APPENDIX A

HYDROLOGY AND HYDRAULICS

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HYDROLOGIC AND HYDRAULIC ANALYSES

PART 1 - HYDROLOGIC ANALYSIS

WATERSHED DESCRIPTION

The drainage area of the Trinity River, from its headwaters to the confluence of Five Mile Creek, near the Interstate Highway 20 bridge in south Dallas, was evaluated during this study. This area, which is commonly referred to as the "Upper Trinity" watershed, covers about 6,275 square miles. It includes the majority of the Dallas-Fort Worth (DFW) Metroplex. Terrain in this watershed varies in elevation from about 1,200 feet National Geodetic Vertical Datum (NGVD) at the headwaters of the West Fork of the Trinity River just northeast of Olney, Texas, to about 380 feet NGVD at the confluence of Five Mile Creek. A general watershed map is included as Plate A-1.

Of the five US Army Corps of Engineers (USACE) flood control reservoirs in the study area, three (Lakes Benbrook, Lewisville, and Grapevine) were impounded in the early 1950's. Impoundments in the other two USACE reservoirs (Lakes Joe Pool and Ray Roberts) were initiated in January 1986 and June 1987, respectively. Additional major USACE flood control projects in the study area include the Fort Worth Floodway and Dallas Floodway levee/channel improvement systems.

The two largest non-Federal lakes in the study area, both of which are situated on the West Fork of the Trinity River, are Lake Bridgeport and Eagle Mountain Lake. Lake Bridgeport is located just west of Bridgeport in Wise County. Eagle Mountain Lake is located in northwestern Tarrant County, just upstream from the much smaller Lake Worth, which is owned by the City of Fort Worth. Eagle Mountain Lake has two sets of outlet gates and an emergency spillway, but since it has no dedicated flood control storage, large releases are required during flooding periods. Smaller lakes within the Upper Trinity watershed include: Lake Amon Carter, located on Big Sandy Creek south of Bowie in southwestern Montague County; Lake Weatherford, located on the Clear Fork of the Trinity River northeast of Weatherford in Parker County; Lake Arlington, located on Village Creek in western Arlington in Tarrant County; and Mountain Creek Lake, located on its namesake in Grand Prairie in western Dallas County.

CLIMATOLOGY

The climate in the Upper Trinity watershed is humid subtropical with hot summers and mild winters. Snowfall and subfreezing temperatures are experienced occasionally during the winter season. Generally, the winter temperatures are mild with occasional cold periods of short duration resulting from the rapid movement of cold pressure air masses from the Northwestern polar regions and the continental western highlands.

Recorded temperatures at the DFW International Airport have ranged from a high of 113°F in June 1980 to a low of -1°F in December 1989. The average annual temperature over the watershed varies from 64°F at Bridgeport in the northwestern extremity of the watershed to 66°F at DFW International Airport. The mean annual relative humidity for the DFW Metroplex is about 65 percent. The average annual precipitation over the watershed varies from about 30 inches at Jacksboro, in the northwestern extremity of the watershed, to about 32 inches in the DFW Metroplex. The extreme annual precipitation amounts since 1887 include a maximum of 53.54 inches in 1991 at the DFW International Airport and a minimum of 17.91 inches in 1921 at Fort Worth. The maximum recorded precipitation in a 24 hour period was 9.57 inches, at Fort Worth on the 4th and 5th of September 1932. A large part of the annual precipitation results from thunderstorm activity, with occasional very heavy rainfall over brief periods of time. Thunderstorms occur throughout the year, but are more frequent in the late spring and early summer.

The average length of the warm season (freeze-free period) in the DFW Metroplex is about 249 days, extending from mid-March to mid-November.

MODEL DEVELOPMENT

A watershed runoff model for the area was developed utilizing the USACE computer program "HEC-1". The drainage area was divided into 110 subareas in order to be responsive to the timing of each major tributary's runoff contribution to the total flood hydrograph and also to obtain detailed flow information (flood hydrographs) at all major points of interest on the Clear, West, and Elm Forks, as well as the mainstem of the Trinity River. Plate A-1 shows the subarea arrangement. A one-hour computation time interval was used in the model. Each reservoir having flood control storage was assumed to be at conservation pool level at the start of the hypothetical, frequency related storms/floods and at a level corresponding to that at which one-third of the full flood control pool (except at Lewisville Lake which was started at 89 percent full) would already be occupied at the start of the USACE" Standard Project Flood (SPF). All reservoirs without flood control storage were assumed to be at normal (conservation pool) levels at the start of all storm/flood events. Lake Bridgeport, Eagle Mountain Lake, Lake Worth, and Lake Arlington were assumed to reside at a level corresponding to 2, 3, 2, and 3, feet, respectively, above normal (conservation pool) level at the start of the SPF event.

MODEL CALIBRATION

The Upper Trinity River "HEC-1" model was calibrated by reproducing the significant historical flood hydrographs of May-June 1989, April-May 1990, and December 1991. Initial abstractions, infiltration rates, and Snyder"s unit hydrograph parameters (lag time and peaking coefficient) were adjusted in order to generate computed hydrographs that would reasonably match the observed flood hydrographs at the streamflow gages and lakes (inflow) throughout the basin. Additionally, the Muskingum "X", "K", and number of routing steps (in both the Muskingum and modified Puls routing methods) were adjusted during the calibration efforts. The results of the flood hydrograph reproductions for the May-June 1989, April-May 1990, and December 1991 events were tabulated and compared with the results of hydrograph reproductions for the October 1974, March 1977, October-November 1981, and May 1982 events, as published in the "Upper Trinity Reconnaissance Study", dated May 1990. The results of these analyses, for the seven storm/flood reproductions, were used to assign each of the specific parameters noted above.

The model was further calibrated by adjusting infiltration rates, within reasonable limits, in order to match as closely as possible the peak values of eight different frequency related flood peaks, based on analyses of historical peaks at six streamflow gaging stations. These streamflow gaging sites include the Clear Fork of the Trinity River at Fort Worth, the West Fork of the Trinity River at Fort Worth, the West Fork of the Trinity River at Grand Prairie, the Elm Fork of the Trinity River near Carrolton, the Trinity River at Dallas, and the Trinity River below Dallas. The target values of the peak flows for hypothetical frequency related floods at any particular gage were determined by performing a flood flow frequency analysis from the record of flows at that gage. The time period covered by the gage record of flows was selected to extend from water year 1953 through water year 1992. Water year 1953 was used as the starting point since all of the major flood control reservoirs, except Lakes Joe Pool and Ray Roberts, were in place by 1952. Water year 1992 was used as the "cut-off" point for the statistical analyses since the last significant flood events on the major branches and the main stem of the Trinity River occurred in December 1991 (water year 1992). It should be noted that the degree of urbanization and conditions of available valley storage changed gradually, but significantly throughout this gaging period; therefore, a direct (perfect) calibration would not necessarily represent present day or projected baseline conditions. The flood flow frequency analysis was performed using the procedures described in "Guidelines for

Determining Flood Flow Frequency, Bulletin No. 17B, Revised September 1981", and using USACE" Southwestern Division's skew criteria. The USACE computer program "HEC-FFA" (dated May 1992) was used to statistically estimate the frequency versus discharge relationship at each of the investigated gaging sites. A graphical representation of these statistical frequency curves is presented on Plates A-2 through A-7. Plates A-7A and A-7B provide samples of the flood hydrograph reproductions.

MODEL RAINFALL

The hypothetical precipitation for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year frequency storms was developed using data from the National Weather Service (NWS) "Technical Paper 40 (TP40)" and the National Oceanic and Atmospheric Administration (NOAA) Memorandum "NWS Hydro-35". Precipitation for the 500-year frequency storm was computed by extrapolation. Figure 15 of TP40, "Depth-Area-Duration" curves, was used to adjust the point rainfall to representative average values over the contributing watershed size at each point of interest. One-hour computation time intervals were used with a 24-hour storm duration for each of the frequency related storm events. As an example, the point rainfall amounts for the 24-hour duration storms, with the storm center positioned approximately at the streamflow gage for the West Fork of the Trinity River at Grand Prairie, are as follows: 1-year, 3.20 inches; 2-year, 4.00 inches; 5-year, 5.38 inches; 10-year, 6.43 inches; 25-year, 7.54 inches; 50-year, 8.55 inches; 100-year, 9.55 inches; and 500-year, 13.10 inches. The area-adjusted 100-year frequency storm rainfall distribution is presented in Table A-1.

The Standard Project Storm (SPS) was assumed to have a total rainfall amount equal to 50 percent of the Probable Maximum Storm (PMS) rainfall amount, as adjusted in accordance with USACE" Hydrometeorological Report Number 52 (HMR 52). The PMS precipitation (commonly referred to as the PMP) was determined in accordance with the method described in "HMR 51", dated June 1978, Subject: "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," and "HMR 52", dated August 1982, Subject: "Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian." The computer program used to develop the SPS was the USACE" "HMR52". The SPS duration was 72 hours. Four separate elliptical storm positions were used to obtain "critical-centerings" on the West, Clear, and Elm Forks, and on the main stem of the Trinity River. One of these storm centers was critically centered for the "Trinity River at Dallas" streamflow gage, for which the dominant major storm axis orientation from "HMR52" is 220 degrees bearing and the critical storm orientation angle is 246 degrees bearing. The average SPS precipitation over the 6,275 square miles of drainage area is 5.64 inches. This average precipitation is based on a critical centering of the hypothetical elliptical SPS at Hurst, in northeastern Tarrant County. As an example, the SPS rainfall amount for Subarea 50, located near the storm center, is 19.52 inches. The SPS rainfall distribution for that subarea is presented in Table A-2.

INITIAL ABSTRACTIONS AND INFILTRATION RATES

The rainfall loss values were assumed to vary with the frequency of each storm event and the nature of the soil surface. The USACE, Fort Worth District (FWD) standard values are presented in Table A-3. Data on soils was obtained using generalized soils maps from the USDA Natural Resources Conservation Service, formerly the Soil Conservation Service (SCS) , which had been linked electronically with the detailed subbasin layout mapping in a geographic information system (GIS). The "percent sand" for each subarea was determined by first assigning a value to each soil type and then weighting the value for each applicable soil type in proportion to the area of each soil type in a particular subarea. Engineering judgment was used for some subareas to override the "percent sand" values obtained by the GIS. The initial abstraction and infiltration rate for each subarea was weighted in accordance with the previously tabulated values for clayey (zero "percent sand") and sandy (100 "percent sand") soils.

Comparisons were made between the frequency versus discharge relationships determined based on the statistical analysis of historical data at the major streamflow gages and those based on results of the "HEC-1" modeling. Adjustments were made to the rainfall losses at some subareas in order to produce a better correlation. The adjusted values were then used in this study. The loss rates for the SPF event varied regionally and were identical to those used in the "Upper Trinity Reconnaissance Study".

DEVELOPMENT OF UNIT HYDROGRAPHS

Unit hydrographs for the subareas above Eagle Mountain, Benbrook, Grapevine, and Lewisville Lakes were based on the adopted Snyders lag times and peaking coefficients obtained through the historical flood hydrograph reproductions of the May-June 1989, April-May 1990, and December 1991 events. Previously developed relationships between measurable subbasin parameters and Snyders unit hydrograph lag time, for both clayey and sandy soils, with consideration for the degree of urbanization, were used for the smaller, more urban subareas within the "HEC-1" model , downstream of the lakes.

Land use data for baseline conditions (year 2000) were obtained from the North Central Texas Council of Governments (NCTCOG). This data and a table correlating land use to "percent urbanization" and "percent imperviousness" was input into the GIS. Net values of these parameters at each subarea were derived from the GIS by weighting the land uses within each subarea by the default values associated with each land use.

The Snyders unit hydrograph lag time ("time-to-peak") was developed for each small, urban subarea using methodology described in "Synthetic Hydrograph Relationships, Trinity River Tributaries, Fort Worth-Dallas Urban Area" by T. L. Nelson, 1970. These mathematical relationships, which are referred to as "Urbanization Curves", are available for both Cross Timbers sandy loam- and Blackland Prairie clay- dominated watersheds in the general vicinity of the DFW Metroplex. The geographical characteristics of each subarea, including the length of the major stream (L), the distance from the subarea outflow point to the location of the subarea centroid (L_{ca}), the weighted slope (S_{st}) of the major stream, and the "percent urbanization" are the data used in the equations to determine the Snyders lag time for the two general extremes of soil type. The Snyders lag for each subarea was then generated mathematically from the "Cross Timbers Sandy Loam" and "Blackland Prairie Clay" Urbanization Curves through direct interpolation, based on the percentage of each soil type within that subarea. These urbanization curves are shown on Plates A-8 and A-9.

The subbasin parameters (both measured and computed) for baseline conditions (year 2000) are presented in Table A-4.

ROUTING PROCEDURES

The modified Puls routing method was used along the reaches downstream of Lake Worth and Benbrook, Grapevine, and Lewisville Lakes. The valley storage versus discharge relationships were based on USACE "HEC-2" backwater analyses, using the latest (February 1991) 2-foot contour interval topography along the Clear, West, and Elm Forks and the mainstem of the Trinity River. The modified Puls routing method was also used along the reach of Denton Creek below Grapevine Lake, but in this particular case, the valley storage versus discharge relationships were based on "HEC-2" backwater analyses used in the Denton County Flood Insurance Study of 1985.

The Muskingum routing method was generally used along the reaches upstream from Lake Worth, Benbrook Lake, and Lewisville Lake. The Muskingum "X", "K", and number of routing steps (in both the Muskingum and modified Puls routing methods) were calibrated by reproducing the historical flood hydrographs of May-June 1989, April-May 1990, and December 1991.

BASELINE CONDITIONS

Baseline conditions represent estimated watershed development for the year 2000, based on land use data obtained from NCTCOG. The "percent urbanization" and "percent imperviousness" values for each subarea were derived from the GIS as previously mentioned. Unit hydrographs for each subarea in the DFW metropolitan area were adjusted for this "baseline" urbanization. Rainfall losses for each subarea were also adjusted for "baseline" imperviousness.

The valley storage versus discharge relationships for the Clear, West, and Elm Forks, and the mainstem of the Trinity River, within the DFW metropolitan area, were based on "HEC-2" backwater analyses which had been adjusted to account for any known and/or anticipated future projects, and/or those which were under construction since the development of the February 1991 aerial photography and subsequent topographic mapping.

A summary of the computed probability peak discharges for baseline conditions is presented in Table A-5.

FLOOD CONTROL ALTERNATIVES

Many flood control alternatives were considered in this study. They are described in detail in Part 2 of this appendix ("Hydraulic Analysis"), and in the main report and other appendices. The two structural alternatives on the main stem of the Trinity River which were analyzed with hydrologic models were: the National Economic Development (N.E.D.) Plan, which is a 1,200-foot long overflow swale in the vicinity of the confluence of White Rock Creek; and the Recommended Plan, which is a combination of a proposed Chain of Wetlands Plan and the implementation of both the Lamar Street and Cadillac Heights Levee projects. Summaries of the computed probability peak discharges for baseline conditions on these two scenarios are presented in Tables A-6 and A-7, respectively.

RISK ANALYSIS

In accordance with recent USACE study criteria, a risk-based economic analysis was performed in this study. From a hydrologic standpoint, the three key locations along the Trinity River which were assessed for this purpose are: the downstream end of the existing Dallas Floodway (i.e. at the abandoned Atchison,Topeka, & SanteFe Railroad crossing); the Central Wastewater Treatment Plant Levee; and the "Below Dallas" streamflow gage site (at State Highway Loop 12). This analysis was performed in accordance with the procedure described in USACE Engineering Circular "EC 1105-2-205", dated 25 February 1994, entitled "Risk-Based Analysis for Evaluation of Hydrology/Hydraulics and Economics in Flood Damage Reduction Studies."

Computed probability discharge versus frequency curves for baseline conditions were developed for the Trinity River at these three locations using the "HEC-1" model as previously described. The log-transformed statistics of mean, standard deviation, and skew were then developed by trial-and-error to reasonably reproduce the discharge versus frequency curve at each location. The "HEC-FFA" program was used in this process, with 40 years assigned as an equivalent gaging record length. A summary of the calibrated statistics at each location, for each analysis condition, is presented in Table A-8. These statistical parameters were supplied as a part of the input to the "HEC Risk Analysis Program" which was then used to crudely simulate the long-term average annual equivalent flood damages under baseline conditions. Graphical representations of the resulting frequency curves are presented on Plates A-10 through A-18.

INTERIOR DRAINAGE ANALYSES

While providing a substantial degree of riverine flood damage reduction to existing properties in the reach downstream from the present end of the Dallas Floodway (at the Dallas Area Rapid Transit crossing), the proposed Lamar Street and Cadillac Heights Levees would trap a major portion of the surface runoff from about 1,264 and 337 acres of localized subbasin area, respectively. Interior drainage facilities (sumps and sluice outlets) would be required to insure that this runoff does not contribute to any "induced" flood damage. Since the levee alternatives have been intentionally designed to provide at least a 100-year frequency level of Trinity River flood containment, it was deemed inappropriate, by the local sponsor (the City of Dallas), to propose anything less than a 100-year frequency level of protection from potential interior drainage flood damage. This design level was thus necessary to meet the City"s criteria for the Recommended Plan. Developing at least a 100-year frequency level of protection from interior drainage also fits well within the City"s goals and incentives relative to their participation in the National Flood Insurance Program (NFIP).

Interior Runoff

A combination of the previously mentioned February 1991, 2-foot contour interval topographic mapping, older and less detailed City of Dallas topographic mapping, and the standard 7.5-minute US Geological Survey (USGS) 10-foot contour interval topographic mapping was used to delineate the subbasins which would contribute to interior drainage behind the proposed Lamar Street and Cadillac Heights Levee alignments. In cases where the topographic detail was too limited, the City"s storm sewer plans were used to further define the boundaries of the contributing drainage areas. A generalized watershed layout map for the interior drainage areas is presented on Plate A-19.

Subbasin runoff models, including both USACE"s "HEC-1" and the USACE-FWD"s "SWFHYD" (Southwestern Division, Fort Worth District, Hydrologic Analysis Package) were developed for these areas. Point rainfall, and its adjustment to represent basin average precipitation, was developed as previously mentioned, using a combination of the NWS "TP-40" and NOAA "Hydro. 35" data sources. Since the overwhelming majority of the localized basins are already fully developed, with a more intensive degree of urbanization expected to prevail once the levee projects are implemented, both the unit hydrograph lag times and rainfall losses were assumed to be minimized. In this particular case, the Snyder"s unit hydrographs were uniformly assigned a lag time of 5 minutes and a peaking coefficient of 0.71875 (which relates to a 640 Cp value of 460). This peaking coefficient matches the general value determined appropriate for the DFW vicinity, based on extensive flood hydrograph reproductions performed by USACE- FWD in the late 1970's. For design purposes, each subbasin was assumed to be fully covered with impervious surface material.

Riverine Tailwater Assessment

An extensive statistical evaluation was made of Trinity River flows (mean daily values) and their correlation with localized precipitation at Dallas. The analysis period of May 1957 to present (actually September 1994, due to limits of available electronic data) was used since most of the major USACE flood control reservoirs which affect flows along this reach of the river were in full operation beginning with the major inflows resulting from the April-May 1957 flood events. Ray Roberts and Joe Pool Lakes, which were implemented in the 1980's, have a fairly limited impact at this point in the river system. A generally weak correlation between localized storms and high mean flows in the river was noted. This is due to the fact that substantial rainfall upon the central and upper portions of the Clear, West, and Elm Forks of the Trinity River watershed, runoff from which is gradually routed through the long stream reaches and assorted reservoirs, is necessary to produce large, sustained flows at the proposed project reach. The runoff from the small localized interior basin watersheds at Dallas is often fully evacuated prior to the arrival significant flows on the river itself. For design purposes however, it is reasonable to assume that a closer coincidence will occur occasionally over the project life. Therefore, the prevailing (limited) steady state release rate used in evacuating water from the flood control pools of the upstream USACE reservoirs was used as a basis for the Trinity River conditions at the time of potentially intense localized precipitation. This flow value, 15,000 cubic feet per second (cfs), was assumed to be supplemented with 5,000 cfs from uncontrolled Trinity River inflows, downstream from the USACE reservoirs, for a total design tailwater flowrate of 20,000 cfs. Such a mean daily streamflow has been exceeded 110 times over the 37.3-year analysis period. Even though this amounts to only about 0.81 percent of the overall time, it must be understood that the lengthy periods during which the flood control storage in the USACE reservoirs is being evacuated offer prime opportunities for small localized thunderstorms to produce interior runoff at the proposed project.

The 20,000 cfs flowrate on the Trinity River relates to slightly less than the 2-year frequency peak discharge under the proposed project(s) conditions. It was applied within the "HEC-2" model being used concurrently for the hydraulic analysis, in order to establish design-condition tailwater elevations at potential outlet sluice locations along both levee reaches. These tailwater elevations range from about 400 to 404 feet NGVD along the Lamar Street Levee alignment and from about 397 to 400 feet NGVD along the Cadillac Heights Levee alignment. Due to the fact that the existing terrain (and improved properties) behind the Lamar Street Levee is generally lower than its counterpart behind the Cadillac Heights Levee alignment, the outlet sluices at the two projects would operate quite differently, from an hydraulic standpoint. The outlets along the Lamar Street Levee would experience a more significant tailwater and would have a generally limited conveyance while operating under "full pipe" flow conditions. Those along the Cadillac Heights Levee would not be impacted by the design condition tailwater and would have a generally higher conveyance while operating under "inlet control" flow conditions. This disparity has a dramatic impact upon the amount of required sump storage along each project.

Existing Storm Sewers

For purposes of this preliminary design, it was assumed that each of the major storm sewers that pass beneath the areas proposed for levee protection would be retained during implementation of the levee projects. This would allow for a considerable volume of otherwise "trapped" local runoff to be passed directly into the Trinity River. These storm sewers would be realigned, where necessary to avoid excavated sumps, etc. and would have their outlets modified to include flapgates, to prevent high river stages from forcing floodwaters to spill from any low gutter inlets in the areas proposed for the levee protection. Consideration was given to omitting the flapgates and sealing-off any relatively low access points (gutter inlets, manholes, etc.), but there was simply too low of a degree of confidence that these older and possibly poorly maintained sewer lines could sustain under the pressure created during high river stages.

Information on the storm sewer alignments and sizes were generally obtained from older City of Dallas plans. These systems appear to be fairly complex, with numerous interceptors and cross connections, making it difficult to clearly establish their capacities and reliabilities. During a site visit, one major storm sewer outlet as shown on the older plans was nonexistent on the ground. The uncertainties regarding these facilities will have to be significantly reduced, and preferably eliminated, prior to actual implementation of the two levee projects. Further research will be undertaken during the Plans and Specifications stage. If the decision is made to allow the existing storm sewers to drain directly into the proposed sumps, considerable enlargements and/or deepening of those sumps, or significantly increased outlet sluice capacities, would be required to insure the desired degree of interior flood damage protection.

Potential capacities of the existing storm sewers were computed based on direct application of the Bernoulli Equation to the reach of each sewer line between its lowest curb/gutter inlet and its outlet, near the Trinity River.

Sump and Outlet Sluice Design

Localized (interior) runoff was evaluated using the "HEC-1" program as previously described, except that the potential storm sewer capacities were deducted as simple "diversions". Emphasis was placed on the evaluation of the 100-year frequency flood event, i.e. the design flood for the interior drainage facilities. The inflow flood hydrographs at each of the sump locations were stored electronically using USACE"s Data Storage System (DSS). USACE"s Interior Flood Hydrology Package "HEC-IFH" was used to perform the actual routings through the alternative sump/outlet sluice configurations. This was accomplished by importing the inflow hydrograph (from "HEC-1" and "DSS") at each sump location and applying the design condition riverine tailwater stage against the outlet sluice(s). Each sump was assumed to be one third full at the onset of the 100-year frequency inflow. Target values for the resulting peak flood stages around the sumps were initially determined based on providing flood protection to the lowest improved property (buildings, etc.); however, later in this process the local sponsor requested that the design insure that the 100-year frequency flood pool at each sump be fully contained within its excavated, or otherwise topographically defined, boundary.

Storage volumes in each potential sump were computed based on discrete mathematical integration of thin horizontal slices (surface areas) for each associated elevation. The side slopes for excavation were limited to 3:1 (horizontal:vertical) ratios, while those along the levee faces were held to 5:1 ratios. Since the sumps were arranged to fit the existing alignments of railway and/or roadway embankments, property lines, and the proposed levees themselves, etc., it was not possible to apply simple volumetric equations for common prismatic shapes. Instead, the alternative configurations were mapped and the excavations contoured, providing a measurable surface area at each interval of elevation. The actual volumetric integration was performed within the "HEC-IFH" program based on the assigned area versus depth values.

Repetitive runs were required in order to establish a series of cross combinations of sump storage and outlet sluice capacities that would meet the design requirements. The recommended scenario at each sump was based on consideration of real estate, excavation, and outlet sluice costs. The number and sizes of sluice pipes at each outlet were adjusted to allow for a predominance of shapes that have been successfully applied at the existing Fort Worth and Dallas Floodways. These are simple rectangular conduits with both a flapgate (at the outlet end) and a manually operated sluice gate (positioned beneath the levee crown). Pertinent data on the sumps and outlet sluice structures, including hydrologic effects, are presented in Table A-9. Detailed drawings, including the plan view of each sump and the plan/profile views of the outlet sluice structures are presented in Appendix C - Civil, Relocations, and Structural Engineering.

General Considerations

The location of each sump is primarily based upon the availability of segments of the most low-lying terrain along the landward side of the proposed levees. Specific care was taken to avoid the use of any lands with existing improvements, active commercial/industrial land uses, or the likelihood of signifcant hazardous, toxic, or radioactive waste (HTRW) problems. Anticipated flood inflows at each site were determined based upon standard rainfall-runoff (hydrologic) modeling, using the available 2-foot contour detailed topographic mapping as the basis for delineating each of the applicable runoff subbasins.

In the case of the Lamar Street Levee, all existing storm sewers which provide drainage to or beyond Lamar Street are proposed to be extended (where necessary) beyond the levee, and to be backflow-controlled via sluice gate and flap gate devices. The total inflow hydrograph at each sump site is based upon the difference between the total inflow hydrograph and the total capacity of the applicable storm sewers. By allowing for the continued use of these storm sewers, substantial portions of the total runoff volume can be diverted past the proposed sumps. Two of the five sump sites already provide plenty of existing storage capacity to be combined with minimally sized sluices for interior drainage facilities. A third already provides an overwhelming majority of the necessary storage. The remaining two sump sites would require extensive excavation. In summary, optimization of facilities was only required for the two sites requiring excavation, if minimally sized facilities are used. "Minimally sized" is that size which is cost effective from a construction standpoint, but can still be maintained with relative ease.

In the case of the Cadillac Heights Levee, flood runoff from the upper portion of the primary contributing drainage area is proposed to be diverted around the south end of the levee system, thereby eliminating the need for an otherwise substantial sump storage capacity. The remaining (contributing) portion of the watershed is proposed to be handled via interior drainage facilities at four existing ditch locations. Due to the fact that the existing terrain in the Cadillac Heights area is situated several feet higher in elevation than its counterpart along the Lamar Street Levee, the anticipated tailwater effects from the Trinity River are virtually negligible. This results in a prevalent condition whereby the interior drainage can be sufficiently passed through minimally sized outlet sluice structures, without the need for temporary storage of floodwaters in sumps.

In keeping with the City of Dallas' local drainage ordinance, each interior drainage facility was designed to prevent inundation of "non-sump" lands, during the passage of a "100-year" frequency flood runoff event. In practice, an acceptable design elevation (a target) was selected around the periphery of each sump area, and the combination of sump storage capacity and outlet sluice capacity was varied, until the design conditions could be satisfied. The sump capacities were allowed to vary from that provided by the existing terrain to that necessary to store virtually the entire flood runoff volume. The outlet sluice capacities (both size and number of conduits) was allowed to vary upwards from that provided by a simple 4-foot by 4-foot box culvert, deemed as minimally sized. This is also the smallest structure size of this type to have been applied along the existing reaches of the Dallas and the Fort Worth Floodways.

Alternative Solution Scenarios

Optimization of the two sumps requiring excavation and/or larger than minimal facilities was performed by (1) holding the 100-year target elevation constant, and (2) varying the size and number of outlets inversely to the excavation required to obtain the target elevation. Corresponding costs were developed for these scenarios, and can be found in the Table 9A. The scheme selected for the recommended plan corresponds to the lowest cost of the combined variables, and thus is considered as being optimized.

PART 2 - HYDRAULIC ANALYSIS

GENERAL

Hydraulic analysis was performed on the reach of the Trinity River in Dallas, Texas, that extends from the Interstate Highway 20 (I.H. 635) bridge upstream to the confluence of the West Fork and the Elm Fork of the Trinity River at the upstream end of the Dallas Floodway Levee System. The primary focus of the study has been on the reach between the Loop 12 bridge and the abandoned Atchison, Topeka, and Santa Fe Railroad bridge at the downstream end of the Dallas Floodway Levees in which various plans of improvement were found to be feasible. Analysis was performed to determine the hydraulic characteristics of the existing river and to develop plans for reducing flood damages within the city. All references to elevation are given in feet above the National Geodetic Vertical Datum (NGVD).

The HEC-2 Water Surface Profiles computer program was used to hydraulically model and compute water surface profiles for a broad range of flood events. Traditional expression of the frequency of flood events has been in terms of the recurrence interval in years, such as, the "100-Year Flood". The more appropriate expression of the probability of a particular flood magnitude is in terms of "percent chance exceedance", especially as it relates to a risk-based analysis. Therefore, the "100-Year Flood", which is defined as "the magnitude of flooding which has a 1 percent probability of being equaled or exceeded in any given year", is expressed as, the "1 percent chance flood". The nine flood events computed for this study that were traditionally referred to as the 1-year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 500-year, and the Standard Project Flood (SPF) are now referred to respectively as the 99 percent, 50 percent, 20 percent, 10 percent, 4 percent, 2 percent, 1 percent, 0.2 percent chance flood, and the SPF.

The Standard Project Flood is defined as the flood that may be expected from the most severe combination of meteorological and hydrologic conditions that are considered to be reasonably characteristic of the geographical region involved, excluding extremely rare combinations. The SPF usually has a 0.3 to 0.08 percent probability of being equaled or exceeded in any year, and is usually between 40 and 60 percent of a Probable Maximum Flood (PMF). The SPF represents a "standard" against which the degree of protection for a project may be judged and compared with protection provided at similar projects in other localities. The SPF for purposes of this study has been estimated to have a 0.125 percent probability of being exceeded in any year.

EXISTING CONDITIONS

Trinity River

The Trinity River channel within the study reach has an average depth of about 30 feet and a top width of about 200 feet. The river historically has an average discharge of about 2,000 cubic feet per second (cfs) over the period of record from 1958 to 1990. The overbanks are generally very wide and flat. The river channel has an average bottom slope of about 0.05 percent or about 2.6 feet per mile and has historically been very stable. Analysis of aerial photographs taken periodically over the past 47 years as well as historical topographic data has shown no channel migration. There have been no known bank stability problems within the study reach. The overbank areas in the floodplain are generally covered with heavy vegetation with some areas having been disturbed by mining operations or landfills. The areas of the floodplain that have the greatest density of vegetation are covered with mature trees of sufficient height to extend above the water surface of the highest flows considered in this analysis. Therefore, a consistent hydraulic roughness value has been used for all depths of flows considered in this study. Examination of historical aerial photographs revealed that a gradual increase in the density of the vegetative cover on the floodplain has occurred and increased the hydraulic roughness of the floodplain over time. This has resulted in the computation of higher flood levels within the study reach than in previous studies. Several landfills placed within the floodplain in the study reach have also raised flood levels due to the reduction of flow conveyance area and the reduction of available valley storage of floodwater.

Landfills

Four significant landfill areas are located within the floodplain of the study reach. The McCommas Bluff Landfill is located just upstream of Interstate Highway 20 (I.H. 635) and is currently operated by the City of Dallas. This landfill is a primary site for solid waste disposal for the City of Dallas. The South Loop Landfill is located immediately downstream of Loop 12 on the left overbank looking downstream and was closed in 1983. The Elam Landfill is located immediately upstream of Loop 12 on the left overbank and was closed in 1980. The Linfield Landfill located on Linfield Road on the right bank of the Trinity River was closed in 1975. The Linfield Landfill is a significant influence to flood water surface elevations due to its close proximity to the river channel as well as fill extending above the 1 percent chance flood water surface. The landfill is located opposite the river channel from a natural narrowing of the left overbank. These conditions combine to create a significant encroachment of the floodplain at this location. The locations of the South Loop Landfill, the Elam Landfill, and the Linfield Landfill and their relationship to the limits of the 1 percent chance flood and the SPF floodplain are shown on Plates A-37 and A-38.

Sleepy Hollow Country Club Levee

The Sleepy Hollow Country Club Golf Course is located between the Linfield Landfill and the Loop 12 bridge on the right bank of Trinity River. A small levee approximately 10 feet in height is located along the right bank of the river channel and protects the golf course from approximately the 10 percent chance flood event based on observance of recent floods. For flows less than the 10 percent chance flood, the levee encroaches upon the main bridge opening of the Loop 12 bridge for about 50 percent of its length. The Loop 12 highway crossing of the floodplain consists of two additional relief bridges that are not effected by the golf course levee.

Dallas Floodway Levees

The Dallas Floodway Levee System is a federally sponsored project currently maintained by the City of Dallas. The Dallas Floodway Extension Study initially had a primary focus to evaluate current conditions and proposed improvements for those areas downstream of the Dallas Floodway that are susceptible to flood damages up to and including the SPF event. However, due to changes in the floodplain and the backwater effects on the downstream end of the Dallas Floodway Levees, the risk of overtopping of the Dallas Floodway Levees has become a major consideration. The design of the Dallas Floodway Levees was based on construction of the levee crest to the SPF water surface elevation plus four feet of freeboard. The SPF flood elevations used to establish the original design grade of the Dallas Floodway Levees were computed using hand backwater calculations. Subsequent studies have confirmed the original SPF flood elevations using an LRD-1 hydraulic model to compute water surface profiles. The most recent LRD-1 model was based on U.S.G.S. quadrangle map topography combined with surveyed cross-section data and estimated hydraulic roughness values from the 1960's. As a result, the hydraulic model compiled for this study, which has been updated for current conditions as accurately as possible, computes significantly higher water surfaces than those computed with the earlier model downstream of the Dallas Floodway.

The downstream end of the Dallas Floodway East and West Levees is located near the abandoned Atchison, Topeka, and Santa Fe Railroad Bridge. The East Levee has a terminal section that extends perpendicular to the river along the abandoned A.T. & S.F Railroad and directly beneath the newly constructed DART Rail Line Bridge to high ground. A portion of this extension of the East Levee is an earthen embankment with a design crest elevation of 425.2 feet and the remainder is a concrete flood wall up to 7 feet in height that extends the levee to high ground. The concrete flood wall portion of the levee has a design crest elevation of 423.0 feet and has two integral stop log closure sections. One stop log structure provides passage for a double track Union Pacific Railroad line. The other stop log structure formerly served the same purpose but the tracks have been removed as part of the construction of the DART Rail Line Bridge. For the purposes of this study the stop log structures have been assumed to be in place prior to the occurrence of a major flood event and reliable up to the flood wall design crest elevation of 423.0 feet.

The most recent topographic survey of the region was compiled from aerial photographs taken in February of 1991 and indicates that portions of both the East and West Levee crests have degraded below the design grade. However, the original design grade of the levees has been used to evaluate the frequency of overtopping based on the fact that, maintaining the levee crest height is a City of Dallas responsibility and the City was already implementing projects to address the problem of the degraded crest heights. The overtopping elevation chosen for the Dallas Floodway East Levee and used in the risk based analysis was based on the crest elevation of the concrete flood wall portion for plans without new levees. The current hydraulic study has computed a baseline condition SPF water surface elevation at the DART Rail Line Bridge of 426.0 feet and a 0.2 percent chance (500-year) water surface elevation of 422.4 feet.

A project undertaken by the City Of Dallas to improve the flow conditions for low flows within the Trinity River channel in the Dallas Floodway has been initiated. The project design provides for excavation (dredging) of the river channel and placement of the excavated material on the East and West Levees. The placement of the fill on the levees provides for restoration of the crest height to the original design grade plus two feet of overbuild. Additionally, it provides for greater side slope stability on the riverside by placing fill on a flatter slope. Phase 1 of the project, beginning downstream of the A.T. & S.F. Railroad bridge and extending upstream to the Houston Street bridge, has been funded and thus has been hydraulically modeled in the baseline conditions hydraulic model. The design does not provide for the raising of the extension of the East Levee under the DART Rail line bridge described above for either the embankment portion or the concrete flood wall portion. Therefore, under current conditions, the crest of the flood wall extension of the East Levee remains the critical overtopping point.

Computation of water surface profiles through the Dallas Floodway reflecting the backwater effect of the changed downstream conditions was performed to evaluate the existing levees risk of overtopping. When comparing the original design grade of the East and West Levees to the water surface profiles, a gradual increase in the levees height above the computed water surface profiles from the downstream end to the upstream end of the floodway was observed. This observation is the result of the assumption that within the floodway periodic maintenance has been performed as designed to maintain a consistent hydraulic roughness and no encroachments or other hydraulic changes have occurred within the floodway. The hydraulic analysis also reveals that conditions within the floodplain downstream of the Dallas Floodway have a strong influence on the performance of the floodway.

Rochester Park Levee

The Rochester Park Levee was designed and constructed prior to the completion of the current hydrologic and hydraulic analysis. The levee has been hydraulically modeled in the current Baseline Conditions hydraulic model but not included in the Existing Conditions Model as discussed under "Hydraulic Models" in Appendix A. The design of the levee was based on the SPF water surface computed from the previous LRD-1 hydraulic analysis discussed above plus four feet of freeboard. The LRD-1 hydraulic analysis was, at the time the levee was being designed, the most up to date hydraulic analysis available. The SPF water surface elevation at the upstream end of the levee yielded a design crest elevation of 417.0 feet. This design crest elevation was used for the entire levee crest. However, the upstream end of the Rochester Park Levee terminates at a natural ground elevation of approximately 415.5 feet. Based on the earlier hydraulic study this allowed for about two feet of freeboard above the SPF water surface at that location. The levee includes floodgate structures at the Central Expressway Service Road , the Bexar Street underpass at the C.F. Hawn Freeway, the Union Pacific Railroad underpass at the C.F. Hawn Freeway, and two levee crossings of the Union Pacific Railroad.

As originally designed, flood discharges exceeding the capacity of the levee system would initially enter the levee protected area upstream of the end of the levee across broad natural ground areas prior to a levee overtopping. Because no floodgate structure was constructed at the underpass of Hatcher Street and South Central Expressway, floodwater would enter the areas protected by the Rochester Park Levee at an elevation lower than the upstream end of the levee. The elevation at the underpass above which floodwater would begin to inundate those areas protected by the Rochester Park Levee north of the C.F. Hawn Freeway is estimated to be 413.0 feet. The elevation above which floodwaters would begin to inundate those areas south of the C.F. Hawn Freeway after floodwater had entered through the Hatcher Street underpass is estimated to be elevation 414.5 feet. A portion of the C.F. Hawn Freeway located north of the Rochester Park area forms a ridge that causes this difference in initial inundation levels for the two areas. The ground elevation of 413.0 feet at the Hatcher Street underpass was used as the critical overtopping elevation for evaluation of the existing Rochester Park Levee and used as input to the risk based analysis for determination of the residual damages and relative levee performance for baseline conditions. The current hydraulic study has computed a 1 percent chance (100-year) water surface elevation at Hatcher Street for baseline conditions of 412.0 feet and a 0.2 percent chance (500-year) water surface elevation of 418.1 feet.

Central Wastewater Treatment Plant Levee

The Central Wastewater Treatment Plant (CWWTP) is located on the right overbank of the Trinity River between the Missouri-Kansas-Texas Railroad bridge and the Interstate Highway 45 bridge. It is protected from flooding by a ring levee system that surrounds the treatment plant. The levee survived the flood of 1990 without overtopping but required emergency repairs during the flood. The City of Dallas has since implemented a plan to upgrade the CWWTP Levee and other plant facilities to comply with the Texas Water Commission requirements to provide 1 percent chance (100-year) flood protection plus three feet of freeboard. The levee improvement plan was designed by the engineering firm of Halff Associates, Inc., of Dallas. The results of the hydraulic analysis used to establish the design levee crest grade of elevation 415.0 feet compares very closely with the current baseline water surface profiles presented in this report. Elevation 415.0 feet was used as input to the risk based analysis for determination of the residual damages and relative levee performance for baseline conditions.

HYDRAULIC MODELS

General

The PC version 4.6 of the HEC-2 Water Surface Profiles computer program was used to hydraulically model and compute water surface profiles for this study. Several HEC-2 backwater models with differing input data sets have been used for this study. Initially HEC-2 models were produced using cross-sections obtained from the City of Dallas topographic maps developed in 1977 and was the most recent topographic information available at the time the model was prepared. When the topographic mapping used for the Upper Trinity River Feasibility Study became available later in the study, the decision was made to update the models with the more recent topographic data. Therefore, models for this study would be consistent with the HEC-2 models used for the Upper Trinity Feasibility Studies.

The City of Dallas topographic maps used for the "existing conditions" HEC-2 models developed initially were updated as much as possible to represent current conditions. The City of Dallas topographic maps were compiled from aerial photography flown in March 1977, and have a contour interval of two feet and a scale of one inch equals two hundred feet. Cross-sections for the model were taken directly from the topographic maps on average every 1,000 feet of river distance. Channel geometry was input from surveyed cross-sections used in previous Trinity River LRD-1 hydraulic models. The 1977 topographic maps were updated to reflect the contours of two City of Dallas landfills located in the floodplain of the Trinity River that were completed after 1977. One of these is the Elam Landfill located immediately upstream of the Loop 12 bridge on the left overbank and the other is the South Loop Landfill located immediately downstream of the Loop 12 bridge on the left overbank. Another landfill is located on Linfield Road and was completed prior to 1977 and was reflected in the City of Dallas topographic maps. Information relating to current conditions for the McCommas Bluff Landfill located near I.H. 20 was not available to update the 1977 topography. a calibration of this model was accomplished by the methods described under "Calibration Model" to closely match the May 1990 Flood. This model was used for initial plan formulation and the initial determination of the National Economic Development (N.E.D.) Plan.

In 1994, the existing conditions model discussed above was abandoned and a new model was created which was based on mapping made available as a result of the concurrent Upper Trinity Feasibility Study. Basic input data for the current model was obtained from cross-sections taken from digitized topographic mapping produced by photogrammetry. The cross-sections were taken electronically from the digitized mapping data rather than from topographic maps and contain ground points having elevations mapped to one tenth of one foot. The cross-section locations are identical to those used in the initial HEC-2 models. The mapping was compiled from aerial photography flown in February 1991. The mapping complies with National Map Accuracy Standards and has a vertical accuracy of plus or minus 0.5 ft.

The following description applies to the development of HEC-2 models derived from both sets of topographic data described above. Four highway bridges and three railroad bridges were modeled by the HEC-2 Normal Bridge method using the best available as-built bridge plans. The I-45 bridge was not modeled in the normal manner because of several factors. First, the bridge crosses the entire floodplain with no contraction of flows caused by the bridge abutments. Secondly, the bridge crosses the floodplain on an extreme skew making it impractical to model by usual methods. Thirdly, the low steel of the bridge is sufficiently high that it would not influence the highest flood flow that would be analyzed. Therefore, the pier losses were accounted for by the use of the Manning's roughness coefficient in each successive cross-section. Due to the broad and varied nature of the floodplain, "NH" records were used in the models to vary the Manning's roughness coefficients horizontally, to more accurately model the floodplain roughness.

The White Rock Creek confluence to the Trinity River and the low lying residential areas north of the Rochester Park Levee store significant volumes of flood water during major flood events. This created a need to compile separate HEC-2 models to calculate flood volumes. One model was used to compute water surface profiles by representing only conveyance areas of the floodplain. Another was used to compute storage volumes for the various floods under consideration so that peak discharges would be more accurately computed. This was done for both the initial HEC-2 model and the current one. The stage-discharge relationship of the conveyance model was retained during computation of the storage volumes by use of rating curve input to the model cross-sections.

Calibration Model

A recent major flood event occurring in May 1990 provided a reasonable basis for calibrating the HEC-2 backwater models because the flood was estimated to be the highest magnitude since 1942 and high watermarks were established for the study reach following the flood. When the Upper Trinity Feasibility Study topographic data became available, development of a common HEC-2 model to be used for each of the two concurrent studies was needed. Therefore, another model calibration was needed to establish the hydraulic roughness values in the floodplain consistent with the new topographic data. The 1991 topographic data represented hydraulic conditions at the time of the May 1990 flood sufficiently to be used without revision for the calibration. The following description of the model calibration applies to both the model derived from the 1977 topographic data and the model derived from the 1991 topographic data. The data given for Manning"s roughness coefficients and flow velocities are for the current model.

Initial Manning's roughness coefficients were estimated by field surveys, aerial photographs, and using the "Guide for Selecting Manning"s Roughness Coefficients for Natural Channels and Flood Plains" by Arcement and Schneider. Calibration of the hydraulic model was accomplished by using the U.S.G.S. gage data at both the Below Dallas Gage and the Dallas Gage. The Dallas Gage is located 90 feet downstream of the Commerce Street Bridge and the Below Dallas Gage is located at the downstream side of the Loop 12 bridge. Calibration of the model by adjusting the Manning's roughness coefficients resulted in a reasonable reproduction of the most recent major flood for the study area which occurred in May of 1990. Measured peak discharges and corresponding gage readings published by the U.S.G.S. were used as reference points. High watermarks for the May 1990 flood established by the Corps of Engineers and Halff Associates, Inc., at various locations in the study area were also used in the calibration of the model. The measured peak discharges published by the U.S.G.S for the May 1990 flood were 82,300 cfs at the Below Dallas Gage and 87,000 cfs at the Dallas Gage. Manning's roughness coefficients used in the study for the channel vary from 0.035 to 0.063 and range from 0.084 for open grassy areas to 0.210 for densely wooded areas in the overbanks. The channel flow capacity is approximately 6000 cfs. Computed channel flow velocities are in the range of 0.7 to 4.0 feet per second (fps) for the 50 percent chance event, 0.3 to 6.0 fps for the 10 percent chance event, 0.4 to 9.0 fps for the 2 percent chance event, and 0.5 to 10.0 fps for the 1 percent chance event. The calibration model water surface profiles and the high watermark locations are shown on Plates A-25 and A-26. Stage-Discharge rating curves for the Below Dallas Gage, the calibration model, and baseline conditions are shown on Plate A-21.

Baseline Model

The development of the Baseline model was based on the requirements of the Upper Trinity River Feasibility Study to have certain projects that influence the hydraulic and hydrologic conditions within the floodplain incorporated into the HEC-2 model to form a basis for future hydraulic studies within the Trinity River corridor. The following projects have been incorporated into the approved Upper Trinity River Feasibility Study Baseline Conditions HEC-2 models and modeled as completed per the design plans for each project.

Southside Sewage Treatment Plant Levee modification
McCommas Bluff Landfill and Swale
Rochester Park Levee
Central Wastewater Treatment Plant Levee modification
DART OC-2 Rail Line bridge
Dixie Metals Company Landfill
Dallas Floodway channel and levee modification (A.T. & S.F. Railroad bridge to Houston St. bridge)
Various small permitted fill areas

These projects are permitted fills or projects constructed or under construction following the development of the 1991 aerial photography and mapping which was the basic input for the baseline model. All landfills have been represented as completed. Water surface profiles for baseline conditions are shown on Plates A-29 and A-30.

Existing Conditions Model

Due to the configuration of proposed levees in the Recommended Plan, economic analysis of conditions prior to the construction of the Rochester Park Levee and the Central Wastewater Treatment Plant Levee Modification was necessary. Therefore, an "Existing Conditions" hydraulic model was compiled representing floodplain conditions prior to 1991 before either project was constructed. This model is essentially the same as the baseline model without the effects of the Rochester Park Levee and the CWWTP Levee Modification. Water surface profiles for existing conditions are presented on Plates A-27 and A-28.

NATIONAL ECONOMIC DEVELOPMENT PLAN

General

The N.E.D. Plan for the reduction of flood damages within the study reach calls for excavation of overbank swales within two sections along the Trinity River. The lower swale is located on the left overbank looking downstream and extends from about 2,000 feet downstream of Loop 12 to the oxbow river bend near the State Highway 310 (Central Expressway) bridge. The upper swale is located on the right overbank and extends from the upstream side of the Central Mitigation Swale adjacent to the Central Wastewater Treatment Plant to the confluence with Cedar Creek. The swale is designed to function as a grass-lined floodway to be maintained free of woody vegetation to provide an efficient means of conveying flood water. The swale design provides for ease of maintenance and to minimize negative impacts to the existing river channel and the local environment.

Both the upper and lower swales are designed to function similarly to a bypass channel with a very wide and shallow trapezoidal cross-section with low hydraulic roughness characteristics. The trapezoidal swale cross-section has a flat bottom with side slopes of a minimum four horizontal to one vertical (4H:1V). Excavation depth of both the upper and lower swales ranges from zero to 14 feet with the average depth at about three feet. Both the upper and lower swales are designed on a longitudinal slope of 0.05 percent for most of their length. This slope is consistent with both the average slope of the natural channel bottom and the average downstream slope of the overbanks. Hydraulic efficiency of the floodplain is improved primarily by the reduction of the existing hydraulic roughness in the areas where the swale is located. The swale also increases hydraulic efficiency by the addition of floodplain conveyance area. Additionally, the swales increase hydraulic efficiency by providing for more uniform flow due to the removal of floodplain ridges and filling of low areas. The uniform slope of the excavated swale is designed to be free draining for efficient conveyance of local runoff and receding flood waters as an aid to providing effective seasonal maintenance. Water surface profiles for the N.E.D. Plan are shown on Plates A-31 and A-32. Average flow velocities for the N.E.D. Plan for the 1 percent chance and SPF floods are provided in Table A-10.

Lower N.E.D. Swale

The downstream limit of the lower swale at the centerline is located about 2,200 feet downstream of the Loop 12 bridge. The downstream end terminates at an approximate elevation of 386.0 feet. The downstream limit of the swale is along an existing tree line break adjacent to an existing dirt road. The tree line break is skewed 45 degrees to the swale centerline. Dense cover of mature trees exists downstream and less dense vegetation is located upstream of the dirt road. The swale excavation and clearing was limited to this downstream location because very little additional benefits would be achieved by continuing the swale farther downstream. Significant impacts to the higher quality habitat areas downstream was also reduced by limiting the swale at this location.

From the downstream end, the swale has a bottom width of 800 feet and extends upstream on a 0.14 percent adverse down slope approximately 1,100 feet to a natural tributary crossing. The tributary conveys local runoff to the main Trinity River channel. The slope of this portion of the swale is adverse to the flow of the river at flood stage and serves to provide good drainage for local runoff without negatively effecting the performance of the swale during a flood event. From this tributary crossing, the 800 foot bottom width swale extends upstream on a slope of 0.05 percent through the largest of two relief bridges for the Loop 12 highway crossing to a point about 4,500 feet from the downstream end of the swale. The extension of the swale downstream of Loop 12 is designed to significantly increase flow through the relief bridge. Loop 12 crosses the Trinity River floodplain and conveys flood water by means of three bridge structures. The main bridge structure crosses the river channel and is approximately 1,340 feet in length. Its flow capacity is reduced for the more frequent flood events by the Sleepy Hollow Country Club levee located immediately upstream from the bridge. The largest of the two relief bridges is located 470 feet from the main bridge eastward along Loop 12 and is 800 feet in length. Dense vegetation exists immediately upstream and downstream of this relief bridge. Another relief bridge is located eastward along the highway about 1,200 feet from the 800 foot relief bridge and is 200 feet in length.

Several ponds of water are located in the area upstream of Loop 12 near the Elam Landfill within the proposed boundaries of the lower swale. The portions of these ponded areas that lie within the swale are to be filled to the design grade of the swale bottom and turfed in like manner as the excavated areas of the swale. The portions of the ponded areas that lie outside the boundaries of the swale are to remain.

From a location about 4,500 feet from the downstream end of the lower swale, the swale extends upstream on a 0.05 percent slope and begins the transition from 800 foot bottom width to 1,200 foot bottom width over a distance of 2,000 feet. The 800 foot bottom width portion of the swale has been designed to extend upstream sufficiently to avoid excavation of the documented landfill areas located upstream of the Loop 12 bridge. From the upstream end of the 800 foot to 1,200 foot transition, the swale extends upstream on a 0.05 percent slope and has a 1,200 foot bottom width to its upstream limit at the oxbow river bend near the S.H. 310 bridge. The White Rock Creek channel crosses the proposed swale at about 10,000 feet upstream from the downstream end of the swale. Excavation of the swale on the design grade is continuous through the creek banks on both sides. Otherwise, the creek is to remain in its natural meandering condition across the swale. Erosion resistant grasses are to be maintained on the creek bank slopes and Willow trees and other woody type vegetation are to be removed from the upper creek banks at least annually.

The design requires excavation in the vicinity of the bridge piers that are located within the swale for the three bridges that cross the lower swale. Excavation at the Loop 12 relief bridge piers is required to a depth of about 3 feet. The Southern Pacific Railroad bridge crosses the lower swale near the upper end of the swale and excavation around the bridge trestle pilings is required to a depth of about 2 to 3 feet. Excavation around the bridge piers within the swale for the S.H. 310 bridge is required to a depth of 3 to 5 feet. Bridge piers or pilings located within the swales are protected from possible increased risk of scour damage or strengthened where it is deemed necessary.

The upstream limit of the lower swale is excavated on the design grade through the left bank of the Trinity River at the centerline elevation of 392.7 feet. Total centerline length of the lower swale is about 17,500 feet. Channel flow would initially divert to the upstream end of the lower swale at the approximate bank full capacity of 6,000 cfs.

Upper N.E.D. Swale

The downstream limit of the upper swale is located along the upstream bank line of the existing Central Mitigation Swale adjacent to the Central Wastewater Treatment Plant. The downstream end of the swale at the centerline is at elevation 392.5 feet and meets the approximate natural ground surface at that location. The design of the downstream limit of the upper swale and the discontinuity of the upper and lower swale is based on several factors. The area immediately downstream of the I.H. 45 bridge to the right bank of the river has a dense mature tree cover consisting mostly of hardwood trees. This overbank area between the upper and lower swale has been determined to have a high value in terms of natural habitat and aesthetics. Even though this area creates a significant resistance to the flow under flood conditions, its natural condition should be preserved. In addition, the presence of the dense vegetation in this area may have some benefit to preserving the natural condition of the channel under flood conditions by acting as a buffer for flows crossing the river channel at a right angle to the channel at the oxbow bend. The lower swale has been designed to extend upstream through the Southern Pacific Railroad Bridge and the State Highway 310 bridge to the river bank to improve flow capacity through the bridges and to compensate for the high floodplain roughness created by the dense vegetation in the river oxbow area. The ground surface beneath the I.H. 45 bridge has been maintained clear of vegetation and provides for adequate access to the area for maintenance.

The downstream end of the upper swale has a bottom width of about 1,100 feet and extends upstream on a slope of 0.05 percent to the upstream limit of the upper swale at the confluence of Cedar Creek and the Trinity River. The portion of the swale extending from the upstream bank line of the existing Central Mitigation Swale to about 1,400 feet from the downstream end varies in bottom width from 1,000 feet to 1,200 feet due to space limitations between the CWWTP levee and the river channel. Another portion of the upper swale located about 3,200 feet from the downstream end is reduced in bottom width to about 900 feet due to the sharp river channel bend extending southwesterly into the swale.

The Missouri-Kansas-Texas Railroad trestle bridge crosses the upper swale at about 4,000 feet upstream from the downstream end of the swale. Excavation required beneath the trestle is estimated to be a depth of less than one foot for the entire length of the trestle. This trestle crosses the right overbank of the Trinity River floodplain and is approximately 1,200 feet in length. Another M-K-T Railroad Bridge spanning the river channel is approximately 320 feet in length is not effected by the swale. Excavation beneath the Martin Luther King Boulevard Bridge is estimated to be a depth ranging between zero and 2.5 feet. This bridge crosses the upper swale at about 5,900 feet upstream of the downstream end of the swale.

The bottom elevation at the upstream limit of the upper swale is 396.0 feet at the centerline of the swale. The upstream limit will intersect the banks of both the Trinity River and Cedar Creek near the confluence. The natural alignment of either of the two channels in the confluence area will not be changed by construction of the swale. The total centerline length of the upper swale is about 7,000 feet. Channel flow would initially divert to the upstream end of the upper swale at the approximate bank full capacity of about 7,000 cfs.

Local Drainage

The upper and lower swales are in a trapezoidal shape and are designed to a grade generally lower than the natural ground. This creates side bank slopes that potentially are at risk from erosion damage if significant concentrations of flows from local runoff are allowed to flow over the top of the banks. The side slopes of the swale are designed with four horizontal to one vertical (4H:1V) side slopes. An analysis of upland drainage areas on the sides of the proposed swale has produced the following observations and recommendations.

Two conditions where local runoff could flow over the banks of the swale are at the beginning of a rainfall event and when the flood water recedes from the floodplain. At the beginning of a flood event, the inundation of the swale to a depth of several feet would be expected before there would be significant local runoff over the swales banks. This is due to the location of the swales bottom grade at near the top of bank elevation along the natural channel and the very large drainage area upstream of the project reach. The design grades of the upper and lower swales are such that excavation of the swales will result in side bank heights for most of the swales length ranging from zero to four feet. This would result in a head differential of flow from the overbanks of the swale to the water surface in the swale that is negligible for most of the swales length.

An area of the lower swale with a side bank height significantly higher is located on the left bank of the lower swale between 4,500 feet and 7,200 feet from the downstream end. This area has a bank height varying from 11 to 15 feet. To prevent possible overbank flow from local drainage areas above the top of bank in this reach of the lower swale, the side slope will be extended above the natural ground to create a small berm. The reach of the lower swale located between 4,500 feet and 6,200 feet from the downstream end will have a left top of bank minimum elevation of 398.0 feet. The reach of the lower swale located between 6,500 feet and 7,200 feet from the downstream end has a left top of bank minimum elevation of 403.0 feet.

The natural channels crossing the swales such as the White Rock Creek channel provide for complete drainage of the swales following a flood event. Floodwaters recede very gradually within the floodplain following a flood event; therefore, no significant erosion of cross channel banks or swale banks is expected.

RECOMMENDED PLAN

General

The Recommended Plan for reduction of flood damages within the study reach is for excavation of wetland swales in a longitudinal configuration paralleling the Trinity River on the right overbank extending from Loop 12 upstream to the confluence of Cedar Creek and the Trinity River. The wetland swales are comprised of a series of wetland cells linked closely together and referred to as the Chain of Wetlands. The Recommended Plan also includes construction of earthen levees and/or flood walls on both sides of the Trinity River. The proposed Lamar Street Levee is designed to provide SPF flood protection for portions of the industrial and residential development downstream of the existing Dallas Floodway Levees. The Lamar Street Levee will be located on the left bank of the Trinity River between the Dallas Area Rapid Transit (DART) Rail Line Trinity River bridge and the Southern Pacific Railroad Trinity River bridge. The proposed Cadillac Heights Levee is designed to provide SPF flood protection for the industrial and residential areas located on the right bank of the Trinity River from the confluence of Cedar Creek to the Central Wastewater Treatment Plant and to high ground near Kiest Boulevard. Water surface profiles for the Recommended Plan are shown on Plates A-33 and A-34. Water surface profiles comparing the Recommended Plan to the Baseline Conditions are shown on Plates A-35 and A-36. The floodplain areas for the 1 percent chance flood and the SPF comparing the Baseline Conditions with the Recommended Plan are shown on Plates A-37 and A-38. A plan view of the Recommended Plan may be found in Appendix C - Civil, Structural, and Relocations Engineering.

Chain of Wetlands

The Chain of Wetlands portion of the Recommended Plan has been designed to reduce flood damages by increasing the overall hydraulic efficiency of the floodplain. The Chain of Wetlands swales function as a "floodway" by performing two primary functions relating to the conveyance of flood water. First, the design of the Chain of Wetlands swales provides a flow zone where areas of dense vegetation having high hydraulic roughness characteristics are replaced with vegetation having a much lower resistance to the flow. Secondly, the excavation of the swales provides for increased flowage area by the removal of soil from the floodplain. The swales have been designed to be generally continuous and aligned along the flowline of the river to enhance both hydraulic functions. The effect of the increased efficiency is that flood water moves through the project area at a slightly faster rate as the flood wave passes, thereby lowering the maximum water surface at the peak of the flood. The term "swale" is used to describe an excavated flow path having a very low depth to width ratio. A typical channel has a much higher depth to width ratio. The wetlands, which have been designed to have consistent water levels for long periods, are generally only 2 to 3 feet below the natural ground surface and several hundred feet in width. Therefore, the wetland swales will have the appearance of a slight depression rather than the appearance of a typical channel.

The Chain of Wetlands is divided into upper and lower reaches within the study area by the oxbow river bend between S.H. 310 and I.H. 45. The lower portion of the Chain of Wetlands extends from the upstream side of the Loop 12 bridge to the downstream side of the I.H. 45 bridge on the right overbank looking downstream. The lower portion of the Chain of Wetlands has been aligned through the Linfield Landfill and the Sleepy Hollow Country Club Golf Course to provide for a shorter and more efficient flow path for floodwater. The Linfield Landfill is located on the inside of a natural river bend where flood water is currently forced to follow a long path around the landfill through dense vegetation on the opposite side of the river. The excavation of the swale through the landfill results in excavation depths of up to 30 feet, but elsewhere the maximum excavation depth is 10 to 13 feet. The excavation through the Linfield Landfill has been designed with a swale width of about 500 feet at elevations consistent with the natural ground surface upstream and downstream of the landfill. Grass-lined side slopes on a 4H:1V slope will be utilized through the landfill to facilitate maintenance. The portion of the proposed lower Chain of Wetlands swales located upstream of the Linfield Landfill and downstream of the Southern Pacific Railroad bridge will be adjacent to an existing wetland swale previously constructed for mitigation of the Central Wastewater Treatment Plant Levee Improvement Project. This proposed swale and the existing swale will be separated by an approximate 120-foot wide strip of existing forested land. Another existing wetland swale lies between the S.H. 310 bridge and the I.H. 45 bridge on the right bank of the river. Neither of these existing wetland swales will be modified or included in the management scheme for the proposed wetlands. The most upstream wetland cell of the lower Chain of Wetlands is approximately 300 feet in width and 1,100 feet in length. This wetland cell is located along the downstream right-of way line of the I.H. 45 bridge on the right bank of the river. This cell is separated from the remainder of the lower Chain of Wetlands by the oxbow river bend of the Trinity River.

The upper portion of the Chain of Wetlands extends from the upstream side of the I.H. 45 bridge to the confluence of Cedar Creek and the Trinity River. The proposed upper wetland swales extend from the upstream bank line of the Central Mitigation Swale to the bank of Cedar Creek. The CWWTP effluent discharges into the Central Mitigation Swale and outflows to the river channel beneath the I.H. 45 bridge. Water from the CWWTP in the Central Mitigation Swale will be used to periodically supply water to the wetlands by means of a pumping facility. The normal water level of the Central Mitigation Swale is elevation 382.5 feet. The discharge of water from the pumping facility to the upper wetlands will be at elevation 392.0 feet to the wetland cell at the downstream side of the I.H. 45 bridge and at elevation 394.0 feet to the wetland cell that is upstream of the Central Mitigation Swale. The pumping facility will be used to re-supply the wetland cells during dry periods and following periodic draw down of the wetlands for maintenance and management.

The wetland swales are divided into wetland cells to allow management of the water levels in the wetlands in order to maximize the environmental functions of the wetlands. Wetland cells are divided at each location where the swale intersects a bridge structure so that no excavation in the vicinity of the bridge piers will be required. However, the area beneath the bridge in the vicinity of the swale is considered a functional part of the swale and is to be maintained clear of woody vegetation. Each wetland cell is excavated to create a variety of water depths for desirable wetland vegetation. A typical cross section of the wetland swales and the design Manning"s hydraulic roughness values for the various zones within the wetland swale is shown on Plate A-20. The Manning"s hydraulic roughness values shown for the various swale zones are used in the Recommended Plan HEC-2 hydraulic model and are based on the varying types of vegetation in the swale. Each wetland cell has a specified design water surface elevation and is controlled by a small outlet structure with a simple stoplog weir. The stoplog weir design allows for the draw down of each cell up to 3 vertical feet below the design water level and allows the option of controlling the water levels at increments between 0 and 3 feet below the design water level. The stoplog weir structure also provides for energy dissipation for flow into or out of the wetland cell. Transfer of water from cell to cell will be by means of a subsurface 36 inch diameter reinforced concrete pipe (RCP). The outflow pipe from each wetland cell will discharge to either the next downstream wetland cell or existing creeks depending on the cell location. There are no direct flow connections between the proposed wetland cells and the Trinity River channel. Distribution of water at low river conditions for the upper Chain of Wetlands from the pumping facility will be downstream to the wetland cell located adjacent to the I.H. 45 bridge and upstream to the wetland cell located upstream of the Central Mitigation Swale. Water supplied to the wetland cell adjacent to the I.H. 45 bridge will flow downstream to the other wetland cells in the Lower Chain of Wetlands by means of a 36-inch RCP. Outflow from the lower Chain of Wetlands at low river conditions will be directly into Honey Springs Branch. Water supplied to the wetland cell upstream of the Central Mitigation Swale will flow in the upstream river direction to the other wetland cells extending to Cedar Creek. Outflow from the upper Chain of Wetlands at low river conditions will be directly into Cedar Creek. The wetlands cells in the upper portion of the Chain of Wetlands between the Central Mitigation Swale and Cedar Creek have been designed with descending water levels in the upstream river direction in order to take advantage of the consistent water source available at the Central Mitigation Swale.

Lamar Street Levee

The Lamar Street Levee will extend from the existing Rochester Park Levee at the downstream side of the Southern Pacific Railroad bridge to the existing Dallas Floodway East Levee at the DART Rail Line Bridge. The Lamar Street Levee will become an extension of each of the existing levees at the points of juncture and become an integral part of both existing earthen levees to provide SPF flood protection for residences and business along Lamar Street and the Rochester Park area. The alignment of the Lamar Street Levee has been designed to preserve as much of the natural forest along the river channel as possible and provide flood protection for most of the businesses located along the riverside of the Southern Pacific Railroad. Floodgates will be required at the levee intersection of the Southern Pacific Railroad and the M.K.T. Railroad. The levee has been designed with turn-back sections at the Martin Luther King Boulevard intersection in order to tie the levee crest to the highway embankment at the levee design grade. This alignment eliminates the need for an additional floodgate at the M.L.K. Blvd. bridge abutment. The levee crosses S.H. 310 near the Trinity River bridge north abutment where there is currently a grade separation of about 4 feet between the south bound lanes and the northbound lanes. The southbound lanes are at an elevation approximately 1 foot higher than the design levee crest elevation at that point. Therefore, the levee crest has been designed to tie into the highway embankment on the upstream side of the bridge abutment. The northbound lanes are currently about 3 feet below the crest design grade of the levee. The Texas Department of Transportation (TXDOT) has indicated that a replacement of the older northbound S.H. 310 Trinity River bridge is in the planning stages. The northbound bridge and approaches to the bridge are to be replaced at approximately the same elevation as the southbound bridge. The current design for the northbound bridge approach closure to the levee design grade is by emergency flood fighting methods, such as sandbagging, pending further development of the bridge replacement by TXDOT.

The Lamar Street Levee crest design grade has been set at 2 feet above the SPF water surface for "Baseline" conditions with the Lamar Street Levee, the Cadillac Heights Levee, and the Chain of Wetlands in place. The design crest height of the Lamar Street Levee was not based on a particular "freeboard" requirement. However, the levee height is reasonably optimized because the design satisfies the basic requirements of providing a reasonably low risk of overtopping by the SPF while taking advantage of the full protection potential of existing levees without incurring significant costs to raise or modify them. It should be noted that, while the computed design water surface profile must be used as a guide for establishing the design crest profile for the levee, the design water surface is not an absolute. The design water surface profile is regarded as a "most likely" value derived from best estimates of key factors, parameters and data components that have some inherent variability or uncertainty. This most likely value of the flooding level is used in the risk based analysis along with probability distributions of the key parameters and data components which may take on a range of values. Information relating to performance and probability of overtopping for this levee height is presented in Appendix D - Economic Analysis. The design grade of the Lamar Street Levee at the juncture with the existing Rochester Park Levee is elevation 417.0

feet. The design grade of the proposed levee at the juncture with the existing Dallas Floodway East Levee is elevation 426.5 feet. The Lamar Street Levee will be constructed of earth fill with the exception of the floodgate structures at the two railroad crossings. Compacted impervious fill will be placed to the height of the design grade of the levee and a minimum 8 inches of road base material will be placed above this level. The levee design crest is defined as the top of the road base material. The proposed levee has a crest width of 20 feet and side slopes of 4 horizontal to 1 vertical (4H:1V).

The Recommended Plan floodplain area for the 1 percent chance flood and the SPF is shown on Plates A-37 and A-39. The floodplain area located north of U.S. Highway 175 (Central Expressway and C.F. Hawn Freeway) receives significant flood damage reduction benefits by the Recommended Plan in terms of frequency of flooding. However, some of this area will remain subject to the SPF at elevation 414.8 feet because Trinity River flood water will pond back into this area from the lower portion of the White Rock Creek drainage basin. This is due to the present alignment of the downstream end of the Rochester Park Levee. The levee extends westward along the C.F. Hawn Freeway and ties to high ground at the floodgate at Bexar Street and the C.F. Hawn Freeway. The Trinity River SPF water surface elevation of 414.8 feet is the computed Recommended Plan water surface elevation at the downstream limit of the existing Rochester Park Levee. Because low lying areas north of the Central Expressway and the C.F. Hawn Freeway are below the 414.8 feet elevation, the Hatcher Street underpass at Central Expressway would be subject to the SPF with the Recommended Plan. The approximate street level at the Hatcher Street underpass is elevation 413.0 feet. Therefore, some method of closure of the underpass up to the SPF water surface of 414.8 feet is required to prevent floodwater from entering the Lamar Street Levee protected area. The most appropriate method of closure for the underpass is sandbagging because of the relatively low height of about 2 to 3 feet needed to contain the SPF. The length of closure required across the underpass would be approximately 160 feet making the use of permanent floodgates impractical. The sandbagging effort would be a rare occurrence since the flood event required to reach the underpass from backup flooding would be greater than a 500-year event. Consideration was given to extending the downstream end of the Rochester Park Levee to high ground to provide SPF flood protection to the predominately residential structures remaining in the SPF floodplain. Preliminary investigation of potential levee extension alternatives for this area has indicated that these alternatives would not be economically feasible. This conclusion is based on the low number of structures remaining in the more frequent flood zones, the length of levee required to tie to high ground, the high cost of providing for relocation of structures along the levee alignments and interior flood protection requirements.

A preliminary plan to provide for initial overtopping of the Lamar Street Levee in the least hazardous location has been developed. The Plan complies with the guidelines of Engineer Technical Letter No. 1110-2-299 titled, "Overtopping of Flood Control Levees and Floodwalls". The plan is designed to prevent a sudden failure or washout of the earth embankment due to overtopping of the levee in a particularly hazardous location if a levee overtopping is determined to be unavoidable due to a flood event greater than the SPF. The plan provides for controlled inundation of the levee-protected area in the event of an imminent overtopping. The design of the levee requires a low level flood protection effort in the form of sandbagging at the Hatcher Street underpass as described above to allow the SPF to pass without damaging the levee protected area. The sandbagging effort would be used to provide an easily controlled initial access point for flood water into the levee protected area if a levee overtopping is determined to be unavoidable. The Hatcher Street underpass will serve to localize flow into the levee-protected area and the street surfaces will minimize erosion potential at the sandbagged release point. Since floodwater access to the levee protected area through the Hatcher Street underpass is backup flooding from the downstream end of the levee, the flood level could potentially pond up to elevation 417.0 feet prior to overtopping of the earth embankment at some upstream point. A potential levee overtopping head differential between the interior ponded level of elevation 417.0 feet and the crest of the levee at the upstream end of the Lamar Street Levee near the DART Rail Line bridge is nine feet. Therefore, a notch 1 foot lower than the design crest of the Lamar Street Levee will be constructed at approximately 600 feet downstream of the DART Rail Line bridge in order to force overtopping at the least damaging point near the upstream end of the levee. The weir notch will have a concrete crest and a mildly sloping riprap chute on the upland side of the levee to allow flood water to enter the protected area gradually, thus preventing a sudden washout of the levee. The notch and the chute will be located at the point of the highest natural ground elevation of approximately 420.0 feet to minimize erosion potential.

Cadillac Heights Levee

The Recommended Plan Cadillac Heights Levee extends easterly from the intersection of Kiest Boulevard and McGowan Street to the existing Central Wastewater Treatment Plant Levee. The levee also extends from the northwestern end of the CWWTP Levee and along the Texas Utilities power line easement to high ground near the intersection of 11th Street and Avenue J. The levee will require floodgate structures at two crossing locations on the M.K.T. Railroad, the railroad spur service track to the CWWTP, and at Martin Luther King Boulevard. A portion of the Recommended Plan Cadillac Heights levee alignment coincides with a portion of the existing alignment of the CWWTP levee. This portion of the proposed levee will raise the existing levee to the design height and the widening of the base of the levee will be on the outside of the existing CWWTP ring levee. The CWWTP entrance roads will be relocated to convey traffic over the design crest of the levee.

The Recommended Plan Cadillac Heights Levee crest design grade has been set at 2 feet above the same SPF water surface elevations as described above for the Lamar Street Levee. The levee height was based on, requests by the local sponsor and the public to provide equal flood protection on both sides of the river. Information relating to performance and risk of overtopping for this levee height is presented in Appendix D - Economic Analysis. The design grade of the Recommended Plan Cadillac Heights Levee, extending from the south end of the levee at the intersection of Kiest Boulevard and McGowan Street to the northeast corner of the CWWTP levee is elevation 421.5 feet. The design grade of the levee at the upstream end near the intersection of 11th Street and Avenue J is elevation 426.0 feet. The Cadillac Heights Levee will be constructed of earth fill with the exception of the floodgate structures. Compacted impervious fill will be placed to the height of the design grade and a minimum 8 inches of road base material will be placed above this level. The levee design crest is defined as the top of the road base material. The proposed levee has a crest width of 20 feet and side slopes of 4 horizontal to 1 vertical (4H:1V).

A preliminary plan to provide for initial overtopping of the Cadillac Heights Levee in the least hazardous location has been developed. A weir notch 1 foot lower than the design crest of the levee will be located at the terminal end of the levee near the intersection of Kiest Boulevard and McGowan Street. The notch will be 200 feet in length and will allow flood water to enter the levee protected area gradually to prevent a sudden washout of the levee if it is determined that a levee overtopping is unavoidable due to a flood event greater than the SPF. Early warning of an eminent inundation of the protected area will be sufficient to facilitate a complete evacuation of the protected area.

Local Drainage

The local drainage design features related to the Chain of Wetlands are minimal since most of the excavated slopes are very gradual. The exception to this is the excavation through the Linfield Landfill which has 4H:1V side slopes extending out from the swale. Concentration of local runoff to the top of the 4H:1V side slopes is expected to be minimal; therefore, no collector ditches at the top of the slopes through the landfill are required. Local runoff from the interior of the Cadillac Heights Levee will be conveyed by means of short grass-lined channels from the sluicegate outlet locations in the levee to the wetland cells adjacent to the levee. Local runoff from the interior of the Lamar Street Levee will be conveyed to the river channel by utilizing existing runoff drainage channels.

Sedimentation

There has been no significant channel migration, bank stability problems, or erosion documented in the last fifty years within the project reach that would indicate that there has been a net loss of sediment from the project reach transported either by normal daily flows or by flood events. The apparent stability of the channel in the project reach and minor changes in the overbank topography in undisturbed areas also indicate that any sediment being supplied from the Dallas Floodway area or from the White Rock Creek drainage area is being transported through the project reach without significant deposition. The upper and lower Chain of Wetlands swales have been designed to function only when the river is at flood stage, and therefore will have a very minor effect on the hydraulic characteristics of the natural river channel at flows less than the 50 percent chance (2-year) flood. Flows through the swales during a flood event will have higher velocities than under existing conditions and will reduce the chance of deposition of suspended sediments. However, flow velocities are not high enough to cause an increased risk of erosion. Average flow velocities for the 1 percent chance flood and the SPF are provided in Table A-11.

Risk and Uncertainty Analysis

Risk and Uncertainty Analysis was performed for the three existing major levee systems and the proposed Recommended Plan Levees discussed above. The purpose of the analysis is to provide a measure of the uncertainty of the performance of the levees as they relate to various alternatives that have been considered. This information is used to aid in the determination of acceptable risk and selection of a plan.

A component of this analysis is the stage-discharge uncertainty and is represented in the analysis by the stage-discharge rating and the standard deviation of the computed water surface elevations. The stage-discharge ratings used in the analysis were computed at the selected index points for the three levees under consideration. The index locations used in the risk analysis are at river station 998+00 for the Rochester Park Levee, at river station 1,011+38 for the Central Wastewater Treatment Plant Levee and the Cadillac Heights Levee, and at river station 1,083+80 for the Dallas Floodway East Levee. Plates A-22, A-23, and A-24 show the stage-discharge rating curves for existing conditions, baseline conditions, and the Recommended Plan at these respective locations.

The water surface profile analysis was performed using cross-sectional data taken from topographic data having an estimated accuracy of plus or minus 0.5 feet. The model was calibrated to the Trinity River Below Dallas Gage and to high watermarks for the 1990 flood event. The calibration results indicated that all of the high watermarks are within 0.7 feet of the 2 percent chance flood profile and the majority of them are within 0.3 feet, as shown on Plates A-25 and A-26. The calibration of the model for the range of frequencies from the 50 percent chance (2-year) flood to the 2 percent chance (50-year) flood at the gage was considered to be good. A comparison of the computed rating curve with the Below Dallas Gage rating curve is shown on Plate A-21. A minimum standard deviation from Appendix A, Stage-Discharge Uncertainty Section, Table 1, of the draft engineering circular "Risk Analysis Framework for Evaluation of Hydrology/Hydraulics and Economics in Flood Reduction Studies" is estimated to be 0.6 feet. A sensitivity analysis to estimate upper and lower limits for a range of flood events was performed by adjusting the Manning's roughness coefficients. The result was about 1 foot difference between the limits and the computed profile for flood events of the 2 percent chance flood and below. This range was estimated to encompass 95 percent of the entire population of measured gage reading data points that could be expected for the Below Dallas Gage at Loop 12. The maximum discharge recorded at the gage is about a 2 percent chance flood discharge. The difference between the upper and lower limits and the computed profile for the 1 percent chance flood, the 0.2 percent chance flood, and the SPF through the project reach ranges from 1.5 to 2.0 feet. Based on these results, a standard deviation of one foot for the SPF frequency stage was adopted, since this frequency was of primary interest in the analysis.

Care of Water During Construction

Excavation of the swales should proceed from downstream to upstream relative to the design flow of water from cell to cell following completion of the outlet control structure at the discharge point to the tributary creeks. This will provide for efficient drainage of the work site following a local rainfall event or high river conditions to minimize construction delays and costs to dewater the site. Temporary flow control devices on the outlet structures discharging to the tributary creeks shall be used to prevent backflow into the swales in the event of high river conditions during construction. Turfing and planting of wetland vegetation on the excavated swale shall be established as soon as practical within seasonal limits following completed excavation of each wetland cell. Construction of each wetland cell shall be completed prior to commencing construction of the next upstream wetland cell. Downed trees, cleared brush, or other debris loosened from the floodplain will not be stored within the floodplain. These materials will be removed from the floodplain as soon as practical following clearing and grubbing operations to prevent the possible blockage of bridge structures during a flood event. Any suspected hazardous or toxic materials discovered during the excavation and construction of the project will be reported to the Corps of Engineers District Office personnel to ensure proper removal and disposal of such materials.

Surveillance Plan

The local sponsor will be furnished an Operation and Maintenance (O&M) manual. This manual will provide information showing the requirements of CFR Section 208.10, and will also contain copies of the construction plans which show the features along the project such as bridges and roadway surfaces. The plan and profile sheets show the horizontal and vertical control with reference bench marks. The O&M manual will also specify that the local sponsor will appoint a superintendent who will have responsibility for maintenance of the project. The manual will provide that annual maintenance inspections will be performed which will include participation by a representative from the Corps of Engineers District Office. This inspection will include evaluation of such items as mowing requirements and repair of damages experienced by the project in recent flood events. In addition to the annual inspections, inspections will be made after each significant flow event, and eroded reaches of the project will be repaired. CFR 208.10 also requires that any improvement passing over, through, or under the floodway must be given prior approval by the District Engineer.

Plan for Relocation of the Trinity River Channel at I.H. 45

The Interstate Highway 45 bridge over the Trinity River was constructed following the Authorized Plan of improvement which provided for a 250-foot bottom width navigation channel to be constructed on the right overbank of the river near the point where the bridge would cross the river. In order to accommodate the requirements for the navigation channel, the bridge was designed with three continuous steel girder spans of 320 feet, 480 feet, and 320 feet measured along the centerline of the bridge and centered over the proposed navigation channel. The pier bents are skewed 50 degrees to the centerline of the bridge so that the piers would be parallel with the navigation channel and the flow of the river. The resulting pier bent spacing for the 480-foot span and the 320-foot outside spans normal to the flow are respectively 308 feet and 205 feet. The bridge beam spans over the existing river channel are 78 feet in length and have the same 50 degree skew to the centerline of the bridge. The resulting pier bent spacing for the bridge over the existing river channel normal to the flow is about 50 feet. The bridge was designed to accommodate construction of the large navigation channel and also retain the existing river channel. However, most of the river flow would have been carried through the navigation channel and a significantly reduced amount of flow would have been carried through the existing channel. The navigation channel was never approved for construction, but since the completion of the bridge, the Texas Department of Public Transportation (TXDOT) has experienced significant maintenance and repair costs due to floating debris accumulation and large trees striking the piers in the existing channel. The pier bent spacing at the current channel location was not designed to accommodate all of the normal river flows in the existing channel. The top width of the existing river channel is about 200 feet and several pier bents are located within the channel.

Interstate Highway 45 has been designated as a major transportation corridor for national defense, and TXDOT has considered replacement of the bridge spans over the existing channel as a solution to the ongoing maintenance costs and to provide long-term integrity of the structure. Alternatively, TXDOT has proposed a plan to relocate the existing river channel to pass normal river flow beneath the existing 320-foot bridge span that is located nearest the existing river channel. A plan to relocate a portion of the existing river channel has been designed to accomplish these goals at a significantly lower cost than replacement of the short bridge spans. The plan calls for realignment of about 3,300 feet of the existing river channel. The proposed channel has a trapezoidal cross section with a 30-foot bottom width, 3H:1V side slopes, and a top width of approximately 180 feet. The existing river channel in the reach where the realignment is proposed has an average bottom slope that is nearly zero. Therefore, the proposed channel realignment section has been designed with a zero bottom slope from beginning to end. The proposed channel has an average depth of 15 feet and has been designed to closely approximate the channel flow capacity and the flow velocities of the existing river channel. The proposed channel alignment will be centered between the nearest 320-foot span of the I.H. 45 bridge which has a face to face clearance distance between the piers of about 200 feet normal to the flow. Excavation around the piers will not be required. The proposed realignment will result in the channel being moved laterally a maximum distance of about 350 feet. The existing channel will be filled to the existing top of bank elevation 396.0 feet, to prevent further collection of debris. Relocation of the channel will result in modifications to the existing Central Mitigation Swale. The Central Mitigation Swale will be reduced in size by filling of the portion of the swale near the proposed channel realignment. A minimum of 150 feet from the top of bank of the proposed river channel realignment to the top of the bank of the Central Mitigation Swale, will be required.

Several alternatives regarding filling of the old river channel have been investigated. The investigated alternatives accomplish the primary goals of the I-45 bridge channel realignment project to some degree, but the proposed plan for the channel realignment accomplishes these goals with a minimal risk to the bridge structure and a minimal filling of the old channel. The primary objective of the project is to reduce the risk of damage to the bridge piers from floating debris and reduce or eliminate the cost of continual maintenance to remove the debris and periodically repair the structure. The proposed plan to fill the old channel is to fill from the upstream diversion of the river channel to the downstream side of the bridge. The fill will be placed up to the level of the existing overbank areas at the approximate elevation of 396.0 feet and will be placed around the existing bridge piers located within the old channel. This is the only partial channel fill plan that will ensure complete diversion of channel confined flows and minimize the risk to the existing bridge piers. The channel fill will terminate at the downstream end with a very gradual slope of the fill to the streambed of the old channel just downstream of the bridge piers. A portion of the old channel downstream of the I-45 bridge is to remain unfilled as existing. This unfilled portion of the old channel will provide a slack water area for use as a possible river access point and may provide some habitat diversity near the river. However, slack water areas such as this have a tendency to collect trash and debris both from flood events and from the ease of public access. Therefore, additional maintenance to remove trash may be required for the unfilled portion of the old river channel. The filled portion of the old river channel will maximize the diversion of channel confined river flows to the new channel alignment, stabilize the bridge piers in the old channel, and minimize the risk of floating debris collecting on the bridge piers. The Texas Department of Transportation (TXDOT) maintains an access road directly beneath the I-45 bridge which provides access to the river channel from either side of the river. Filling of the old river channel beneath the bridge as proposed will provide continued access to the river channel within the TXDOT right-of-way for inspection and maintenance. A plan view of the proposed relocation of the Trinity River channel at I.H. 45 may be found in Appendix C - Civil, Structural, and Relocations Engineering.

Table A-1
100-Year Frequency Storm Rainfall Distribution

Time (hour)	Rainfall (inches)
1	0.09
2	0.10
3	0.11
4	0.12
5	0.13
6	0.14
7	0.21
8	0.24
9	0.28
10	0.38
11	0.50
12	1.04
13	2.79
14	0.62
15	0.43
16	0.31
17	0.26
18	0.23
19	0.15
20	0.13
21	0.12
22	0.11
23	0.10
24	0.10
Total	8.69

Table A-2
Standard Project Storm (SPS) Rainfall Distribution
For Subarea 50

Time (hour)	Rainfall (inches)	Time (hour)	Rainfall (inches)	Time (hour)	Rainfall (inches)
1	0.05	25	0.07	49	0.89
2	0.05	26	0.07	50	1.42
3	0.05	27	0.07	51	2.00
4	0.05	28	0.07	52	3.64
5	0.05	29	0.07	53	1.78
6	0.05	30	0.07	54	1.26
7	0.05	31	0.10	55	0.29
8	0.05	32	0.10	56	0.26
9	0.05	33	0.10	57	0.23
10	0.05	34	0.10	57	0.22
11	0.05	35	0.10	59	0.20
12	0.05	36	0.10	60	0.19
13	0.06	37	0.14	61	0.12
14	0.06	38	0.14	62	0.12
15	0.06	39	0.15	63	0.12
16	0.06	40	0.16	64	0.12
17	0.06	41	0.17	65	0.12
18	0.06	42	0.18	66	0.12
19	0.06	43	0.32	67	0.08
20	0.06	44	0.35	68	0.08
21	0.06	45	0.39	69	0.08
22	0.06	46	0.45	70	0.08
23	0.06	47	0.52	71	0.08
24	0.06	48	0.60	72	0.08
				Total	19.52

Table A-3
Standard Rainfall Losses

Recurrence Interval (years)	Annual Exceedance Probability (%)	Clayey Soil		Sandy Soil	
		Initial Abstraction (inches)	Infiltration Rate (inches/hour)	Initial Abstraction (inches)	Infiltration Rate (inches/hour)
1	NA	1.35	0.18	1.89	0.23
2	50	1.20	0.16	1.68	0.21
5	20	1.30	0.16	1.80	0.21
10	10	1.12	0.14	1.50	0.18
25	4	0.95	0.12	1.30	0.15
50	2	0.84	0.10	1.10	0.13
100	1	0.75	0.07	0.90	0.10
500	0.2	0.50	0.05	0.60	0.08

Table A-4
Subbasin Parameters for Baseline Conditions

Subarea Number	Drainage Area (sq. mi.)	C_t	t_{pR} (hours)	C_p	Q_{pR} (cfs)	Percent Sand (%)	Urbanization (%)	Imperviousness (%)
1	683.00	2.3	18.00	.35	8785	100	<1	<1
2	149.25	1.7	9.00	.35	3805	100	2	1
3	97.78	1.4	8.00	.35	2785	100	7	4
4	160.97	1.3	6.00	.35	6047	100	4	2
5	20.00		**		12907			
6	71.17	1.6	7.00	.66	4370	100	8	4
7	97.46	1.6	6.00	.66	6927	100	5	3
8	2.34		**		1510			
9	69.90	2.0	10.00	.66	3045	100	2	1
10	90.10	1.8	9.00	.66	4346	100	5	3
11	73.20	1.8	7.00	.66	4495	100	2	<1
12	209.83	1.7	9.00	.66	10120	97	2	<1
13	55.65	1.5	5.00	.66	4734	75	3	2
14	47.52	1.2	3.00	.66	6567	20	8	6
15	127.45	1.6	8.00	.66	6885	100	7	5
16	14.38		**		9280			
17	13.60	***	0.82	.70	4170	30	22	15
18	74.84	***	4.82	.70	6970	50	9	7
19	5.56		**		3588			
20	20.99	***	3.55	.70	2609	60	57	36
21	107.11	1.6	8.00	.70	6155	100	1	1
22	1.89		**		1220			
23	142.38	1.8	13.00	.70	5056	100	3	2
24	62.47	1.3	6.00	.70	4719	89	2	1
25	33.61	1.2	5.00	.70	3023	80	2	<1
26	33.94	0.9	3.00	.70	4963	24	5	3
27	39.11	***	2.37	.70	7548	80	3	2

Subarea Number	Drainage Area (sq. mi.)	C_t	t_{pR} (hours)	C_p	Q_{pR} (cfs)	Percent Sand (%)	Urbanization (%)	Imperviousness (%)
28	5.89		**		3801			
29	8.45	***	1.63	.70	2121	50	30	17
30	54.70	***	3.90	.60	5365	10	14	8
31	24.56	***	1.70	.70	5920	40	69	42
32	3.96	***	0.94	.70	1278	40	70	56
33	0.40	***	0.87	.70	129	40	70	54
34	8.91	***	1.16	.70	2875	5	10	6
35	0.38		**		245			
36	13.71	***	1.24	.70	4321	0	53	34
37	10.89	***	1.85	.70	2440	40	67	47
38	37.33	***	2.53	.70	6375	10	59	37
39	18.25	***	1.86	.70	4082	40	60	39
40	18.45	***	2.35	.70	3384	5	42	30
41	54.70	***	3.97	.70	6111	1	24	15
42	11.30	***	3.39	.70	1484	30	29	20
43	114.76	***	6.73	.70	7715	50	16	10
44	14.42	***	1.36	.70	4197	60	62	40
45	10.38	***	1.64	.70	2589	100	73	42
46	3.44		**		2220			
47	48.63	***	5.51	.70	3981	90	48	29
49	1.79	***	1.53	.70	474	30	36	23
50	27.29	***	3.05	.70	3936	60	59	38
51	29.47	***	4.97	.70	2660	70	48	35
52	21.60	***	3.47	.70	2756	65	69	51
53	2.85	***	0.78	.70	920	10	60	39
54	4.12	***	1.40	.70	1172	5	46	31
55	83.16	***	9.10	.70	4200	90	37	20
56	9.64	***	3.44	.70	1243	80	37	27
57	8.85	***	2.33	.70	1636	5	30	23

Subarea Number	Drainage Area (sq. mi.)	C_t	t_{pR} (hours)	C_p	Q_{pR} (cfs)	Percent Sand (%)	Urbanization (%)	Imperviousness (%)
58	33.00	***	3.08	.70	4745	0	9	7
59	68.00	***	6.51	.70	4742	85	11	7
60	77.08	***	2.72	.70	12257	7	9	6
61	42.25	***	1.62	.70	10663	8	9	6
62	11.67		**		7531			
63	30.58	***	2.49	.70	5301	5	34	20
64	17.84	***	1.39	.70	5104	5	57	40
65	10.35	***	1.13	.70	3340	5	24	14
66	4.23		**		2730			
67	9.00	***	1.32	.70	2697	5	36	26
68	9.23	***	2.38	.70	1663	75	75	47
69	110.00	1.5	7.00	.70	7157	100	1	<1
70	164.00	1.2	7.00	.70	10670	91	1	<1
71	58.00	1.0	4.00	.70	6453	54	2	2
72	68.00	.94	4.00	.70	7565	12	<1	<1
73	61.32	1.0	5.00	.70	5516	23	1	<1
74	36.86	1.4	5.00	.70	3316	5	6	4
75	102.44	1.6	7.00	.70	6665	0	4	3
76	83.01	1.45	4.00	.70	9235	80	14	9
77	11.37		**		7337			
78	23.63	***	4.02	.70	2625	25	24	15
79	295.00	1.95	16.00	.794	9717	74	2	2
80	55.34	1.95	9.50	.794	3032	24	4	3
81	275.10	1.95	14.28	.794	10105	50	3	3
82	92.80	1.95	*	.794	15714	25	2	1
83	145.60	1.95	8.04	.794	9373	86	2	2
84	45.86		**		29595			
85	37.60	1.95	7.00	.794	2767	80	1	<1
86	221.61	1.95	*	.794	18397	50	11	8

Subarea Number	Drainage Area (sq. mi.)	C_t	t_{pR} (hours)	C_p	Q_{pR} (cfs)	Percent Sand (%)	Urbanization (%)	Imperviousness (%)
87	75.50	1.45	9.00	.794	4371	21	2	1
88	236.71	1.95	*	.794	37998	50	5	3
89	46.24		**		29840			
90	19.95	***	3.33	.70	2639	15	24	16
91	15.93	***	2.43	.70	2826	0	19	12
92	24.98	***	5.24	.70	2155	80	26	16
93	19.51	***	1.76	.70	4567	0	45	29
94	12.81	***	1.37	.70	3707	0	52	37
95	15.22	***	2.27	.70	2885	5	42	28
96	13.70	***	1.21	.70	4403	0	68	51
97	24.12	***	1.88	.70	5346	0	48	30
98	21.62	***	1.09	.70	6976	0	67	48
99	12.59	***	1.01	.70	4062	0	87	49
100	5.12	***	0.74	.70	1652	40	55	42
101	2.95	***	1.12	.70	592	0	76	56
102	6.03	***	0.81	.70	1946	0	75	52
103	98.25	***	3.67	.70	11794	0	62	40
104	1.75		**		1129			
105	32.99	***	2.39	.70	5921	0	63	39
106	22.43	***	1.98	.70	4796	5	66	41
107	12.10	***	1.62	.70	3054	5	37	27
108	60.72	***	2.79	.70	9420	0	42	27
109	45.56	1.95	*	.794	18637	100	3	3
110	33.80	1.95	7.67	.794	2282	100	5	5
111	53.28	1.95	*	.794	24782	74	2	2

* a composite unit hydrograph was made from combining numerous subarea unit hydrographs.
** a 1-hour instantaneous unit hydrograph was used for the lake surface area.
*** a "C_t" value was not required. Urbanization curve methodology was used.

Table A-5

Peak Discharges on the Trinity River for Baseline Conditions

Location along the Trinity River	Computed Probability Peak Discharges (cfs) for:								
	Recurrence Interval (years)								SPF Event
	1	2	5	10	25	50	100	500	
	Annual Exceedance Probability (percent)								
	NA	50	20	10	4	2	1	0.2	NA
Below the confluence of the West and Elm Forks	18300	24500	38700	51500	73400	95100	115800	202700	270100
At the "Dallas" Streamflow Gage	18000	24100	38100	50800	72500	94600	115200	201400	269200
Above the confluence of White Rock Creek	14100	20900	35200	48400	69100	90200	111800	188500	251100
Below the confluence of White Rock Creek	15700	22400	37900	55200	74200	96700	119400	200300	268300
At the "Below Dallas" Streamflow Gage	15700	22300	37700	54700	74100	96500	119300	200100	267700
Above the confluence of Five Mile Creek	15300	21900	37300	53200	73700	95700	118800	197800	264700
Below the confluence of Five Mile Creek	15300	21900	37300	53200	73700	95700	118800	197800	264700

Table A-6

Peak Discharges on the Trinity River
for Baseline Conditions plus a 1,200-foot Swale

Location along the Trinity River	Computed Probability Peak Discharges (cfs) for:								
	Recurrence Interval (years)								SPF Event
	1	2	5	10	25	50	100	500	
	Annual Exceedance Probability (percent)								
	NA	50	20	10	4	2	1	0.2	NA
Below the confluence of the West and Elm Forks	18300	24500	38700	51500	73400	95100	115800	202700	270100
At the "Dallas" Streamflow Gage	18000	24100	38200	51000	72600	94600	115300	201500	269300
Above the confluence of White Rock Creek	16800	23100	36000	48800	70300	91700	112900	192500	255000
Below the confluence of White Rock Creek	19800	26900	42600	57700	76400	99000	122100	205900	273600
At the "Below Dallas" Streamflow Gage	19800	26900	42500	57500	76300	99000	122000	205700	273400
Above the confluence of Five Mile Creek	18900	26100	41600	56300	76000	98400	121700	203700	270000
Below the confluence of Five Mile Creek	18900	26100	41600	56300	76000	98400	121700	203700	270200

Table A-7

Peak Discharges on the Trinity River for Baseline Conditions, plus "Chain of Wetlands", and with Lamar Street and Cadillac Heights Levees in place

Location along the Trinity River	Computed Probability Peak Discharges (cfs) for:								
	Recurrence Interval (years)								SPF Event
	1	2	5	10	25	50	100	500	
	Annual Exceedance Probability (percent)								
	NA	50	20	10	4	2	1	0.2	NA
Below the confluence of the West and Elm Forks	18300	24500	38700	51500	73400	95100	115800	202700	270100
At the "Dallas" Streamflow Gage	18000	24200	38200	51000	72500	94500	115200	201300	269300
Above the confluence of White Rock Creek	16500	22800	35700	48200	69700	91200	112900	192200	256800
Below the confluence of White Rock Creek	19200	26300	41400	55700	75200	97900	121700	205500	275700
At the "Below Dallas" Streamflow Gage	19200	26200	41300	55600	75100	97800	121600	205200	275300
Above the confluence of Five Mile Creek	18500	25600	40500	54700	74700	97100	121100	203000	271700
Below the confluence of Five Mile Creek	18500	25600	40500	54700	74700	97100	121100	203000	271800

Table A-8
Calibrated Statistics for Risk Analysis

Condition	Trinity River Location	Equivalent Record Length (years)	Log-transformed Values of:		
			Mean	Standard Deviation	Skew
Baseline	At the Downstream End of the Dallas Floodway	40	4.2779	0.3341	0.13
Baseline	At the Central Wastewater Treatment Plant Levee	40	4.2770	0.3300	0.13
Baseline	At the "Below Dallas" Streamflow Gage (Loop 12)	40	4.2779	0.3341	0.13
N.E.D Plan	At the Downstream End of the Dallas Floodway	40	4.2779	0.3341	0.13
N.E.D Plan	At the Central Wastewater Treatment Plant Levee	40	4.2800	0.3335	0.14
N.E.D Plan	At the "Below Dallas" Streamflow Gage (Loop 12)	40	4.2779	0.3390	0.14
Recommended Plan	At the Downstream End of the Dallas Floodway	40	4.2779	0.3341	0.13
Recommended Plan	At the Central Wastewater Treatment Plant Levee	40	4.2770	0.3300	0.14
Recommended Plan	At the "Below Dallas" Streamflow Gage (Loop 12)	40	4.2779	0.3390	0.14

Table A-9
Pertinent Data on Sumps and Outlet Sluices

Sump #	100-year Frequency Flood Event					Excavation				Outlet Sluices				
	Drainage Area	Peak Inflow	Peak Outflow	Peak Pool Elevation	Peak Surface Area	Surface Area	Top-of-Cut Elevation	Toe-of-Cut Elevation	Invert Elevation	# of Conduits	Size	Inlet Elevation	Outlet Elevation	Levee Station
	(acres)	(cfs)	(cfs)	(feet)	(acres)	(acres)	(feet)	(feet)	(feet)		(feet)	(feet)	(feet)	(feet)
LS-1	32	258	133	402.7	1.68	0.00	NA	NA	388	1	4 x 4	393	391	40+10
LS-2	28	226	111	402.0	1.80	1.80	409	393	392	1	4 x 4	392	390	24+90
LS-3	795	3483	1614	402.5	17.10	17.10	407	393	392	4	6 x 6	392	390	118+60
LS-4	141	837	187	403.8	8.08	0.00	NA	NA	398	1	4 x 4	395	393	92+30
LS-5	268	1809	713	400.0	12.20	12.20	412	393	392	3	5 x 5	392	390	50+60
CH-1	102	798	798	412.0	NA	NA	NA	406	405	3	5 x 5	405	404	118+00
CH-2	140	1025	1025	405.8	NA	NA	NA	398	397	3	5 x 5	397	396	90+80
CH-3	34	280	280	400.6	NA	NA	NA	398	397	3	5 x 5	397	396	82+00
CH-4	61	469	469	400.0	NA	NA	NA	396	395	3	5 x 5	395	394	43+70

Notes: "LS" refers to the Lamar Street Levee segment and "CH" refers to the Cadillac Heights Levee segment.
All elevations are referenced to the National Geodetic Vertical Datum (NGVD).

Table A-9A
Incremental Cost Items for Alternative Solution Scenarios
Sumps LS-3 and LS-5

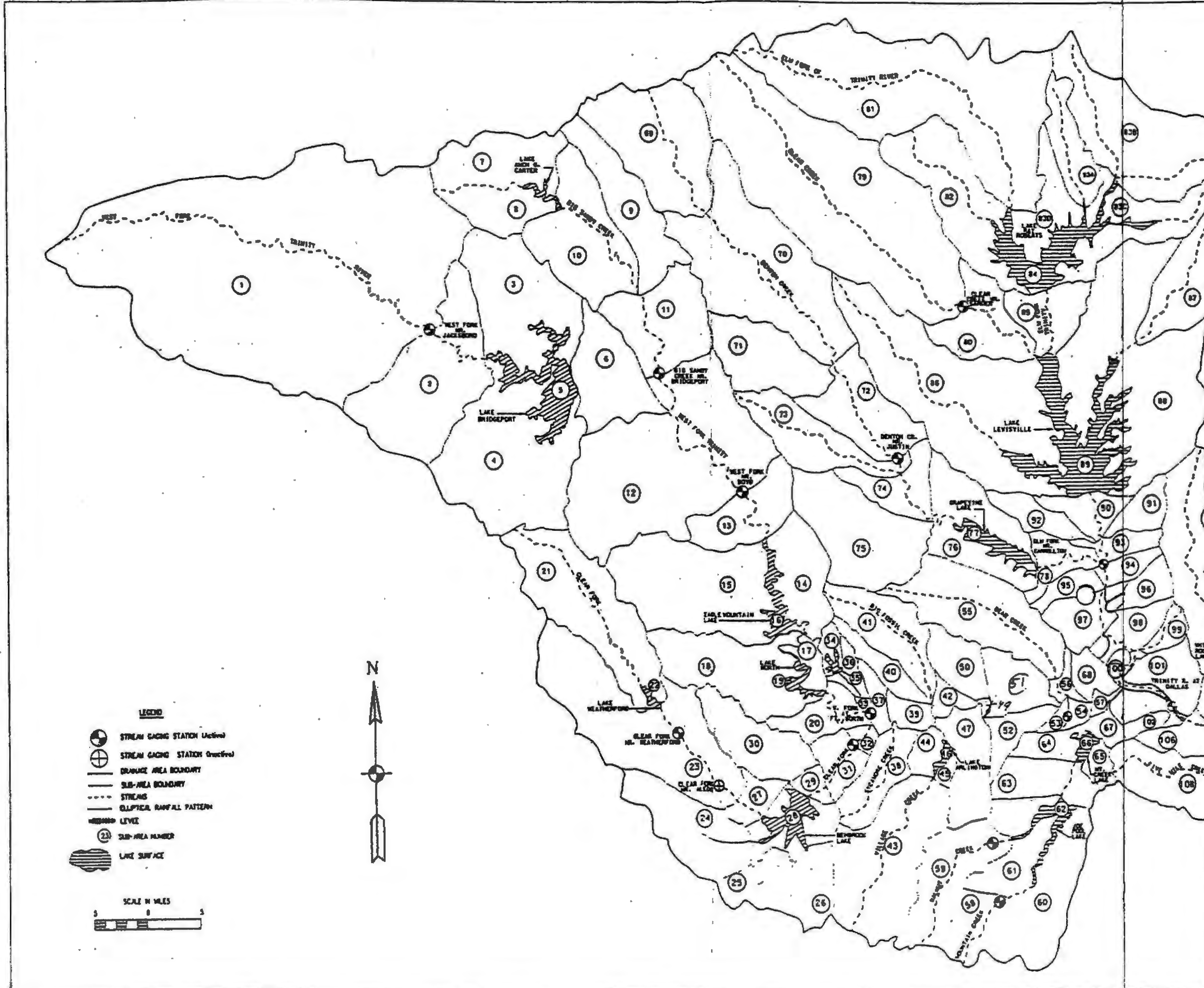
Sump #	Alternative #1 (1 – 4' x 4' outlet)			Alternative #2 (Recommended Plan)			Alternative #3 (no excavation)		
	Outlet Sluices	Excavation (cu.yds)	Cost of items	Outlet Sluices	Excavation (cu. yds.)	Cost of items	Outlet Sluices	Excavation (cu.yds)	Cost of items
L.S.-3	1 – 4 x 4	543,300	$2.95 million	4 - 6 x 6	317,300	$2.08 million	9 – 6 x 6	0	$2.30 million
L.S.-5	1 – 4 x 4	400,500	$2.24 million	3 – 5 x 5	232,500	$1.38 million	7 – 5 x 5	0	$1.43 million

Table A-10
Average Flow Velocities (feet per second) for Baseline Conditions versus N.E.D. Plan

Reach	Event	Baseline Conditions			N.E.D. Plan		
		LOB	CHAN	ROB	LOB	CHAN	ROB
Downstream limit of N.E.D. Swale to Loop 12 Bridge	1 %	0.9	3.0	2.5	3.4	2.2	2.0
	SPF	1.1	3.5	3.5	4.4	2.6	2.9
Through Loop 12 bridge	1 %	2.4	10.1	2.5	3.0	7.6	1.9
	SPF	3.2	12.0	3.6	4.2	9.1	2.8
Loop 12 to White Rock Creek	1 %	1.0	2.5	1.7	2.8	1.8	0.6
	SPF	1.4	2.9	2.3	2.7	1.9	1.5
White Rock Creek to Southern Pacific RR	1 %	1.0	3.3	0.9	1.5	2.2	0.5
	SPF	1.4	4.3	1.1	2.1	2.9	0.8
Through Southern Pacific RR Bridge	1 %	1.4	7.2	1.8	3.9	4.5	1.1
	SPF	2.3	10.5	2.6	6.1	6.5	1.6
Southern Pacific RR to State Highway 310 bridge	1 %	1.3	5.0	2.3	3.5	3.8	1.6
	SPF	1.9	6.7	3.2	4.9	4.9	2.3
Through State Highway 310 bridge	1 %	1.5	6.7	2.8	3.5	4.6	1.8
	SPF	2.0	9.9	3.9	5.2	6.3	2.7
State Highway 310 to I.H. 45 bridge	1 %	0.9	3.6	1.6	1.7	4.0	1.8
	SPF	1.2	3.9	2.0	1.9	4.6	2.3
I.H. 45 bridge to M-K-T RR	1 %	1.2	4.6	1.7	1.0	4.3	3.1
	SPF	1.4	4.4	1.8	1.3	4.8	3.3
Through M-K-T RR bridge	1 %	1.6	8.3	1.7	0.9	5.4	5.3
	SPF	2.4	7.1	2.0	1.6	6.0	4.6
M-K-T RR to Martin Luther King Boulevard	1 %	1.3	7.3	1.4	0.8	5.1	3.8
	SPF	2.0	8.7	1.8	1.2	6.3	4.7
Through Martin Luther King Boulevard Bridge	1 %	1.4	7.4	1.9	0.6	3.9	4.2
	SPF	2.4	8.4	2.5	1.0	5.0	5.6
Martin Luther King Boulevard to Cedar Creek	1 %	1.6	3.8	1.9	1.9	6.5	3.9
	SPF	2.2	4.4	2.3	2.3	7.5	5.0

Table A-11
Average Flow Velocities (feet per second)
for Baseline Conditions versus Recommended Plan

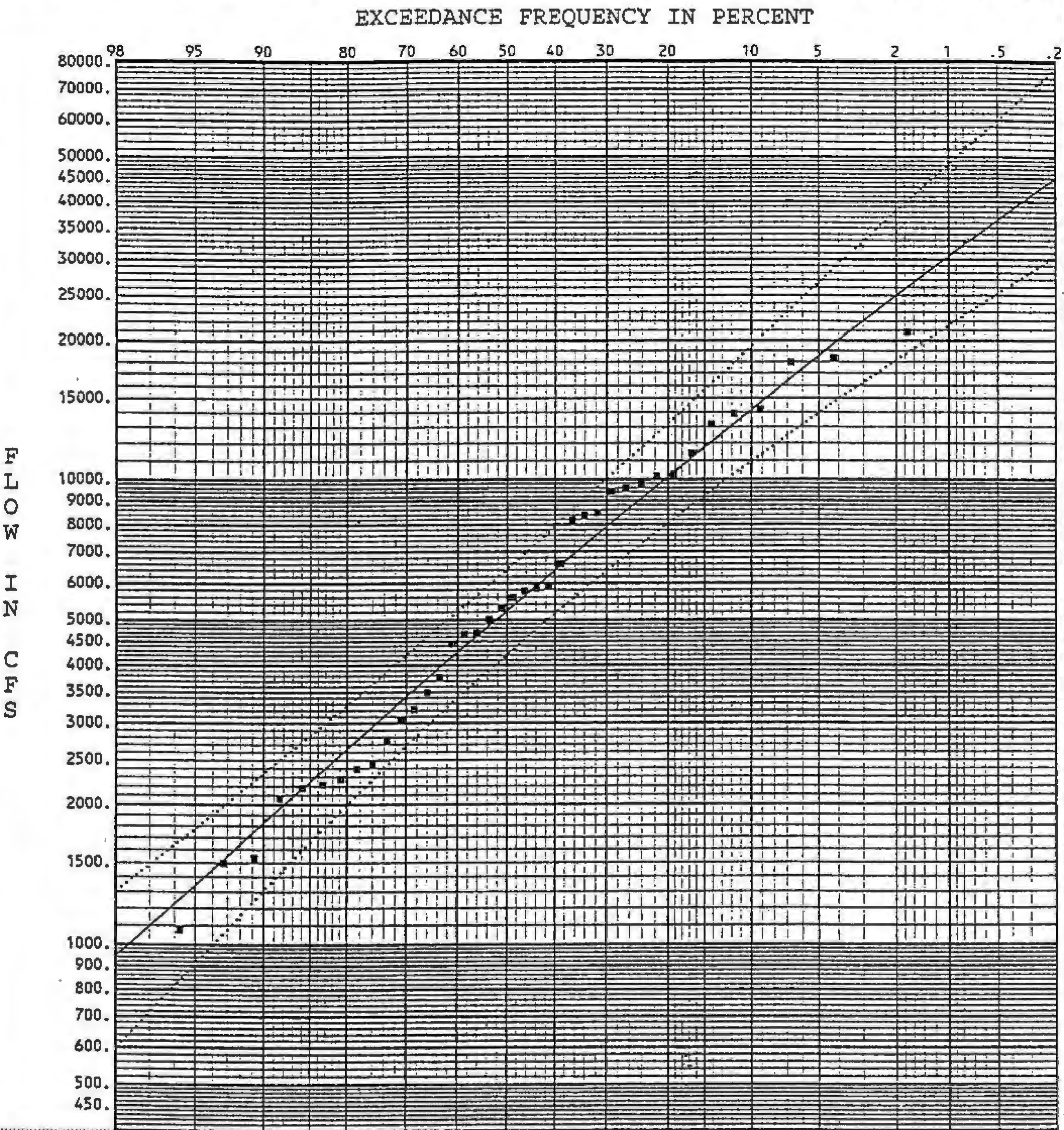
Reach	Event	Baseline Conditions			Recommended Plan		
		LOB	CHAN	ROB	LOB	CHAN	ROB
Downstream limit of N.E.D. Swale to Loop 12 Bridge	1 %	0.9	3.0	2.5	0.9	5.3	2.9
	SPF	1.1	3.5	3.5	1.3	6.4	3.8
Through Loop 12 bridge	1 %	2.4	10.1	2.5	2.2	8.1	2.3
	SPF	3.2	12.0	3.6	2.7	10.6	2.9
Loop 12 to White Rock Creek	1 %	1.0	2.5	1.7	0.8	3.3	2.9
	SPF	1.4	2.9	2.3	1.1	4.1	3.3
White Rock Creek to Southern Pacific RR	1 %	1.0	3.3	0.9	0.6	2.5	3.0
	SPF	1.4	4.3	1.1	0.9	3.3	3.2
Through Southern Pacific RR Bridge	1 %	1.4	7.2	1.8	1.0	4.9	4.2
	SPF	2.3	10.5	2.6	1.6	6.7	5.7
Southern Pacific RR to State Highway 310 bridge	1 %	1.3	5.0	2.3	0.6	2.7	4.1
	SPF	1.9	6.7	3.2	1.0	3.9	5.5
Through State Highway 310 bridge	1 %	1.5	6.7	2.8	1.9	5.4	4.2
	SPF	2.0	9.9	3.9	3.0	7.4	6.0
State Highway 310 to I.H. 45 bridge	1 %	0.9	3.6	1.6	1.3	4.4	2.8
	SPF	1.2	3.9	2.0	2.0	5.5	3.3
I.H. 45 bridge to M-K-T RR	1 %	1.2	4.6	1.7	1.3	4.9	3.8
	SPF	1.4	4.4	1.8	2.0	6.3	4.7
Through M-K-T RR bridge	1 %	1.6	8.3	1.7	3.0	11.2	3.6
	SPF	2.4	7.1	2.0	3.9	11.3	4.6
M-K-T RR to Martin Luther King Boulevard	1 %	1.3	7.3	1.4	0.9	4.5	2.8
	SPF	2.0	8.7	1.8	1.6	6.6	4.4
Through Martin Luther King Boulevard Bridge	1 %	1.4	7.4	1.9	1.4	7.0	2.7
	SPF	2.4	8.4	2.5	2.7	9.5	4.2
Martin Luther King Boulevard to Cedar Creek	1 %	1.6	3.8	1.9	1.7	4.5	2.7
	SPF	2.2	4.4	2.3	2.7	6.5	4.0



Dallas Floodway Extension
General Reevaluation Report

Upper Trinity River
Watershed Map

Plate A-1



—— FLOW Frequency (without Exp. Prob.)

▪ Median Plotting Positions

······· 5% and 95% Confidence Limits

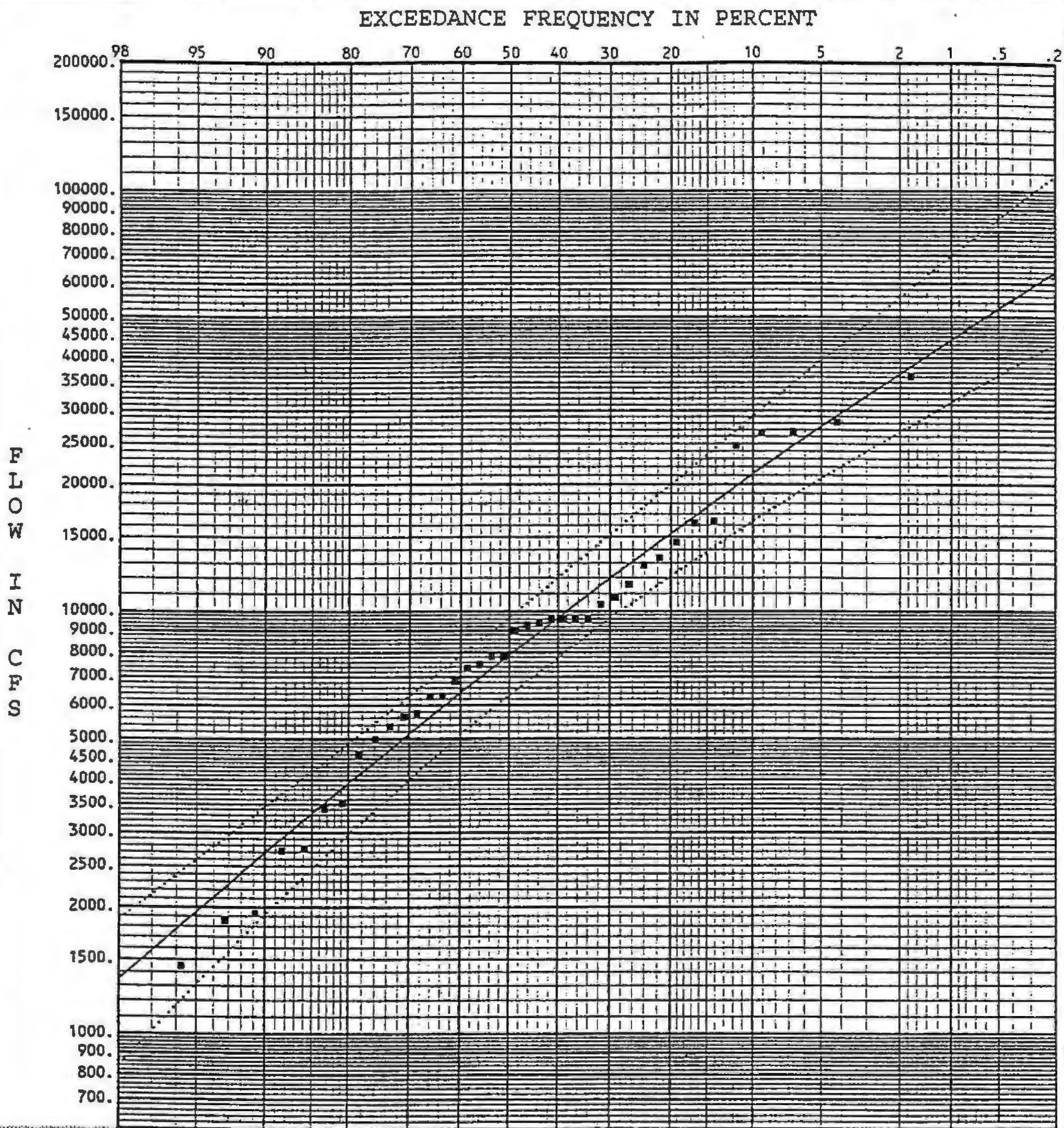
FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	3.7110	HISTORIC EVENTS	0
STANDARD DEV	.3466	HIGH OUTLIERS	0
SKEW	-.1846	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	-.1289	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Clear Fork of the Trinity
River at Fort Worth
for Water Years 1953-1992

Plate A-2



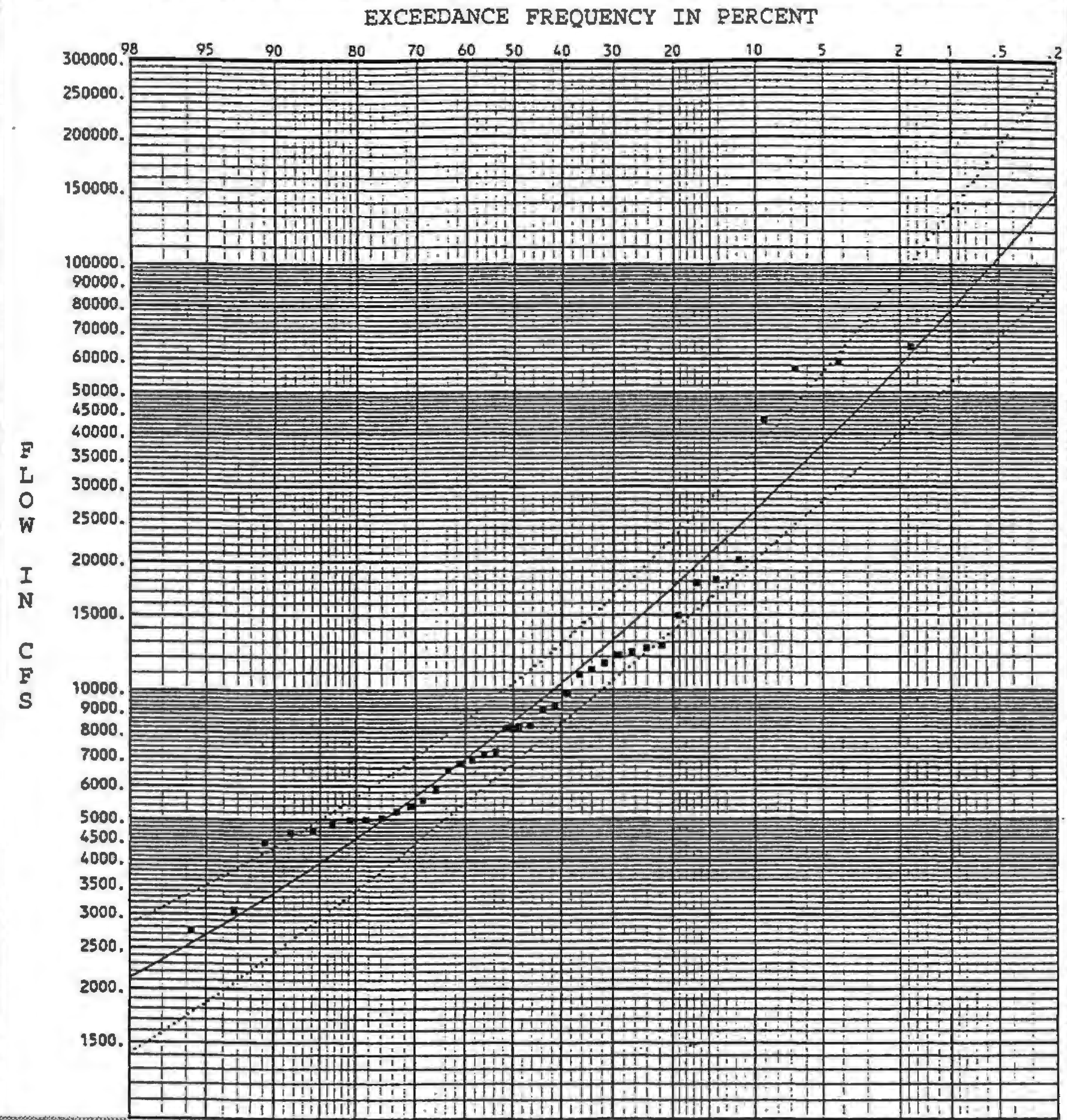
—— FLOW Frequency (without Exp. Prob.)
■ Median Plotting Positions
········ 5% and 95% Confidence Limits

FREQUENCY STATISTICS LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	3.8852	HISTORIC EVENTS	0
STANDARD DEV	.3498	HIGH OUTLIERS	0
SKEW	-.2964	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	-.2031	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
West Fork of the Trinity
River at Fort Worth
for Water Years 1953-1992

Plate A-3



—— FLOW Frequency (without Exp. Prob.)

▪ Median Plotting Positions

······ 5% and 95% Confidence Limits

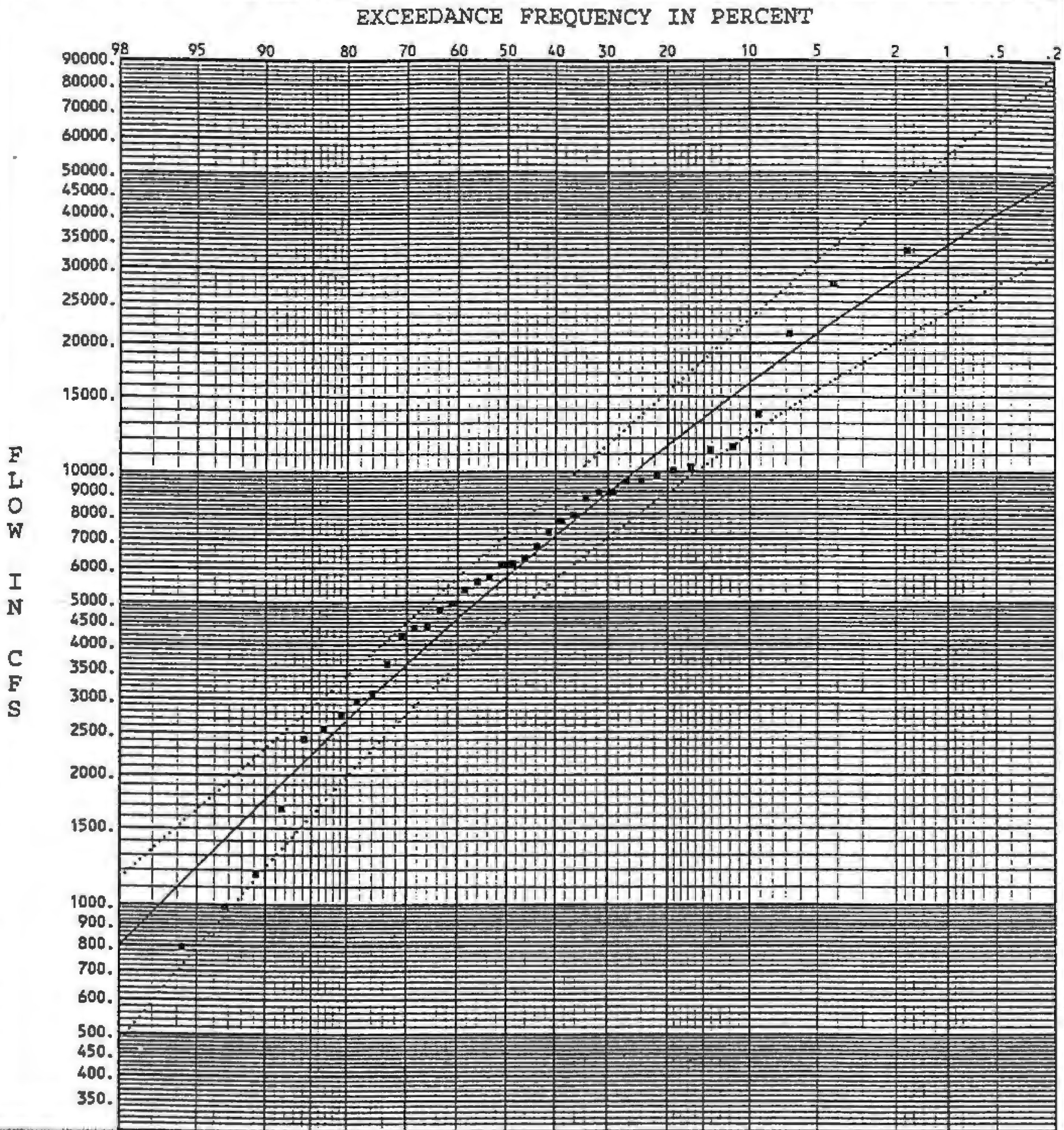
FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	3.9551	HISTORIC EVENTS	0
STANDARD DEV	.3514	HIGH OUTLIERS	0
SKEW	.7718	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.4838	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
West Fork of the Trinity
River at Grand Prairie
for Water Years 1953-1992

Plate A-4



—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······ 5% and 95% Confidence Limits

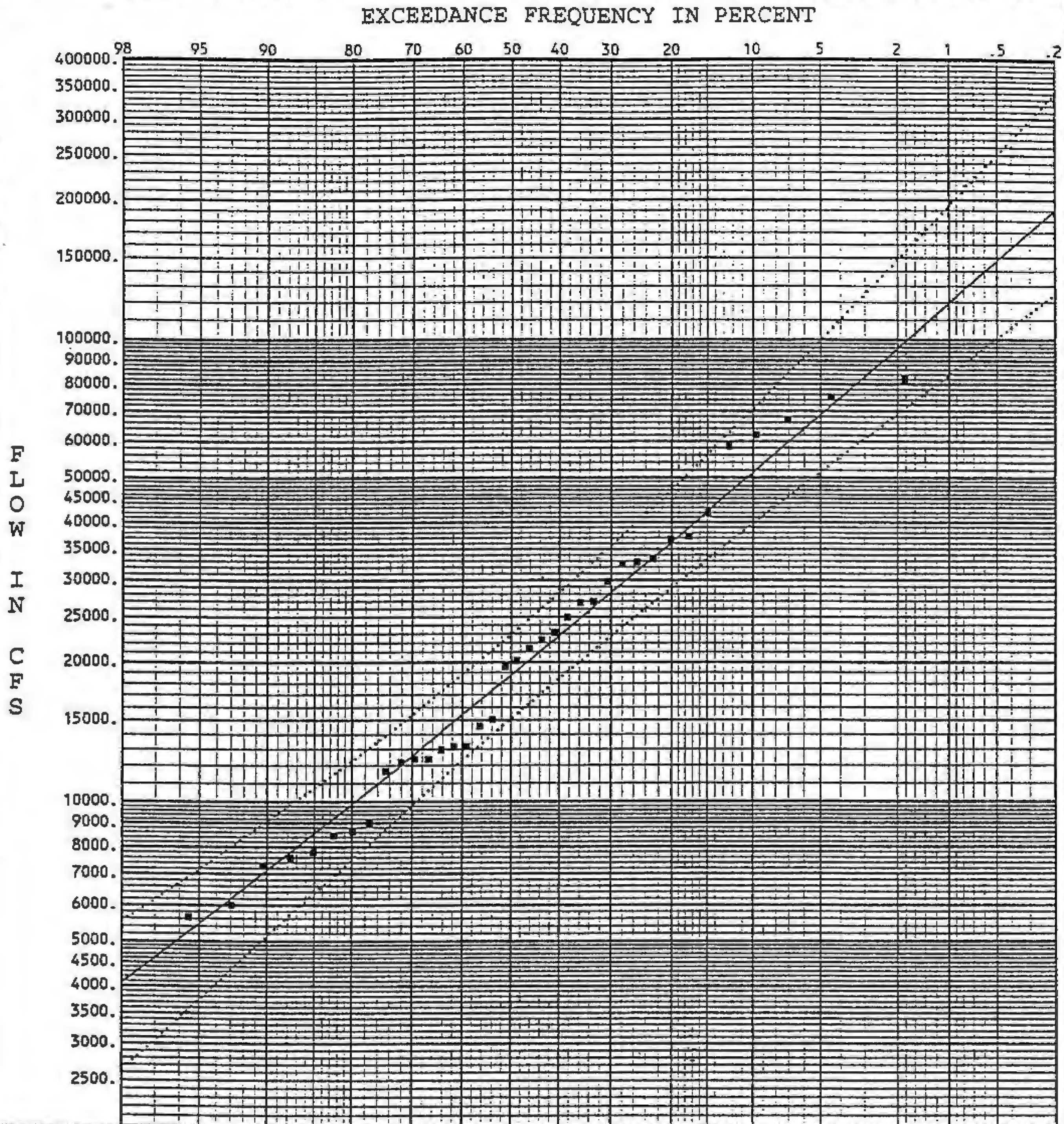
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

		NUMBER OF EVENTS	
MEAN	3.7390	HISTORIC EVENTS	0
STANDARD DEV	.3775	HIGH OUTLIERS	0
SKEW	-.4652	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	-.3094	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Elm Fork of the Trinity
River near Carrolton
for Water Years 1953-1992

Plate A-5



—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
········ 5% and 95% Confidence Limits

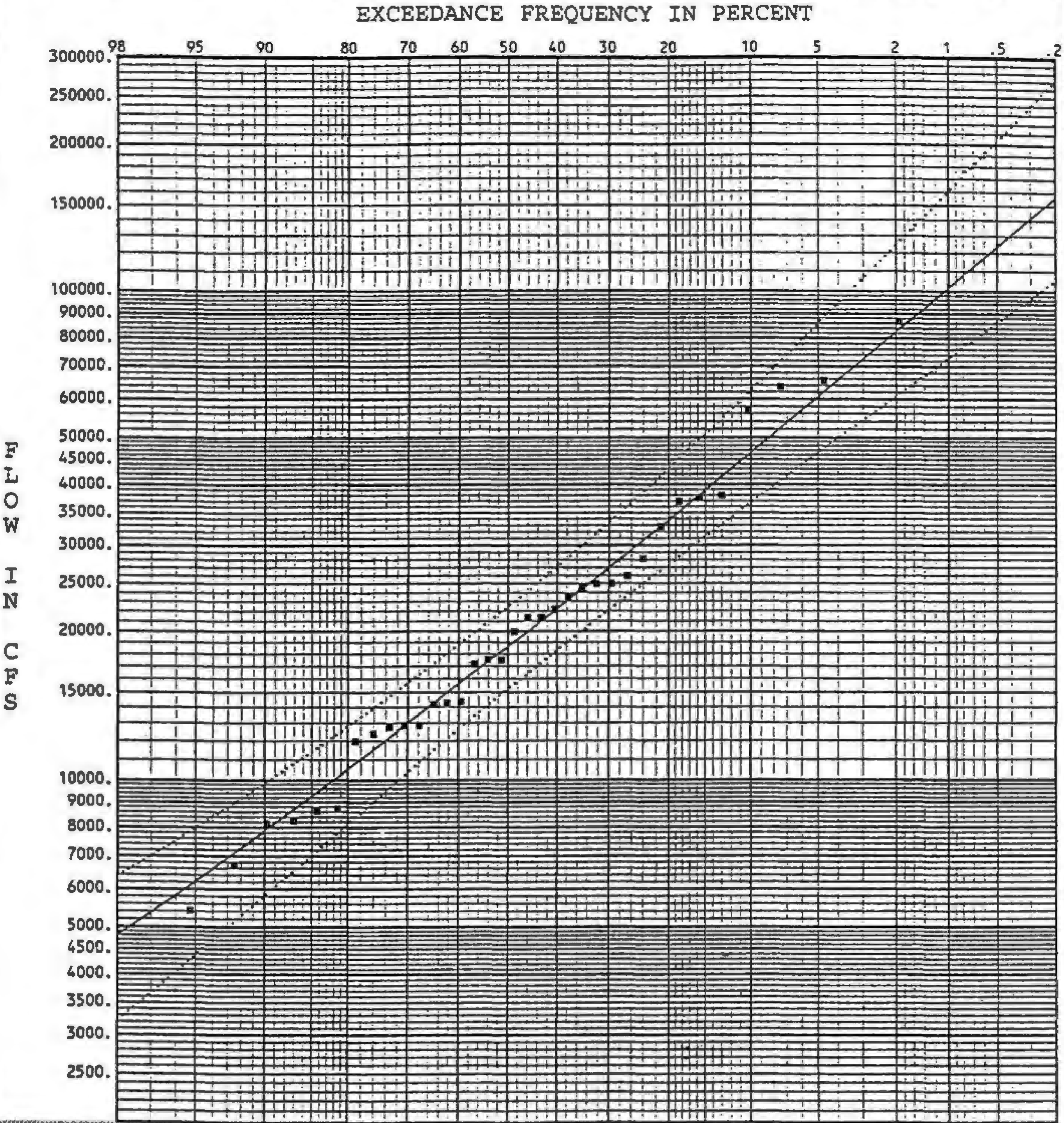
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

		NUMBER OF EVENTS	
MEAN	4.2779	HISTORIC EVENTS	0
STANDARD DEV	.3341	HIGH OUTLIERS	0
SKEW	.1416	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.0982	SYSTEMATIC EVENTS	38

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at Dallas
for Water Years 1953-1992

Plate A-6



—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······ 5% and 95% Confidence Limits

FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

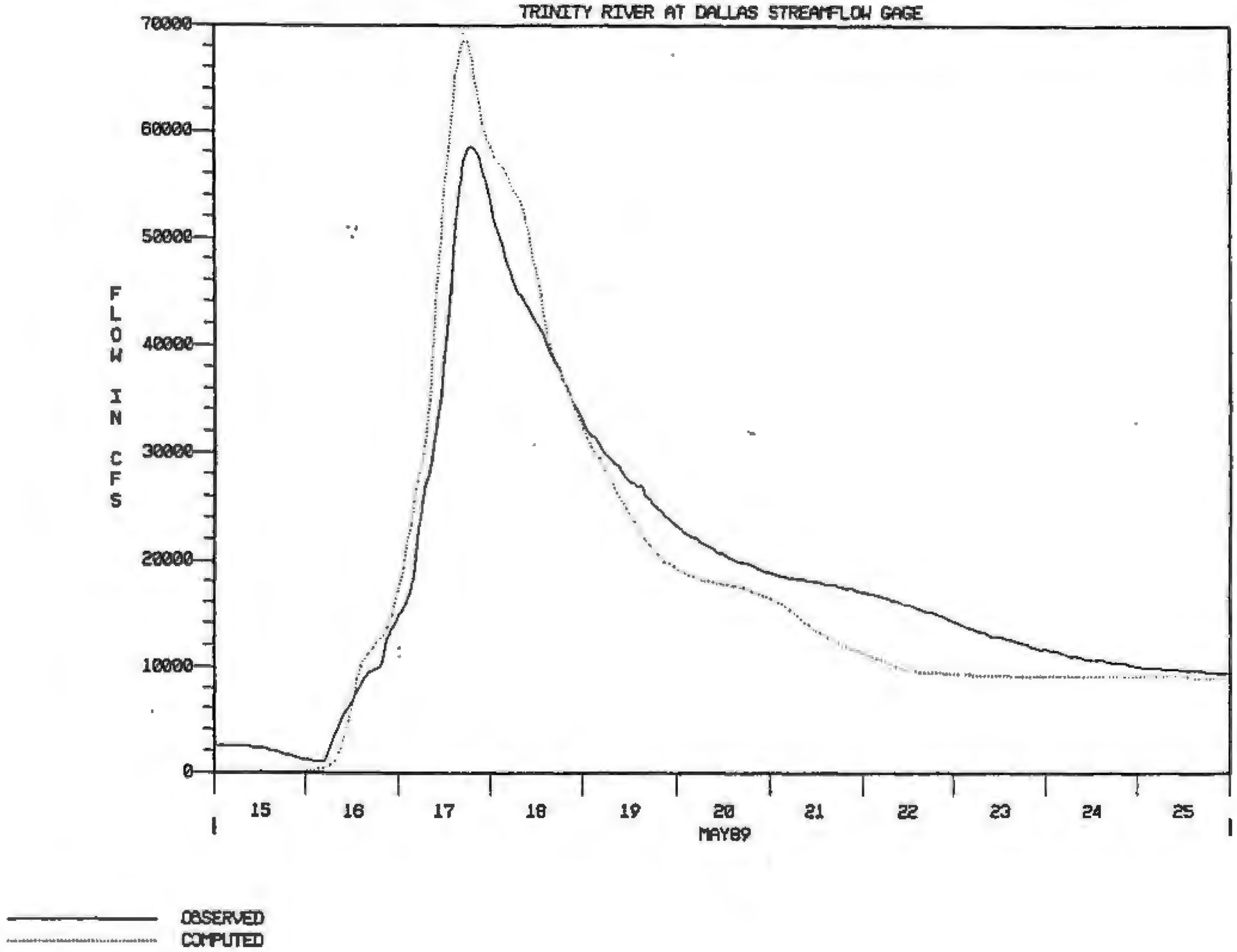
MEAN	4.2770	HISTORIC EVENTS	0
STANDARD DEV	.3010	HIGH OUTLIERS	0
SKEW	.2070	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1398	SYSTEMATIC EVENTS	36

NUMBER OF EVENTS

. Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River below Dallas
for Water Years 1953-1992

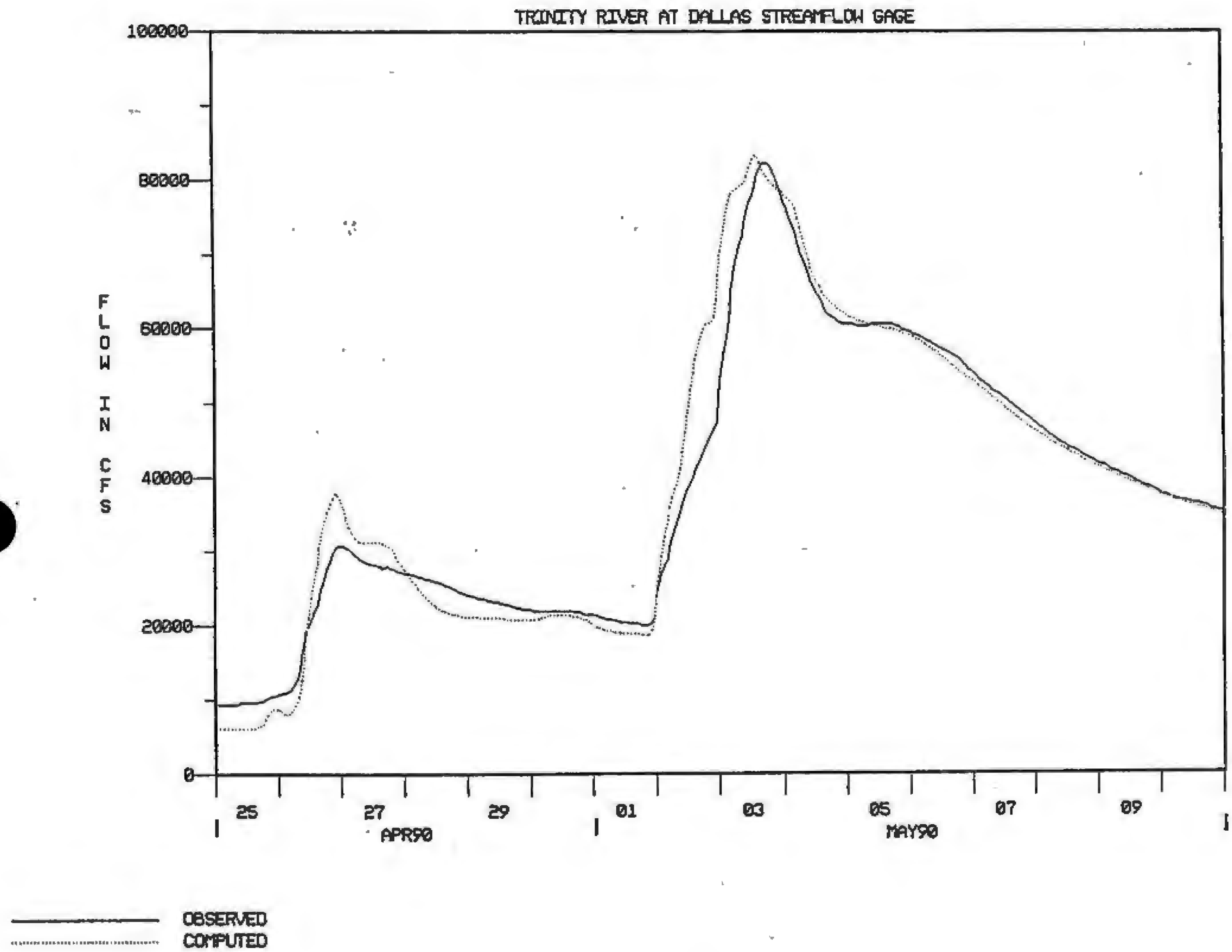
Plate A-7



Dallas Floodway Extension
General Reevaluation Report

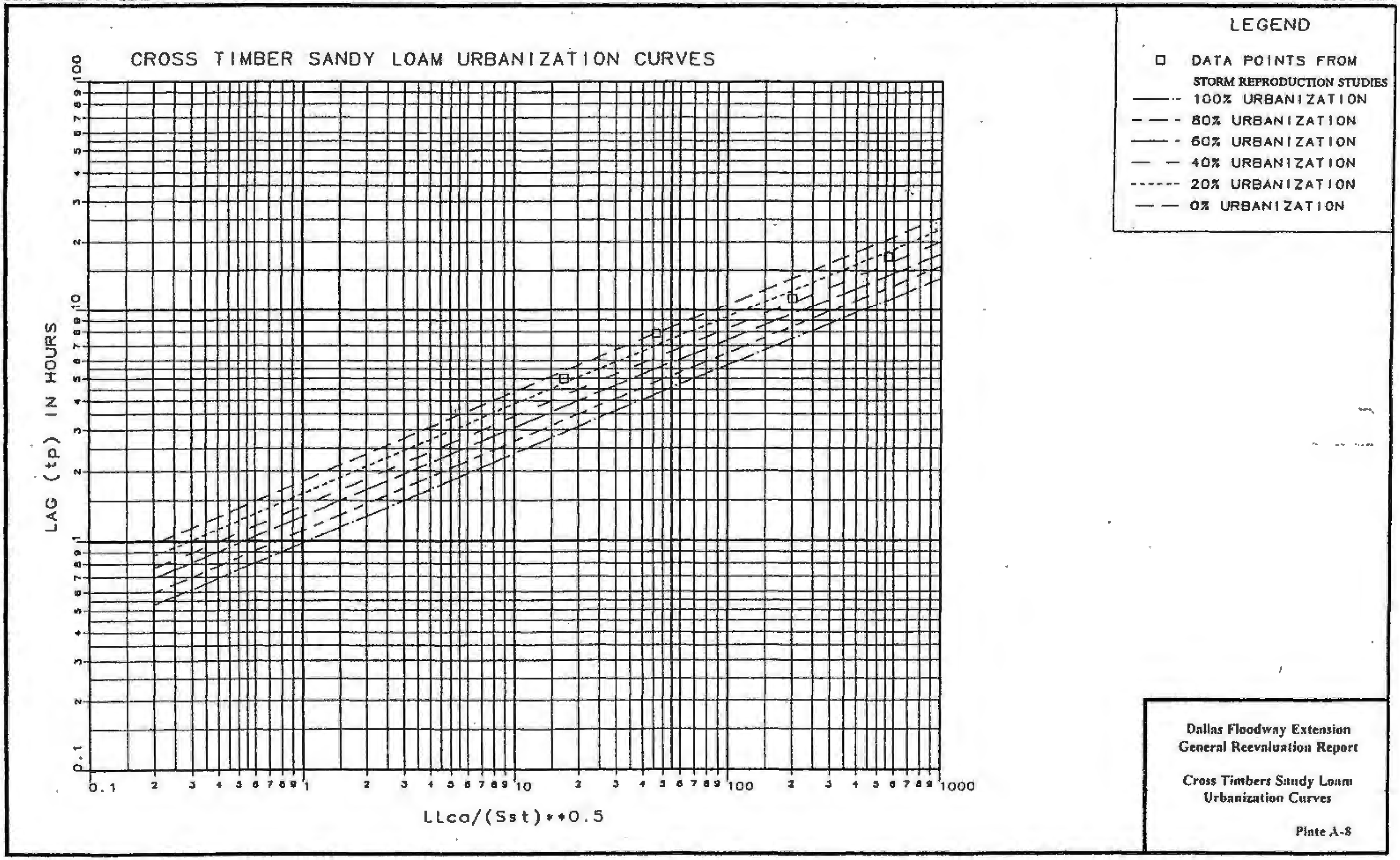
Flood Hydrograph Reproduction
May 1989 Event

Plate A-7A



Dallas Floodway Extension
General Reevaluation Report

Flood Hydrograph Reproduction
May 1990 Event

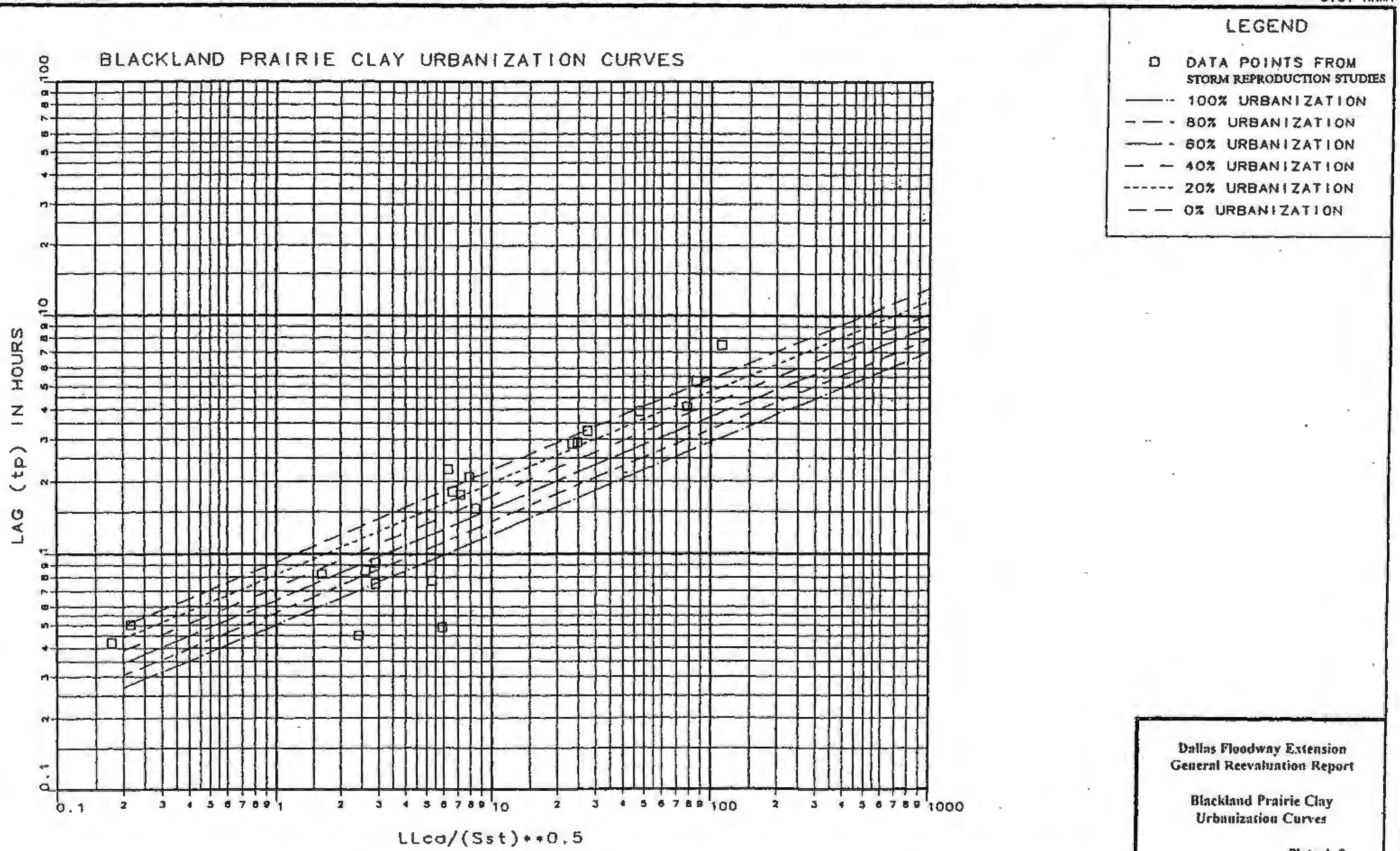


Dallas Floodway Extension
General Reevaluation Report

Cross Timbers Sandy Loam
Urbanization Curves

Plate A-8

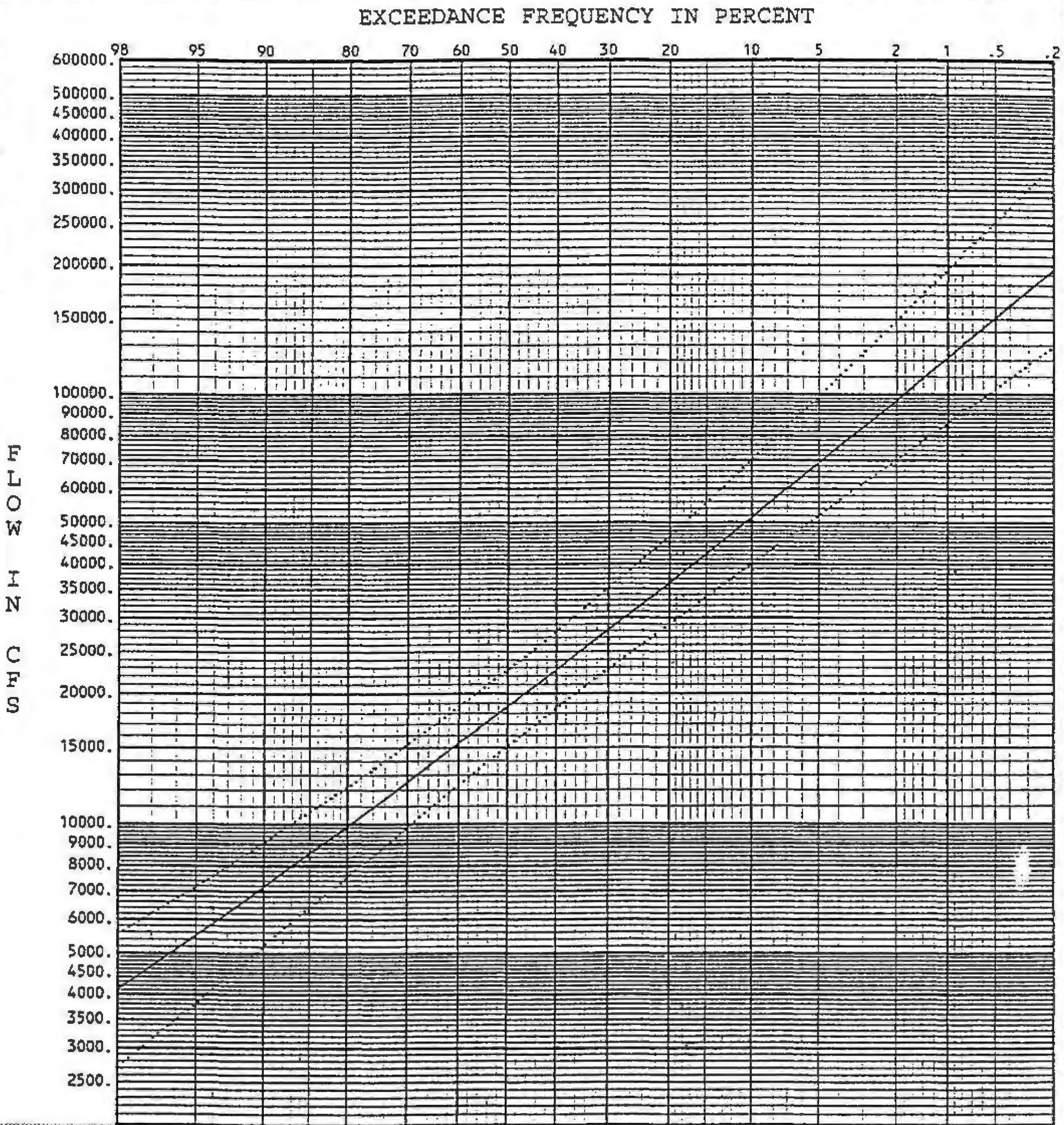
CORPS OF ENGINEERS U.S. ARMY



Dallas Floodway Extension
General Reevaluation Report

Blackland Prairie Clay
Urbanization Curves

Plate A-9



—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
.... 5% and 95% Confidence Limits

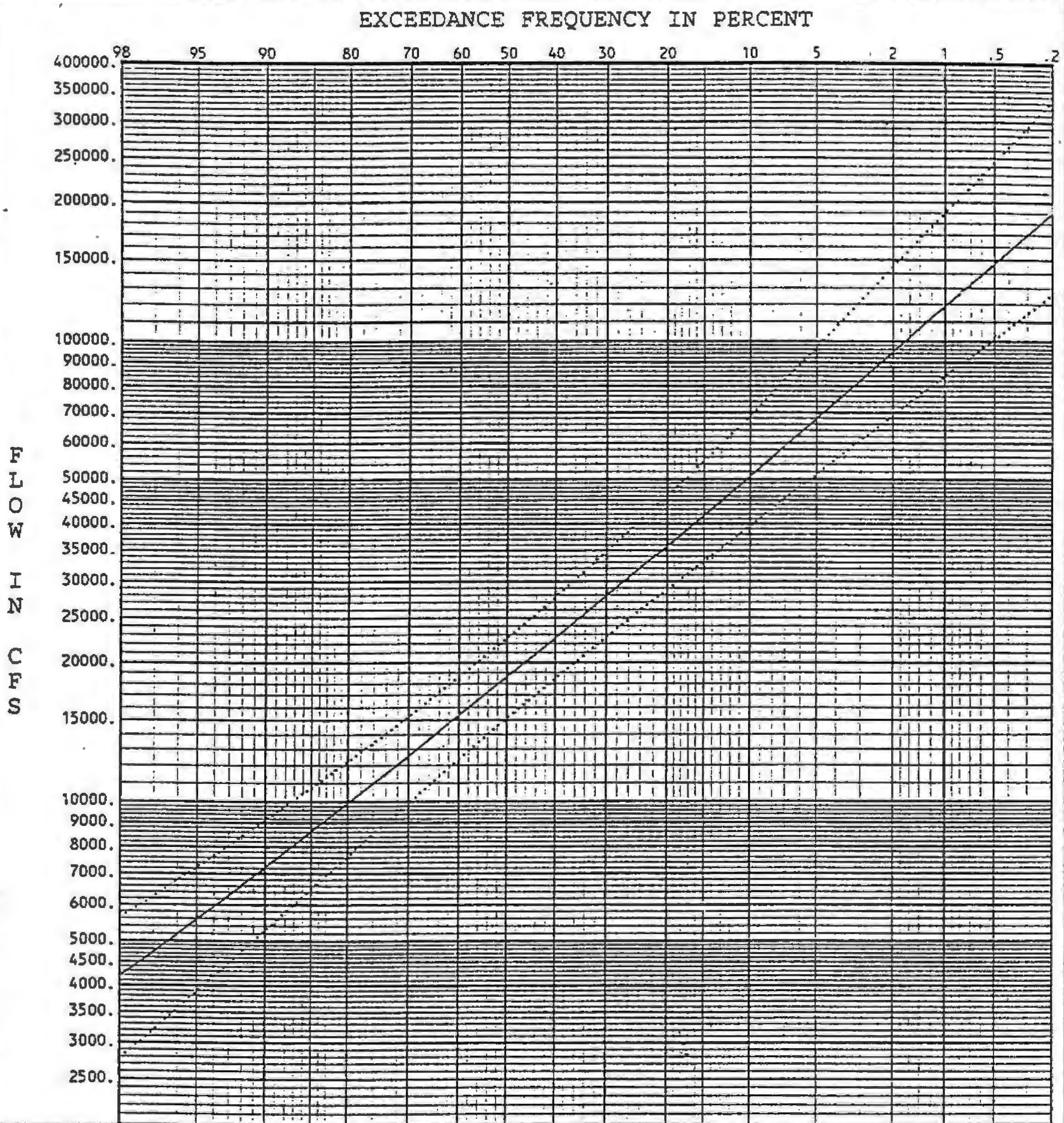
FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2779	HISTORIC EVENTS	0
STANDARD DEV	.3341	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1300	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at Downstream
End of the Dallas Floodway
For Baseline Conditions

Plate A-10



—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······· 5% and 95% Confidence Limits

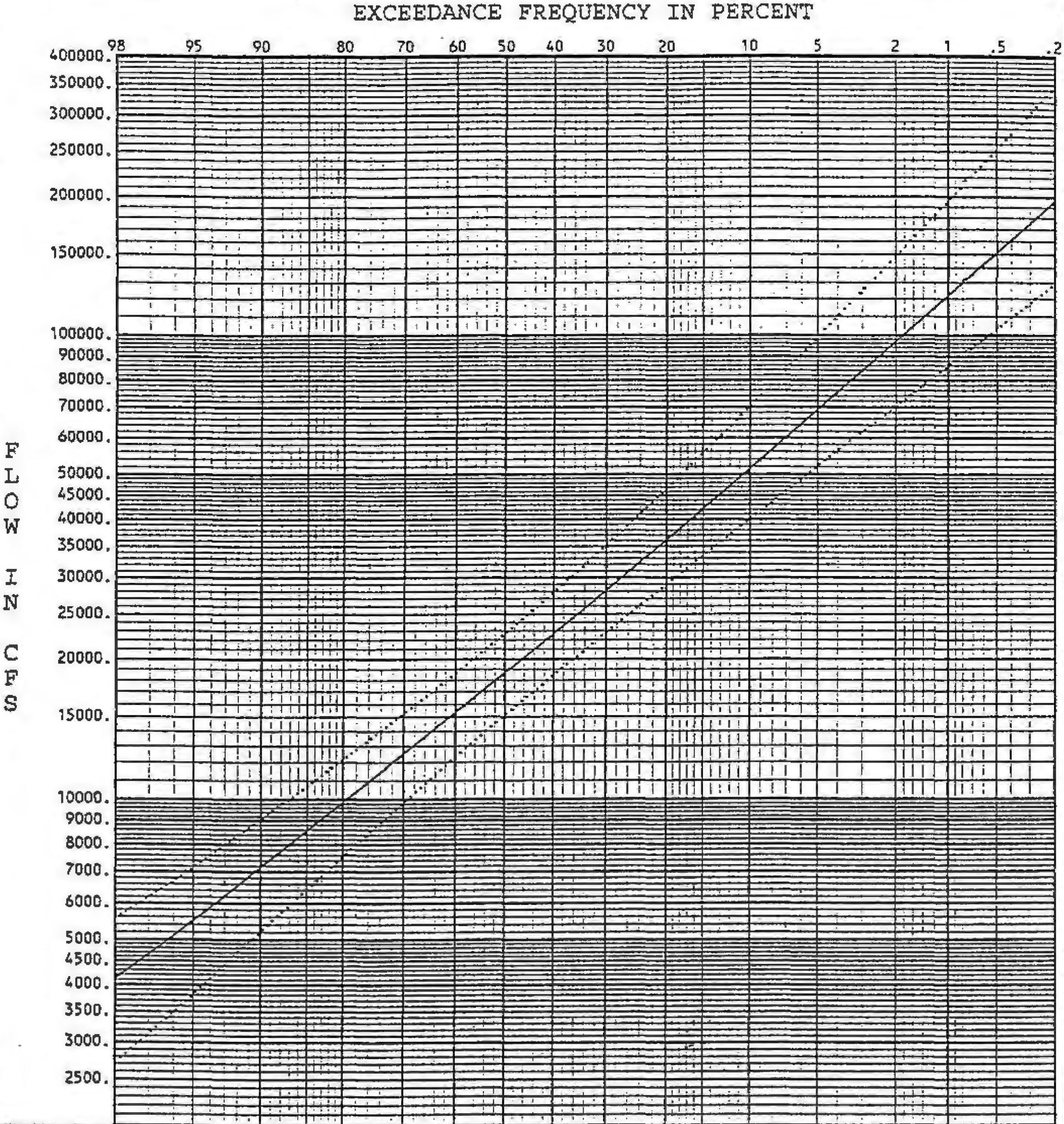
FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2770	HISTORIC EVENTS	0
STANDARD DEV	.3300	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1300	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at the Central
Wastewater Treat. Plant Levee
For Baseline Conditions

Plate A-11



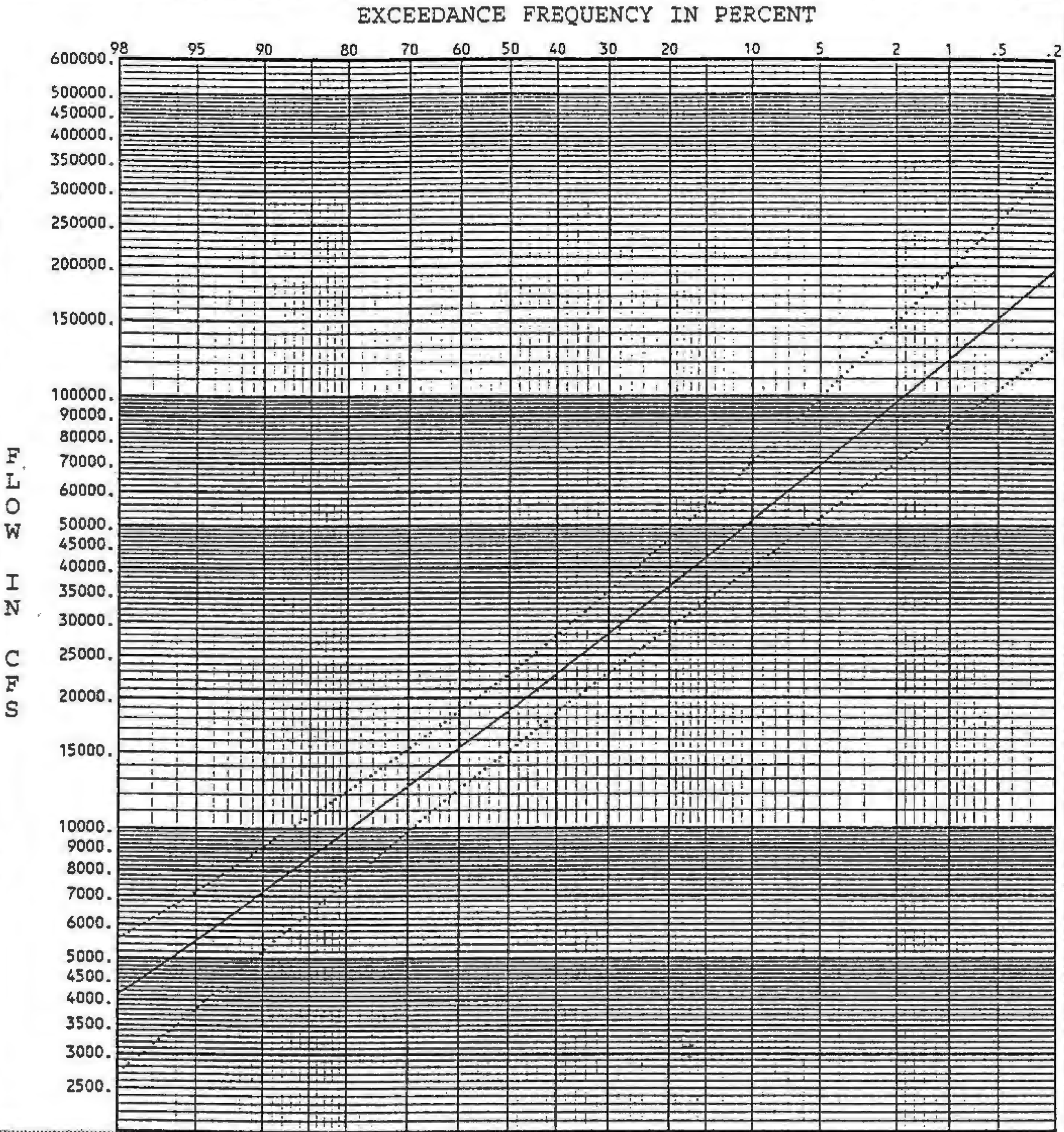
—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······· 5% and 95% Confidence Limits

FREQUENCY STATISTICS LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2779	HISTORIC EVENTS	0
STANDARD DEV	.3341	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1300	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at the "Below Dallas" Gage (Loop 12)
For Baseline Conditions

Plate A-12



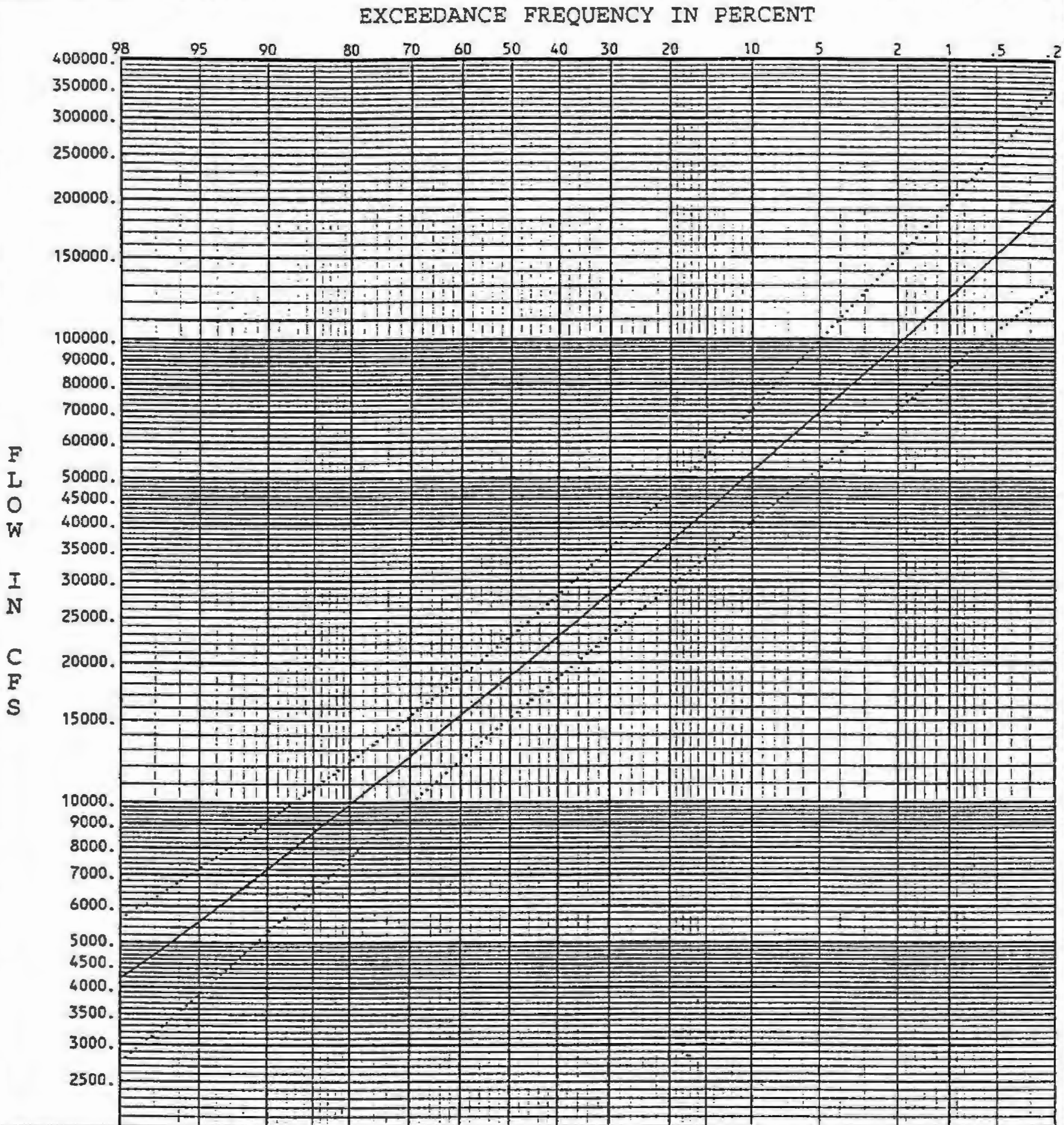
—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······· 5% and 95% Confidence Limits

FREQUENCY STATISTICS LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2779	HISTORIC EVENTS	0
STANDARD DEV	.3341	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1300	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at Downstream
End of Dallas Floodway for
1,200-Foot Swale (NED) Plan

Plate A-13



—— FLOW Frequency (without Exp. Prob.)

▪ Median Plotting Positions

········ 5% and 95% Confidence Limits

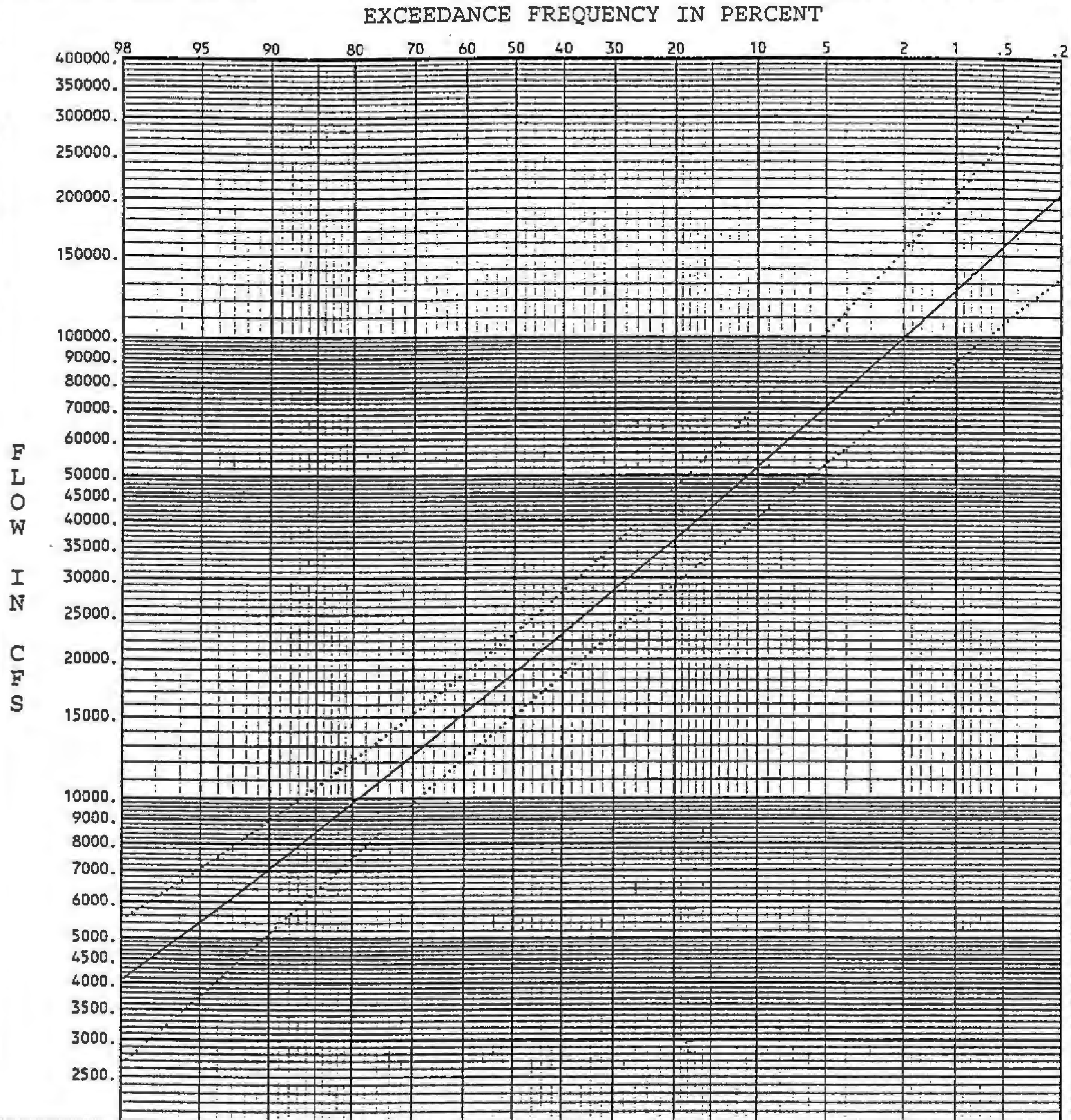
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

		NUMBER OF EVENTS	
MEAN	4.2800	HISTORIC EVENTS	0
STANDARD DEV	.3335	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1400	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at the Central
Wastewater Treat. Plant Levee
1,200-Foot Swale (NED) Plan

Plate A-14



—— FLOW Frequency (without Exp. Prob.)

▪ Median Plotting Positions

········ 5% and 95% Confidence Limits

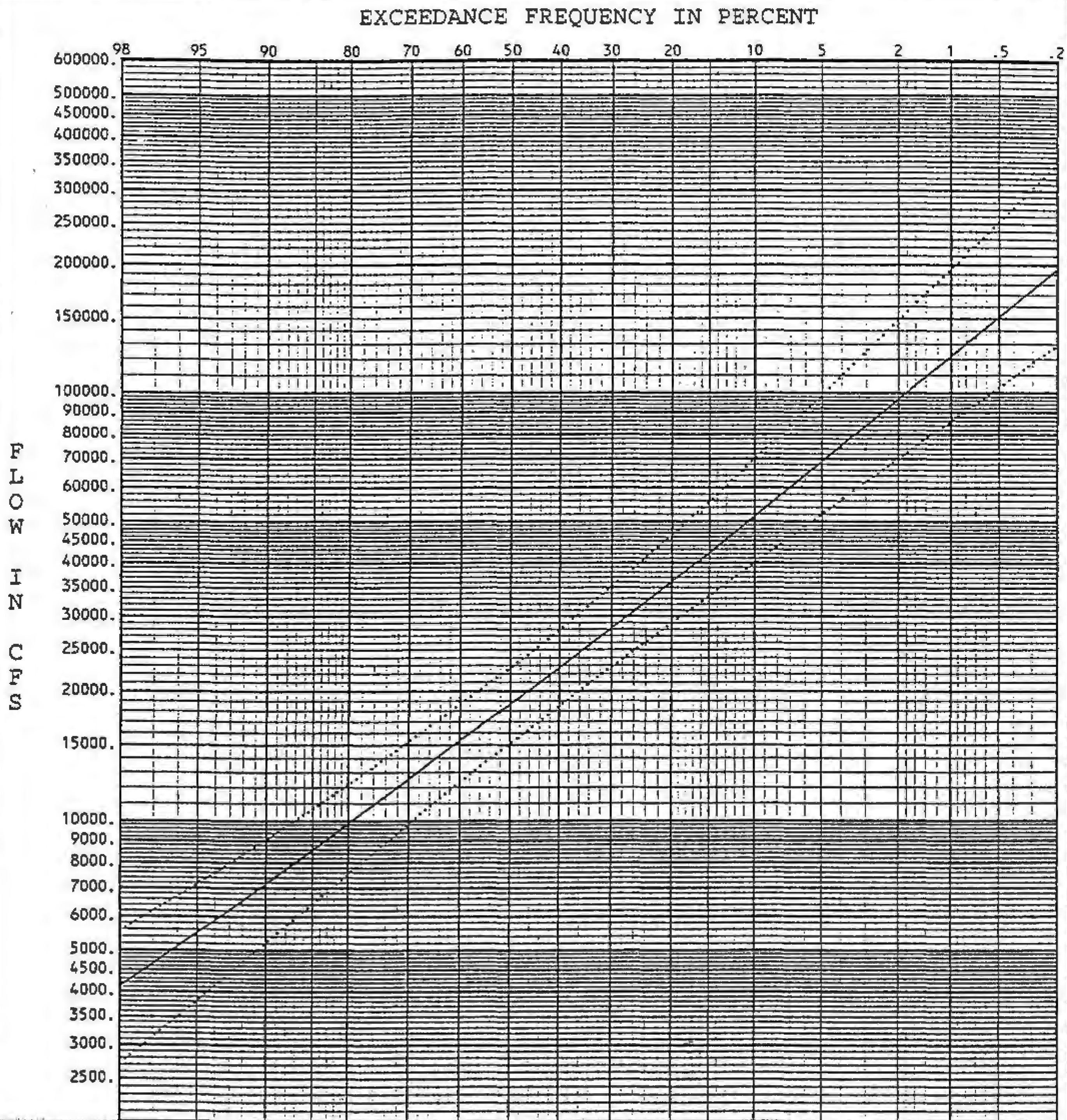
FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2779	HISTORIC EVENTS	0
STANDARD DEV	.3390	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1400	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at the "Below Dallas" Gage (Loop 12) For
1,200-Foot Swale (NED) Plan

Plate A-15



—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······· 5% and 95% Confidence Limits

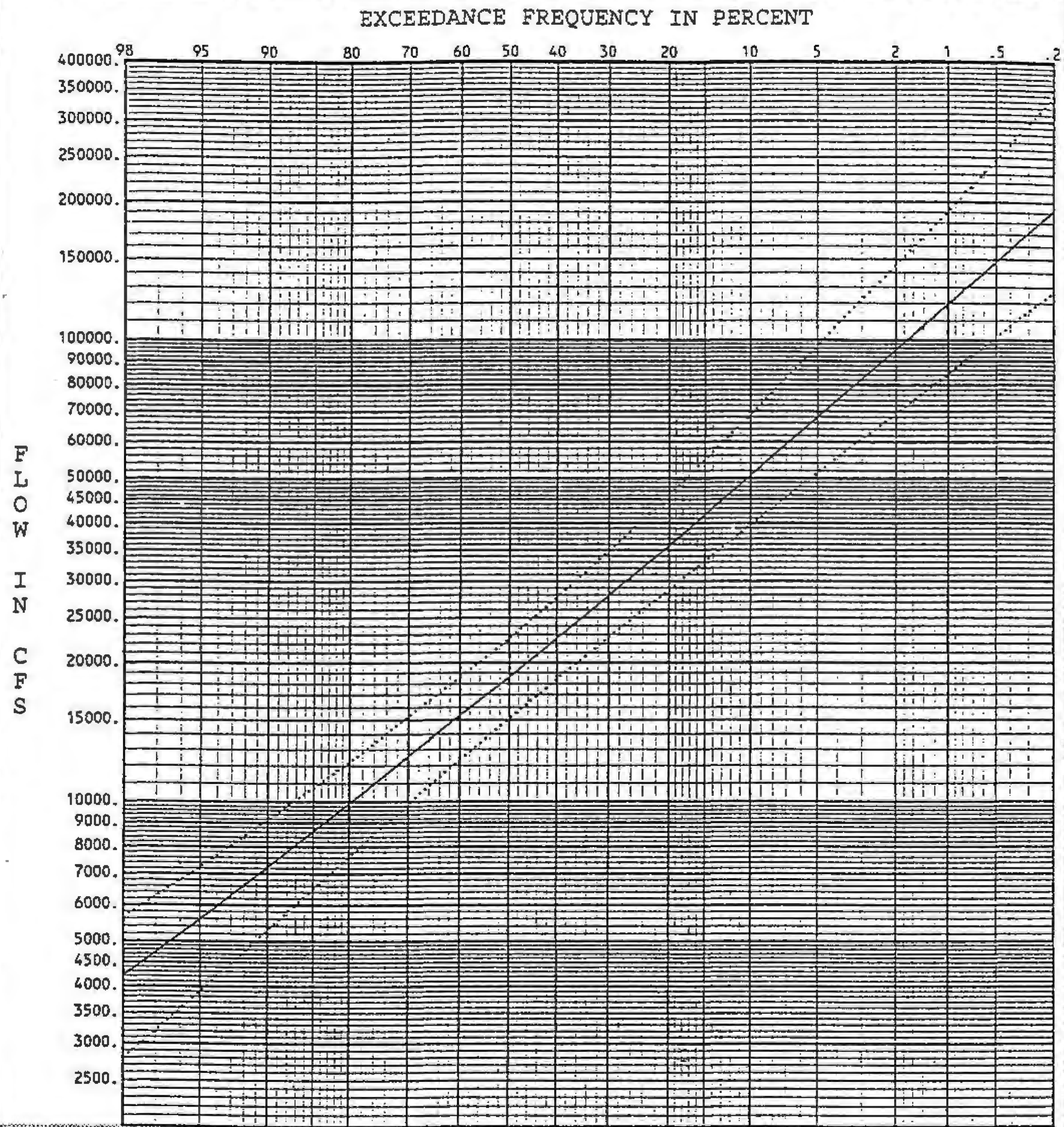
FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2779	HISTORIC EVENTS	0
STANDARD DEV	.3341	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1300	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at Downstream
End of the Dallas Floodway
For Recommended Plan

Plate A-16



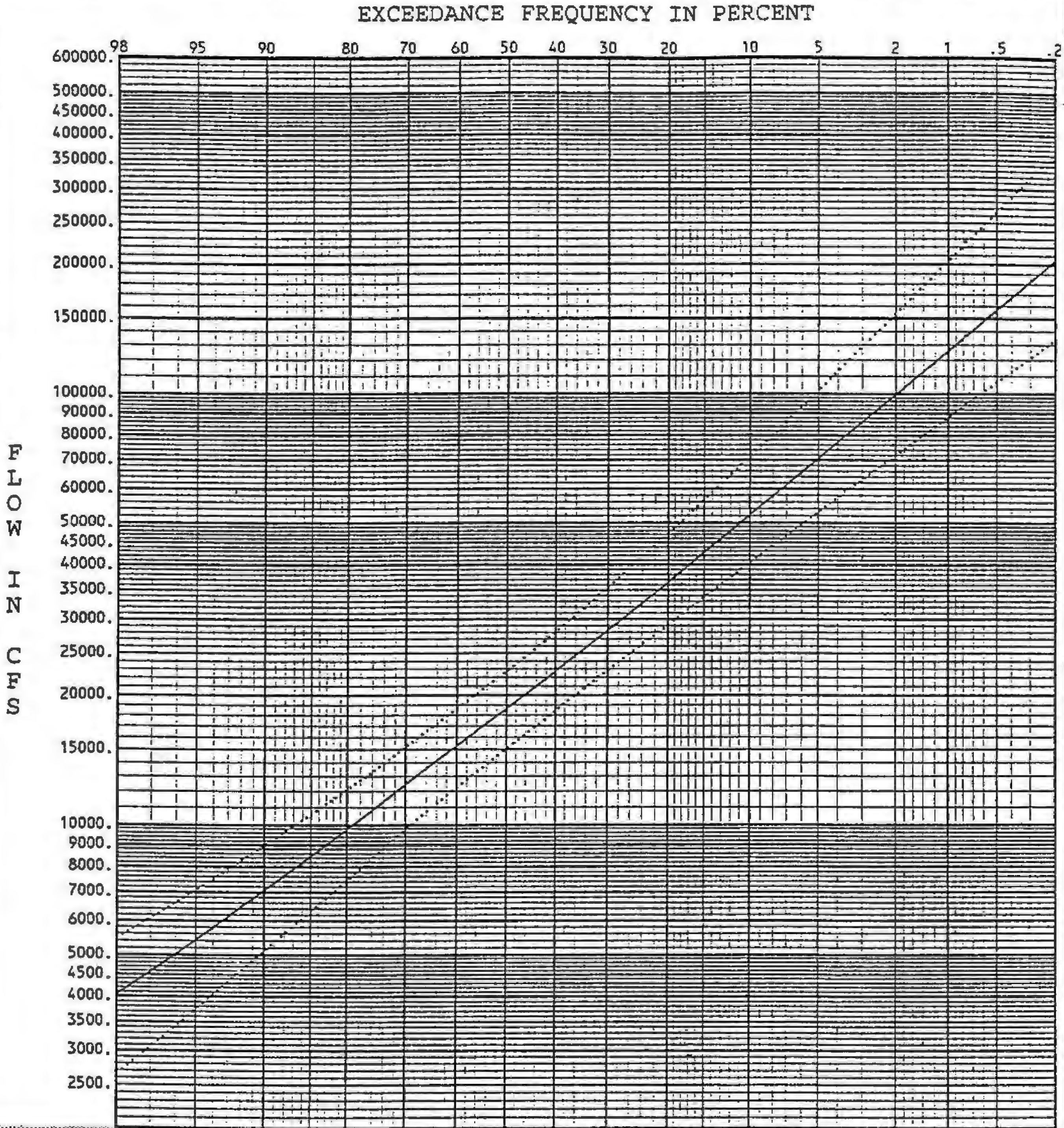
—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······· 5% and 95% Confidence Limits

FREQUENCY STATISTICS LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2770	HISTORIC EVENTS	0
STANDARD DEV	.3300	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1400	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at the Central
Wastewater Treat. Plant Levee
For Recommended Plan

Plate A-17



—— FLOW Frequency (without Exp. Prob.)
▪ Median Plotting Positions
······· 5% and 95% Confidence Limits

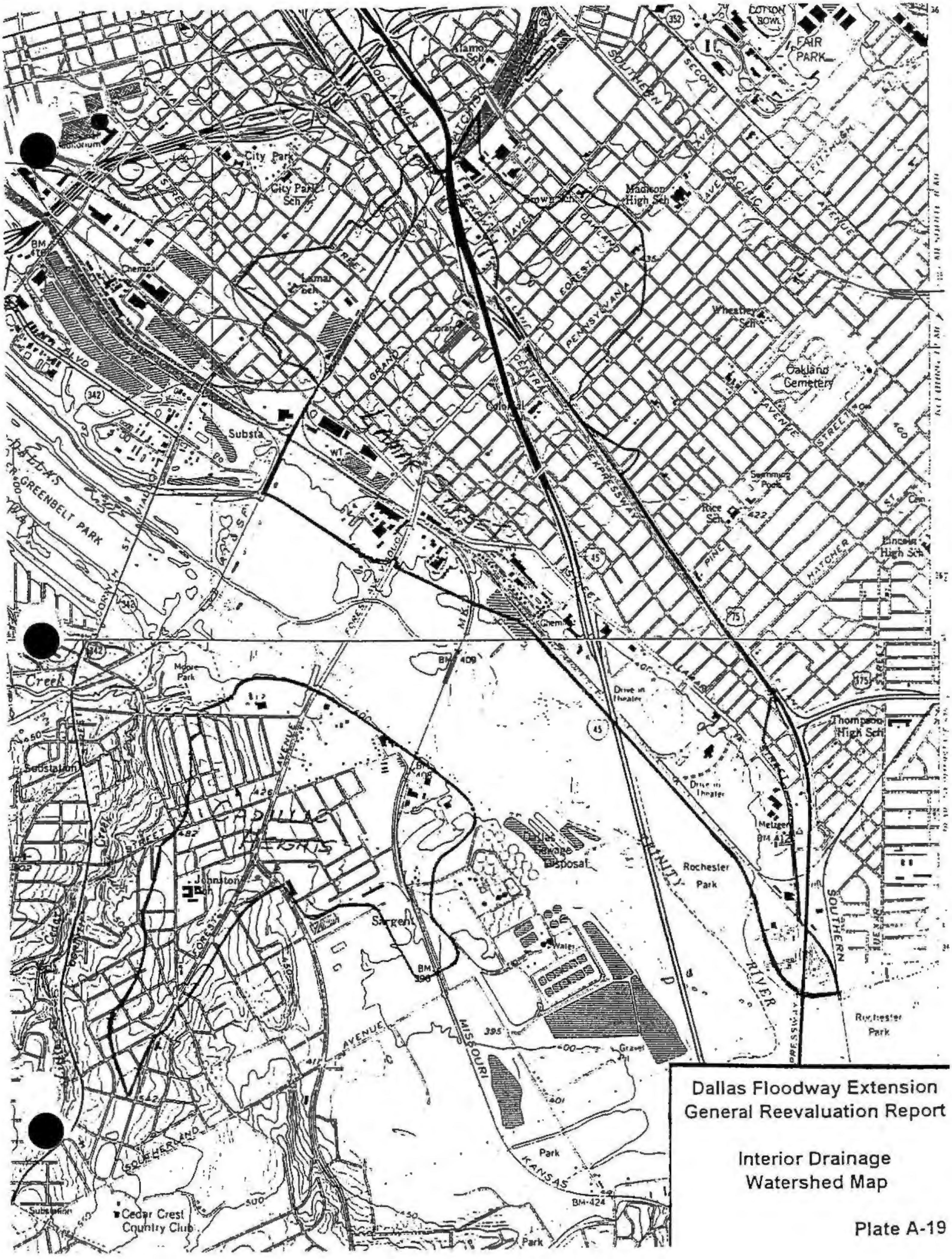
FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	4.2779	HISTORIC EVENTS	0
STANDARD DEV	.3390	HIGH OUTLIERS	0
SKEW	.0000	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.1400	SYSTEMATIC EVENTS	40

Dallas Floodway Extension
General Reevaluation Report

Discharge vs. Frequency Curve
Trinity River at the "Below Dallas" Gage (Loop 12)
For Recommended Plan

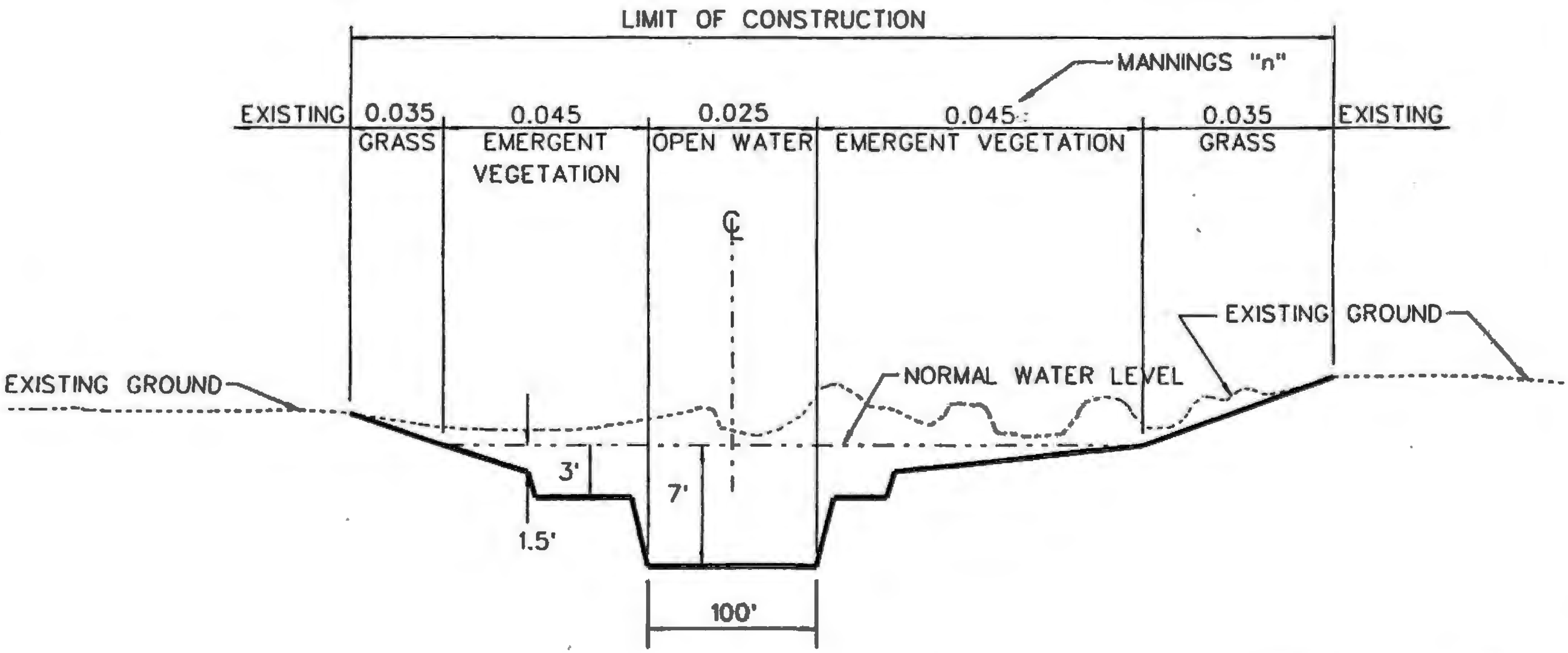
Plate A-18



Dallas Floodway Extension
General Reevaluation Report

Interior Drainage
Watershed Map

Plate A-19



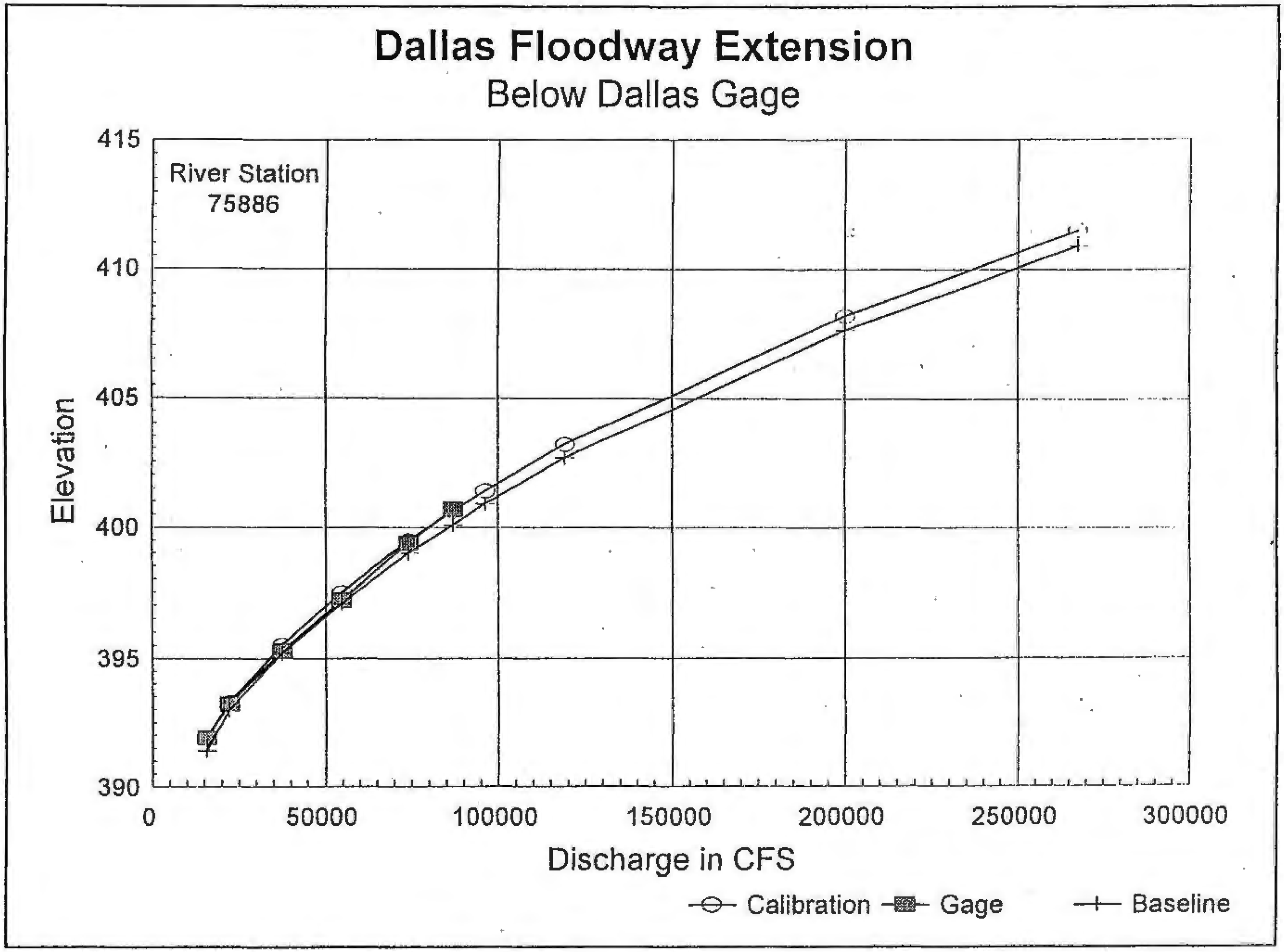
TYPICAL WETLAND CROSS-SECTION

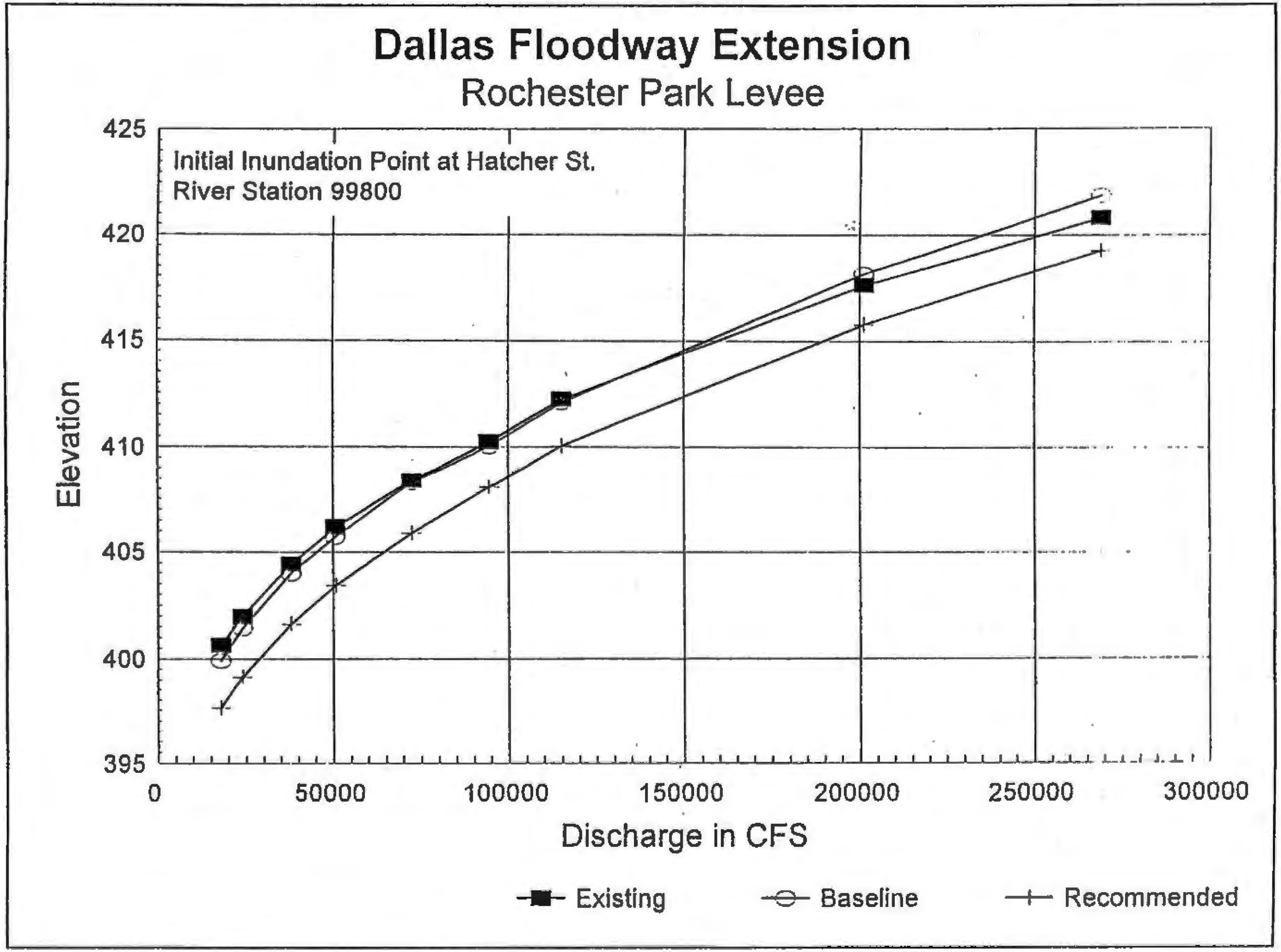
DALLAS FLOODWAY EXTENSION
GENERAL REEVALUATION REPORT

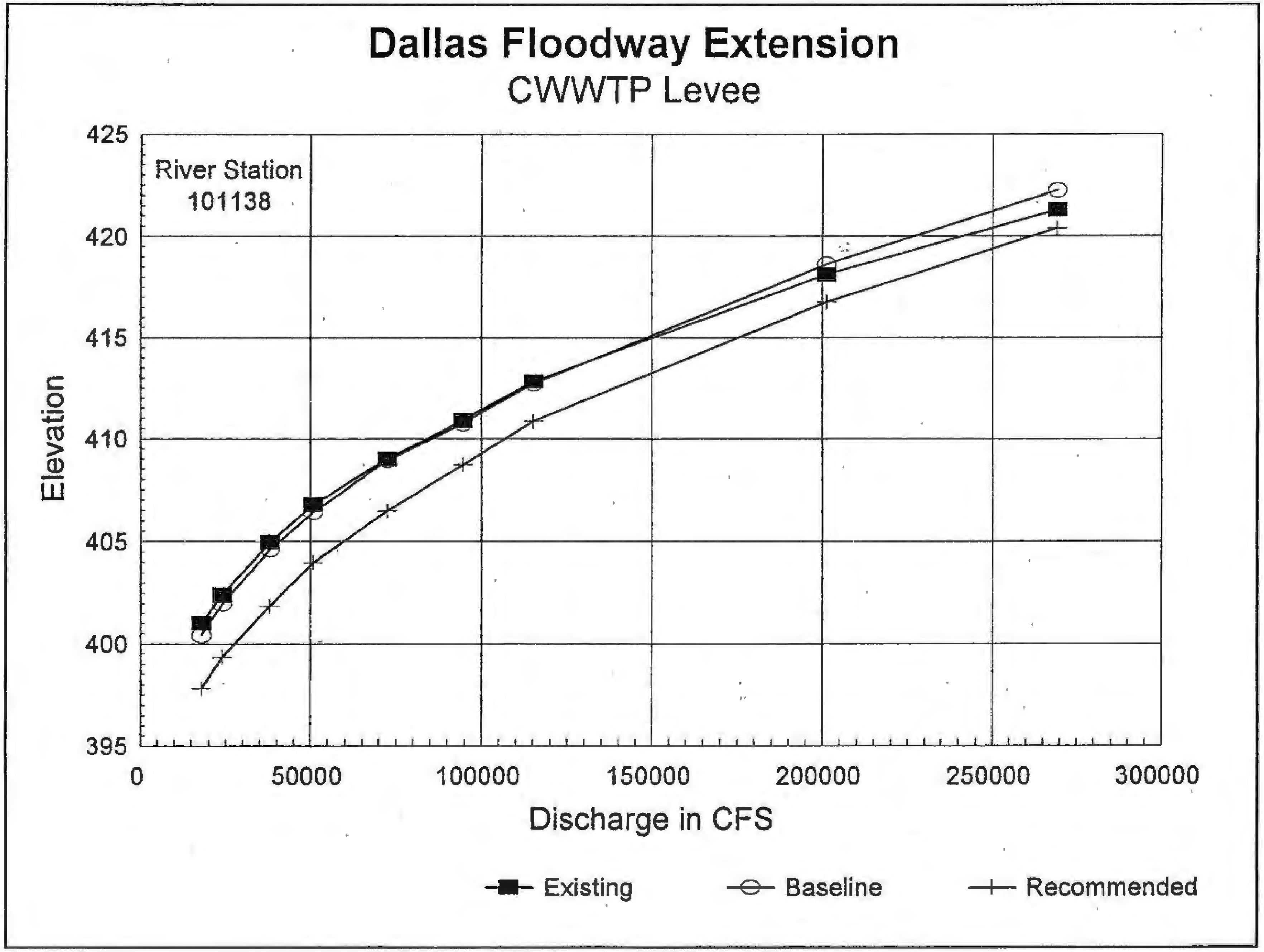
TYPICAL WETLAND CROSS SECTION

U.S. ARMY ENGINEER DISTRICT FT.WORTH

PLATE A-20


Dallas Floodway Extension
Below Dallas Gage
River Station
75886
Elevation
415
410
405
400
395
390
0
50000
100000
150000
200000
250000
300000
Discharge in CFS
Calibration
Gage
Baseline


Dallas Floodway Extension
Rochester Park Levee
Initial Inundation Point at Hatcher St.
River Station 99800
Elevation
425
420
415
410
405
400
395
0
50000
100000
150000
200000
250000
300000
Discharge in CFS
Existing
Baseline
Recommended


Dallas Floodway Extension
CWWTP Levee
River Station
101138
425
420
415
410
405
400
395
0
50000
100000
150000
200000
250000
300000
Elevation
Discharge in CFS
Existing
Baseline
Recommended

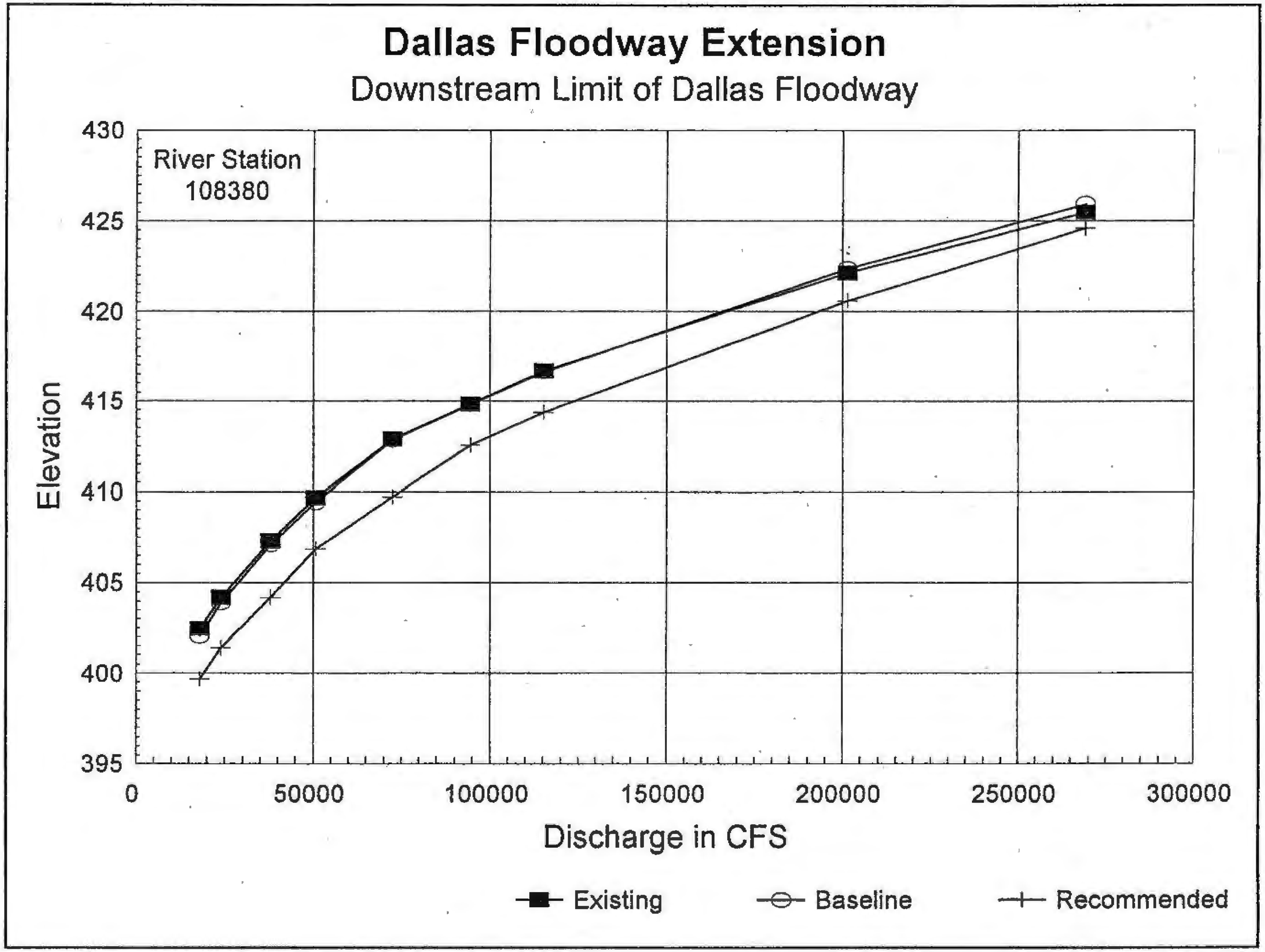
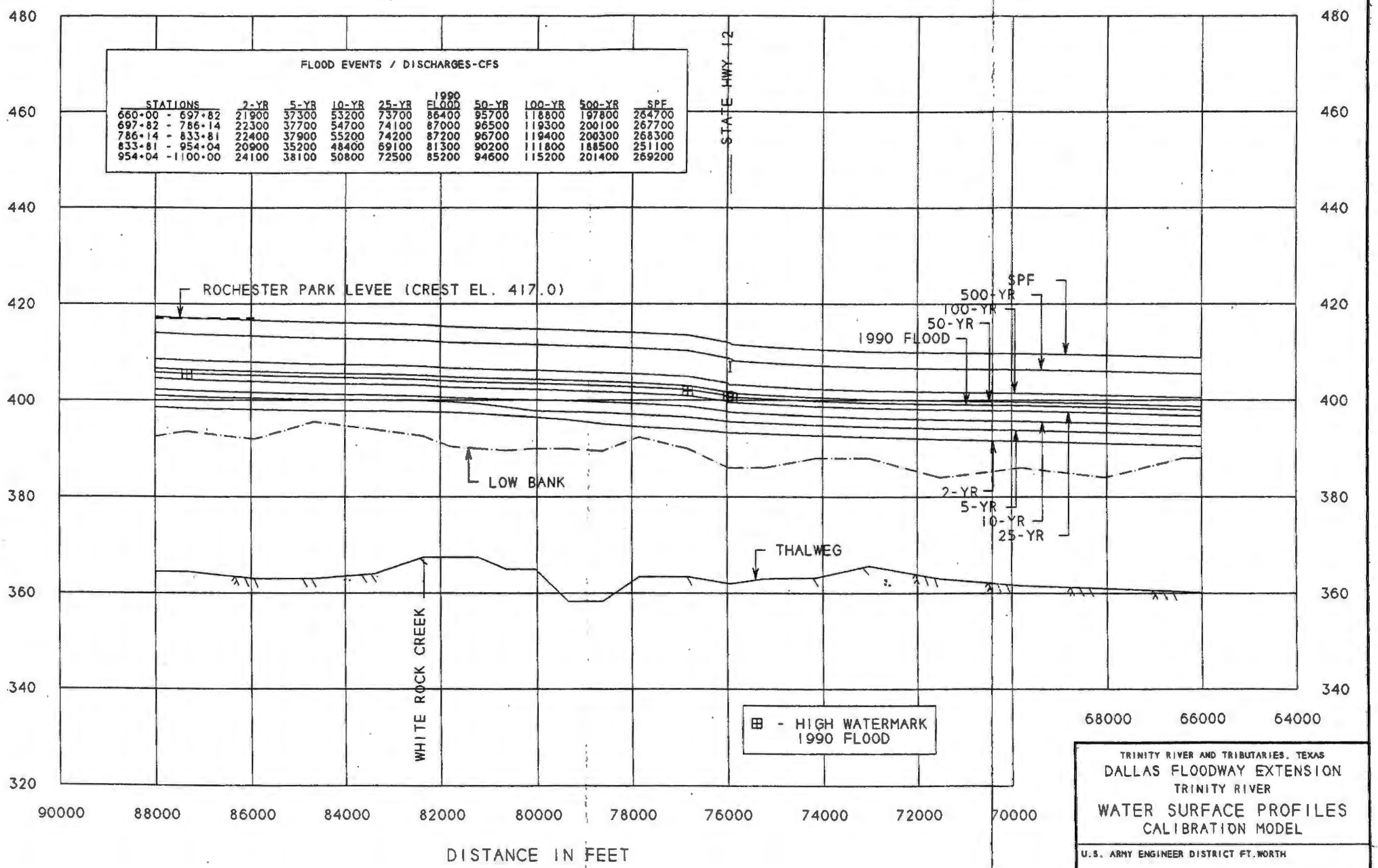

Dallas Floodway Extension
Downstream Limit of Dallas Floodway
River Station
108380
Elevation
430
425
420
415
410
405
400
395
0
50000
100000
150000
200000
250000
300000
Discharge in CFS
Existing
Baseline
Recommended

PLATE A-24



FLOOD EVENTS / DISCHARGES-CFS

STATIONS	2-YR	5-YR	10-YR	25-YR	1990 FLOOD	50-YR	100-YR	500-YR	SPF
660+00 - 697+82	21900	37300	53200	73700	86400	95700	118800	197800	264700
697+82 - 786+14	22300	37700	54700	74100	87000	96500	119300	200100	267700
786+14 - 833+81	22400	37900	55200	74200	87200	96700	119400	200300	268300
833+81 - 954+04	20900	35200	48400	69100	81300	90200	111800	188500	251100
954+04 -1100+00	24100	38100	50800	72500	85200	94600	115200	201400	269200

TRINITY RIVER AND TRIBUTARIES, TEXAS

DALLAS FLOODWAY EXTENSION

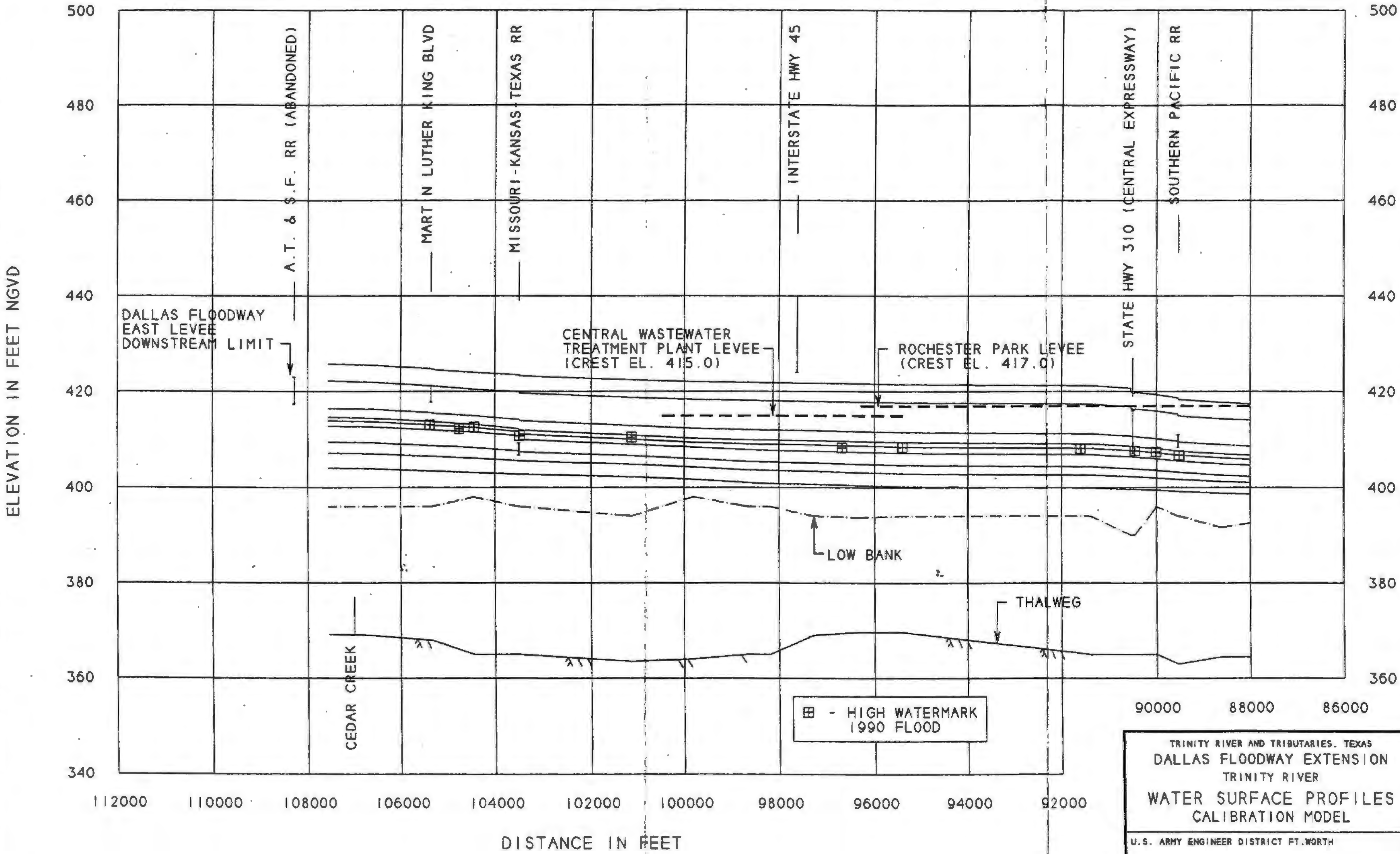
TRINITY RIVER

WATER SURFACE PROFILES

CALIBRATION MODEL

U.S. ARMY ENGINEER DISTRICT FT. WORTH

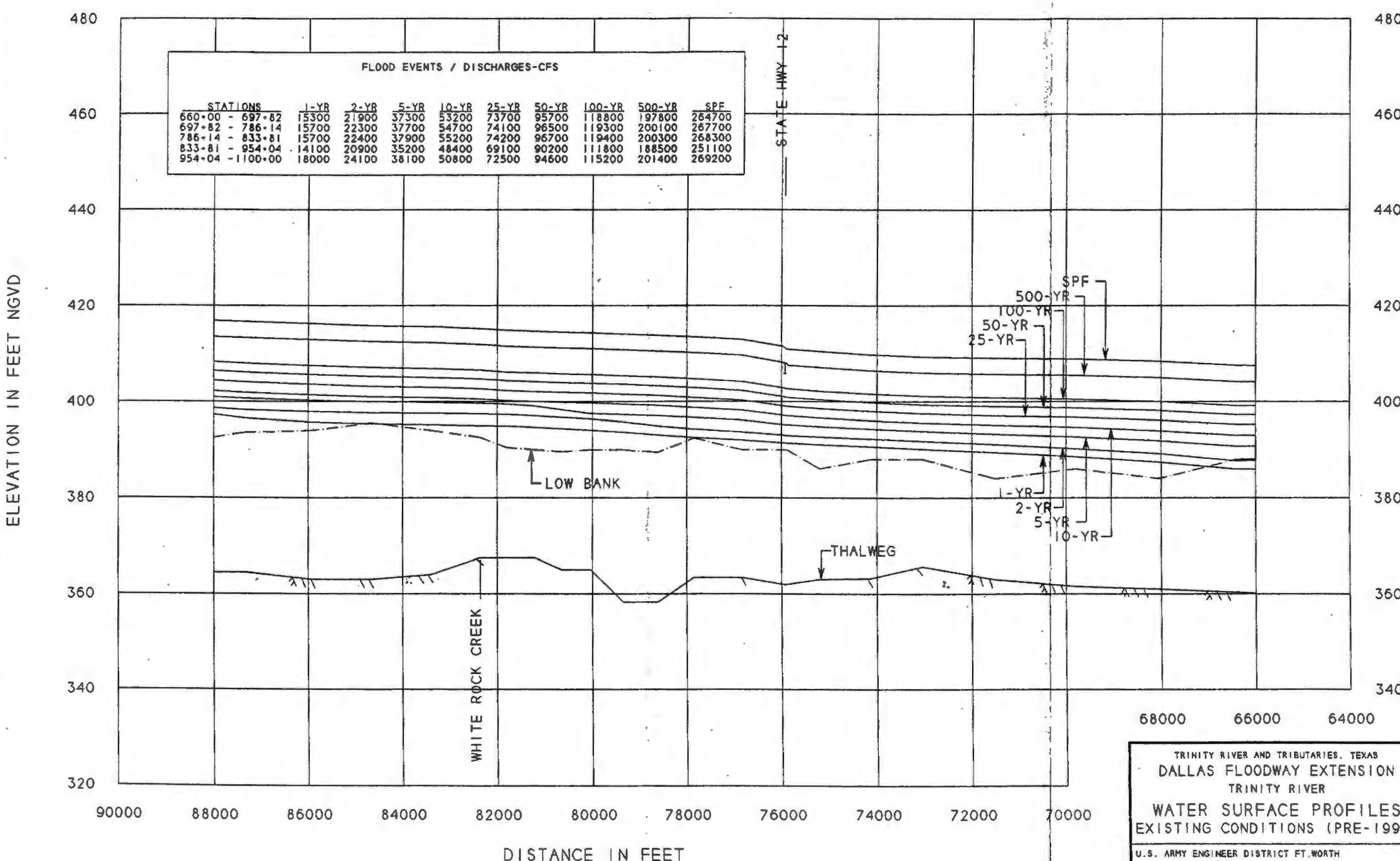
CORPS OF ENGINEERS U.S. ARMY



TRINITY RIVER AND TRIBUTARIES, TEXAS
DALLAS FLOODWAY EXTENSION
TRINITY RIVER
WATER SURFACE PROFILES
CALIBRATION MODEL

U.S. ARMY ENGINEER DISTRICT FT.WORTH

PLATE A-26



FLOOD EVENTS / DISCHARGES-CFS

STATIONS	1-YR	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	SPF
660+00 - 697+82	15300	21900	37300	53200	73700	95700	118800	197800	264700
697+82 - 786+14	15700	22300	37700	54700	74100	96500	119300	200100	267700
786+14 - 833+81	15700	22400	37900	55200	74200	96700	119400	200300	268300
833+81 - 954+04	14100	20900	35200	48400	69100	90200	111800	188500	251100
954+04 - 1100+00	18000	24100	38100	50800	72500	94600	115200	201400	269200

TRINITY RIVER AND TRIBUTARIES, TEXAS

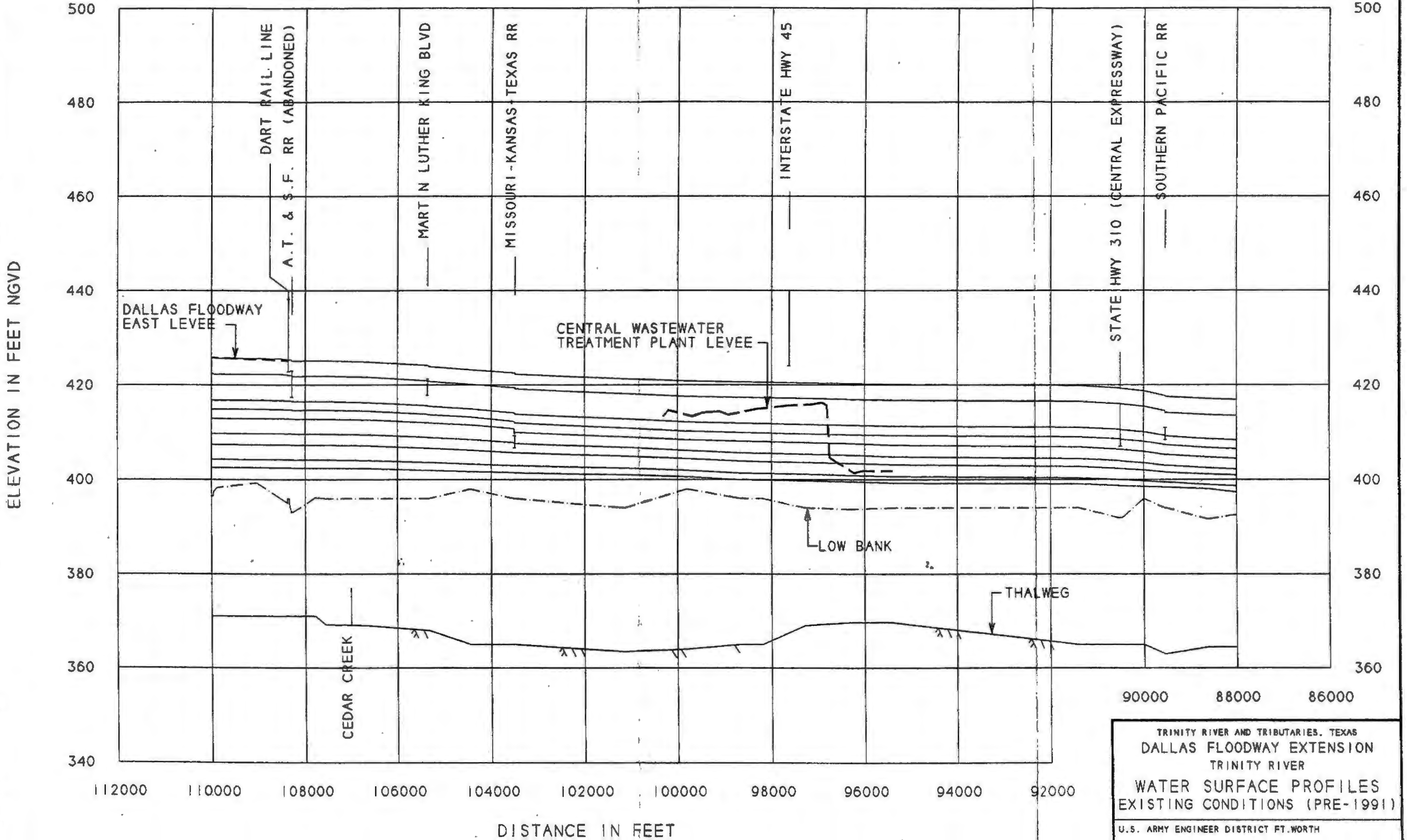
DALLAS FLOODWAY EXTENSION

TRINITY RIVER

WATER SURFACE PROFILES

EXISTING CONDITIONS (PRE-1991)

U.S. ARMY ENGINEER DISTRICT FT.WORTH



TRINITY RIVER AND TRIBUTARIES, TEXAS
DALLAS FLOODWAY EXTENSION
TRINITY RIVER
WATER SURFACE PROFILES
EXISTING CONDITIONS (PRE-1991)

U.S. ARMY ENGINEER DISTRICT FT.WORTH

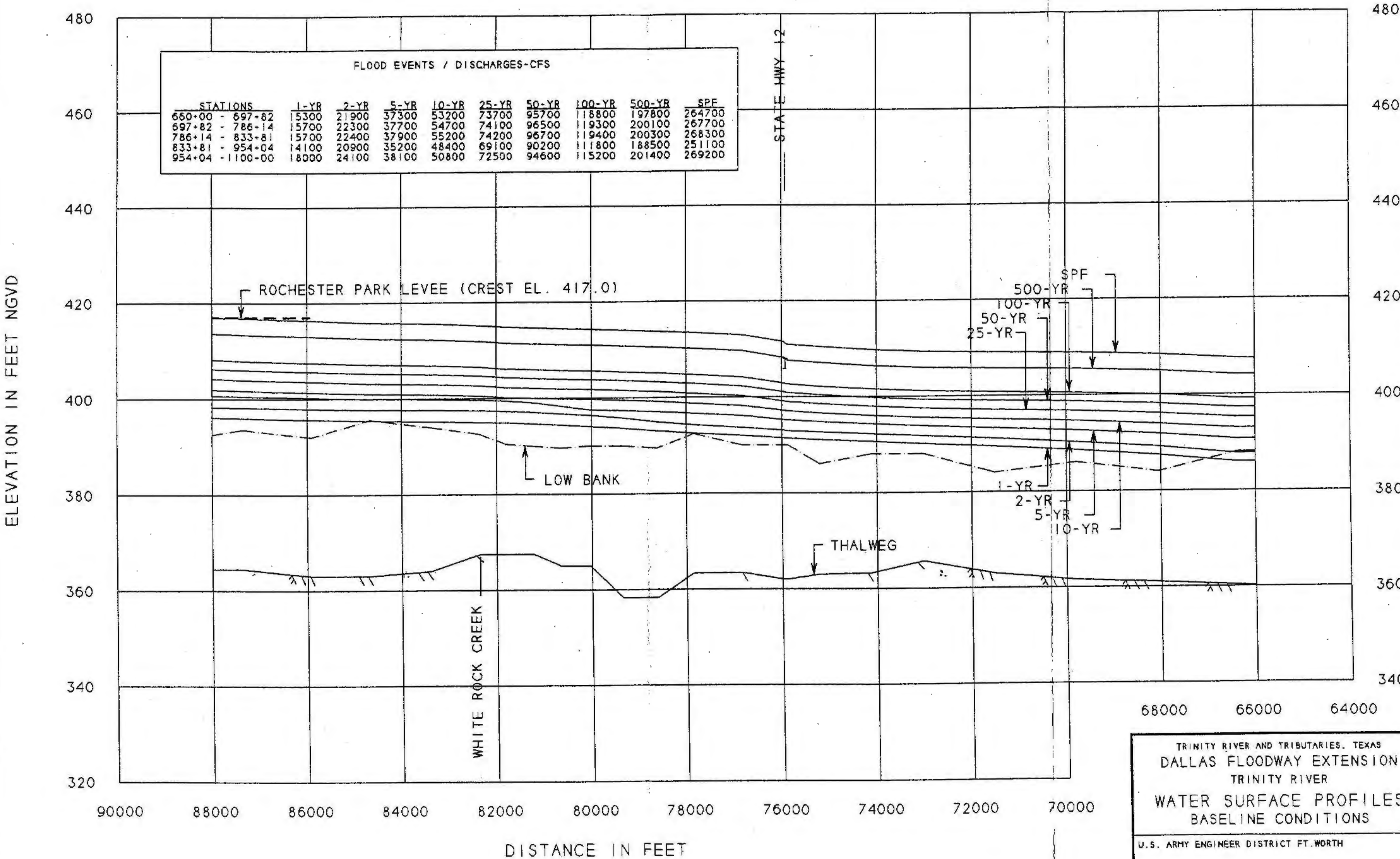
PLATE A-28

CORPS OF ENGINEERS

U.S. ARMY

FLOOD EVENTS / DISCHARGES-CFS

STATIONS	1-YR	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	SPF
660+00 - 697+82	15300	21900	37300	53200	73700	95700	118800	197800	264700
697+82 - 786+14	15700	22300	37700	54700	74100	96500	119300	200100	267700
786+14 - 833+81	15700	22400	37900	55200	74200	96700	119400	200300	268300
833+81 - 954+04	14100	20900	35200	48400	69100	90200	111800	188500	251100
954+04 - 1100+00	18000	24100	38100	50800	72500	94600	115200	201400	269200



TRINITY RIVER AND TRIBUTARIES, TEXAS

DALLAS FLOODWAY EXTENSION

TRINITY RIVER

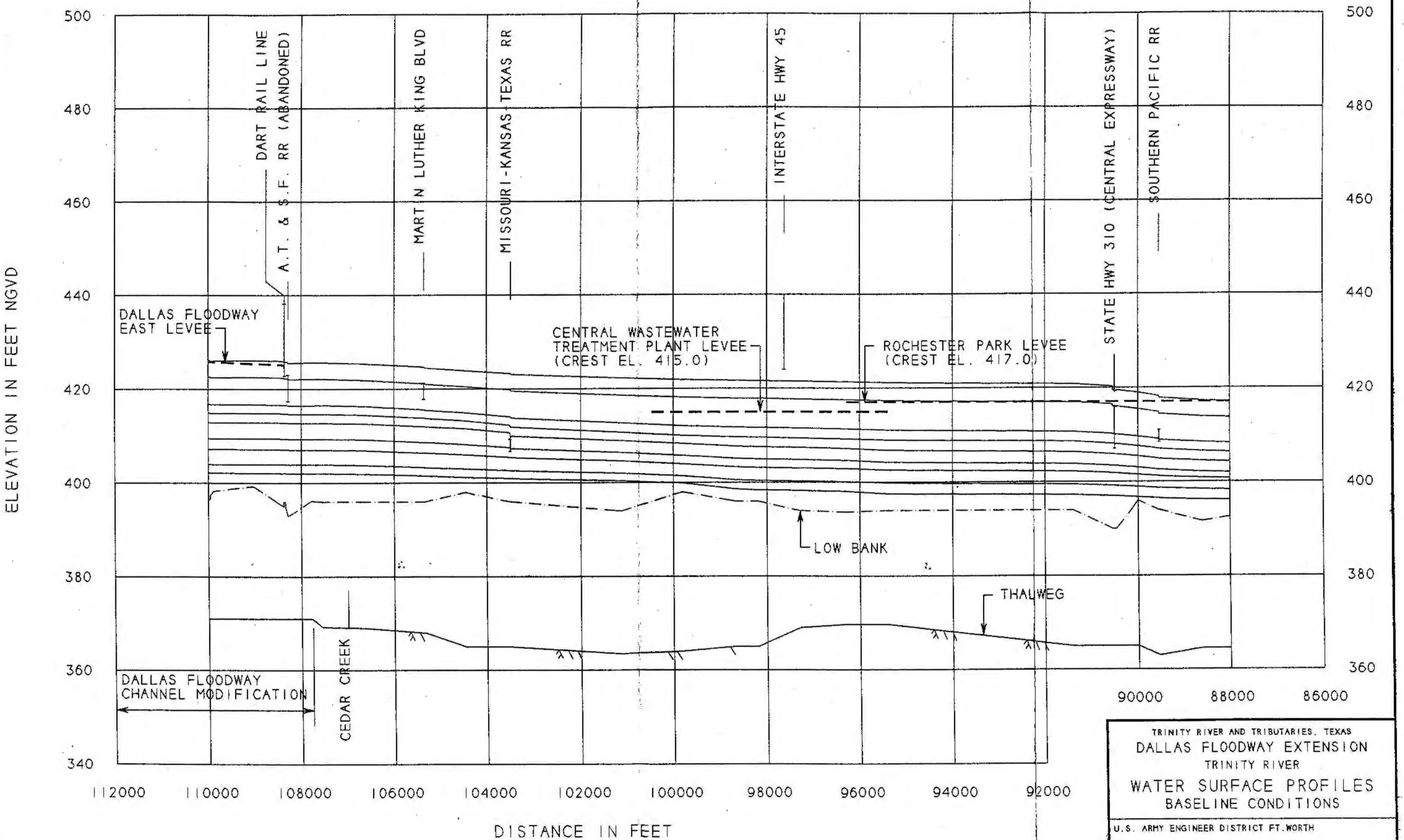
WATER SURFACE PROFILES

BASELINE CONDITIONS

U.S. ARMY ENGINEER DISTRICT FT. WORTH

PLATE A-29

CORPS OF ENGINEERS U.S. ARMY



TRINITY RIVER AND TRIBUTARIES, TEXAS
DALLAS FLOODWAY EXTENSION
TRINITY RIVER

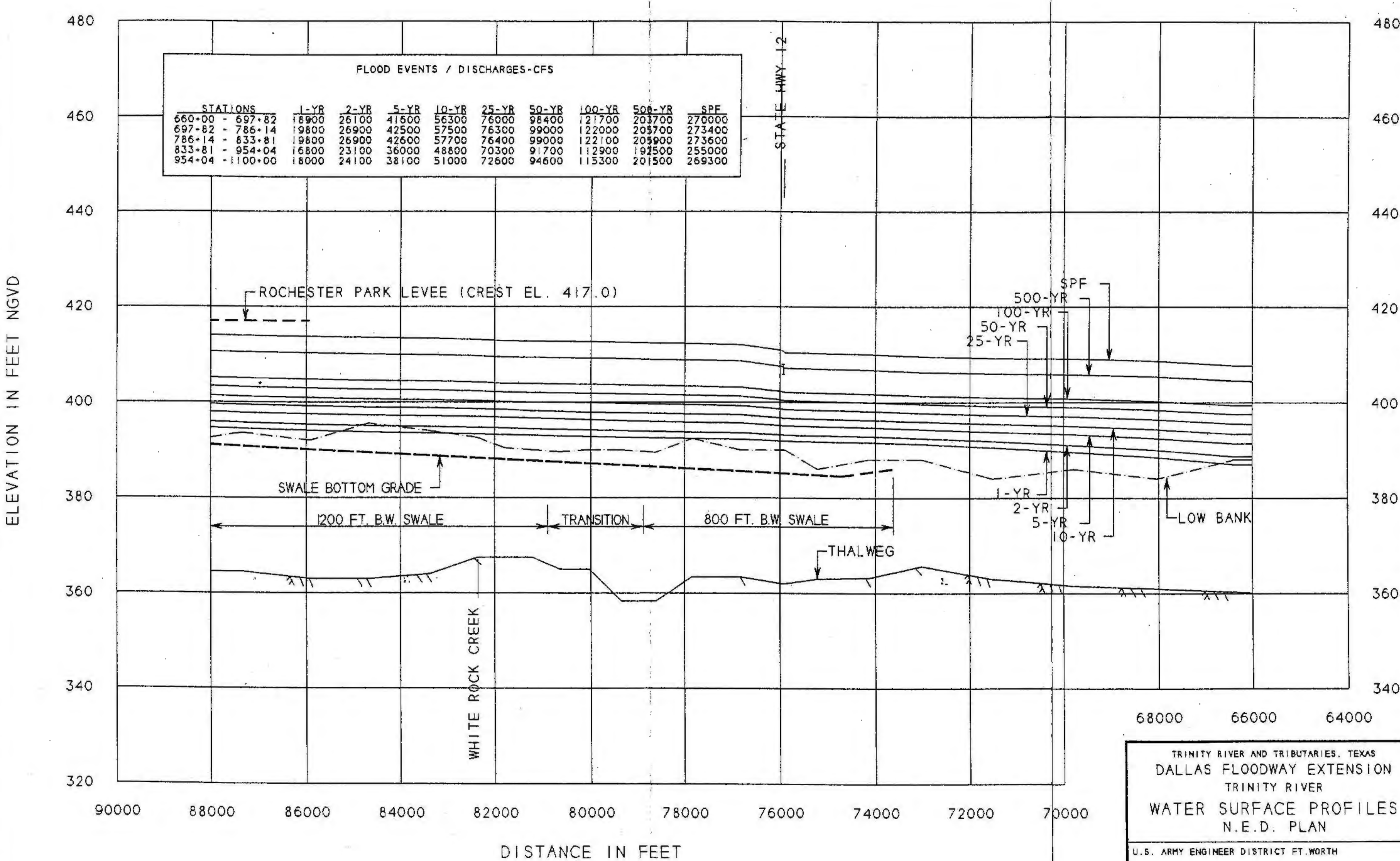
WATER SURFACE PROFILES
BASELINE CONDITIONS

U.S. ARMY ENGINEER DISTRICT FT. WORTH

PLATE A-30

CORPS OF ENGINEERS

U.S. ARMY



FLOOD EVENTS / DISCHARGES-CFS

STATIONS	1-YR	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	SPF
660+00 - 697+82	18900	26100	41600	56300	76000	98400	121700	203700	270000
697+82 - 786+14	19800	26900	42500	57500	76300	99000	122000	205700	273400
786+14 - 833+81	19800	26900	42600	57700	76400	99000	122100	205900	273600
833+81 - 954+04	16800	23100	36000	48800	70300	91700	112900	192500	255000
954+04 - 1100+00	18000	24100	38100	51000	72600	94600	115300	201500	269300

TRINITY RIVER AND TRIBUTARIES, TEXAS

DALLAS FLOODWAY EXTENSION

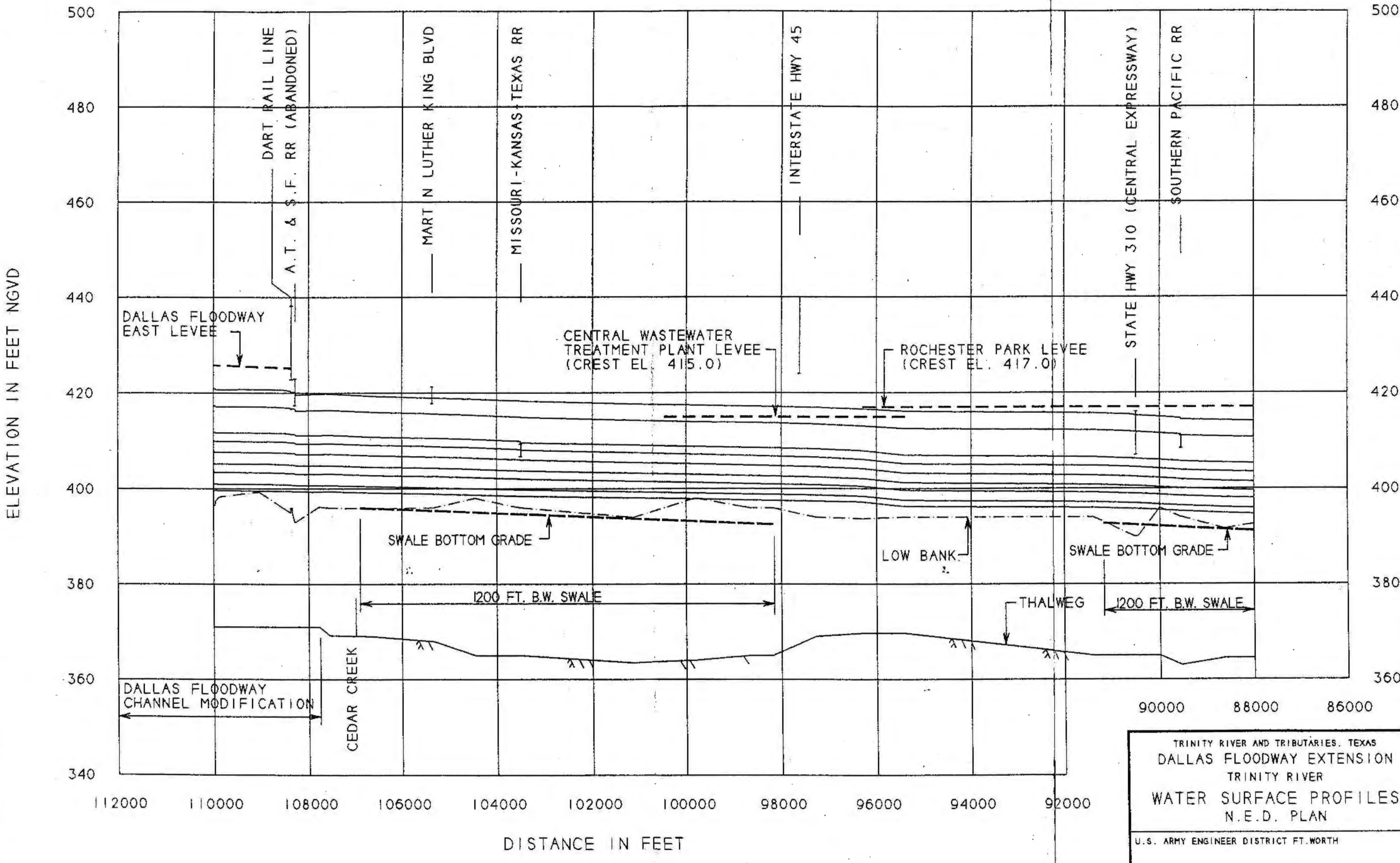
TRINITY RIVER

WATER SURFACE PROFILES

N.E.D. PLAN

U.S. ARMY ENGINEER DISTRICT FT. WORTH

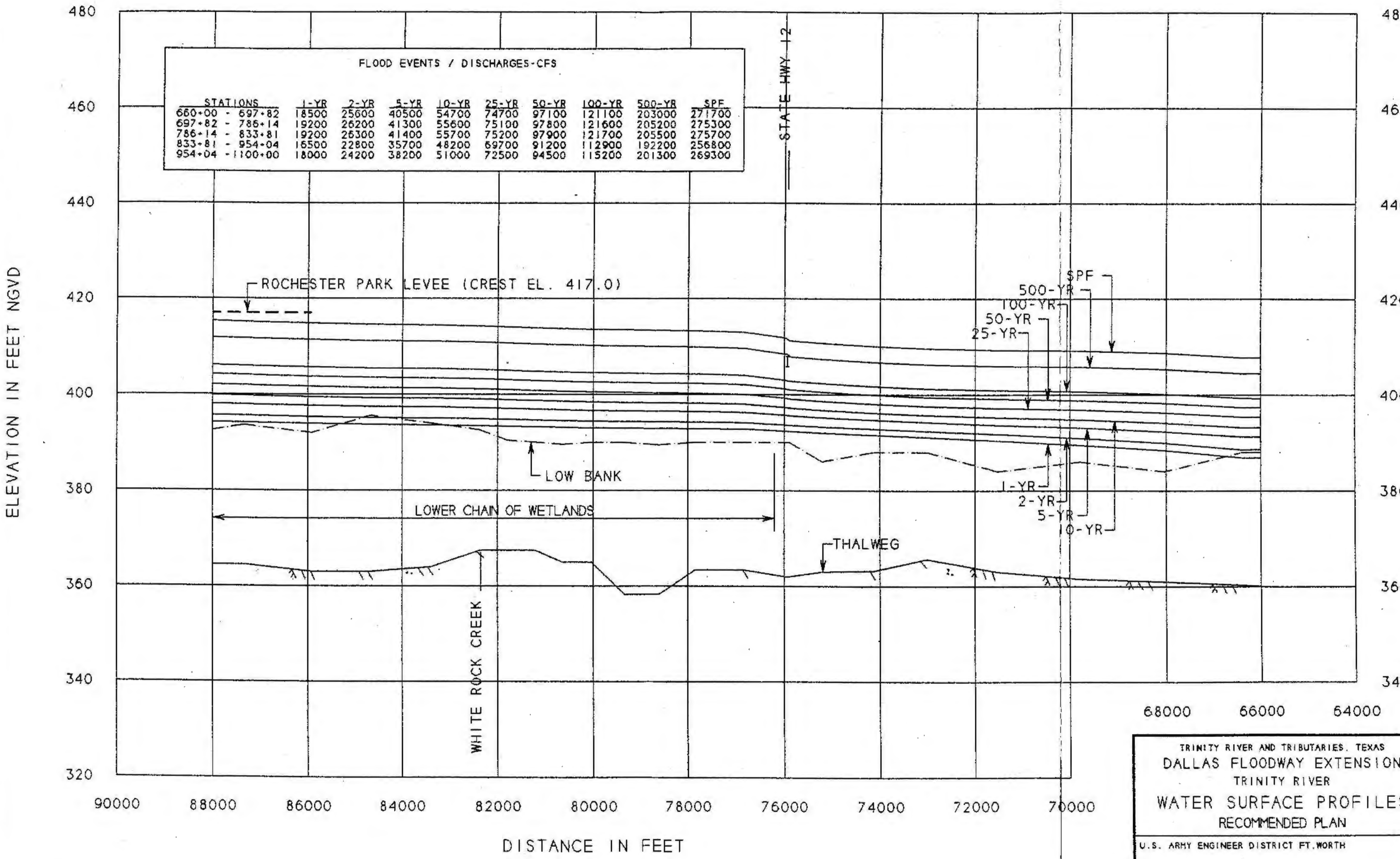
PLATE A-31



TRINITY RIVER AND TRIBUTARIES, TEXAS
DALLAS FLOODWAY EXTENSION
TRINITY RIVER
WATER SURFACE PROFILES
N.E.D. PLAN

U.S. ARMY ENGINEER DISTRICT FT. WORTH

PLATE A-32



FLOOD EVENTS / DISCHARGES-CFS

STATIONS	1-YR	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	SPF
660+00 - 697+82	18500	25600	40500	54700	74700	97100	121100	203000	271700
697+82 - 786+14	19200	26200	41300	55600	75100	97800	121600	205200	275300
786+14 - 833+81	19200	26300	41400	55700	75200	97900	121700	205500	275700
833+81 - 954+04	16500	22800	35700	48200	69700	91200	112900	192200	256800
954+04 - 1100+00	18000	24200	38200	51000	72500	94500	115200	201300	269300

TRINITY RIVER AND TRIBUTARIES, TEXAS

DALLAS FLOODWAY EXTENSION

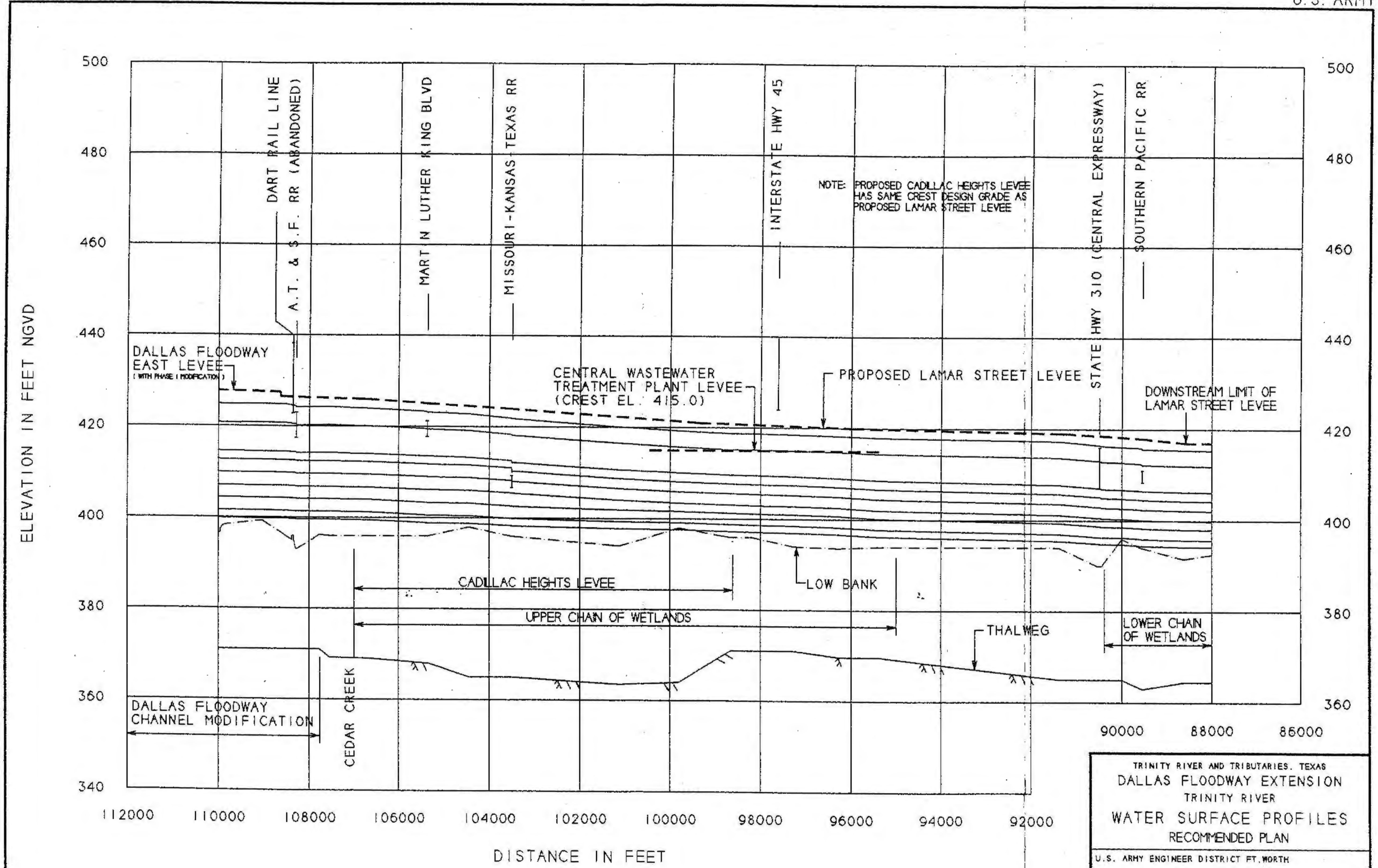
TRINITY RIVER

WATER SURFACE PROFILES

RECOMMENDED PLAN

U.S. ARMY ENGINEER DISTRICT FT. WORTH

PLATE A-33



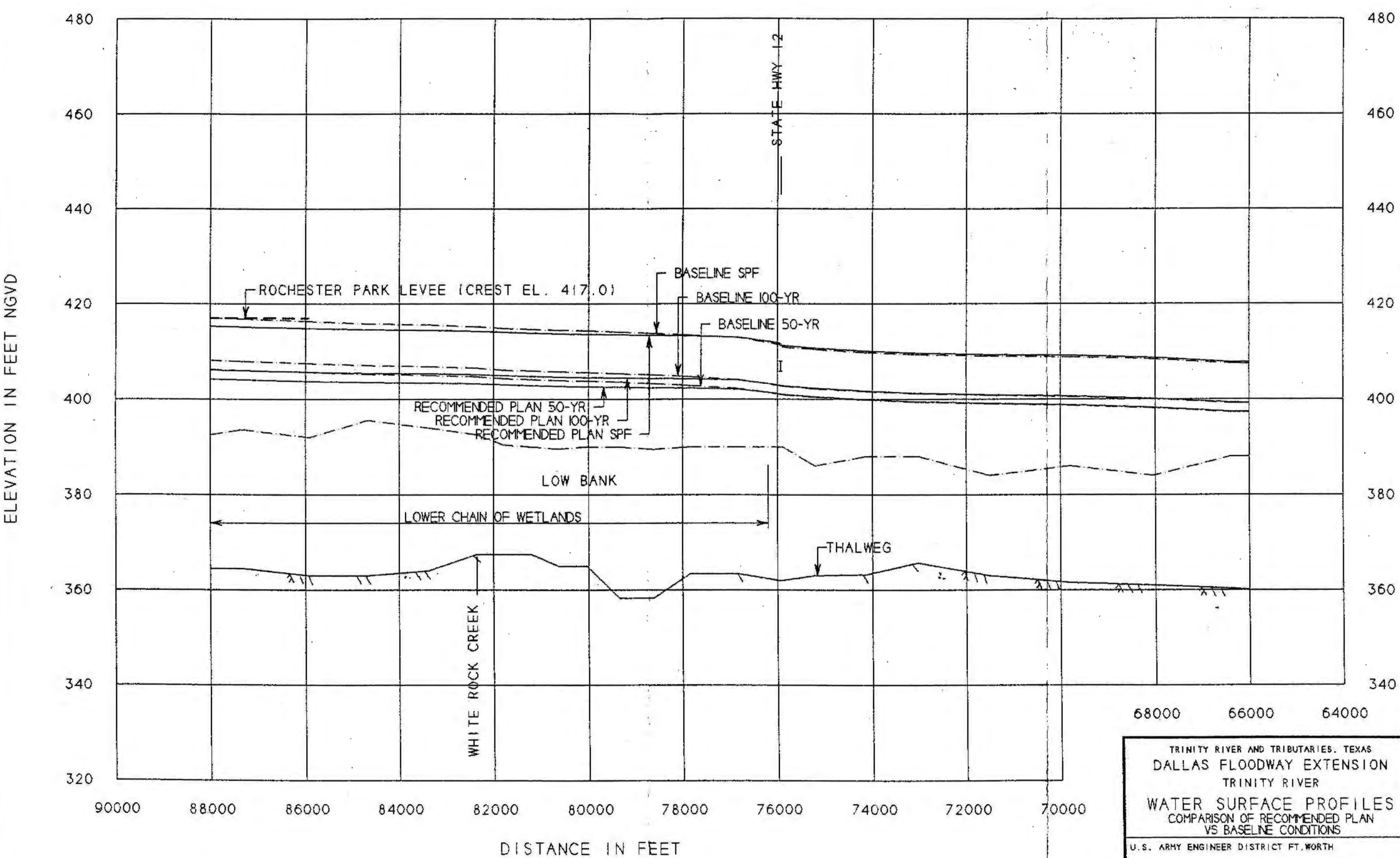
TRINITY RIVER AND TRIBUTARIES, TEXAS
DALLAS FLOODWAY EXTENSION
TRINITY RIVER
WATER SURFACE PROFILES
RECOMMENDED PLAN

U.S. ARMY ENGINEER DISTRICT FT. WORTH

PLATE A-34

CORPS OF ENGINEERS

U.S. ARMY



TRINITY RIVER AND TRIBUTARIES, TEXAS

DALLAS FLOODWAY EXTENSION

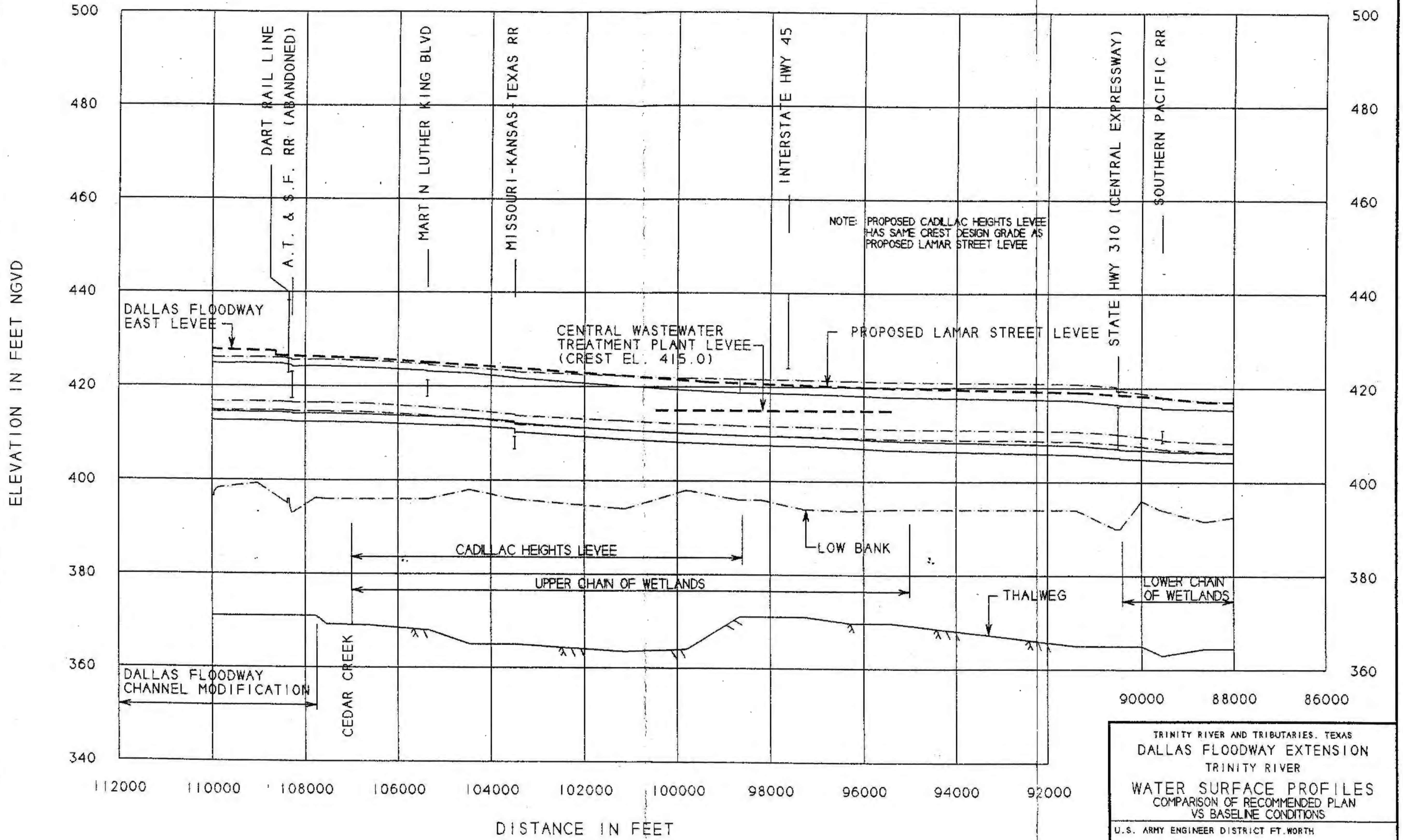
TRINITY RIVER

WATER SURFACE PROFILES

COMPARISON OF RECOMMENDED PLAN VS BASELINE CONDITIONS

U.S. ARMY ENGINEER DISTRICT FT. WORTH

PLATE A-35



TRINITY RIVER AND TRIBUTARIES, TEXAS

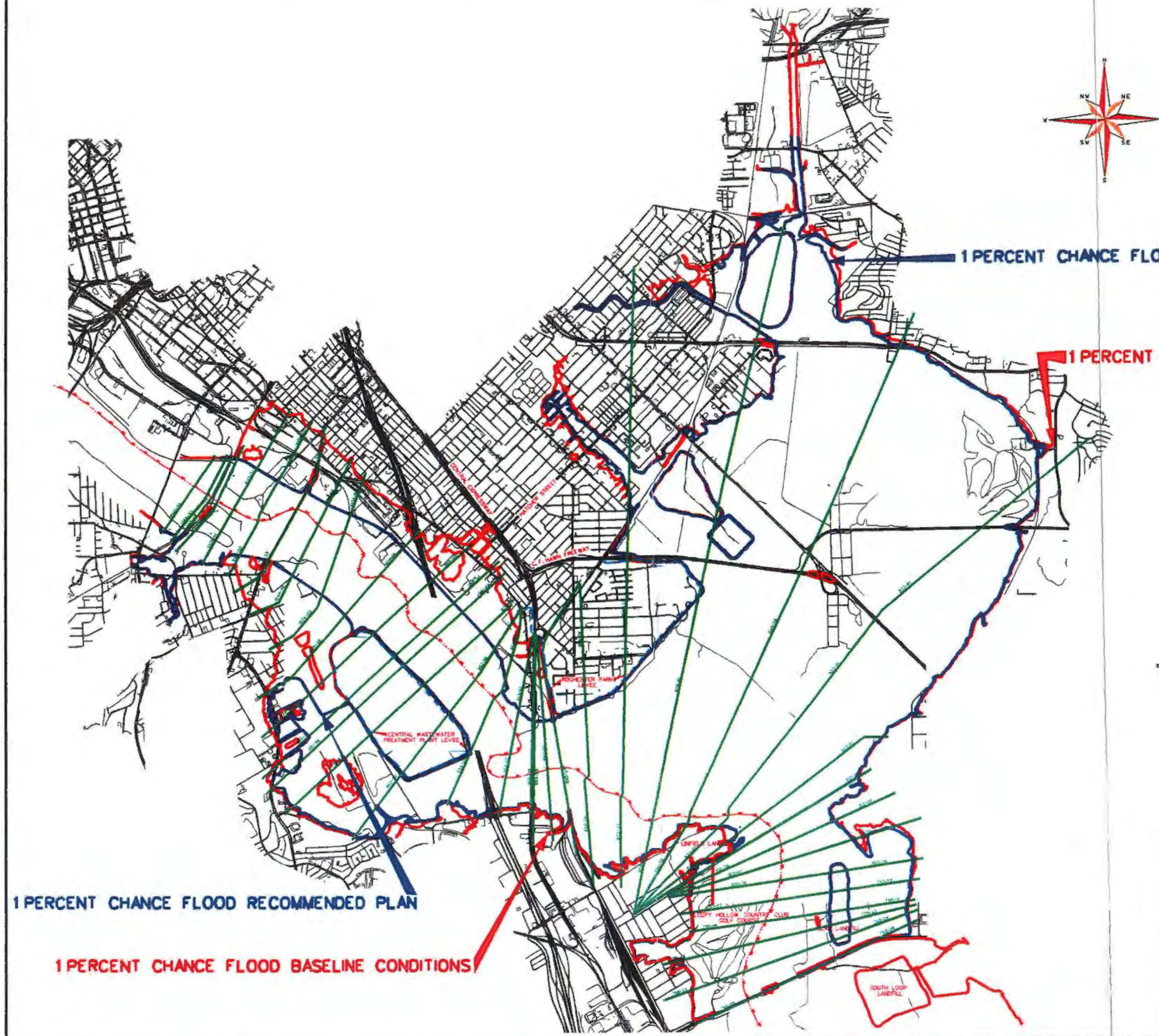
DALLAS FLOODWAY EXTENSION

TRINITY RIVER

WATER SURFACE PROFILES

COMPARISON OF RECOMMENDED PLAN VS BASELINE CONDITIONS

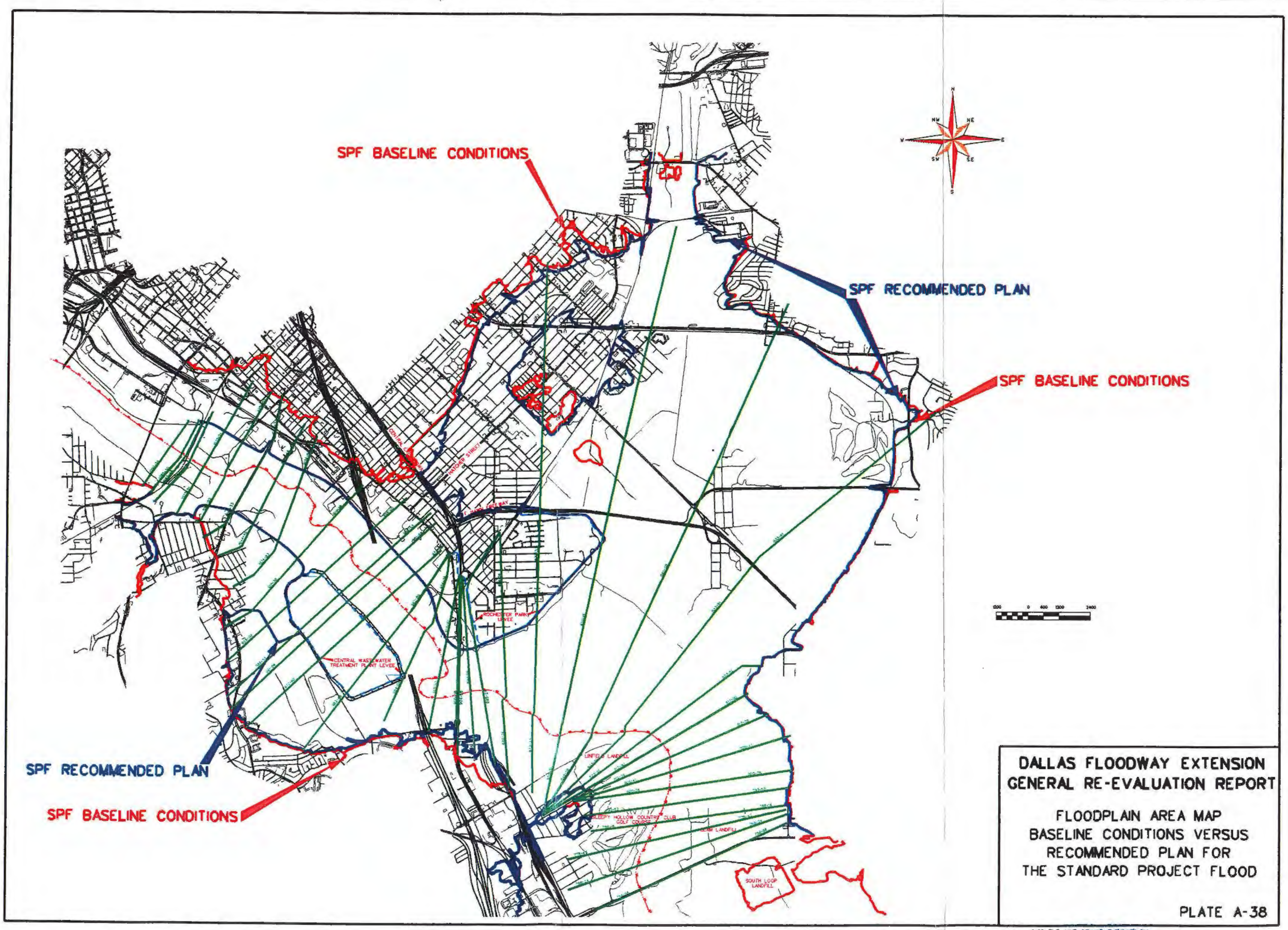
U.S. ARMY ENGINEER DISTRICT FT. WORTH



DALLAS FLOODWAY EXTENSION
GENERAL RE-EVALUATION REPORT

FLOODPLAIN AREA MAP
BASELINE CONDITIONS VERSUS
RECOMMENDED PLAN FOR
THE 1 PERCENT CHANCE FLOOD

PLATE A-37



DALLAS FLOODWAY EXTENSION
GENERAL RE-EVALUATION REPORT

FLOODPLAIN AREA MAP
BASELINE CONDITIONS VERSUS
RECOMMENDED PLAN FOR
THE STANDARD PROJECT FLOOD

PLATE A-38