacuate these overflows without overbank flooding. Flooding problems due to the overflow of Chiltipin Creek are not limited to the City of Sinton and encompass an expansive rural area as well as the San Patricio County Airport west of the City. Composite floodplain mapping clearly illustrates the extent of flooding in the Sinton area. A recent report (Ref. 20) prepared for the San Patricio County Drainage District (SPCDD) entitled: "Update Report on Flood Mitigation and Improvements to Drainage, Chiltipin Creek Watershed," provides a detailed description of historical flooding and improvements to Chiltipin Creek.

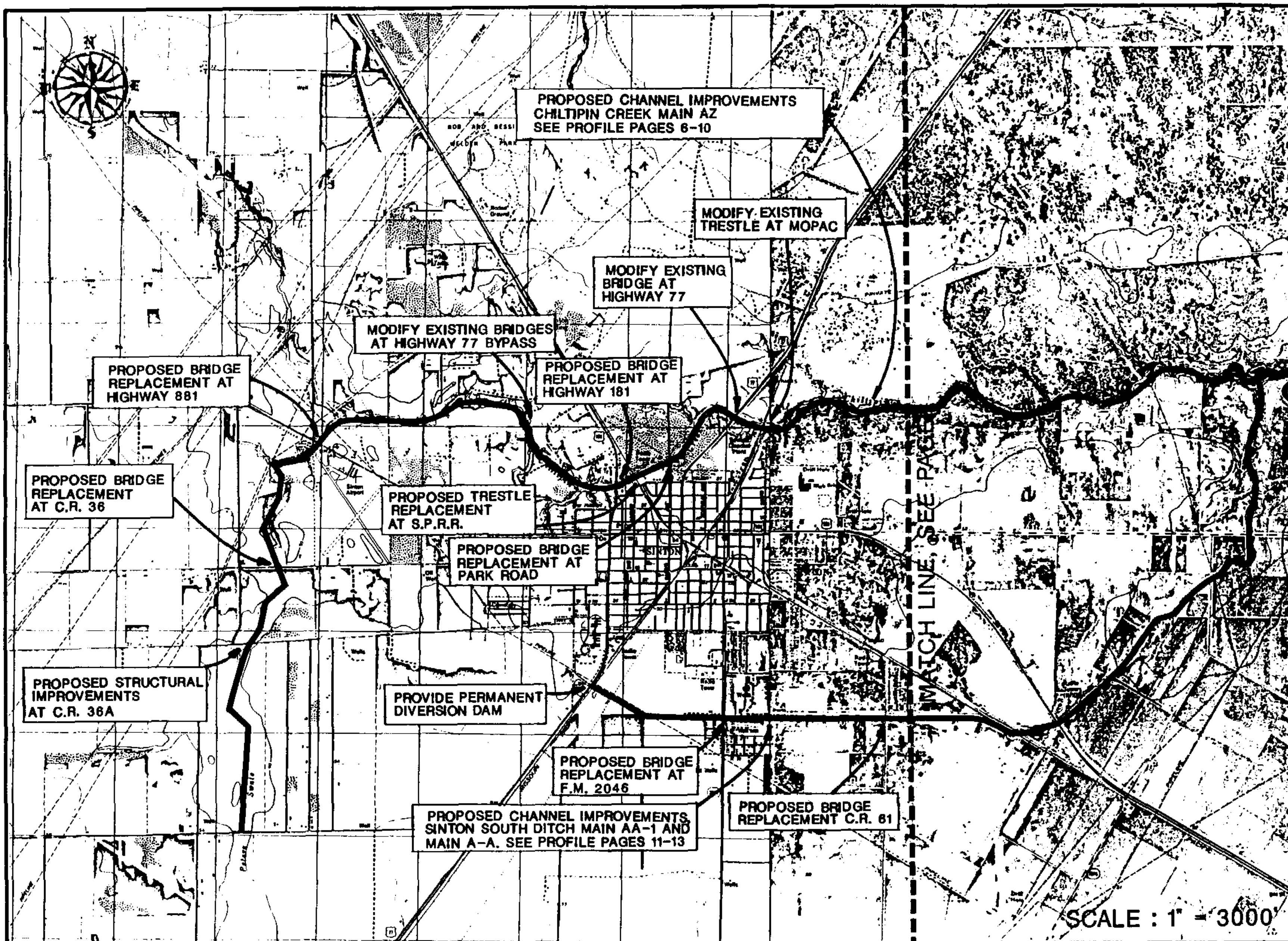
In June of 1985, the San Patricio County Drainage District (SPCDD) implemented an Early Flood Warning System and an emergency response program in an effort to reduce flood damages in the event of a major storm. The System is comprised of three major subsystems: 1) Remote floodwater stage sensors located in the upper reaches of Chiltipin Creek and on Peters Swale; 2) A rainfall observer network; and 3) Stream staff gauges installed at road crossings throughout the County. A letter report was prepared for the SPCDD by Naismith Engineers, Inc. outlining the need for and requirements of the Early Flood Warning System. In addition, the City of Sinton has recently completed the installation of two voice synthesized command speakers to facilitate notification of City residents of the potential for rising floodwaters from the Chiltipin Creek watershed.

Direct computation of peak discharge estimates for various locations along Chiltipin Creek and the Sinton South Ditch using the USGS Region 2 equations was complicated by the overflow of floodwaters from the Peters Swale sub-watershed to the South Ditch rather than these waters following the natural drainage path to Chiltipin Creek. This overflow is a result of the limited hydraulic capacity of the MPRR and U.S. 77 structures crossing Peters Swale and the fact that, according to topographic maps, the floodwaters will overflow to the South Ditch watershed before overtopping U.S. 77. Information provided by the SPCDD Staff, local observers, and recorded high water marks indicate that floodwaters have not overtopped U.S. 77 in the past. Hydraulic computations showed that only about 2500 cfs would pass through the box culverts at U.S. 77 and on to Chiltipin Creek upstream of Sinton, while the remainder would tend to move overland toward the South Ditch. Adjustments to the peak discharge estimates obtained by direct application of the USGS Region 2 equations were accomplished using the methodology described in Section 3.3 of this report. The revised peak discharge estimates for various return periods based on existing watershed conditions are presented in Table 4.2-1 for Chiltipin Creek and Table 4.2-2 for the Sinton South Ditch. Table 4.2-3 presents peak discharge estimates for the South Ditch assuming a channel to convey the Peters Swale overflows to South Ditch was constructed and the structures at the MPRR and U.S. 77 crossings were not modified.

Utilizing extensive surveyed cross-section data and existing topographic mapping, HEC-2 models of Chiltipin Creek from a point 9.56 miles downstream of the South Ditch confluence upstream to FM 1945 and of South Ditch from the Chiltipin Creek confluence upstream to MPRR were developed. Water surface elevations based on the peak discharge estimates for existing watershed conditions were computed by the HEC-2 computer program. The existing 50- and 100-year water surface profiles for Chiltipin Creek and the existing 100-year water surface profile for South Ditch are presented in Appendix A (pp. 5-10 and pp. 11-13, respectively).

A number of potential flood control alternatives including channel and structural improvements, channel rectification or straightening, upstream detention storage, levee construction, and interbasin diversion were considered as alternatives to reduce flooding in the Sinton area. The HEC-2 computer program was utilized to evaluate channel and structural improvements along Chiltipin Creek. Review of the existing water surface profile indicated that channel improvements to Chiltipin Creek would have to begin approximately 10.9 miles downstream of the MPRR bridge to effectively decrease flood elevations within the City of Sinton. A 130' bottom width channel with 3:1 sideslopes extending from station 135+00 east of Sinton to FM 1945 west of the City is proposed herein. Channel rectification downstream of the South Ditch confluence is recommended as it will substantially reduce channel reach length and the associated head losses and excavation volumes. Numerous structural improvements are recommended and these are itemized in Table 4.2-4. Recommended improvements are summarized in plan view in Figure 4.2-1 and profile along with the proposed 100-year water surface elevations and right-of-way requirements on pages 6-10 of Appendix A. Output from the HEC-2 model of Chiltipin Creek indicates that while implementation of the recommended improvements will maintain the 100-year flood elevations within the channel banks through the City of Sinton, some shallow overbank flooding would occur both downstream of the City and upstream of State Highway 881. The 100-year floodplain with and without recommended improvements is indicated on the aerial mapping acquired in the performance of this study.

Construction of a levee immediately west of Sinton extending from U.S. 77 near the South Ditch crossing northward beyond State Highway 881 was proposed by the U.S. Army Corps of Engineers (Ref. 29) in 1975 The proposed levee was intended to provide 50-year protection and woul average 9.5 feet in height and have a topwidth of 12 feet. Channel an structural improvements were also proposed along Chiltipin Creek through Sinton and extending approximately 2 miles downstream. The hy draulic performance of the levee and channel improvements recommended



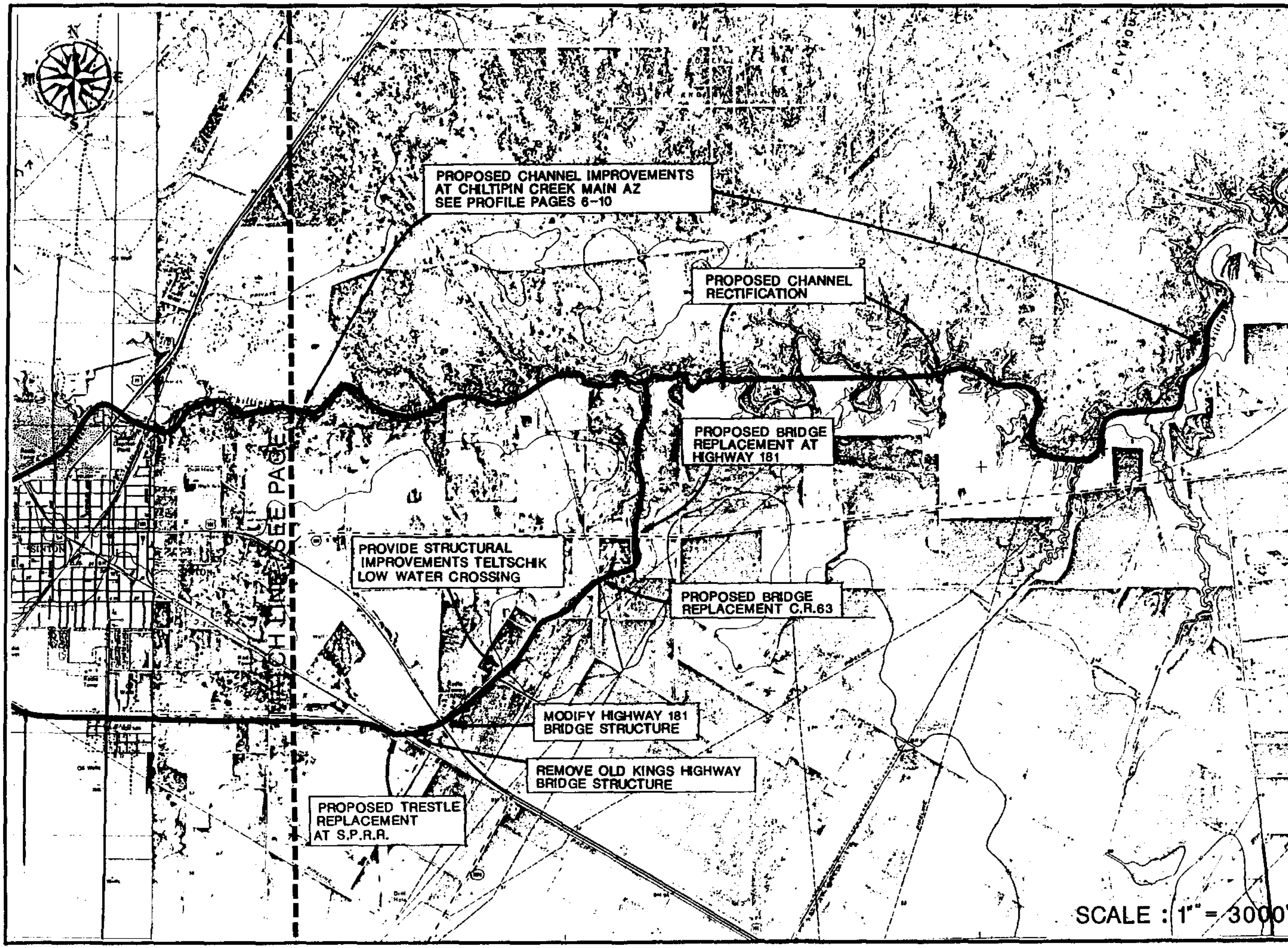
LOCATION MAP FOR
PROPOSED IMPROVEMENTS

THE CITY OF
SINTON, TEXAS

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FIG# 4.2-1a



LOCATION MAP FOR
PROPOSED IMPROVEMENTS

THE CITY OF
SINTON, TEXAS

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

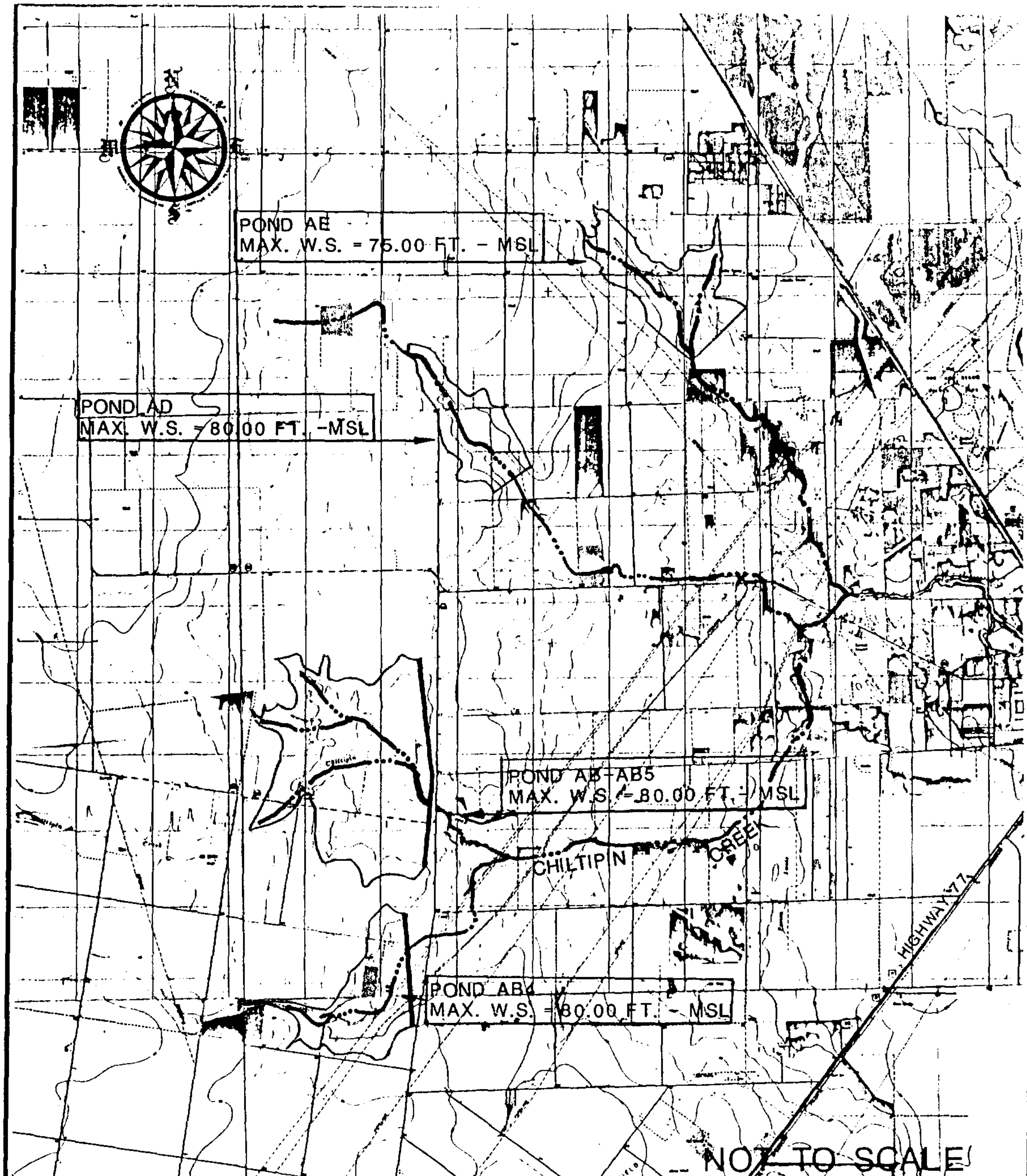
NAISMITH ENGINEERS, INC.
Consulting Engineers

FIG#
4.2-1b

by the Corps of Engineers were not reevaluated in the performance of this study, however, the levee alternative has been considered in the Economic Evaluation portion of this section.

Upstream detention storage was considered as a possible means of reducing the peak discharge of Chiltipin Creek near the City of Sinton. The four potential detention storage sites evaluated are located in Figure 4.2-2. Location of these sites was based primarily on topographic considerations. Available storage volumes were computed and the impacts of full utilization of the storage capacity for each site on the estimated 100-year peak flowrates for Chiltipin Creek at Sinton were quantified using methods described in Section 3.1. Only one of the detention storage sites, Pond AB-AB5, would actually reduce the peak discharge of Chiltipin Creek at Sinton. Due to watershed configuration and the timing of runoff peaks, implementation of the other three ponds would have a negligible impact on the peak discharges at Sinton. Hence, Pond AB-AB5, which could reduce the 100-year peak flowrate at Sinton by 12 to 15 percent, was retained for economic evaluation, while the other sites are not recommended for further consideration.

Channel improvements to Chiltipin Creek would greatly improve the capacity of the Sinton South Ditch by the reduction of backwater effects at its outfall. Even with improvements to Chiltipin Creek, however, the South Ditch is not adequately sized to handle overflows from Peters Swale without flooding the southern portion



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	ALTERNATIVE DETENTION STORAGE LOCATIONS	FIG.# 4.2-2
NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF SINTON	DATE 7/87

of Sinton. Channel and structural improvements for the South Ditch were evaluated using the HEC-2 computer program, and 100-year peak discharge estimates were based on the assumed implementation of a channel to convey Peters Swale overflows to the South Ditch upstream of FM 2046. Recommended improvements include an improved channel section with a 125' bottom width and 3:1 sideslopes and either modification or replacement of existing structures. Recommended structural improvements for the South Ditch are summarized in Table 4.2-5 and Figure 4.2-1 and shown along with the proposed 100-year water surface elevations and right-of-way requirements in the Flood Profile (App. A, pp. 11-13).

Neither the recommended improvements to Chiltipin Creek nor to Sinton South Ditch will alone solve the flooding problems in Sinton. Implementation of both sets of improvements will be required to attain full flood damage reduction and potential development benefits.

Economic Evaluation

Economic evaluation of various flood control alternatives considered for the Sinton area was achieved by the performance of benefit-cost analyses. Tables were prepared summarizing the results of these analyses for the various alternatives affecting flood levels in Chiltipin Creek including channel and structural improvements with rectification (Table 4.2-6), without rectification (Table 4.2-7), and with levee construction (Table 4.2-8). Analysis of the levee alternative was based on construction of a levee west of Sinton as proposed by the U.S. Army Corps of Engineers (Ref. 29) and channel and structural improvements downstream of State Highway 881 as recommended herein. Economic evaluation of the upstream detention storage alternative is summarized in Table 4.2-9. As mentioned above, the effectiveness of improvements to the South Ditch is dependent on improvements to Chiltipin Creek. Hence, the benefits and costs associated with the recommended South Ditch improvements were lumped with those for Chiltipin Creek (channel and structural improvements including rectification) and both were evaluated as one comprehensive flood control project (Table 4.2-10). Detailed cost estimates for alternatives evaluated are included in Appendix B (pp. 3-6).

Upon review of the benefit-cost ratios presented in Tables 4.2-6 through 4.2-10, it is clear that upstream detention storage is not a cost effective means of reducing flood damages in the City of Sinton. In fact, none of the alternatives with the exception of the combined Chiltipin Creek and South Ditch improvements showed a benefit-cost ratio greater than unity for existing conditions. Inclusion of channel rectification as a portion of the channel and structural improvements to Chiltipin Creek saves excavation cost and provides additional flood damage reduction benefits. Although the levee alternative appears competitive with other alternatives evaluated for Chiltipin Creek, the costs of modifying existing internal drainage facilities to effectively drain the portions of the City protected by the levee could make this option prohibitive. On the basis of the economic evaluations for alternative types and combinations of flood control improvements performed for this study, it is concluded that improvements to both Chiltipin Creek and the Sinton South Ditch must be implemented as one comprehensive flood control project to maximize flood control benefits for the Sinton area.

TABLE 4.2-1

Chiltipin Creek - Peak Discharge*
Sinton Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
	0+00	216.8	17250	25010	31500	38650
	373+00	177.6	15910	23090	29090	35690
Upstream of South Ditch Confluence	533+00	141.06	13780	18750	22880	27410
2900' Downstream of MPRR	680+20	130.9	12990	17580	21440	25560
U.S. 77 Bypass	809+10	118.24	12180	16430	20250	23830
Downstream of Hwy 881	887+50	100.47	10760	14370	17860	20630
2200' Downstream of Peters Swale Confluence	976+00	73.45	7900	10160	11990	14010

* Peak Discharge estimates reflect Peters Swale overflow to South Ditch for both existing and improved conditions.

TABLE 4.2-2

South Ditch - Peak Discharge*
Sinton Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Confluence Chiltipin Creek	0+00	34.62	4910	7880	10290	12880
Upstream of U.S. 181	149+50	8.48	1790	3470	4930	6500

* Peak discharge estimates include overflow from Peters Swale for existing conditions.

TABLE 4.2-3

South Ditch - Peak Discharge*
Sinton Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Confluence Chiltipin Creek	0+00	34.62	6500	9980	12780	15790
Hwy 881	45+50	34.02	6410	9790	12470	15330
Downstream of U.S. 181	144+00	30.76	5700	8660	11020	13550
Upstream of U.S. 181	149+50	8.48	3120	5290	7030	8890
FM 2046	256+00	4.97	2570	4560	6160	7870

* Peak discharge estimates include overflow from Peters Swale delivered by a diversion channel upstream of FM 2046 for improved conditions.

TABLE 4.2-4

Chiltipin Creek Main AZ-00 - Structural Improvements
Sinton Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
MPRR	709+50	25560	0.1	Bridge Modification Incl. Slope Paving and Excavation
U.S. 77	719+00	25560	0.3	Bridge Modification Incl. Slope Paving and Excavation
Park Rd.	749+00	25560	0.1	240' Bridge Span
SPRR	762+50	25560	0.2	250' Railroad Trestle
U.S. 181	772+00	25560	0.3	230' Bridge Span
U.S. 77 Bypass	810+50	23830	0.15	Bridge Modification Incl. Slop Paving and Excavation
Hwy 881	888+00	14010	0.2	150' Bridge Span
CR 36A	939+00	14010	0.15	200' Bridge Span
CR 36	970+00	14010	0.15	200' Bridge Span

TABLE 4.2-5

South Ditch - Structural Improvements
Sinton Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
Hwy 881	46+50	15330	0.5	Proposed Bridge Span
CR 63	77+50	15330	0.5	Proposed Bridge Span
Teltschik	123+00	15330	0.5	Replace Low Water Crossing
U.S. 181	146+50	13550	0.1	Bridge Modification Incl. Slope Paving and Excavation
Old Kings Highway	160+00	8890	0.0	Remove Exist. Struct.
SPRR	161+50	8890	0.5	200' Railroad Trestle
CR 61	203+50	8890	0.1	Proposed Bridge Span
FM 2046	256+00	7870	0.5	Proposed Bridge Span

TABLE 4.2-6

Economic Evaluation
Chiltipin Creek Improvements
Channel Rectification and Structural Improvements
Sinton Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	934,800.00
Emergency Cost Reduction	46,700.00
Total Benefits - Existing Conditions	$ 981,500.00
Potential Development Benefits	199,700.00
Total Benefits - Future Conditions	$ 1,181,200.00
Costs:	
Proposed Improvements	1,082,500.00
Operations and Maintenance	129,700.00
Total Costs	$ 1,212,200.00

Benefit-Cost Ratios:

Existing Conditions = 0.8
Future Conditions = 1.0

TABLE 4.2-7

Economic Evaluation
Chiltipin Creek Improvements
Channel and Structural Improvements
Sinton Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	828,600.00
Emergency Cost Reduction	41,400.00
Total Benefits - Existing Conditions	$ 870,000.00
Potential Development Benefits	199,700.00
Total Benefits - Future Conditions	$ 1,069,700.00
Costs:	
Proposed Improvements	1,096,300.00
Operations and Maintenance	131,400.00
Total Costs	$ 1,227,700.00

Benefit-Cost Ratios:

Existing Conditions = 0.7
Future Conditions = 0.9

TABLE 4.2-8

Economic Evaluation
Chiltipin Creek Improvements
Channel Rectification, Structural Improvements,
and Levee Construction
Sinton Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	784,900.00
Emergency Cost Reduction	39,200.00
Total Benefits - Existing Conditions	$ 824,100.00
Potential Development Benefits	79,800.00
Total Benefits - Future Conditions	$ 903,900.00
Costs:	
Proposed Improvements	961,900.00
Operations and Maintenance	115,300.00
Total Costs	$ 1,077,200.00

Benefit-Cost Ratios:

Existing Conditions = 0.8
Future Conditions = 0.8

TABLE 4.2-9

Economic Evaluation
Chiltipin Creek Upstream Detention Storage
Sinton Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	9,400.00
Emergency Cost Reduction	500.00
Total Benefits - Existing Conditions	$ 9,900.00
Potential Development Benefits	10,300.00
Total Benefits - Future Conditions	$ 20,200.00
Costs:	
Proposed Improvements	257,200.00
Operations and Maintenance	30,800.00
Total Costs	$ 288,000.00

Benefit-Cost Ratios:

Existing Conditions = 0.0
Future Conditions = 0.1

TABLE 4.2-10

Economic Evaluation
Chiltipin Creek and South Ditch Improvements
Channel Rectification and Improvements and Structural Improvements
Sinton Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	1,668,900.00
Emergency Cost Reduction	83,400.00
Total Benefits - Existing Conditions	$ 1,752,300.00
Potential Development Benefits	321,800.00
Total Benefits - Future Conditions	$ 2,074,100.00
Costs:	
Proposed Improvements	1,592,500.00
Operations and Maintenance	190,800.00
Total Costs	$ 1,783,300.00

Benefit-Cost Ratios:

Existing Conditions = 1.0
Future Conditions = 1.2

4.3 Odem Area

Description of Flood Control Considerations

The City of Odem is located at the intersection of the Missouri Pacific Railroad and U.S. Highway 77 approximately 5.0 miles north of the Nueces River. The City is situated on the drainage divide between the San Patricio and Chiltipin Creek watersheds. The drainage divide generally runs east-west with the area to the north draining to Peters Swale, a tributary of Chiltipin Creek, and the area to the south draining to the Nueces River and Nueces Bay.

Flooding problems in Odem occur primarily in the northwest portion of the City which outfalls to Peters Swale. Peters Swale is the only major natural drainageway in or near the City of Odem. The drainage area of Peters Swale at U.S. 77 is approximately 16.3 square miles and consists mainly of agriculturally developed land.

Elevations within the City of Odem range from 55 to 77 ft-msl, with the majority of the City between elevation 70 and 75 ft-msl. Water surface elevations in excess of 75.0 ft-msl during major storm events in recent years have caused extensive flooding in northwest Odem where many existing homes and businesses have finished floor elevations of less than 73.0 ft-msl. Most of the flooding in this portion of Odem is attributed to inadequate channel capacity upstream of CR 42 and structural capacity at the CR 42, MPRR, and U.S. 77 crossings. Peters Swale, in its existing state, has insufficient capacity to evacuate runoff rapidly. With an approximate flowline of 66.0 ft-msl in northwest Odem, only 2.0 feet of water in Peters Swale produces street flooding in the area of Cook and Bullard Streets.

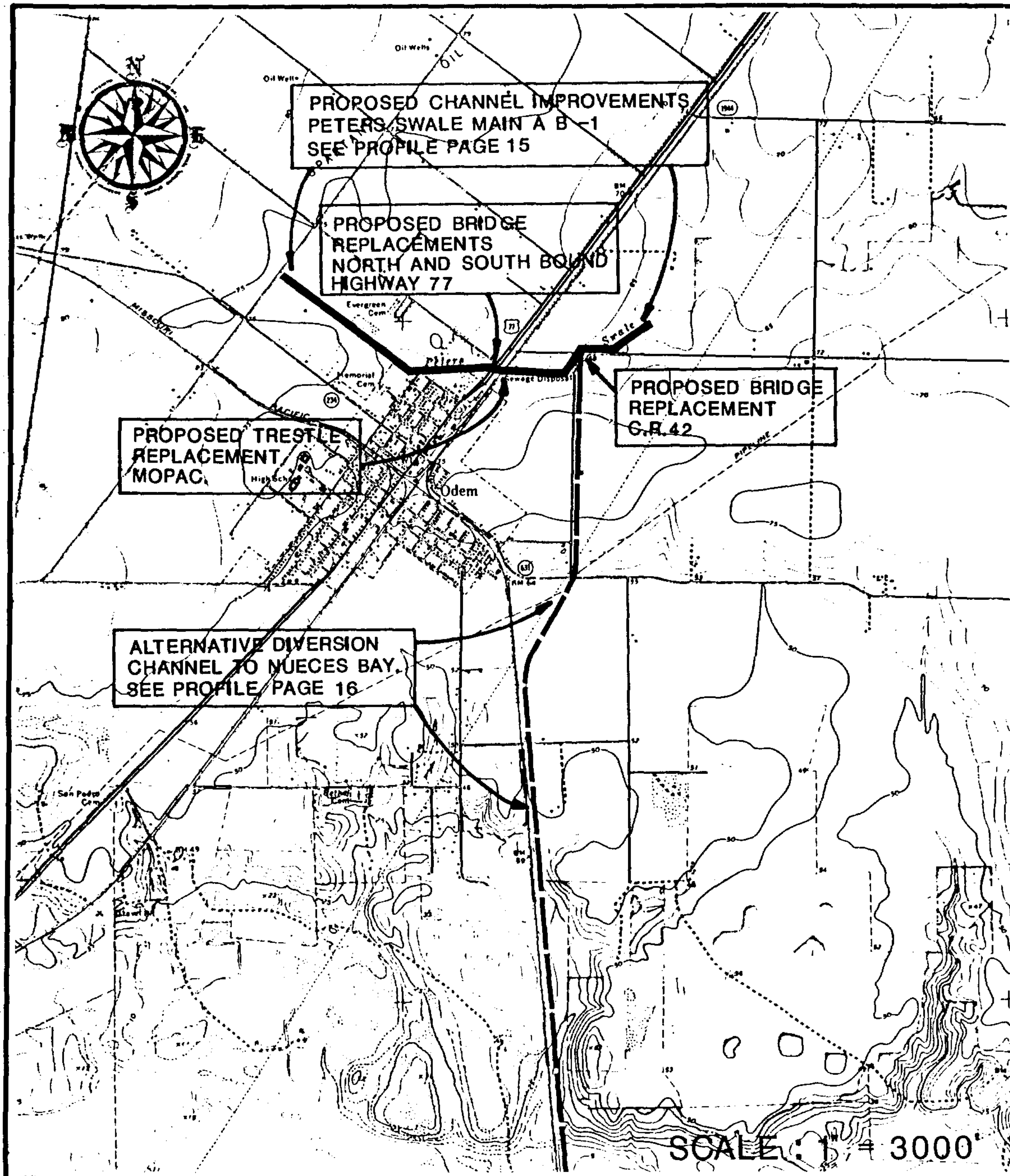
Runoff from the southern portion of the City does not result in the type of flooding that has been experienced in northwest Odem. One area known to have suffered flooding problems south of Odem, however, is the Bethel Estates area. Recommended improvements to alleviate problems in this area have been presented in a recent report (Ref. 17) prepared by Naismith Engineers, Inc. and were not addressed in this study.

Basin Analyses and Recommended Improvements

Using surveyed stream cross-section data and structural dimensions for the various stream crossings, a HEC-2 model of Peters Swale extending from approximately 2800' downstream of FM 1944 to CR 37 north and west of the City was developed. Peak discharge estimates were computed using the USGS Region 2 equations and are presented in Table 4.3-1. Backwater analysis of Peters Swale in its existing condition indicated that the 100-year runoff would cause considerable overbank flooding throughout the entire reach modelled. Head losses due to inadequate channel and structural capacity totalled more than 6.0 feet between points immediately downstream of CR 42 and immediately upstream of U.S. 77. The existing 100-year Flood Profile is included as pages 14 and 15 of Appendix A.

The HEC-2 computer program was used to determine that flooding in northwest Odem could be substantially reduced by widening and deepening portions of the existing channel from a point 1000' downstream of CR 42 upstream to CR 37 and replacing the structures at CR 42, MPRR, and U.S. 77. A 200' bottom width earthen channel with 3:1 sideslopes is proposed and the recommended structural improvements are noted in Figure 4.3-1 and Table 4.3-2. Implementation of these improvements would lower the 100-year water surface elevation upstream of U.S. 77 by approximately 4.5 feet as is apparent in the proposed water surface profile (App. A, p. 15). These improvements will not alleviate all street flooding in northwest Odem; however, they will reduce street flooding during a 100-year storm event to depths of less than 2.0 feet and eliminate the flooding of homes. More frequent storms will remain within the banks of the channel upstream of U.S. 77.

A diversion channel originating at Peters Swale upstream of CR 42 and proceeding southward paralleling the MPRR to outfall at Nueces Bay was evaluated in this study. Based on a 100-year design discharge of 6760 cfs, a 75' bottom width channel section at



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	LOCATION MAP FOR PROPOSED IMPROVEMENTS	FIG.# 4.3-1
NAISMITH ENGINEERS, INC. Consulting Engineers HDR Infrastructure, Inc. A Centerra Company	THE CITY OF ODEM, TEXAS	DATE 7/87

a slope of 0.25 percent is proposed. The alignment of this diversion channel is shown in Figure 4.3-1 and the recommended channel dimensions, structural improvements, and right-of-way requirements are presented in the Flood Profile (App. A, p. 16). Note that most of the recommended channel and structural improvements for Peters Swale would have to be implemented in conjunction with the diversion channel in order to prevent the flooding of homes in northwest Odem.

Economic Evaluation

The economic evaluations of recommended Peters Swale improvements and the diversion channel from Peters Swale to Nueces Bay were accomplished by the performance of benefit-cost analyses. The results of these analyses are presented in Tables 4.3-3 and 4.3-4 and detailed alternative cost estimates are included in Appendix B (pp. 7-8).

Review of Table 4.3-3 indicates that the flood control benefits associated with the recommended Peters Swale improvements are slightly less than the cost of implementation and maintenance. It should be noted, however, that planning by the State Department of Highways and Public Transportation and the Missouri Pacific Railroad is underway at this time to replace the existing structures at U.S. 77 and MPRR. If the costs of these structural improvements were deducted from the total cost of recommended improvements, the benefit-cost ratios for existing and proposed conditions would increase to 1.6 and 1.8, respectively.

With regard to the alternative diversion channel to Nueces Bay, flood damage reduction benefits associated with substantially reduced flooding in the southern portion of Sinton and the northwest portion of Odem contribute to a benefit-cost ratio of 1.0 for existing conditions. Reduction of flood damages to rural and agricultural areas along Peters Swale east of U.S. 77 were not quantified and, therefore, neglected in the computation of benefits. Flood control benefits to the City of Odem associated with this alternative are essentially the same as those estimated for channel and structural improvements to Peters Swale only. If the costs of structural replacement at U.S. 77 and MPRR are deducted, the benefit-cost ratio for existing conditions would increase from 1.0 to 1.2.

TABLE 4.3-1

Peters Swale - Peak Discharge
Odem Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
FM 1944	29+00	20.58	3690	5060	6160	7340
CR 42	166+50	16.37	3380	4650	5670	6760
MPRR	187+50	15.76	3280	4500	5480	6530
U.S. 77	189+00	15.67	3270	4490	5470	6510
CR 37	242+00	12.53	2770	3780	4580	5440

TABLE 4.3-2

Peters Swale - Structural Improvements
Odem Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
CR 42	166+50	6760	0.1	220' Bridge Span
MPRR	187+50	6530	0.1	220' Railroad Trestle
U.S. 77	189+00	6510	0.1	220' Bridge Span

TABLE 4.3-3

Economic Evaluation
Peters Swale Improvements
Odem Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	180,000.00
Emergency Cost Reduction	9,000.00
Total Benefits - Existing Conditions	$ 189,000.00
Potential Development Benefits	58,800.00
Total Benefits - Future Conditions	$ 215,000.00
Costs:	
Proposed Improvements	197,900.00
Operations and Maintenance	23,700.00
Total Costs	$ 221,600.00

Benefit-Cost Ratios:

Existing Conditions = 0.9
Future Conditions = 1.0

TABLE 4.3-4

Economic Evaluation
Peters Swale Improvements and Diversion
Odem Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	543,300.00
Emergency Cost Reduction	27,200.00
Total Benefits - Existing Conditions	$ 570,500.00
Potential Development Benefits	66,300.00
Total Benefits - Future Conditions	$ 604,000.00
Costs:	
Proposed Improvements	504,000.00
Operations and Maintenance	60,400.00
Total Costs	$ 564,400.00

Benefit-Cost Ratios:

Existing Conditions = 1.0
Future Conditions = 1.1

4.4 Taft Area

Description of Flood Control Considerations

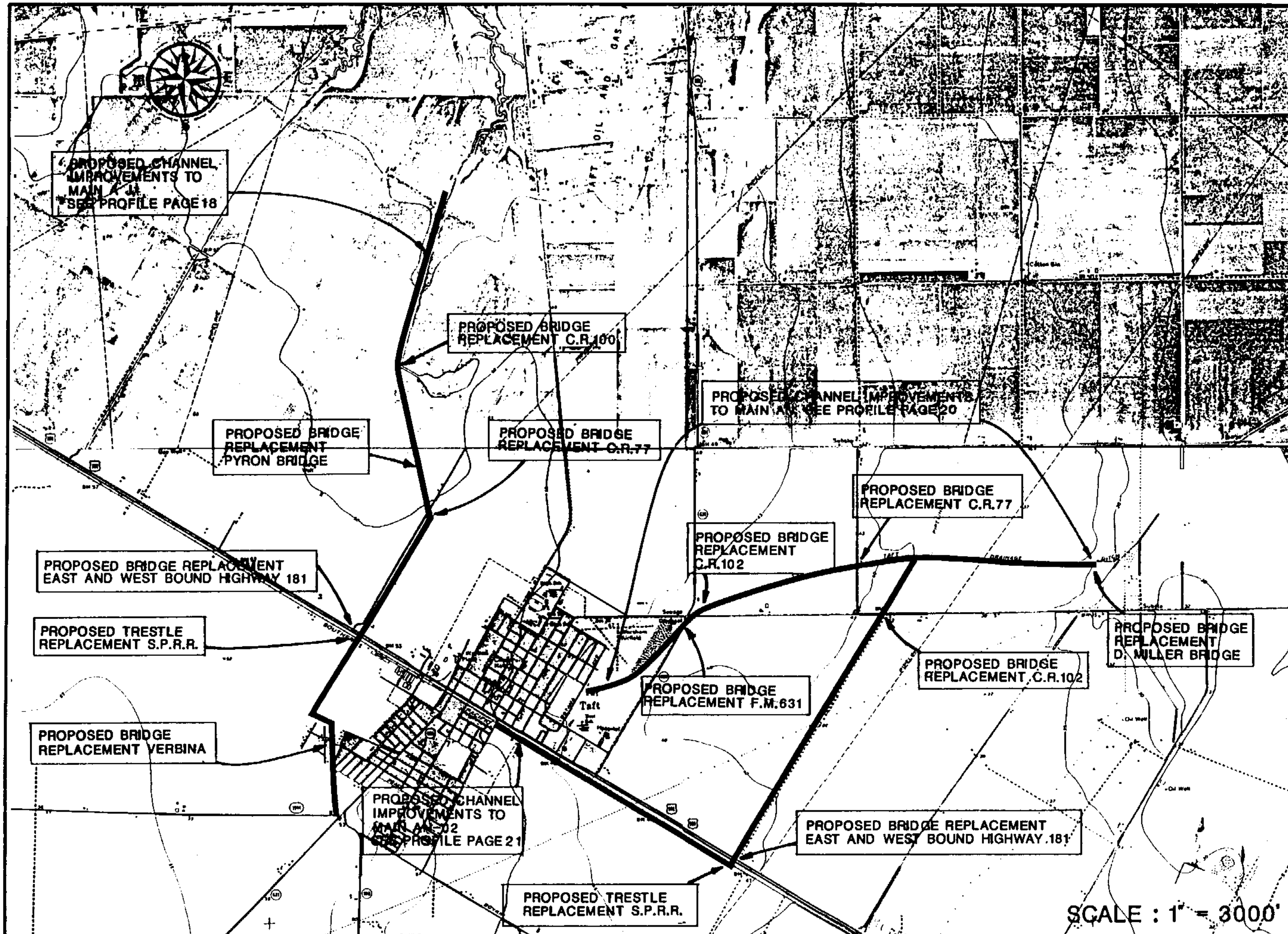
The City of Taft is located in central San Patricio County approximately 8.0 miles southeast of the City of Sinton along U.S. Highway 181. The City is located on a drainage basin divide which runs generally north-south and sheds runoff to the northwest and to the northeast. Primary outfall drainage of the City and surrounding area is provided by two outfall ditches. Main AJ (John Deere Ditch) drains the western portion of the City and a large agricultural area south and west of the City and outfalls to Chiltipin Creek approximately 3.0 miles downstream of U.S. 181. The drainage area of Main AJ at the SPRR crossing is approximately 6.4 square miles. Main AN provides drainage for the eastern areas of the City and the residential development in northern Taft. Main AN also drains a considerable amount of agricultural land to the east of the City and outfalls to the Aransas River near Copano Bay. AN-02, which is a tributary of Main AN, drains areas of southeastern Taft along Toland Avenue and a considerable amount of farmland.

Flooding has occurred in the southwestern portion of the City due to overflow of Main AJ and its tributary ditches. Several areas in northeastern Taft as well as adjacent farmland have been subject to flooding from Main AN. Since Taft is located on the top of the drainage divide, it is felt that flooding problems in these areas have been due primarily to insufficient channel capacity of Main AN and Main AJ. This, in turn, affects the performance of the internal drainage systems in the developed areas of the City.

Basin Analyses and Recommended Improvements

The HEC-2 computer program was used to model Main AJ, Main AN, and AN-02 in an effort to determine the existing 100-year flood levels and floodplain boundaries in and around the City of Taft. Peak discharges were computed at various stream locations using the USGS Region 2 equations and are presented in Table 4.4-1 (Main AJ), Table 4.4-2 (Main AN), and Table 4.4-3 (AN-02). The existing 100-year Flood Profiles for Main AJ, Main AN, and AN-02 are included in Appendix A, pages 17 and 18, 19 and 20, and 21, respectively.

Recommended improvements for Main AJ include channel enlargement from CR 98 to FM 1944 and structural improvements to all major transportation route crossings. Specific channel improvements are summarized in Table 4.4-4 and structural improvements are noted in Table 4.4-5 and Figure 4.4-1. The proposed 100-year water surface profile is included in Appendix A (p. 17-18) along



LOCATION MAP FOR PROPOSED IMPROVEMENTS

THE CITY OF TAFT, TEXAS

FLOOD CONTROL STUDY SAN PATRICIO COUNTY

SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC. Consulting Engineers

FIG# 4.4-1

with required right-of-way requirements. The 100-year water surface elevation in the southwestern potion of the City can be reduced from an existing 56.5 ft-msl to approximately 53.0 ft-msl with implementation of the proposed improvements. This reduction will accommodate existing and future drainage facilities in the area and eliminate a considerable amount of overland flooding. Implementation of the recommended improvements will not keep the 100-year flood completely within the banks of the channel at all locations; however, street flooding will be reduced to less than 1.0 foot and floodwaters should not enter any homes.

Overbank flooding in northeast Taft can be substantially reduced by implementation of the improvements recommended for Main AN and extension of AN-02. Recommended improvements to Main AN begin at or near the David Miller Bridge located upstream of CR 81 and continue upstream to a point near the elevated storage tank in northeast Taft. Improvements to AN-02 begin at its confluence with Main AN and extend south beyond the SPRR tracks and westward along the SPRR right-of-way to Toland Avenue. Structure replacement is recommended at all transportation route crossings of either drainageway. Recommended channel improvements to Main AN and AN-02 are summarized in Tables 4.4-6 and 4.4-7 and structural improvements are noted in Figure 4.4-1 and Tables 4.4-8 and 4.4-9. The Flood Profiles included in Appendix A show the proposed 100-year water surface elevations and right-of-way requirements for Main AN (pp. 19-20) and AN-02 (p. 21). Reduction of the 100-year floodplain is apparent upon visual inspection of the aerial mapping of San Patricio County prepared as a portion of this study. Implementation of the recommended improvements to Main AN and AN-02 will facilitate future development of land between Retama Avenue and CR 77 by removing it from the floodplain and providing adequate outfall drainage.

Economic Evaluation

Detailed cost estimates for implementation of the proposed improvements to Main AJ, Main AN, and AN-02 are included in Appendix B (pp. 9-11). Benefit-cost analyses for Main AJ and for Main AN and AN-02 are summarized in Tables 4.4-10 and 4.4-11, respectively. The relatively high computed benefit-cost ration of 1.6 for Main AJ improvements may be attributed primarily to the removal of developed areas in southwestern Taft from the floodplain. Although substantial potential development benefits may be associated with improvements to Main AN and AN-02, flood damage reduction to existing development is insufficient and the cost of improvements too great to generate a benefit-cost ratio greater than 0.7 for future conditions.

TABLE 4.4-1

Main AJ-00 - Peak Discharge
Taft Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Confluence with Chiltipin Creek		14.59	2990	4080	4950	5880
FM 881	1+00	14.35	2940	4000	4850	5750
5000' Upstream of FM 881	51+00	12.82	2390	3210	3850	4530
4500' Downstream of CR 71	133+00	9.36	1970	2620	3130	3670
CR 71	178+00	6.90	1550	2040	2420	2820
SPRR	226+00	6.40	1480	1960	2320	2700
Hidalgo St.		4.92	1270	1660	1960	2280

TABLE 4.4-2

Main AN-00 - Peak Discharge
Taft Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
CR 98	1+00	17.78	3300	4520	5480	6510
CR 85	40+50	17.55	3150	4280	5180	6140
CR 100	141+00	15.20	3000	4100	4960	5890
Confluence AN-02	262+00	7.76	2040	2750	3310	3910
CR 77	287+00	7.25	1920	2590	3110	3660
FM 631	350+00	2.16	930	1220	1440	1680
Retama Ave.	387+00	0.49	400	520	610	700

TABLE 4.4-3

Main AN-02 - Peak Discharge
Taft Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs)			
			10	25	50	100
CR 102	27+00	3.50	1150	1510	1800	2090
U.S. 181	118+00	2.30	870	1140	1340	1560
FM 631	173+00	1.00	600	780	920	1060

TABLE 4.4-4

Main AJ-00 - Channel Improvements
Taft Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
CR 98 to Pyron Bridge	75+00 to 164+00	4530	75	0.05
Pyron Bridge to Downstream FM 1944	164+00 to 267+00	3670	50	0.05

TABLE 4.4-5

Main AJ-00 - Structural Improvements
Taft Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
CR 100	130+00	3670	0.1	100' Bridge Span
Pyron Bridge	164+00	2820	0.1	100' Bridge Span
CR 71	178+00	2820	0.1	100' Bridge Span
U.S. 181	223+00	2700	0.1	100' Bridge Span
SPRR	225+00	2700	0.1	100' Railroad Trestle
Verbina	247+00	2280	0.1	75' Bridge Span

TABLE 4.4-6

Main AN-00 - Channel Improvements
Taft Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
David Miller Bridge to Upstream AN-02	209+00 to 270+00	5150	100	0.05
Upstream AN-02 to Hwy 631	270+00 to 350+00	3660	75	0.05
Hwy 631 to 1000' Downstream Retama	350+00 to 386+00	1680	50	0.05

TABLE 4.4-7

Main AN-02 - Channel Improvements
Taft Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
Confluence AN-00 to SPRR	0+00 to 120+00	2090	50	0.05
SPRR to Toland Ave.	120+00 to 188+00	1560	30	0.164

TABLE 4.4-8

Main AN-00 - Structural Improvements
Taft Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
David Miller Bridge	209+00	5150	0.1	125' Bridge Span
CR 77	287+00	3660	0.1	125' Bridge Span
CR 102	343+00	1680	0.1	125' Bridge Span
Hwy 631	350+00	1680	0.1	105' Bridge Span

TABLE 4.4-9

Main AN-02 - Structural Improvements
Taft Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
CR 102	27+00	2090	0.1	100' Bridge Span
U.S. 181	118+00	1560	0.1	100' Bridge Span
SPRR	120+00	1560	0.1	100' Bridge Span

TABLE 4.4-10

Economic Evaluation
Main AJ-00 Improvements
Taft Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	366,800.00
Emergency Cost Reduction	18,300.00
Total Benefits - Existing Conditions	$ 385,100.00
Potential Development Benefits	25,700.00
Total Benefits - Future Conditions	$ 410,800.00
Costs:	
Proposed Improvements	208,500.00
Operations and Maintenance	25,000.00
Total Costs	$ 233,500.00

Benefit-Cost Ratios:

Existing Conditions = 1.6
Future Conditions = 1.8

TABLE 4.4-11

Economic Evaluation
Main AN-00 and AN-02 Improvements
Taft Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	125,800.00
Emergency Cost Reduction	6,300.00
Total Benefits - Existing Conditions	$ 132,100.00
Potential Development Benefits	121,800.00
Total Benefits - Future Conditions	$ 253,900.00
Costs:	
Proposed Improvements	310,200.00
Operations and Maintenance	37,200.00
Total Costs	$ 347,400.00

Benefit-Cost Ratios:

Existing Conditions = 0.4
Future Conditions = 0.7

4.5 Portland Area

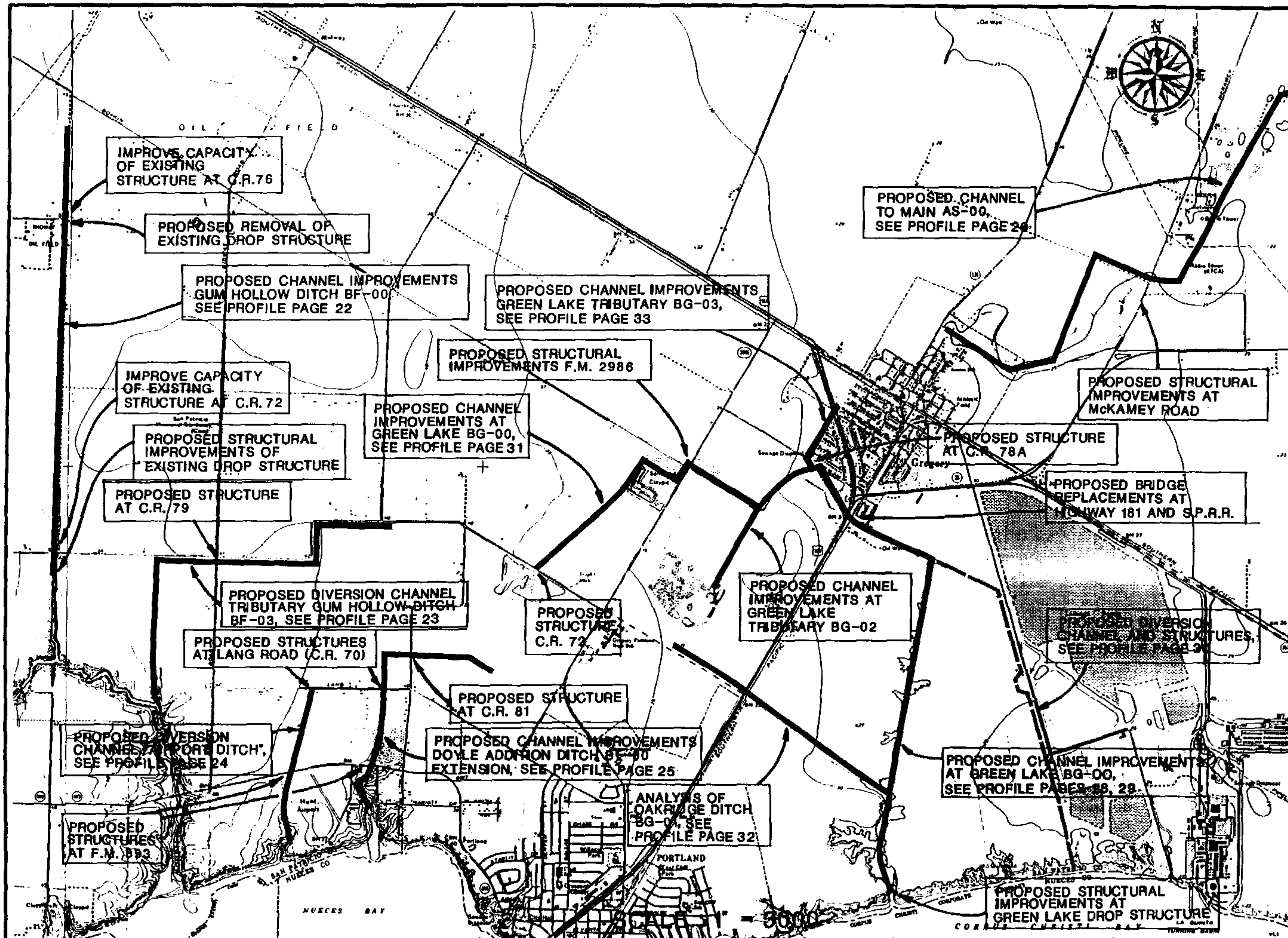
Description of Flood Control Considerations

The City of Portland is the largest community in San Patricio County, with an estimated population of 12,000. Linked directly to the City of Corpus Christi by U.S. Highway 181 across the mouth of Nueces Bay, Portland is felt to have a great potential for future population growth and land development. Discussions with members of the San Patricio County Commissioners Court as well as the City Council of Portland (App. C, pp. 2-3 amd pp. 10-11) indicated that the primary areas of interest with regard to flood control in the Portland area pertain to providing adequate outfall drainage to areas west, north, and east of the city. Specific projects in which an interest was expressed included a potential diversion channel to Gum Hollow (BF-00), evaluation of drainage near the Doyle Addition (BT-00) and Hunt Airport, and evaluation of the performance of Green Lake (BG-00) and the tributary Oakridge Ditch (BG-01). Green Lake and the Oakridge Ditch are addressed in Section 4.7 concerning the Portland/Gregory Area.

Basin Analyses and Recommended Improvements

Peak discharge estimates were computed and hydraulic analyses were performed for Gum Hollow (BF-00) using the HEC-2 computer program in order to facilitate the preliminary design of a potential tributary diversion channel paralleling CR 72. The peak discharge estimates for the Gum Hollow watershed are presented in Table 4.5-1 and were used in the computation of the existing 100-year water surface profile. Proposed channel and structural improvements to prevent overbank flooding in the Gum Hollow watershed upstream of Drop Structure #2 near CR 72 are presented in Tables 4.5-2 and 4.5-3, respectively, and also indicated in the Flood Profile (App. A, p. 22) and Location Map for proposed improvements (Fig. 4.5-1). Improvements to Gum Hollow downstream of Drop Structure #2 are required neither for existing conditions nor in the event of the construction of a tributary diversion channel from the Green Lake watershed.

Construction of a tributary diversion channel from the Green Lake watershed which would outfall to Gum Hollow approximately 3500' upstream of FM 893 is considered herein as a potential means of providing a major outfall to an area roughly bounded by CR 70, CR 74, CR 79, and CR 81. The proposed alignments of this diversion channel and others considered in this section are noted in Figure 4.5-1. This is a very flat area which tends to drain very slowly to the east and, ultimately, into Green Lake. Removal of this area from the Green Lake watershed can substantially reduce peak discharge along Main BG-00 through Gregory and at Green Lake.



LOCATION MAP FOR PROPOSED IMPROVEMENTS

GREGORY / PORTLAND AREA

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FIG# 4.5-1

Design (100-year) peak discharges for the proposed Gum Hollow tributary channel are presented in Table 4.5-4, and channel and structural improvements are presented in Table 4.5-5 and 4.5-6, respectively. The proposed water surface profile for this diversion channel is in Appendix A (p. 23).

A small channel has been proposed west of the Hunt Airport and extending north to Lang Road. This channel is intended to serve several purposes, including reduction of the existing Green Lake (BG-00) and Doyle Addition (BT-00) watersheds and establishment of a primary outfall for the area north of the Hunt Airport. Design (100-year) peak discharges for this watershed are presented in Table 4.5-7, recommended channel and structural improvements are summarized in Tables 4.5-8 and 4.5-9, and a channel and water surface profile is provided in Appendix A (p. 24).

Improvements to and extension of the Doyle Addition Ditch (BT-00) located west of Portland have been considered in the performance of this study to provide primary outfall drainage to the developing area near the intersection of Lang Road and CR 81. Drainage improvements north of the Doyle Addition and the Hunt Airport are included in the "Comprehensive Plan Summary" (Ref. 2) for the City of Portland. Design (100-year) peak discharge estimates, recommended channel dimensions, and recommended structural improvements are presented in Tables 4.5-10, -11, and -12, respectively. A profile detailing the proposed water surface elevations, channel invert, and various structural improvements is included in Appendix A (p. 25).

Economic Evaluation

An economic evaluation of the recommended improvements to Gum Hollow is presented in Table 4.5-13, and a detailed cost estimate for these improvements is included in Appendix B (p. 12). It is apparent in Table 4.5-13 that the benefits of these improvements are not great as damages to existing development in the area are limited and future development is not expected at this time. Hence, the benefit-cost ratio for this project is quite low. Computed existing 100-year water surface elevations indicate that some stream runoff could spill eastward across CR 77 north of CR 72 into the Green Lake watershed and ultimately impact flood levels in the Portland/Gregory area. Implementation of recommended improvements in Gum Hollow could, therefore, slightly reduce flood damage and enhance property values in the Green Lake watershed. These benefits are marginal, however, and were not incorporated in the economic analysis.

The recommended flood control improvements including the Gum Hollow tributary diversion channel, Airport Ditch, and Doyle Addition Ditch, were not subjected to independent economic evaluation as the associated benefits are directly related to flood damage reduction and potential development benefits in the Green Lake watershed. Detailed cost estimates for these proposed channels and improvements are included in Appendix B (pp. 13-15). Economic evaluation of flood control alternatives affecting the Green Lake watershed is included in the Portland/Gregory sub-section of this Section.

TABLE 4.5-1

Gum Hollow - Peak Discharge
Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
FM 893 (Existing Watershed)	19+50	17.57	4040	5640	6930	8320
FM 893 (Proposed Watershed)						
Confluence BF-01	87+00	11.64	2910	4000	4870	5800
CR 72	120+00	10.50	2780	3810	4640	5530
1500' Upstream of CR 74	187+00	4.73	1740	2360	2850	3380
CR 76	222+00	3.32	1340	1800	2160	2540

TABLE 4.5-2

Gum Hollow - Channel Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
Upstream CR 72	112+00 to 173+00	5530	100	0.071
Downstream CR 76	173+00 to 220+00	3380	80	0.10
Upstream CR 76	220+00 to 252+00	2540	80	0.10

TABLE 4.5-3

Gum Hollow - Structural Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
Struct. #2	112+00	5530	4.3	71' Weir Notch Width, Crest = 30.0 ft-msl
CR 72	120+00	5530	0.1	130' Bridge Span
Struct. #3	220+00	2540	0.0	Remove Structure
CR 76	222+00	2540	0.7	Conc. Channel Lining

TABLE 4.5-4

Proposed Gum Hollow Tributary - Peak Discharge
Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs)			
			10	25	50	100
3500' Downstream of CR 79	30+00	3.28				2080
CR 79	65+00	2.13				1550
3200' Upstream of CR 79	97+00	1.14				1000

TABLE 4.5-5

Proposed Gum Hollow Tributary - Channel Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
Downstream CR 79	5+00 to 65+00	2080	40	0.06
Upstream CR 79	65+00 to 97+00	1550	40	0.08
	97+00 to 131+00	1000	40	0.08

TABLE 4.5-6

Proposed Gum Hollow Tributary - Structural Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
CR 79	65+00	1550	0.70	3-10'X10' Conc. Boxes

TABLE 4.5-7

Proposed Airport Ditch - Peak Discharge
Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs)			
			10	25	50	100
Nueces Bay	0+00	0.67				735
FM 893	36+00	0.45				560
Lang Road	63+00	0.19				255

TABLE 4.5-8

Proposed Airport Ditch - Channel Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
Downstream FM 893	20+00 to 36+00	740	8	0.10
FM 893 to Lang Rd	36+00 to 63+00	560	8	0.10

TABLE 4.5-9

Proposed Airport Ditch - Structural Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
FM 893	36+00	560	0.9	2-6'X8' Conc. Boxes
Lang Road	63+00	260	0.6	2-5'X5' Conc. Boxes

TABLE 4.5-10

Proposed Doyle Addition Ditch - Peak Discharge
Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Nueces Bay	0+00	1.22				1240
Confluence BT-01	25+00	1.01				1080
FM 893	28+00	0.80				830
Lang Road	53+00	0.63				580
CR 81	75+00	0.41				430

TABLE 4.5-11

Proposed Doyle Addition Ditch - Channel Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
Downstream FM 893	17+00 to 28+00*	1080	15	0.20
FM 893 to Lang Rd	28+00 to 53+00	830	10	0.20
Lang Rd to CR 81	53+00 to 75+00	580	10	0.075
Upstream CR 81	75+00 to 98+00	430	10	0.05

* Concrete lined channel with 1:1 sideslopes.

TABLE 4.5-12

Proposed Doyle Addition Ditch - Structural Improvements
Portland Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
FM 893	28+00	830	0.10	40' Bridge Span
Lang Rd	53+00	580	0.85	2-7'X7' Conc. Boxes
CR 81	75+00	430	0.50	2-7'X7' Conc. Boxes

TABLE 4.5-13

Economic Evaluation
Gum Hollow Improvements
Portland Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	11,800.00
Emergency Cost Reduction	600.00
Total Benefits - Existing Conditions	$ 12,400.00
Potential Development Benefits	Unquantified
Total Benefits - Future Conditions	$ 12,400.00
Costs:	
Proposed Improvements	129,700.00
Operations and Maintenance	15,500.00
Total Costs	$ 145,200.00

Benefit-Cost Ratio:

Existing Conditions = 0.1

4.6 Gregory Area

Description of Flood Control Considerations

The City of Gregory is located approximately 3 miles northeast of the City of Portland. Although the natural topography in the Gregory area indicates that drainage should generally move in an east or northeasterly direction, the drainage for the majority of the city has been directed to the south and Green Lake (BG-00). U.S. 181 and Highway 361 serve as the watershed boundary between portions of Gregory draining south to Green Lake and northeast in a somewhat poorly defined channel designated Main AS-00 to McCampbell Slough and Port Bay. Flooding problems and flood control alternatives associated with Green Lake are considered in Section 4.7.

Limited flooding has been experienced in northern and eastern Gregory due to the apparent inadequacy of the Main AS-00 channel. The extent of urban flooding in this area is unknown as the Flood Insurance Rate Map (Ref. 4) for the City of Gregory does not indicate flooding within the city limits. The Flood Insurance Rate Map for the Unincorporated Areas of San Patricio County (Ref. 9), however, does indicate flooding up to the northern city limits of Gregory. Requesting a FEMA Restudy of the area to resolve this discrepancy should be considered. Improvements to this outfall channel have been limited in the past due to obstructions of the natural drainageway, including a sanitary landfill and oil refinery.

Basin Analyses and Recommended Improvements

Peak discharge estimates for the Main AS-00 watershed were developed using the USGS Region 2 equations (Ref. 31) and are presented in Table 4.6-1. A realignment of the existing drainageway as indicated in Figure 4.5-1 has been proposed, and channel and structural dimensions have been prepared and included in Tables 4.6-2 and 4.6-3, respectively. Channel dimensions and structural improvements were prepared to convey the peak runoff from the 100-year design storm within the banks. A profile indicating the proposed channel invert and 100-year water surface elevation is included in Appendix A (pp. 26-27).

Economic Evaluation

Insufficient information is available to determine the extent of urban areas flooded in northeast Gregory and rural areas beyond the city limits during the 100-year design storm; therefore, a benefit-cost analysis consistent with others presented herein could not be prepared. A preliminary cost estimate has been prepared, however; and it is included in Appendix C (p. 16).

TABLE 4.6-1

Main AS-00 - Peak Discharge
Gregory Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
McCampbell Road	84+00	5.38	1270	1650	1950	2260
Richardson Road	164+00	4.09	1040	1340	1570	1820
McKamey Road	254+00	2.72	940	1230	1450	1680

TABLE 4.6-2

Main AS-00 - Channel Improvements
Gregory Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
McCampbell Road to Richardson Road	84+00 to 164+00	2260	75	0.05
Richardson Road to McKamey Road	164+00 to 254+00	1820	75	0.033
McKamey Road to Downstream Hwy 136	254+00 to 336+00	1680	75	0.033

TABLE 4.6-3

Main AS-00 - Structural Improvements
Gregory Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
McCampbell Road	84+00	2260	0.1	130' County Road Bridge Span
Richardson Road	164+00	1820	0.8	5-8'X8' Conc. Boxes
McKamey Road	254+00	1680	1.1	4-8'X8' Conc. Boxes

4.7 Portland/Gregory Area

Description of Flood Control Considerations

The San Patricio County Commissioners and the Cities of Portland and Gregory have expressed concern with regard to the performance of Green Lake (Main BG-00) and its tributary channels including the Oakridge Ditch (App. C, pp. 2-4 and pp. 10-11). This system of channels currently provides outfall drainage to an 11.44 square mile area upstream of the spillway structure at Green Lake. The drainage area extends east to the Reynolds Aluminum Plant, includes the area between the two cities, is bounded on the north by U.S. 181, and extends west beyond CR 79. Approximately 70 percent of the watershed concentrates upstream of the U.S. 181 and SPRR crossings of Main BG-00 immediately south of Gregory. Due to their proximity to the Green Lake channel and a lack of topographic relief, the south and western portions of Gregory have been subjected to frequent flooding. Inadequate primary outfall channel and structural capacity also contribute to frequent flooding of the area. In addition, limited spillway capacity and an existing spillway crest elevation of 22.36 ft-msl at Green Lake have a significant impact on upstream flood levels. Plans to modify and improve the hydraulic capacity of the Green Lake spillway have been prepared by Naismith Engineers, Inc. and partial improvements were completed in 1986.

For the purposes of this study, it was decided that the following hydraulic analyses would be completed for the Portland/Gregory Area. The existing 100-year water surfaces and floodplains for Green Lake and the Main BG-00 channel and for the Oakridge Ditch (BG-01) would first be determined. Two alternative improvement scenarios would then be evaluated for Green Lake:

Alternative 1 - Consider necessary channel and structural improvements to Main BG-00 assuming the spillway structure at Green Lake is modified to enhance hydraulic efficiency and the crest is lowered to 15.0 ft-msl.

Alternative 2 - Consider necessary channel and structural improvements to Main BG-00 including a bypass diversion channel around Green Lake assuming the existing modifications to the Green Lake spillway are complete.

Finally, improvements to the Oakridge Ditch (BG-01), BG-02, and BG-03 would be considered along with extension of the BG-00 channel from FM 2986 to CR 72. Preliminary specification of recommended improvements is based on within-bank containment of the 100-year peak discharge assuming the Green Lake watershed would be reduced by the proposed tributary diversion channel to Gum Hollow, the Airport Ditch, and the Doyle Addition Ditch Extension discussed in Section 4.5.

Basin Analyses and Recommended Improvements

Peak discharge estimates for the existing Green Lake watershed were developed using the USGS Region 2 equations and are summarized in Table 4.7-1. Using these discharge values and the HEC-2 computer program, the existing 100-year water surface was computed for the Green Lake (BG-00) channel from the Green Lake spillway structure upstream to FM 2986. Water surface elevations and flow distribution data from the HEC-2 model indicated that only about one-third of the 100-year peak discharge would actually pass through the Green Lake spillway. The remainder of the flow would leave the channel southeast of Gregory and bypass Green Lake to the east along the private road near the Reynolds Aluminum Plant. Review of the water surface profile presented in Appendix A (p. 28 or 29) indicates substantial head losses (in excess of 3.0 feet) through the bridges at SPRR and U.S. 181. The water surface upstream of these structures is sufficiently high that most of the area between U.S. 181 and FM 2986 including most of western and central Gregory would be completely inundated. The computed water surface downstream of FM 2986 indicates that extensive acreage west of FM 2986 would be subject to ponding and very slow moving runoff.

The implementation of interbasin diversion channels including the Gum Hollow tributary diversion, the Airport Ditch, and the

Doyle Addition Ditch extension discussed in the Section 4.5 can substantially reduce the peak discharge rates affecting the Portland/Gregory area. Peak discharge estimates for the 100-year event based on the proposed watershed revisions are presented in Table 4.7-2. Using the 100-year peak discharge values as the design flow rates, channel and structural improvements were developed for the Alternative 1 and Alternative 2 scenarios described above using the HEC-2 computer program. The improvements proposed for each scenario were selected to attain a comparable water surface elevation or level of protection in the Gregory Area. Channel and structural improvements for Alternative 1 are summarized in Tables 4.7-3 and 4.7-4, respectively, and noted on the profile (App. A, p. 28) along with the proposed 100-year water surface. Minimum floor slab elevations of approximately 28.0 ft-msl are recommended for the area southeast of Gregory between U.S. 181 and the private road.

Recommended channel and structural improvements for Alternative 2 are presented in Tables 4.7-5 and 4.7-6, respectively. As described above, Alternative 2 includes a bypass channel around Green Lake along the private road adjacent to Reynolds Aluminum. The 100-year design flow rates for which this diversion channel was designed are presented in Table 4.7-7. A 70' bottom width channel at a slope of 0.06 percent is proposed except within 1600' of the entrance where the proposed slope is increased to 0.3

percent. Structural improvements associated with the diversion channel including a free overfall drop structure and splash pad at the outlet, are summarized in Table 4.7-8. The two profiles for Alternative 2 (Green Lake BG-00 and Diversion Channel) are included in Appendix A (pp. 29-30). Minimum floor slab elevations of approximately 28.0 ft-msl are recommended for the area southeast of Gregory between U.S. 181 and the private road for Alternative 2 also.

In order to provide adequate primary drainage to the area immediately west of FM 2986 and along CR 72, extension of the Main BG-00 channel was proposed. Peak discharge estimates for the 100-year event for both the existing and proposed watershed are presented in Table 4.7-9. A 60' bottom width channel at a slope of 0.06 percent is adequate to convey the design peak discharge from the proposed upstream watershed within the channel banks. Recommended structural improvements at FM 2986 and CR 72 are presented in Table 4.7-10, and a profile including the proposed channel invert and 100-year water surface given is in Appendix A (p. 31).

The hydraulic capacity of the Oakridge Ditch (BG-01) was evaluated using the HEC-2 computer program. Peak discharge estimates for the Oakridge Ditch watershed are presented in Table 4.7-11, and the existing 100-year water surface profile is plotted in Appendix A (p. 32). The existing channel and hydraulic structures are adequate to convey the 100-year peak runoff rate without overbank flooding, and no improvements are recommended at this time.

The small tributary channel of Main BG-00 designated as BG-02 is intended to provide outfall drainage to an area northeast of the Gregory-Portland High School bounded by U.S. 181 and FM 2986 on the east and west, respectively. This area is very flat and poorly drained. Channel improvement to a 10' bottom width ditch with a slope of 0.08 percent in conjunction with the recommended improvements to BG-00 will provide sufficient capacity to handle the estimated 100-year peak discharge (640 cfs) and enhance the future development potential of the area.

The Green Lake tributary channel designated BG-03 outfalls to BG-00 near the Gregory sewage treatment plant. This channel is intended to provide primary outfall drainage to most of the western portion of the City of Gregory. Under existing conditions, however, the tailwater elevation in BG-00 is often too high to facilitate adequate drainage subject to peak flow rates. Peak discharge estimates for the BG-03 watershed are presented in Table 4.7-12. Based on the 100-year design peak discharge, excavation of a 10' bottom width channel at a slope of 0.22 percent and replacement of the existing corrugated metal pipe culverts at CR 78A with a 70' County Road bridge span. These improvements are noted on the profile (App. A, p. 33) along with the proposed water surface profile and right-of-way requirements. In conjunction with the proposed improvements to BG-00, these improvements will assure adequate outfall drainage of western Gregory and minimize damages to existing properties.

Economic Evaluation

An economic evaluation of the two proposed alternative sets of projects and improvements intended to provide flood control benefits to the Portland/Gregory Area has been prepared in the form of benefit-cost analyses. The costs and benefits associated with the proposed improvements to the Green Lake spillway, Main BG-00 channel and extension, and the various tributary channels have been evaluated as a group because improvement of the tributary channels without improvement of the Main BG-00 and Green Lake spillway will not prove effective in the control of flooding during major storm events. Cost estimates for each of the individual project components are presented in Appendix B (pp. 13-15 and 17-22). As flood levels in the Green Lake watershed would be affected by the implementation of the three diversion channels discussed in the Portland Area sub-section, the costs and benefits associated with these improvements have also been included in the economic evaluation of Alternatives 1 and 2. Tables 4.7-13 and 4.7-14 summarize the annual benefits and costs attributable to

Alternative 1 and Alternative 2, respectively.

The benefit-cost ratio for existing conditions for Alternative 1, though less than one, is greater than that noted for Alternative 2. The cost of the diversion channel bypassing Green Lake is the primary component causing the lesser benefit-cost ratio for Alternative 2. The benefits associated with the two alternative improvement scenarios were assumed equal in these analyses because the channel and structural improvements were intentionally selected to achieve approximately the same proposed water surface elevations and provide a similar level of flood protection. The intangible benefits associated with the maintenance of a higher normal pool elevation in Green Lake, however, have not been considered herein. Although the same cost differential is apparent in the benefit-cost ratios for future conditions, both alternatives have benefit-cost ratios in excess of unity with the inclusion of potential development benefits derived from flood control improvements.

TABLE 4.7-1

Green Lake Main BG-00 - Peak Discharge
Proposed Watershed
Gregory/Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Green Lake Structure	0+00	9.06				4240 2160*
Upstream of BG-01 Confluence	32+00	7.61				3730 1700*
2800' Downstream of SPRR	110+00	6.04				3180
SPRR	138+00	5.49				3010
Downstream of BG-02 Confluence	183+00	4.22				2290
FM 2986	210+00	3.61				1990

* Peak discharge values reflect implementation of proposed diversion channel (Alternative 2) bypassing Green Lake structure.

TABLE 4.7-2

Green Lake Main BG-00 - Peak Discharge
Existing Watershed
Gregory/Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Green Lake Structure	0+00	11.44	2580	3500	4230	5010 2160*
Upstream of BG-01 Confluence	32+00	9.99	2350	3190	3850	4550 1700*
2800' Downstream of SPRR	110+00	8.42	2105	2840	3420	4030
SPRR	138+00	7.87	2020	2720	3270	3850
Downstream of BG-02 Confluence	183+00	6.60	1710	2280	2730	3200
FM 2986	210+00	5.99	1620	2160	2580	3030

* Peak discharge values reflect implementation of proposed diversion channel (Alternative 2) bypassing Green Lake structure.

TABLE 4.7-3

Green Lake Main BG-00 - Channel Improvements
Alternative 1
Gregory/Portland Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
BG-01 Confluence to Downstream SPRR	38+00 to 137+00	4240	125	0.035
Downstream SPRR to Upstream U.S. 181	137+00 to 140+50*	3010	80	0.08
Upstream U.S. 181 to Wildcat Road	140+50 to 207+50	3000	80	0.08

* Proposed concrete slope paving and 1:1 sideslopes.

TABLE 4.7-4

Green Lake Main BG-00 - Structural Improvements
Alternative 1
Gregory/Portland Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
Green Lake Structure	0+00	4240		Rect. Notch Weir Drop Struct., 70' Notch Width
SPRR	138+00	3010	0.1	110' Railroad Trestle
U.S. 181 N&S Lanes	139+00	3010	0.1	2 - 110' Highway Bridge Spans

TABLE 4.7-5

Green Lake Main BG-00 - Channel Improvements
Alternative 2
Gregory/Portland Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
BG-01 Confluence to Diversion Channel	38+00 to 110+00	2160	150	0.035
Diversion Channel to Downstream SPRR	110+00 to 137+00	3180	125	0.035
Downstream SPRR to Upstream U.S. 181	137+00 to 140+50*	3010	70	0.08
Upstream U.S. 181 to Wildcat Road	140+50 to 207+50	3010	70	0.08

* Proposed concrete slope paving and 1:1 sideslopes.

TABLE 4.7-6

Green Lake Main BG-00 - Structural Improvements
Alternative 2
Gregory/Portland Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
Green Lake Structure	0+00	2160		Per Existing NEI Plans
SPRR	138+00	3010	0.1	100' Railroad Trestle
U.S. 181 N&S Lanes	139+00	3010	0.1	2 - 100' Highway Bridge Spans

TABLE 4.7-7

Proposed Green Lake Diversion Channel - Peak Discharge
Alternative 2
Gregory/Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Corpus Christi Bay	0+00	0.60*				2310
3500' Downstream of Green Lake BG-00	65+00	0.10*				2090
Green Lake BG-00	116+00					1990

* Values represent drainage area of diversion channel only and do not include Green Lake watershed.

TABLE 4.7-8

Proposed Green Lake Diversion Channel - Structural Improvements
Alternative 2
Gregory/Portland Area

Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
2+00 to 3+00	2310		Free Overfall Drop Structure
3+00 to 3+40	2310		Concrete Lined Channel Transition 70' BW to 25' BW
32+00	2310	0.1	130' County Road Bridge Span
116+00 to 116+50	1990		Conc. Lined Diversion Channel Section

TABLE 4.7-9

Proposed Green Lake Channel Extension - Peak Discharge
Gregory/Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
FM 2986	0+00					
Existing Watershed		4.17				2250
Proposed Watershed		1.77				1250
CR 72	68+00					
Existing Watershed		2.22				1450
Proposed Watershed		0.26				370

TABLE 4.7-10

Proposed Green Lake Channel Extension - Structural Improvements
Gregory/Portland Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
FM 2986	0+00	1250	1.1	3-8'X8' Conc. Boxes
CR 72	68+00	370	0.7	2-6'X6' Conc. Boxes

TABLE 4.7-11

Oakridge Ditch BG-01 - Peak Discharge
Gregory/Portland Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
Pipeline Crossing	16+50	0.73		370	420	470
SPRR	48+50	0.44		260	300	330

TABLE 4.7-12

Green Lake Tributary BG-03 - Peak Discharge
Portland/Gregory Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
CR 78A	1+50	0.52		530	620	710
U.S. 181	18+00	0.15		180	200	230

TABLE 4.7-13

Economic Evaluation
Alternative 1 Improvements
Gregory/Portland Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	340,100.00
Emergency Cost Reduction	17,000.00
Total Benefits - Existing Conditions	$ 357,100.00
Potential Development Benefits	434,500.00
Total Benefits - Future Conditions	$ 791,600.00
Costs:	
Proposed Improvements	481,400.00
Operations and Maintenance	57,700.00
Total Costs	$ 539,100.00

Benefit-Cost Ratios:

Existing Conditions = 0.7
Future Conditions = 1.5

TABLE 4.7-14

Economic Evaluation
Alternative 2 Improvements
Gregory/Portland Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	340,100.00
Emergency Cost Reduction	17,000.00
Total Benefits - Existing Conditions	$ 357,100.00
Potential Development Benefits	434,500.00
Total Benefits - Future Conditions	$ 791,600.00
Costs:	
Proposed Improvements	609,500.00
Operations and Maintenance	73,000.00
Total Costs	$ 682,500.00

Benefit-Cost Ratios:

Existing Conditions = 0.5
Future Conditions = 1.2

4.8 Ingleside Area

Description of Flood Control Considerations

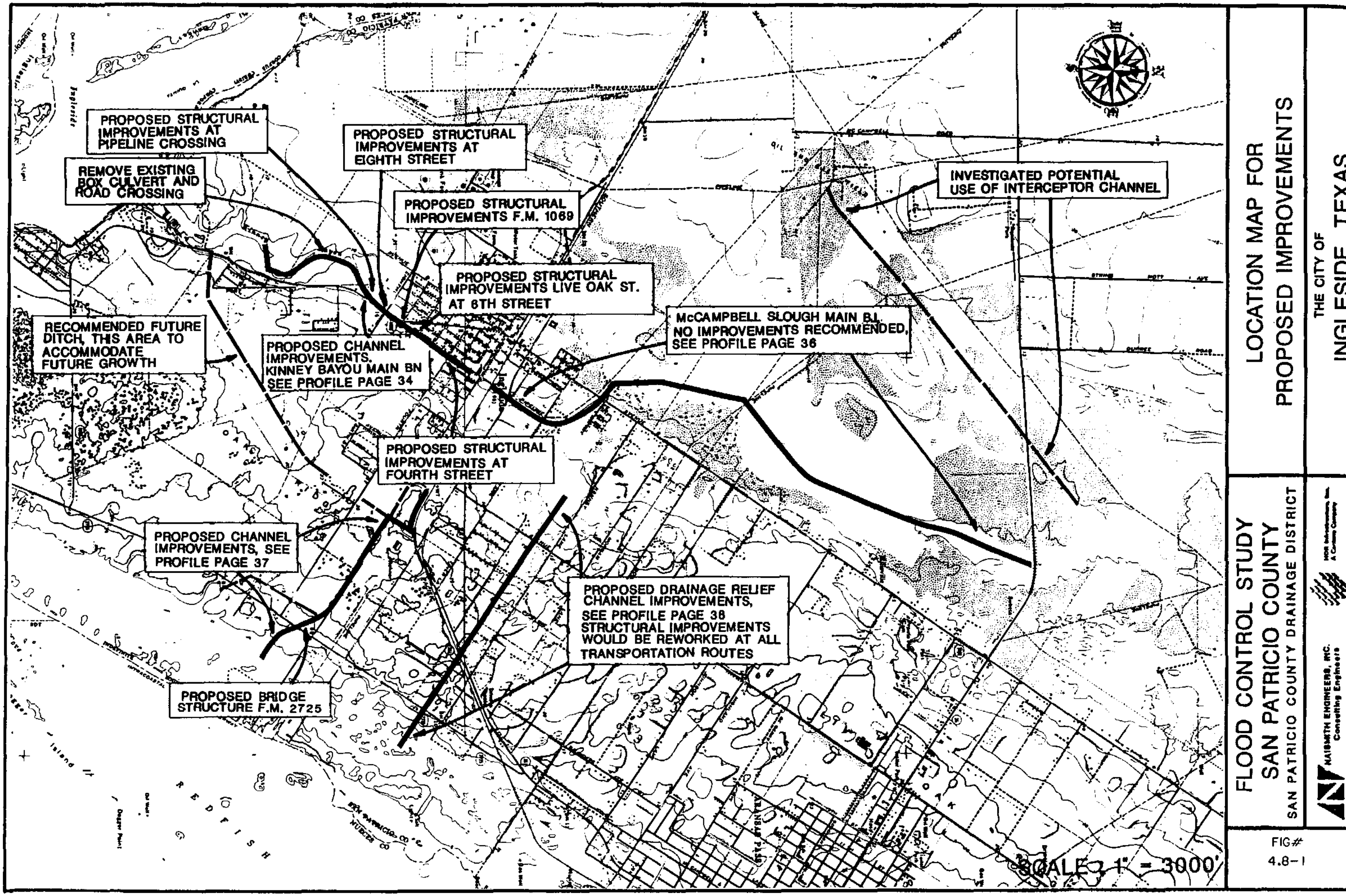
The City of Ingleside is located in the southeastern portion of San Patricio County at the crossroads of State Highway 361 and FM 1069. Drainage of Ingleside is generally divided by Highway 361 and Southern Pacific Railroad (SPRR), which cross the city in an east-west direction. Primary outfall drainage for the the majority of the City is provided by two major outfall drainageways. These are Kinney Bayou which drains the southern portions of the City and McCampbell Slough which drains the northern areas.

The City of Ingleside is located west of Live Oak Ridge which naturally protects the City from tidal surges associated with hurricanes. Elevations in the City range from 10 to 20 ft-msl in the southern part of the City and from 8 to 20 ft-msl in the northern sections. Primary flooding problems in Ingleside are a result of overflows from Kinney Bayou and McCampbell Slough. Storm surges may also contribute to flooding particularly along Kinney Bayou. For the purposes and scope of this study, however, the major outfalls were analyzed on the basis of peak runoff rates neglecting tidal effects, as high tides do not always coincide with peak discharge for inland drainageways.

Basin Analyses and Recommended Improvements

In order to address the magnitude of flooding problems within the developed areas of Ingleside, HEC-2 models were developed for both Kinney Bayou and McCampbell Slough. The model for Kinney Bayou extends from a point approximately 6300' downstream of Eighth St. to about 300' upstream of Eighth St. McCampbell Slough was modelled from the State Highway 35 bridge upstream to Tiner Lane. These channel reaches are noted in Figure 4.8-1. Peak discharges for these watersheds were computed using the USGS Region 2 equations and are summarized in Tables 4.8-1 and 4.8-2. The computed existing 100-year water surface profiles for Kinney Bayou and McCampbell Slough are presented in Appendix A, (pp. 34-36).

Review of the existing 100-year water surface profile for Kinney Bayou shows a water surface elevation in excess of 13.0 ft-msl at Eighth St. Extensive flooding of residential areas upstream of Eighth St. and along Avenue G will occur subject to this tailwater elevation. Several improvement options were modelled for Kinney Bayou beginning simply with the removal of an 8' X 10' concrete box culvert located about 3000' downstream of Eighth St. Removal of this structure results in a 100-year flood level reduction of about 2.0 feet immediately downstream of Eighth St. This reduction is insufficient, however, to prevent flooding along Avenue G.


PROPOSED STRUCTURAL IMPROVEMENTS AT PIPELINE CROSSING
REMOVE EXISTING BOX CULVERT AND ROAD CROSSING
PROPOSED STRUCTURAL IMPROVEMENTS AT EIGHTH STREET
PROPOSED STRUCTURAL IMPROVEMENTS F.M. 1069
PROPOSED STRUCTURAL IMPROVEMENTS LIVE OAK ST. AT 6TH STREET
INVESTIGATED POTENTIAL USE OF INTERCEPTOR CHANNEL
McCAMPBELL SLOUGH MAIN BJ. NO IMPROVEMENTS RECOMMENDED, SEE PROFILE PAGE 36
RECOMMENDED FUTURE DITCH, THIS AREA TO ACCOMMODATE FUTURE GROWTH
PROPOSED CHANNEL IMPROVEMENTS, KINNEY BAYOU MAIN BN SEE PROFILE PAGE 34
PROPOSED STRUCTURAL IMPROVEMENTS AT FOURTH STREET
PROPOSED CHANNEL IMPROVEMENTS, SEE PROFILE PAGE 37
PROPOSED DRAINAGE RELIEF CHANNEL IMPROVEMENTS, SEE PROFILE PAGE 38 STRUCTURAL IMPROVEMENTS WOULD BE REWORKED AT ALL TRANSPORTATION ROUTES
PROPOSED BRIDGE STRUCTURE F.M. 2725
REDFISH
SCALE : 1" = 3000'
LOCATION MAP FOR PROPOSED IMPROVEMENTS
THE CITY OF
INGLESIDE, TEXAS
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT
NAISMITH ENGINEERS, INC.
Consulting Engineers
FIG#
4.8-1

Recommended channel and structural improvements to adequately decrease flood levels along Kinney Bayou are summarized in Tables 4.8-3 and 4.8-4, respectively. Implementation of these improvements will result in a computed water surface elevation of approximately 7.6 ft-msl at Eighth St. as observed in the proposed 100-year water surface profile presented in Appendix A. Reductions in the 100-year floodplain due to the implementation of the recommended improvements are indicated on the aerial mapping of San Patricio County prepared as a portion of this Flood Control Study.

The McCampbell Slough watershed includes expansive low-lying areas which are periodically subject to inundation from overflow of the shallow and sometimes undefined channel of McCampbell Slough. An area containing scattered residential and multi-family development bounded by Mooney Lane, First St., Avenue B, and Avenue A drains naturally to McCampbell Slough. Natural ground elevations in this area range from 8 to 20 ft-msl. Review of the existing 100-year water surface profile for McCampbell Slough indicates water surface elevations of 11.0 to 12.0 ft-msl between Avenue B and Tiner Lane. Hence, portions of this area are subject to backwater inundation from McCampbell Slough up to 4 feet in depth.

HEC-2 modelling of various channel improvements to McCampbell Slough indicated that very little could be done to reduce the

100-year flood elevations downstream of Avenue B. An interceptor channel west of and paralleling McCampbell Slough was investigated as a means of collecting runoff from much of the watershed and conveying it downstream of Highway 35. Extensive channel improvements including bottom widths up to 400 feet and extending from Highway 35 to Avenue B were also considered. Neither of these altenatives resulted in a significant reduction in flood elevations and recommendation of larger channel dimensions would prove cost prohibitive.

As a result of these analyses, it is recommended that the City of Ingleside recognize this area as being subject to flooding and stipulate that future development conform to minimum finished floor elevations at least one foot above the computed 100-year water surface elevation. Future development in this area need not be discouraged; however, all development should respect designated flood elevations. Internal or initial drainage systems should be provided to handle runoff from rainfall events up the 25-year event and the performance of these systems should be evaluated subject to the 100-year event. Future development in the area between Tiner Lane and Kinney Lane can occur where excavated material is utilized to elevate development sites above the floodplain. Excavated areas can serve as recreational lakes or be incorporated into small-scale detention storage systems. Caution should be exercised, however, as excessive fill placement can encroach on existing drainageways and increase flood levels.

One item of singular interest noted in the analysis of the McCampbell Slough watershed is that the Federal Emergency Management Agency (FEMA) does not include the area northeast of Ingleside in a designated flood hazard zone (Ref. 5). As the results of this study indicate widespread flooding of this area, it is recommended that the City of Ingleside request a restudy of the area to establish flood hazard zones.

A diversion channel extending from the SPRR near the old refinery at Sun Ray Road to Redfish Bay (Fig. 4.8-1) was considered as a means of reducing runoff to McCampbell Slough upstream of Avenue B. This channel could provide primary drainage relief to the Sun Ray Road area and outfall drainage for present and future development along Live Oak Ridge. Recommended channel improvements consist of a 10' bottom width ditch with a slope of 0.30 percent. The proposed channel invert, 100-year design water surface, and right-of-way requirements are included in the Flood Profile (App. A, p. 37).

A diversion channel extending from near Kinney Lane and Avenue A to Redfish Bay (Fig. 4.8-1) was also investigated to help reduce flood elevations for McCampbell Slough. This channel would also provide outfall drainage to the area north and east of Avenue A. Presently, no major outfall crossing Live Oak Ridge to Redfish Bay exists in this area. Preliminary channel size was based on

100-year peak runoff rates ranging from 1290 to 1510 cfs. Recommended channel dimensions and structural improvements are summarized in Tables 4.8-5 and 4.8-6, respectively. A profile indicating the existing natural ground elevations and proposed channel invert and 100-year water surface are included in Appendix A (p. 38). As the proposed channel is fairly deep, structural and slope stability should be carefully investigated. In addition, construction of this diversion channel would constitute a breach in the natural levee (Live Oak Ridge) protecting Ingleside from tidal surges in Redfish Bay which can reach 14.0 ft-msl (Ref. 25).

Economic Evaluation

A summary of the economic evaluation of the recommended improvements to Kinney Bayou is presented in Table 4.8-7. As implementation of proposed improvements will remove some developed areas from the floodplain, flood damage reduction benefits are relatively high and a benefit-cost ratio of 3.3 is computed for existing conditions. Detailed cost estimates for the Kinney Bayou improvements as well as the two diversion channels are included in Appendix B (pp. 23-25).

TABLE 4.8-1

Kinney Bayou - Peak Discharge
Ingleside Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
7200' Downstream of 8th St.		2.10	910	1200	1420	1660
2300' Downstream of 8th St.	40+00	1.39	700	910	1070	1240
600' Downstream of 8th St.	57+00	1.25	610	790	930	1070
Ave. G Above 8th St.	66+00	0.71	400	510	590	670
Ave. G Above Live Oak St.	70+00	0.59	370	460	540	610

TABLE 4.8-2

McCampbell Slough - Peak Discharge
Ingleside Area

Location	Profile Station	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs) 10	25	50	100
16700' Downstream of FM 1069	15+00	17.10	3240	4430	5370	6380
8500' Downstream of FM 1069	97+00	12.76	2740	3730	4510	5350
2300' Downstream of FM 1069	159+00	4.41	1480	1980	2370	2780
FM 1069	182+00	2.46	1050	1390	1650	1930
Tiner Lane	200+00	1.08	700	920	1090	1270
Hwy 361	213+00	0.71	540	700	830	960

TABLE 4.8-3

Kinney Bayou - Channel Improvements
Ingleside Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
6300' Downstream of 8th St. to 8th St.	0+00 to 63+00	1660	100	Exist.
Liveoak St. to 6th St.	69+50 to 74+50*	470	10	0.08
6th St. to 2nd St.	74+50 to 96+50*	240	8	0.08

* Concrete lined channel with 1:1 sideslopes.

TABLE 4.8-4

Kinney Bayou - Structural Improvements
Ingleside Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
Private Bridge	32+50	1240	0.0	Remove Existing Box Culvert & Roadway
Pipeline Crossing	58+00	670	0.1	Add Culvert Capacity
Eighth St.	63+00	670	0.1	Add 8'X5' Conc. Box
FM 1069 and and Ave. G	67+00	520	0.5	Add 8'X5' Conc. Box
Live Oak Street	69+50	470	0.2	Add Box Culvert and Lower Existing Box
Fourth St.	87+00	240	0.2	Add Box Culvert and Lower Existing Box

TABLE 4.8-5

Proposed Diversion Channel - Channel Improvements
McCampbell Slough to Redfish Bay
Ingleside Area

Location	Profile Station	Design Discharge (cfs)	Bottom Width (ft)	Slope (%)
Redfish Bay to West of Hwy 361	0+00 to 38+00	1510	75	0.05
West of Hwy 361 to West of FM 1069	38+00 to 99+00	1290	60	0.05

TABLE 4.8-6

Proposed Diversion Channel - Structural Improvements
McCampbell Slough to Redfish Bay
Ingleside Area

Location	Profile Station	Design Discharge (cfs)	Allowable Head Loss (ft)	Example Structure
FM 2275	10+00	1510	0.1	120' Bridge Span
SPRR	32+00	1470	0.1	125' Railroad Trestle
Hwy 361	33+50	1470	0.1	125' Bridge Span
Ave. A	81+00	1290	0.1	90' Bridge Span

TABLE 4.8-7

Economic Evaluation
Kinney Bayou Improvements
Ingleside Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	259,300.00
Emergency Cost Reduction	13,000.00
Total Benefits - Existing Conditions	$ 272,300.00
Potential Development Benefits	Unquantified
Total Benefits - Future Conditions	$ 272,300.00
Costs:	
Proposed Improvements	74,700.00
Operations and Maintenance	9,000.00
Total Costs	$ 83,700.00

Benefit-Cost Ratio:

Existing Conditions = 3.3

4.9 Aransas Pass Area

Description of Flood Control Considerations

The City of Aransas Pass is located in the eastern most portion of San Patricio County at the crossroads of State Highways 35 and 361. The City borders on Redfish Bay to the east and is protected from tidal inundation and hurricane surge by a 2.5-mile long earthen seawall constructed in 1927 (Ref. 3). Primary outfall drainage for the City is of two distinct types. Outfall ditches transport stormwater runoff from the areas immediately north and south of the City directly to Redfish Bay, and stormwater pump stations serve the central portion of the City protected by the seawall. A detailed drainage basin analysis (Ref. 18) was prepared in 1983 by Naismith Engineers, Inc. for the City and the San Patricio County Drainage District.

Flooding problems affecting the City of Aransas Pass are a result of: 1) Development in low-lying areas throughout the City; 2) Lack of adequate internal and primary outfall drainage facilities to effectively remove storm runoff; and 3) Inadequate pumping capacity and collector channels serving the existing stormwater pump stations located inside the seawall. Since the need for adequate internal drainage relief facilities as well as identification of low-lying areas have been addressed in past studies (Ref. 13 and 18), no attempt was made in this study to locate deficiencies in the the existing internal drainage systems.

Basin Analyses and Recommended Improvements

In an effort to identify the magnitude of flooding problems within the City of Aransas Pass, an analysis of flooding within and around the existing seawall was performed. Since the seawall area includes the major portion of the populated and commercially developed areas of the City, it was felt that analysis of this area was of primary concern. Existing stormwater pump stations, along with collector channels and sumps, were analyzed.

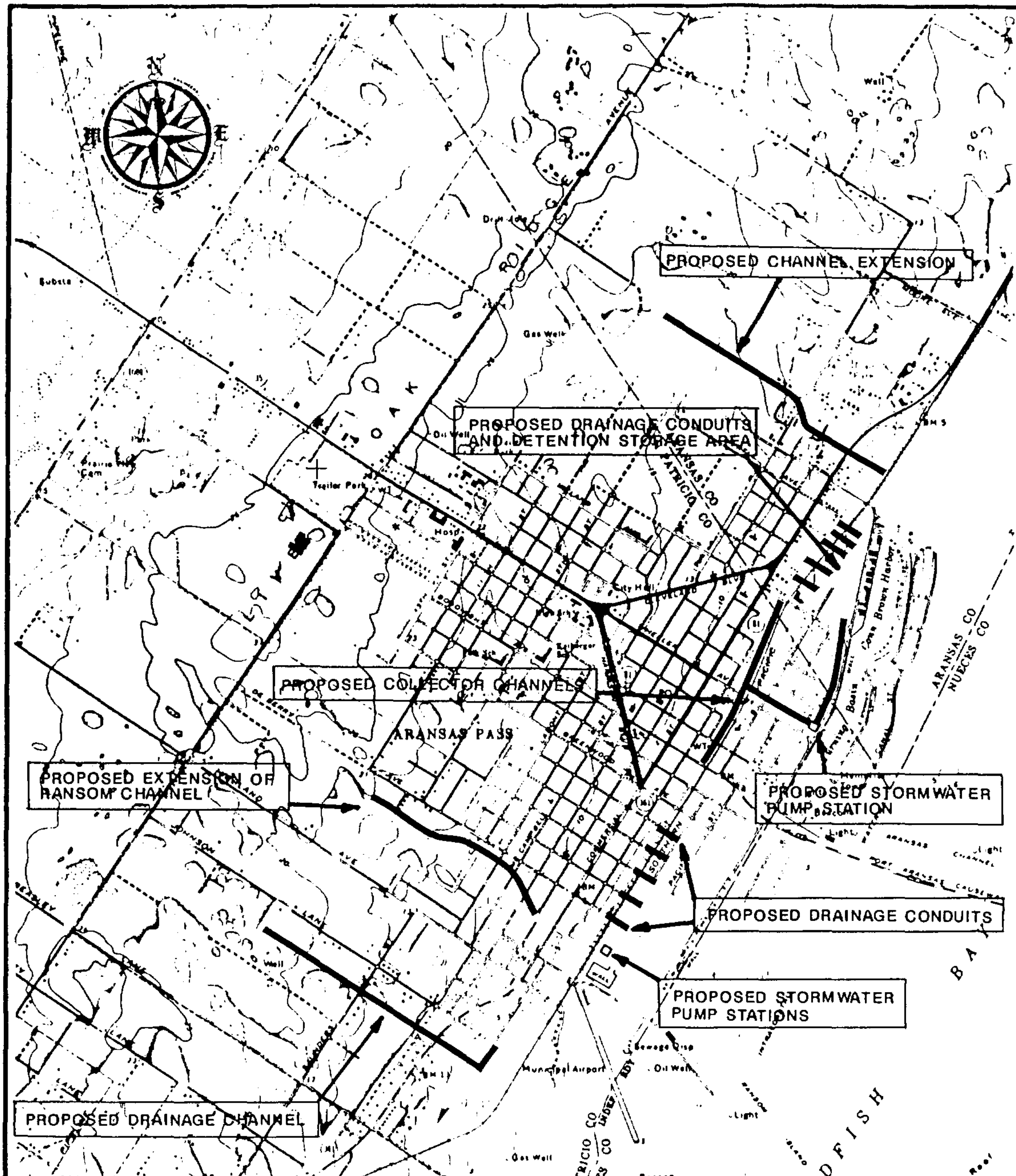
The area protected by the seawall may be divided into three primary drainage basins. Storm runoff from these basins is pumped beyond the seawall by the Euclid St., Goodnight St., and Wheeler Ave. Pump Stations. Two of these three areas also have the capability to drain storm runoff through the seawall by gravity via storm sewer culverts equipped with flap gates or control valves. The area draining to the Euclid St. Pump Station, however, no longer has any gravity drainage due to recent development east of the SPRR tracks. All runoff to this area, whether it is runoff from a 2-year or 100-year event, must be pumped beyond the seawall by the Euclid St. Pump Station.

Each of the existing pump stations has a rated capacity of between 30 and 50 cfs. Each station is drastically undersized to evacuate runoff from the 100-year storm for the areas they serve in a timely fashion. Rather than attempt to increase the capacity of the existing pump stations, an analysis was performed to provide new pumping facilities at locations that would optimize the number and capacity of stations. Natural topography divided the area behind the seawall into two nearly equally sized drainage areas. These are referred to herein as the North Basin (including Areas IV, V, and VI in Ref. 18) and the South Basin (including Areas III and IIIA in Ref. 18). The drainage areas and estimated runoff peaks for these basins are presented in Table 4.9-1.

Alternatives considered for the North Basin included installation of a stormwater pump station and collector channels capable of removing runoff at a rate equal to the peak discharge for the basin and of developing a detention storage or sump area to reduce required pump capacity. The proposed locations of the pump station and collector channels are indicated in Fig. 4.9-1. Methodologies described in Section 3.1 were applied to estimate the reduction in peak discharge which can be attributed to detention storage development. Incorporation of the detention storage area reduced the peak runoff rate to the proposed pump station from 1410 to 1060 cfs. Recommended improvements for the North Basin include:

1) Development of a 29 acre detention storage area in the region bounded by Matlock Ave., SPRR, and the seawall.

2) Installation of drainage conduits beneath the SPRR leading



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	LOCATION MAP FOR PROPOSED IMPROVEMENTS	FIG.# 4.9-1
NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF ARANSAS PASS	DATE 7/87

directly to the detention storage area.

3) Construction of a new stormwater pump station with a rated capacity of approximately 1060 cfs located near Wheeler Ave. adjacent to the seawall.

Although the capital costs for installation of a large pump station and for a smaller pump station and detention storage area were almost identical, the detention storage option was felt to have certain hydraulic, economic, and aesthetic advantages. These advantages include increased hydraulic efficiency of the pump station, lower operation and maintenance costs, and the potential for park development of the detention storage area.

The primary alternative considered for the South Basin involved the use of the canals located in the "Pelican Cove" development as a sump and detention storage area. As development of this area has blocked the historical drainage paths under the SPRR and blocked access to the former ponded storage area between SPRR and the seawall, increased ponding during major storm events will occur west of the SPRR. Utilization of the available storage capacity of these canals up to elevation 5.0 ft-msl provides a reduction in required pump station capacity from 1120 cfs to 750 cfs for a 100-year design runoff rate. This, in turn, results in substantial cost savings for the proposed stormwater pump station. Recommended improvements for the South Basin include:

1) Installation of drainage conduits beneath the SPRR and outfalling to the canals in the Pelican Cove development.

2) Construction of a new stormwater pump station to be located near the pass through the seawall connecting the Pelican Cove canals to Redfish Bay. The proposed rated capacity of the pump station is 750 cfs, and water surface elevations in the canals will not exceed 5.0 ft-msl for the 100-year design storm event.

Maintenance of the existing Euclid St. Pump Station should be considered as a means of preserving water quality in the Pelican Cove canals during minor storm events. Installation of the recommended conduits could be achieved in such a way as to intercept the initial runoff which typically carries the accumulated debris and pollutants from the streets and storm sewer systems and direct it to the existing Euclid Street Pump Station.

Diversion of runoff from areas above 16.0 ft-msl (top of seawall) was not evaluated in detail in the performance of this study because: 1) Diversion would not solve all flooding problems in downtown Aransas Pass; 2) Increased stormwater pump station capacity would still be required; 3) Major improvements in the internal drainage system would be necessary to redirect stormwater outside the seawall; and 4) Major improvements to the drainageways outside of the seawall would be required to accommodate the increased flow rates.

Three major outfall channels outside of the seawall were, however, evaluated to provide an indication of the improvements necessary to upgrade them. The alignment of these channels is noted in Fig. 4.9-1 and design discharge values are presented in Table 4.9-1. One outfall channel is located south of Johnson Lane and extends from its existing outfall at Redfish Bay to approximately 9th St. Recommended improvement of this channel includes upgrading approximately 6,400 feet of channel to a 25' bottom width, eight street crossings, and one crossing of the SPRR. The second channel improvement investigated was the extension of Ransom Channel from near McCampbell St. to near 13th St. This improvement includes the upgrading of approximately 4,200 feet of channel to a 30' bottom width, nine street crossings, and one crossing of the SPRR. The final major outfall analyzed is located in the northern portion of the City in Aransas County and extends from its outfall at Redfish Bay to about 10th St. This improvement includes upgrading of approximately 5,360 feet of channel to 30' bottom width, three street crossings, and one crossing of the SPRR.

Economic Evaluation

Benefit-cost analyses were performed to evaluate recommended improvements for the North and South Basins including pump station and detention storage implementation on an economic basis. Detailed cost estimates for these recommended improvements are included in Appendix B (pp. 26-27) and the results of the benefit-cost analyses are summarized for the North and South Basins in Tables 4.9-2 and 4.9-3, respectively. Economic evaluation of the proposed improvements affecting the South Basin for existing conditions indicates that implementation costs would be approximately balanced by flood damage and emergency cost reduction benefits. The computed benefit-cost ratio of 0.9 for the North Basin improvements indicates that the costs of implementation slightly exceed estimated benefits. If detention storage could be developed in this area in conjunction with a development similar to Pelican Cove, benefits could increase substantially.

TABLE 4.9-1

Peak Discharge
Aransas Pass Area

Location	Drainage Area (sq. mi.)	Peak Discharge (cfs) per Return Period (yrs)	
		25	100
North Basin	1.76	1130	1410
South Basin	1.30	900	1120
Ditch South of Johnson Ave.	0.54	430	540
Ransom Channel Extension	0.72	550	690
Ditch North of Seawall			
Above SPRR	0.51	410	510
Above McCampbell Road	0.39	340	430

TABLE 4.9-2

Economic Evaluation
North Basin Improvements
Detention Storage, Channel and Structural Improvements, and
Stormwater Pump Station
Aransas Pass Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	183,400.00
Emergency Cost Reduction	9,200.00
Total Benefits - Existing Conditions	$ 192,600.00
Potential Development Benefits	Unquantified
Total Benefits - Future Conditions	$ 192,600.00
Costs:	
Proposed Improvements	196,500.00
Operations and Maintenance	23,500.00
Total Costs	$ 220,000.00

Benefit-Cost Ratio:

Existing Conditions = 0.9

TABLE 4.9-3

Economic Evaluation
South Basin
Structural Improvements and Stormwater Pump Station
Aransas Pass Area

	Average Annual Dollars
Benefits:	
Flood Damage Reduction	174,200.00
Emergency Cost Reduction	8,700.00
Total Benefits - Existing Conditions	$ 182,900.00
Potential Development Benefits	Unquantified
Total Benefits - Future Conditions	$ 182,900.00
Costs:	
Proposed Improvements	155,100.00
Operations and Maintenance	18,600.00
Total Costs	$ 173,700.00

Benefit-Cost Ratio:

Existing Conditions = 1.1

4.10 Priority Ranking

The benefit-cost analyses performed for the major alternative flood control projects considered in the Flood Control Study for San Patricio County are intended to provide a means of prioritizing improvements on an area by area and on a county-wide basis. Therefore, consistent unit costs and benefits were applied in the economic evaluation of flood control improvements affecting each incorporated community and the unincorporated areas of the County. The computed benefit-cost ratios presented in Sections 4.1 through 4.9 of this report are not intended to ultimately determine the feasibility of specific projects, as feasibility of a given project is not always simply a function of economics, but may also involve social, political, and environmental concerns which have not been directly addressed in the performance of this study.

Tables presenting the priority ranking of the various alternative flood control projects considered are presented in the following pages. The rank assigned to each project is determined directly from the computed benefit-cost ratio for that project; therefore, the project with the highest benefit-cost ratio is assigned the most favorable ranking (Rank = 1). A more favorable rank was assigned to the project with greater associated benefits if two or more projects had the same computed benefit-cost ratio. The flood control projects are ranked subject to existing development conditions on an area by area basis in Table 4.10-1 and on a county-wide basis in Table 4.10-2. Table 4.10-3 presents the ranking of alternative projects on a county-wide basis for future development conditions.

TABLE 4.10-1

Priority Ranking of Alternative Improvements by Area
Existing Development Conditions

Area	Drainageway(s)	Type of Improvements	Ranking City	Ranking County	Benefit/ Cost Ratio
Mathis	Sixmile Crk. & Extension	Channel & Structures	1	4	1.1
Mathis	BU-01 & BU-02	Channel & Structures	2	11	0.8
Sinton	Chiltipin Crk. & South Ditch	Channel & Structures & Rectification	1	5	1.0
Sinton	Peters Swale	Channel & Structures & Interbasin Diversion	3	6	1.0
Sinton	Chiltipin Crk.	Channel & Structures & Rectification	2	9	0.8
Sinton	Chiltipin Crk.	Channel & Structures & Levee	4	10	0.8
Sinton	Chiltipin Crk.	Channel & Structures	5	12	0.7
Sinton	Chiltipin Crk.	Detention Storage	6	17	0.0
Odem	Peters Swale	Channel & Structures	1	8	0.9
Taft	Main AJ	Channel & Structures	1	2	1.6
Taft	Main AN & AN-02	Channel & Structures	2	15	0.4
Portland/ Gregory	Alternative 1	Channel & Structures & Interbasin Diversion	1	13	0.7
Portland/ Gregory	Alternative 2	Channel & Structures & Interbasin Diversion	2	14	0.5
Portland/ Gregory	Gum Hollow	Channel & Structures	3	16	0.1
Ingleside	Kinney Bayou	Channel & Structures	1	1	3.3
Aransas Pass	South Basin	Stormwater Pump Station & Detention Storage	1	3	1.1
Aransas Pass	North Basin	Stormwater Pump Station & Detention Storage	2	7	0.9

TABLE 4.10-2

Priority Ranking of Alternative Projects
Existing Development Conditions

Area	Drainageway(s)	Type of Improvements	Ranking County	Benefit/ Cost Ratio
Ingleside	Kinney Bayou	Channel & Structures	1	3.3
Taft	Main AJ	Channel & Structures	2	1.6
Aransas Pass	South Basin	Stormwater Pump Station & Detention Storage	3	1.1
Mathis	Sixmile Crk. & Extension	Channel & Structures	4	1.1
Sinton	Chiltipin Crk. & South Ditch	Channel & Structures & Rectification	5	1.0
Sinton	Peters Swale	Channel & Structures & Interbasin Diversion	6	1.0
Aransas Pass	North Basin	Stormwater Pump Station & Detention Storage	7	0.9
Odem	Peters Swale	Channel & Structures	8	0.9
Sinton	Chiltipin Crk.	Channel & Structures & Rectification	9	0.8
Sinton	Chiltipin Crk.	Channel & Structures & Levee	10	0.8
Mathis	BU-01 & BU-02	Channel & Structures	11	0.8
Sinton	Chiltipin Crk.	Channel & Structures	12	0.7
Portland/ Gregory	Alternative 1	Channel & Structures & Interbasin Diversion	13	0.7
Portland/ Gregory	Alternative 2	Channel & Structures & Interbasin Diversion	14	0.5
Taft	Main AN & AN-02	Channel & Structures	15	0.4
Portland/ Gregory	Gum Hollow	Channel & Structures	16	0.1
Sinton	Chiltipin Crk.	Detention Storage	17	0.0

TABLE 4.10-3

Priority Ranking of Alternative Projects
Future Development Conditions

Area	Drainageway(s)	Type of Improvements	Ranking County	Benefit/ Cost Ratio
Ingleside	Kinney Bayou	Channel & Structures	1	3.3
Mathis	BU-01 & BU-02	Channel & Structures	2	2.4
Taft	Main AJ	Channel & Structures	3	1.8
Mathis	Sixmile Crk. & Extension	Channel & Structures	4	1.6
Portland/ Gregory	Alternative 1	Channel & Structures & Interbasin Diversion	5	1.5
Sinton	Chiltipin Crk. & South Ditch	Channel & Structures & Rectification	6	1.2
Portland/ Gregory	Alternative 2	Channel & Structures & Interbasin Diversion	7	1.2
Aransas Pass	South Basin	Stormwater Pump Station & Detention Storage	8	1.1
Sinton	Chiltipin Crk.	Channel & Structures & Rectification	9	1.0
Sinton	Peters Swale	Channel & Structures & Interbasin Diversion	10	1.0
Odem	Peters Swale	Channel & Structures	11	1.0
Sinton	Chiltipin Crk.	Channel & Structures	12	0.9
Aransas Pass	North Basin	Stormwater Pump Station & Detention Storage	13	0.9
Sinton	Chiltipin Crk.	Channel & Structures & Levee	14	0.8
Taft	Main AN & AN-02	Channel & Structures	15	0.7
Portland/ Gregory	Gum Hollow	Channel & Structures	16	0.1
Sinton	Chiltipin Crk.	Detention Storage	17	0.1

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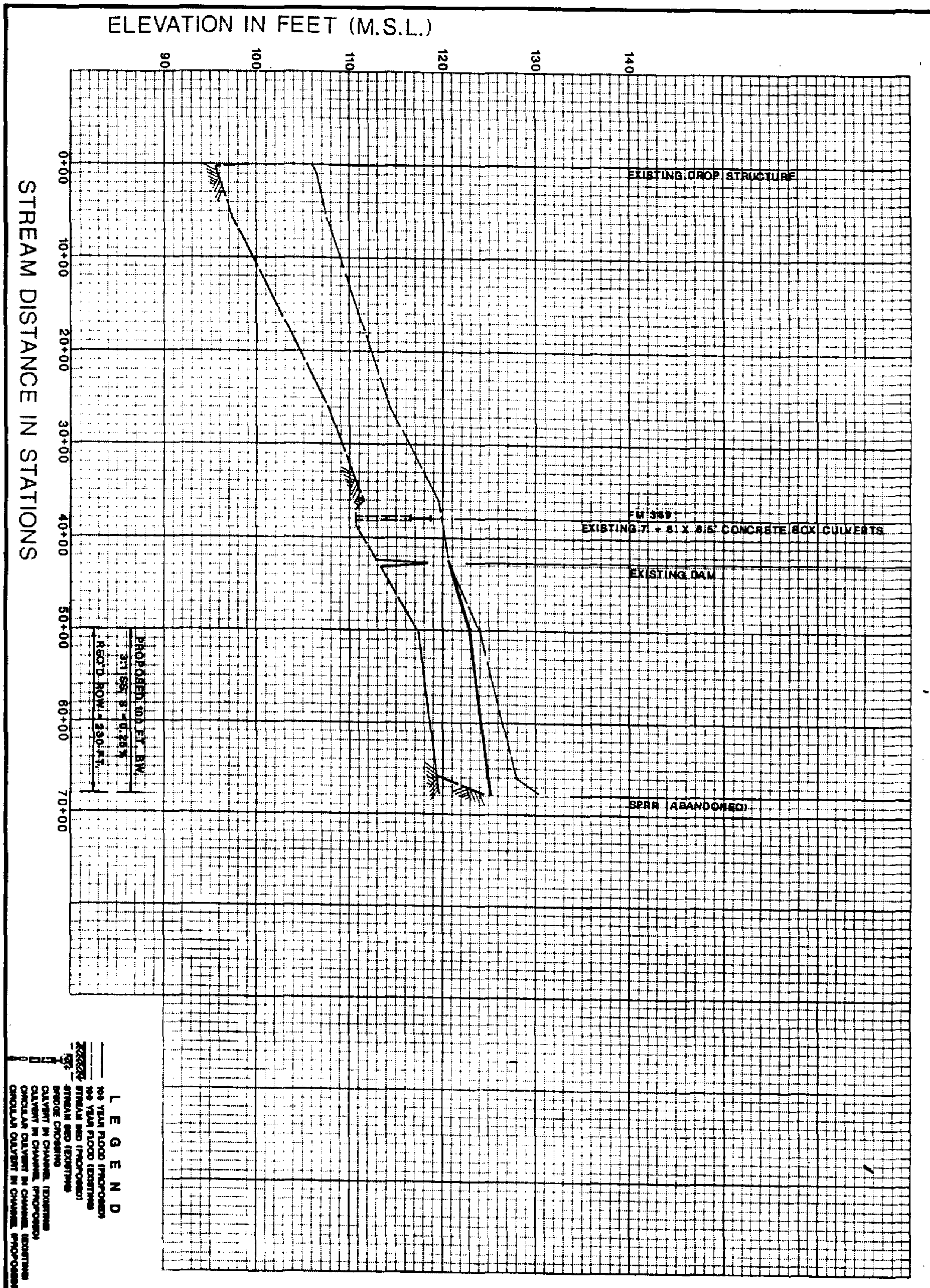
APPENDIX A

WATER SURFACE PROFILES

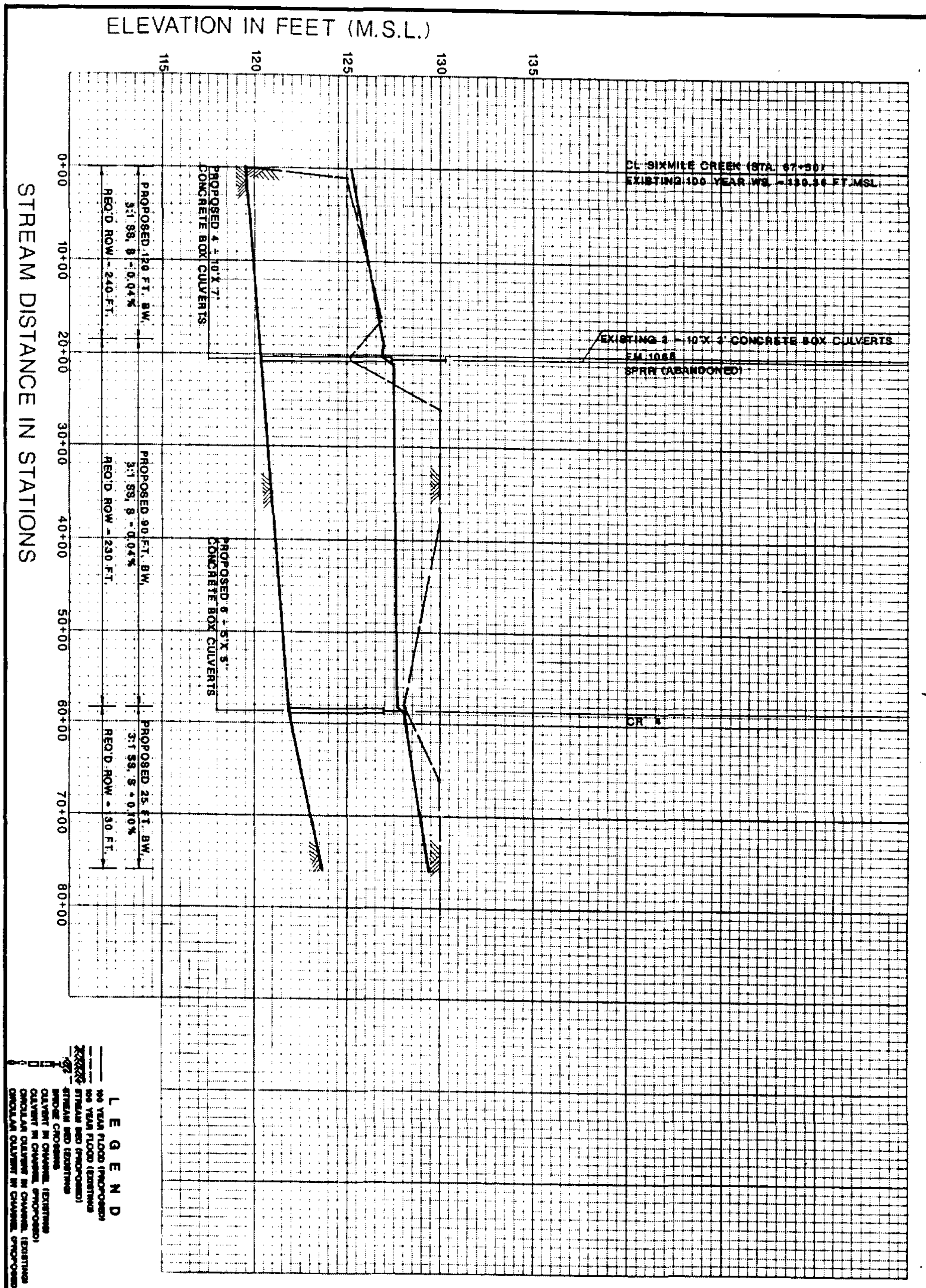


PAGE 1

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FLOOD PROFILE
SIXMILE CREEK (MAIN BU - 00)
CITY OF MATHIS



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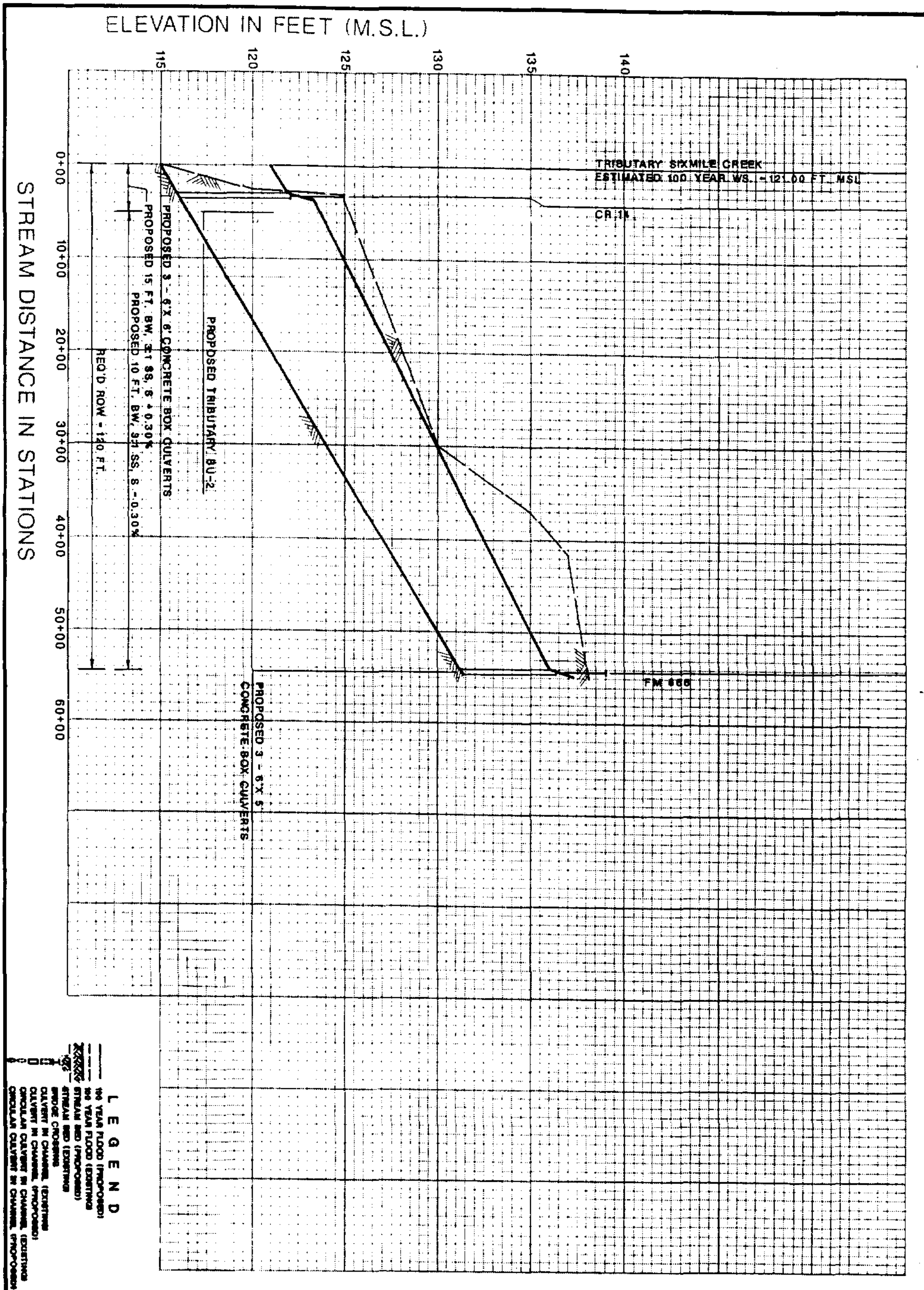
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FLOOD PROFILE
SIXMILE CREEK
EXTENSION (MAIN BU - 00)

CITY OF MATHIS



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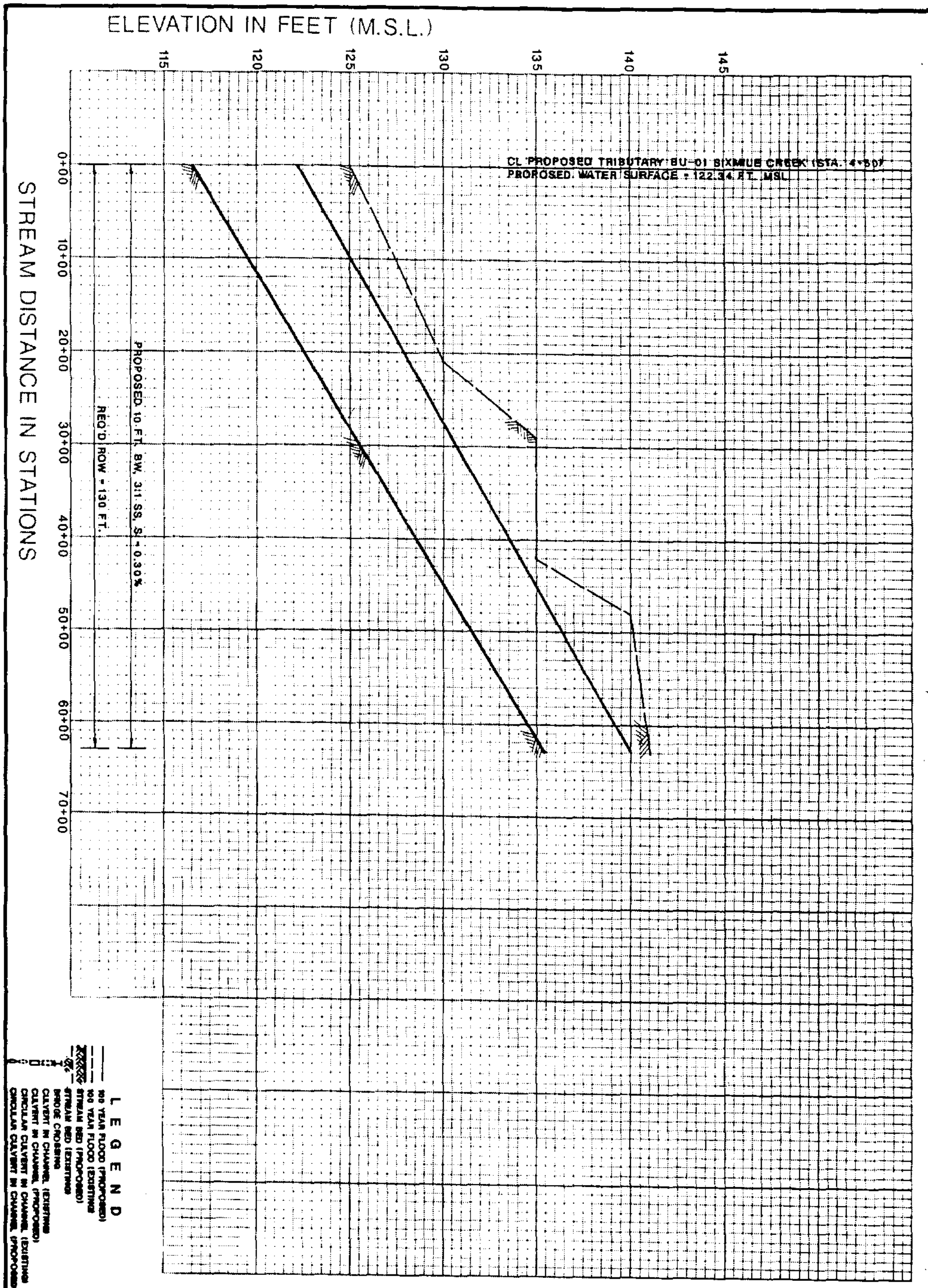
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF MATHIS



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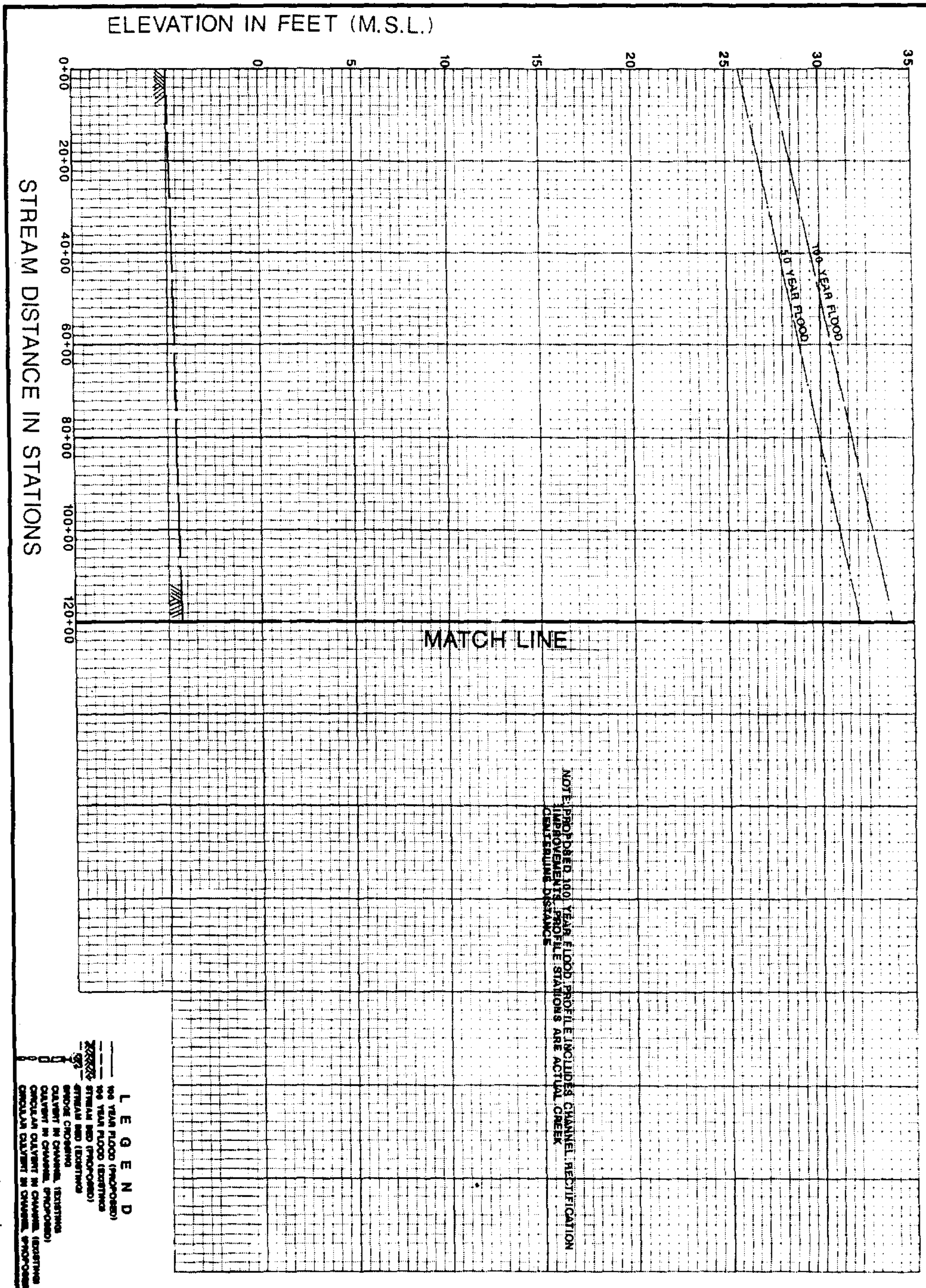

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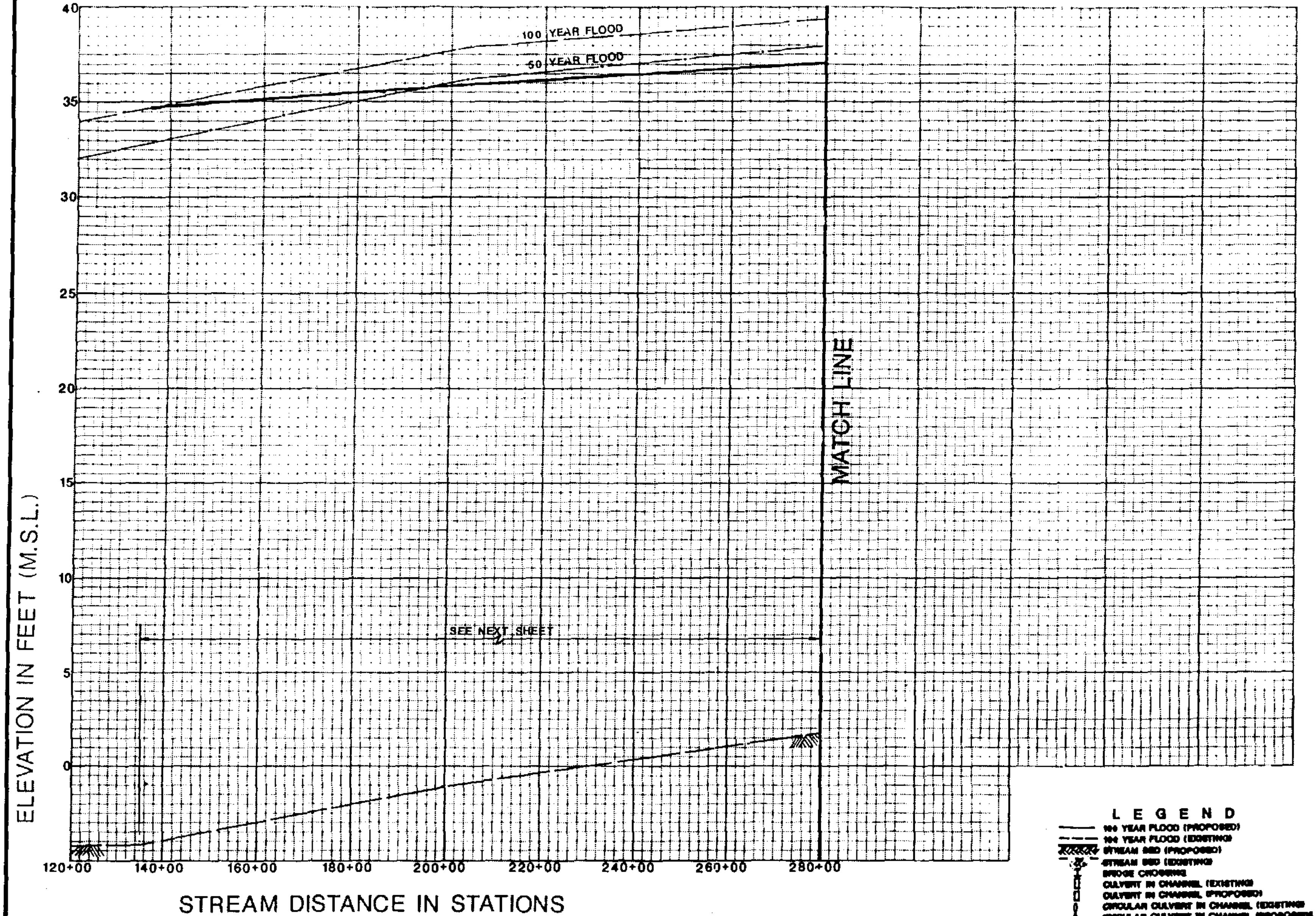
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF SINTON



FLOOD PROFILE

CHILTIPIN CREEK (MAIN AZ)

CITY OF SINTON

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

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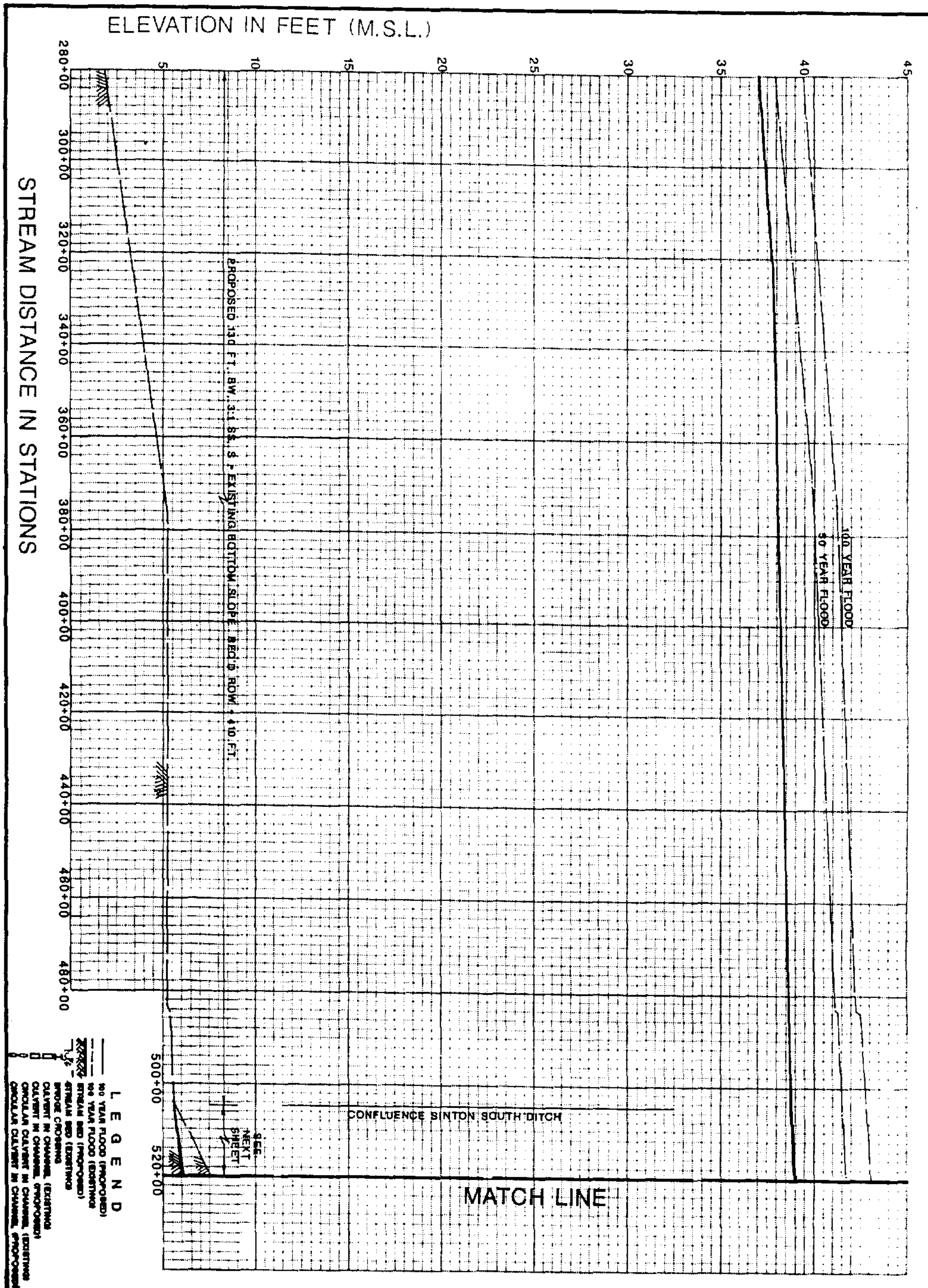

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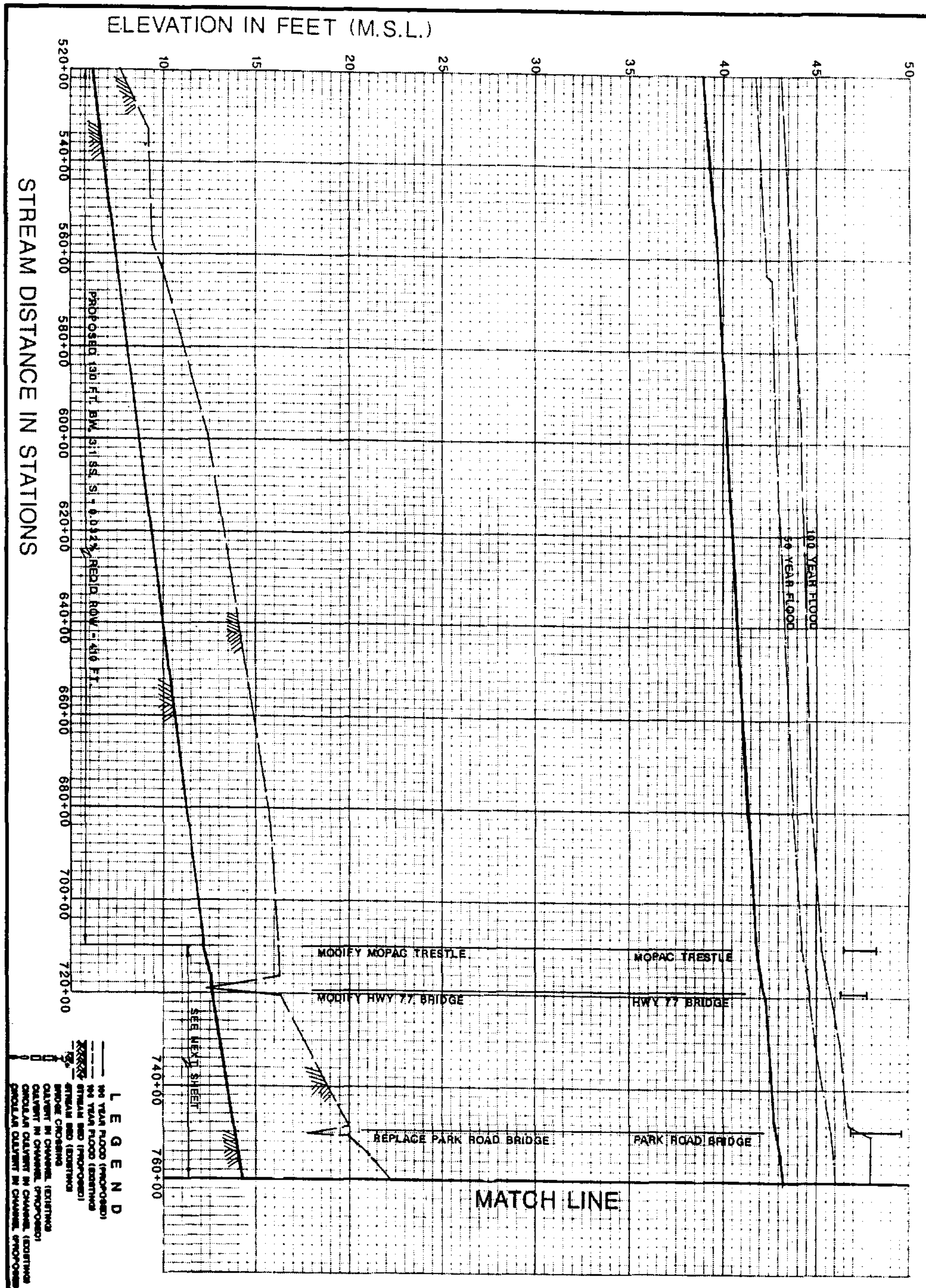
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FLOOD PROFILE

CHILTIPIN CREEK (MAIN AZ)

CITY OF SINTON



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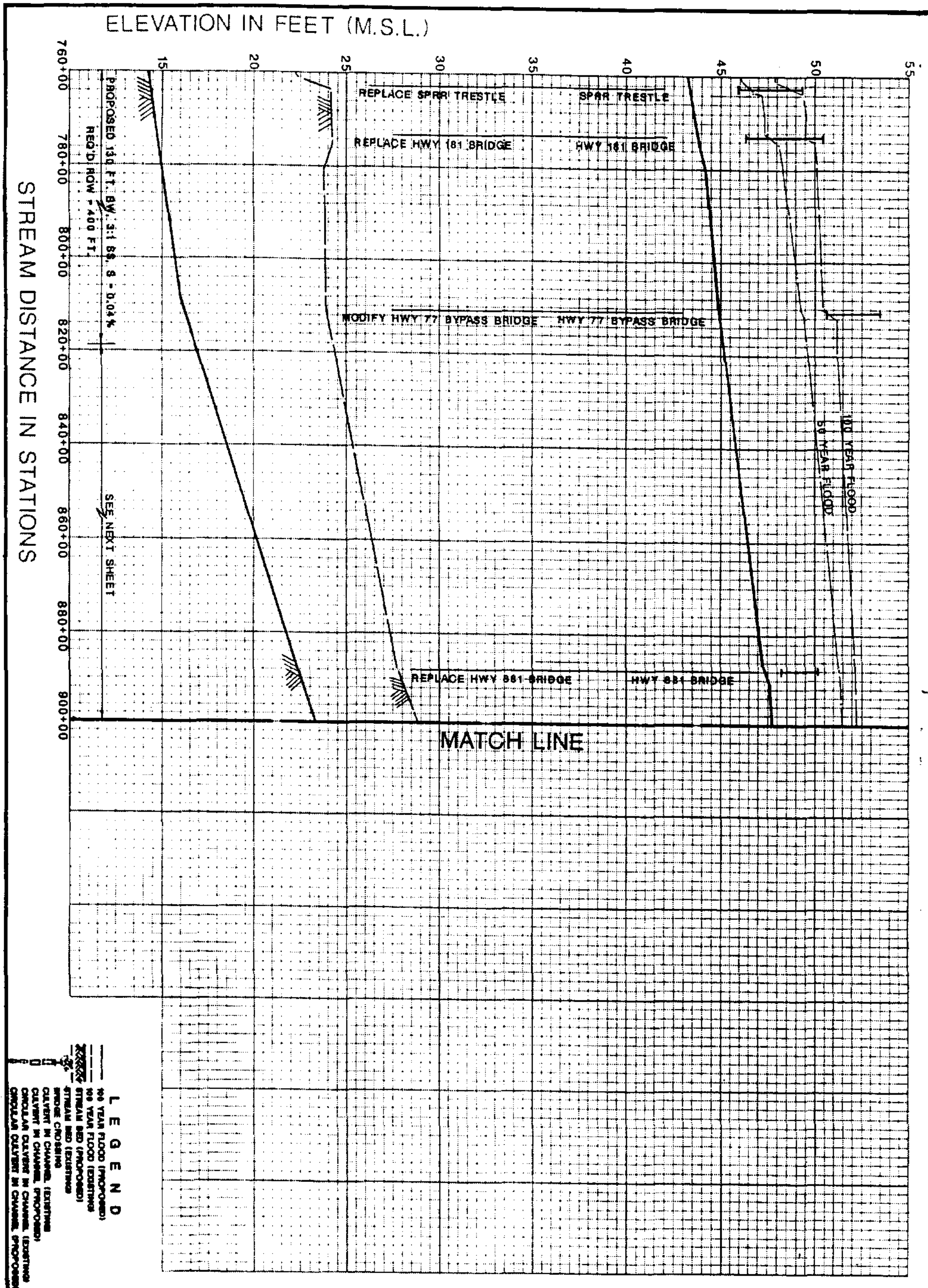
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF SINTON



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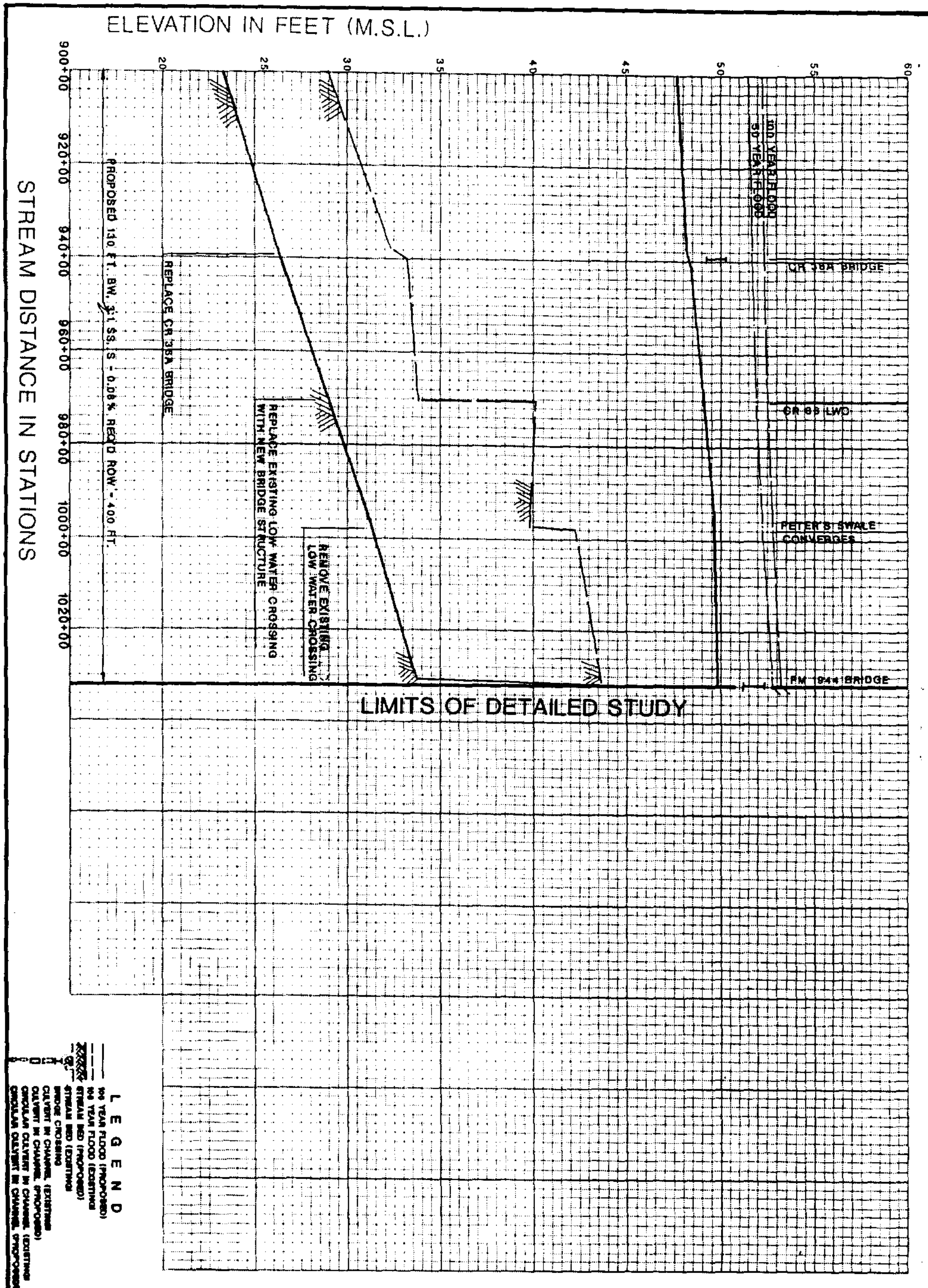
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF SINTON



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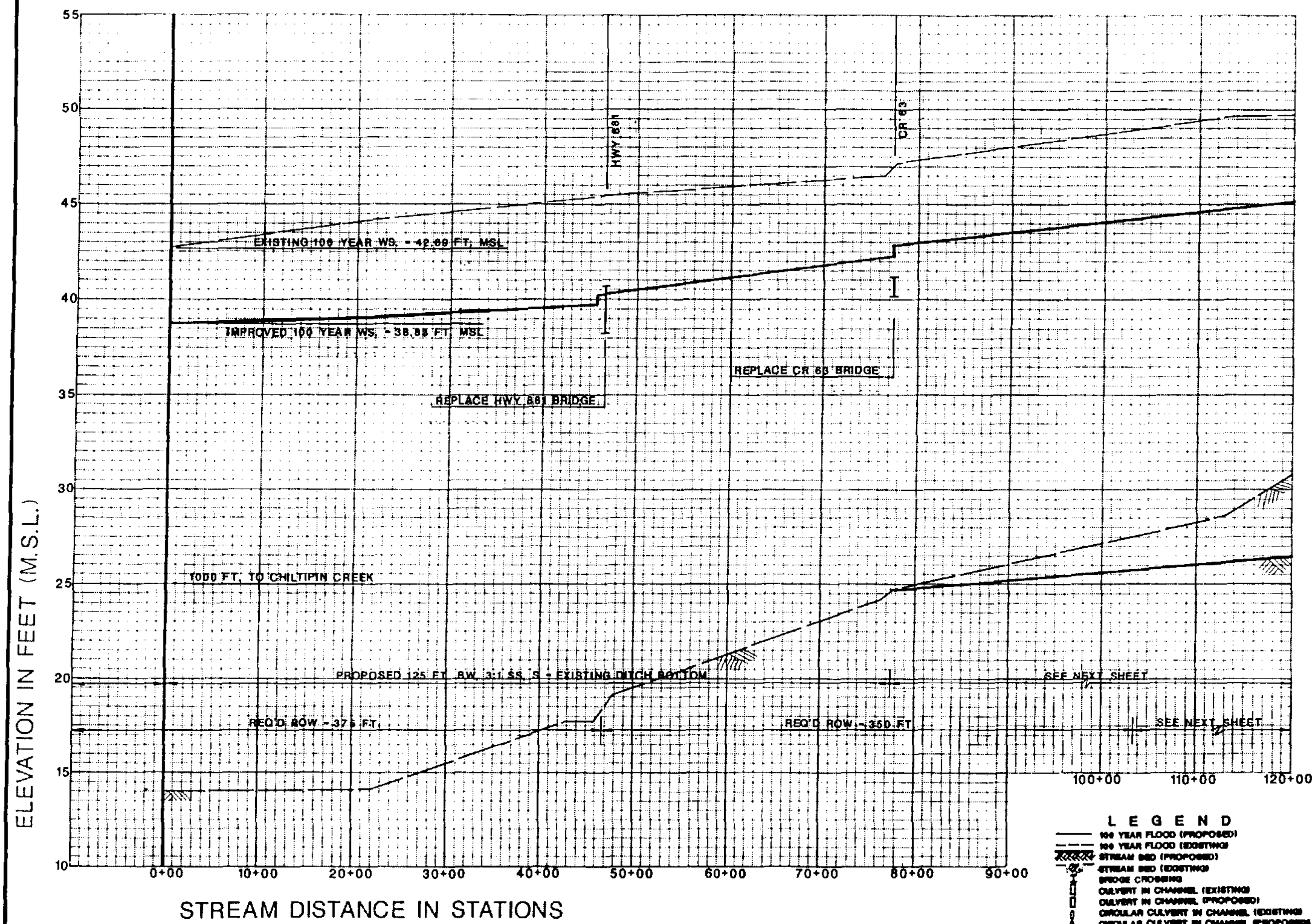
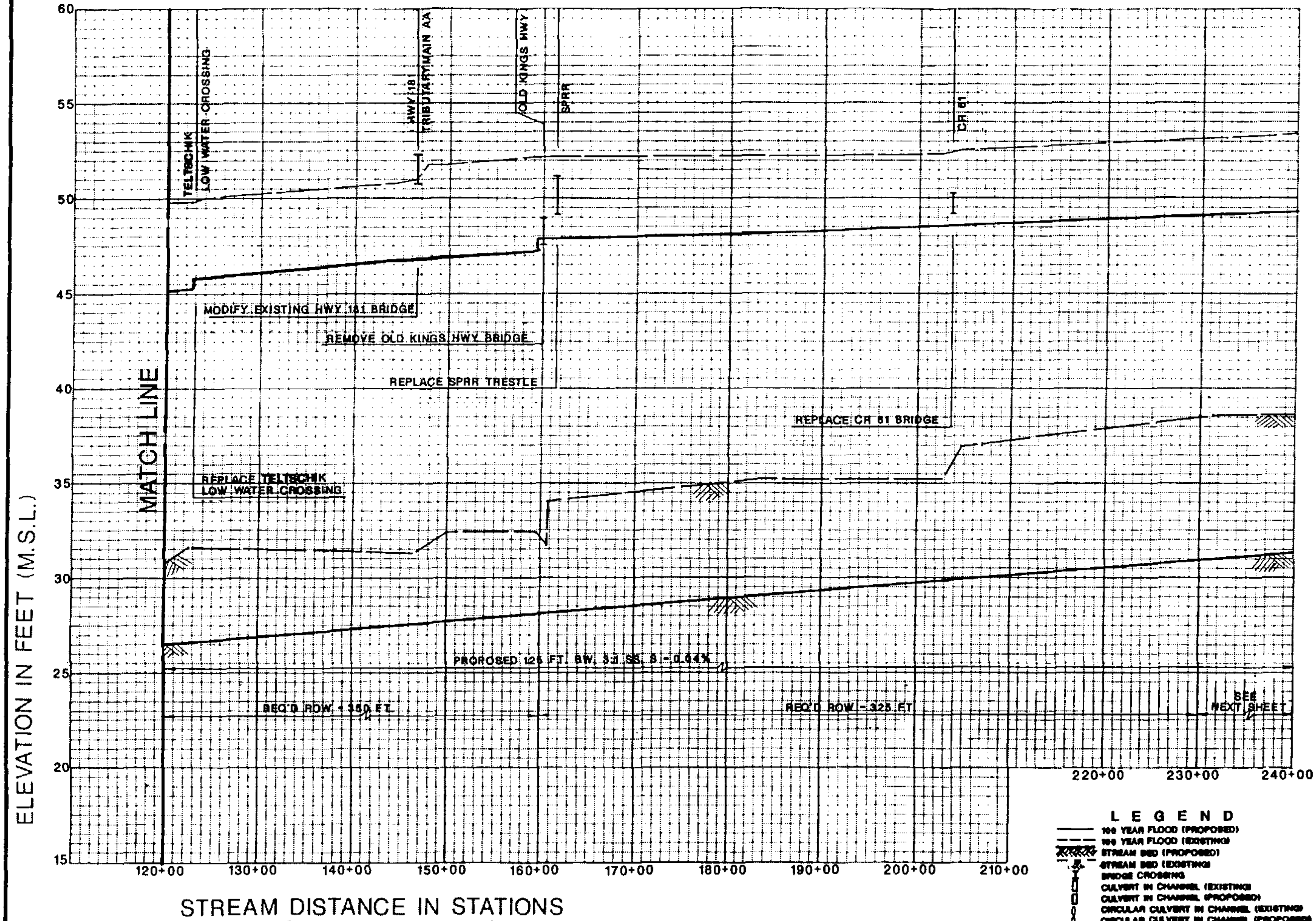
	NAISMITH ENGINEERS, INC. Consulting Engineers — HDR Infrastructure, Inc. A Centerra Company	CITY OF SINTON



FLOOD PROFILE
SINTON SOUTH DITCH
(MAIN AA, AA - 1)
CITY OF SINTON

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers



FLOOD PROFILE

SINTON SOUTH DITCH
(MAIN AA, AA – 1)

CITY OF SINTON

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

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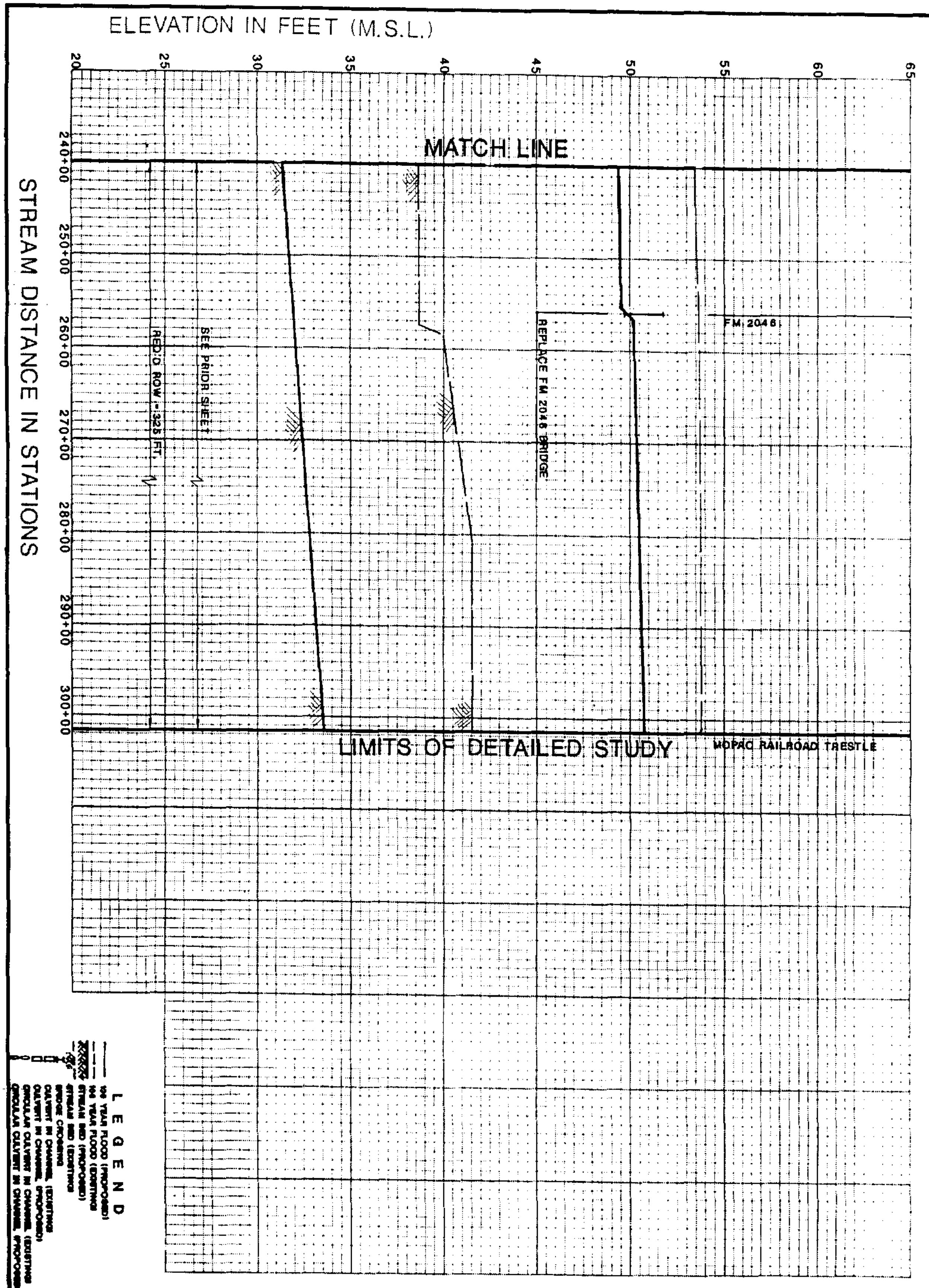

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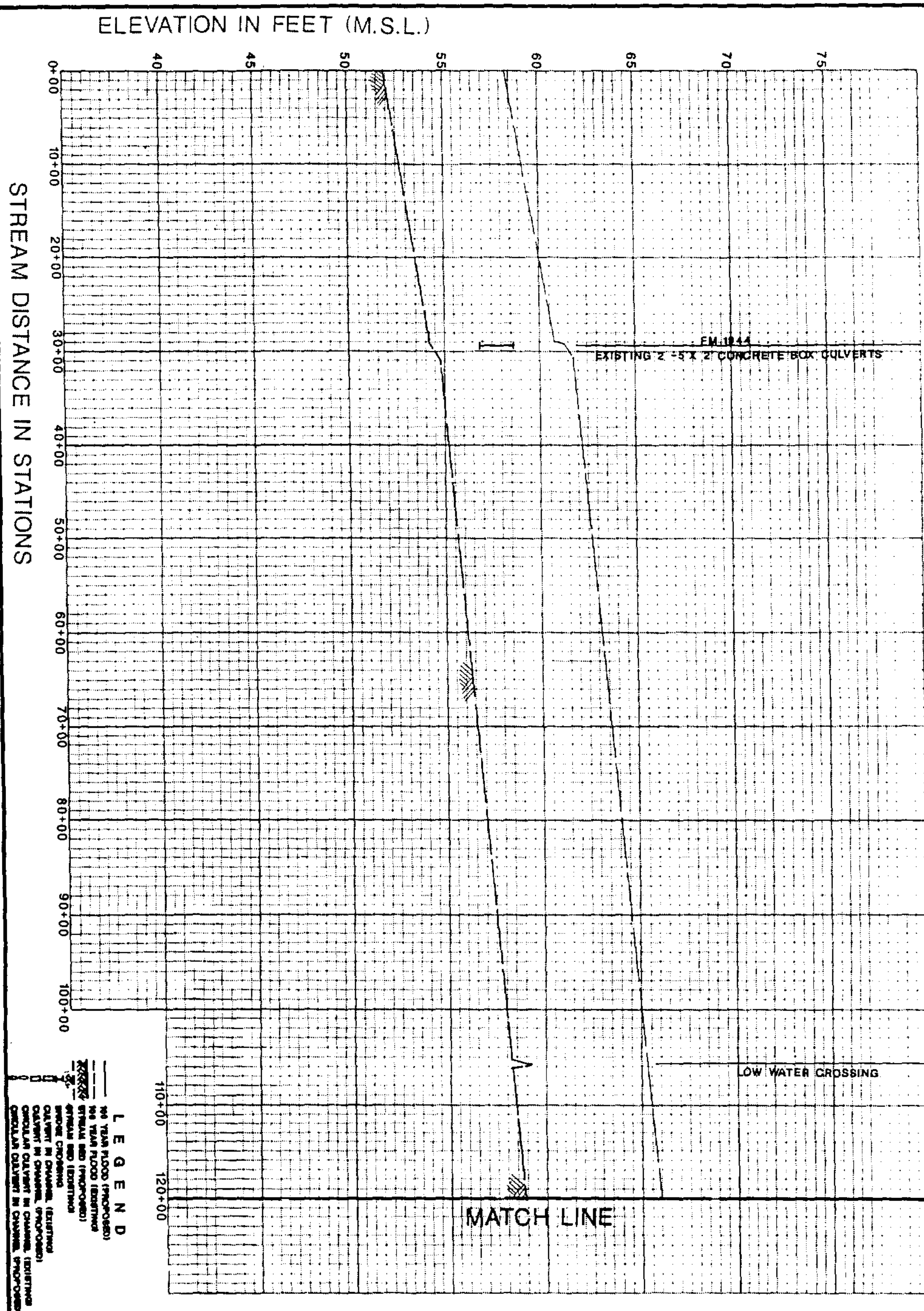
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF SINTON



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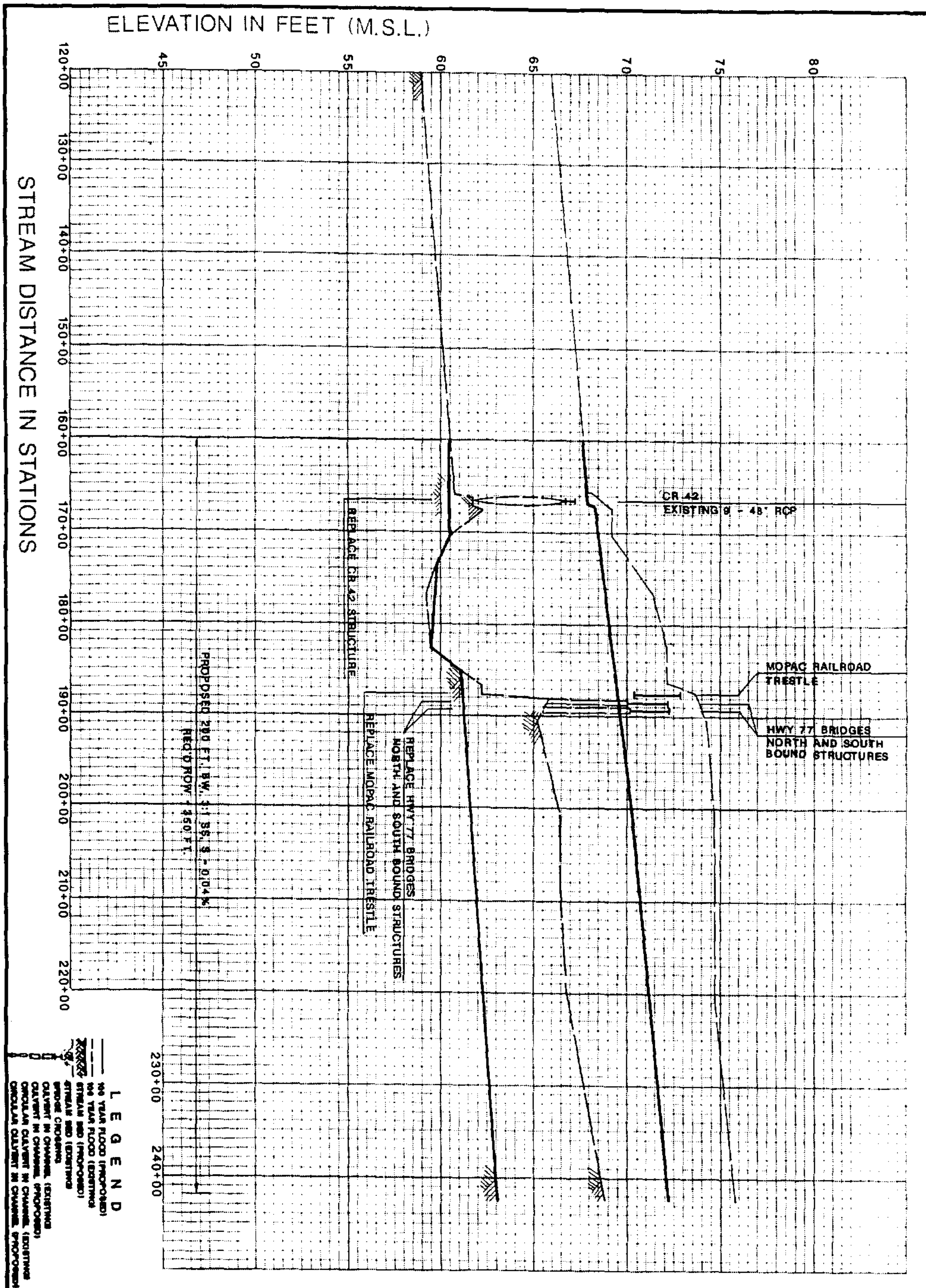
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF ODEM



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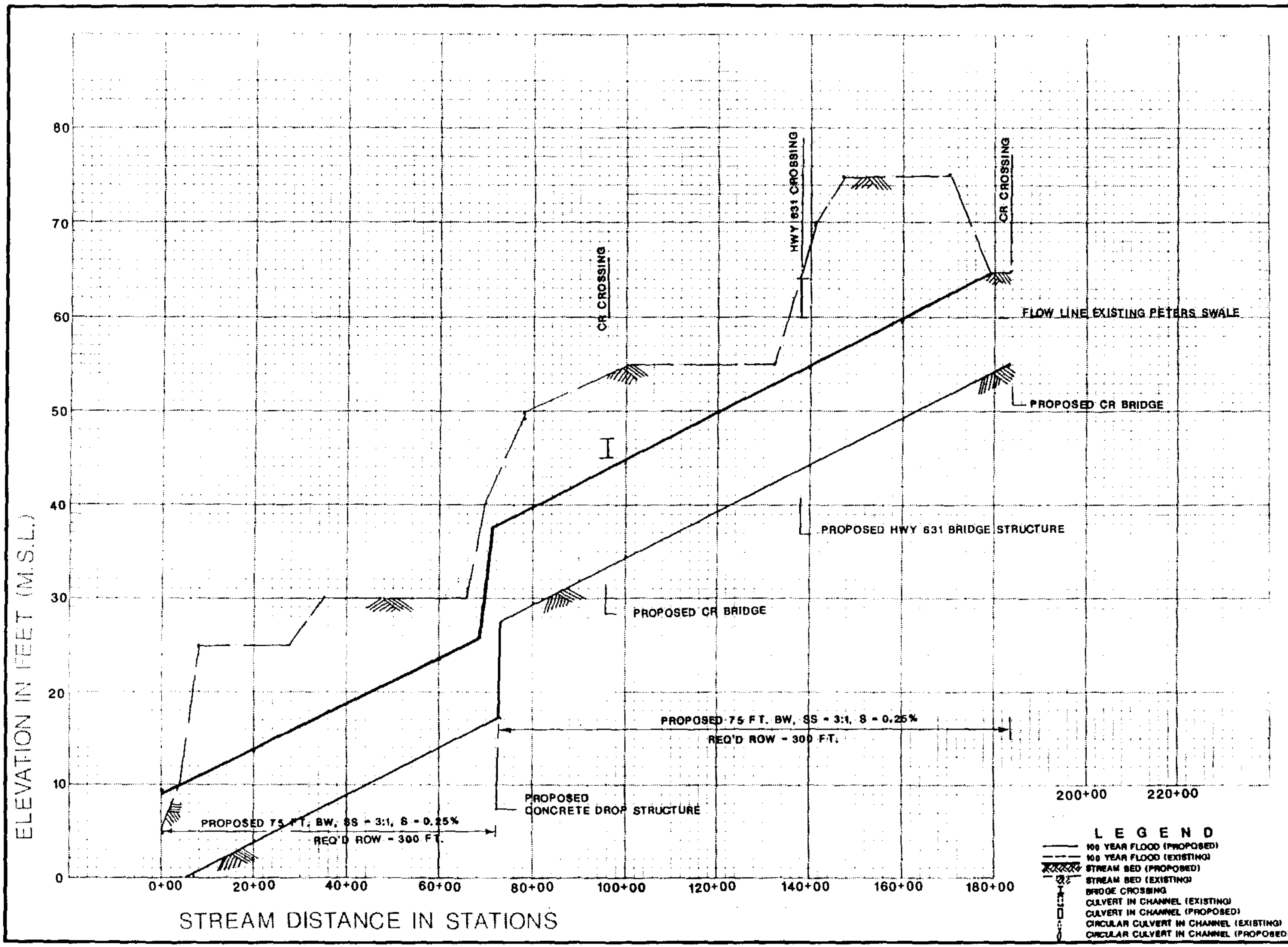
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FLOOD PROFILE
PETERS SWALE
(MAIN AB - 1)

CITY OF ODEM



FLOOD PROFILE
ALTERNATE DIVERSION CHANNEL

CITY OF ODEM

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

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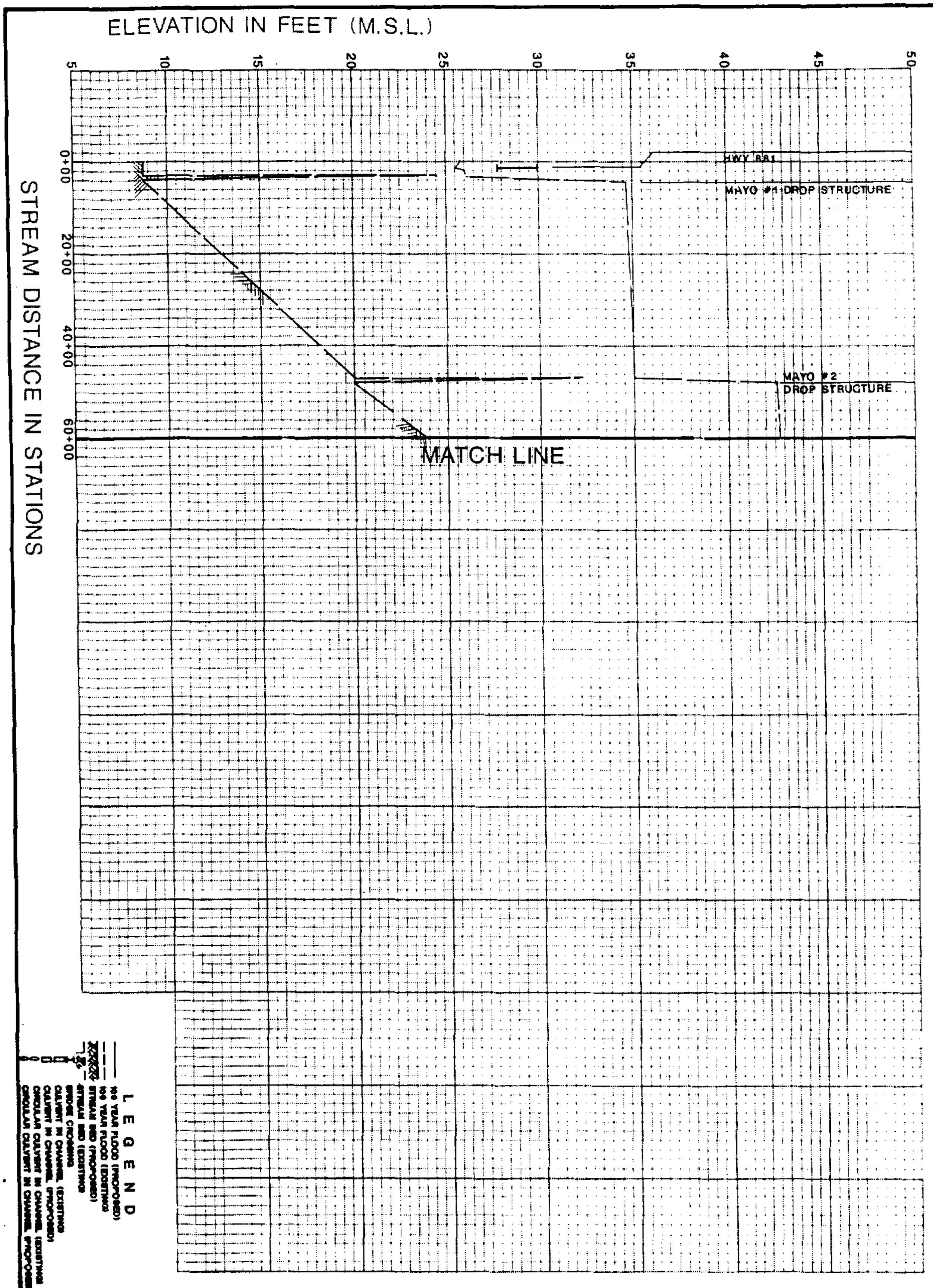

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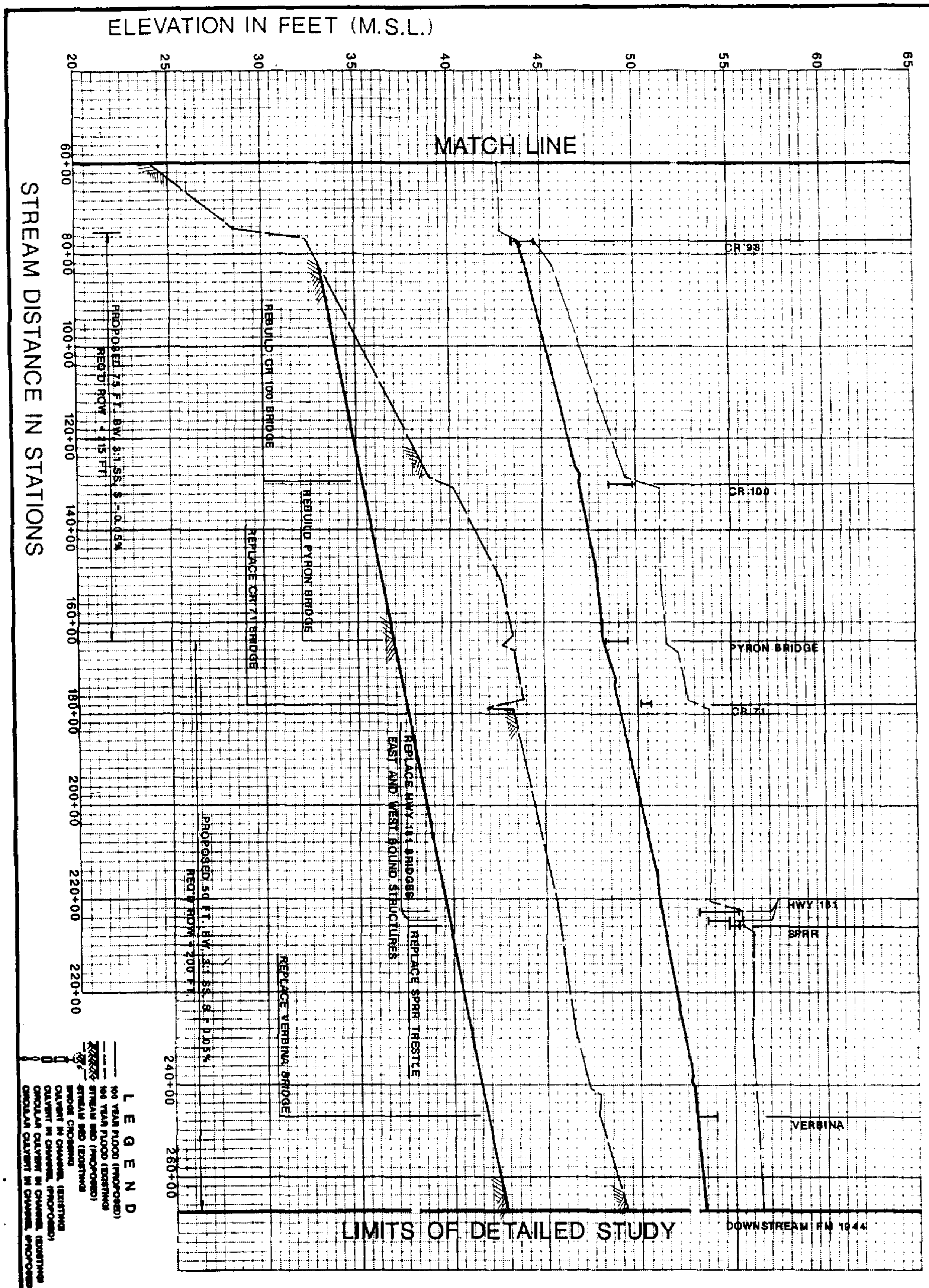
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FLOOD PROFILE
(MAIN AJ)

CITY OF TAFT



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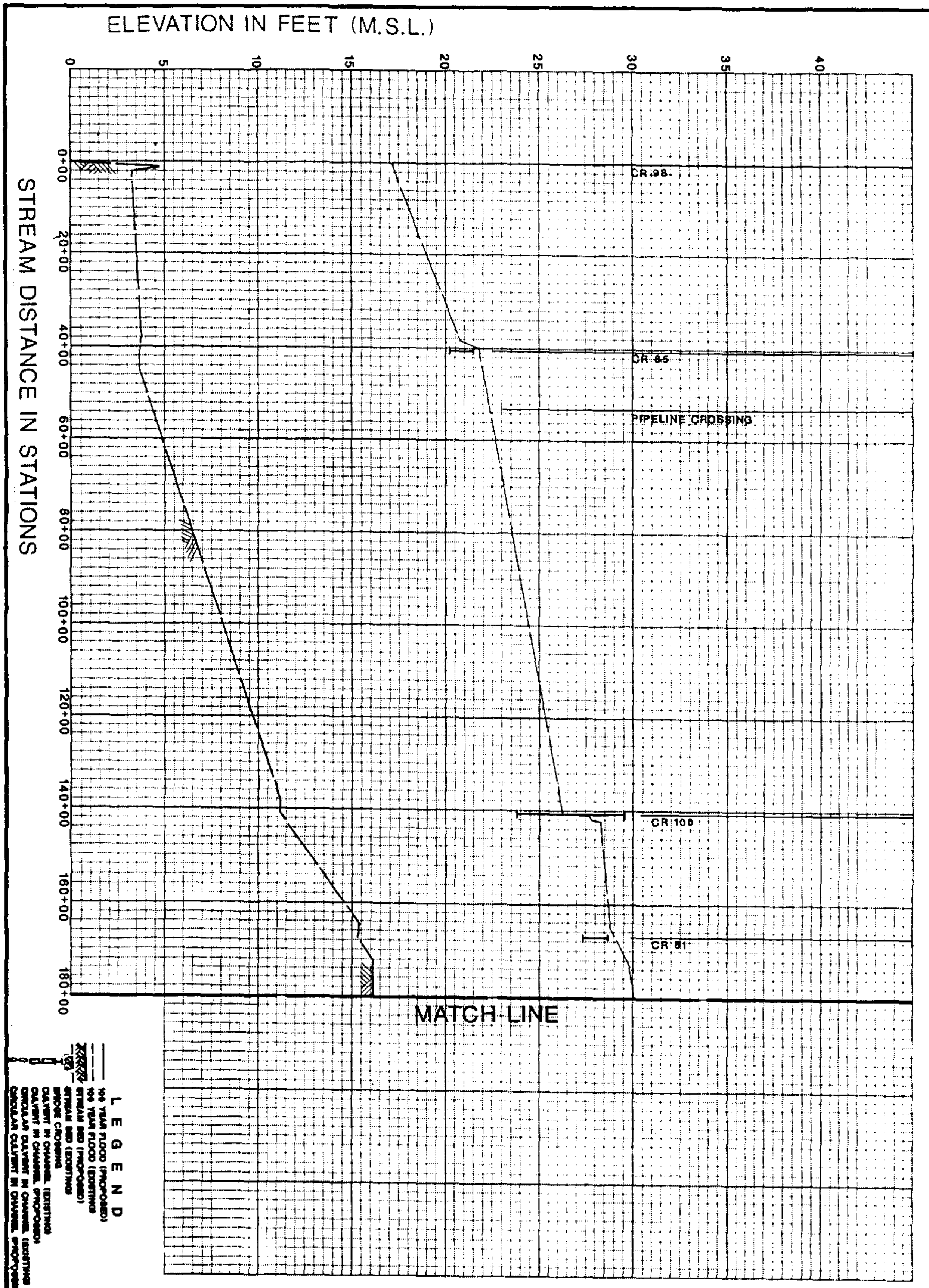
	NAISMITH ENGINEERS, INC. Consulting Engineers — HDR Infrastructure, Inc. A Centerra Company	CITY OF TAFT



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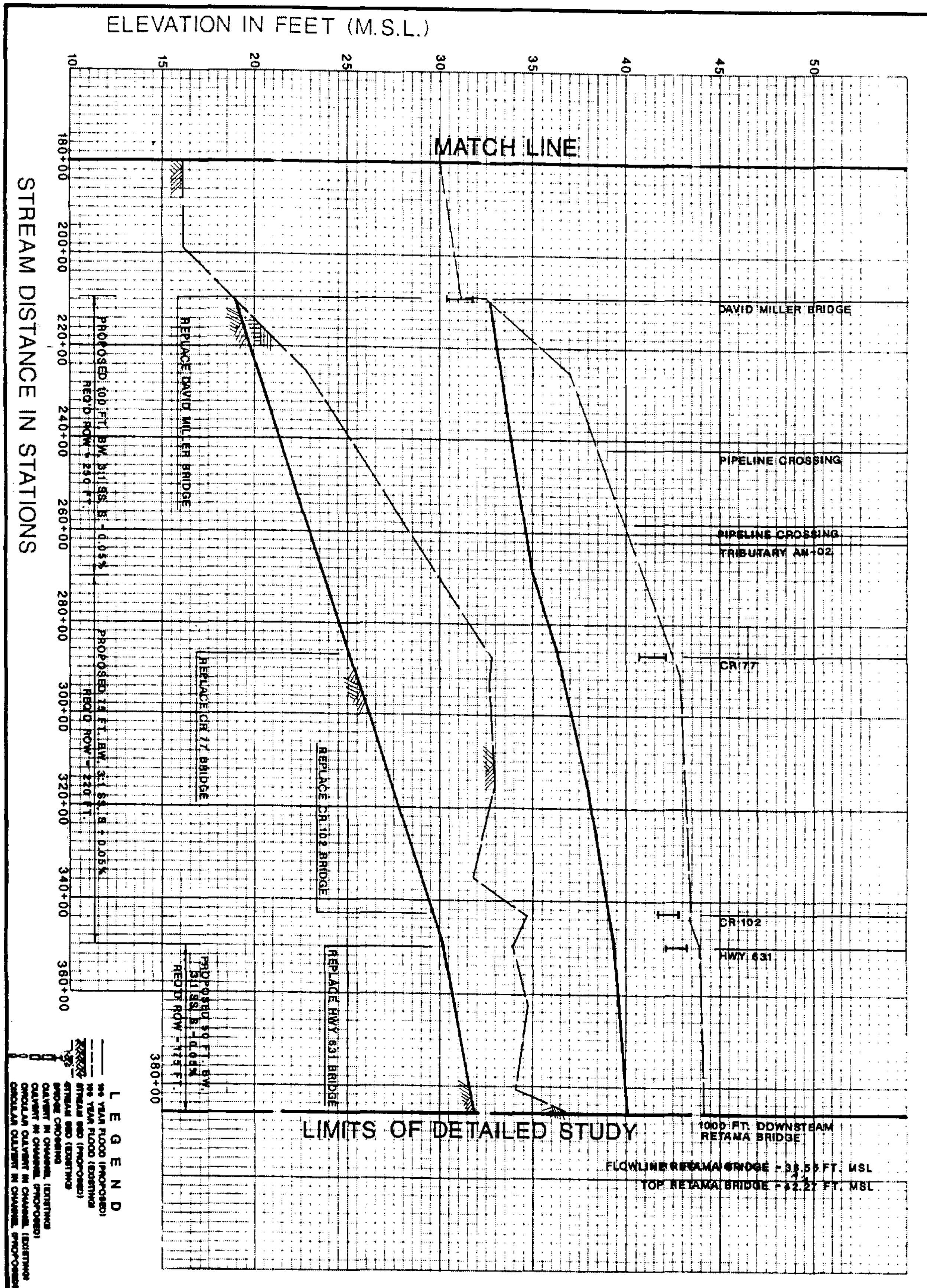
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF TAFT



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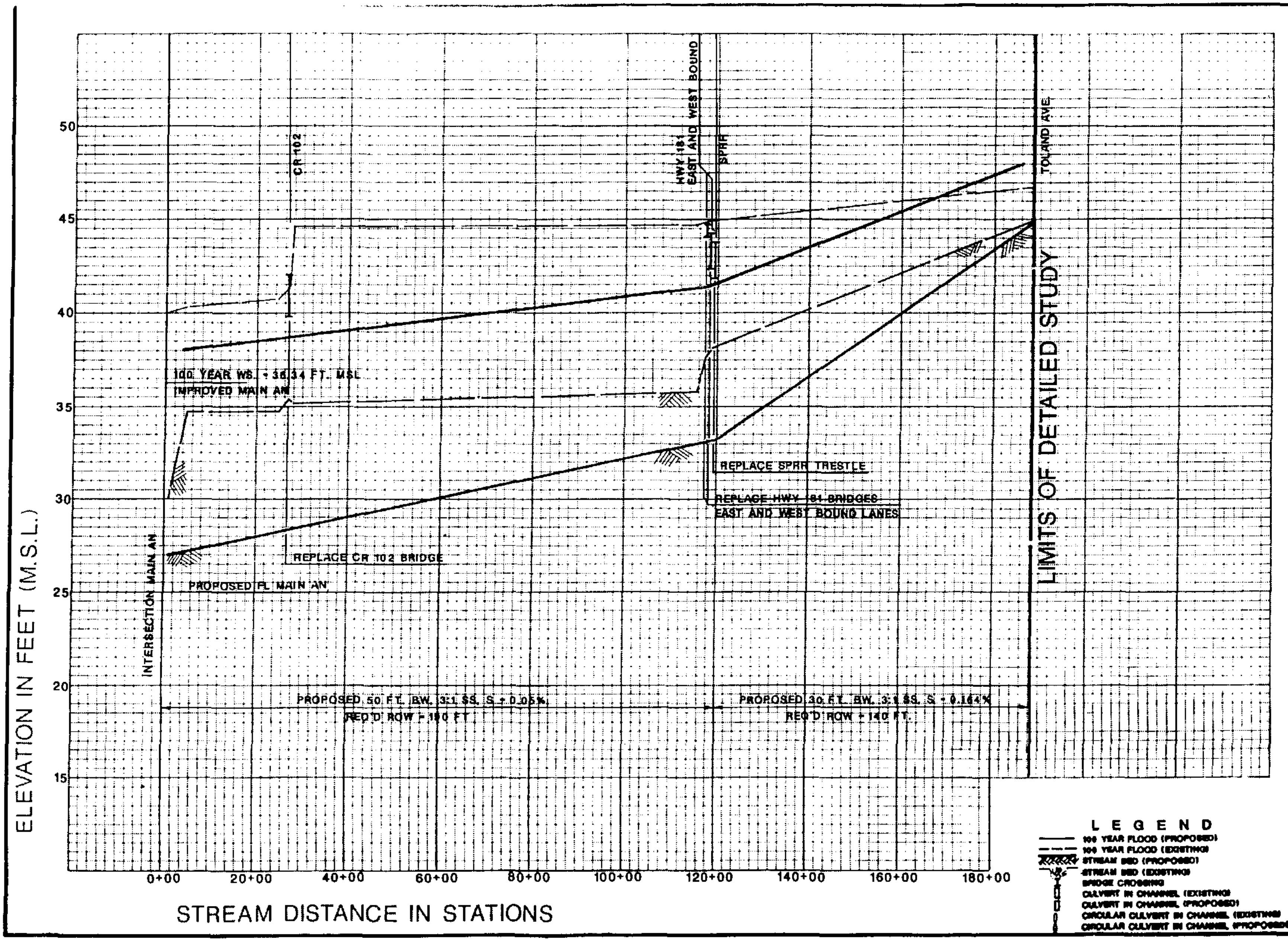
	NAISMITH ENGINEERS, INC. Consulting Engineers

FLOOD PROFILE

(MAIN AN)

CITY OF TAFT



FLOOD PROFILE

(AN - 02)

CITY OF TAFT

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

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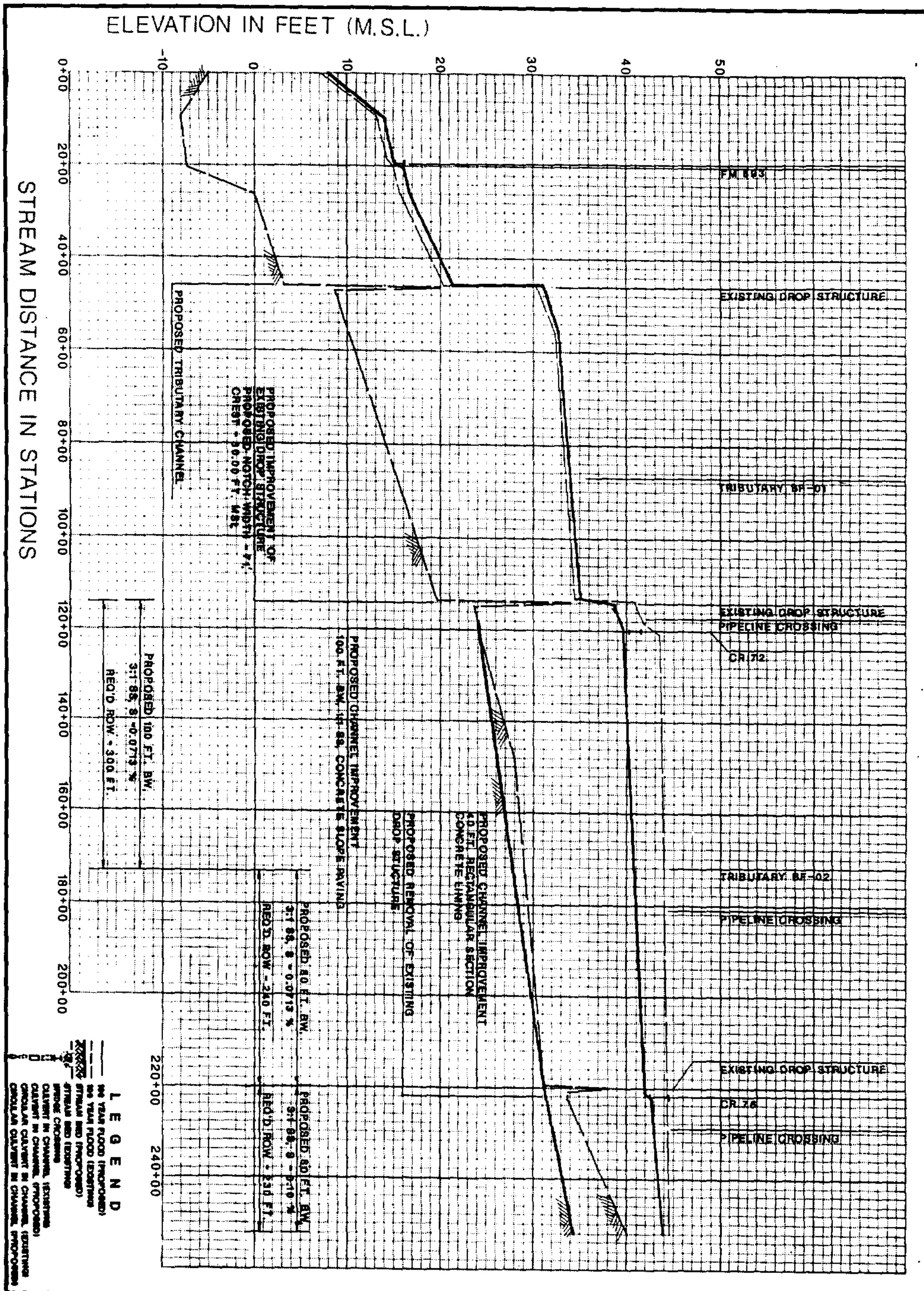

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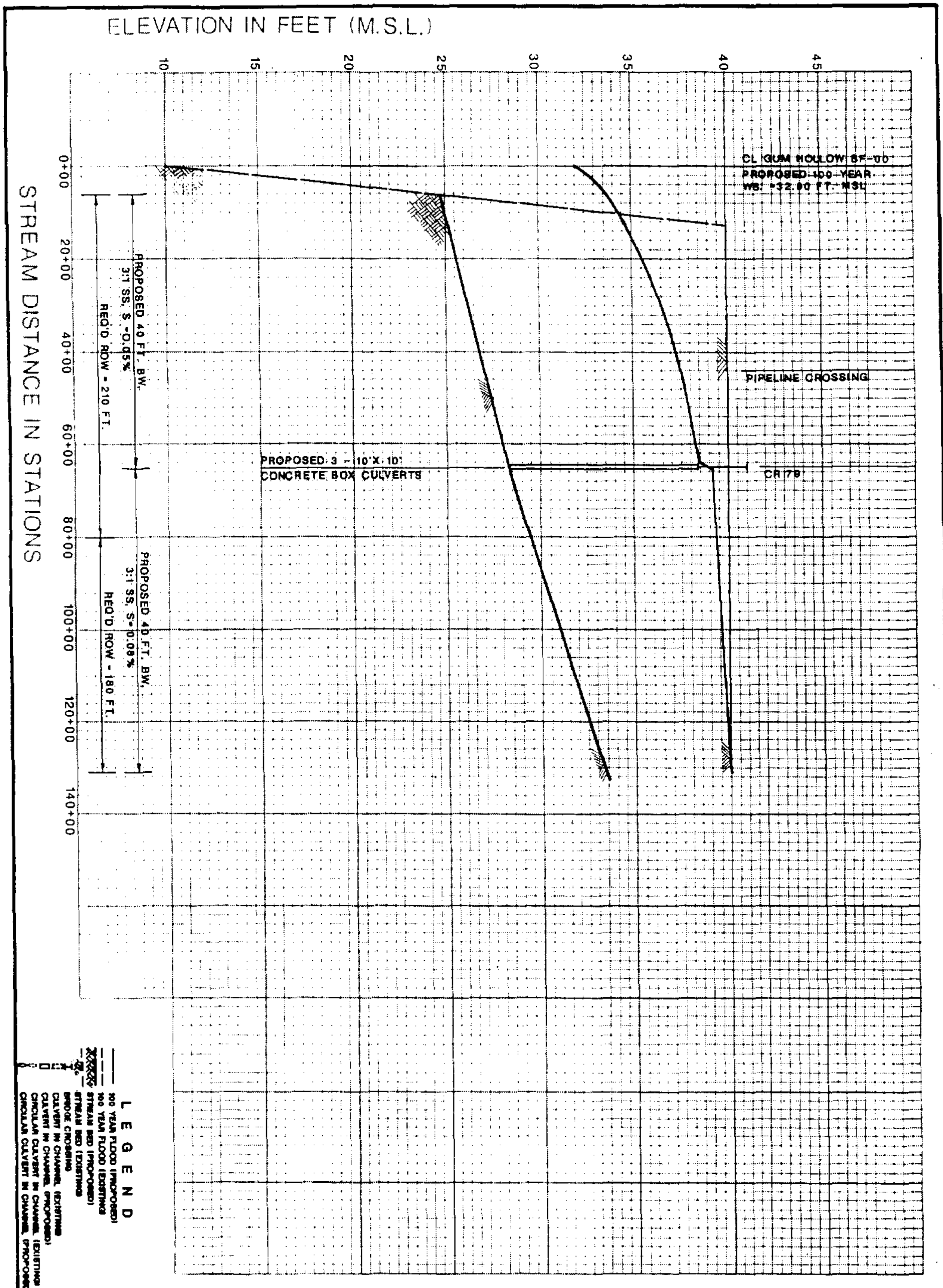
	NAISMITH ENGINEERS, INC. Consulting Engineers	GREGORY/PORTLAND AREA



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FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

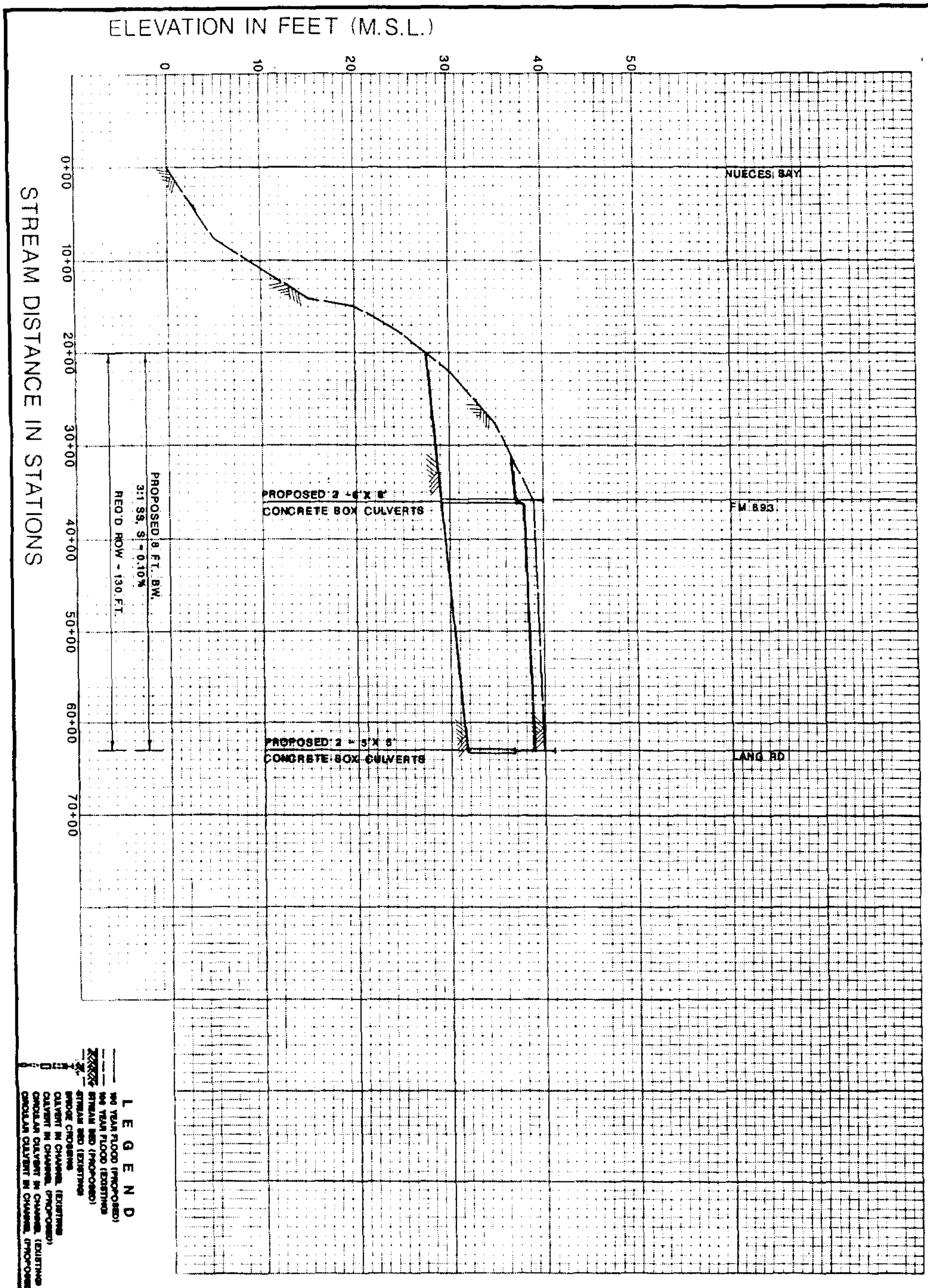
NAISMITH ENGINEERS, INC.
Consulting Engineers

HDR Infrastructure, Inc.
A Centerra Company

FLOOD PROFILE

PROPOSED GUM HOLLOW
TRIBUTARY (BF - 03)

GREGORY/PORTLAND AREA

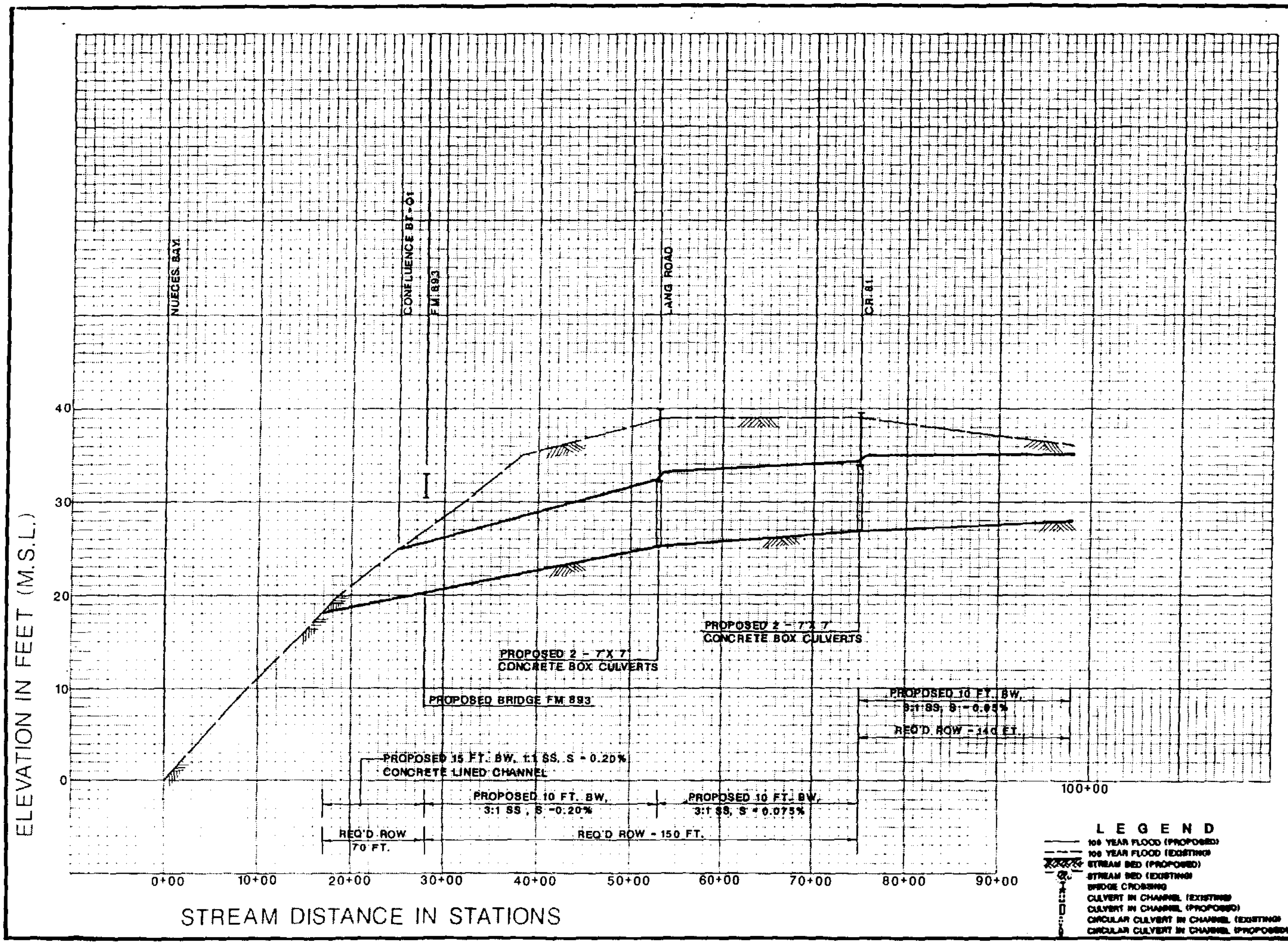


FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

FLOOD PROFILE
PROPOSED AIRPORT DITCH

GREGORY/PORTLAND AREA



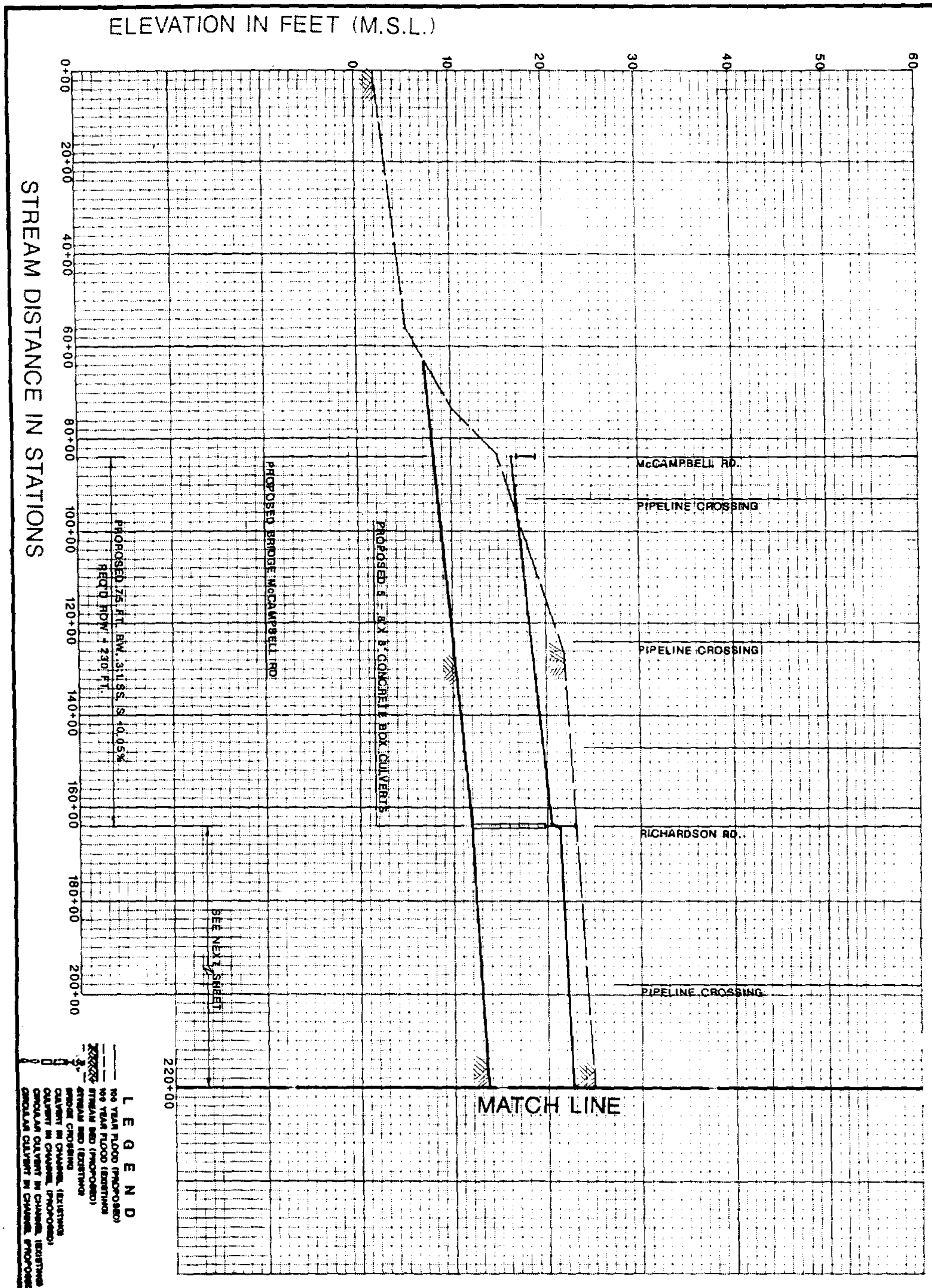
FLOOD PROFILE
PROPOSED DOYLE ADDITION
DITCH IMPROVEMENTS AND EXTENSION
(BT - 00)

GREGORY/PORTLAND AREA

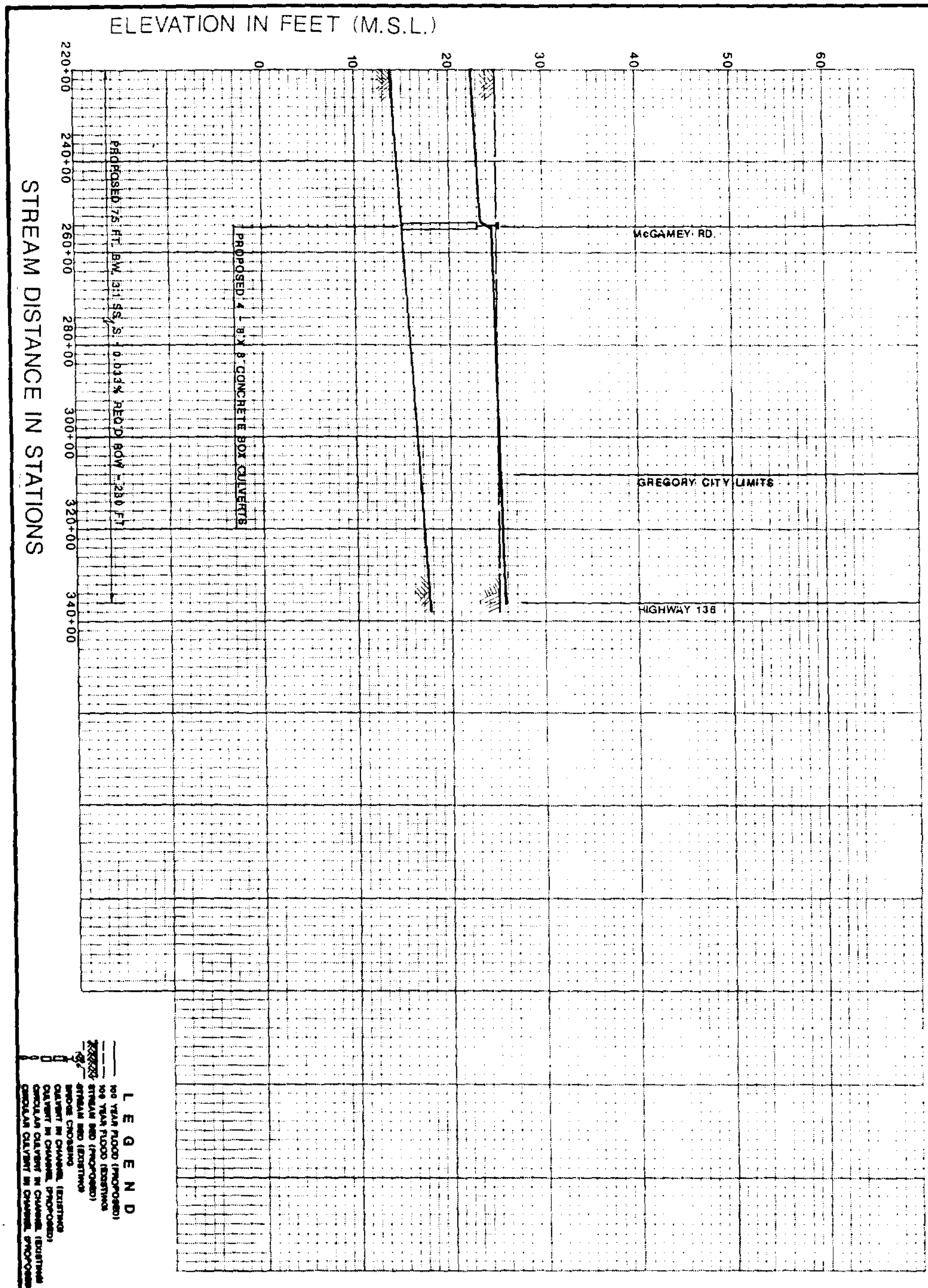
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

PAGE



PAGE 26	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	FLOOD PROFILE (MAIN AS)
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF GREGORY



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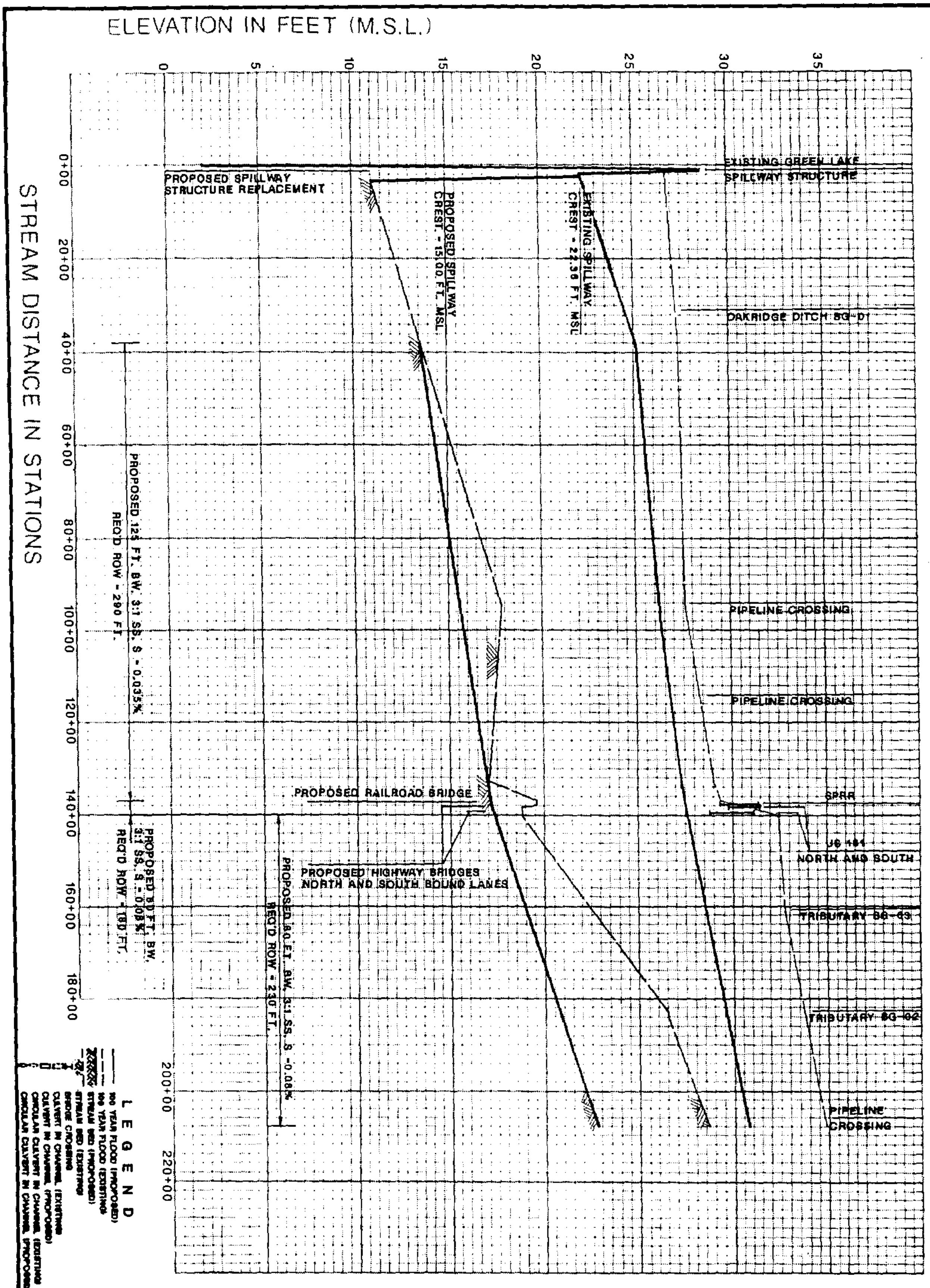
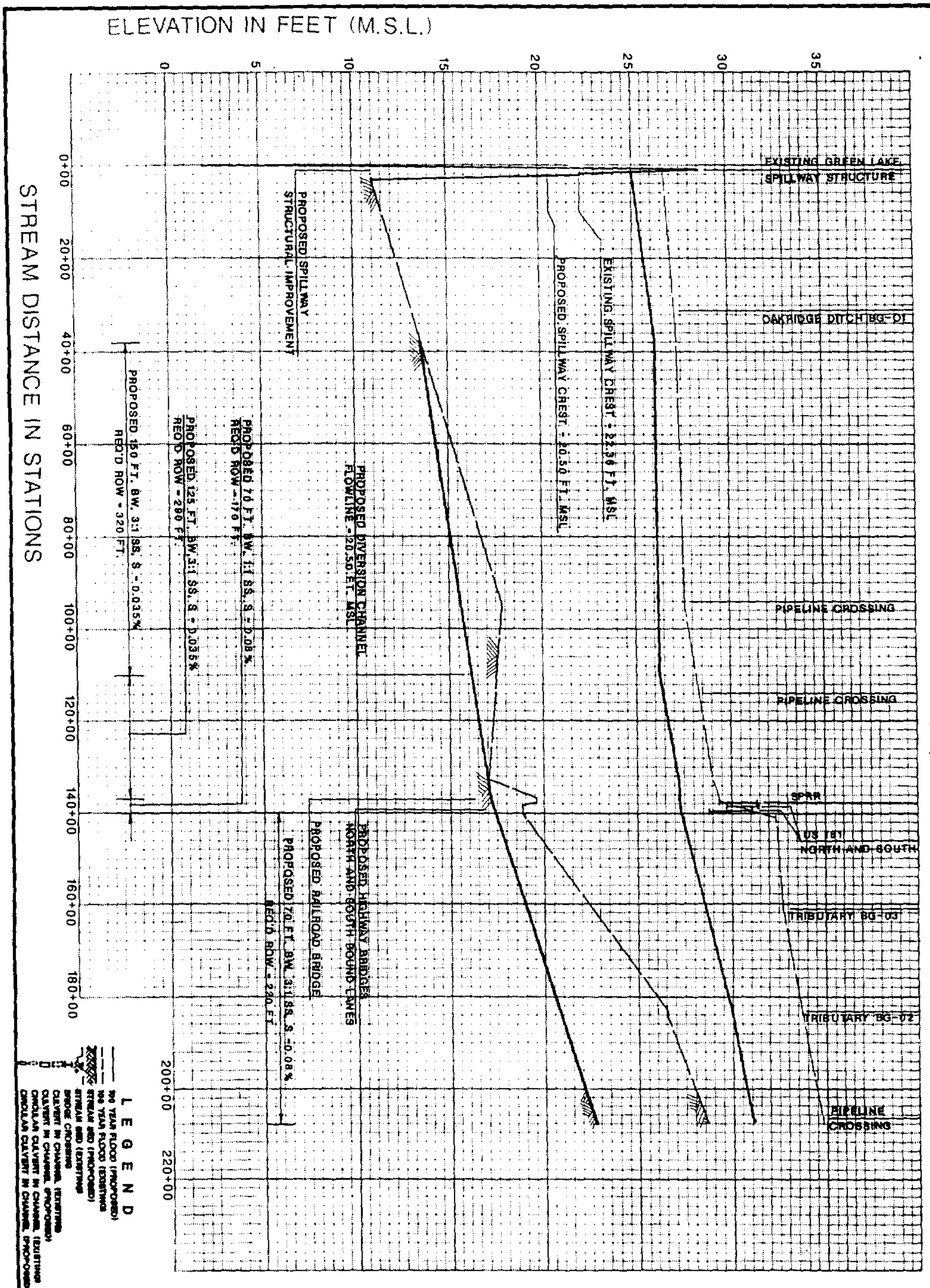
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF GREGORY



PAGE 28	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	FLOOD PROFILE GREEN LAKE (MAIN BG - 00) ALTERNATIVE 1
	NAISMITH ENGINEERS, INC. Consulting Engineers	GREGORY/PORTLAND AREA



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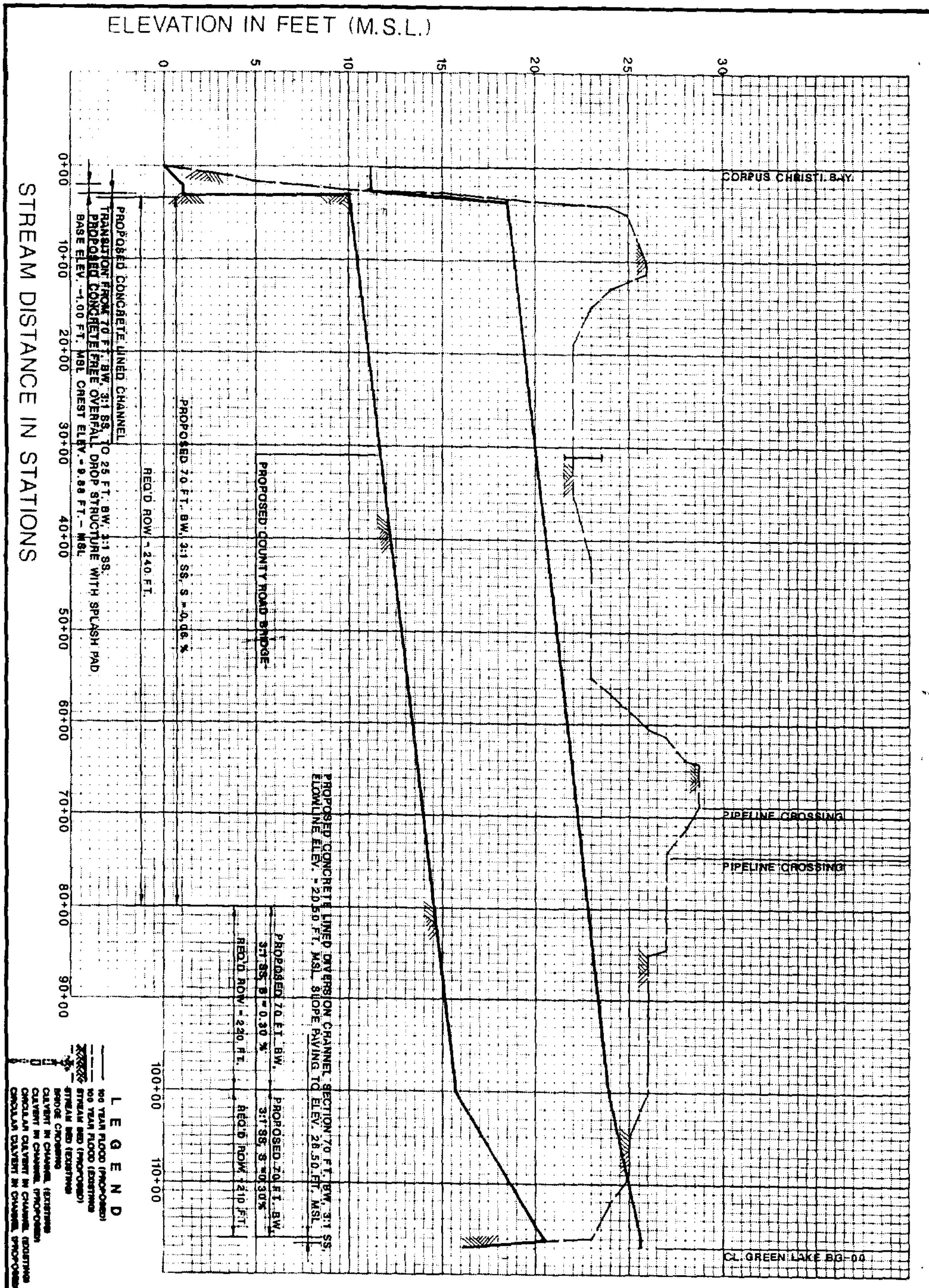
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC. Consulting Engineers

FLOOD PROFILE
GREEN LAKE (MAIN BG - 00)
ALTERNATIVE 2

GREGORY/PORTLAND AREA



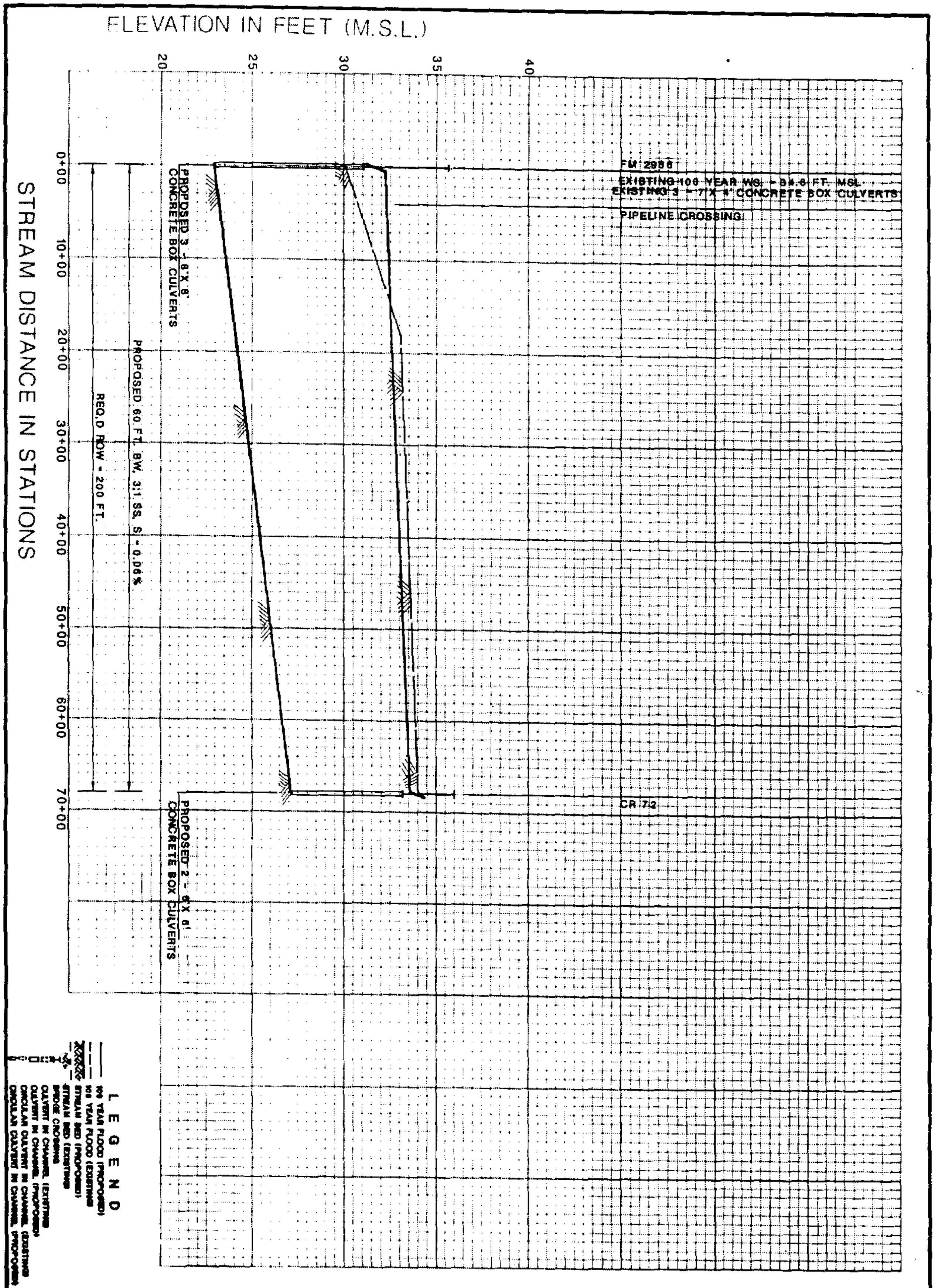
FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC. Consulting Engineers

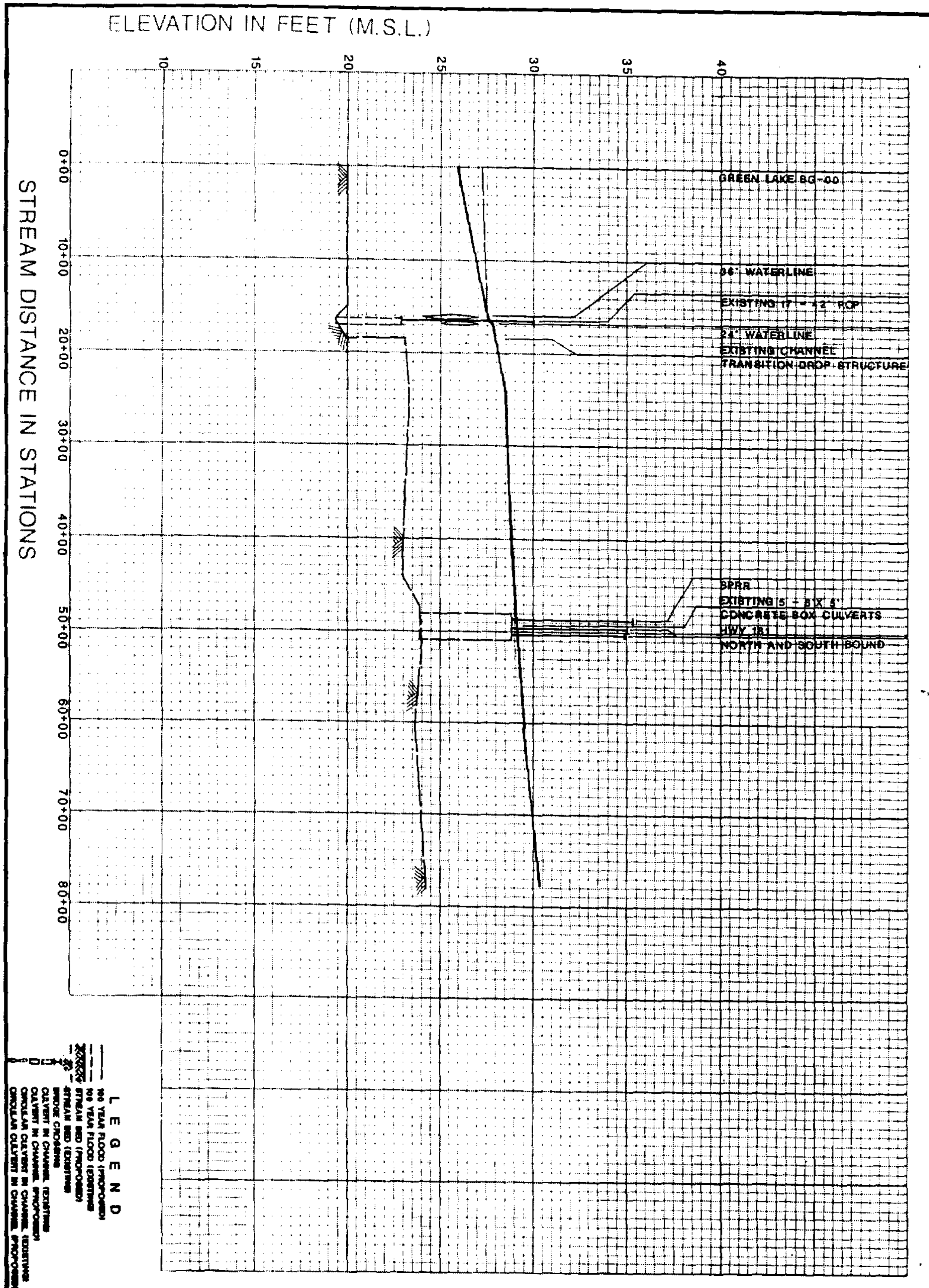
FLOOD PROFILE

GREEN LAKE DIVERSION CHANNEL
PROPOSED WATERSHED

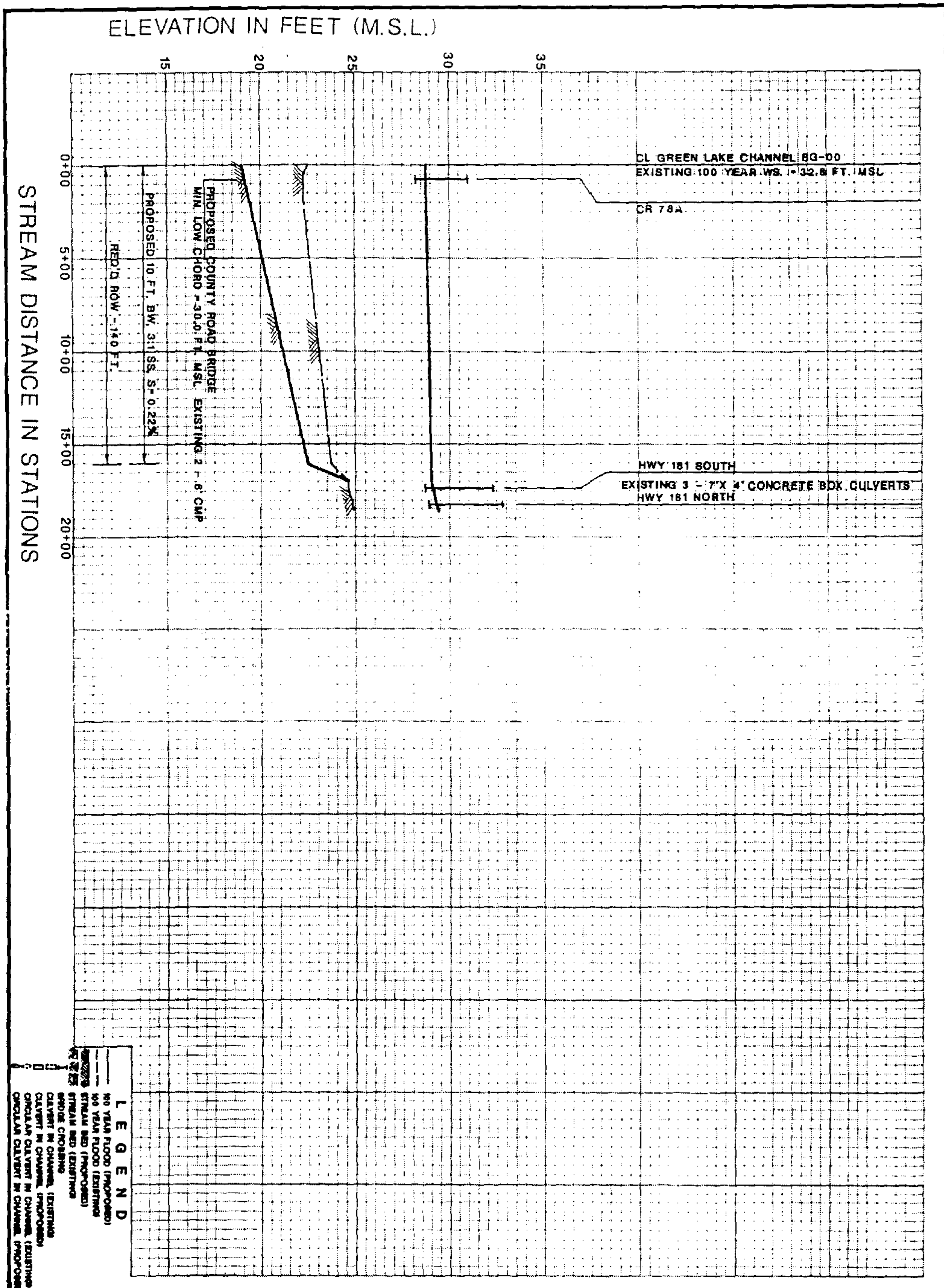
GREGORY/PORTLAND AREA



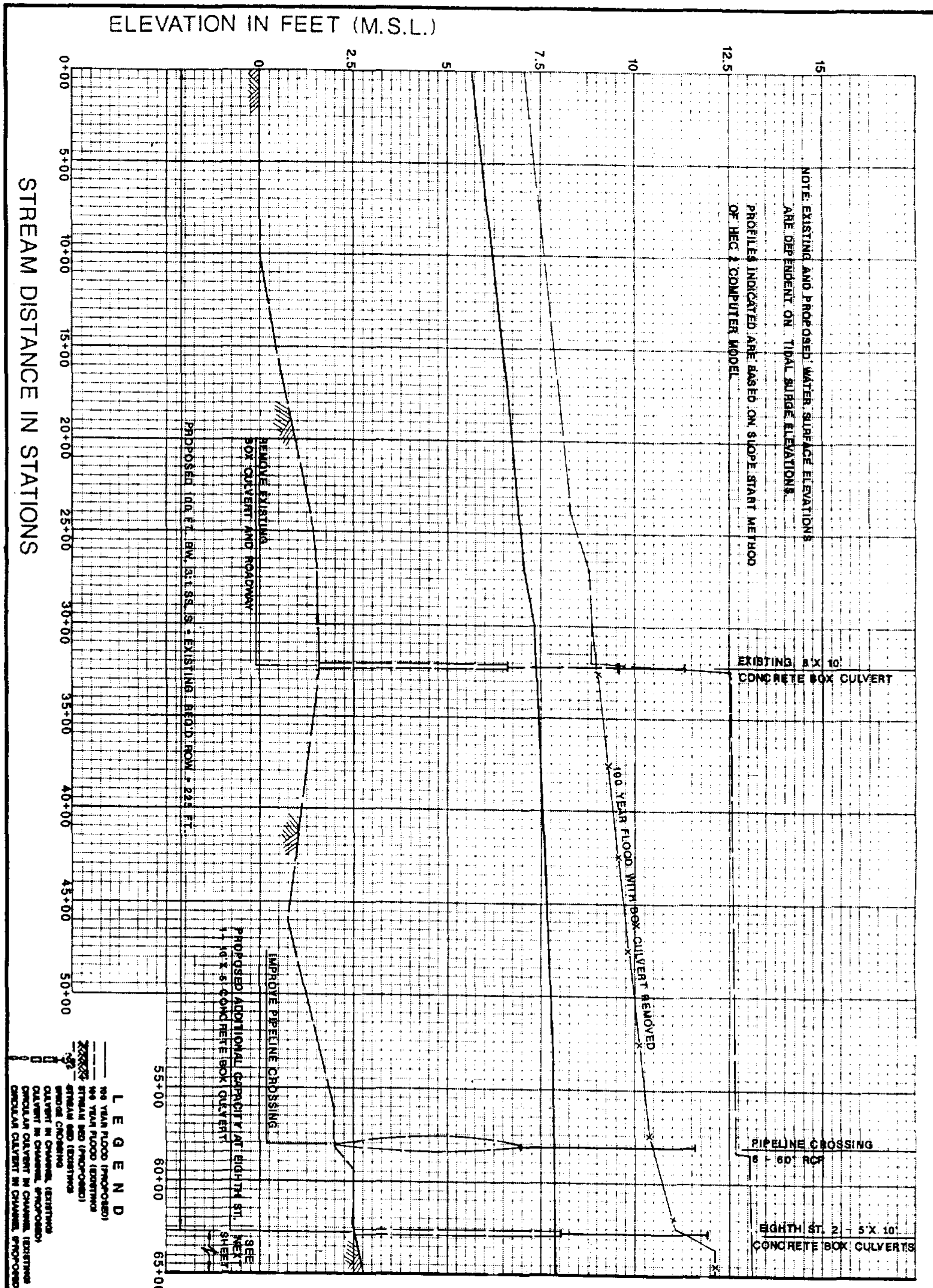
PAGE 31	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	FLOOD PROFILE GREEN LAKE CHANNEL EXTENSION (MAIN BG - 00) PROPOSED WATERSHED
	NAISMITH ENGINEERS, INC. Consulting Engineers	GREGORY/PORTLAND AREA



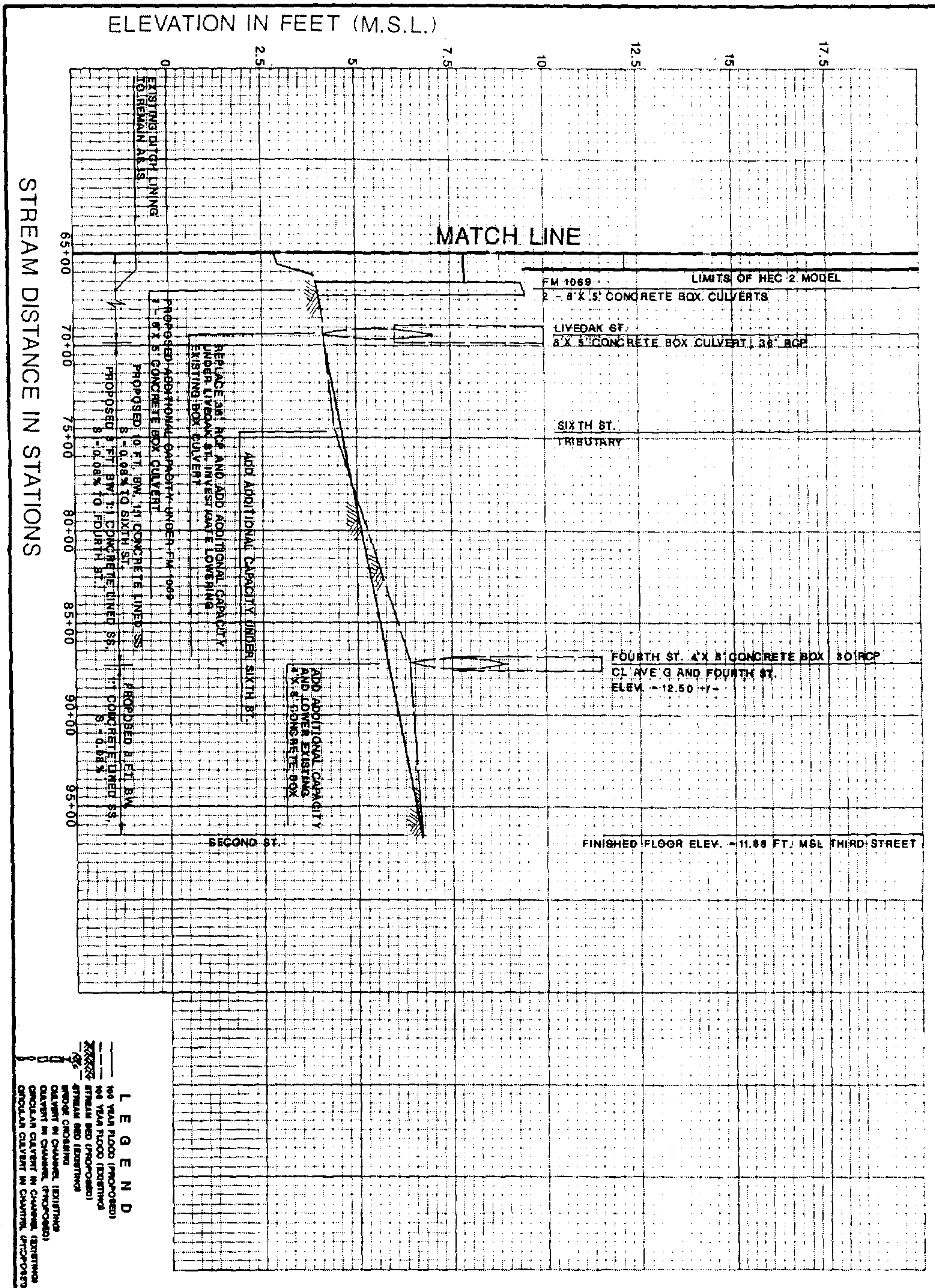
PAGE 32	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	FLOOD PROFILE OAKRIDGE DITCH (BG – 01)
	NAISMITH ENGINEERS, INC. Consulting Engineers	GREGORY/PORTLAND AREA



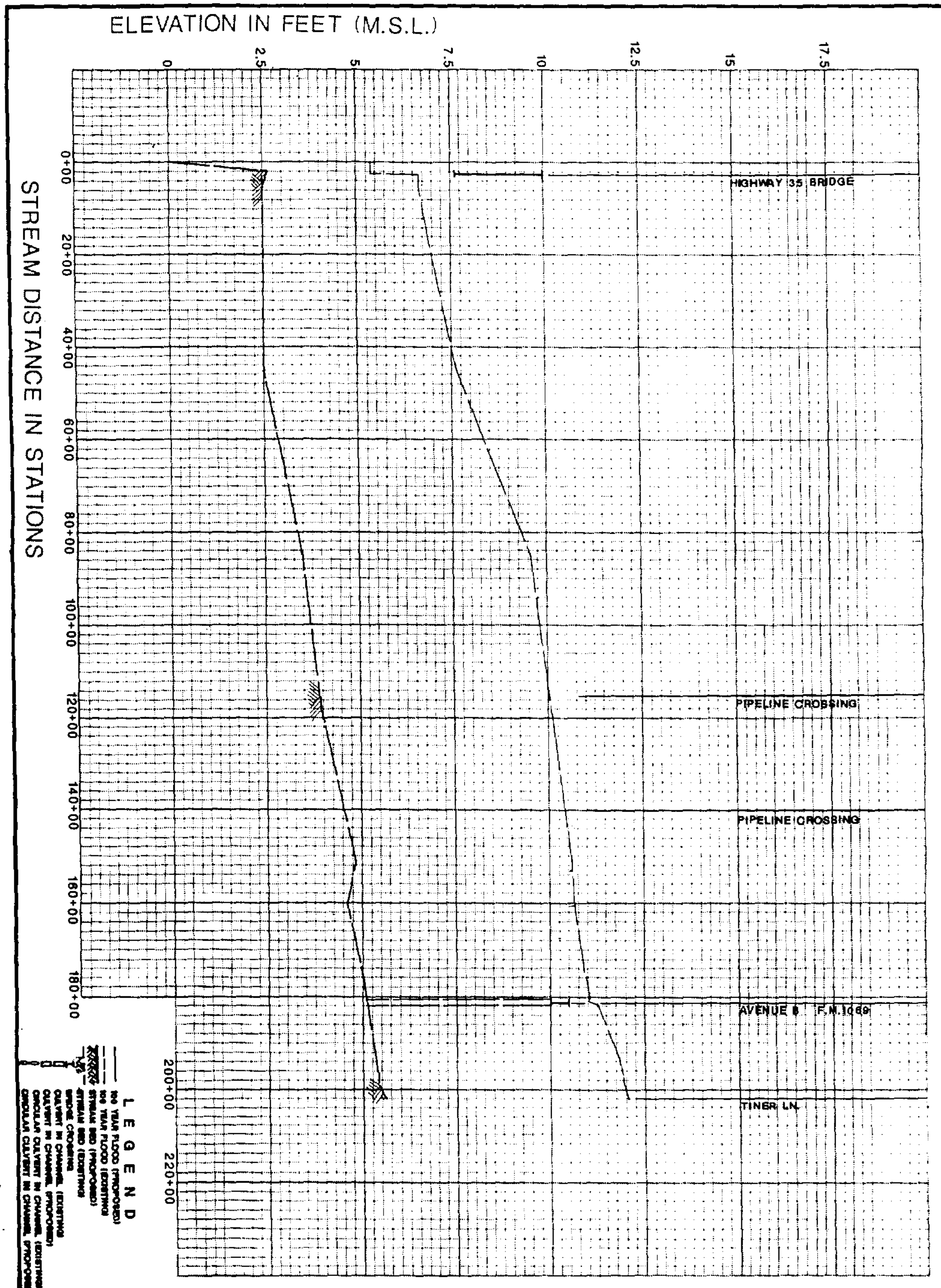
PAGE 33	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	FLOOD PROFILE GREEN LAKE TRIBUTARY (BG - 03)
	NAISMITH ENGINEERS, INC. Consulting Engineers	GREGORY/PORTLAND AREA



FLOOD CONTROL STUDY **SAN PATRICIO COUNTY** SAN PATRICIO COUNTY DRAINAGE DISTRICT	**FLOOD PROFILE** **KINNEY BAYOU (MAIN BN)**
NAISMITH ENGINEERS, INC. Consulting Engineers	**CITY OF INGLESIDE**



FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	FLOOD PROFILE KINNEY BAYOU (MAIN BN)
NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF INGLESIDE

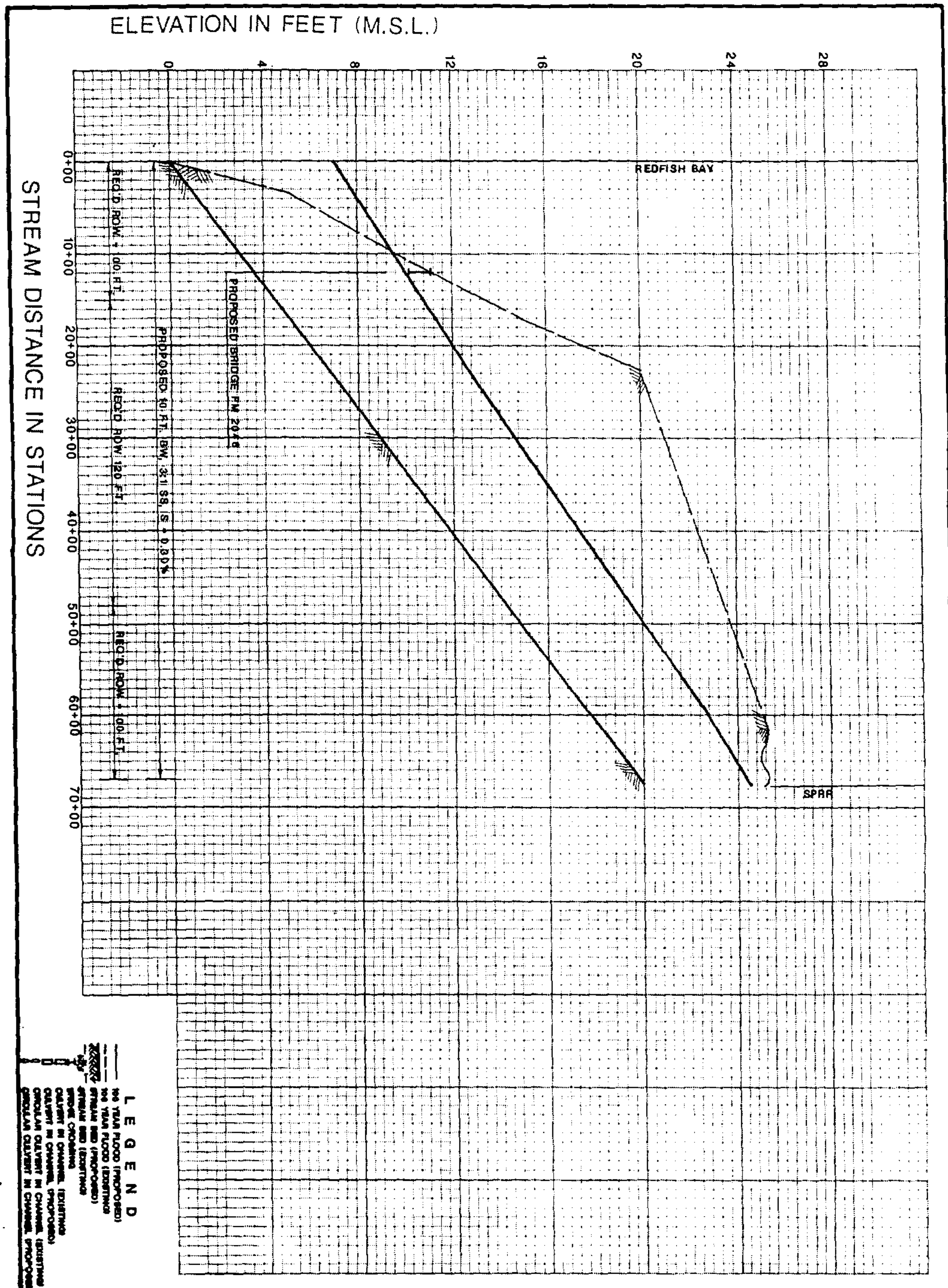


FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

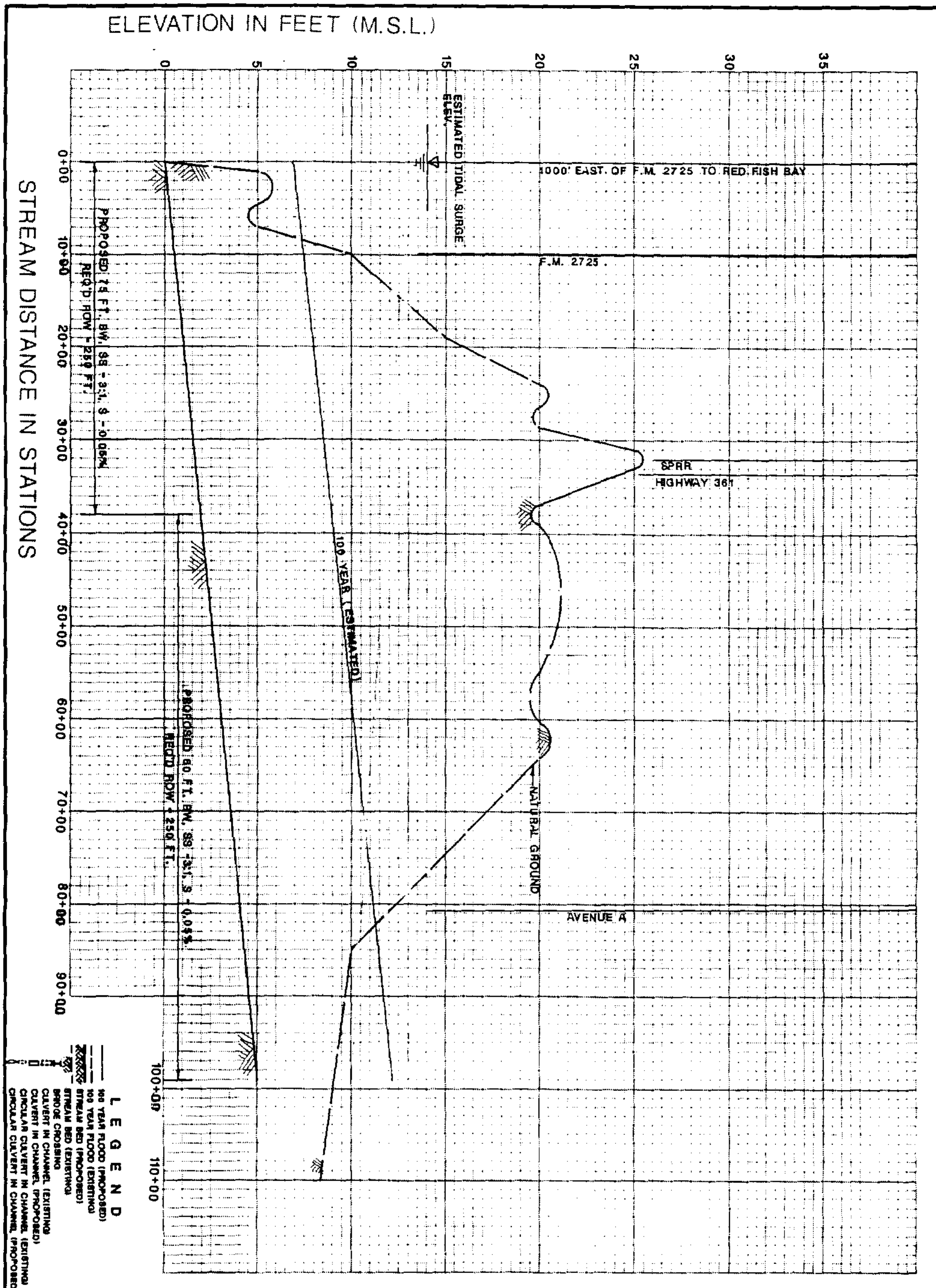
NAISMITH ENGINEERS, INC.
Consulting Engineers

FLOOD PROFILE
McCAMPBELL SLOUGH (MAIN BJ)

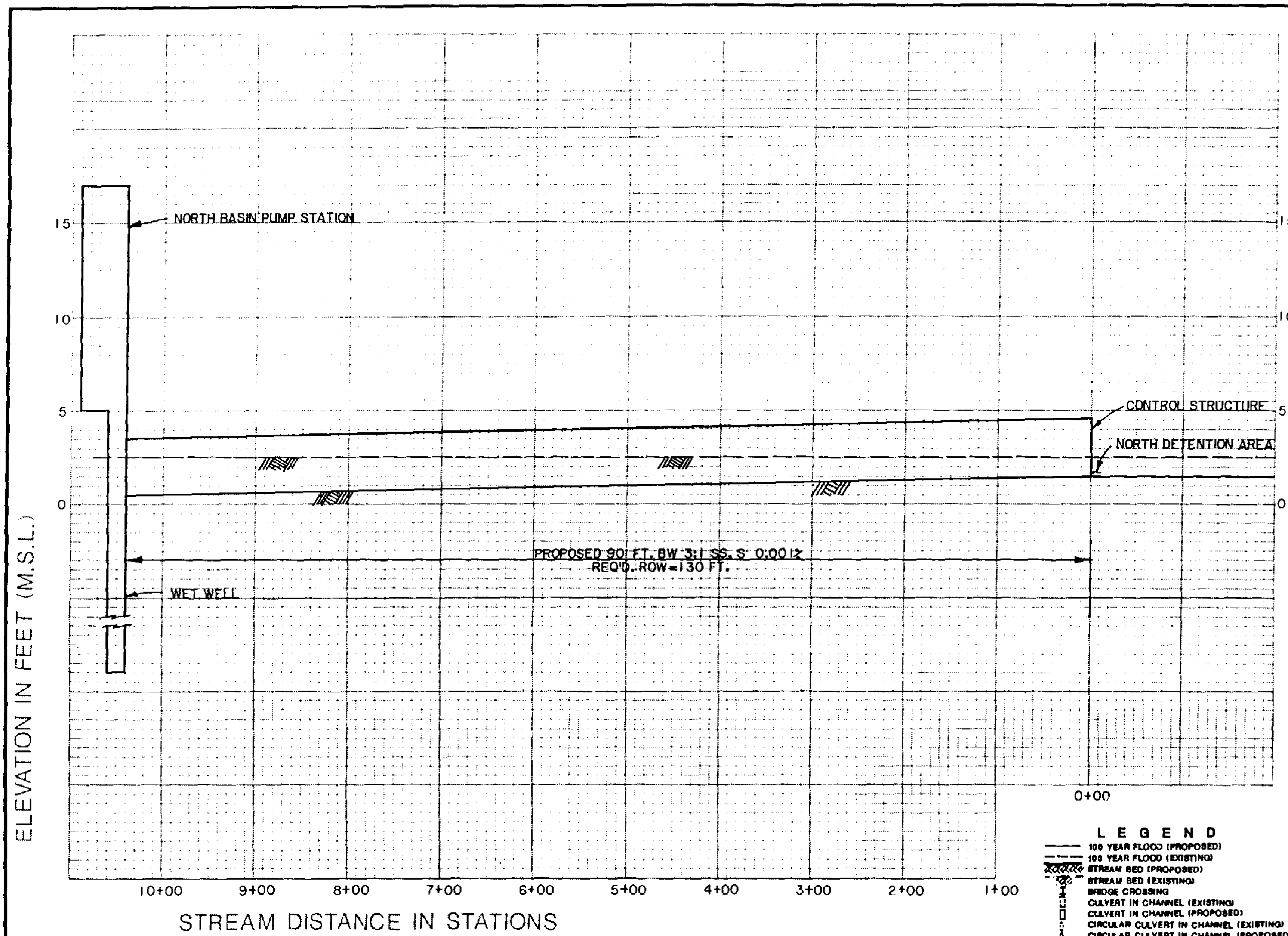
CITY OF INGLESIDE



PAGE 37	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	FLOOD PROFILE PROPOSED CHANNEL INGLESIDE TO REDFISH BAY SOUTH OF F.M. 361
	NAISMITH ENGINEERS, INC. Consulting Engineers	CITY OF INGLESIDE



PAGE 38	FLOOD CONTROL STUDY SAN PATRICIO COUNTY SAN PATRICIO COUNTY DRAINAGE DISTRICT	PROPOSED DITCH McCAMPBELL SLOUGH TO REDFISH BAY
	NAISMITH ENGINEERS, INC. Consulting Engineers HDR Infrastructure, Inc. A Centerra Company	CITY OF INGLESIDE



FLOOD PROFILE

NORTH BASIN OUTLET CHANNEL

ARANSAS PASS AREA

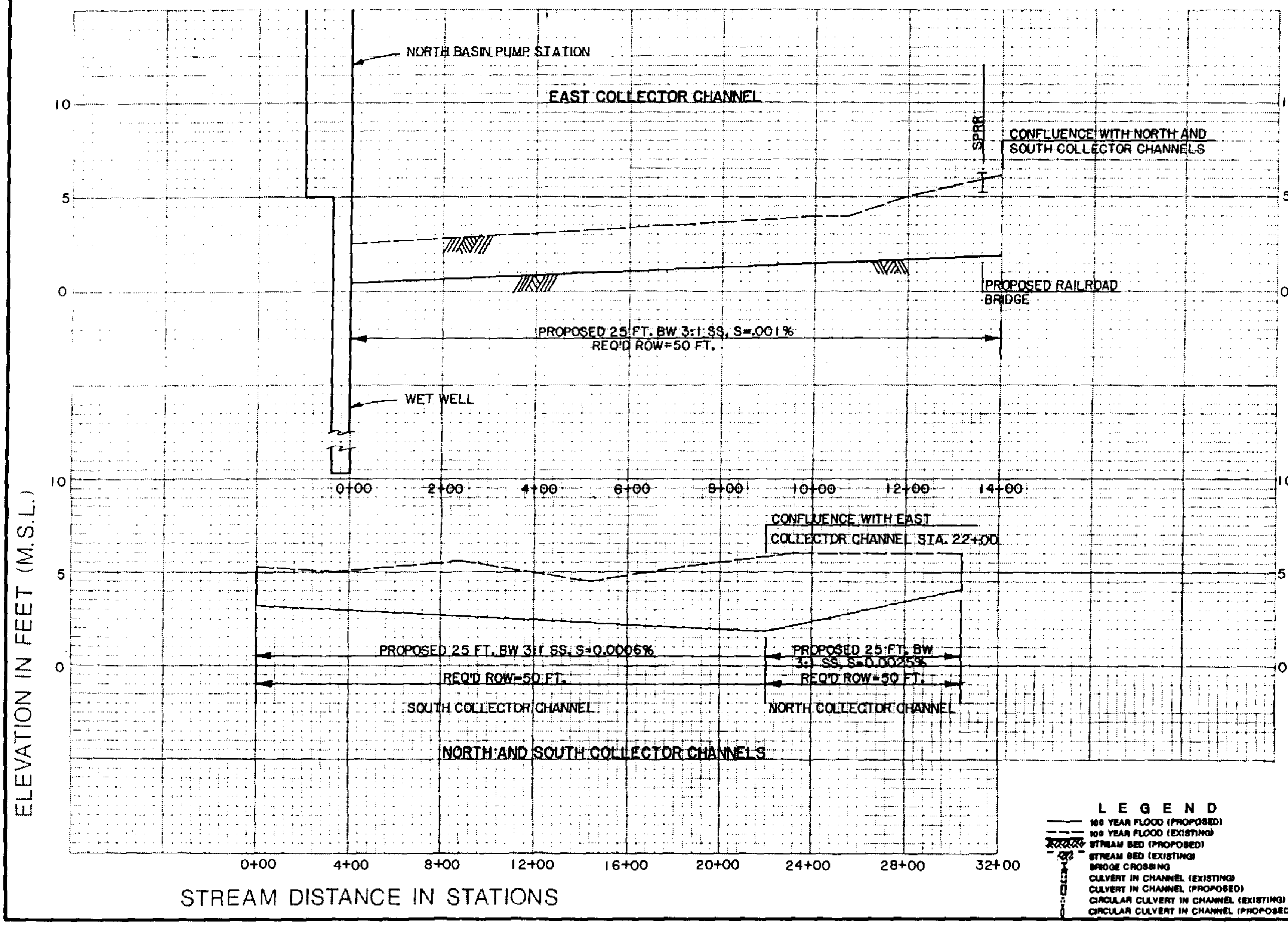
FLOOD CONTROL STUDY

SAN PATRICIO COUNTY

SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

PAGE 39



ARANSAS PASS AREA

NORTH BASIN COLLECTOR CHANNELS

FLOOD CONTROL STUDY
SAN PATRICIO COUNTY
SAN PATRICIO COUNTY DRAINAGE DISTRICT

NAISMITH ENGINEERS, INC.
Consulting Engineers

APPENDIX B

ALTERNATIVE COST ESTIMATES

Cost Estimate

Mathis Area - Sixmile Creek (BU-00) and Extension

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	256,750	yd^3	1.25	320,940
2)	Vegetation Establishment	45.77	acre	1,200	54,920
3)	Spoil Disposal	6.4	acre	2,000	12,800
4)	FM 1068 Culverts	139.89	yd^3	350.00	48,960
5)	FM 1068 Road Replacement	357	yd^2	40.00	14,280
6)	CR 4 Culverts	52.49	yd^3	350.00	18,370
7)	CR 4 Road Replacement	156	yd^2	40.00	6,240
8)	Right-of-Way Potential Development	45.77	acre	2,000	91,540

Subtotal Basic Construction	$	568,050
10% Contingencies		56,810
Total Construction	$	624,860
15% Eng., Legal, Admin., & Finance		93,730
TOTAL COST	$	718,590

Cost Estimate

Mathis Area - Proposed Sixmile Creek Tributaries (BU-01 & BU-02)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	115,630	yd^3	1.25	144,540
2)	Spoil Disposal	2.5	acre	2,000	5,000
3)	Vegetation Establishment	33.81	acre	1,200	40,570
4)	CR 14 Culverts	33.07	yd^3	350.00	11,580
5)	CR 14 Road Replacement	116	yd^2	40.00	4,640
6)	FM 666 Culverts	36.65	yd^3	350.00	12,830
7)	FM 666 Road Replacement	166	yd^2	40.00	6,640
8)	Right-of-Way Potential Development	33.81	acre	2,000	67,620

Subtotal Basic Construction	$	293,420
10% Contingencies		29,340
Total Construction	$	322,760
15% Eng., Legal, Admin., & Finance		48,410
TOTAL COST	$	371,170

Cost Estimate

Sinton Area - Chiltipin Creek (AZ-00)

		Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	7,500,000	yd^3	1.00	7,500,000
2)	Clear & Grub	1,005	acre	350.00	351,750
3)	Pipeline Crossing	6	EA	100,000	600,000
4)	Modify Mopac Bridge	1	LS	90,000	90,000
5)	Modify US 77 Bridge	1	LS	90,000	90,000
6)	Rebuild Park Road Bridge	10,560	ft^2	50.00	528,000
7)	Rebuild SPRR Trestle	250	LF	1,000	250,000
8)	Rebuild Hwy US Bridge	10,032	ft^2	50.00	501,600
9)	Modify US 77 Bypass	1	LS	90,000	90,000
10)	Rebuild Hwy 881 Bridge	6,600	ft^2	50.00	330,000
11)	Rebuild CR 36A Bridge	5,200	ft^2	40.00	208,000
12)	Rebuild CR 36 LWC	5,200	ft^2	40.00	208,000
13)	Remove LWC on Chiltipin Creek	1	LS	10,000	10,000
14)	Miscellaneous Utility Adjustments	1	LS	50,000	50,000
15)	Right-Of-Way				
	Undeveloped	875	acre	1,000	875,000
	Potential Development	130	acre	2,000	260,000
				Subtotal Basic Construction Cost	11,942,350
				10% Contingencies	1,194,240
				Total Construction Cost	13,136,590
				15% Legal, Admin., Engr., & Finance	1,970,490
				TOTAL COST	$15,107,080

Cost Estimate

Sinton Area - Chiltipin Creek (AZ-00)
Channel Rectification

		Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	7,350,000	yd^3	1.00	7,350,000
2)	Clear & Grub	1,005	acre	350.00	351,750
3)	Pipeline Crossings	6	EA	100,000	600,000
4)	Modify Mopac Bridge	1	LS	90,000	90,000
5)	Modify US 77 Bridge	1	LS	90,000	90,000
6)	Rebuild Park Road Bridge	10,560	ft^2	50.00	528,000
7)	Rebuild SPRR Trestle	250	LF	1,000	250,000
8)	Rebuild US 181 Bridge	10,032	ft^2	50.00	501,600
9)	Modify US 77 Bypass	1	LS	90,000	90,000
10)	Rebuild Hwy 881 Bridge	6,600	ft^2	50.00	330,000
11)	Rebuild CR 36A Bridge	5,200	ft^2	40.00	208,000
12)	Rebuild CR 36 LWC	5,200	ft^2	40.00	208,000
13)	Remove LWC on Chiltipin Creek	1	LS	10,000	10,000
14)	Miscellaneous Utility Adjustments	1	LS	50,000	50,000
15)	Right-Of-Way				
	Undeveloped	875	acre	1,000	875,000
	Potential Development	130	acre	2,000	260,000
	Subtotal Basic Construction Cost				11,792,350
	10% Contingencies				1,179,240
	Total Construction Cost				12,971,590
	15% Legal, Admin., Engr., & Finance				1,945,740
	TOTAL COST				$14,917,330

Cost Estimate

Sinton Area - Chiltipin Creek
Upsteam Detention Alternative

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Embankment	170,000	yd^3	3.00	510,000
2)	Spillway Structure	341	yd^3	350.00	119,350
3)	Vegetation Establishment	50	acre	1,200	60,000
4)	Land Purchase	50	acre	1,000	50,000
5)	Easement Purchase	1,250	acre	750.00	937,500
6)	Relocations	15	EA	75,000	1,125,000

Subtotal Basic Construction	$ 2,801,850
10% Contingencies	280,190
Total Construction	$ 3,082,040
15% Eng., Legal, Admin., & Finance	462,310
TOTAL COST	$ 3,544,350

Cost Estimate

Sinton Area - South Ditch

		Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	3,500,000	yd^3	1.00	3,500,000
2)	Clear & Grub	52	acre	350.00	18,200
3)	Pipeline Crossings	2	EA	100,000	200,000
4)	Rebuild US 881 Bridge	6,300	ft^2	50.00	315,000
5)	Rebuild CR 63 Bridge	3,900	ft^2	40.00	156,000
6)	Rebuild Teltshik LWC	4,800	ft^2	4.00	19,200
7)	Modify US 181 Bridge	1	LS	100,000	100,000
8)	Remove Old Kings Hwy Bridge	1	LS	10,000	10,000
9)	Rebuild SPRR Trestle	200	LF	1,000	200,000
10)	Replace CR 61 Bridge	3,900	ft^2	40.00	156,000
11)	Replace FM 2046 Bridge	6,600	ft^2	50.00	330,000
12)	Miscellaneous Utility Adjustments	1	LS	50,000	50,000
13)	Right-Of-Way				
	Undeveloped	229	acre	1,000	229,000
	Potential Development	76	acre	2,000	152,000
	Developed	30	acre	4,000	120,000
	Subtotal Basic Construction Cost				5,555,400
	10% Contingencies				555,540
	Total Construction Cost				6,110,940
	15% Legal, Admin., Engr., & Finance				916,670
	TOTAL COST				$ 7,027,570

Cost Estimate

Odem Area - Peters Swale

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	490,000	yd^3	1.25	612,500
2)	Vegetation Establishment and Clear & Grub	72	acre	1,500	108,000
3)	Pipeline Relocation (Water Lines, Sanitary Sewer)	1	LS	50,000	50,000
4)	Rebuild CR 42 Bridge	5,720	ft^2	40.00	228,800
5)	Rebuild MPRR Trestle	220	LF	1,000	220,000
6)	Rebuild US 77	15,840	ft^2	50.00	792,000
7)	Right-of-Way Potential Development	72	acre	2,000	144,000
				Subtotal Basic Construction	$ 2,155,300
				10% Contingencies	215,300
				Total Construction	$ 2,370,830
				15% Eng., Legal, Admin., & Finance	355,620
				TOTAL COST	$ 2,726,450*

* NOTE: If Items 5 and 6 paid by others, the Total Cost is reduced to $ 1,446,270.

Cost Estimate

Odem Area - Diversion Channel Alternative
Diversion Channel Component

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	1,800,000	yd^3	1.25	2,250,000
2)	Clear & Grub	45	acre	350	15,750
3)	Vegetation Establishment	140	acre	1,500	210,000
4)	CR 51 Bridge Structure	3,510	ft^2	40	140,400
5)	Hwy 631 Bridge Structure	8,970	ft^2	50	448,500
6)	CR 64 Bridge Structure	5,070	ft^2	40	202,800
7)	Low Water Crossing	1	LS	75,000	75,000
8)	Outfall Structure	1	LS	100,000	100,000
9)	Right-of-Way	150	acre	1,500	225,000
				Subtotal Basic Construction	$ 3,667,450
				10% Contingencies	366,750
				Total Construction	$ 4,034,200
				15% Eng., Legal, Admin., & Finance	605,130
				TOTAL COST	$ 4,639,320

Peters Swale Component

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	430,000	yd^3	1.25	537,500
2)	Vegetation Establishment	64	acre	1,500	96,000
3)	Pipeline Relocation (Water & Sewer Lines)	1	LS	50,000	50,000
4)	Rebuild MPRR Trestle	220	LF	1,000	220,000
5)	Rebuild US 77 Bridge	15,840	ft^2	50	792,000
6)	Right-of-Way	64	acre	2,000	128,000
				Subtotal Basic Construction	$ 1,823,500
				10% Contingencies	182,350
				Total Construction	$ 2,005,850
				15% Eng., Legal, Admin., & Finance	300,880
				TOTAL COST	$ 2,306,730
				Total 1	$ 4,639,330
				Total 2	+ 2,306,730
				GRAND TOTAL	$ 6,946,060

Cost Estimate

Taft Area - Main AJ

		Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	730,000	yd^3	1.25	912,500
2)	Vegetation Establishment	100	acre	1,500	150,000
3)	Rebuild CR 100 Bridge	2,600	ft^2	40.00	104,000
4)	Rebuild Pyron Bridge	1,600	ft^2	40.00	64,000
5)	Rebuild CR 71 Bridge	2,600	ft^2	40.00	104,000
6)	Rebuild US 181 Eastbound	4,400	ft^2	50.00	220,000
7)	Rebuild US 181 Westbound	4,400	ft^2	50.00	220,000
8)	Rebuild SPRR Trestle	100	LF	1,000	100,000
9)	Rebuild Verbina Bridge	1,950	ft^2	40.00	78,000
10)	Pipeline Relocation	1	EA	100,000	100,000
11)	Miscellaneous Utility Adjustments	1	LS	50,000	50,000
12)	Right-Of-Way				
	Undeveloped	67.50	acre	1,000	67,500
	Potential Development	50.50	acre	2,000	101,000
	Subtotal Basic Construction				$ 2,271,000
	10% Contingencies				227,100
	Total Construction Cost				2,498,100
	15% Legal, Admin., Engr., & Finance				374,720
	TOTAL COST				$ 2,872,820

Cost Estimate

Taft Area - Main AN

		Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	900,500	yd^3	1.25	1,125,630
2)	Pipeline Relocations	2	EA	100,000	200,000
3)	Rebuild D. Miller Bridge	2,000	ft^2	40.00	80,000
4)	Rebuild CR 77 Bridge	3,250	ft^2	40.00	130,000
5)	Rebuild CR 102 Bridge	3,250	ft^2	40.00	130,000
6)	Rebuild Hwy 631 Bridge	4,620	ft^2	50.00	231,000
7)	Right-Of-Way				
	Undeveloped	68.50	acre	1,000	68,500
	Potential Development	55.00	acre	2,000	110,000
	Subtotal Basic Construction				$2,075,130
	10% Contingencies				207,510
	Total Construction Cost				2,282,640
	15% Legal, Admin., Engr., & Finance				342,400
	TOTAL COST				$ 2,625,040

Cost Estimate

Taft Area - AN-02

		Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	350,000	yd^3	1.25	437,500
2)	Vegetation Establishment	75	acre	1,500	112,500
3)	Rebuild CR 102 Bridge	2,600	ft^2	40.00	104,000
4)	Rebuild West US 181 Bridge	4,400	ft^2	50.00	220,000
5)	Rebuild East US 181 Bridge	4,400	ft^2	50.00	220,000
6)	Rebuild SPRR Trestle	100	ft^2	1,000	100,000
7)	Right-Of-Way				
	Undeveloped	65.00	acre	1,000	65,000
	Potential Development	22.50	acre	2,000	45,000
			Subtotal Basic Construction		$1,304,000
			10% Contingencies		130,400
			Total Construction Cost		1,434,400
			15% Legal, Admin., Engr., & Finance		215,160
			TOTAL COST		$ 1,649,560

Cost Estimate

Portland Area - Gum Hollow (BF-00)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	565,760	yd^3	1.25	707,200
2)	Vegetation Establishment	83.43	acre	1,200	100,120
3)	Spoil Disposal Areas	12.2	acre	1,000	12,200
4)	CR 72 Bridge	3,380	ft^2	40.00	135,200
5)	CR 72 Slope Paving	1,092	ft^2	4.00	4,370
6)	CR 76 Concrete Lining	42.40	yd^3	350.00	14,840
7)	Modify Drop Structure Near CR 72				
	Concrete - Removal	26	yd^3	200.00	5,200
	Concrete - Additional	87	yd^3	350.00	30,450
8)	Drop Structure Removal Near CR 76	99	yd^3	200.00	19,800
9)	Right-of-Way - Undeveloped	83.43	acre	1,000	83,430
10)	Pipeline Crossings	3	LS	100,000	300,000
				Subtotal Basic Construction	$ 1,412,810
				10% Contingencies	141,280
				Total Construction	$ 1,554,090
				15% Eng., Legal, Admin., & Finance	233,110
				TOTAL COST	$ 1,787,200

Cost Estimate

Portland Area - Proposed Gum Hollow Tributary (BF-03)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	381,000	yd^3	1.25	476,250
2)	Vegetation Establishment	57.23	acre	1,200	68,680
3)	Spoil Disposal	8.3	acre	1,000	8,300
4)	CR 79 Culverts	65.0	yd^3	350	22,750
5)	CR 79 Road Replacement	153.00	yd^2	40.00	6,120
6)	Right-Of-Way Undeveloped	57.23	acre	1,000	57,230
7)	Pipeline Crossings	1	LS	100,000	100,000

Subtotal Basic Construction	$	739,330
10% Contingencies		73,930
Total Construction	$	813,260
15% Eng., Legal, Admin., & Finance		121,990
TOTAL COST	$	935,250

Cost Estimate

Portland Area - Proposed Airport Ditch

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	44,090	yd^3	1.25	55,110
2)	Vegetation Establishment	12.83	acre	1,200	15,400
3)	Spoil Disposal	1.1	acre	2,000	2,200
4)	FM 893 Culverts	48.44	yd^3	350.00	16,950
5)	FM 893 Road Replacement	166	yd^2	40.00	6,640
6)	Lang Road Culverts	19.73	yd^3	350.00	6,910
7)	Lang Rd. Road Replacement	92	yd^2	40.00	3,680
8)	Right-Of-Way Potential Development	12.83	acre	2,000	25,660

Subtotal Basic Construction	$	132,550
10% Contingencies		13,260
Total Construction	$	145,810
15% Eng., Legal, Admin., & Finance		21,870
TOTAL COST	$	167,680

Cost Estimate

Portland Area - Proposed Doyle Addition Ditch Improvement & Extension (BT-00)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	138,450	yd^3	1.25	173,060
2)	Vegetation Establishment	23.57	acre	1,200	28,280
3)	Spoil Disposal	3.5	acre	2,000	7,000
4)	Concrete Channel Lining	28,380	ft^2	4.00	113,520
5)	FM 893 Bridge	1,760	ft^2	50.00	88,000
6)	Lang Road Culverts	28.24	yd^3	350.00	9,880
7)	Lang Rd. Road Replacement	104	yd^2	40.00	4,160
8)	CR 81 Culverts	28.24	yd^3	350.00	9,880
9)	CR 81 Road Replacement	104	yd^2	40.00	4,160
10)	Right-Of-Way				
	Developed	5.21	acre	4,000	20,840
	Potential Development	20.13	acre	2,000	40,260

Subtotal Basic Construction	$	499,040
10% Contingencies		49,900
Total Construction	$	548,940
15% Eng., Legal, Admin., & Finance		82,340
TOTAL COST	$	631,280

Cost Estimate

Gregory Area - Proposed Northeast Ditch (AS-00)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	946,800	yd^3	1.25	1,183,500
2)	Vegetation Establishment	133	acre	1,200	159,600
3)	Spoil Disposal	20.4	acre	1,000	20,400
4)	McCampbell Rd. Bridge	3,380	ft^2	40.00	135,200
5)	Richardson Rd. Culverts	80.2	yd^3	350.00	28,070
6)	Richardson Road Road Replacement	185	yd^2	40.00	7,400
7)	McKamey Rd. Culverts	65.4	yd^3	350.00	22,890
8)	McKamey Road Road Replacement	159	yd^2	40.00	6,360
9)	Pipeline Crossings	4	LS	100,000	400,000
10)	Clearing & Grubbing	15.8	acre	350.00	5,530
8)	Right-Of-Way Undeveloped	133	acre	1,000	133,000
				Subtotal Basic Construction	$ 2,101,950
				10% Contingencies	210,200
				Total Construction	$ 2,312,150
				15% Eng., Legal, Admin., & Finance	346,820
				TOTAL COST	$ 2,658,970

Cost Estimate

Portland/Gregory Area - Green Lake Improvements (BG-00)

ALTERNATIVE 1

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	759,170	yd^3	1.25	948,960
2)	Vegetation Establishment	91.57	acre	1,200	109,880
3)	Spoil Disposal	16.0	acre	2,000	32,000
4)	Green Lake Drop Structure Outfall	1	LS	200,000	200,000
5)	SPRR Bridge	110	LF	1,000	110,000
6)	US 181 Bridges (North and South Bound)	9,680	ft^2	50.00	484,000
7)	Pipeline Crossings	3	LS	100,000	300,000
8)	Clearing & Grubbing				
9)	Right-Of-Way Potential Development	103.05	acre	2,000	206,100
10)	Concrete Slope Paving	8,485	ft^2	4.00	33,940

Subtotal Basic Construction	$ 2,424,880
10% Contingencies	242,490
Total Construction	$ 2,667,370
15% Eng., Legal, Admin., & Finance	400,110
TOTAL COST	$ 3,067,480

Cost Estimate

Portland/Gregory Area - Green Lake Improvements (BG-00)

ALTERNATIVE 2

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	808,020	yd^3	1.25	1,010,030
2)	Vegetation Establishment	60.07	acre	1,200	72,080
3)	Spoil Disposal	17.4	acre	2,000	34,800
4)	Green Lake Structure Improvements (NEI)	1	LS	200,000	200,000
5)	SPRR Bridge	100	LF	1,000	100,000
6)	US 181 Bridges (North and South Bound)	8,800	ft^2	50.00	440,000
7)	Pipeline Crossings	3	LS	100,000	300,000
8)	Clearing & Grubbing				
9)	Right-Of-Way Potential Development	106.38	acre	2,000	212,760
10)	Concrete Slope Paving	8,485	ft^2	4.00	33,940
				Subtotal Basic Construction	$ 2,403,610
				10% Contingencies	240,360
				Total Construction	$ 2,643,970
				15% Eng., Legal, Admin., & Finance	396,600
				TOTAL COST	$ 3,040,570

Cost Estimate

Portland/Gregory Area - Green Lake Diversion Channel

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	478,460	yd^3	1.25	598,080
2)	Vegetation Establishment	61.03	acre	1,200	73,240
3)	Spoil Disposal	11.4	acre	2,000	22,800
4)	Free Overfall Drop Structure & Splash Pad	360	yd^3	350.00	126,000
5)	Concrete Lined Channel Trans.				
	Bottom	37.1	yd^3	350.00	12,990
	Side Slopes	2,320	ft^2	4.00	9,280
6)	Concrete Lined Diversion Sect.				
	Bottom	64.8	yd^3	350.00	22,680
	Side Slopes	1,900	ft^2	4.00	7,600
7)	Pipeline Crossings	2	LS	100,000	200,000
8)	Right-Of-Way Potential Development	61.03	acre	2,000	122,060
9)	County Road Bridge	3,380	ft^2	40.00	135,200

Subtotal Basic Construction	$ 1,329,930
10% Contingencies	132,990
Total Construction	$ 1,462,920
15% Eng., Legal, Admin., & Finance	219,440
TOTAL COST	$ 1,682,360

Cost Estimate

Portland/Gregory Area - Green Lake Channel Extension (BG-00)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	168,520	yd^3	1.25	210,650
2)	Vegetation Establishment	31.22	acre	1,200	37,460
3)	Spoil Disposal	3.8	acre	2,000	7,600
4)	FM 2986 Culverts				
	3-8' x 8' Boxes	89.1	yd^3	350.00	31,190
	Road Replacement	230	yd^2	40.00	9,200
5)	FM 2986 Culverts				
	3-7' x 7' Boxes	77.1	yd^3	350.00	26,990
	Road Replacement	239	yd^2	40.00	9,560
6)	CR 72 Culverts				
	2-6' x 6' Boxes	23.4	yd^3	350.00	8,190
	Road Replacement	98	yd^2	40.00	3,920
7)	Pipeline Crossing	1	LS	100,000	100,000
8)	Clearing & Grubbing	5.5	acre	350.00	1,930
8)	Right-Of-Way				
	Potential Development	31.22	acre	2,000	62,440

Subtotal Basic Construction	$	509,130
10% Contingencies		50,910
Total Construction	$	560,040
15% Eng., Legal, Admin., & Finance		84,010
TOTAL COST	$	644,050

Cost Estimate

Portland/Gregory Area - Green Lake Tributary (BG-02)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	43,200	yd^3	1.25	54,000
2)	Vegetation Establishment	10.45	acre	1,200	12,540
3)	Spoil Disposal	1.2	acre	2,000	2,400
4)	Pipeline Crossing	1	LS	100,000	100,000
5)	Right-Of-Way Potential Development	10.45	acre	2,000	20,900
				Subtotal Basic Construction	$ 189,840
				10% Contingencies	18,980
				Total Construction	$ 208,820
				15% Eng., Legal, Admin., & Finance	31,320
				TOTAL COST	$ 240,140

Cost Estimate

Portland/Gregory Area - Green Lake Tributary (BG-03)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	13,040	yd^3	1.25	16,300
2)	Vegetation Establishment	5.14	acre	1,200	6,170
3)	Spoil Disposal	0.5	acre	2,000	1,000
4)	CR 78A Bridge	1,820	ft^2	40.00	72,800
5)	Right-Of-Way Potential Development	5.14	acre	2,000	10,280
6)	Clearing & Grubbing	5.14	acre	350.00	1,800

Subtotal Basic Construction	$	108,350
10% Contingencies		10,840
Total Construction	$	119,190
15% Eng., Legal, Admin., & Finance		17,880
TOTAL COST	$	137,070

Cost Estimate

Ingleside Area - Kinney Bayou (Main BN)

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Excavation & Disposal	165,000	yd^3	1.50	247,500
2)	Vegetation Establishment	45	acre	1,500	67,500
3)	Modify Pipeline Location	1	LS	50,000	50,000
4)	Provide Additional Capacity 8th street	1	LS	20,000	20,000
5)	Remove Existing 8 x 10 Box	1	LS	10,000	10,000
6)	Concrete Ditch Lining Including Excavation & Disposal	60,000	ft^2	4.00	240,000
7)	Provide Additional Capacity 1069	1	LS	35,000	35,000
8)	Provide Additional Capacity Live Oak Street	1	LS	20,000	20,000
9)	Provide Additional Capacity 4th Street	1	LS	20,000	20,000
10)	Right-of-Way Required	56	acre	1,500	84,000
11)	Miscellaneous Utility Adjustments	1	LS	20,000	20,000

Subtotal Basic Construction	$ 814,000
10% Contingencies	81,400
Total Construction	$ 895,400
15% Eng., Legal, Admin., & Finance	134,310
TOTAL COST	$ 1,029,710

Cost Estimate

Ingleside Area - Diversion Channel
Ingleside to Redfish Bay

	Item	Units	Quantity	Cost/Unit	Cost ($)
1)	Excavation & Disposal	75,000	yd^3	2.00	150,000
2)	Clear & Grub	20	acre	350.00	7,000
3)	Vegetation Establishment	50	acre	1,500	30,000
4)	Bridge Structure @ FM 2725	3,220	ft^2	40.00	128,800
5)	Miscellaneous Utility Adjustments	1	LS	50,000	50,000
6)	Right-of-Way	20	acre	4,000	80,000

Subtotal Basic Construction	$	445,800
10% Contingencies		44,580
Total Construction	$	490,380
15% Eng., Legal, Admin., & Finance		73,560
TOTAL COST	$	563,940

Cost Estimate

Ingleside Area - Diversion Channel
McCampbell Slough to Redfish Bay

	Item	Units	Quantity	Cost/Unit	Cost ($)
1)	Excavation & Disposal	450,000	yd^3	1.50	675,000
2)	Clear & Grub	50	acre	350.00	17,500
3)	Vegetation Establishment	50	acre	1,200	60,000
4)	Avenue A Bridge	3,680	ft^2	40.00	147,200
5)	Highway 361 Bridge	5,470	ft^2	50.00	273,500
6)	SPRR	125	LF	1,000	125,000
7)	FM 2725 Bridge	5,290	ft^2	40.00	211,600
8)	Miscellaneous Utility Adjustments	1	LS	75,000	75,000
9)	Right-of-Way	50	acre	4,000	200,000

Subtotal Basic Construction	$	1,784,800
10% Contingencies		178,480
Total Construction	$	1,963,280
15% Eng., Legal, Admin., & Finance		294,490
TOTAL COST	$	2,257,770

Cost Estimate

Aransas Pass Area - North Basin Stormwater Pump Station

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Pump Station (~ 1060 cfs)	1	LS	1,550,000	1,550,000
2)	Channel Excavation & Disposal	37,100	yd^3	1.25	46,380
3)	Pond Excavation & Disposal	187,150	yd^3	1.25	233,940
4)	Right-of Way	36.70	acre	4,000	146,800
5)	Culverts (Installed)	5	LF	23,750	118,750
6)	Bridge at Railroad	1	LS	45,000	45,000

Subtotal Basic Construction	$	2,140,870
10% Contingencies		214,090
Total Construction	$	2,354,960
15% Eng., Legal, Admin., & Finance		353,240
TOTAL COST	$	2,708,200

Cost Estimate

Aransas Pass Area - South Basin Stormwater Pump Station

	Item	Quantity	Units	Cost/Unit	Cost ($)
1)	Pump Station	1	LS	1,300,000	1,300,000
2)	Culverts (Installed)	8	LF	48,750	390,000

Subtotal Basic Construction	$	1,690,000
10% Contingencies		169,000
Total Construction	$	1,859,000
15% Eng., Legal, Admin., & Finance		278,850
TOTAL COST	$	2,137,850

APPENDIX C

SUMMARY OF PUBLIC INFORMATION AND WORKSHOP MEETINGS

RECORD OF PUBLIC INFORMATION AND WORKSHOP MEETINGS

City of Mathis
Date: June 17, 1986
Public Information Meeting - City Council Meeting

Mr. Steve Elliott (San Patricio County Drainage District) and John Michael (Naismith Engineers, Inc.) introduced the scope and objectives of the flood control study to the City Council with copies of the project completion schedule and detailed budget estimate. It was explained that this was a study outlining major flood control improvements to relieve flooding in present or potential growth areas. Mr. Elliott and Mr. Michael presented their understanding of some of the community drainage problems and asked the Council and public for their comments. The Mayor of the City was not present at the meeting, however, several Council Members and representatives from the public voiced their concerns relating to areas where house flooding has occurred. One representative from the public had photographs where water stood for days in the street. It was explained that this study was not aimed at solving small, localized street drainage problems, but rather investigating major outfall structures that may relieve some of the other problems existing due to a lack of drainage. It was requested that a workshop meeting be set up with the City Staff and other interested persons to discuss in detail the City of Mathis drainage concerns and future growth needs.

City of Mathis
Date: August 18, 1986
Workshop Meeting
Present were Mayor Knight, Public Works Director, Steve Elliott(SPCDD), John Michael (NEI).

Mayor Knight presented a city street map and a copy of the 20 year comprehensive plan for the City of Mathis prepared in 1971. Included in the plan is an overall drainage plan and 5' contour map of the City and adjacent area. As discussed, most of the city drainage is surface type, with streets carrying the majority of the runoff to very poorly defined outfalls. The City is situated on a ridge with very little runoff coming into the city from adjacent areas. It was discussed that the major drainage problems exist due to the lack of any major outfall ditches and drainage easements, specifically in the west and south sections of the city. Mayor Knight indicated that the City's first priority should be to relieve the area west of Aransas Street and south of San Patricio Avenue with a major drainage ditch which would eventually outfall across State Highway 1068 to Sixmile Creek. Drainage easements would need to be acquired. The Mayor indicated that the area near Texas Street, located on the southeast side of town, was constructed in a low area and that a lift station was constructed to relieve storm runoff. It was suggested that an outfall ditch be completed to the south, extending to Sixmile Creek. Easements would be required. The Mayor indicated that the area between I.H. 37 and FM 3024 would be the next area for future development and that the study consider this area in the analysis.

San Patricio County Commissioner's Court
Date: June 23, 1986
Public Information Meeting: Present were the County Judge, Commissioners, Steve Elliott (SPCDD), L. J. Luedke (SPCDD), Pete Manz (HDR), J. P. Naismith (NEI), Sam Vaugh (HDR), and John Michael (NEI).

Mr. Elliott, San Patricio County Drainage District Manager, introduced the scope and objectives of the study to the Court and introduced the members of the study team. Mr. Elliott gave the Commissioners a copy of the project completion schedule and budget estimate. John Michael discussed the steps involved in the project completion and community meetings in progress. Pete Manz addressed the Court concerning the type of data which was needed from each County Precinct, and discussed several of the project deliverables. Public input and questions were solicited and workshop meetings with the Commissioners were scheduled.

San Patricio County Commissioner's Court
Date: July 1, 1986
Workshop Meetings: Present were San Patricio County Commissioners at the San Patricio County Drainage District Office, Mr. Joe Zapata (Precinct 1), Mr. Carl Duncan (Precinct 2), Mrs. Hazel Edwards (Precinct 4), Pete Manz (HDR), Bob Wear (Texas Water Development Board), Sam Vaugh (HDR), J. P. Naismith (NEI), John Michael (NEI), Steve Elliot (SPCDD), L. J. Luedke (SPCDD).

Commissioner Zapata indicated that his Precinct included the cities of Sinton, Taft, and portions of Gregory. He generally outlined on maps the areas of the County for which he is responsible. He discussed known flood prone areas. The design team discussed the objectives of the study with regard to major outfalls and asked Mr. Zapata for additional problem areas in his Precinct. Mr. Zapata addressed several County roadways where standing water was a problem, including County Roads 57, 41, 39, and 37, and FM 881, 630, and 1945. He also indicated a concern for drainage south of Gregory into Green Lake. Possible meetings with the State Department of Highways and Public Transportation and railroads were considered so that potential drainage recommendations could be coordinated with the existing plans. It was also decided that a meeting with the County Engineer, Mr. Julius Petrus, would be arranged in the future.

Commissioner Duncan indicated the limits of his Precinct, which included the City of Portland. USGS maps of the area were viewed and Mr. Duncan discussed known flood prone areas. His primary concern was drainage in the areas north and west of Portland which is experiencing rapid population growth. The existing drainage into Green Lake via Oakridge Ditch was discussed as well as potential alternate flow diversions to Gum Hollow or the Doyle Addition west of Portland.

Commissioner Edwards indicated the limits of Precinct 4, which includes Aransas Pass and Ingleside. Steve Elliott (San Patricio County Drainage District) explained the purpose for the study and the types of information needed by her for the completion of tasks. Mrs. Edwards' primary concerns were drainage problems in the rapidly developing areas between Aransas Pass and Ingleside. Outfall drainage via McCampbells Slough was discussed in detail and Mr. Elliott explained the need for additional drainage relief from FM 2725 to Corpus Christi Bay. Commissioner Edwards was also visited on August 12, 1986, to further discuss drainage concerns in her Precinct.

City of Gregory
Date: June 23, 1986
Public Information Meeting - City Council Meeting
Present were Steve Elliott (SPCDD), and John Michael (NEI).

Mr. Elliott and John Michael introduced the scope and objectives of the flood control study to the Mayor and City Council. John Michael handed out copies of the project completion schedule and detailed budget estimate. It was explained that this was a study outlining major flood control improvements to relieve present or potential growth areas that may experience severe flooding. Steve Elliott and John Michael explained their understanding of some of the city's drainage problems and asked the Council and public for their comments. The Mayor and Council thanked the San Patricio County Drainage District for their continued assistance in the Gregory area and specific items of concern were left to be discussed in workshop meetings.

City of Gregory
Date: August 11, 1986
Workshop Meeting - City Council Meeting

Steve Elliott and John Michael explained the nature of the workshop and passed out a USGS map of Gregory to use as an aid in discussion. Mr. Elliott explained how the natural fall of the land surface and subsequent surface runoff was from the southwest to northeast through the City. With the development of the County, including highways, railroads, and industrial growth a portion of the drainage pattern had been altered and diverted to Green Lake and to the lake's final outfall to Nueces Bay. Considerable development, both industrial and municipal, appears to be blocking the natural drainageway to the northeast. It was explained that Gregory lacked major outfall structures and easements for the construction of those improvements. Discussion was then opened to the Council. One Council Member requested that the study address Green Lake dam and the possibility of diverting Portland drainage waters away from Gregory and the lake. It was stated that the project engineers would investigate the capacity, with computer modeling of Green Lake. Another Council Member asked if the San Patricio County Drainage District could line the existing ditch that runs through the new TDCA project area and outfalls to the main SG Ditch south of town and eventually to Green Lake. Mrs. Saldivar, City Secretary, gave John Michael the FEMA maps for Gregory to be utilized in the study. The Council appeared to be very interested in obtaining the Drainage Design Criteria Manual for future use. Steve Elliott concluded the meeting and remarked that continual coordination with the City Staff would be required throughout the study.

City of Sinton
Date: July 1, 1986
Public Information Meeting - City Council Meeting

Steve Elliot (San Patricio County Drainage District) introduced the scope and objectives of the flood control study to the City Council and introduced Pete Manz and Sam Vaugh (HDR), J. P. Naismith and John Michael (NEI), and Bob Wear (Water Development Board). John Michael handed out copies of the project completion schedule and detailed budget estimates. John Michael and Pete Manz discussed the items outlined in the completion schedule. Jim Naismith added that all work would be closely coordinated with the work being performed in the TDCA drainage study presently ongoing. A favorable response was received from the Council and Mayor Pro-Tem Daryl Lemke asked if the possibility of a public meeting would assist in educating the public to the project. No action was taken. Steve Elliot asked that a workshop meeting with the City Staff, Council, and public be scheduled to discuss in detail the City's needs and problems.

City of Sinton
Date: August 12, 1986
Workshop Meeting: Present were Walter W. Hill, Jr. (City Manager), Ron Garrison (Director of Public Works), Steve Elliott, L. J. Luedke (SPCDD), Sam Vaugh (HDR), John Michael and James P. Naismith (NEI).

Steve Elliott and John Michael briefed Mr. Hill on the progress to date of the Phase I work. Mr. Hill indicated that Sinton is aware that this study was dedicated to flood control as opposed to local drainage problems. He also emphasized that the City of Sinton will be concerned about the impacts of outside improvements (i.e., Peter's Swale near Odem) on flooding problems at Sinton. Numerous flood control alternatives were discussed. The alternatives given the most attention included detention storage, reservoir(s) located west of Sinton and the diversion of Peter's Swale to Nueces Bay. Mr. Hill indicated that the study should first evaluate the effects of channel improvements on Chiltipin Creek downstream of Sinton to the Aransas River. It was stated that this would be evaluated during the study period; however, it was indicated that channel improvements very far downstream should have little influence on flooding problems in the City of Sinton. It was speculated that channel improvements (possibly including channel lining) through the City of Sinton might prove to be the most cost effective measure. Mr. Hill also requested that the study consider the idea of a levee on the north side of the South Ditch and investigate the achieving of topographic maps in the area east of and along Highway 77, south of Sinton. Steve Elliott indicated that the state may have some topographic information available along Texas State Highway 181 from Sinton, north several miles.

After the Sinton workshop meeting, Jim Naismith, John Michael, and Sam Vaugh visited with Mr. James Johnson, Resident Engineer at the Texas Department of Highways and Public Transportation Sinton office. This meeting was to inform the State of the scope of the study and to discuss specifically the State's existing box structure at Peter's Swale and Highway 77 at Odem, Texas. Mr. Johnson appeared to understand the objectives of the study and expressed a desire to be kept informed. It was stated that since the scope of the study will include evaluation of all existing major drainage structures throughout the county, that considerable coordination efforts with the State would be required.

City of Odem
Date: August 5, 1986
Public Information Meeting - City Council Meeting

John Michael introduced the scope and objectives of the flood control study and introduced Mr. L. J. Luedke (SPCDD) and Michael Vanecek (SPCDD), Jim Naismith (NEI), and Sam Vaugh (HDR). Several copies of the project completion schedule and budget estimates were passed out and Sam Vaugh and Jim Naismith discussed the items outlined in the handouts. It was requested that a workshop type meeting with the Council to discuss specific drainage problems and solutions be set up for the near future. The Council appeared to be interested and agreed that the meeting could be arranged by Mr. Elliott through the City Secretary, Miss Billie Jo Tennill. The floor was open to discussions and the only specific item expressed was by one of the Council members asking that the workshop meeting address the redirection of flow in Peter's Swale southward to Nueces Bay.

City of Odem
Date: August 12, 1986
Workshop Meeting, City Hall, Odem, Texas
Present were Miss Billie Jo Tennill (City Secretary, City of Odem), L. J. Luedke, Steve Elliott (SPCDD), Jim Naismith and John Michael (NEI), Sam Vaugh (HDR).

Miss Tennill indicated that Odem is primarily concerned with localized flooding in the northwest portion of town, which is located immediately upstream of the Highway 77 crossing of Peter's Swale. Mr. Michael indicated that most of the flooding problems are probably due to a lack of capacity in Peter's Swale prior to flooding of streets. The structures located on Peter's Swale from Highway 77 to County Road 42 were discussed in detail, and it was noted that the 100 year discharge at this point is approximately 3000 CFS and that further improvements to these structures should be designed around this parameter. Miss Tennill indicated that the FEMA base flood elevation for Odem is seventy-four feet MSL and that Naismith Engineers has surveyed floor slab elevations in northwest Odem which should prove helpful in benefit-cost analysis. Miss Tennill also indicated that Odem is aware of Sinton's concern that structural and channel improvements in Odem may increase flood peaks in Sinton. It was emphasized, however, that it was felt that no significant effects of improvements in the Odem area will be noticed at Sinton, The people of Odem are very interested in the possibilities of diverting flow in Peter's Swale to Nueces Bay. It was stated that this possibility would be thoroughly evaluated, but that the benefit-cost analysis may not favor such a project. Miss Tennill indicated that the City Staff and

Council of Odem are very aware of the fact that several events in excess of the "100 year" event have impacted Odem in recent years, hence consideration should be given to modeling the effects of proposed improvements using one or more historical events in addition to the 100 year design flood.

City of Aransas Pass
Date: July 7, 1986
Public Information Meeting - City Council Meeting

Mr. Steve Elliott (SPCDD) introduced the scope and objectives of the flood control study and introduced Steven Hiltpold and John Michael (NEI). Mr. Michael handed out copies of the project completion schedule and budget estimates to all the Council Members for their review and discussion. The Council thanked the San Patricio County Drainage District for their continued support. Mr. Elliott requested that a workshop meeting be set up to discuss in detail the City of Aransas Pass's flood and drainage problems with City Staff and Council.

City of Aransas Pass
Date: August 12, 1986
Workshop Meeting: City Hall, Aransas Pass
Present were Rick Ewaniszyk (City Manager), Allen Berna (Public Works Director), Steve Elliott (SPCDD), Sam Vaugh (HDR), Jim Naismith (NEI), and John Michael (NEI).

Mr. Berna indicated that he and others were concerned with the potential flooding problems caused by development of the Golden Palms (subdivision with canals) in an area just inside the seawall which once acted as a collection area or sump for storm runoff prior to being pumped over the seawall. It was expressed to the City Staff that recent meetings with the Fish and Wildlife Service by Naismith Engineers, Inc. and San Patricio County Drainage District, indicate that some storm runoff may be routed to the new canals if measures are taken to prevent the entrance of undesirable chemicals and debris. This appeared as good news to the Staff who had been previously informed otherwise by the developer's engineer. Mr. Ewaniszyk indicated that none of the improvements outlined in the 1983 twenty-five year design return period drainage study had been implemented. City Staff was very interested in the results of the 100 year return period event. Mr. Naismith indicated that the area bounded Euclid Street and the seawall, and State Highway 361 and Blossom Avenue streets located behind the seawall might be used as a drainage sump for lift stations. He indicated that the design of such a detention facility could be incorporated into a City Park or other residential development. Mr. Ewaniszyk indicated that there are many property owners in the area, but that very few are residents and that present tax revenues are minimal. Concern was implied by City Staff with regard to drainage of presently undeveloped areas south and west of town. They indicated that this would be a fast growing area in the next few years and that drainage relief was needed. The possibility of diverting runoff from behind the seawall around Ransom Channel was also discussed. The area west of town (west of Avenue A), drains to McCampbells Slough and the area south of town will probably be drained around and to the south of the seawall. These drainage areas will also need to be investigated as future growth areas.

City of Taft
Date: July 8, 1986
Public Information Meeting - City Council Meeting

Steve Elliott (SPCDD) introduced the scope and objectives of the flood control study and introduced Jim Naismith (NEI) and John Michael (NEI), who passed out copies of the project completion schedule and budget estimate to all the Council Members. Mr. Michael briefly explained the items outlined in the handouts. Steve Elliott explained his understanding of the drainage and flood problems in the City of Taft and surrounding area. Mr. Naismith spoke briefly about some of the project deliverables, such as aerial photography and the Drainage Criteria Manual. Council Members thanked the San Patricio County Drainage District for their continued support. One Council Member expressed a concern that the report not change past FEMA work done in the area. The Mayor agreed and requested that a workshop meeting be set up in the near future to discuss specific needs.

City of Taft
Date: July 28, 1986
Workshop Meeting - City Council Meeting

Steve Elliott (SPCDD) and John Michael (NEI) passed out copies of the USGS Map for the Taft area for discussion purposes. Steve Elliott had a 2' contour map of the City that was also used in discussion. Mr. Michael explained that generally, runoff flows from south to northeast naturally; however, with the construction of Highway 181, MOPAC, and the city proper; runoff has been diverted to flow west and east along the railroads until it reaches outfall structures and ditches finally outfalling at Chiltipin Creek. It was indicated that backwater analysis would be completed on the John Deere Ditch west of town and the other major drainage ditch east of town, known as AN-01. Mr. Veselka, City Manager, requested that the study address improving the drainageway by the elevated storage tank and old sewage disposal plant, by using detention or other forms of landscaping to improve runoff and aesthetics, instead of the existing ditch. City of Taft presently owns this property. Council Members expressed that the growth area for the City would be in northeast Taft, some of which is at elevation 40-45 MSL. The question of tidal effects on drainage was addressed and explained that it was probably not a real concern. Since the John Deere Ditch was to be investigated, so will the effects on the Verbina and Hidalgo Ditch in south Taft. The Council Members asked also that the study address the overall drainage of the underground system in north Taft, as well as its outfall ditch that runs out by the airfield. Mr. Elliott recommended that the study look at bypassing the existing bar ditch and cutting across a field and intersecting the major ditch, east of town by the old sewer plant. It was expressed that continual assistance from City Staff would be received throughout the study.

City of Ingleside
Date: August 12, 1986
Public Information Meeting

Steve Elliott (SPCDD) introduced the scope and objectives of the flood control study and introduced Jim Naismith (NEI) and John Michael (NEI) who passed out copies of the project completion schedule and budget estimate to all Council Members. Mr. Michael explained the items outlined in the handouts. Mr. Naismith followed with a brief explanation about the deliverables for the project. The Mayor and City Council thanked the San Patricio County Drainage District for their continued support and agreed that a workshop meeting with City Staff be set up to discuss the City's drainage and flood control needs.

City of Ingleside
Date: August 26, 1986
Workshop Meeting at City Hall, City of Ingleside
Present were Dell Lewis (City Manager), George Kneupel (Director of Public Works), City Building Official, Steve Elliott (SPCDD), and John Michael (NEI).

Steve Elliott explained briefly the reason for the workshop meeting and introduced John Michael from Naismith Engineers, Inc., who passed out USGS Maps of the area and explained his understanding of the City's flooding problems. Dell Lewis was given a copy of the South Eastern County Drainage Plan and the City gave Mr. Michael copies of the FEMA Rate and Floodways Maps. A City Drainage Basin Map was also obtained. The City Staff expressed concern with drainage in the Villanova Subdivision which divides to McCampbells Slough to the north and Kinney Bayou to the south. These areas appear to have been constructed in a naturally occurring low area. The City Building Official asked why some areas were shown in flood zones on FEMA maps, when the are actually on high ground and do not flood. Mr. Michael explained briefly how the HEC-2 Models were prepared and how this situation can occur. City Staff indicated that FEMA Maps end at Avenue "G". However, flooding occurs to the north of this area. It was explained that some of the work to be performed in the study will be to evaluate existing FEMA Maps and revise if necessary. The City Public Works Director indicated a concern for the area east of FM 2725 where commercial development has blocked natural drainage for the west areas. There was also concern for an area east of town along Fourth Street that is low and drains in a pond. The Public Works Director has to operate a flood gate to control runoff to Avenue "G" via 48" RCP. It was agreed that this problem be addressed in the report. A master plan for Ingleside at least ten years ago was obtained as well as drainage calculations for Avenue "G" watershed. Mr. Michael explained that backwater analysis on Kinney Bayou and McCampbells Slough would be performed. Ms. Lewis explained that growth potential for the City would probably be in all directions.

City of Portland
Date: August 19, 1986
Public Information Meeting - City Council Meeting

Mr. Steve Elliott (SPCDD) introduced the scope and objectives of the flood control study to the Council and introduced John Michael (NEI) who passed out copies of the project completion schedule and budget estimate. Mr. Michael explained the items covered in the handouts and explained briefly his understanding of drainage in the City of Portland, then requested input from the Council and general public. The Mayor thanked the San Patricio County Drainage District for their support over the past years and agreed that a workshop meeting with City Staff be set up in the near future. One member from the audience asked if the 100 year storm was going to be evaluated and expressed that he thought the results would prove to be cost prohibitive to design for this event storm. Mr. Michael explained that it was not feasible under most circumstances to design typical urban drainage for the 100 year event due to economics. However, the effect of the 100 year event on major drainageways and outfall structures must be considered.

City of Portland
Date: August 26, 1986
Workshop Meeting at City Hall, Portland, Texas

Present were Richard Burdine (City Manager), Hilliard Goode (City Engineer), Steve Elliott (SPCDD), Jim Naismith (NEI), and John Michael (NEI).

Mr. Burdine presented a Master Plan and survey that was prepared for the City in 1971. He expressed that the City's big growth areas are to the north and west of town. He asked that the study address future drainage to the west of town, that it investigate the drainage in the Doyle Subdivision located by the airport and that the future completion work on Memorial Ditch be investigated. Mr. Burdine also questioned the full capacity of the Oakridge Ditch which outfalls to Green Lake and asked how far to the west could it be lengthened to possibly alleviate problems west of town. Mr. Elliott indicated the probable limits that the Oakridge Ditch could be extended to. Mr. Burdine also requested that the San Patricio County Drainage District investigate the outfall ditch in the East Cliff area along southeastern Portland. This area has plenty of fall with several drop structures. The meeting was ended and Jim Naismith and John Michael met with Mr. Goode at the City Annex to discuss existing available mapping for the City. It was explained that future meetings with Staff would be required throughout the duration of the study.