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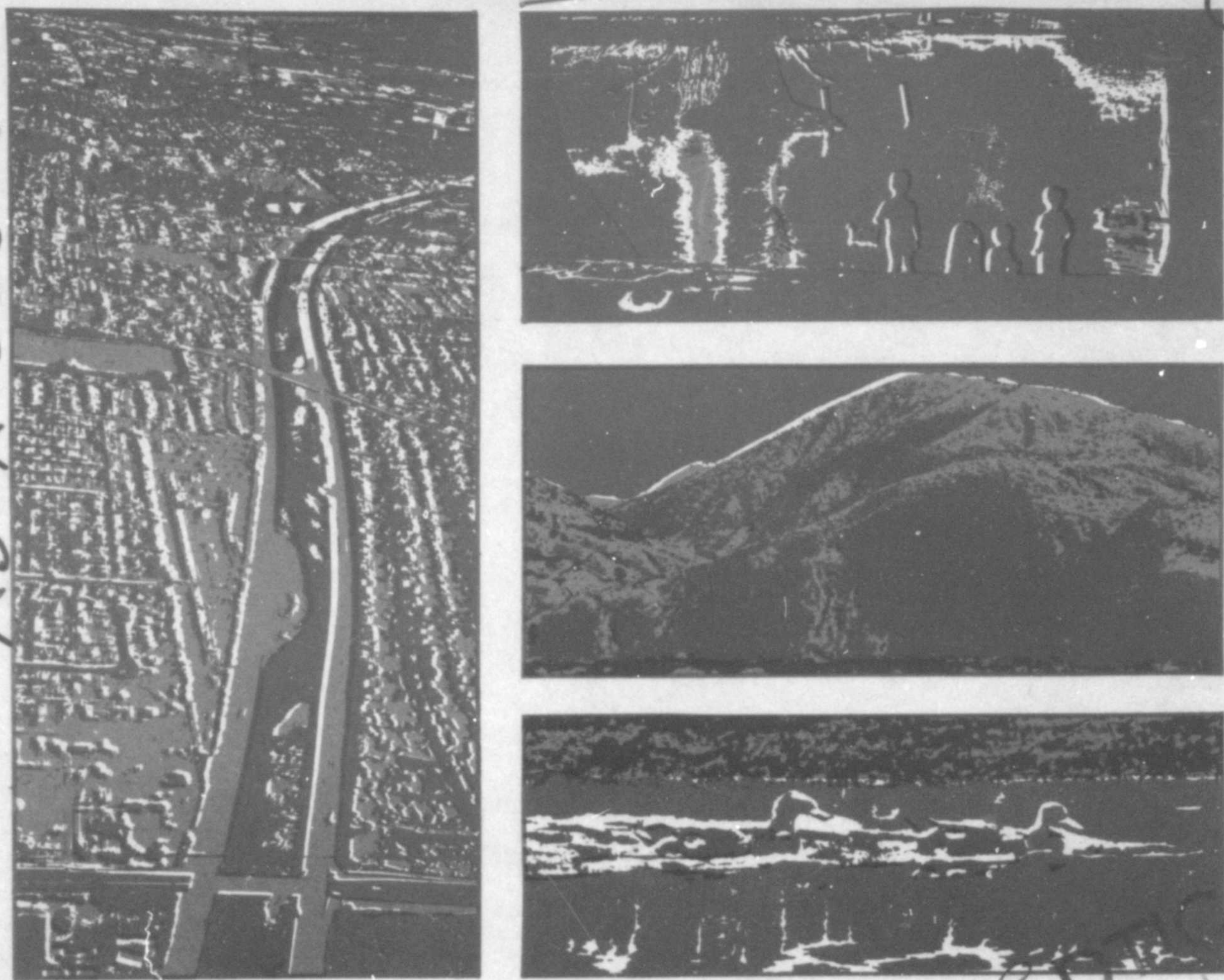

Santa Ana River

PHASE I GDM ON THE SANTA ANA RIVER MAIN STEM including Santiago Creek

TECHNICAL APPENDIXES (B,C,D,E&F)

SEPTEMBER 1980

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

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19. Real Estate (Appendix E), and Design and Cost Estimates (Appendix F).

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Appendix C (Hydraulic Design) describes the important parameters of the design work of the project: overflow analysis, drainage considerations, sedimentation, channel capacity, dam spillways and other structures.

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B Hydrology

APPENDIX B
HYDROLOGY

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Plates--Continued

SANTA ANA RIVER, CALIFORNIA

PHASE I--GENERAL DESIGN MEMORANDUM

HYDROLOGY

APPENDIX B

I. INTRODUCTION

PURPOSE AND SCOPE. This section of the Phase I--General Design Memorandum presents the results of investigations made for the Santa Ana River Basin in connection with flood control planning efforts not covered in the 1975 Review Report plus updating for changed conditions and new information. Primary emphasis was placed on new studies concerned with flood control on Santiago Creek, Oak Street Drain, and water conservation at Prado Dam. Generally, hydrology for the Santa Ana River Basin not discussed in this report can be found in the Review Report.

PREVIOUS REPORTS.

a. The most recent hydrology developed by the Corps of Engineers for the study area was presented in the "Review Report on the Santa Ana River Main Stem--Including Santiago Creek and Oak Street Drain, Appendix 2, Volume 2, Technical Information," dated December 1975.

b. Probable maximum and standard project flood inflow hydrographs for Prado Dam, presented in the report titled "Interim Report on Design Features of Existing Dams, Hydrology and Hydraulic Review for Prado, Brea, Fullerton, and Salinas Dams," dated November 1969, were approved by the Office of the Chief of Engineers on May 1970, for use in further studies related to the review of design features of Prado Dam.

c. Hydrology presented in the report titled "Hydrology, Santa Ana River Below Prado Dam, Orange County, California," dated July 1974, was approved by the South Pacific Division in the fifth endorsement, dated July 31, 1974, for use in survey report investigations.

II. GENERAL DESCRIPTIONS OF DRAINAGE AREA

PHYSIOGRAPHY AND TOPOGRAPHY.

a. The Santa Ana River Basin drains approximately 2,450 square miles, excluding a closed area of 32 square miles tributary to Baldwin Lake and 10 square miles tributary to Perris Reservoir. Of the total basin, 2,255 square miles are above Prado Dam, which is the major flood control structure on the Santa Ana River. The Santa Ana River Basin boundary is shown on plate B-1. Approximately 23 percent of the basin is within the rugged San Gabriel and San Bernardino Mountains about 9 percent is in the San Jacinto Mountains, and 5 percent is within the Santa Ana Mountains. Most of the remaining area is in the valleys formed by the broad alluvial fan along the base of these mountains. The numerous low hills in the alluvial valley areas include a few low hills north of San Bernardino; the Crafton Hills east of Redlands; the Jurupa Mountains north and west of Riverside; the Box Springs Mountains and the Badlands east of Riverside; and the Chino and Peralta Hills northeast of Anaheim. In general, the mountain ranges are steep and sharply dissected. Maximum elevation at San Antonio Peak in the San Gabriel Mountains is 10,080 feet; at San Gorgonio Mountain in the San Bernardino Mountains, 11,485 feet; and at Mount San Jacinto in the San Jacinto Mountains, 10,804 feet. The San Bernardino Mountains are the source of the Santa Ana River and of two of its principal tributaries, Bear and Mill Creeks. Lytle Creek, the largest tributary originating in the San Gabriel Mountains, is in the northwest part of the drainage area. The San Jacinto River has its origin in the San Jacinto Mountains southeast of Beaumont. The major tributary in the lower part of the basin is Santiago Creek, which originates in the Santa Ana Mountains as shown on plate B-1. The Santa Ana River has an average gradient of about 240 feet per mile in the mountains, about 20 feet per mile near Prado Dam, and about 15 feet per mile below Prado Dam. The average gradient of the tributaries is about 700 feet per mile in the mountains and 30 feet per mile in the valleys.

b. Santiago Creek, draining approximately 101 square miles, has its headwaters in the Santa Ana Mountains. It flows northwestward through Santiago Canyon and then southwestward through the Cities of Orange and Santa Ana into the Santa Ana River. Handy Creek is a major tributary to Santiago Creek in the study region. Most of the watershed is within Orange County, with a small portion of the headwaters in Riverside County. Elevations in the basin range from 110 feet at the confluence with the Santa Ana River to 5,687 feet at Santiago Peak in the Santa Ana Mountains. Stream gradients range from 25 feet per mile in the lower reaches of Santiago Creek to 305 feet per mile in the upper reaches. The basin is located on a portion of the coastal plain, the gradually sloping lowland apron that extends from the base of the Santa Ana Mountains to the Pacific Ocean. The soils of the coastal plain have a limited capability to absorb floodwaters; therefore, the greater part of the water must escape to the ocean. Vegetation varies considerably in the watershed. The mountain and foothill areas are covered with oaks and other trees, brush, and native grasses. Large segments of the valley area have been cleared of most of the native vegetation because of extensive development in the area. The remaining valley areas are mainly covered with orchards, crops, and eucalyptus and sycamore trees. Plate B-2 shows the Santiago Creek drainage basin.

c. Hagador, Tin Mine, and Kroonen Canyons rise in the steep eastern slopes of the Santa Ana Mountains and combine at the Oak Street debris basin to form the beginning of the Oak Street channel. The channel flows northward over a wide alluvial plain, through the western portion of the City of Corona to Temescal Wash. Flows from Mabey Canyon and Lincoln Avenue Drain enter Oak Street channel upstream of its confluence with Temescal Wash. The total drainage area is about 12 square miles. Elevations vary from 3,800 feet at the headwaters to 1,000 feet at the debris basin to 570 feet at the mouth. Slopes range from about 600 feet per mile in the upper basin to 200 feet per mile in the lower basin. Vegetation in the basin is similar to that found in the Santiago Creek basin. Plate B-3 presents the Oak Street Drain drainage area.

STORM TYPES.

a. Three types of storms produce precipitation in the Santa Ana River Basin: general winter storms, local storms, and general summer storms.

b. General winter storms usually occur during the period from December through March. They originate over the Pacific Ocean as a result of the interaction between polar Pacific and tropical Pacific air masses and move eastward over the basin. These storms, which often last for several days, reflect orographical influences and are accompanied by widespread precipitation in the form of rain and, at higher elevations, some snow.

c. Local storms can occur at any time of the year, either during general storms or as isolated phenomena. Those that occur in the winter are generally associated with frontal systems. These storms cover comparatively small areas but result in high-intensity precipitation for durations of up to 6 hours.

d. General summer storms in this area are usually associated with tropical cyclones and occur very infrequently. They are known to have occurred in the late summer and early fall months but have not resulted in any major floods during the period of record.

EXISTING STRUCTURES.

a. Four major flood control dams are located in the Santa Ana River Basin. Three of these structures, Prado Dam, San Antonio Dam, and Carbon Canyon Dam, were built by the Corps of Engineers. The fourth, Villa Park Dam, was built by the Orange County Flood Control District. The locations of these dams are shown on plate B-4. Other existing flood control improvements have been constructed by the Corps of Engineers and local interests. The improvements include channelization, storm drains, levees, stone and wire-mesh fencing, and stone walls along the banks of stream channels. The principal existing water conservation improvements are spreading grounds and reservoirs. The more than 100 water conservation and recreational reservoirs within the basin have storage capacities ranging in sizes from 5 to about 182,000 acre-feet. The locations of the large water conservation reservoirs are indicated on plate B-4.

b. The Santiago Creek channel has been improved over the years by local interests. During the 1930's, masonry walls were constructed from the Santa Ana Freeway through Hart Park. Within Hart Park, the channel bottom has been paved for use as a parking lot. Riprap was placed along the west bank upstream from Chapman Avenue for the protection of homes along the bank. Downstream from Prospect Avenue, concrete sideslope protection has been placed to protect homes damaged by the 1969 floods. On Handy Creek, a concrete channel runs from its confluence with Santiago Creek to a point just downstream of Orange Park Boulevard. Several large gravel pits in the upper reaches of Santiago Creek act as reservoirs for floodwaters. Minor floods are completely contained by these pits, and flows never reach the downstream channel. However, during major floods, these pits will already be filled when the peak flow occurs, consequently, flood protection downstream is significantly reduced. Villa Park Dam, just upstream from the study reach, is a flood control facility constructed by the Orange County Flood Control District in 1963. Santiago Reservoir, just upstream from Villa Park Dam, is a water supply reservoir constructed by the Irvine Company in 1933. The general location of the gravel pits and improved channel are shown on plate B-4. Plate B-25 presents existing flood control improvements, in greater detail, for the lower subareas of Santiago Creek.

c. Within the Oak Street Drain drainage basin, an existing debris basin has been completed by Riveside County Flood Control District in October 1979. Located immediately north of Chase Drive, it controls about 6 square miles of runoff and debris from Kroonen Canyon, Hagador Canyon, and Tin Mine Canyon. The design provides an estimated debris storage of 253 acre-feet and a spillway to pass 7,700 cfs. The location of the debris basin is shown on plate B-3. Steel rail and wire revetment line Oak Street Drain from the debris basin to the confluence with Mangular Drain. A concrete channel runs from this point down to Railroad Street. The remaining reach to Temescal Wash is natural channel. The existing channel capacity for the entire reach is about 600 cfs.

RUNOFF CHARACTERISTICS. Streamflow, which is perennial in the canyons of the Santa Ana River and in the headwaters of most of the tributaries, is generally intermittent in the valley sections. Streamflow increases rapidly in response to effective precipitation. High-intensity precipitation in combination with the effects of steep gradients and possible denudation by fire result in intense sediment-laden floods, with some debris in the form of shrubs and trees. Deposition of the sediment occurs on the mountain streams as they flow into the valley when stream gradients become flatter. The urbanization taking place in the valley areas of the Santa Ana River Basin tends to make the basin more responsive to rainfall. Hence, the same rainfall occurring over an urbanized part of the basin will generate higher peak discharges with a shorter peaking time and a greater volume than if it occurred over the natural basin without urbanization.

III. PRECIPITATION AND RUNOFF

STORMS AND FLOODS OF RECORD.

a. Although historical references to flood conditions in the general region date back to about 1769, little information is available regarding the magnitude of floods prior to 1850. Historical references indicate that (from 1769 to 1850) medium-to-large floods occurred in 1825, 1833, 1840, and 1850. Some quantitative data are available to show that from 1850 to 1897, medium-to-large winter floods occurred in 1859, 1862, 1867, 1876, 1884, 1886, 1889, and 1894. Recorded data from 1897 to the present show that medium-to-large winter floods occurred in 1903, 1910, 1914, 1916, 1921, 1922, 1927, 1938, 1943, 1965, 1966, 1969, and 1978. Since the historical floods of the 1800's and early 1900's, considerable change has occurred in the drainage basin. The runoff characteristics of the majority of the valley area have been changed by urbanization and agriculture. The mountain areas have remained relatively unchanged, but several small reservoirs, detention dams, and debris basins have been constructed at the canyon mouths. If some of the big, historical storms occurred today, the mountain runoff would be about the same as in the past because the small structures would have little effect on major floods on the main stem of the Santa Ana River above Prado Dam. The valley runoff would be considerably higher in both peak and volume because of the impervious areas and channelized flows.

b. Not much information is available about the storms that led to the great flood of 1862. No rainfall amounts are available in or near the Santa Ana River Basin. However, recounts from settlers tell of a flood which "wrought great destruction and desolation". The storm and flood of 21-24 January 1943 was in nearly every respect, the most severe of its kind on record. Isohyets of the maximum 24-hour precipitation is shown on plate B-5. The storm of 3-4 March 1943 is described as a local thunderstorm which resulted in short-period precipitation of near record breaking magnitude for the southern California coastal region. Plate B-6 presents an isohyetal map of the maximum 3-hour precipitation. The storms of 18-27 January and 22-25 February 1969 were a storm series which brought extremely heavy precipitation to the southern California area. Because ground conditions were more conducive to runoff than during the January 1943 storm, each of the 1969 floods produced peak discharges greater than January 1943 flood.

c. Storms and floods of February and March 1978 were caused when an unusually strong high pressure cell built up over Alaska and western Canada from mid-December through mid-March, forcing the Pacific storm track far to the south of its normal position. As a result, several series of intense storms moved directly onto the coast of southern California from out of the west, bringing large quantities of warm, moist tropical air into the region. Widespread moderate to heavy precipitation with snow levels generally 7,000 to 8,500 feet occurred throughout coastal southern California. This precipitation was greatly enhanced in many mountain areas by the orographic uplifting of the moist air. During the 5-11 February storms the total precipitation ranged from 1-2 inches around San Diego, 4-7 inches in the coastal valleys of Los Angeles, Ventura, and Santa Barbara Counties, and up to 20 inches in the eastern San Gabriel Mountains and foothills plus parts of Santa Barbara County. In the 27 February-6 March storm period the total precipitation ranged from 2-4 inches in coastal portions of San Diego and San Luis Obispo Counties to 6-10 inches in coastal valleys from Santa Barbara to Orange County, with more than 25 inches recorded in the eastern San Gabriel Mountains. Areas hard hit by damaging rainfall in this storm extended from Santa Barbara County eastward to San Bernardino and Riverside Counties. Peak discharges recorded for the February storm were: Santa Ana River near Mentone, 2,170 cfs; Santa Ana River at the MWD crossing, 25,000 cfs (estimate); Plunge Creek near East Highland, 1,050 cfs; Mill Creek near Yucaipa, 5,400 cfs; and Santiago Creek at Villa Park Dam, 2,800 cfs. Peak discharges recorded for the March storm were: Santa Ana River near Mentone, 4,000 cfs; no estimate at Santa Ana River at the MWD crossing; Plunge Creek near East Highland, 1,830 cfs; Mill Creek near Yucaipa, 4,100 cfs; and Santiago Creek at Villa Park Dam, 2,000 cfs. Pertinent data on these and other streamgages are given in table B-1 and station locations are shown on plate B-7.

IV. SYNTHESIS OF STANDARD PROJECT FLOOD

GENERAL. The standard project flood (SPF) represents the flood that would result from the most severe combination of meteorologic and hydrologic conditions considered reasonably characteristic of the geographical area. The SPF is normally larger than any past recorded flood in the area and would be exceeded in magnitude only on rare occasions. It thus constitutes a standard for design that would provide a high degree of flood protection. Preparation of standard project flood estimates was made in accordance with EM 1110-2-1411 (Standard Project Flood Determinations). The SPF determination was presented in detail in section H of the technical appendix of the Review Report.

STANDARD PROJECT STORM.

a. General. The standard project storm for the Santa Ana Basin was determined by evaluating several storms to determine the event that represents the most severe flood producing rainfall, depth-area duration relationship, and isohyetal pattern that is considered reasonably characteristic of the region. It was determined that a general storm would govern for all points under consideration on the Santa Ana River and for Santiago Creek. Under certain project alternative evaluations where outflow from Prado Dam is limited, however, a local storm centered below Prado Dam was found to govern.

b. General Winter Type. Under present conditions (no additional flood control measures), the critical storm for the Santa Ana River and Santiago Creek is based on the assumed occurrence of a storm equivalent in magnitude to that of 21-24 January 1943 transposed and centered critically over the area tributary to the basin concentration points.

c. Local Type. The 3-hour thunderstorm of March 1943 proved to be the critical storm when centered over the tributary areas of the lower Santa Ana River and Oak Street Drain.

DETERMINATION OF STANDARD PROJECT FLOOD.

a. General. Standard project floods were computed by determining the following: (a) unit-time precipitation for each subarea; (b) effective precipitation by subtraction of loss rates and by application of an imperviousness factor where applicable; (c) subarea surface-runoff hydrograph by application of subarea synthetic unit-hydrograph values to the effective unit period precipitation; (d) subarea total-runoff hydrograph by addition of base flow; and (e) total flood hydrograph by reservoir and channel routing, subtraction of percolation losses, and combining subarea hydrographs as required.

b. Mentone. Mentone Dam was operated for two alternative regulation plans under SPF conditions: (1) gated outlets and (2) ungated outlets. With gated outlets, Mentone Dam would be operated to reduce SPF inflow to an outflow of 2,000 cfs during the storm. This outflow would be held until the flood peak at Prado Dam had passed. The outflow would then be increased to a maximum 6,000 cfs with outflow decreasing with decreasing head. With this plan, the estimated time to empty the reservoir is about 24 days with no additional inflows. The maximum water surface elevation would reach elevation 1548.3 feet (spillway crest elevation is 1548.5 feet). Mentone Dam, operating with ungated outlets, would reduce SPF inflow to a maximum outflow of 5,800 cfs. The estimated time to empty the reservoir is about 21 days, with no additional inflows. The maximum water surface elevation is 1538.4 feet, about 10 feet below spillway crest for gated conditions. Plates 8 and 9 present the comparative results of both plans.

c. Santa Ana River. As discussed previously, the computation of SPF for the Santa Ana River is presented in the Review Report. Plate B-10 shows standard project flood peak discharges for various concentration points, without project, present and future conditions. The with project SPF peak discharges are shown on plate B-11. The SPF inflow and outflow hydrographs at Prado Dam for present and future conditions are shown on plates 12 and 13. These are the same hydrographs developed in the review report. Recent changes in the Lake Elsinore outlet channel have lowered the channel invert by about five feet from the value used in the review report. This change results in about 25,000 acre-feet of surcharge storage with an outflow of only about 250 cfs. The net result is to reduce the 4-day inflow to Prado Dam during the SPF. The results and recommendations of the current planning study for Lake Elsinore and updated capacity tables for Prado Dam will be utilized in the Phase II studies. Plate 14 presents the SPF hydrograph at Prado Dam with the recommended plan and gated outlets for Mentone Dam. Plate 15 presents the SPF hydrograph at Prado Dam with the recommended plan and ungated outlets at Mentone Dam. As shown on the two previous plates, the difference at Prado Dam in the two plans is only 0.3 feet in maximum water surface elevation. Pertinent basin subarea characteristics for the Santa Ana River are presented in table B-2.

d. Santiago Creek. The critical standard project flood peak discharges for concentration points on Santiago Creek were produced by the general standard project storm centered in the Santa Ana Mountains. The local standard project storm was also considered. The critical centering for the local storm was found to be over subareas below Villa Park Dam, but this storm produced a smaller standard project peak discharge at the Santa Ana River than did the general storm; this was primarily because Villa Park reservoir controlled flows from subareas above the reservoir to 3,500 cfs. Pertinent basin subarea characteristics for Santiago Creek are shown in table B-3.

e. Oak Street Drain. The standard project flood peak discharges for concentration points on Oak Street Drain were produced by the critical centering of the local standard project storm. Plate B-16 shows the standard project flood discharges for present and future conditions without project. Plate B-17 shows the standard project peak discharges with the recommended plan. Table B-4 presents pertinent subarea characteristics for Oak Street Drain.

V. PROBABLE MAXIMUM FLOOD

GENERAL. The probable maximum flood (PMF) is the flood that can be expected from the most severe combination of meteorologic and hydrologic conditions reasonably possible in the region. Probable maximum flood, as the name implies, is an estimate of the upper boundary of flood potential for a drainage area. Such a hypothetical flood is required for redesigning the spillway for Prado Dam and for designing the spillway for the proposed Mentone Dam. The determination of the probable maximum storms for the drainage areas above Mentone and Prado Dams was based on data obtained in enclosures one and two of a letter (subject: PMP for 18 Los Angeles basins) dated December 2, 1968, from the Hydrometeorological Branch of the U. S. Weather Bureau. The probable maximum storm, which was based on a general winter storm, was used as a basis for developing the probable maximum flood for Prado and Mentone Dams. A detailed analysis for determining the PMF is presented in section H of the technical appendix of the Review Report.

PEAK DISCHARGES. The probable maximum flood peak discharge at Prado Dam for present conditions is 670,000 cubic feet per second and for future conditions, 700,000 cubic feet per second. The probable maximum flood hydrograph for future conditions for Prado Dam is shown on plate B-17a. The probable maximum flood peak inflow to Mentone Dam under both present and future conditions is 265,000 cubic feet per second. The probable maximum flood hydrograph is shown on plate B-17b.

VI. DEBRIS ESTIMATE

a. The recommended plan in the Review Report for Oak Street Drain included a debris basin at the channel inlet above Ontario Avenue. However, the Riverside County Flood Control District (RCFCD) with funds from the Soil Conservation Service completed their own debris basin immediately below Chase Drive on Oak Street Drain in October 1979. An estimate was made of the debris production for the combined Hagador, Tin Mine, and Kroonen Canyons at the RCFCD debris basin. The debris estimate, based on a major storm event, was computed using the recommendations of the geologist as to debris potential and the procedure outlined in "A New Method of Estimating Debris-Storage Requirements for Debris Basins," by Tatum. The Tatum method for estimating debris storage requirements is derived from facts observed during debris flows from floods originating in the San Gabriel Mountains. Hydrologic and geologic conditions in the Oak Street Drain watershed are similiar to those found in the San Gabriel Mountains, thus allowing the use of the Tatum method for the Oak Street debris estimate. Debris estimates with this method are based on drainage area, slope, drainage density, hypsometric index, 3-hour rainfall and burn effect. Table B-5 presents the resulting data, debris production factors, and correction factors. Correction factors are based on graphs shown on plates B-18 and B-19. The product of the correction factors is then applied to the recommended maximum production rate as determined from plate B-20.

b. Geologic investigations of Hagador, Tin Mine, and Kroonen Canyons indicated a moderate-to-high debris potential. The estimated major storm debris production from the canyon area of 6.2 square miles is 305 acre-feet. The county debris basin is designed for 253 acre-feet of debris storage and a spillway discharge of 7,700 cfs. The California Department of Water Resources Dam Safety Division provided Riverside County with the spillway design discharge of 7,700 cfs. The Corps of Engineers values for debris storage (305 acre-feet) and spillway discharge (16,000 cfs) differ significantly from RCFCD design values. When the Corps of Engineers' probable maximum flood of 16,000 cfs is routed through the debris basin, the maximum water surface elevation reaches within 6 inches of the dam crest (1,034 feet). Since the spillway is able to pass SPF with 7 feet of freeboard, the operation of the project does not appear compromised. Additional wall height is being provided at critical locations in the channel downstream of the debris basin in order to alleviate problems arising from any debris deposition as a result of the basin capacity being exceeded.

VII. DISCHARGE-FREQUENCY AND VOLUME-FREQUENCY ANALYSIS

GENERAL. Development of the updated volume-frequency curves on Santiago Creek and the discharge-frequency curves on Oak Street Drain is outlined in the following paragraphs. No changes were made to the volume-frequency and discharge-frequency curves for the main stem Santa Ana River. Derivation of these curves is discussed in the Review Report. The volume-frequency curves for the Santa Ana River at Prado Dam, for present conditions and future conditions, without project are shown on plates B-21 and B-22. The peak inflow and outflow curves for Prado Dam, for present and future conditions, are shown on plates B-23 and B-24.

SANTIAGO CREEK.

a. The Santiago Creek near Villa Park streamgage was used to develop a frequency curve for inflow into Villa Park Dam. This gage has a 1921 to 1963 period of record. To reflect the effect of Santiago Dam (controls 63.2 sq. mi.) on the inflow to Villa Park Dam, however, the record from 1933 (when Santiago Dam was constructed) to 1963 (when Villa Park Dam was completed) was utilized. In addition, Villa Park Dam inflow data from 1963 to 1979 were utilized. The peak, 1-day, 2-day, and 3-day discharges for the 1933 to 1979 period were plotted using median plotting positions for n equal to 47. The 1969 inflow was the largest that has occurred during the 1921 to 1979 period, therefore, 1969 was plotted at n = 59. Best fit curves were drawn through each data set. Analytical analysis was not considered due to upstream regulation by Santiago Dam. The resulting volume-frequency curves are shown on plate B-25. Table B-6 lists annual maximum runoff values used in the analysis.

b. An analytical discharge frequency curve was drawn for the period 1938-1978 for the Handy Creek streamgage, according to the Water Resources Council Bulletin 17A "Guidelines for Determining Flood Flow

Frequency." Peak discharges for the period of record are tabulated in "Hydrologic Data Report, 1977-78 Season" prepared by Orange County Environmental Management Agency who also operate and maintain the gage. The statistics for the gage are presented in table B-6A. A regional skew of -0.2 was adopted from previous studies. The analytical curve (expected probability) for Handy Creek and recorded data plotted by median plotting positions, are shown on plate B-26.

OAK STREET DRAIN. Discharge-frequency curves for Oak Street Drain were developed according to the procedure outlined in the Review Report. Both present and future curves from the Review Report required restudy to determine the effects of the recently constructed Riverside County debris basin at Chase Avenue and minor area adjustments to some of the subareas. The effect of the debris basin reduced the present conditions, without project SPF developed in the Review Report, from 10,000 cfs to 8,500 cfs. Hence, using the standard deviation (0.744), skew (-0.18), and frequency of SPF used in the Review Report, the analytical curve was adjusted downward to reflect the effect of the debris basin. Other frequency curves at locations downstream of the existing debris basin were constructed in the same manner. All frequency curves were graphically extended beyond the SPF for use in economic evaluation. The frequency curve for Oak Street Drain at the 91 Freeway for without project, present conditions is shown on plate B-27.

VIII. WATER CONSERVATION

GENERAL. The Phase 1 report for the Santa Ana River requires a study on water conservation at Prado Dam. The recommended plan would allow for a seasonally expanded conservation pool. The method and results of the study are discussed in the following paragraphs.

7.02 WATER CONSERVATION ANALYSIS. The following is a discussion on the methodology and data collection for the water conservation study:

a. Records for average monthly inflow to Prado Dam were tabulated for the water years 1920 through 1979. Data from 1920 to 1940 was obtained from the U.S.G.S. gage "Santa Ana River below Prado Dam" (11074000). From 1941 (when Prado Dam was completed) to 1979, inflow data was obtained from Corps of Engineers reservoir computations at Prado Dam.

b. In coordination with the Orange County Water District (OCWD), downstream diversion capacities by month, for the OCWD spreading grounds were determined from present spreading operations. Monthly storage values used for the seasonally expanded conservation pool were determined such that SPF protection would be provided for areas downstream of Prado Dam throughout the year. Essentially, no carry over storage is provided for September and October. The purpose of this operation is to allow for routine maintenance and minimize problems involved with the perennial impoundment of water such as water quality, mosquitoes, and offensive odors. From November through February, the maximum allowable conservation storage ranges from 20,000 to 28,000 acre-feet, respectively. The allowable storages at these levels still provide adequate flood control storage for the standard project flood which has the greatest probability of occurring during this period. The highest maximum allowable storage is allocated for March, April, and May. This storage of 50,000 acre-feet represents the maximum conservation pool level (elevation 512) and is designed to capture spring runoff. This space is available based on the premise that the design standard project flood is not characteristic of that period. The allowable storages for June through August range from 38,000 to 13,000 acre-feet, respectively. The objective of this schedule is to insure a smooth drawdown of the reservoir by the end of September. The downstream diversion capacities and maximum allowable conservation storage are presented in table B-7.

c. Present storage capacity, surface area, release schedules and associated elevations for Prado Dam were compiled from previous Corps of Engineers studies.

d. From the Riverside County Hydrologic Annual Report for 1973-74, the 34-year average pan evaporation for each month at the Lake Mathews station (closest station to Prado Dam) were tabulated. The average precipitation for each month at Corona (closest gage to Prado Dam) was also tabulated. Net evaporation was calculated using the equation:

Net Evaporation = .70 (average monthly pan evaporation minus average monthly precipitation) where .70 is the pan coefficient and all units are in inches per month. Average net evaporation by months is presented in table B-7.

e. The HEC-5 "Reservoir Systems and Flood Control" computer program was utilized to simulate the Prado Dam conservation operation for the 60 years of record. Output data included reservoir storages and elevations, water diverted for spreading and water wasted to the ocean.

f. Additional simulation runs were made to determine the added yield of alternate plans. Allowances for future conditions were also considered by adding 200 cfs baseflow to each month of average inflow to account for the increase due to urbanization and return flow from imported water.

RESULTS. The recommended plan with a seasonally expanded conservation pool resulted in an average inflow of 97,900 acre-feet per year, an average diversion or conservation of 86,300 acre-feet per year, and waste to the ocean of 10,500 acre-feet per year. Adding a 10,000 acre-feet increment of dedicated conservation storage above the seasonally expanded conservation pool adds only an average annual yield of 1,300 acre-feet. Increasing the diversion capacity at the spreading grounds by 150 cfs produces nearly the same added amounts of conserved water as adding an additional 30,000 acre-feet of storage to the seasonally expanded conservation pool. Comparative results of each alternative, present and future condition is presented in table B-8. Plate B-28 shows a bar graph of the recommended operation of Prado Dam with the seasonally expanded conservation pool.

IX. PROJECT ALTERNATIVES

GENERAL. The nine alternatives presented in the Review Report for the main stem of the Santa Ana river can be put into four basic categories: (1) do nothing (no action); (2) correct Prado Dam; (3) modify Prado Dam and improve the channel downstream from Prado Dam (present 100-year protection below Prado, future 100-year protection below Prado, standard project protection below Prado, national economic development, and environmental quality); and (4) build Mentone Dam, modify Prado Dam, and improve the channel downstream from Prado Dam (all river protection, social well-being). Two new alternatives are presented in this report: alternative 10 which is a new environmental quality plan and alternative 11 which includes only enlargement of Prado Dam outlet works and a large downstream channel. These new alternatives along with alternatives 1, 5, 6 and 7 are addressed in detail in this report. Standard project flood peak discharges for alternatives 1, 5, 6, 7, 10, and 11 are presented in table B-9. Tables B-10 through B-14 list peak discharges for alternatives 1, 5, 6, 7, 10, and 11 for various frequency floods. The recommended plan for the Santa Ana River Basin is alternative 6, which includes the elements listed in item (4) and improved flood protection for Santiago Creek and Oak Street Drain. The operation plan at Prado Dam is structured for the purpose of maximizing benefits to the public while minimizing flood damages. Operation for water conservation is considered only when within the framework of the flood control operation. Operational decisions are to be based on factors which include time of year, condition of the upstream watershed, forecasted inflow and Mentone Dam releases, forecasted weather, Prado Reservoir storage, downstream channel conditions and water conservation. The Prado Dam release schedule for alternative 6 is listed in table 15 and will control SPF to a maximum water surface elevation of less than 563 msl. As discussed previously, the Mentone Dam operation plan with gated outlets for alternative 6 requires a constant 2,000 cfs outflow during the storm, then outflow is increased to a maximum 6,000 cfs after the flood peak at Prado Dam has passed. The release schedule for Mentone Dam with ungated outlets (alternative regulation plan at Mentone Dam) is presented in table B-16. Alternatives 5, 7, and 10 provide a maximum controlled release of 30,000 cfs from Prado Dam. Alternative 11 provides a maximum controlled release of 48,000 cfs from Prado Dam with future SPF spilling uncontrolled at 200,000 cfs.

SANTIAGO CREEK.

a. The recommended plan for Santiago Creek provides 100-year flood protection for the residents in the Santiago Creek flood plain. The plan principally consists of a maximum release of 3,500 cfs from Villa Park Dam, routing floodwaters from above Villa Park Road into the detention basin (gravel pits modified to store runoff), regulating outflow to 3,500 cfs from the basin with floodgates and minimal channel improvements to insure the containment of the 100-year flood. Subarea delineations and location of the physical features on Santiago Creek are shown on plates B-2 and B-29.

b. Impact of the recommended plan on floodflows was evaluated by first routing the 100-year balanced hydrograph through Villa Park Dam using an operation schedule with a maximum release of 3,500 cfs (table B-17), future storage and assuming the reservoir elevation at 510 feet (debris pool level) at the start of the inflow hydrograph. During the general storm, spilling occurs at Villa Park Dam when the 100-year balanced hydrograph is routed through the reservoir. The hydrograph was then routed downstream to the detention basin. Rating curves for the basin and the outlet gates (maximum outflow 3,500 cfs) were used in routing the hydrograph through the basin, downstream to the Santa Ana River. The balanced hydrograph for the 100-year exceedence interval was calculated using the volume-frequency curves from plate B-25 and the inflow hydrograph of 22-25 February 1969 as a pattern. The 100-year subarea hydrographs below Villa Park Dam for the general and local storms, used in the Santiago Creek routings, were computed by reducing the SPF hydrographs of the subareas for each storm by a ratio of 100-year peak discharge to SPF peak discharge from frequency curves at Villa Park Dam (general storm) and Handy Creek (local storm). The discharge-frequency curves for Villa Park Dam and Handy Creek are shown on plates 25 and 26, respectively. The 100-year, with project, peak discharges (general and local storms) at various concentration points for future conditions are shown on plates B-30 and B-31. The 100-year, with project hydrographs (general and local storms) for future conditions at the detention basin and the confluence with the Santa Ana River are presented on plates B-32, B-33, B-34, and B-35.

OAK STREET DRAIN. The recommended plan for Oak Street Drain principally consists of channel improvements to provide SPF protection in the reach from the Riverside County debris basin, downstream through the City of Corona to Temescal Wash.

X. ADEQUACY OF ESTIMATES

STANDARD PROJECT FLOOD PEAK DISCHARGES. The occurrence of a storm of the magnitude and intensity of the January and March 1943 storms (which were used as a basis for developing the standard project floods), with ground conditions assumed for this study, would produce a flood that would be exceeded only on rare occasions. The adequacy of the standard project flood peak discharges for Santa Ana River at Prado Dam and Oak Street Drain at the 91 Freeway is indicated by comparison of those discharges with the enveloping curves of peak discharges shown on plate B-36.

DEBRIS ESTIMATE. The adequacy of the debris estimate for Oak Street Drain is indicated by comparison with the enveloping curve of debris inflows shown on plate B-37. The debris estimate for Oak Street Drain, in comparison with other observed and estimated debris flows, clearly shows the high debris potential of this area as indicated by its position near the enveloping curve.

Table B-1. STREAMGAGING STATIONS IN THE SANTA ANA RIVER BASIN

No.*	STATION	Drainage Area** (Square miles)	Geographic coordinates: Latitude (Degrees and minutes)	Longitude (Degrees and minutes)	Period of records: Recording	Non-recording	Maximum discharge of record: Peak Amount (Cubic feet per second)	Peak Date	Mean daily Amount (Cubic feet per second)	Mean daily Date
1	Santa Ana River near Mentone	177.0	34-07	117-06	1917-PR	1896-1917	52,300	Mar. 2, 1938	15,500	Mar. 2, 1938
2	Santa Ana River at Riverside Narrow near Arlington	818.0	33-58	117-28	1927-72 1939-54		100,000	do	***	***
3	Santa Ana River at E Street near San Bernardino	500.0	34-04	117-18	1966-PR		28,000	Feb. 25, 1969	14,800	Feb. 25, 1969
4	Santa Ana River below Prado Dam	2,255.0	33-53	117-39	1940-PR		5,000	Feb. 27, 1969	5,000	Feb. 27, 1969
5	Santa Ana River near Prado Dam	2,255.0	33-52	117-40	1919-42		100,000	Mar. 3, 1938	28,600	Mar. 3, 1938
6	Mill Creek near Mentone	46.3	34-05	117-07	1939-65		1,500	Dec. 23, 1945	170	Dec. 24, 1941
7	Mill Creek near Yucaipa	42.4	34-05	117-02	1919-38 1947-PR		35,400	Jan. 25, 1969	6,300	Mar. 2, 1938
8	Plunge Creek near East Highlands	16.9	34-07	117-08	1919-PR		5,340	Mar. 2, 1938	***	***
9	City Creek near Highland	19.6	34-09	117-11	1919-PR 1954-70		7,000	Feb. 25, 1969	3,360	Feb. 25, 1969
10	Santa Ana River at Waterman Ave. at San Bernardino	322.0	34-04	117-17	1977-PR	1928-37	75,700	Mar. 2, 1938	***	***
11	San Timoteo Creek near Redlands	118.0	34-02	117-12	1973-PR 1926-68		7,460	do	1,860	Mar. 2, 1938
12	San Timoteo Creek near Loma Linda	125.0	34-04	117-17	1954-75		15,000	Feb. 25, 1969	***	***
13	East Twin Creek near Arrowhead Springs	8.8	34-11	117-16	1919-PR		3,360	Mar. 2, 1938	***	***
14	Waterman Canyon Creek near Arrowhead Springs	4.7	34-12	117-16	1919-PR	1911-14	2,350	do	478	Mar. 2, 1938
15	Lytle Creek near Fontana	46.3	34-13	117-27	1918-PR		35,900	Jan. 25, 1969	8,960	do
16	Cajon Creek near Keenbrook	40.6	34-16	117-28	1919-PR		14,500	Mar. 2, 1938	3,800	do
17	Lone Pine Creek near Keenbrook	15.1	34-16	117-28	1919-PR		6,180	do	1,480	do
18	Devil Canyon near San Bernardino	5.5	34-12	117-20	1919-PR	1911-14	3,720	Jan. 25, 1969	556	Jan. 25, 1969
19	Santa Ana River at Imperial Highway	2,306.0	33-52	117-47	1941-PR		100,000	Mar. 2, 1938	***	***

See footnotes at end of table.

Table B-1. Continued

No.*	STATION	Drainage Area** Square miles	Geographic coordinates Latitude Degrees and minutes	Longitude Degrees and minutes	Period of records Recording	Non-recording	Maximum discharge of record Peak Amount Cubic feet per second	Peak Date	Mean daily Amount Cubic feet per second	Mean daily Date
20	Day Creek near Etiwanda	4.6	34-11	117-32	1927-72		9,450	Jan. 25, 1969	4,070	Jan. 25, 1969
21	San Jacinto River near San Jacinto	141.0	33-44	116-50	1920-37 1948-PR	1937-48	45,000	Feb. 16, 1927	***	***
22	Bautista Creek near Hemet	39.4	33-42	116-51	1947-69		1,440	Apr. 3, 1958	***	***
23	San Jacinto River near Elsinore	723.0	33-40	117-18	1921-PR	1916-21	16,000	Feb. 17, 1927	***	***
24	Temescal Creek near Corona	164.0	33-50	117-31	1927-PR		14,900	Mar. 2, 1938	3,460	Mar. 2, 1938
25	San Antonio Creek near Claremont	16.5	34-13	117-40	1917-72	1901-17	21,400	do	4,430	Jan. 25, 1969
26	Cucamonga Creek near Upland	10.1	34-10	117-38	1928-75	1927-28	14,100	Jan. 25, 1969	4,050	Jan. 25, 1969
27	Santiago Creek at Modjeska	12.5	33-43	117-38	1961-PR		6,520	Feb. 25, 1969	3,590	Feb. 24, 1969
28	Santiago Creek near Villa Park	83.8	33-49	117-47	1920-63		11,000	Feb. 16, 1927	7,000	Feb. 16, 1927
29	Santiago Creek at Santa ana	98.6	33-46	117-53	1928-PR		6,600	Feb. 26, 1969	4,270	Feb. 25, 1969
30	Santa Ana River at Santa Ana	2,447.0	33-45	117-55	1923-76		46,300	Mar. 3, 1938	20,300	Mar. 3, 1938
31	Carbon Creek at Olinda	20.0	33-53	117-51	1930-38		1,760	Mar. 2, 1938	***	***
	Carbon Creek near Yorba Linda	20.4			1950-61		935	Apr. 3, 1958	180	Apr. 3, 1958
	Carbon Creek at Carbon Canyon Dam	19.5	33-55	117-50	1961-PR		1,050	Jan. 25, 1969	300	Jan. 25, 1969
32	Brea Creek at Fullerton	26.4	33-52	117-56	1932-40		1,970	Mar. 2, 1938	944	Mar. 2, 1938
	Brea Creek at Brea Reservoir	22.0			1941-72		2,000	Mar. 14, 1941	***	***
33	Fullerton Creek at Fullerton	6.2	33-52	117-54	1936-40		900	Mar. 2, 1938	***	***
	Fullerton Creek at Fullerton Dam	5.1			1941-PR		3,800	Mar. 14, 1941	***	***
34	Little San Gorgonio Creek near Beaumont	1.74	34-02	116-57	1948-PR		11,000	Feb. 25, 1969	1,180	Feb. 25, 1969
35	Lytle Creek at Colton	172.0	34-05	117-18	1957-PR		16,800	Jan. 25, 1969	5,040	Jan. 25, 1969
36	Reche Canyon at Barton Road Near Colton	11.2	34-03	117-17	1952-73		1,175	Feb. 25, 1969	***	***
37	Santa Ana River at MWD Crossing near Arlington, CA	854.0	33-58	117-27	1970-PR		19,500	Mar. 4, 1978	6,800	Mar. 4, 1978
38	Handy Creek (Alameda Storm Channel) Orange, CA	3.2	33-48	117-48	1938-PR		1,220	Feb. 22, 1944	4	Mar. 4, 1978

* See pl. B-7 for location.
**Areas given for points on the Santa Ana River exclude [illegible] sq. miles tributary to Baldwin Lake.
*** Data not available.

Note--Data area from records published in the U.S. Geological Survey Water Supply Papers.

Table B-2. SUBAREA DRAINAGE CHARACTERISTICS, SANTA ANA RIVER BASIN

Subarea designation*	Drainage area	L	L_{CA}	S	Present condition n values	Future condition n values	Lag**	Present condition imperviousness***	Future condition imperviousness+	S graphs
	square miles	Miles	Miles	Feet per mile			Hours	Percent	Percent	
A1	38	8.4	4.1	569.0	0.06	0.06	1.66	0	2	Mountain
A2	91	18.9	9.2	450.0	0.06	0.06	3.20	0	2	Mountain
A3	31	9.8	5.3	628.0	0.06	0.06	1.90	0	2	Mountain
A4	17	7.4	3.9	594.0	0.06	0.06	1.53	0	2	Mountain
B	43	13.0	6.0	565.0	0.06	0.06	2.26	0	2	Mountain
C1	9	8.2	3.8	493.0	0.05	0.05	1.36	5	15	Mountain
C2	13	3.9	2.0	474.0	0.05	0.04	0.65	5	15	Mountain
D	11	5.5	2.8	418.0	0.05	0.04	0.86	5	15	Valley
D1	20	7.6	4.8	609.0	0.05	0.05	1.40	0	2	Mountain
D2	17	7.5	4.6	607.0	0.05	0.05	1.36	0	2	Mountain
E1	36	13.3	7.0	140.0	0.025	0.02	1.05	25	40	Valley
E2	39	12.0	6.0	535.0	0.035	0.03	1.11	20	30	Valley
E3	59	18.6	6.9	374.0	0.04	0.035	1.72	5	10	Valley
E4	30	11.6	7.1	78.0	0.04	0.035	1.96	5	10	Valley
F1	17	7.6	5.1	643.0	0.05	0.05	1.41	0	2	Mountain
F2	29	11.0	6.9	264.0	0.025	0.02	0.86	40	50	Valley
G1	73	19.6	9.7	255.0	0.05	0.04	2.50	0	2	Mountain
G2	52	16.0	7.6	468.0	0.05	0.05	2.31	0	2	Mountain
H1	19	7.8	3.9	516.0	0.045	0.045	1.21	0	2	Mountain
H2	48	13.5	7.0	184.0	0.025	0.02	1.00	40	50	Valley
I	62	21.0	11.0	63.0	0.02	0.02	1.73	40	50	Valley
J	31	16.7	7.6	127.0	0.035	0.03	1.81	15	30	Valley
L	39	17.8	10.4	57.0	0.03	0.025	2.02	15	30	Valley
M	136	25.0	12.1	331.0	0.03	0.02	1.40	30	40	Valley

* See pl. B-1 for location.
** Future conditions.
*** Valley areas only, Mountain area--use 10 percent.
\+ 100-year development, based on percent of valley area.

TABLE B-2. SUBAREA DRAINAGE CHARACTERISTICS, SANTA ANA RIVER BASIN (Continued)

Subarea* desig-nation	Drainage area	L	L_{CA}	S	Present condition n values	Future condition n values	Lag**	Present condition imper-viousness***	Future condition imper-viousness+	S graphs
	Square miles	Miles	Miles	Feet per mile			Hours	Percent	Percent	
N	38	14.6	9.12	47.0	0.03	0.02	1.48	15	25	Valley
O	79	21.6	10.4	382.0	0.03	0.02	1.21	25	40	Valley
P	27	10.3	5.9	769.0	0.05	0.05	1.62	0	2	Mountain
Q	107	20.7	11.8	142.0	0.03	0.015	1.13	30	50	Valley
R1	44	7.2	2.5	41.0	0.05	0.04	1.42	5	5	Valley
R2	146	27.3	11.8	25.0	0.035	0.025	2.92	5	30	Valley
R3	187	23.6	11.3	58.0	0.040	0.03	2.78	10	30	Valley
R4	138	27.0	11.9	117.0	0.040	0.035	3.05	5	10	Valley
R5	245	36.2	13.6	149.0	0.05	0.04	3.91	0	2	Mountain
S	193	24.4	12.2	72.0	0.045	0.04	3.71	5	10	Mountain
S1	45	16.7	8.0	61.0	0.037	0.03	2.12	5	20	Valley
S2	36	7.9	2.7	50.0	0.035	0.030	1.10	5	10	Valley
T	54	16.4	6.5	160.0	0.040	0.035	1.89	10	13	Valley
U	33	13.1	6.8	76.0	0.025	0.022	1.28	33	46	Valley
V	63	15.8	6.5	305.0	0.040	0.040	1.88	0	2	Santa Margarita
W	20	10.5	5.6	210.0	0.045	0.040	1.63	0	2	Santa Margarita
X	18	11.8	6.3	97.0	0.030	0.020	1.03	30	40	Valley
Y	5	10.1	5.0	18.0	0.022	0.020	1.23	50	60	Valley

* See pl. B-1 for location.
** Future conditions.
*** Valley areas only. Mountains area--use 10 percent.
\+ 100-year development, based on percent of valley area.

Table B-3. SUBAREA DRAINAGE CHARACTERISTICS, SANTIAGO CREEK

Subarea*	Drainage area, sq mi	L mi	L_{ca} mi	Slope ft/mi	Basin n Present	Basin n Future	Percent impervious Present	Percent impervious Future	S-Graph
A	63.40	15.80	6.50	305	.040	.040	4	4	Santa Margarita
B	20.40	10.50	5.60	210	.040	.040	4	4	Santa Margarita
C	2.76	3.23	1.44	167	.030	.020	20	25	Valley
D	4.70	6.30	3.40	147	.030	.020	10	25	Valley
E	1.64	3.04	1.90	164	.030	.020	35	40	Valley
F	2.81	2.08	1.44	95	.020	.020	40	45	Valley
G	3.62	2.69	1.89	30	.020	.020	50	55	Valley
H	1.68	2.27	1.21	31	.020	.020	50	55	Valley

* See plate B-2 for location.

Table B-4. SUBAREA DRAINAGE CHARACTERISTICS, OAK STREET DRAIN.

Subarea*	Drainage area (sq mi)	L (mi)	L_{ca} (mi)	Slope (ft/mi)	Basin Present	Basin Future	Percent impervious Present	Percent impervious Future	S-Graph
A	1.50	3.10	1.71	516	0.050	0.050	5	5	Mountain
B	6.13	3.71	1.63	590	0.050	0.050	5	5	Mountain
C	1.24	1.86	0.95	806	0.041	0.041	5	5	Mountain
D	1.06	1.74	0.90	184	0.035	0.025	10	40	Valley
E1	0.70	2.00	1.29	185	0.050	0.045	5	15	Mountain
E2	0.54	2.01	1.22	204	0.050	0.045	5	40	Mountain
E3	0.39	0.95	0.46	126	0.025	0.020	25	45	Valley
F	0.17	0.91	0.53	115	0.025	0.020	25	45	Valley

* See plate B-3 for location.

TABLE B-5

DEBRIS PRODUCTION
FOR OAK STREET DRAIN
DEBRIS PRODUCTION FACTORS

Drainage Area (sq.mi.)	Slope (ft/mi)	Drainage Density	Hypsometric Index	3-Hour Rainfall (inches)
6.2	590	1.81	0.40	3.24

CORRECTION FACTORS

Slope	Drainage Density	Hypsometric Index	3-Hour Rainfall	Total %
64%	96%	89%	82%	45%

RESULTING DATA

Recommended Production For Drainage Area (cubic yards)	Resulting Volume For Drainage Area (acre-feet)
1,100,000	305

Table B-6.
Annual Maximum Runoff Values
Santiago Creek, Orange County, California

Water Year	Analyzed Data: Peak (cfs)	Analyzed Data: 1-Day (cfs)	Analyzed Data: 2-Day (cfs)	Analyzed Data: 3-Day (cfs)	Plotting Position	Ordered Data: (Peak) (cfs)	Ordered Data: 1-Day (cfs)	Ordered Data: 2-Day (cfs)	Ordered Data: 3-Day (cfs)
1933	144	40	28	19	1.17**	10,740**	8,000**	7,450**	5,633**
1934	271	103	54	38	3.60	5,200	3,190	2,925	2,347
1935	194	30	28	23	5.70	3,330	1,796	1,621	1,454
1936	142	20	13	10	7.80	3,300	1,220	1,135	990
1937	1,360	595	460	396	9.90	3,000	1,000	989	924
1938	5,200	3,190	2,925	2,347	12.00	2,420	595	460	396
1939	76	8	5	4	14.10	1,800	593	363	310
1940	50	3	2	2	16.20	1,360	593	336	258
1941	1,800	1,220	1,135	990	18.30	970	219	173	163
1942	2	1	1	1	20.50	955	216	150	123
1943	2,420	1,000	989	924	22.60	900	179	135	122
1944	3,000	593	363	258	24.70	868	175	108	101
1945	330	149	135	123	26.80	825	174	108	81
1946	150	47	37	25	28.90	804	149	103	72
1947	40	10	7	5	31.00	625	116	60	51
1948	1	1	1	1	33.10	500	103	58	41
1949	407	36	25	16	35.20	494	98	58	40
1950	804	116	60	40	37.30	440	95	54	38
1951	139	9	4	3	39.40	407	73	50	34
1952	3,300	593	336	310	41.60	388	67	40	27
1953	955	30	21	14	43.70	330	47	37	25
1954	494	95	50	34	45.80	271	40	28	23
1955	104	11	6	4	47.90	206	38	28	19
1956	868	219	150	101	50.00	194	36	26	19
1957	144	19	10	7	51.50	170	30	25	16
1958	825	175	103	122	53.60	150	30	22	14
1959	128	12	6	4	55.70	144	29	21	14
1960	26	1	1	1	57.80	144	27	19	14
1961	388	15	8	5	59.90	142	23	14	13
1962	98	8	6	4	62.00	139	20	13	10
1963	5	1	1	1	64.10	128	19	10	8

Table B-6 (Continued)
Annual Maximum Runoff Values
Santiago Creek, Orange County, California

	Analyzed Data				Ordered Data				
Water Year	Peak (cfs)	1-Day (cfs)	2-Day (cfs)	3-Day (cfs)	Plotting Position	Peak (cfs)	1-Day (cfs)	2-Day (cfs)	3-Day (cfs)
1964	50	9	6	4	66.20	108	15	8	7
1965	55	9	6	4	68.30	106	14	8	6
1966	440	73*	40*	27*	70.50	104	12	8	5
1967	970	98	58	51	72.60	98	11	7	5
1968	900	216	108	72	74.70	76	10	6	4
1969	10,740	8,000	7,450	5,633	76.80	55	9	6	4
1970	28	27	22	14	78.90	50	9	6	4
1971	170	29	26	19	81.00	50	8	5	4
1972	108	38	19	13	83.10	40	8	4	3
1973	625	179	108	81	85.20	28	3	2	2
1974	206	67	58	41	87.30	26	1	1	1
1975	106	23	14	14	89.40	24	1	1	1
1976	14	9	8	8	91.60	14	1	1	1
1977	24	14	8	6	93.70	5	1	1	1
1978	3,330	1,796	1,621	1,454	95.80	2	0	0	0
1979	500	174	173	163	97.90	1	0	0	0

*Estimated
**1969 flow is largest flood occurring within period from 1921-1979, therefore 1969 is plotted at n=59

TABLE 6A

ANALYTICAL FREQUENCY ANALYSIS OF PEAK FLOWS
HANDY CREEK (ALAMEDA STORM CHANNEL) ORANGE, CALIFORNIA
OCEMA* NO. 152
DRAINAGE AREA = 3.2 SQ. MI.

Analyzed Data		Ordered Data				Final Peak Discharge vs. Frequency Estimates				
Water Year	Peak Discharge (cfs)	Rank	Water Year	Peak Discharge (cfs)	Median Plotting Position	Exceedance Probability	Computed Discharge (cfs)	Expected Probability Discharge (cfs)	0.05 Confidence Limit (cfs)	0.95 Confidence Limit (cfs)
1938	901.	1	1944	1220.	.0182	.002	3930.	4840.	8790.	2220.
1939	1000.	2	1952	1190.	.0443	.005	2980.	3500.	6270.	1750.
1940	322.	3	1978	1050.	.0703	.010	2350.	2680.	4710.	1420.
1941	562.	4	1939	1000.	.0964	.020	1800.	1990.	3430.	1130.
1942	91.	5	1938	901.	.1224	.040	1330.	1430.	2390.	864.
1943	537.	6	1969	870.	.1484	.100	820.	855.	1350.	560.
1944	1220.	7	1941	562.	.1745	.200	510.	522.	776.	362.
1945	267.	8	1943	537.	.2005	.500	195.	195.	270.	141.
1946	251.	9	1971	470.	.2266	.800	69.	67.	97.	46.
1947	256.	10	1956	346.	.2526	.900	39.	37.	58.	24.
1948	30.	11	1940	322.	.2786	.950	24.	22.	37.	13.
1949	27.	12	1958	303.	.3047	.990	9.	8.	16.	4.
1950	78.	13	1967	290.	.3307					
1951	49.	14	1966	285.	.3568					
1952	1190.	15	1945	267.	.3828					
1953	236.	16	1947	256.	.4089					
1954	80.	17	1946	251.	.4349					
1955	39.	18	1953	236.	.4609					
1956	346.	19	1976	202.	.4870					
1957	84.	20	1973	192.	.5130					
1958	303.	21	1959	180.	.5391					
1959	180.	22	1970	178.	.5651					
1960	47.	23	1974	178.	.5911					
1961	6.	24	1975	173.	.6172					
1962	65.	25	1977	164.	.6432					
1966	285.	26	1972	132.	.6693					
1967	290.	27	1942	91.	.6953					
1968	67.	28	1957	85.	.7214					
1969	870.	29	1954	80.	.7474					
1970	178.	30	1950	78.	.7734					
1971	470.	31	1968	67.	.7995					
1972	132.	32	1962	65.	.8255					
1973	192.	33	1951	49.	.8516					
1974	178.	34	1960	47.	.8776					
1975	173.	35	1955	39.	.9036					
1976	202.	36	1948	30.	.9297					
1977	164.	37	1949	27.	.9557					
1978	1050.	38	1961	6.	.9818					

Final Statistics based on 38 years

Mean Logarithm	2.2674
Standard Deviation	.5169
Computed Skew	-.5319
Generalized Skew	-.2000
Adopted Skew	-.2575

No data exists for 1963-65 peak discharges. Since no data is available which points to an abnormal water year, the peak discharge was deleted and the record was treated as continuous.

[illegible] (see plate B-7 for gage location).

Table B-7. PRADO DAM--WATER CONSERVATION PARAMETERS FOR SEASONALLY EXPANDED CONSERVATION POOL

	MAXIMUM ALLOWABLE STORAGE		DIVERSION CAPACITY (CFS)	EVAPORATION (INCHES)
	STORAGE (AC-FT)	ELEVATION (FT)		
OCTOBER	100	475	300	3.65
NOVEMBER	20,000	500	300	1.99
DECEMBER	20,000	500	300	0.53
JANUARY	20,000	500	300	0.22
FEBRUARY	28,000	504	200	0.36
MARCH	50,000	512	200	1.76
APRIL	50,000	512	240	3.16
MAY	50,000	512	260	4.88
JUNE	38,000	508	280	5.84
JULY	26,000	503	300	7.36
AUGUST	13,000	496	300	6.99
SEPTEMBER	100	475	300	5.44

Table B-8. COMPARATIVE RESULTS OF ALTERNATE CONSERVATION PLANS AT PRADO DAM

Alternative	Present Conditions Inflow (AF/YR)	Present Conditions Conserved (AF/YR)	Present Conditions Waste (AF/YR)	Future Conditions Inflow (AF/YR)	Future Conditions Conserved (AF/YR)	Future Conditions Waste (AF/YR)
Present Reservoir Operation Schedule 1978-79	97,900	82,800	14,000	--	--	--
Recommended Plan--Seasonally Expanded Debris Pool	97,900	86,300	10,500	241,000	191,000	48,300
Recommended Plan--Seasonally Expanded Debris Pool with--						
1. Additional 10,000 AF of storage	97,900	87,600	9,200	241,000	193,000	46,000
2. Additional 20,000 AF of storage	97,900	88,600	8,200	241,000	194,000	45,000
3. Additional 30,000 AF of storage	97,900	89,500	7,300	241,000	195,000	44,000
Recommended Plan--Seasonally Expanded Debris Pool with--						
1. Increase diversion capacity by 50 cfs	97,900	87,800	9,000	241,000	211,000	28,300
2. Increase diversion capacity by 100 cfs	97,900	88,600	8,200	241,000	221,000	18,000
3. Increase diversion capacity by 150 cfs	97,900	89,500	7,300	241,000	227,000	12,900

Table B-9. STANDARD PROJECT FLOOD PEAK DISCHARGE

Alternative	Condition	CP1 SAR* at Santa Ana, cfs	CP2 SAR at Imperial Highway, cfs	CP3 SAR Downstream Prado Dam, cfs	CP4 SAR at Prado Dam, cfs	CP5 SAR at Riverside Narrows, cfs	CP6 SAR Downstream Warm Creek, cfs	CP7 SAR at E St., cfs	CP8 SAR at Mentone Damsite, cfs
1-No action	Present	122,000	147,000	150,000	282,000	228,000	228,000	164,000	126,000
	Future	214,000	235,000	239,000	317,000	240,000	234,000	167,000	126,000
5-SPF protection downstream of Prado	Present	43,000	38,000	30,000	282,000	228,000	228,000	164,000	126,000
	Future	45,000	38,000	30,000	317,000	240,000	234,000	167,000	126,000
6-All-river protection	Present	38,000	32,000	30,000	211,000	137,000	132,000	68,000	126,000
	Future	46,000	38,000	30,000	265,000	147,000	137,000	72,000	126,000
7-National economic development	Present	38,000	32,000	30,000	282,000	228,000	228,000	164,000	126,000
	Future	43,000	38,000	30,000	317,000	240,000	234,000	167,000	126,000
10-Environmental quality	Present	38,000	32,000	30,000	282,000	228,000	228,000	164,000	126,000
	Future	43,000	38,000	30,000	317,000	240,000	234,000	167.000	126,000
11-All-channel protection	Present	135,000	132,000	134,000	282,000	228,000	228,000	164,000	126,000
	Future	200,000	195,000	198,000	317,000	240,000	234,000	167,000	126,000

*Santa Ana River

Table B-10. DISCHARGE-FREQUENCY VALUES FOR PRESENT AND FUTURE CONDITIONS

Alternative 1--No action

Frequency of peak discharge		CP1 SAR* at Santa Ana, cfs	CP2 SAR at Imperial Highway, cfs	CP3 SAR Downstream Prado Dam, cfs	CP4 SAR at Prado Dam, cfs	CP5 SAR at Riverside Narrows, cfs	CP6 SAR Downstream Warm Creek, cfs	CP7 SAR at E St., cfs	CP8 SAR at Mentone Damsite, cfs
500-year	Present	290,000	320,000	325,000	490,000	340,000	340,000	230,000	190,000
	Future	350,000	405,000	415,000	540,000	370,000	360,000	235,000	190,000
200-year	Present	130,000	150,000	160,000	360,000	265,000	260,000	165,000	135,000
	Future	220,000	240,000	250,000	380,000	280,000	265,000	170,000	135,000
150-year	Present	90,000	100,000	105,000	300,000	230,000	225,000	135,000	110,000
	Future	160,000	180,000	190,000	325,000	240,000	230,000	140,000	110,000
100-year	Present	45,000	48,000	50,000	230,000	175,000	175,000	105,000	84,000
	Future	90,000	110,000	110,000	270,000	190,000	180,000	111,000	84,000
75-year	Present	32,000	13,000	5,000	185,000	145,000	145,000	84,000	68,000
	Future	60,000	70,000	70,000	220,000	160,000	150,000	87,000	68,000
50-year	Present	21,000	7,500	5,000	132,000	102,000	102,000	60,000	49,000
	Future	30,000	22,000	20,000	155,000	115,000	110,000	64,000	49,000
25-year	Present	13,000	4,500	3,000	72,000	57,000	57,000	33,000	29,000
	Future	16,000	6,600	4,300	85,000	62,000	62,000	36,000	29,000

*Santa Ana River

Table B-11. DISCHARGE-FREQUENCY VALUES FOR PRESENT AND FUTURE CONDITIONS

Alternative 5--Standard project flood protection below Prado

Frequency of peak discharge		CP1 SAR* at Santa Ana, cfs	CP2 SAR at Imperial Highway, cfs	CP3 SAR Downstream Prado Dam, cfs	CP4 SAR at Prado Dam, cfs	CP5 SAR at Riverside Narrows, cfs	CP6 SAR Downstream Warm Creek, cfs	CP7 SAR at E St., cfs	CP8 SAR at Mentone Damsite, cfs
500-year	Present	70,000	70,000	70,000					
	Future	136,000	148,000	150,000					
200-year	Present	43,000	36,000	35,000					
	Future	45,000	38,000	36,000					
150-year	Present	37,000	31,000	30,000					
	Future	38,000	33,000	30,000					
100-year	Present	31,000	26,000	24,000	(Same as alternative 1, no action, see table 10.)				
	Future	33,000	29,000	28,000					
75-year	Present	27,000	22,000	22,000					
	Future	29,000	26,000	25,000					
50-year	Present	21,000	17,000	16,000					
	Future	25,000	21,000	20,000					
25-year	Present	14,000	11,000	8,000					
	Future	18,000	13,000	10,000					

*Santa Ana River.

Table B-[illegible]. DISCHARGE-FREQUENCY VALUES FOR PRESENT AND FUTURE CONDITIONS

Alternative 6--All-river protection

Frequency of peak discharge		CP1 SAR* at Santa Ana, cfs	CP2 SAR at Imperial Highway, cfs	CP3 SAR Downstream Prado Dam, cfs	CP4 SAR at Prado Dam, cfs	CP5 SAR at Riverside Narrows, cfs	CP6 SAR Downstream Warm Creek, cfs	CP7 SAR at E St., cfs	CP8 SAR at Mentone Damsite, cfs
500-year	Present	57,000	57,000	57,000	360,000	210,000	210,000	96,000	190,000
	Future	130,000	140,000	142,000	450,000	225,000	220,000	104,000	190,000
200-year	Present	38,000	32,000	30,000	250,000	155,000	155,000	70,000	135,000
	Future	46,000	38,000	36,000	320,000	160,000	160,000	74,000	135,000
150-year	Present	34,000	31,000	30,000	210,000	135,000	130,000	57,000	110,000
	Future	40,000	34,000	30,000	270,000	140,000	135,000	60,000	110,000
100-year	Present	29,000	26,000	24,000	170,000	110,000	105,000	42,000	84,000
	Future	35,000	32,000	30,000	210,000	115,000	110,000	45,000	84,000
75-year	Present	25,000	23,000	22,000	135,000	88,000	86,000	34,000	68,000
	Future	30,000	28,000	27,000	175,000	92,000	90,000	37,000	68,000
50-year	Present	20,000	18,000	17,000	96,000	62,000	60,000	25,000	49,000
	Future	26,000	23,000	22,000	125,000	66,000	64,000	27,000	49,000
25-year	Present	14,000	12,000	9,000	54,000	34,000	34,000	14,000	29,000
	Future	17,000	15,000	12,000	68,000	37,000	36,000	15,000	29,000

*Santa Ana River.

Table B-13. DISCHARGE-FREQUENCY VALUES FOR PRESENT AND FUTURE CONDITIONS

Alternative 7--National Economic Development
Alternative 10--Environmental Quality

Frequency of peak discharge		CP1 SAR* at Santa Ana, cfs	CP2 SAR at Imperial Highway, cfs	CP3 SAR Downstream Prado Dam, cfs	CP4 SAR at Prado Dam cfs	CP5 SAR at Riverside Narrows, cfs	CP6 SAR Downstream Warm Creek, cfs	CP7 SAR at E St., cfs	CP8 SAR at Mentone Damsite, cfs
500-year	Present	50,000	50,000	52,000					
	Future	122,000	132,000	135,000					
200-year	Present	38,000	32,000	30,000					
	Future	43,000	38,000	36,000					
150-year	Present	35,000	30,000	29,000					
	Future	39,000	32,000	30,000					
100-year	Present	30,000	26,000	25,000	(Same as alternative 1, no action, see table B-10.)				
	Future	34,000	28,000	27,000					
75-year	Present	26,000	22,000	21,000					
	Future	30,000	25,000	24,000					
50-year	Present	21,000	16,000	15,000					
	Future	26,000	22,000	21,000					
25-year	Present	14,000	11,000	10,000					
	Future	18,000	14,000	13,000					

*Santa Ana River.

Table B-14. DISCHARGE-FREQUENCY VALUES FOR PRESENT AND FUTURE CONDITIONS

Alternative 11--All-channel

Frequency of peak discharge		CP1 SAR* at Santa Ana cfs	CP2 SAR at Imperial Highway, cfs	CP3 SAR Downstream Prado Dam, cfs	CP4 SAR at Prado Dam cfs	CP5 SAR at Riverside Narrows, cfs	CP6 SAR Downstream Warm Creek, cfs	CP7 SAR at E St., cfs	CP8 SAR at Mentone Damsite cfs
500-year	Present	266,000	298,000	300,000					
	Future	290,000	328,00	330,000					
200-year	Present	145,000	143,000	145,000					
	Future	208,000	206,000	208,000					
150-year	Present	90,000	80,000	80,000					
	Future	115,000	110,000	110,000					
100-year	Present	58,000	50,000	48,000	(Same as alternative 1, no action, see table B-10.)				
	Future	60,000	52,000	48,000					
75-year	Present	48,000	43,000	41,000					
	Future	50,000	45,000	43,000					
50-year	Present	37,000	35,000	34,000					
	Future	40,000	37,000	36,000					
25-year	Present	22,000	21,000	20,000					
	Future	25,000	24,000	23,000					

*Santa Ana River.

Table 15. PRADO DAM OPERATION SCHEDULE FOR ALTERNATIVE 6--ALL-RIVER PLAN

Elevation, feet	Net storage, acre-feet	Outflow, cfs Outlets	Spillway	Total
500	15,700	200	0	200
501	20,000	300	0	300
510	36,800	2,000	0	2,000
520	63,620	12,000	0	12,000
530	99,240	20,000	0	20,000
540	143,470	30,000	0	30,000
550	199,320	30,000	0	30,000
560	270,360	30,000	0	30,000
563	309,500	30,000	0	30,000
565	330,000	19,000	11,000	30,000
568	366,000	0	53,000	53,000
570	390,000	0	92,000	92,000
576	464,000	0	277,000	277,000
580	516,000	0	395,000	395,000

Note.--Net storage based on 100-year sediment accumulation but reduced by the contribution that would come from the area above Mentone Dam.

Table B-16

MENTONE DAM RELEASE SCHEDULE UNGATED

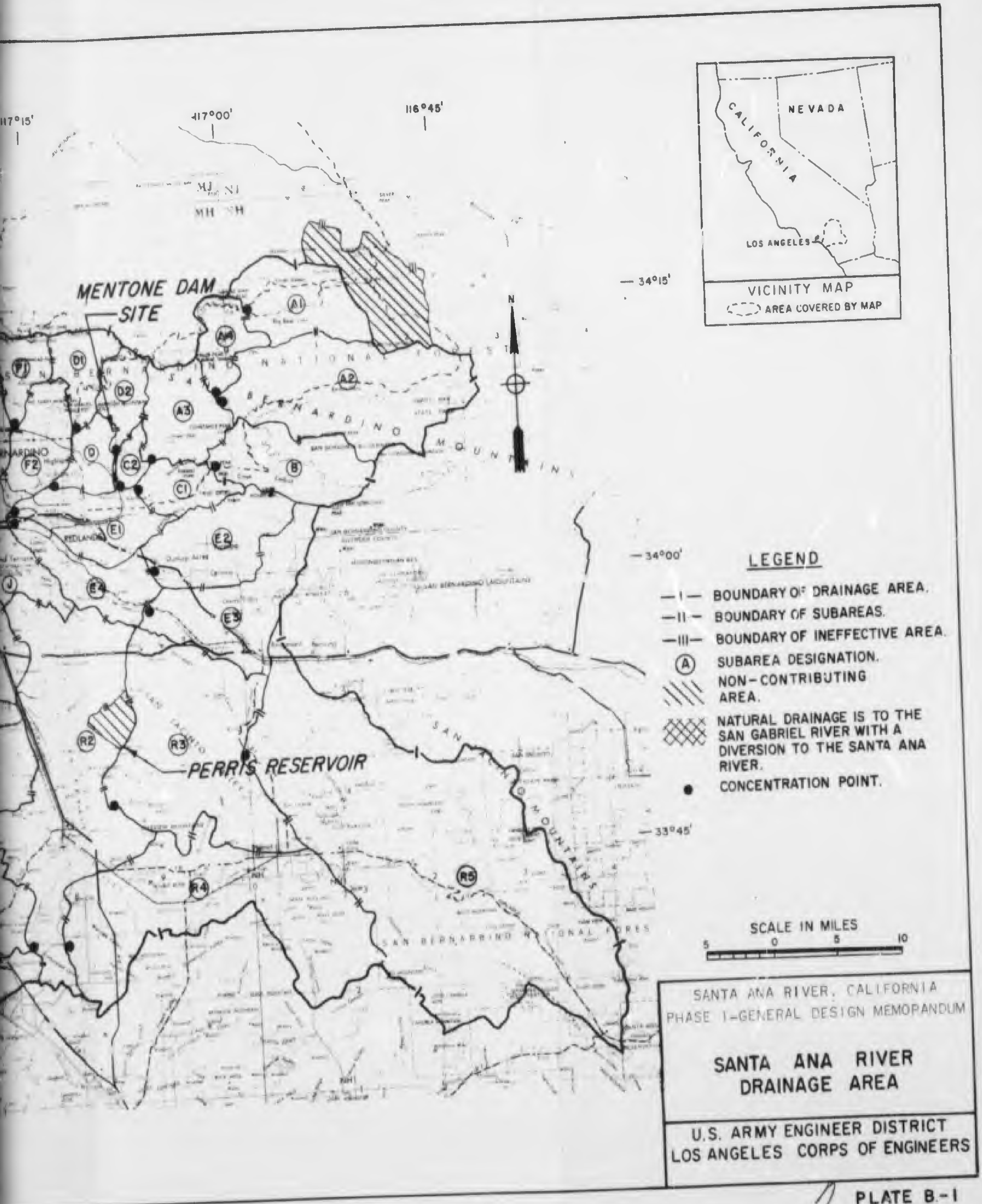
ELEVATION (FT)	NET STORAGE (AC-FT)	OUTFLOW (CFS) OUTLET
1335	0	0
1340	0.1	500
1345	720	750
1350	1,434	1,150
1360	2,869	1,700
1372	5,000	2,200
1386	10,000	2,700
1400	16,338	3,100
1454	46,000	4,100
1500	88,470	5,200
1524	116,000	5,600
1548.5	144,500	6,000
1560	165,700	6,100

Table B-17

VILLA PARK DAM
RELEASE SCHEDULE

ELEVATION (FT)	STORAGE (AC-FT)	OUTFLOW (CFS) OUTLETS	SPILLWAY	TOTAL
510	440	0	0	0
511	510	3500	0	3500
515	785	3500	0	3500
520	1310	3500	0	3500
530	2588	3500	0	3500
540	4957	3500	0	3500
550	7915	3500	0	3500
560	11490	3500	0	3500
566	14044	3500	0	3500
567	14538	3500	0	3500
570	16020	0	5400	5400
575	18655	0	20800	20800
580.5	20655	0	45800	45800

117°45'
117°30'
117°15'
117°00'
NATIONAL
MENTONE DAM
SITE
GLENDORA
MONROVIA
AZUSA
COVINA
BALDWIN PARK
WEST COVINA
LA PUENTE
POMONA
UPLAND
ONTARIO
RIALTO
SAN BERNARDINO
REDLANDS
RIVERSIDE
PRADO
DAM
WHITTIER
LA HABRA
FULLERTON
BUENA PARK
ANAHEIM
CORONA
SANTIAGO
DAM
GARDEN GROVE
WESTMINSTER
VILLA PARK
DAM
FOUNTAIN VALLEY
COSTA MESA
SANTIAGO CREEK
NEWPORT BEACH
PERRIS RESERV
NATIONAL
FOREST



SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER DRAINAGE AREA

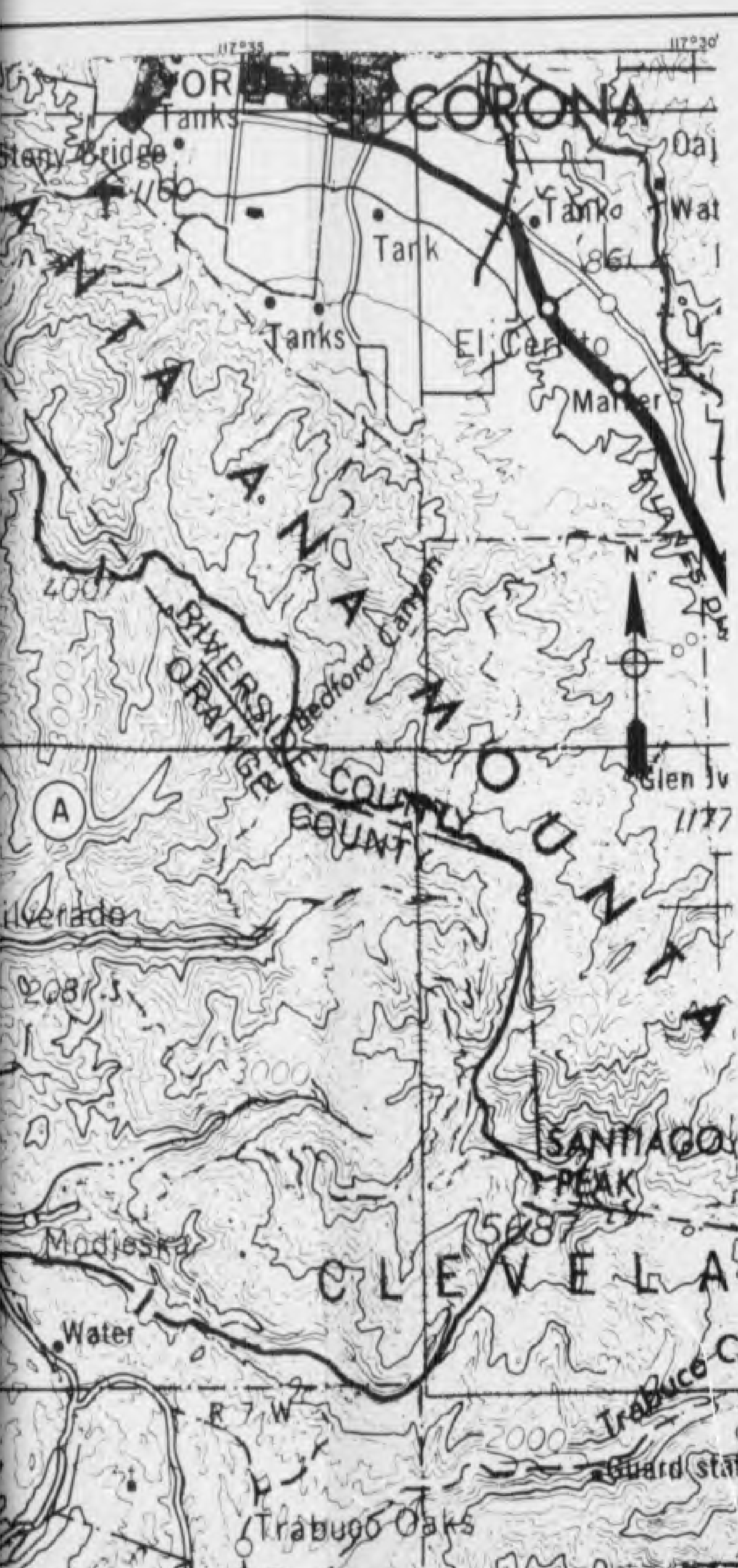
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS

PLATE B.-1

2

ANAHEIM
Peralta Hills
Villa Heights
Villa Park Dam
Fremont Canyon
West Anaheim Junction
Lemon Heights
Limestone
SANTA ANA
LOMAS DE SANTIAGO
Frances
MCAF Santa Ana
Irvine
East Irvine
Orange Co.
R 8 W


117°40'
117°35
117°30'
CORONA
Tanks
Stony Bridge
SIERRA PEAK
SANTA ANA MOUNTAINS
Tank
Tanks
El Cerrito
Guard station
Fremont Canyon
B
Ranch
4007
RIVERSIDE COUNTY
ORANGE COUNTY
Bedford Canyon
N
A
Limestone Canyon
Silverado
Guard station
Santiago Creek
3000
SANTIAGO PEAK
5687
Lookout
Water
Modjeska
CLEVELA
Water
R 8 W
R 7 W
Trabuco C
2000
Guard stat
CAS El Toro
Windmill
Trabuco Oaks



VICINITY MAP

AREA COVERED BY MAP

SCALE IN MILES

LEGEND

—I— BOUNDARY OF DRAINAGE AREA.

—II— BOUNDARY OF SUBAREAS.

(A) SUBAREA DESIGNATION.

● CONCENTRATION POINT.

SCALE IN MILES

SANTA ANA RIVER, CALIFORNIA

PHASE I-GENERAL DESIGN MEMORANDUM

SANTIAGO CREEK

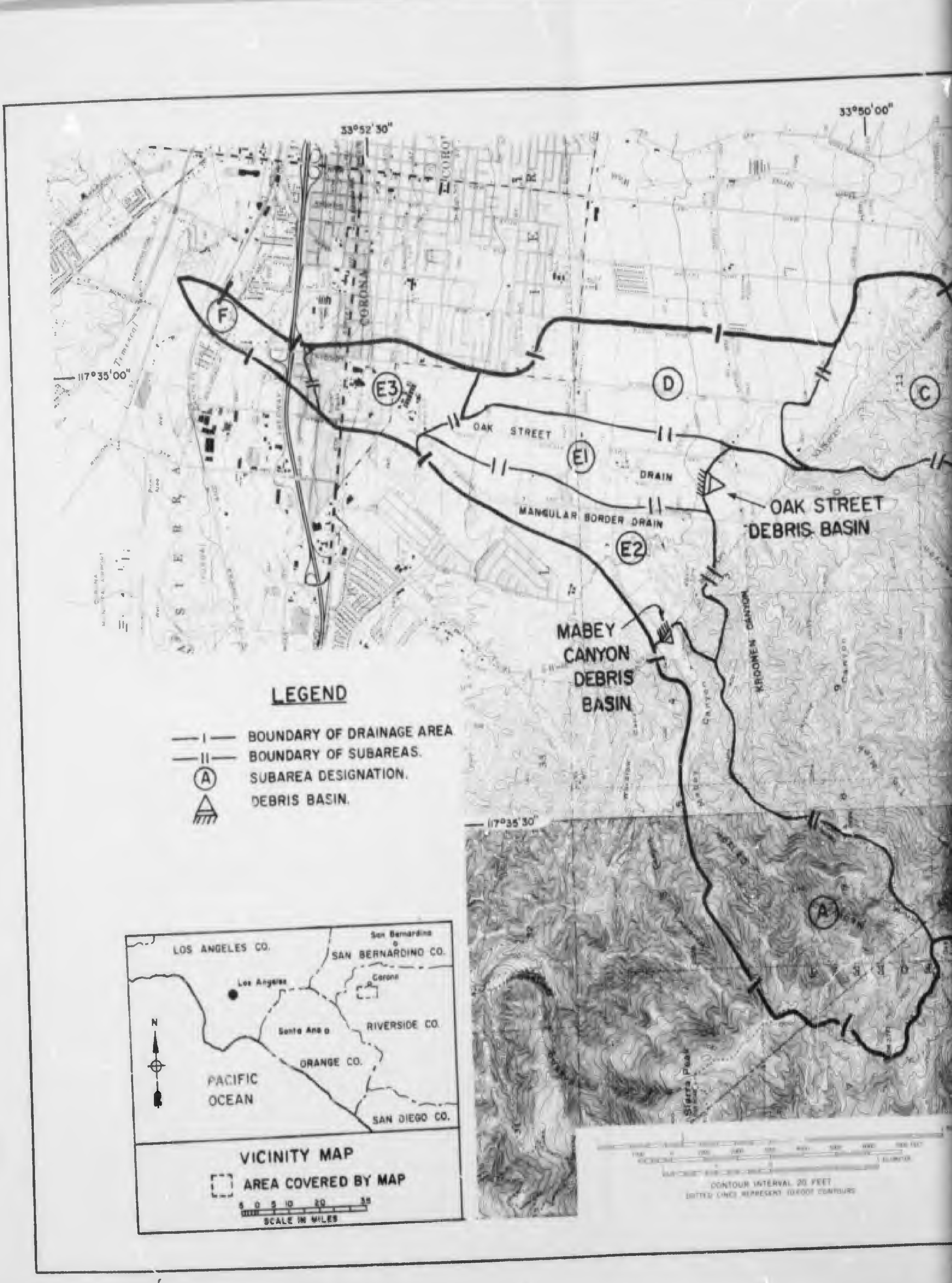
DRAINAGE AREA

U.S. ARMY ENGINEER DISTRICT

LOS ANGELES, CORPS OF ENGINEERS

TO ACCOMPANY REPORT DATED

PLATE B-2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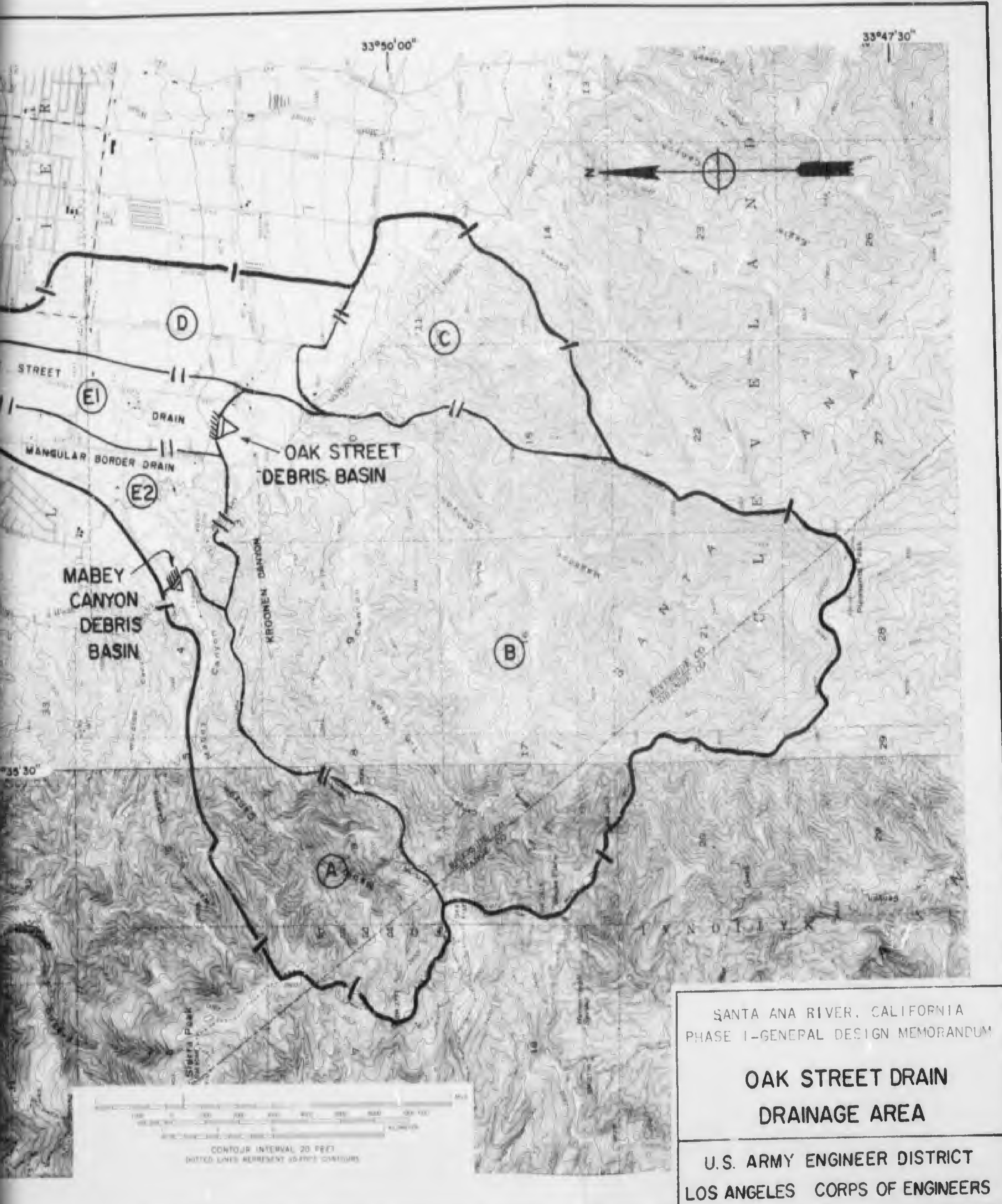
33°50'00"
117°35'00"
117°35'30"
CORONA
OAK STREET
DRAIN
MANGULAR BORDER DRAIN
OAK STREET
DEBRIS BASIN
MABEY
CANYON
DEBRIS
BASIN
KROONEN CANYON
F
E3
D
C
E1
E2
A
LEGEND
BOUNDARY OF DRAINAGE AREA.
BOUNDARY OF SUBAREAS.
SUBAREA DESIGNATION.
DEBRIS BASIN.
LOS ANGELES CO.
San Bernardino
SAN BERNARDINO CO.
Los Angeles
Corona
Santa Ana
RIVERSIDE CO.
ORANGE CO.
PACIFIC
OCEAN
SAN DIEGO CO.
N
VICINITY MAP
AREA COVERED BY MAP
SCALE IN MILES
CONTOUR INTERVAL 20 FEET

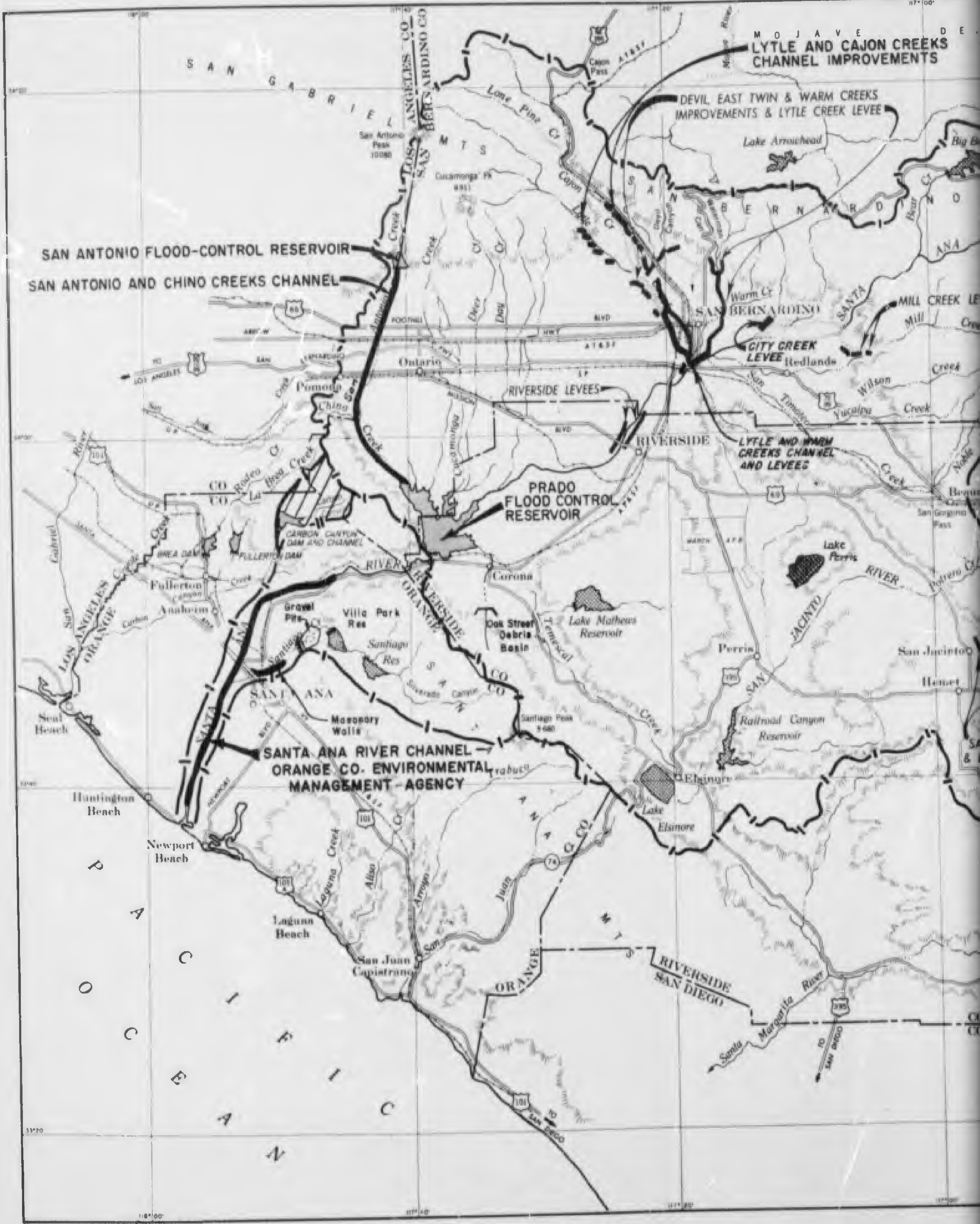


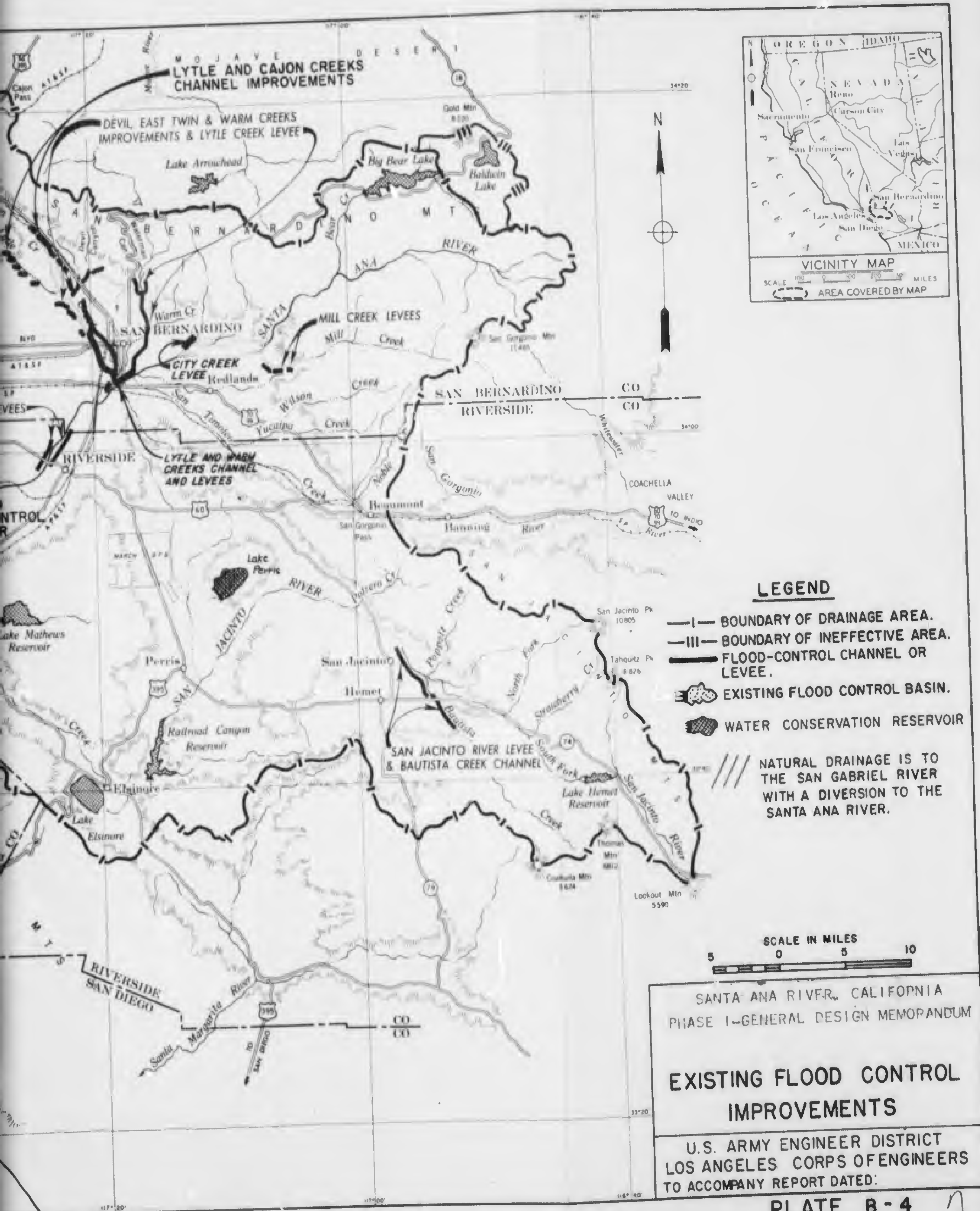
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

OAK STREET DRAIN
DRAINAGE AREA

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

PLATE B-3


LYTLE AND CAJON CREEKS CHANNEL IMPROVEMENTS
DEVIL, EAST TWIN & WARM CREEKS IMPROVEMENTS & LYTLE CREEK LEVEE
SAN ANTONIO FLOOD-CONTROL RESERVOIR
SAN ANTONIO AND CHINO CREEKS CHANNEL
MILL CREEK LE
CITY CREEK LEVEE
RIVERSIDE LEVEES
LYTLE AND WARM CREEKS CHANNEL AND LEVEES
PRADO FLOOD CONTROL RESERVOIR
CARBON CANYON DAM AND CHANNEL
BREA DAM
FULLERTON DAM
Oak Street Debris Basin
SANTA ANA RIVER CHANNEL — ORANGE CO. ENVIRONMENTAL MANAGEMENT - AGENCY
Mossoary Walls
SAN GABRIEL MTS
SAN BERNARDINO
MOJAVE DE
LOS ANGELES CO
SAN BERNARDINO CO
RIVERSIDE
ORANGE
SAN DIEGO
San Bernardino
Redlands
Ontario
Pomona
Riverside
Corona
Fullerton
Anaheim
Santa Ana
Perris
San Jacinto
Hemet
Elsinore
Lake Elsinore
Lake Perris
Lake Mathews Reservoir
Railroad Canyon Reservoir
Lake Arrowhead
Villa Park Res
Santiago Res
Seal Beach
Huntington Beach
Newport Beach
Laguna Beach
San Juan Capistrano
PACIFIC OCEAN

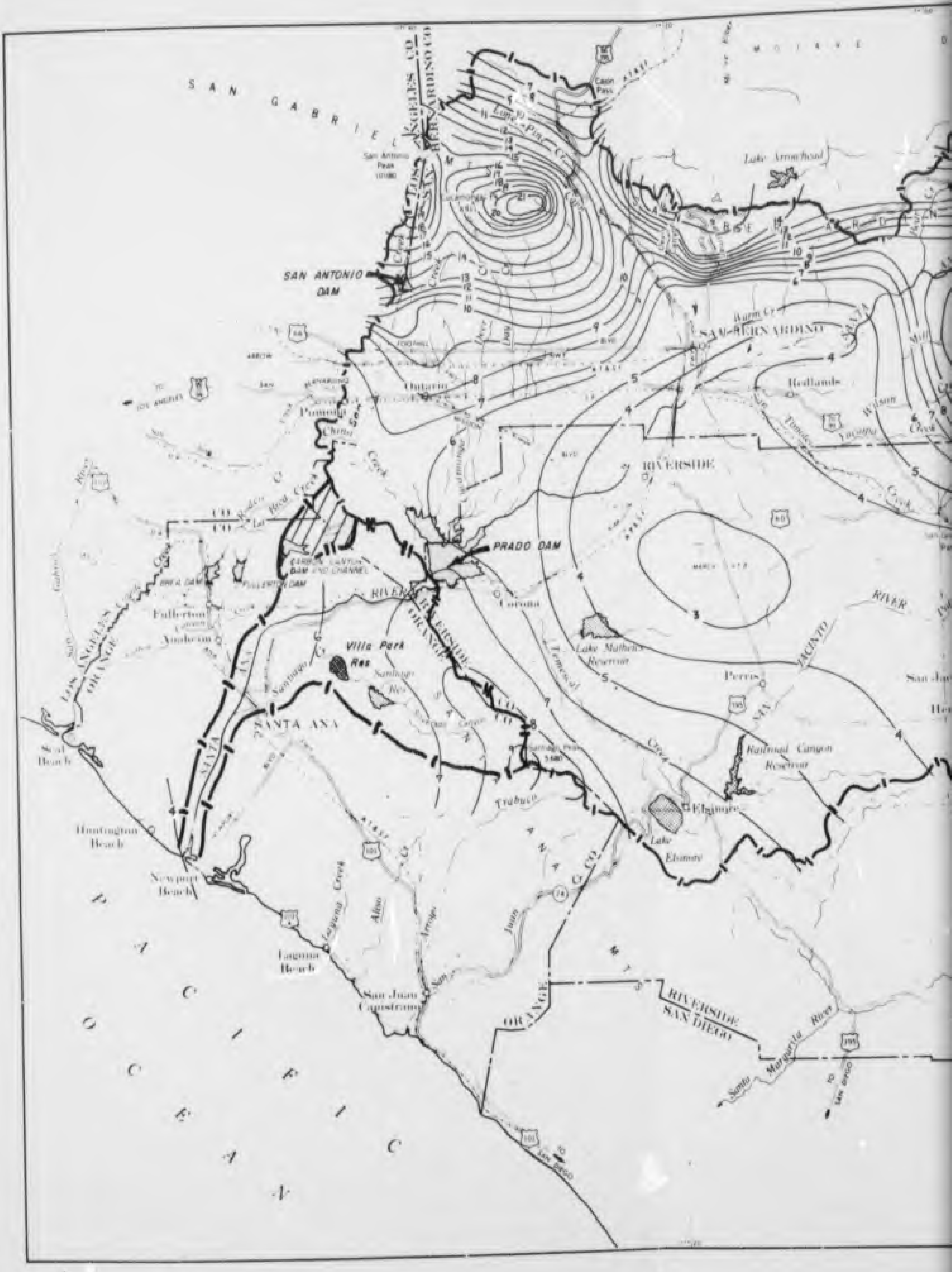


SANTA ANA RIVER, CALIFORNIA
PHASE I—GENERAL DESIGN MEMORANDUM

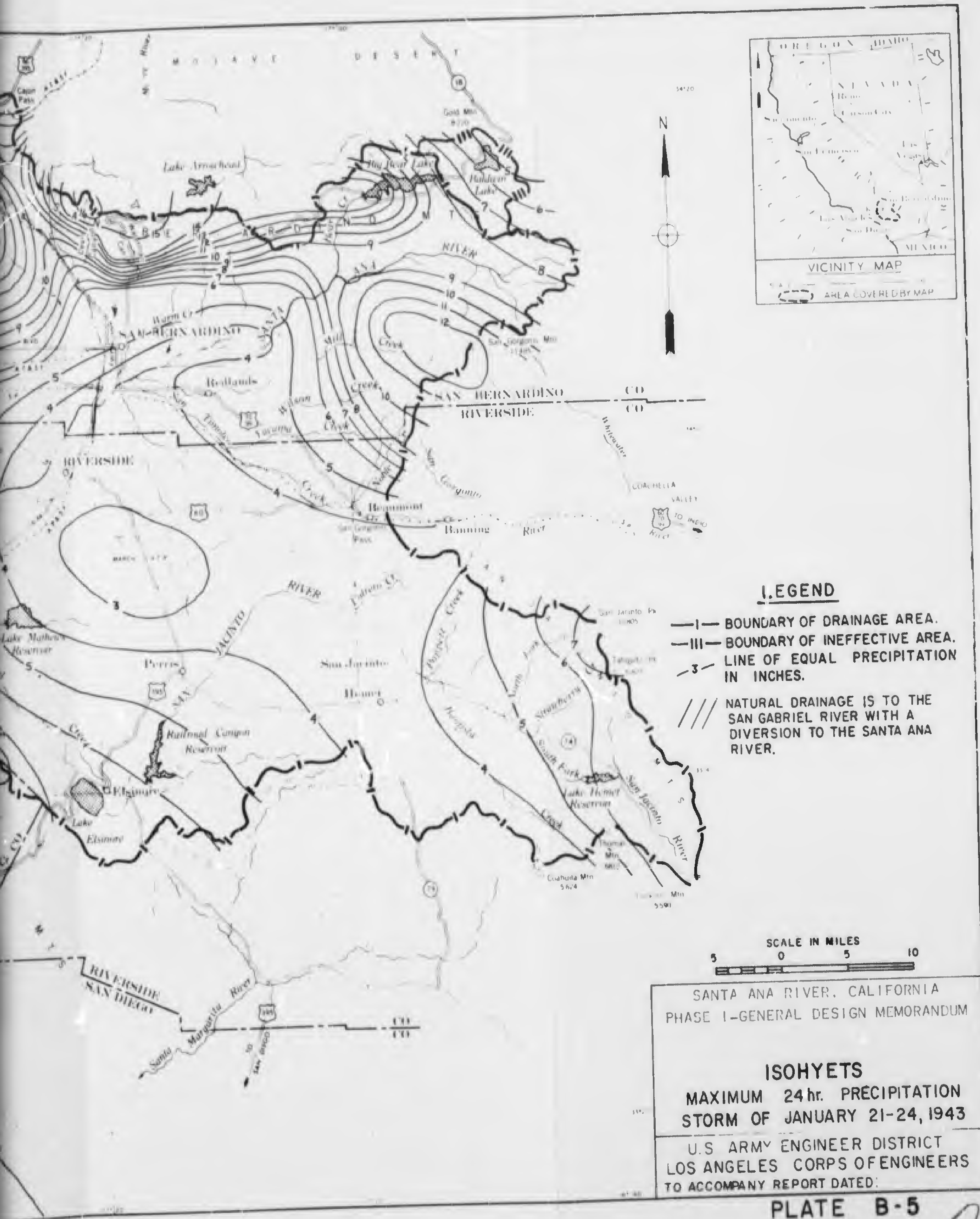
EXISTING FLOOD CONTROL IMPROVEMENTS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-4



SAN GABRIEL MTS
MOJAVE
SAN ANTONIO DAM
PRADO DAM
CARBON CANYON DAM AND CHANNEL
BREA DAM
FULLERTON DAM
Villa Park Res
LOS ANGELES CO
SAN BERNARDINO CO
RIVERSIDE CO
ORANGE CO
RIVERSIDE
SAN DIEGO
SAN BERNARDINO
Redlands
Ontario
Pomona
Fullerton
Anaheim
SANTA ANA
Corona
Perris
Elsinore
Lake Elsinore
Lake Mathews Reservoir
Railroad Canyon Reservoir
Lake Arrowhead
Seal Beach
Huntington Beach
Newport Beach
Laguna Beach
San Juan Capistrano
Santiago Res
Trabuco
SANTA ANA RIVER
SAN JACINTO RIVER
Santa Margarita River
Temescal Creek
Lytle Cr
Cajon Pass
San Antonio Peak
PACIFIC OCEAN



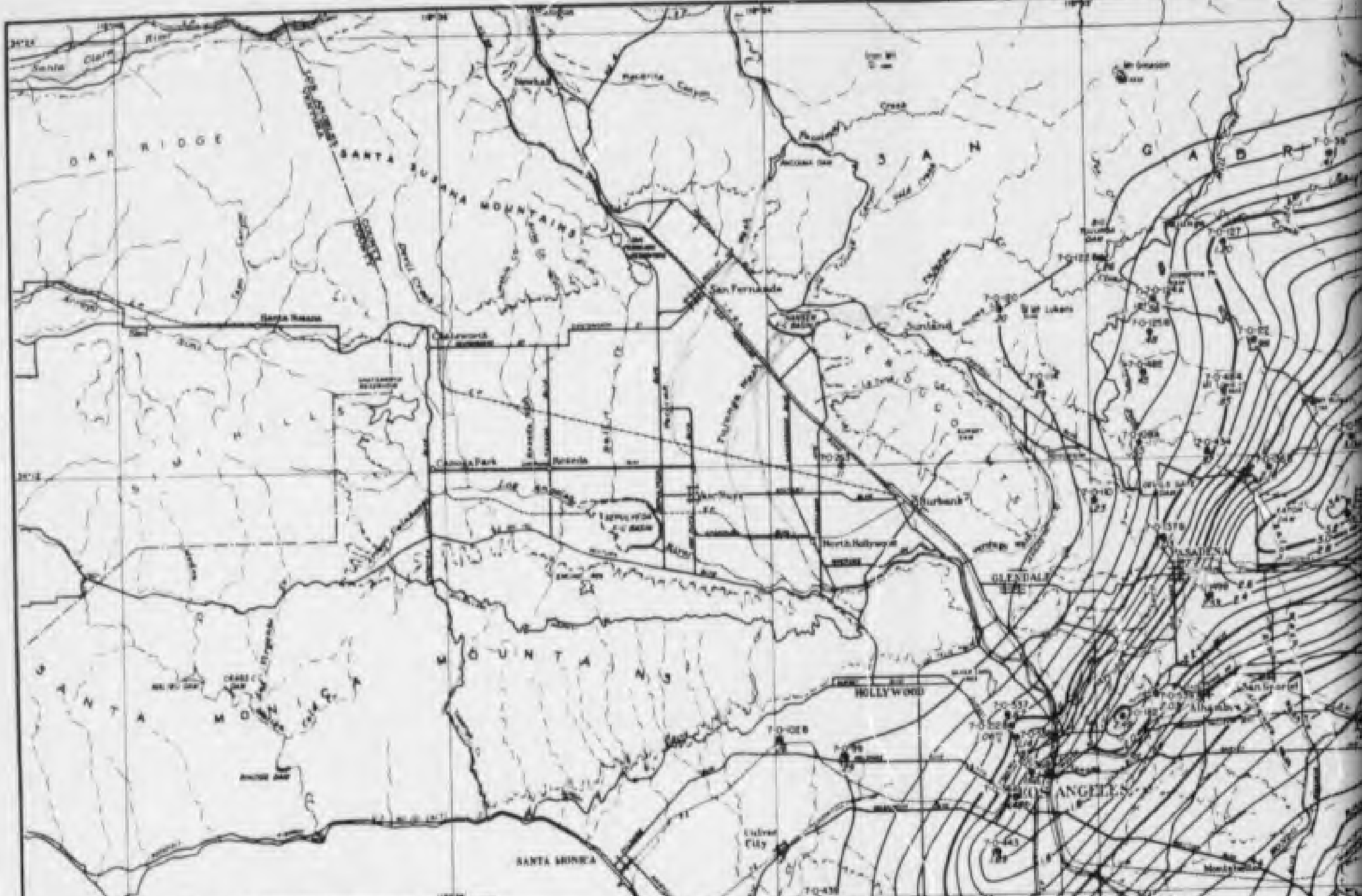
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

ISOHYETS
MAXIMUM 24 hr. PRECIPITATION
STORM OF JANUARY 21-24, 1943

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-5

2



STATIONS REQUIRED FOR THE DETERMINATION OF ISOHYETS WHERE RECORDED PRECIPITATION DEPTHS WERE LESS THAN 1 INCH IN 3 HOURS

STATION	
NUMBER	NAME
7-0-22	OPID'S CAMP - CAMP SINGER
7-0-58	LOOMIS RANCH - ALDER CREEK
7-0-99	HANCOCK PARK
7-0-102B	BEVERLY HILLS - NEW CITY HALL
7-0-108A	ARROYO SECO FLOOD HEADQUARTERS
7-0-110	FLINTRIDGE FIRE STATION
7-0-114	BRIGGS TERRACE - PICKENS CANYON
7-0-122	HAINES CANYON - UPPER
7-0-122B	BIG TUJUNGA DAM - NEW CARETAKER'S HOUSE
7-0-294	CLEAR CREEK
7-0-125B	SWITZER'S CAMP
7-0-148B	EL MONTE FIRE STATION
7-0-285	BURBANK AIRPORT
7-0-434	ALTADENA COUNTY FORESTRY PARK
7-0-143B	BALDWIN HILLS - STANDARD OIL FIELD OFFICE
7-0-480	PINE CANYON - ARROYO SECO
7-0-484	MT. LUKE RIDGE
7-0-537	LOS ANGELES - ECHO PARK & LUCRETIA AVE.
7-F-20	LOS ANGELES - 96TH & CENTRAL AVE.
8-0-48	AZUSA - GRIFFITH
8-0-209A	SAN GABRIEL DAM NO. 1
8-F-29	PUENTE HILLS

RECORDED 15-MIN PRECIPITATION AT STATIONS WHERE DEPTHS EXCEEDED 1 INCH IN 3 HOURS

STATION		PRECIPITATION IN INCHES ON MARCH 3-4 BY 15-MIN PERIODS ENDING AS SHOWN												3-HR. TOTAL IN INCHES
NUMBER	NAME	2215	2230	2245	2300	2315	2330	2345	2400	0015	0030	0045	0100	
7-0-26A	LOS ANGELES - U.S.W.B. - 9TH & MAIN STS.	0.23	0.30	0.21	0.19	0.25	0.05	0.03	0.01	0.01	0.03	0	0	1.40
7-0-26B	LOS ANGELES - U.S.W.B. - 3RD & SPRING STS.	0.11	0.35	0.09	0.16	0.09	0.11	0.09	0.08	0.08	0.08	0.01	0	1.48
7-0-127	COLBY RANCH - SLEEPY HOLLOW RANCH	0.10	0.06	0.04	0.09	0.06	0.16	0.19	0.06	0.14	0.15	0.09	0.20	1.80
7-0-130	HOGEE'S CAMP	0.13	0.10	0.29	0.11	0.37	0.30	0.35	0.46	0.46	0.35	0.15	0.03	3.06
7-0-132B	SIERRA MADRE - CARTER	0.10	0.05	0.01	0.04	0.18	0.16	0.07	0.09	0.49	0.57	1.38	0.44	3.58
7-0-136B	HENNINGER FLATS	0.12	0.06	0.10	0.02	0.46	0.33	0.32	0.21	0.51	0.81	0.09	0.08	3.18
7-0-378	PASADENA - METEOROLOGICAL STATION	0.01	0.01	0.11	0.11	0.21	0.07	0.12	0.09	0.11	0.08	0.09	0.09	1.05
7-0-1458	CALIFORNIA TECH.	0.21	0.15	0.40	0.27	0.60	0.32	0.29	0.44	0.18	0.01	0.08	0.02	2.53
7-0-182	EL SERENO	0.01	0.10	0.25	0.36	0.38	0.33	0.30	0.34	0.11	0.01	0	0.01	2.48
7-0-528	BARLOW SANITORIUM - L.A. CITY	0.08	0.31	0.15	0.05	0.21	0.27	0.09	0.07	0	0.01	0	0	1.06
7-0-443	LOS ANGELES - 30TH & TRINITY STS.	0.21	0.40	0.24	0.40	0.20	0.08	0.13	0.01	0	0.01	0.01	0	1.69
7-0-538	LOS ANGELES - BOYLSTON ST.	0.28	0.30	0.15	0.03	0.16	0.17	0.04	0.17	0.05	0.08	0	0	1.14
7-0-539	LOS ANGELES - STOCKBRIDGE AVE.	0.05	0.15	0.40	0.29	0.16	0.50	0.34	0.18	0	0	0	0.01	2.03
7-F-90	LOS ANGELES - SLAUSON & LONG BEACH AVE.	0.12	0.08	0.08	0.18	0.60	0.25	0.16	0.14	0.03	0	0	0	1.69
8-0-34	SAN GABRIEL DAM NO. 2	0.10	0.04	0.16	0.14	0.14	0.48	0.38	0.13	0.44	0.29	0.32	0.34	2.94
8-J-30	MONROVIA FALLS	0.05	0.12	0.10	0.16	0.27	0.15	0	0.01	0	0.24	0.70	0.04	1.82
8-0-75	CRYSTAL LAKE CO. PARK - E. PINE FLAT	0.07	0.06	0.03	0.10	0.10	0.15	0.20	0.25	0.07	0.06	0.01	0.03	1.15
8-0-117B	STATE PRISON CAMP NO. 37	0.09	0.06	0.15	0.14	0.29	0.09	0.15	0.23	0.14	0.17	0.16	0.18	1.84
8-0-208	SANTA ANITA - SPRING CAMP	0.11	0.25	0.14	0.51	0.35	0.03	0.04	0.15	0.90	0.58	0.03	0.15	3.29

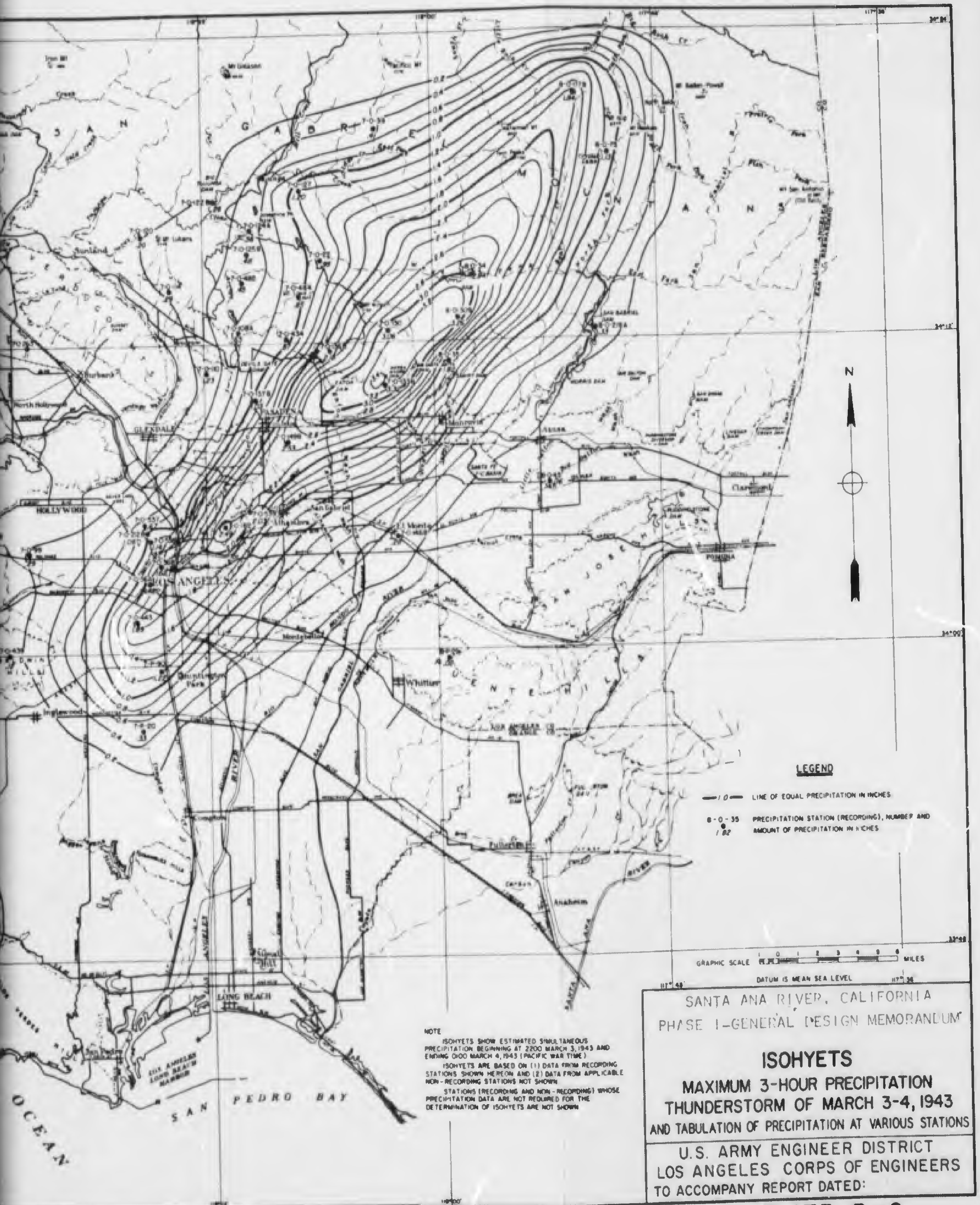
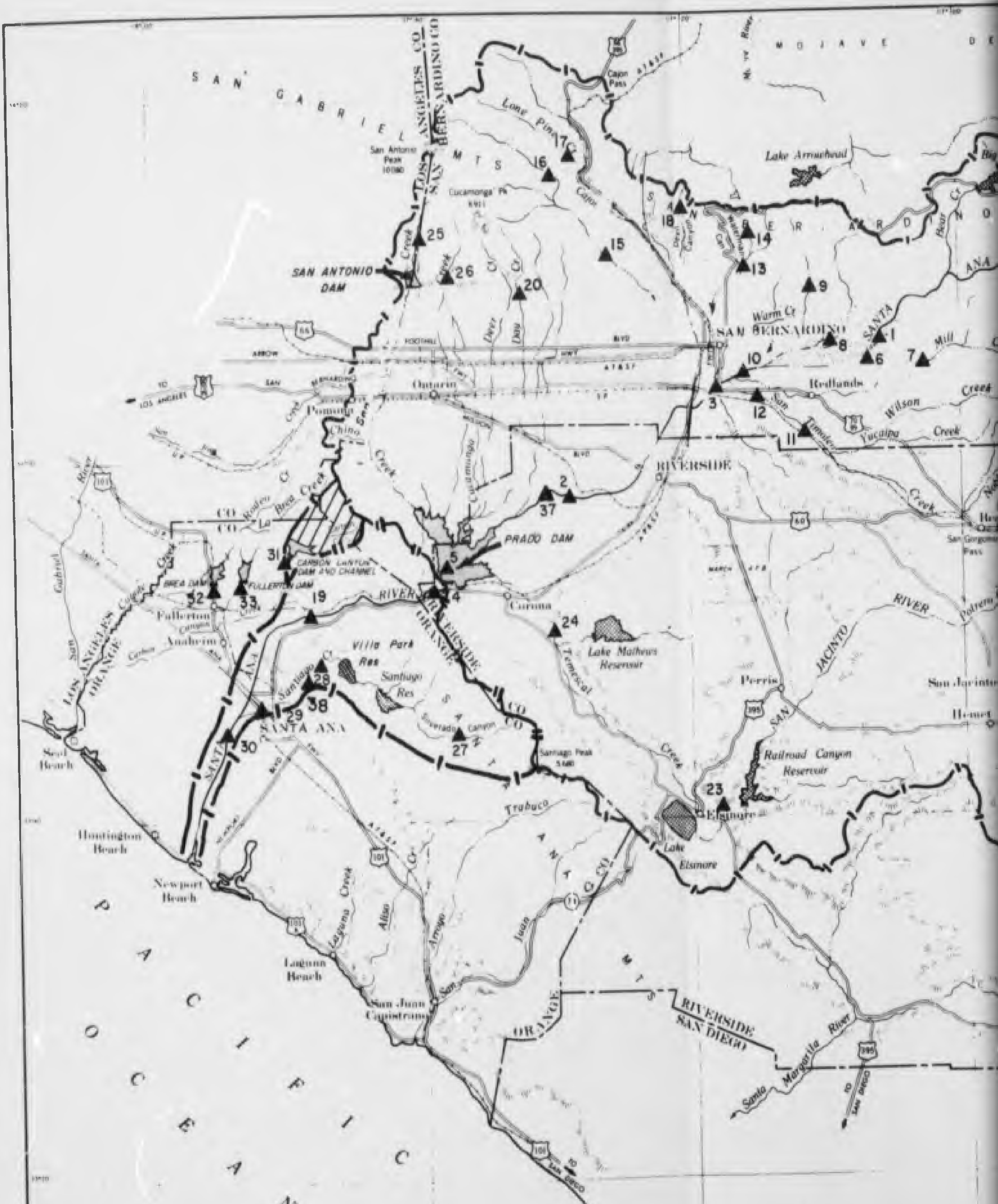

S A N G A B R I E L M O U N T A I N S
HOLLYWOOD
GLENDALE
LOS ANGELES
Whittier
Inglewood
LONG BEACH
Fullerton
Anaheim
Claremont
POMONA
SAN PEDRO BAY
OCEAN
N
LEGEND
1.0 LINE OF EQUAL PRECIPITATION IN INCHES
8-0-35 1.82 PRECIPITATION STATION (RECORDING), NUMBER AND AMOUNT OF PRECIPITATION IN INCHES
GRAPHIC SCALE 1 0 1 2 3 4 5 6 MILES
DATUM IS MEAN SEA LEVEL
NOTE
ISOHYETS SHOW ESTIMATED SIMULTANEOUS PRECIPITATION BEGINNING AT 2200 MARCH 3, 1943 AND ENDING 0100 MARCH 4, 1943 (PACIFIC WAR TIME)
ISOHYETS ARE BASED ON (1) DATA FROM RECORDING STATIONS SHOWN HEREON AND (2) DATA FROM APPLICABLE NON-RECORDING STATIONS NOT SHOWN
STATIONS (RECORDING AND NON-RECORDING) WHOSE PRECIPITATION DATA ARE NOT REQUIRED FOR THE DETERMINATION OF ISOHYETS ARE NOT SHOWN
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM
ISOHYETS
MAXIMUM 3-HOUR PRECIPITATION
THUNDERSTORM OF MARCH 3-4, 1943
AND TABULATION OF PRECIPITATION AT VARIOUS STATIONS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-6

2


SAN GABRIEL MTS
LOS ANGELES CO
SAN BERNARDINO CO
MOJAVE
San Antonio Peak 10080
Cucamonga Pk
SAN ANTONIO DAM
Lake Arrowhead
SAN BERNARDINO
Redlands
Ontario
Pomona
Chino
RIVERSIDE
PRADO DAM
CARBON CANYON DAM AND CHANNEL
BREA DAM
FULLERTON DAM
Fullerton
Anaheim
Corona
Lake Mathews Reservoir
Villa Park Res
Santiago Res
SANTA ANA
Santiago Peak
Perris
Railroad Canyon Reservoir
Elsinore
Lake Elsinore
San Jacinto
Hemet
Seal Beach
Huntington Beach
Newport Beach
Laguna Beach
San Juan Capistrano
RIVERSIDE
SAN DIEGO
ORANGE
PACIFIC OCEAN
Santa Margarita River
San Gorgonio Pass

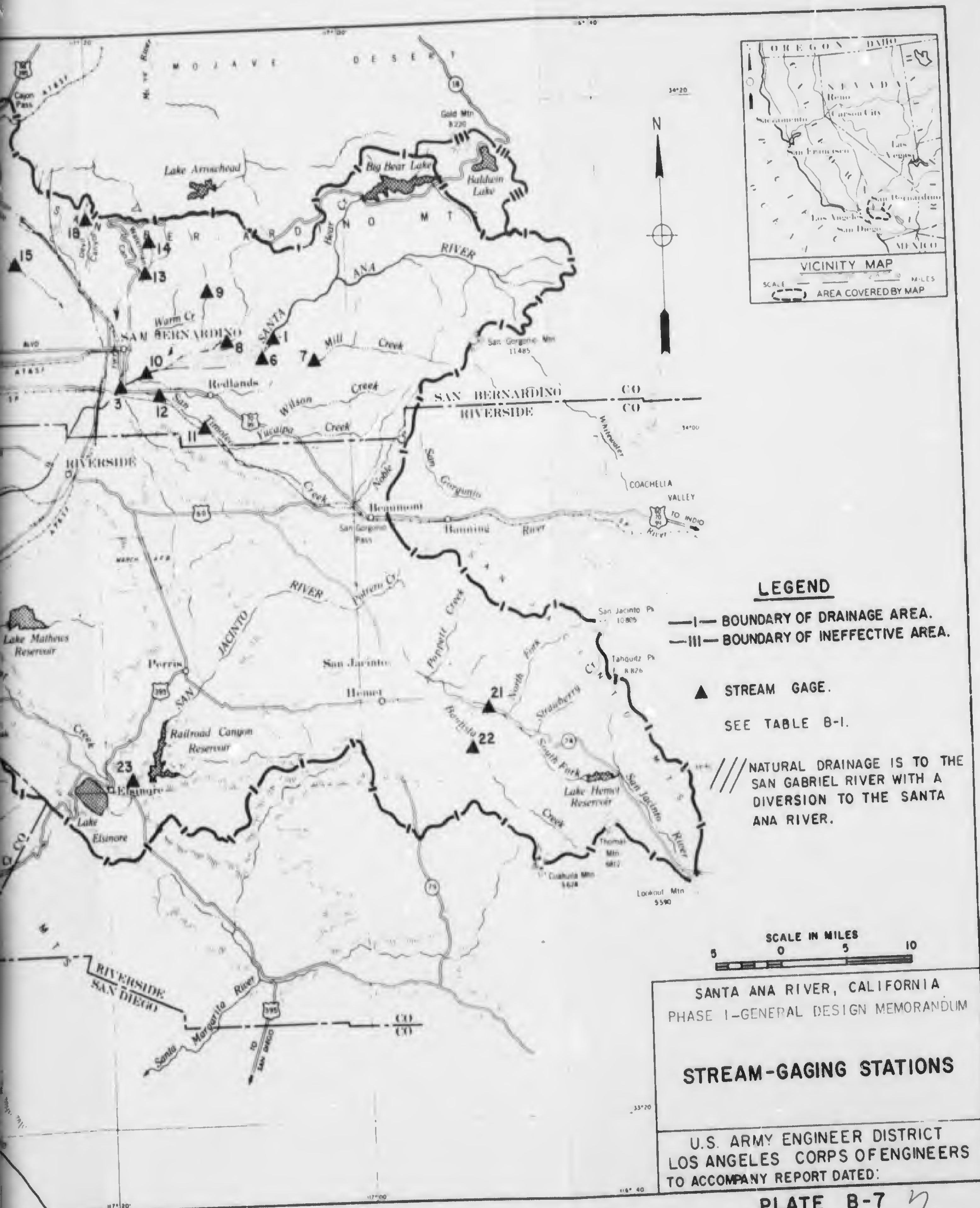
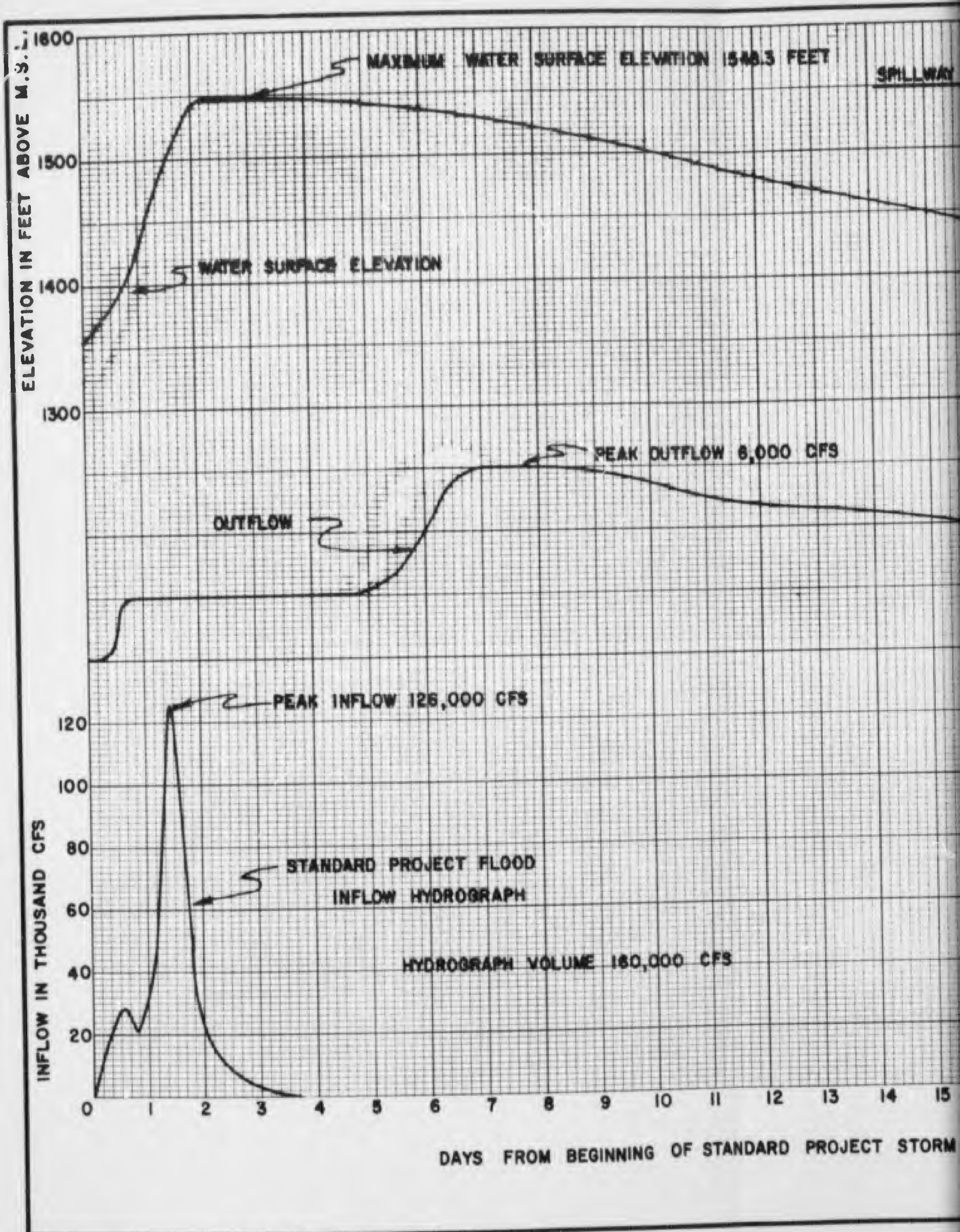
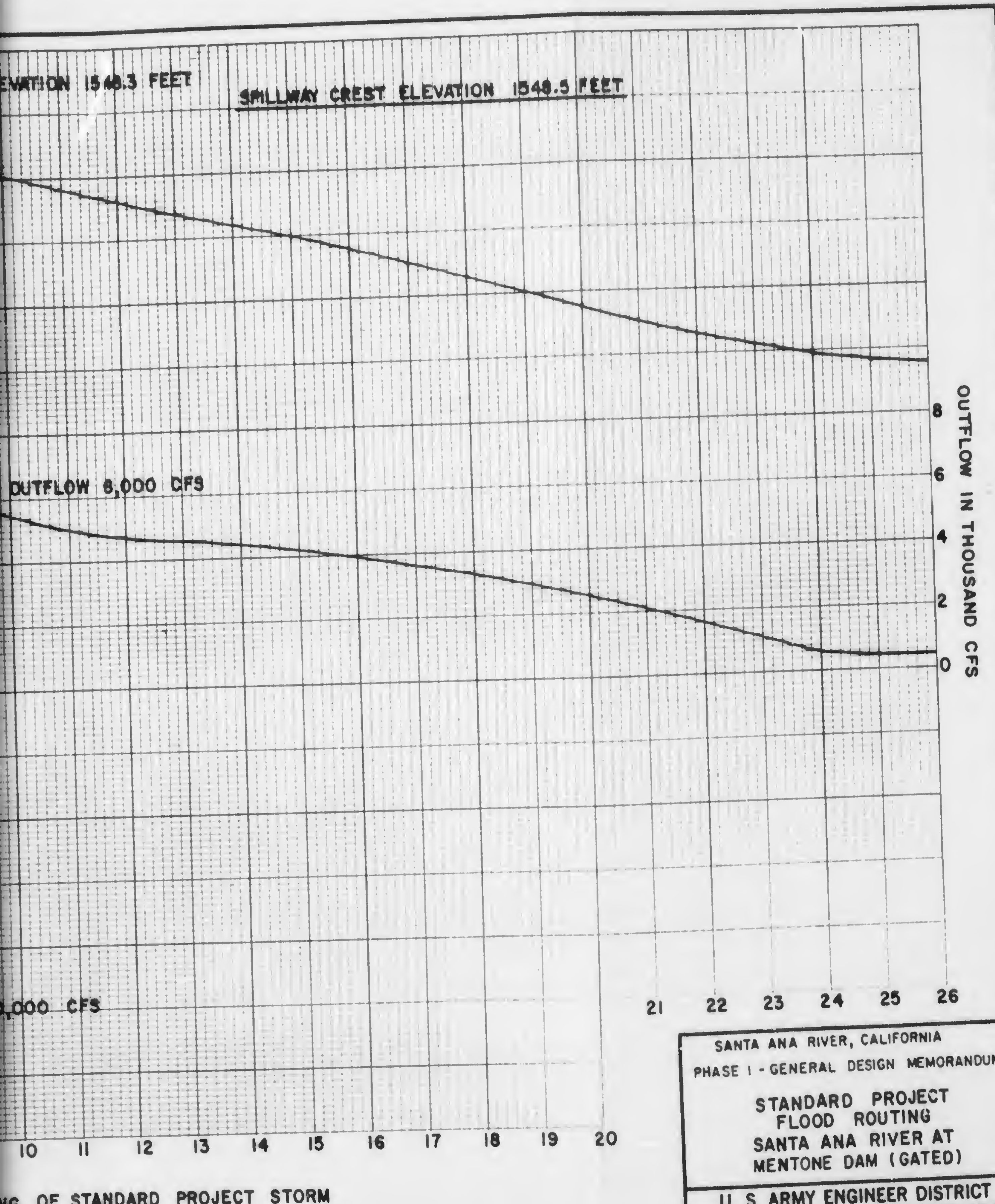

VICINITY MAP
AREA COVERED BY MAP
MOJAVE DESERT
Lake Arrowhead
Big Bear Lake
Baldwin Lake
SANTA ANA RIVER
SAN BERNARDINO
Redlands
Mill Creek
Wilson Creek
Yucaipa Creek
San Gorgonio Mtn 11485
SAN BERNARDINO CO
RIVERSIDE CO
RIVERSIDE
Beaumont
San Gorgonio Pass
Banning
COACHELLA VALLEY
TO INDIO
SAN JACINTO RIVER
Potrero Cr.
Poppet Creek
San Jacinto Pk 10805
Tahquitz Pk 8826
Lake Mathews Reservoir
Perris
San Jacinto
Hemet
Railroad Canyon Reservoir
Elsinore
Lake Elsinore
Lake Hemet Reservoir
Strawberry
North Fork
South Fork
Cahuilla Mtn
Lookout Mtn 5590
RIVERSIDE CO
SAN DIEGO CO
Santa Margarita River
LEGEND
BOUNDARY OF DRAINAGE AREA.
BOUNDARY OF INEFFECTIVE AREA.
STREAM GAGE.
SEE TABLE B-1.
NATURAL DRAINAGE IS TO THE SAN GABRIEL RIVER WITH A DIVERSION TO THE SANTA ANA RIVER.
SCALE IN MILES
5 0 5 10
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM
STREAM-GAGING STATIONS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-7


ELEVATION IN FEET ABOVE M.S.L.
1600
1500
1400
1300
MAXIMUM WATER SURFACE ELEVATION 1548.3 FEET
SPILLWAY
WATER SURFACE ELEVATION
PEAK OUTFLOW 6,000 CFS
OUTFLOW
PEAK INFLOW 126,000 CFS
INFLOW IN THOUSAND CFS
120
100
80
60
40
20
STANDARD PROJECT FLOOD
INFLOW HYDROGRAPH
HYDROGRAPH VOLUME 160,000 CFS
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
DAYS FROM BEGINNING OF STANDARD PROJECT STORM

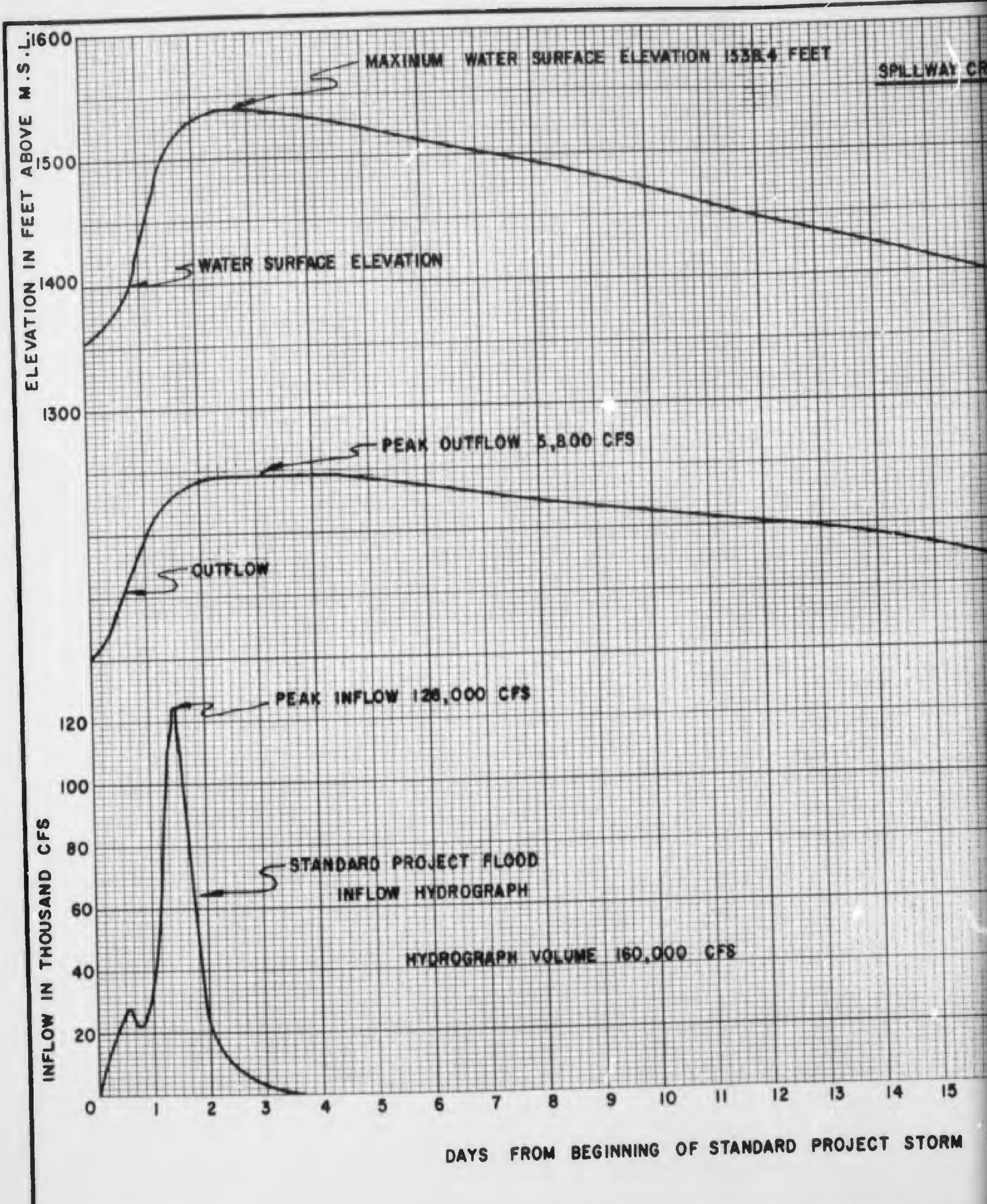


SANTA ANA RIVER, CALIFORNIA

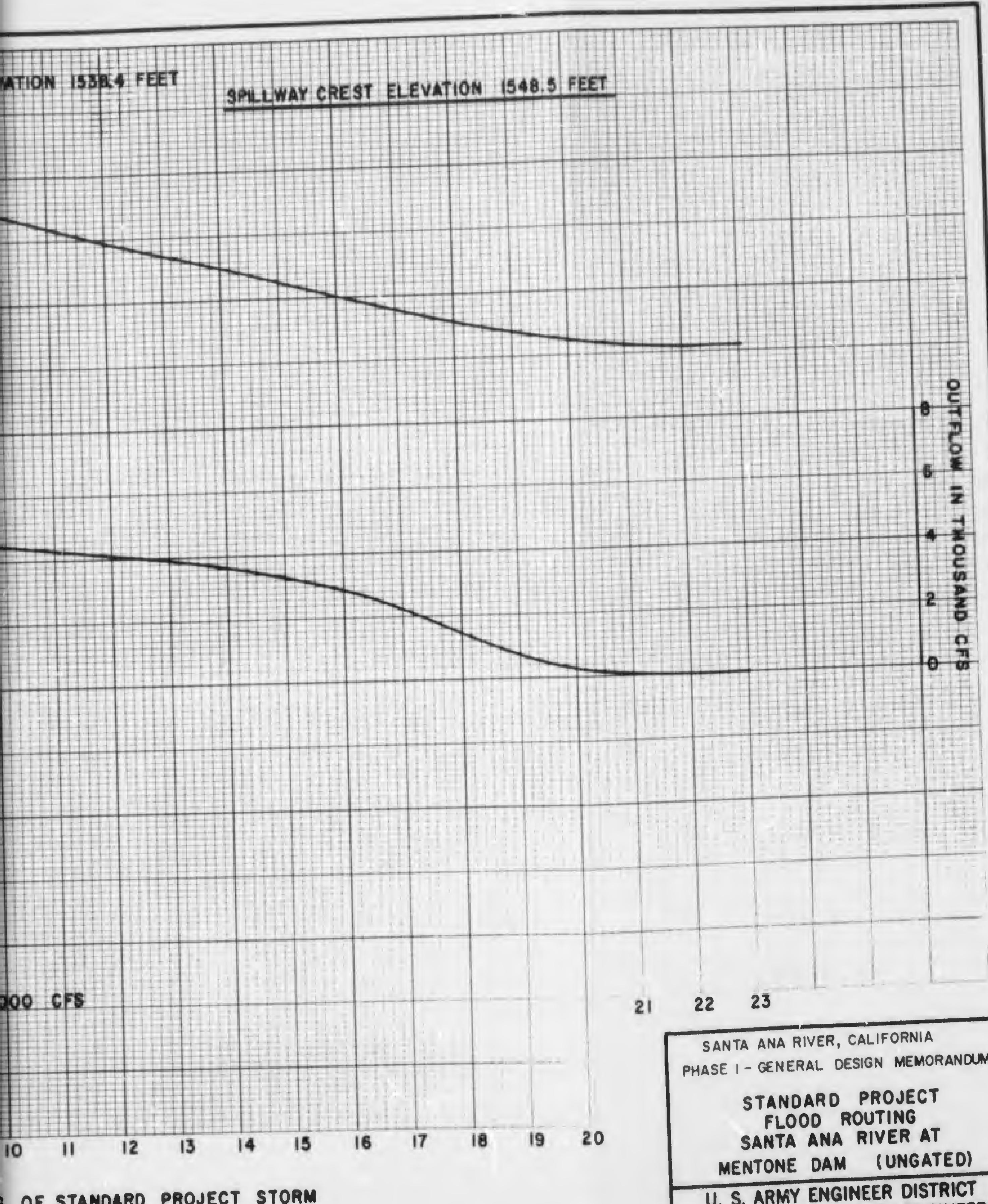
PHASE I - GENERAL DESIGN MEMORANDUM

STANDARD PROJECT
FLOOD ROUTING
SANTA ANA RIVER AT
MENTONE DAM (GATED)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



ELEVATION IN FEET ABOVE M.S.L.
1600
1500
1400
1300
MAXIMUM WATER SURFACE ELEVATION 1538.4 FEET
SPILLWAY CR
WATER SURFACE ELEVATION
PEAK OUTFLOW 5,800 CFS
OUTFLOW
PEAK INFLOW 126,000 CFS
INFLOW IN THOUSAND CFS
120
100
80
60
40
20
STANDARD PROJECT FLOOD
INFLOW HYDROGRAPH
HYDROGRAPH VOLUME 160,000 CFS
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
DAYS FROM BEGINNING OF STANDARD PROJECT STORM

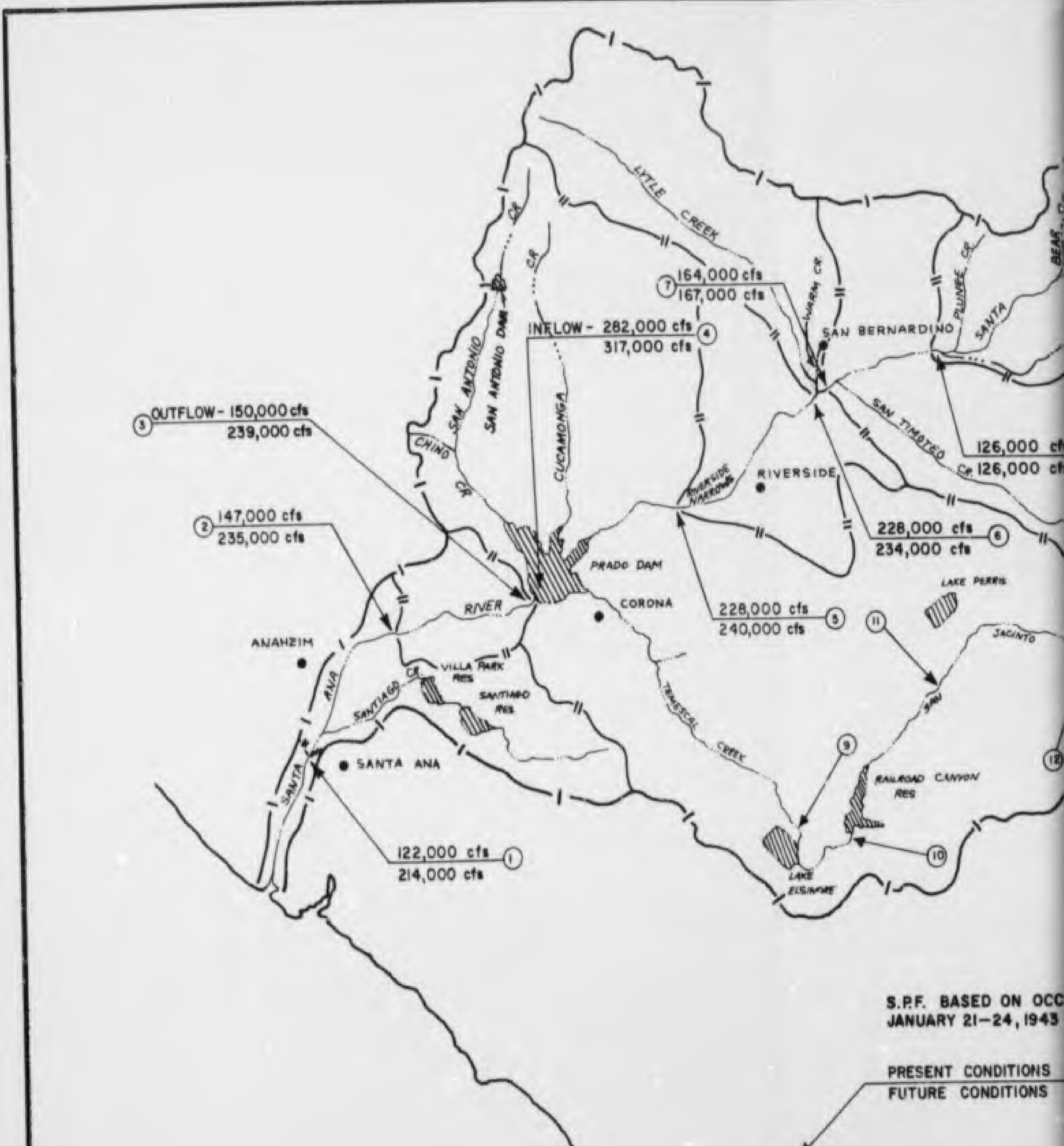


SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

STANDARD PROJECT
FLOOD ROUTING
SANTA ANA RIVER AT
MENTONE DAM (UNGATED)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-9



LYTLE CREEK
SAN ANTONIO CR
SAN ANTONIO DAM
CR
CUCAMONGA CR
CHINO CR
7 164,000 cfs
167,000 cfs
WARM CR
SAN BERNARDINO
PLUNGE CR
SANTA
BEAR
INFLOW - 282,000 cfs 4
317,000 cfs
3 OUTFLOW - 150,000 cfs
239,000 cfs
SAN TIMOTEO CR
126,000 cfs
126,000 cfs
RIVERSIDE NARROWS
RIVERSIDE
2 147,000 cfs
235,000 cfs
228,000 cfs 6
234,000 cfs
PRADO DAM
CORONA
RIVER
228,000 cfs 5
240,000 cfs
11
LAKE PERRIS
JACINTO
ANAHEIM
VILLA PARK RES
SANTIAGO CR
SANTIAGO RES
SANTA ANA
TEMESCAL CREEK
SAN
9
RAILROAD CANYON RES
12
SANTA ANA
122,000 cfs 1
214,000 cfs
10
LAKE ELSINORE
S.P.F. BASED ON OCC
JANUARY 21-24, 1943
PRESENT CONDITIONS
FUTURE CONDITIONS

S.P.F. BASED ON OCCURENCE OF
JANUARY 21–24, 1943 GENERAL STORM.

PRESENT CONDITIONS SPF
FUTURE CONDITIONS SPF

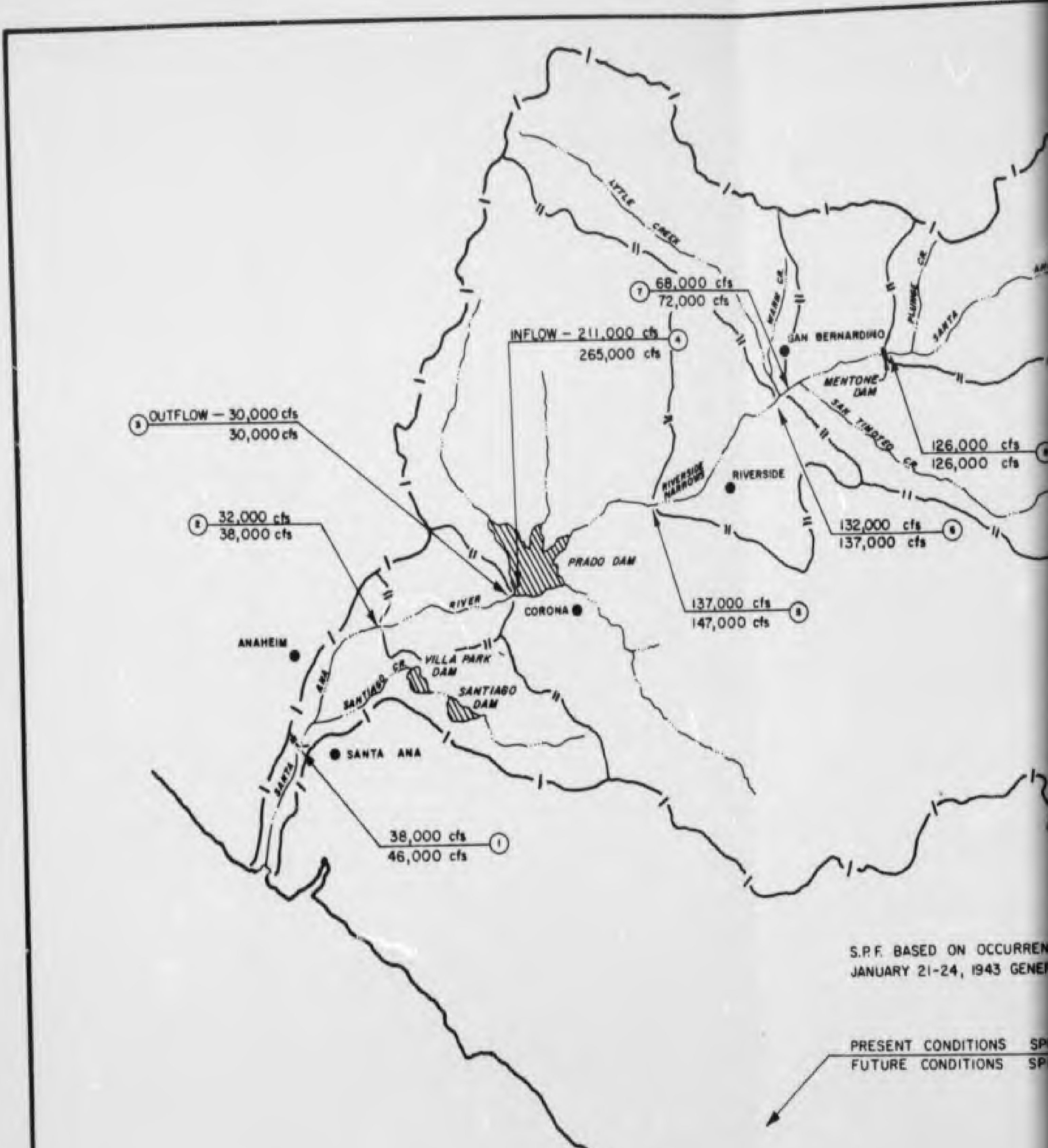
SCALE IN MILES
5 0 5 10

SANTA ANA RIVER, CALIFORNIA
PHASE I–GENERAL DESIGN MEMORANDUM

STANDARD PROJECT FLOOD PEAK DISCHARGES
SANTA ANA RIVER
PRESENT AND FUTURE CONDITIONS
WITHOUT PROJECT

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-10


LYTLE CREEK
68,000 cfs
72,000 cfs
INFLOW — 211,000 cfs
265,000 cfs
OUTFLOW — 30,000 cfs
30,000 cfs
SAN BERNARDINO
MENTONE DAM
SAN TIMOTEO CR
126,000 cfs
126,000 cfs
RIVERSIDE NARROWS
RIVERSIDE
132,000 cfs
137,000 cfs
32,000 cfs
38,000 cfs
PRADO DAM
137,000 cfs
147,000 cfs
RIVER
CORONA
ANAHEIM
VILLA PARK DAM
SANTIAGO CR
SANTIAGO DAM
SANTA ANA
38,000 cfs
46,000 cfs
S.P.F. BASED ON OCCURREN
JANUARY 21-24, 1943 GENE
PRESENT CONDITIONS SP
FUTURE CONDITIONS SP

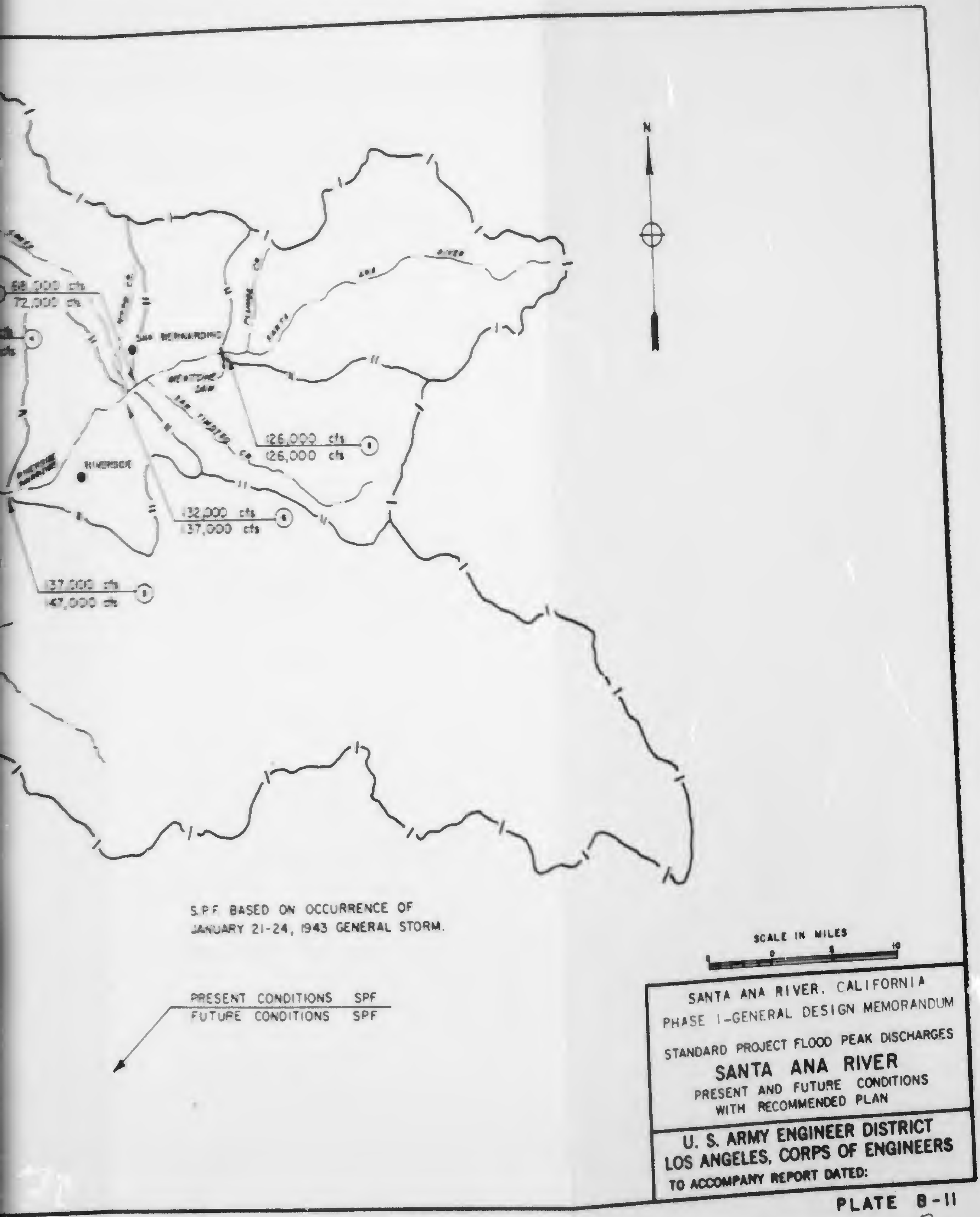
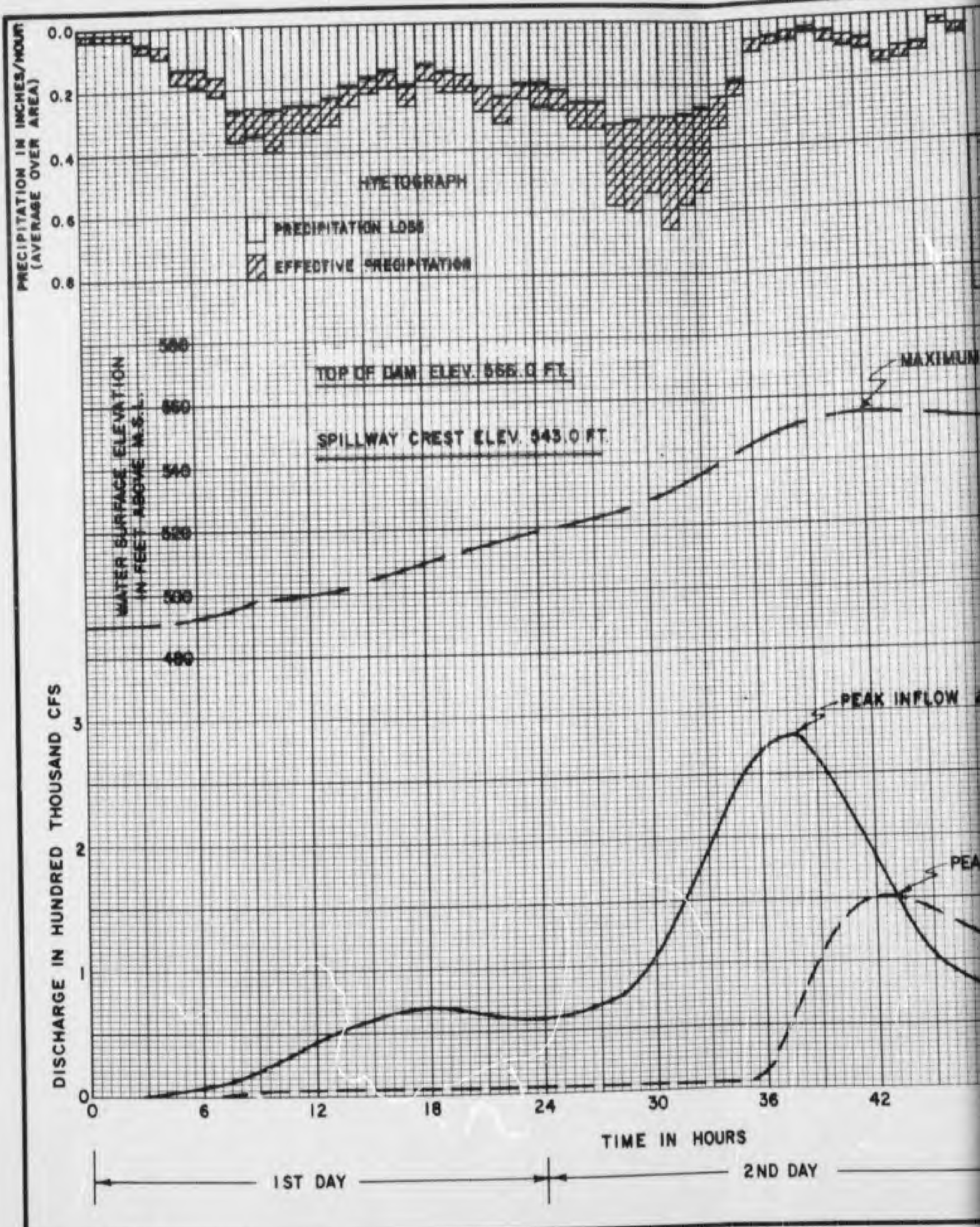
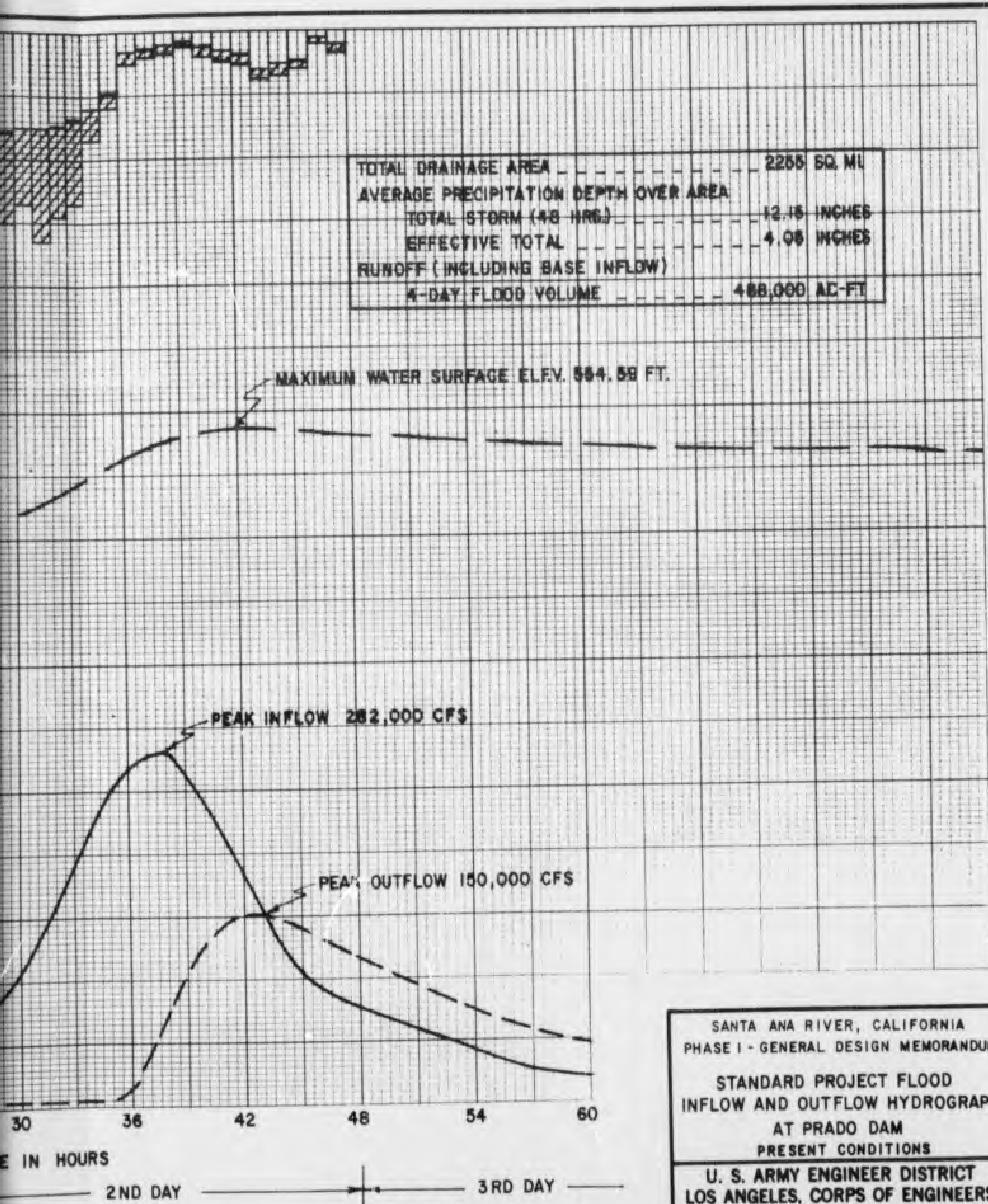

N
RIVER
ANA
SAN BERNARDINO
MENTONE DAM
126,000 cfs
126,000 cfs
RIVERSIDE
132,000 cfs
137,000 cfs
137,000 cfs
147,000 cfs
68,000 cfs
72,000 cfs
S.P.F. BASED ON OCCURRENCE OF
JANUARY 21-24, 1943 GENERAL STORM.
PRESENT CONDITIONS SPF
FUTURE CONDITIONS SPF
SCALE IN MILES
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM
STANDARD PROJECT FLOOD PEAK DISCHARGES
SANTA ANA RIVER
PRESENT AND FUTURE CONDITIONS
WITH RECOMMENDED PLAN
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-11

2



PRECIPITATION IN INCHES/HOUR (AVERAGE OVER AREA)
0.0
0.2
0.4
0.6
0.8
HYETOGRAPH
PRECIPITATION LOSS
EFFECTIVE PRECIPITATION
WATER SURFACE ELEVATION IN FEET ABOVE M.S.L.
580
560
540
520
500
480
TOP OF DAM ELEV. 566.0 FT.
SPILLWAY CREST ELEV. 543.0 FT.
MAXIMUM
PEAK INFLOW
PEA
DISCHARGE IN HUNDRED THOUSAND CFS
3
2
1
0
0
6
12
18
24
30
36
42
TIME IN HOURS
1ST DAY
2ND DAY

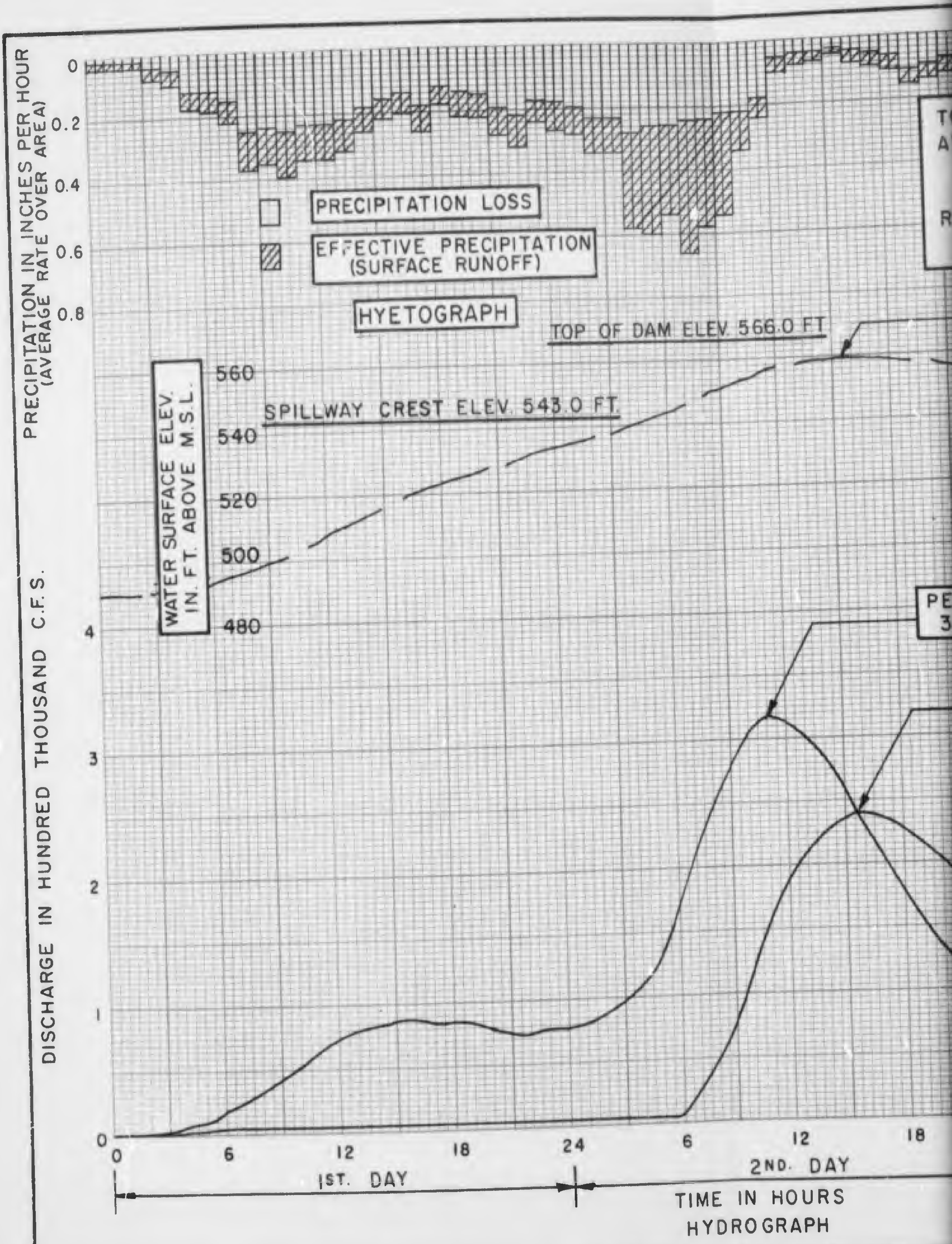


SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

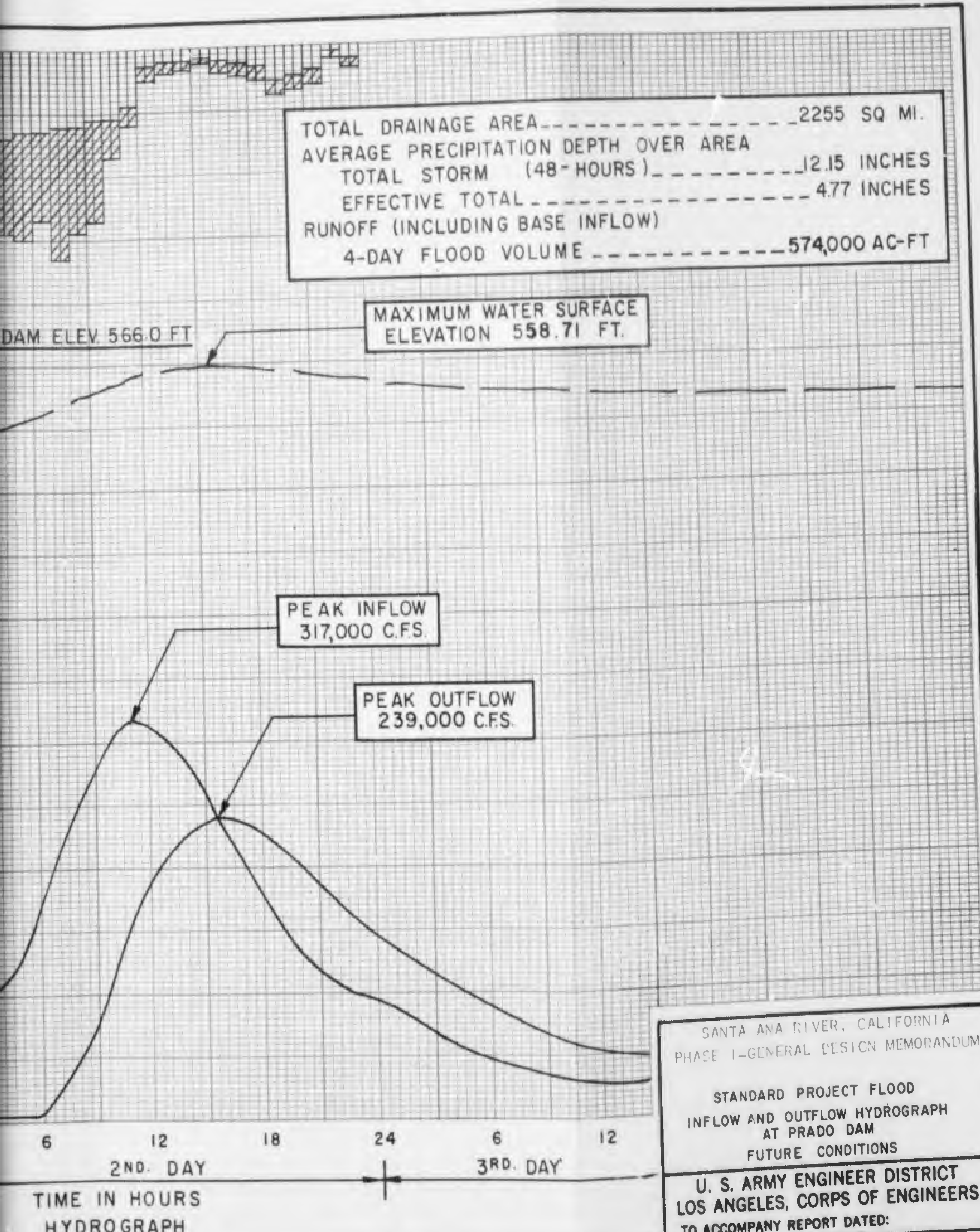
STANDARD PROJECT FLOOD
INFLOW AND OUTFLOW HYDROGRAPHS
AT PRADO DAM
PRESENT CONDITIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-12



PRECIPITATION IN INCHES PER HOUR (AVERAGE RATE OVER AREA)
0
0.2
0.4
0.6
0.8
PRECIPITATION LOSS
EFFECTIVE PRECIPITATION (SURFACE RUNOFF)
HYETOGRAPH
TOP OF DAM ELEV. 566.0 FT.
SPILLWAY CREST ELEV. 543.0 FT.
WATER SURFACE ELEV. IN FT. ABOVE M.S.L.
560
540
520
500
480
DISCHARGE IN HUNDRED THOUSAND C.F.S.
4
3
2
1
0
0
6
12
18
24
6
12
18
1ST. DAY
2ND. DAY
TIME IN HOURS
HYDROGRAPH



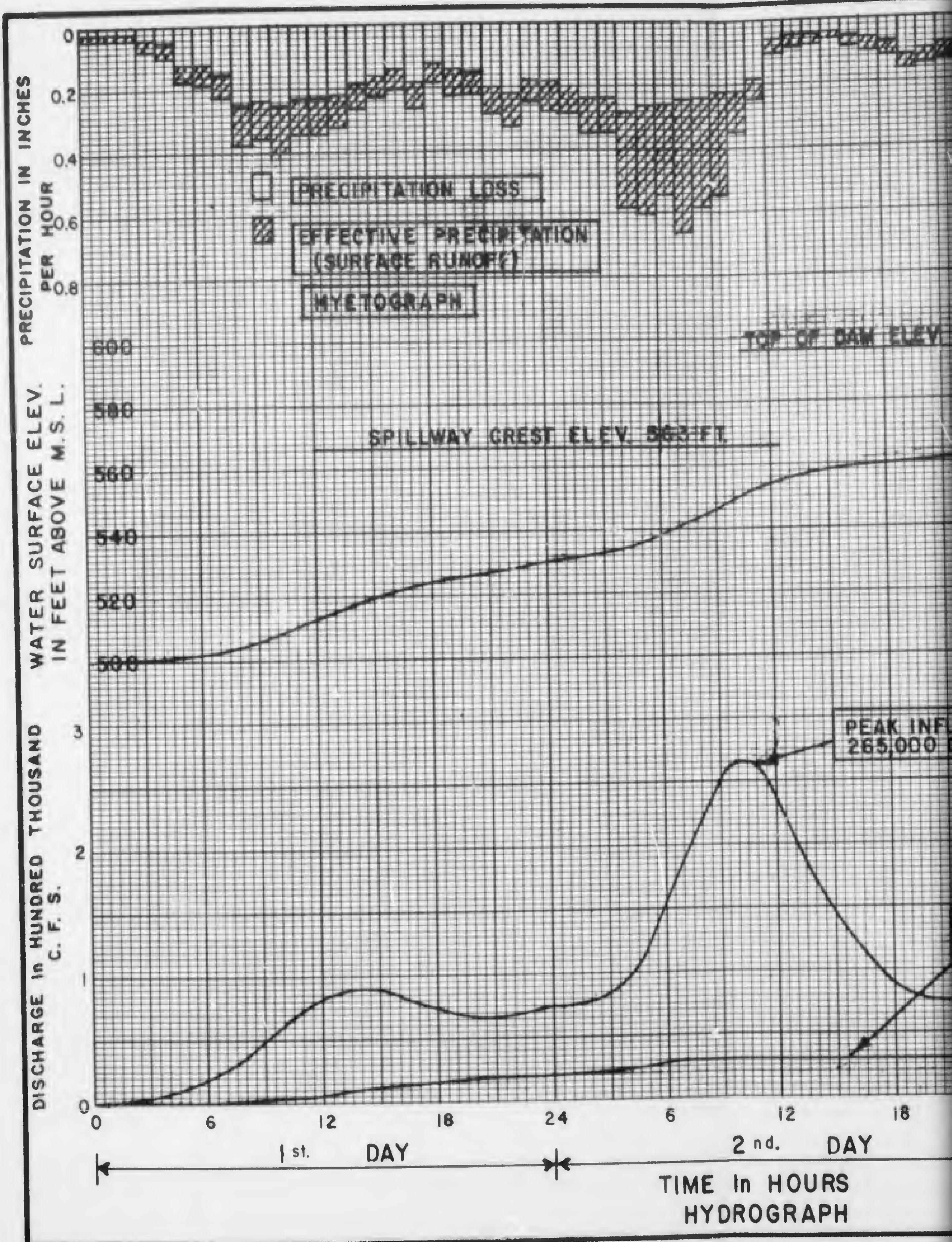
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

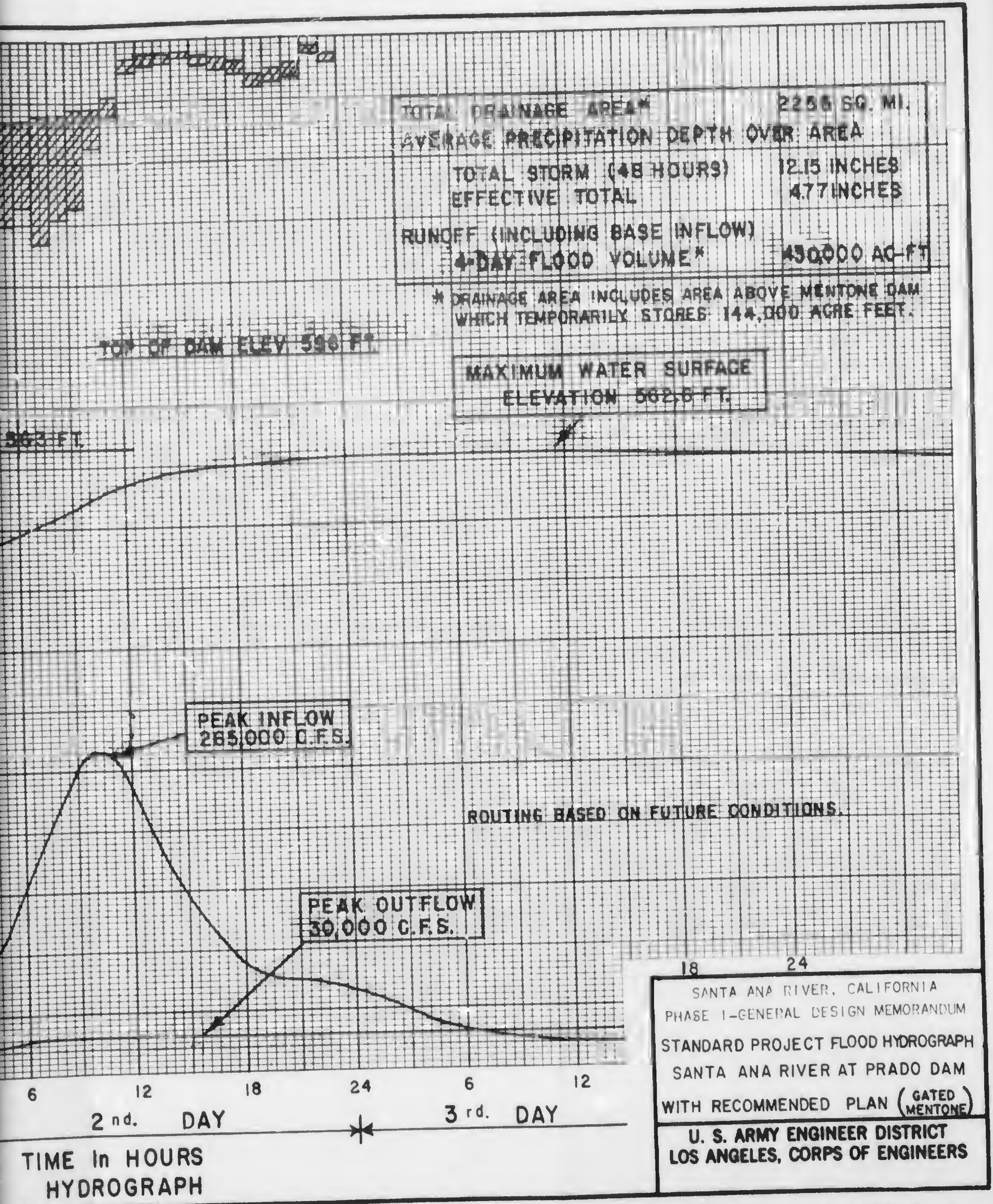
STANDARD PROJECT FLOOD
INFLOW AND OUTFLOW HYDROGRAPH
AT PRADO DAM
FUTURE CONDITIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-13

2


PRECIPITATION IN INCHES PER HOUR
PRECIPITATION LOSS
EFFECTIVE PRECIPITATION (SURFACE RUNOFF)
HYETOGRAPH
TOP OF DAM ELEV
SPILLWAY CREST ELEV. 563 FT.
WATER SURFACE ELEV. IN FEET ABOVE M.S.L.
PEAK INF
265,000
DISCHARGE In HUNDRED THOUSAND C.F.S.
1st. DAY
2nd. DAY
TIME In HOURS
HYDROGRAPH



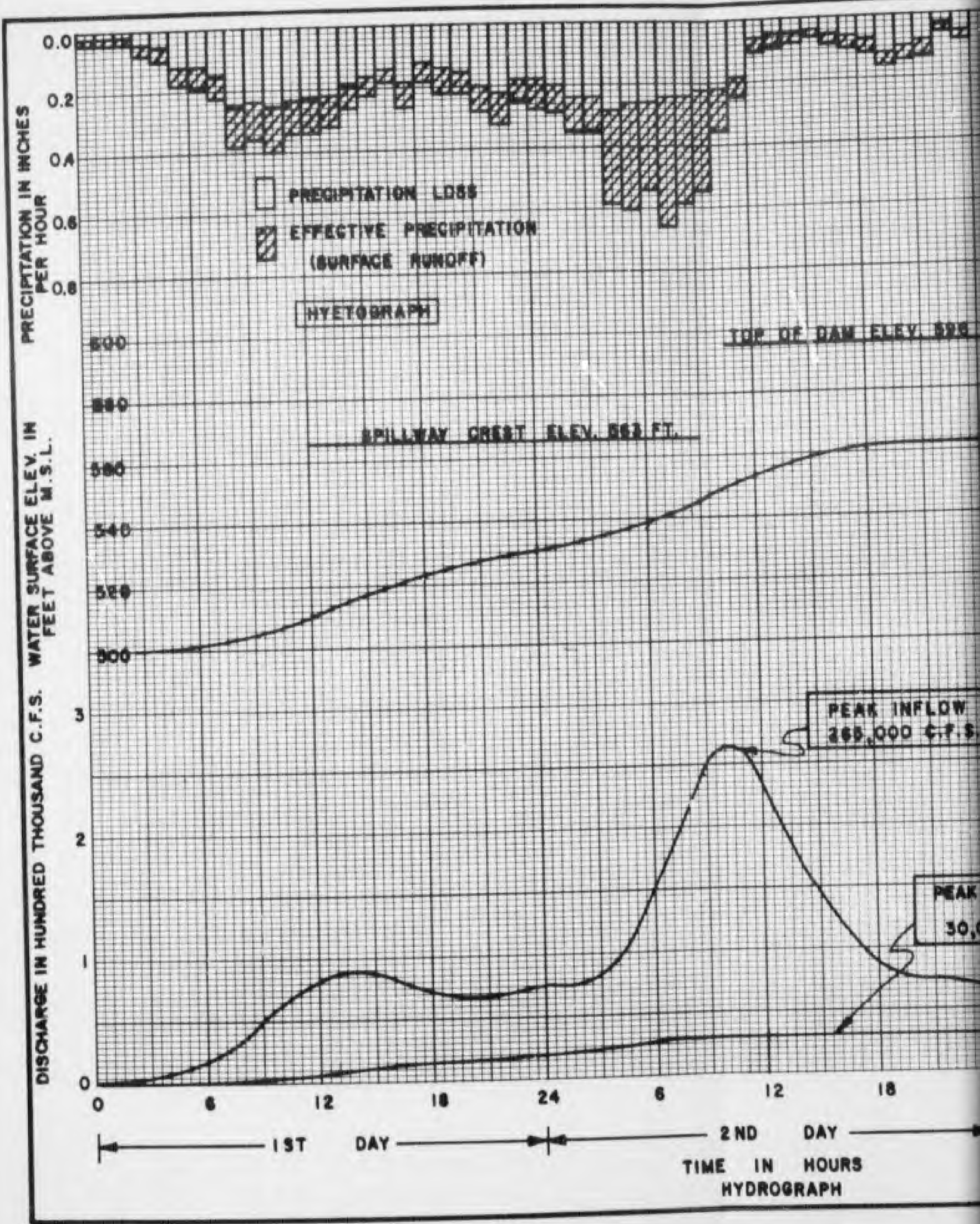
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

STANDARD PROJECT FLOOD HYDROGRAPH
SANTA ANA RIVER AT PRADO DAM
WITH RECOMMENDED PLAN (GATED MENTONE)

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

PLATE B-14

2


PRECIPITATION IN INCHES PER HOUR
0.0
0.2
0.4
0.6
0.8
PRECIPITATION LOSS
EFFECTIVE PRECIPITATION (SURFACE RUNOFF)
HYETOGRAPH
TOP OF DAM ELEV.
SPILLWAY CREST ELEV.
WATER SURFACE ELEV. IN FEET ABOVE M.S.L.
600
580
560
540
520
500
DISCHARGE IN HUNDRED THOUSAND C.F.S.
3
2
1
0
PEAK INFLOW 265,000 C.F.S.
PEAK
0
6
12
18
24
6
12
18
1ST DAY
2ND DAY
TIME IN HOURS
HYDROGRAPH

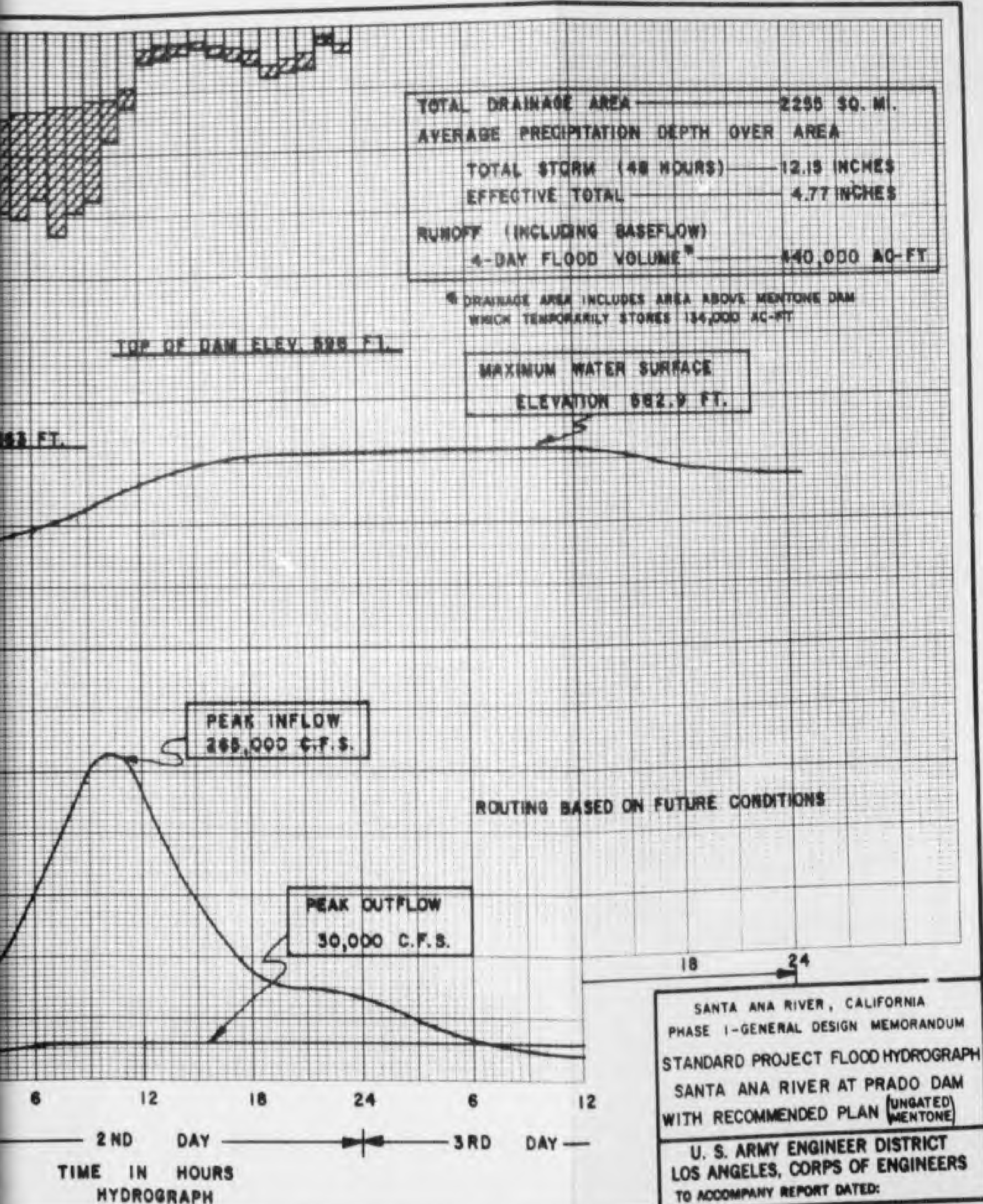
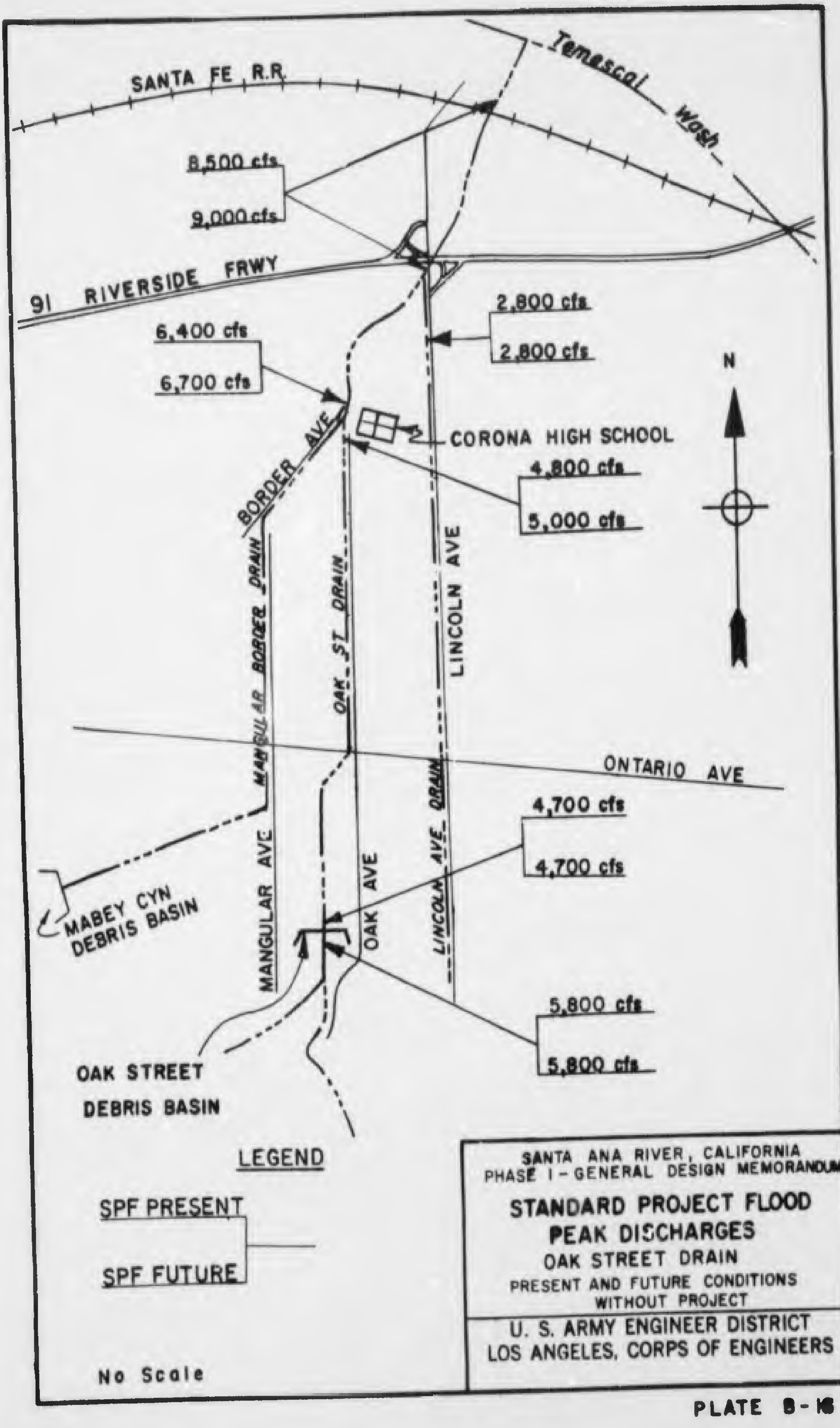

TOTAL DRAINAGE AREA — 2255 SQ. MI.
AVERAGE PRECIPITATION DEPTH OVER AREA
TOTAL STORM (48 HOURS) — 12.15 INCHES
EFFECTIVE TOTAL — 4.77 INCHES
RUNOFF (INCLUDING BASEFLOW)
4-DAY FLOOD VOLUME* — 440,000 AC-FT
* DRAINAGE AREA INCLUDES AREA ABOVE MENTONE DAM WHICH TEMPORARILY STORES 134,000 AC-FT
TOP OF DAM ELEV. 596 FT.
MAXIMUM WATER SURFACE ELEVATION 582.9 FT.
PEAK INFLOW 265,000 C.F.S.
ROUTING BASED ON FUTURE CONDITIONS
PEAK OUTFLOW 30,000 C.F.S.
18
24
6
12
18
24
6
12
2ND DAY
3RD DAY
TIME IN HOURS
HYDROGRAPH
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM
STANDARD PROJECT FLOOD HYDROGRAPH
SANTA ANA RIVER AT PRADO DAM
WITH RECOMMENDED PLAN (UNGATED MENTONE)
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

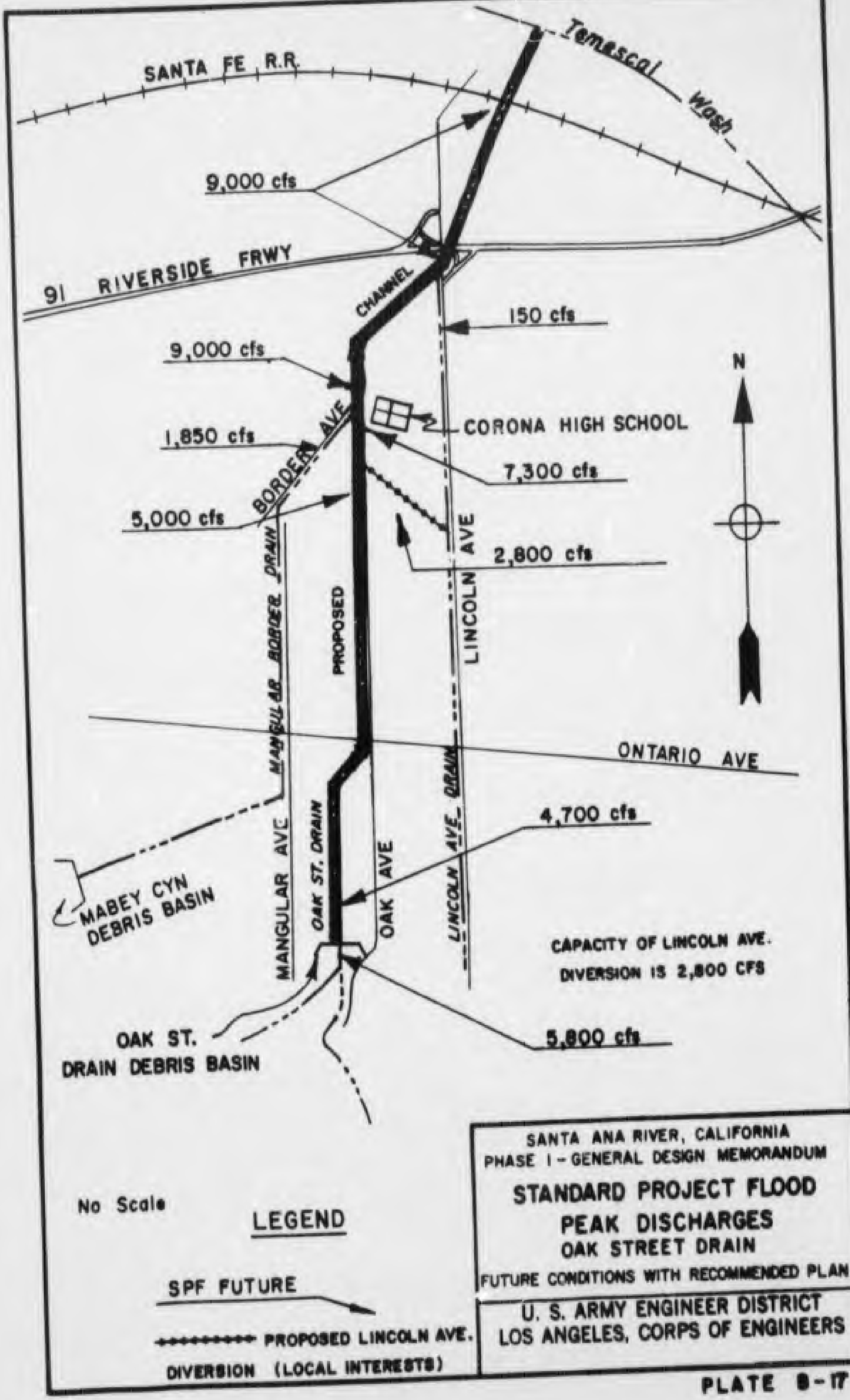
PLATE B-15



SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

STANDARD PROJECT FLOOD
PEAK DISCHARGES
OAK STREET DRAIN
PRESENT AND FUTURE CONDITIONS
WITHOUT PROJECT

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



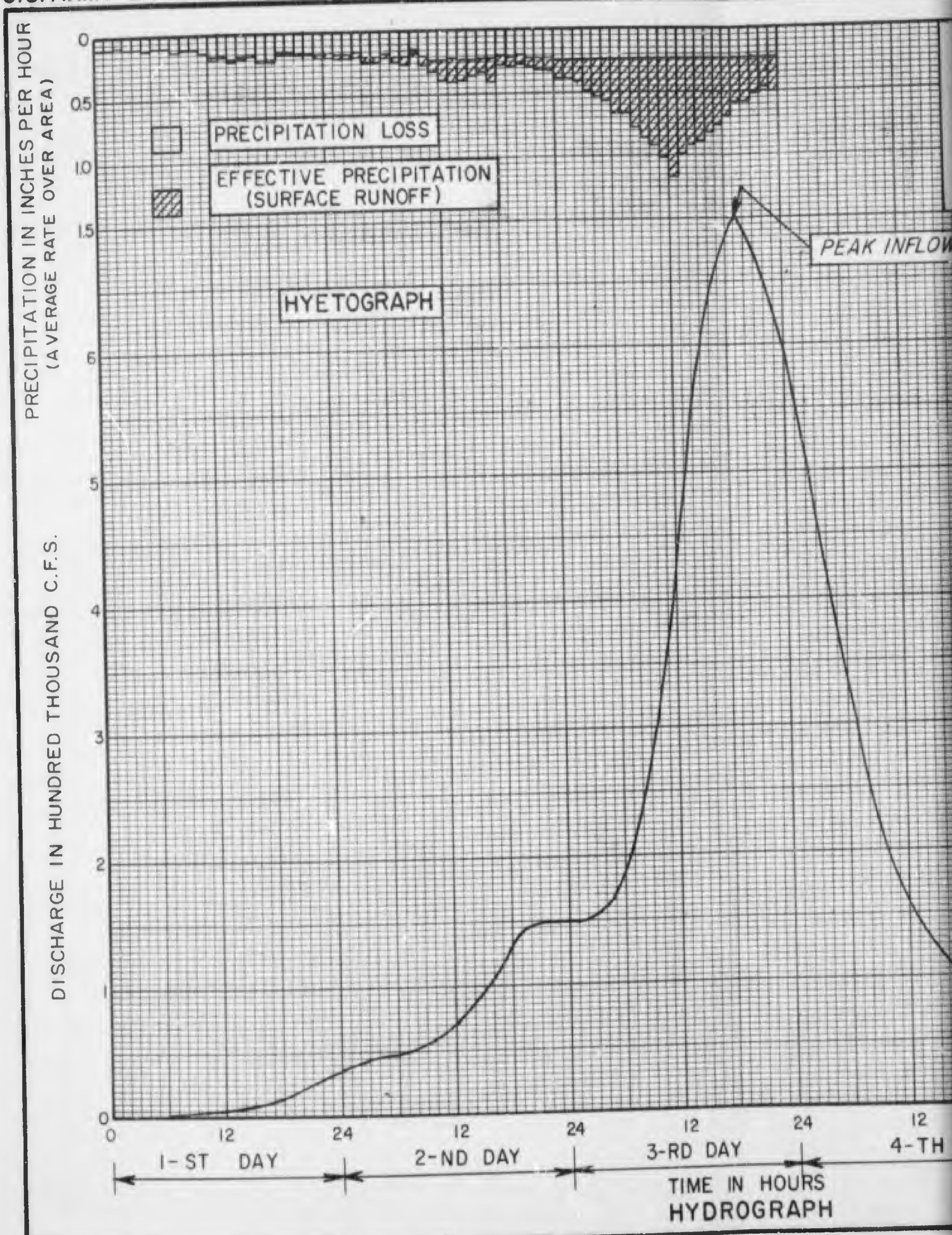
SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

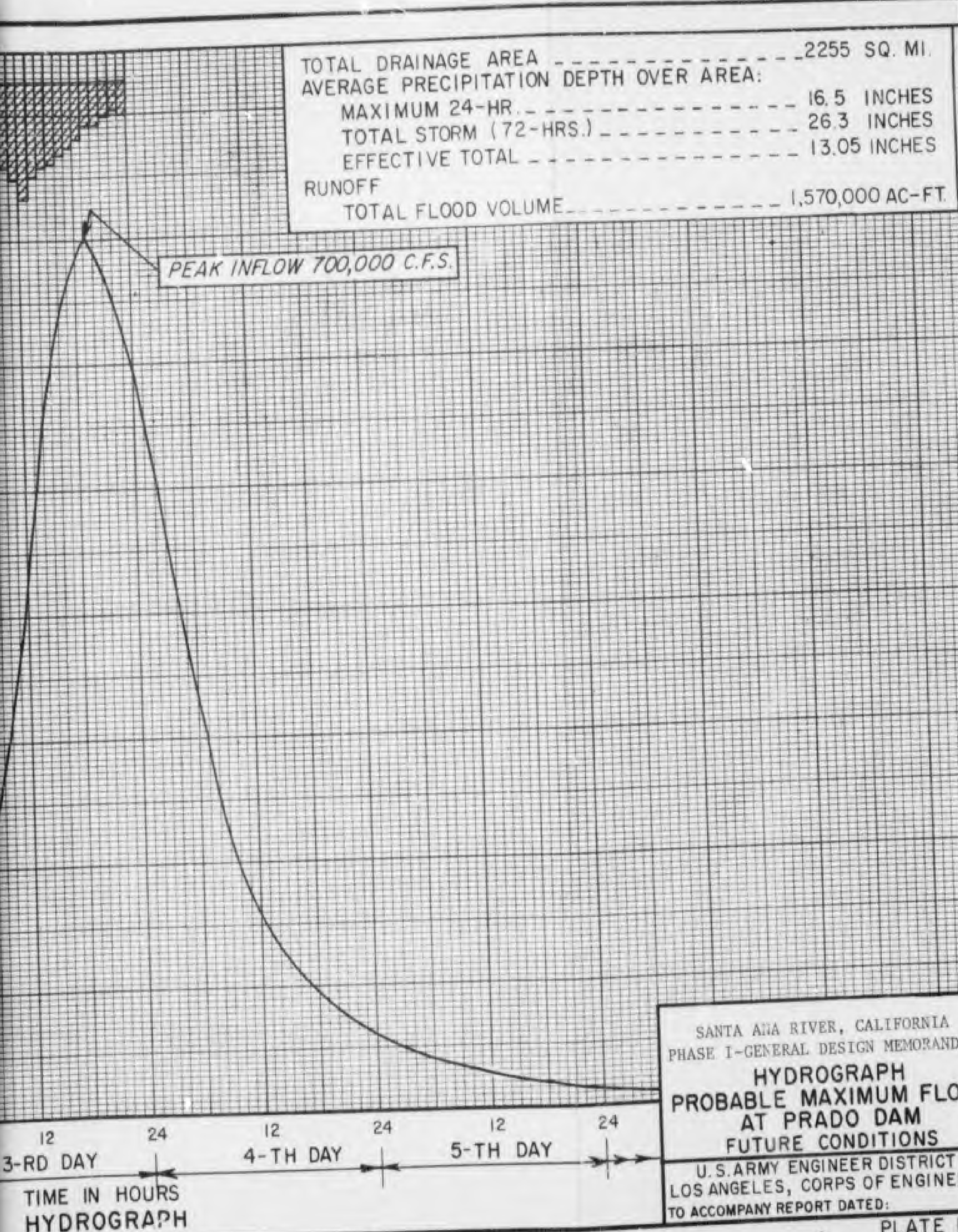
STANDARD PROJECT FLOOD
PEAK DISCHARGES
OAK STREET DRAIN
FUTURE CONDITIONS WITH RECOMMENDED PLAN

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

PLATE B-17

U.S. ARMY ENGINEER DISTRICT





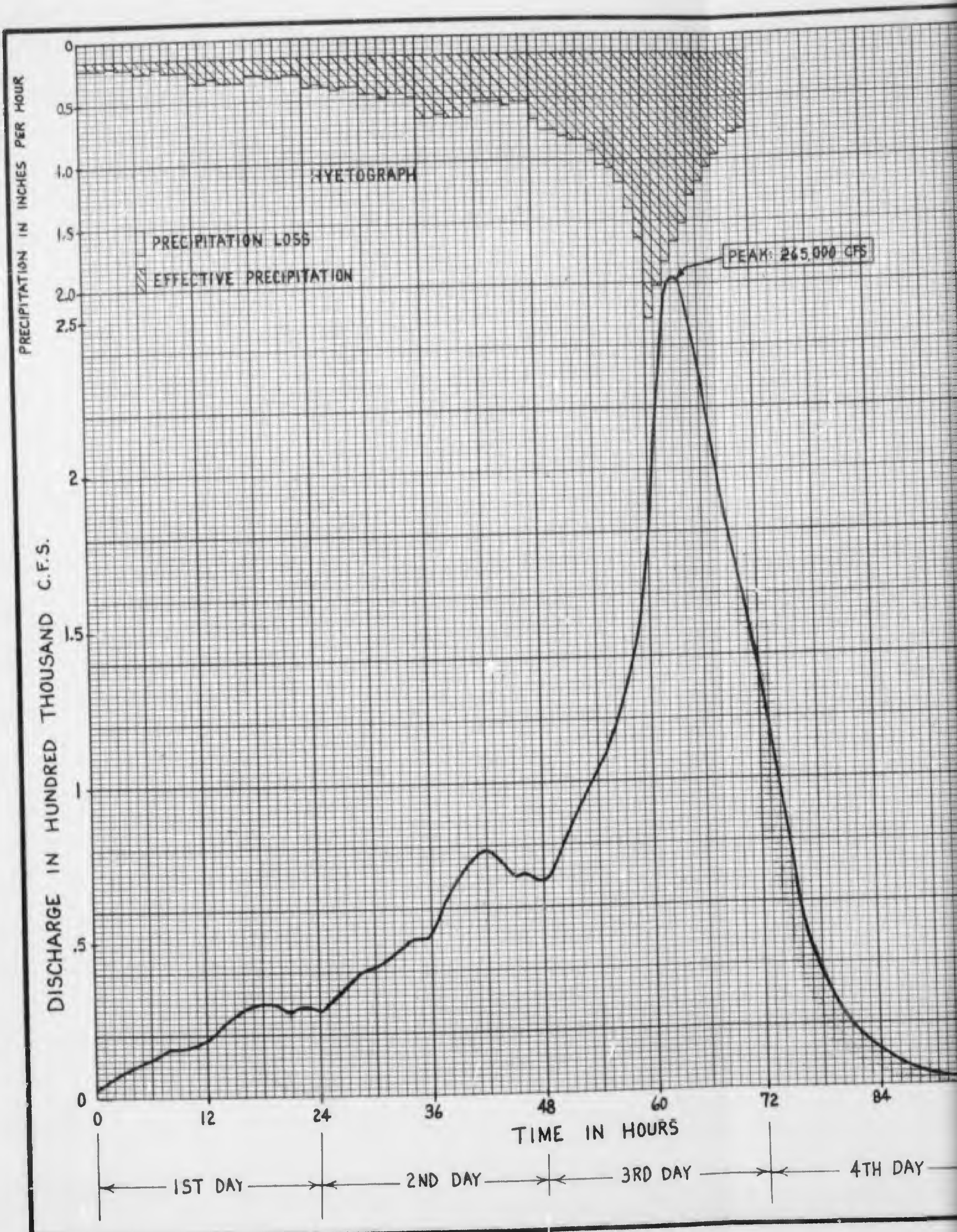
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

HYDROGRAPH
PROBABLE MAXIMUM FLOOD
AT PRADO DAM
FUTURE CONDITIONS

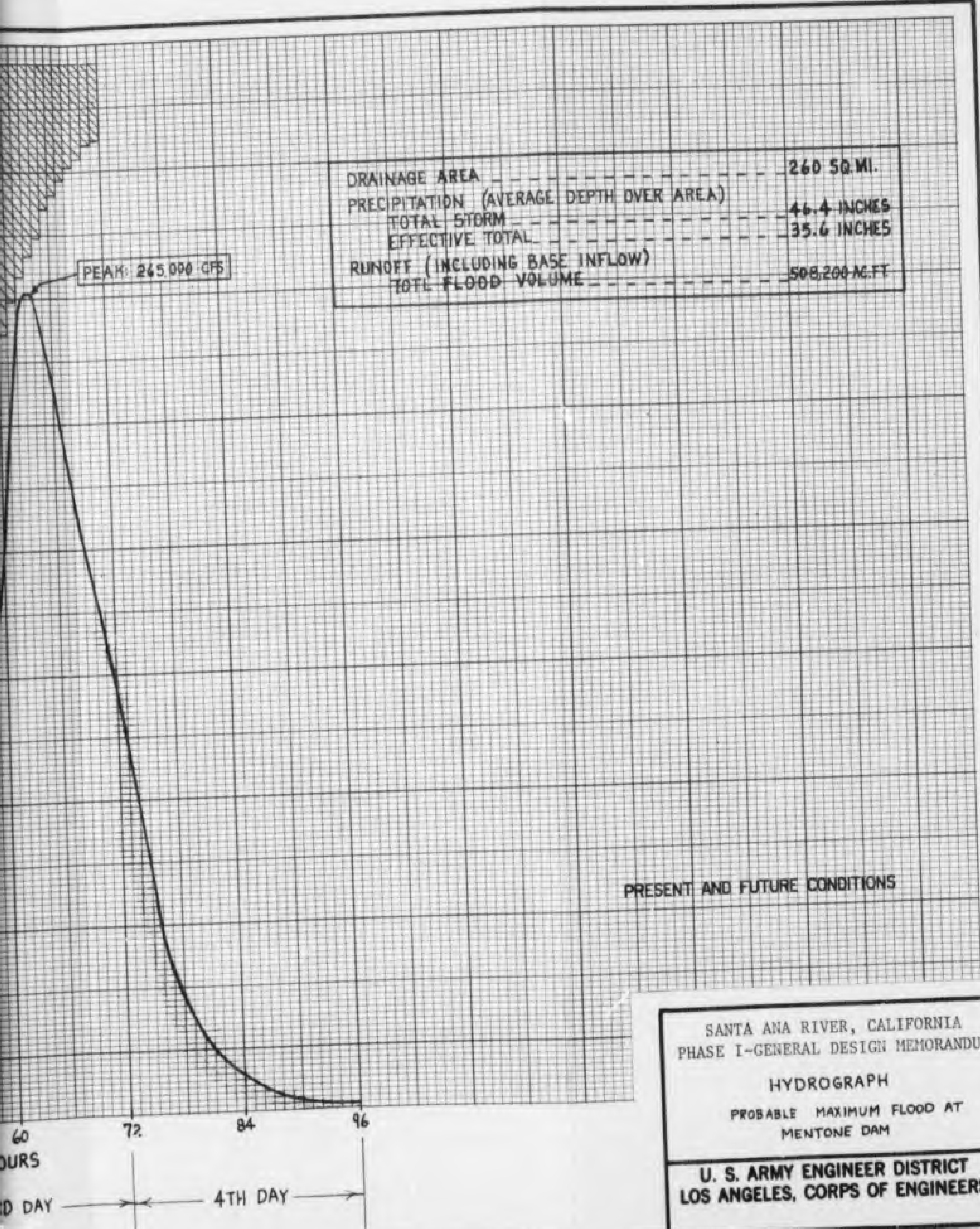
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-17a

2



PRECIPITATION IN INCHES PER HOUR
0
0.5
1.0
1.5
2.0
2.5
HYETOGRAPH
PRECIPITATION LOSS
EFFECTIVE PRECIPITATION
PEAK: 265,000 CFS
DISCHARGE IN HUNDRED THOUSAND C.F.S.
2
1.5
1
.5
0
0
12
24
36
48
60
72
84
TIME IN HOURS
1ST DAY
2ND DAY
3RD DAY
4TH DAY



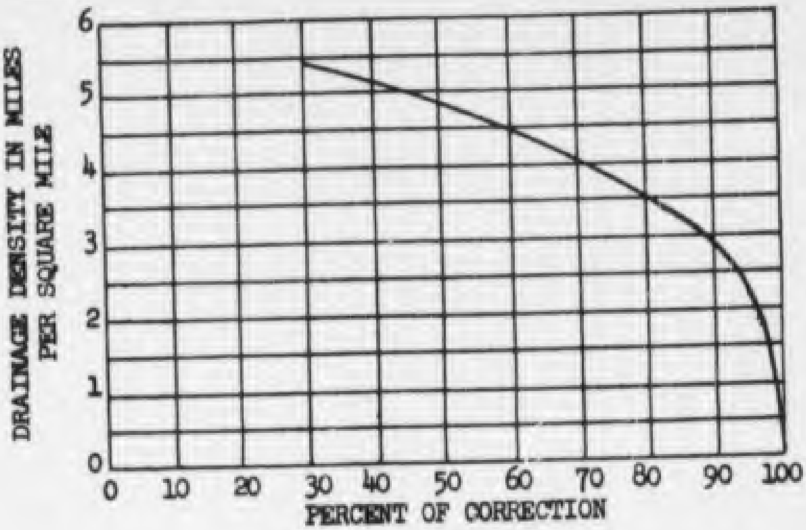
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

HYDROGRAPH

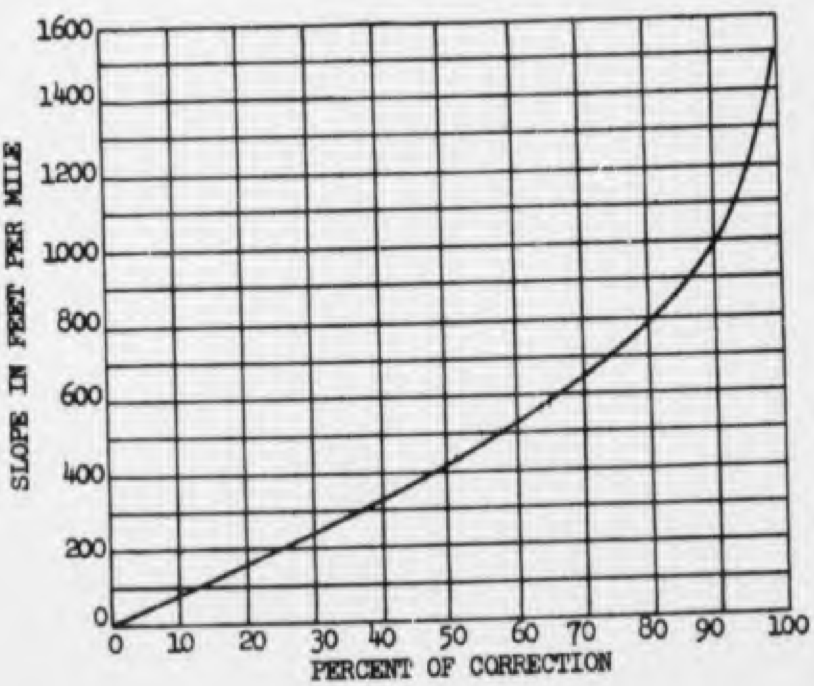
PROBABLE MAXIMUM FLOOD AT MENTONE DAM

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

PLATE B-17b



DRAINAGE DENSITY



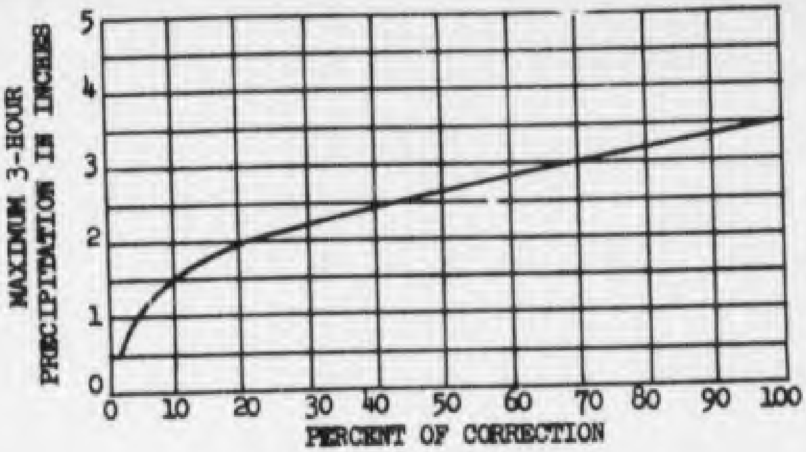
SLOPE

Adopted from plate 6 of "A New Method of Estimating Debris-Storage Requirements for Debris Basins," by Fred E. Tatum

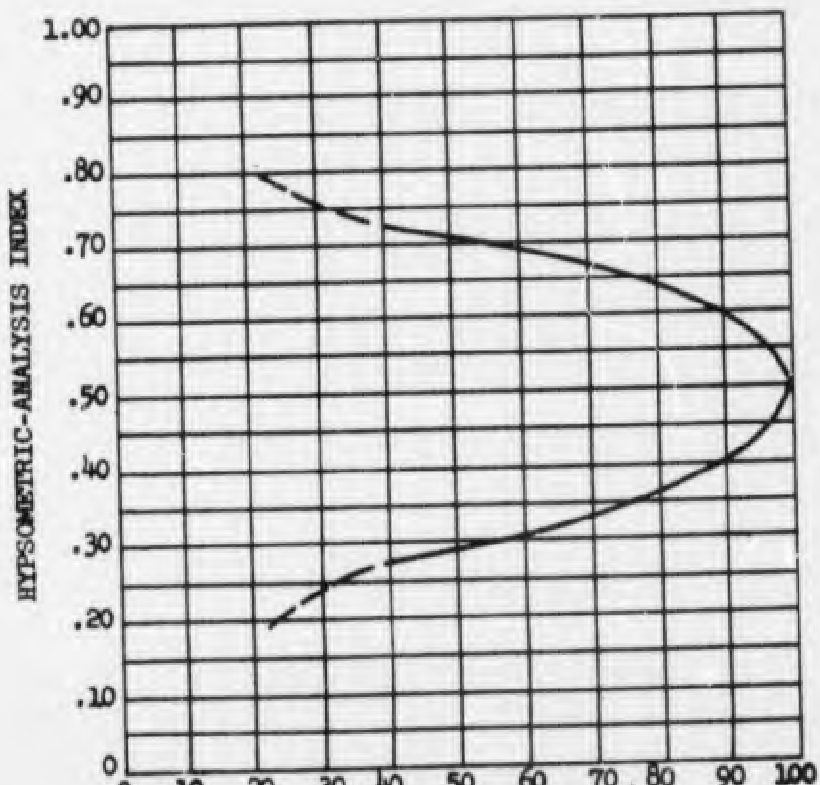
SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

DRAINAGE DENSITY AND SLOPE CORRECTION CURVES OF DEBRIS PRODUCTION

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



MAXIMUM 3-HOUR PRECIPITATION



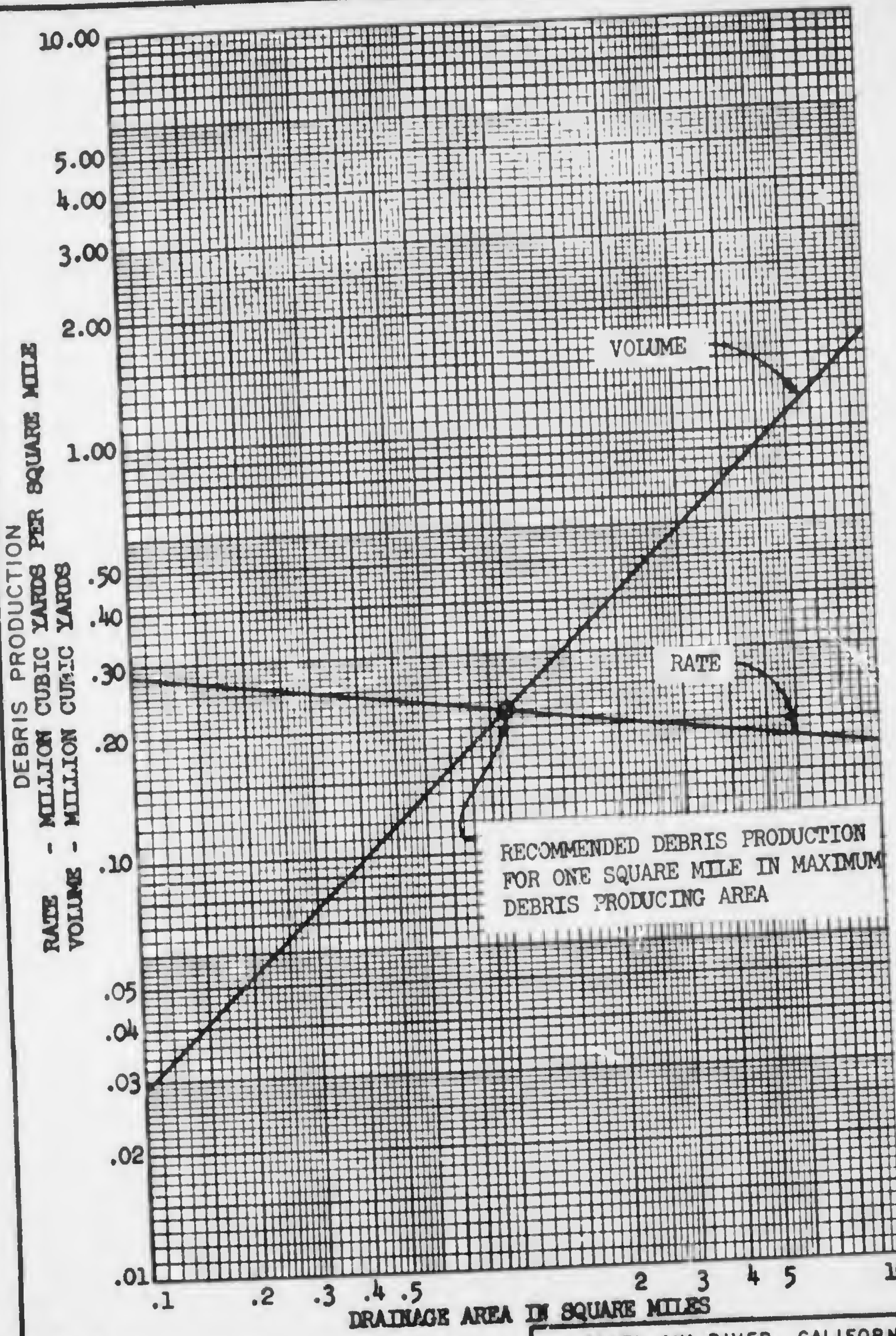
HYPSOMETRIC-ANALYSIS INDEX

Adopted from plate 7 of "A New Method of Estimating Debris-Storage Requirements for Debris Basins," by Fred E. Tatum.

SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

MAXIMUM 3-HOUR PRECIPITATION AND HYPSOMETRIC ANALYSIS INDEX CORRECTION CURVES OF DEBRIS PRODUCTION

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



NOTE:

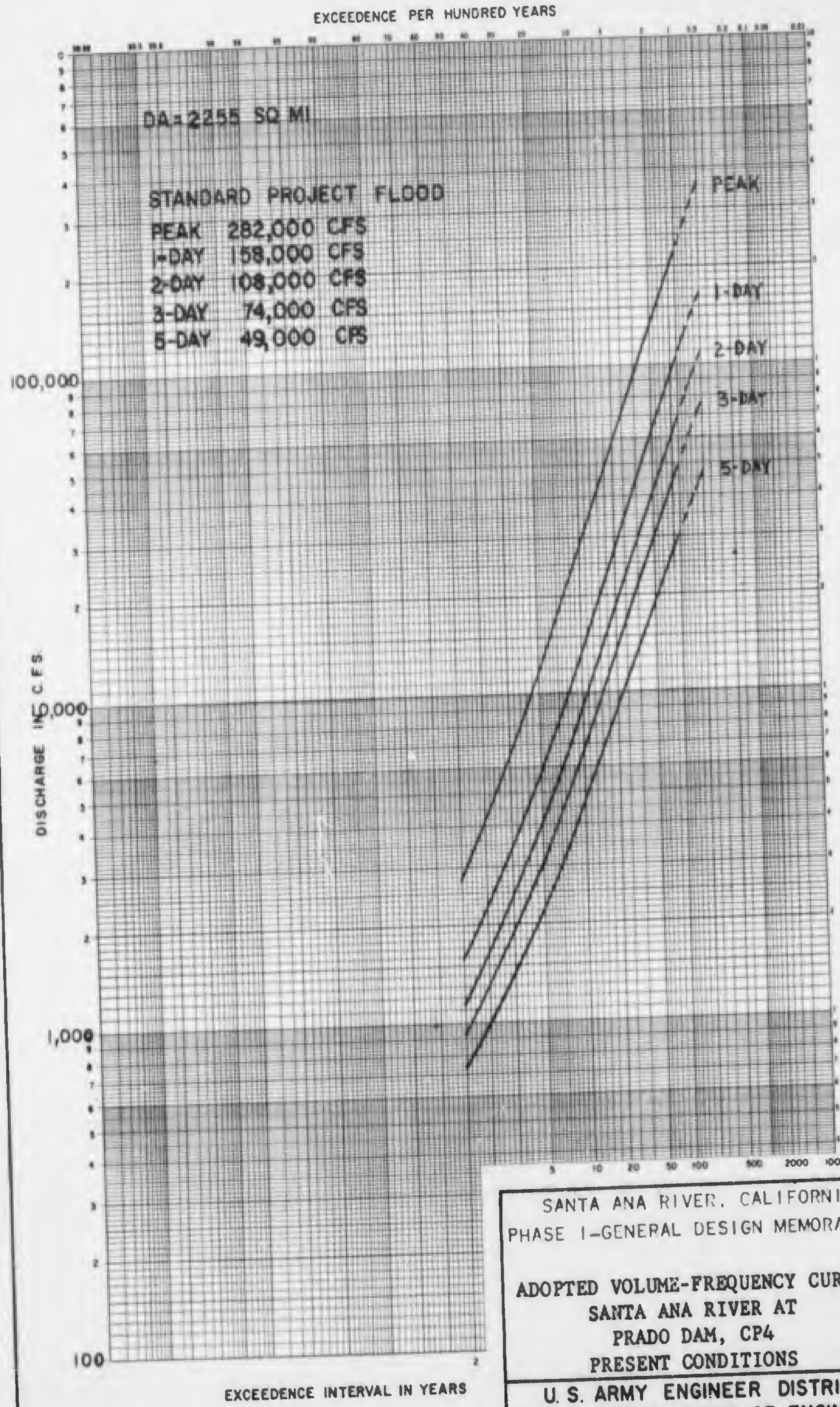
CURVES SHOWN HEREON ARE BASED ON THE ASSUMPTION THAT THE DEBRIS PRODUCING STORM OCCURS 4 TO 5 YEARS AFTER 100 PERCENT BURN IN THE AREA.

Adopted from plate 10 of "A New Method of Estimating Debris-Storage Requirements for Debris Basins," by Fred E. Tatum.

SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

RECOMMENDED DEBRIS PRODUCTION FROM MAXIMUM DEBRIS PRODUCING AREA FOR ONE MAJOR STORM

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

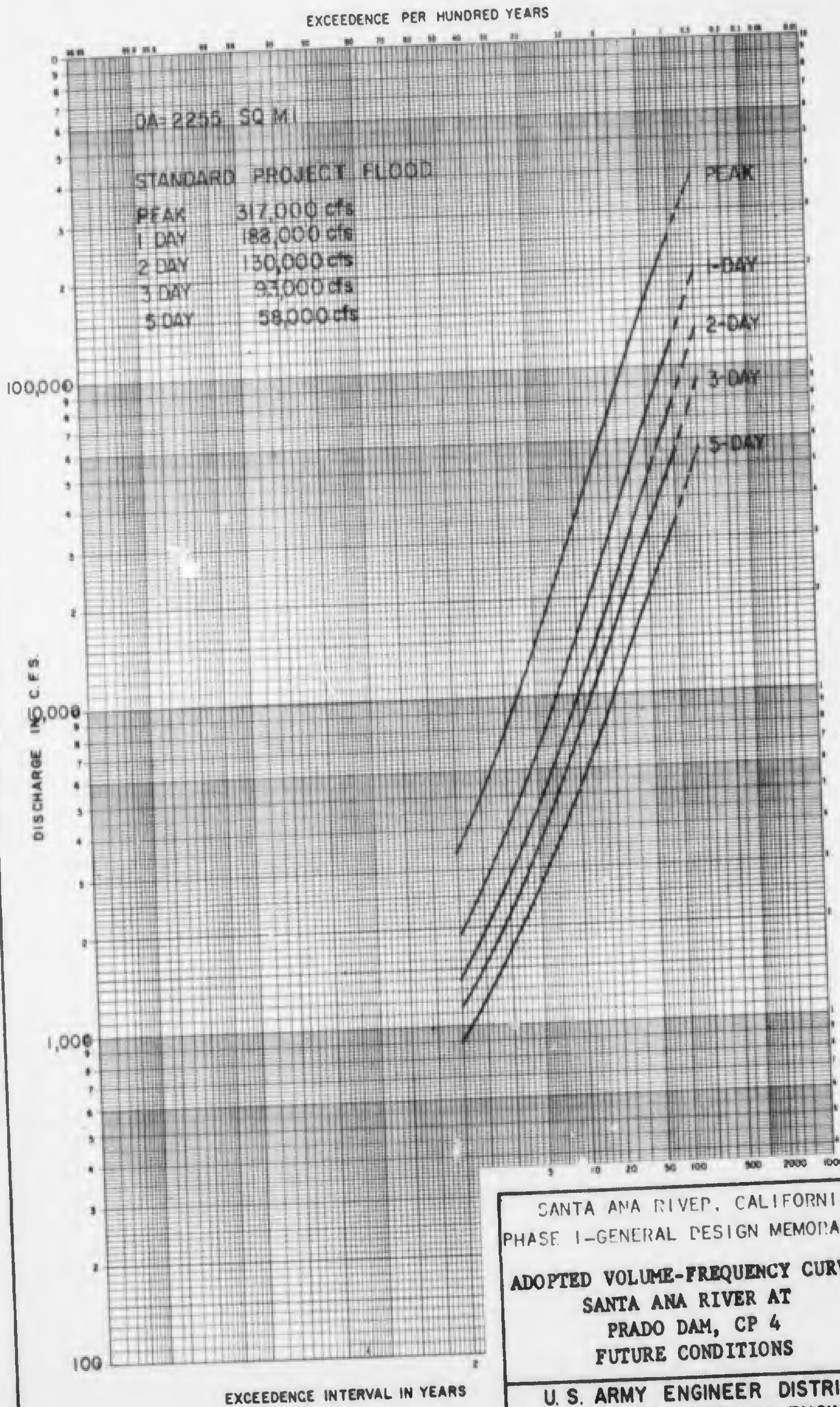


SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

ADOPTED VOLUME-FREQUENCY CURVES
SANTA ANA RIVER AT
PRADO DAM, CP4
PRESENT CONDITIONS

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

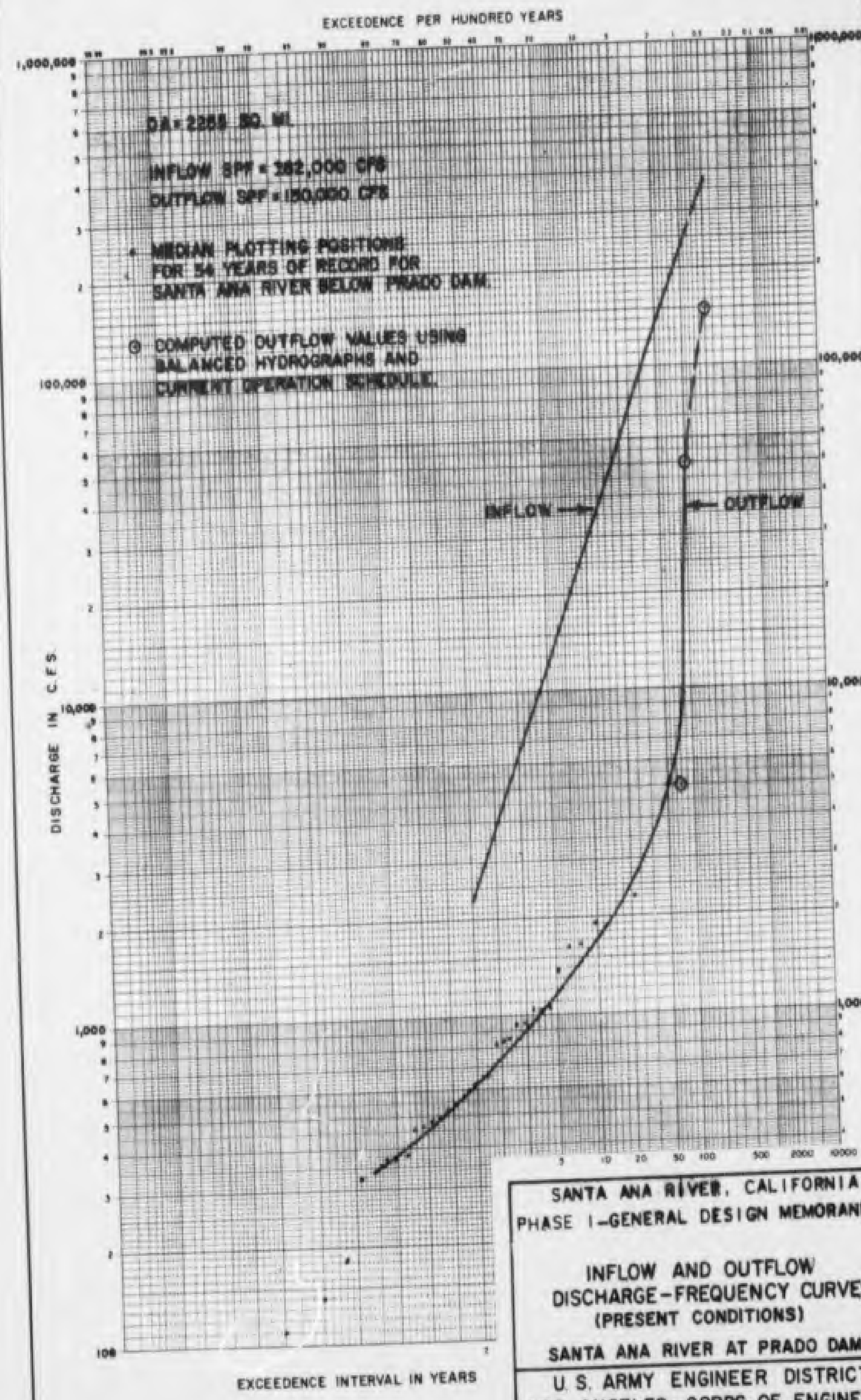
PLATE B-21



SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

ADOPTED VOLUME-FREQUENCY CURVES
SANTA ANA RIVER AT
PRADO DAM, CP 4
FUTURE CONDITIONS

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



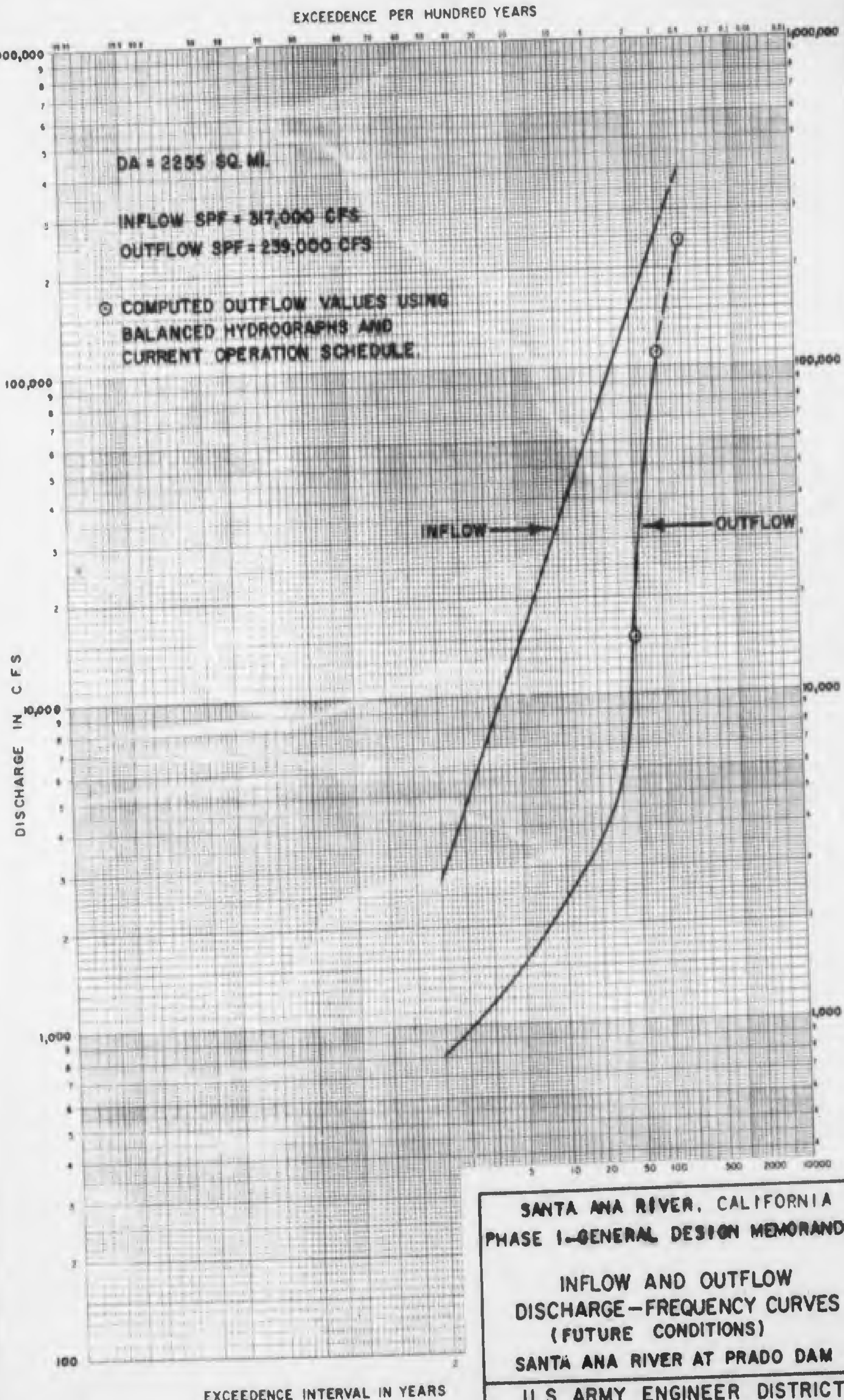
SANTA ANA RIVER, CALIFORNIA
PHASE I–GENERAL DESIGN MEMORANDUM

INFLOW AND OUTFLOW
DISCHARGE–FREQUENCY CURVES
(PRESENT CONDITIONS)

SANTA ANA RIVER AT PRADO DAM

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

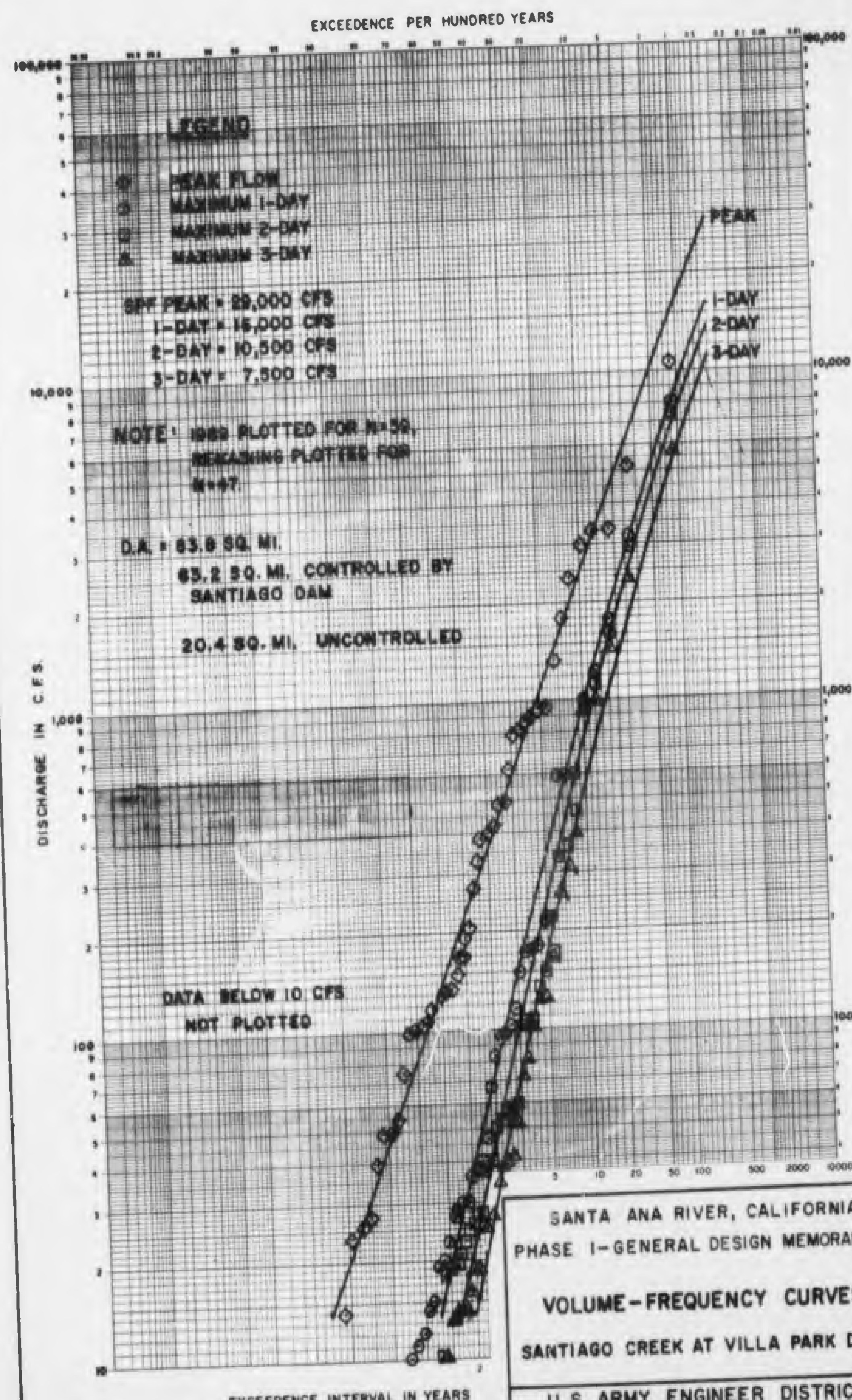
PLATE B-23



SANTA ANA RIVER, CALIFORNIA
PHASE I–GENERAL DESIGN MEMORANDUM

INFLOW AND OUTFLOW
DISCHARGE–FREQUENCY CURVES
(FUTURE CONDITIONS)
SANTA ANA RIVER AT PRADO DAM

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



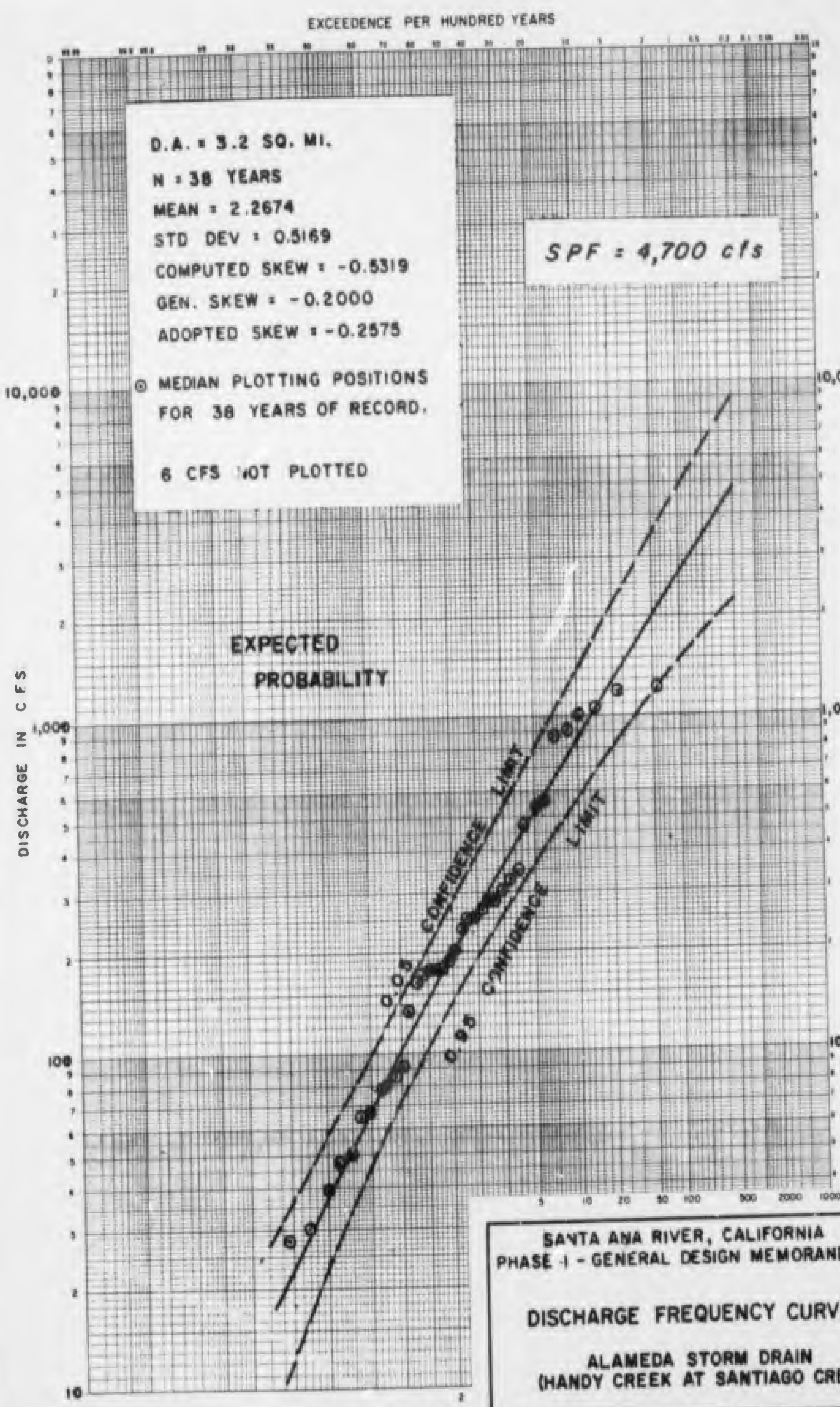
SANTA ANA RIVER, CALIFORNIA

PHASE I-GENERAL DESIGN MEMORANDUM

VOLUME-FREQUENCY CURVES

SANTIAGO CREEK AT VILLA PARK DAM

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



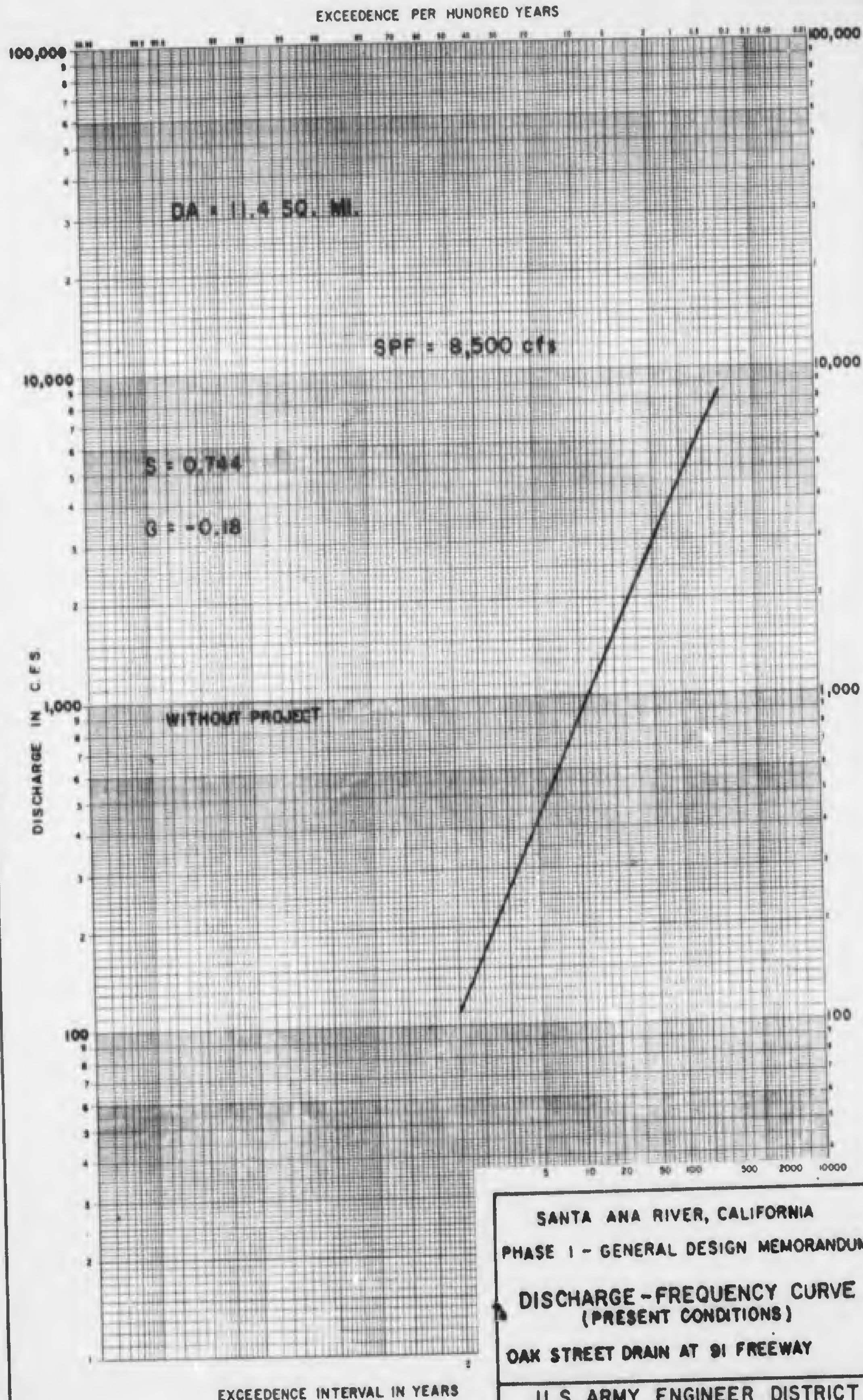
SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

DISCHARGE FREQUENCY CURVE

ALAMEDA STORM DRAIN
(HANDY CREEK AT SANTIAGO CREEK)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED

PLATE 26



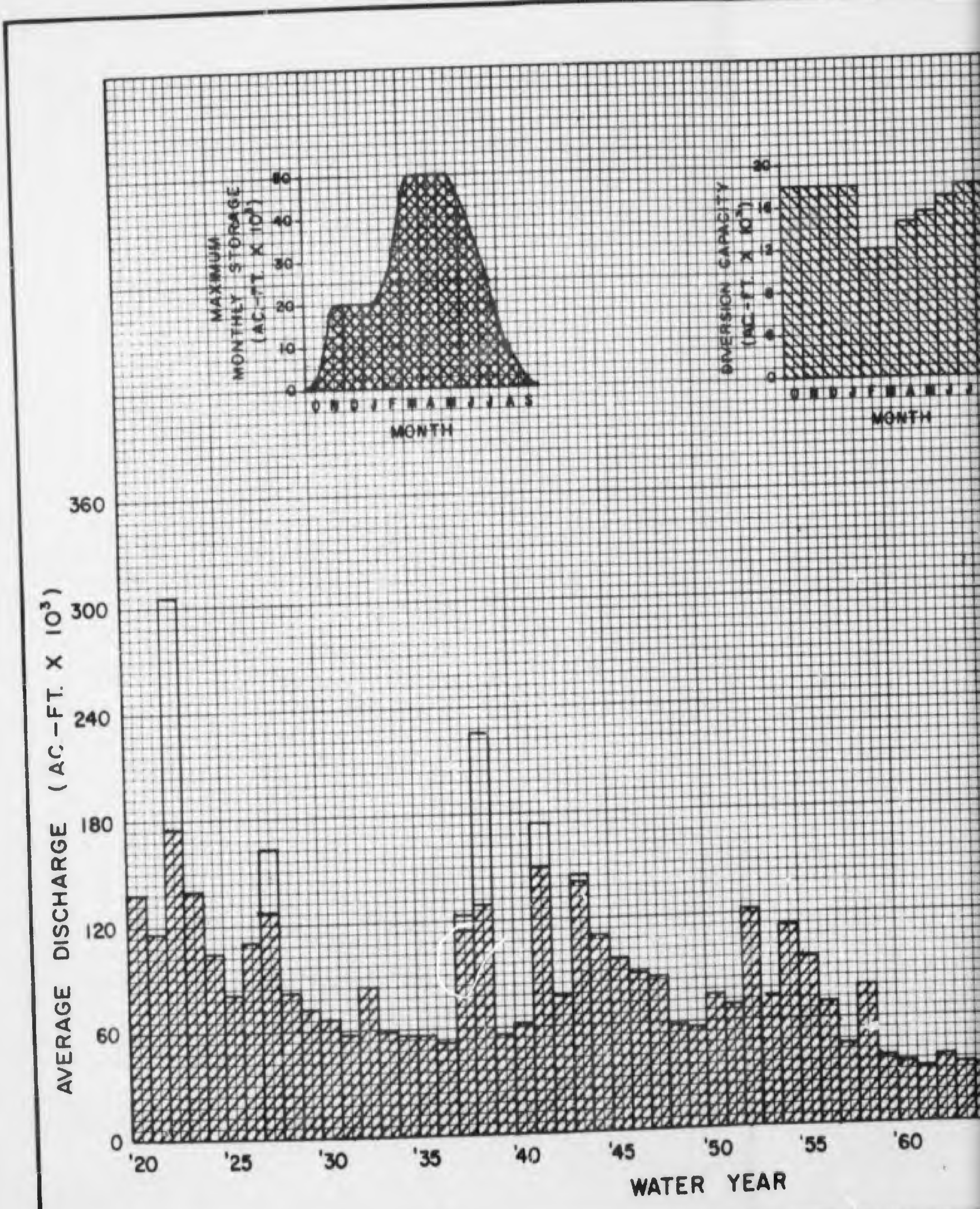
SANTA ANA RIVER, CALIFORNIA

PHASE I - GENERAL DESIGN MEMORANDUM

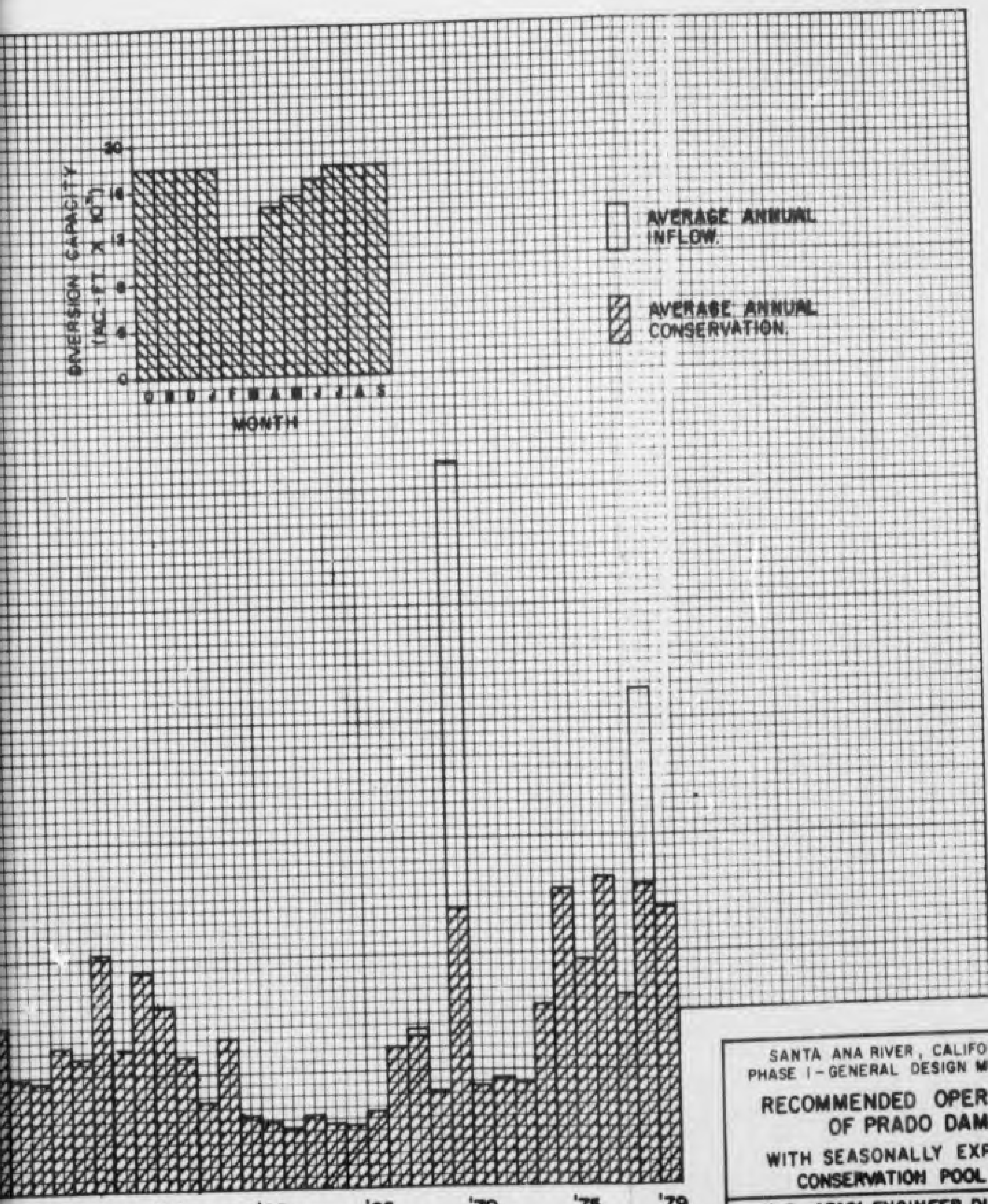
DISCHARGE-FREQUENCY CURVE
(PRESENT CONDITIONS)

OAK STREET DRAIN AT 91 FREEWAY

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



MAXIMUM MONTHLY STORAGE (AC.-FT. X 10^3)
0
10
20
30
40
50
O N D J F M A M J J A S
MONTH
DIVERSION CAPACITY (AC.-FT. X 10^3)
0
4
8
12
16
20
O N D J F M A M J J
MONTH
AVERAGE DISCHARGE (AC.-FT. X 10^3)
0
60
120
180
240
300
360
'20
'25
'30
'35
'40
'45
'50
'55
'60
WATER YEAR



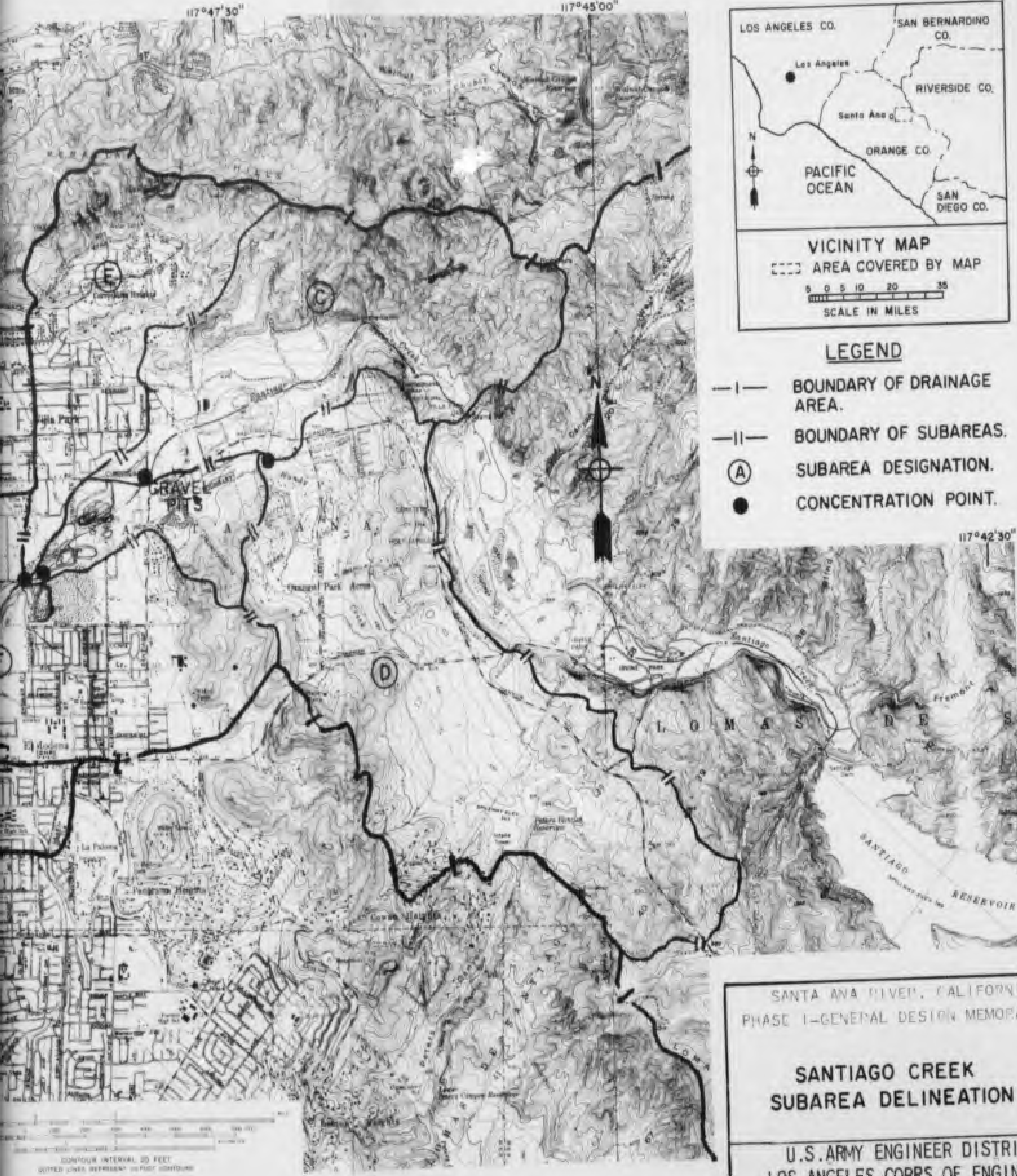
SANTA ANA RIVER, CALIFORNIA
PHASE I – GENERAL DESIGN MEMORANDUM

RECOMMENDED OPERATION OF PRADO DAM WITH SEASONALLY EXPANDED CONSERVATION POOL

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

PLATE B - 28


117°52'30"
117°50'00"
33°50'00"
33°47'30"
SANTA
ANA
RIVER
Olive
Villa Park
ORANGE
El Modena
La Paloma
PARK
SANTA ANA
E
F
G
H



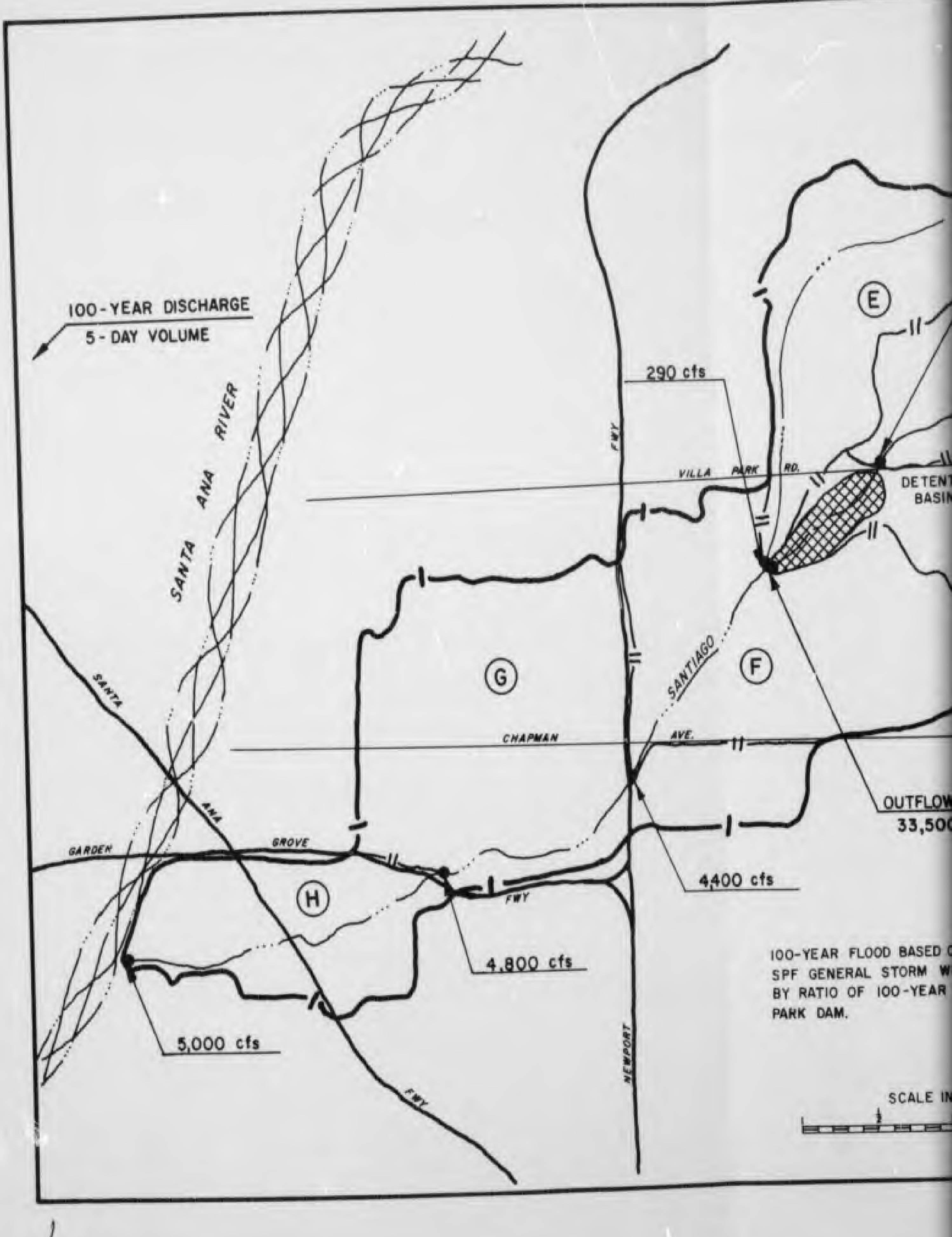
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

SANTIAGO CREEK SUBAREA DELINEATION

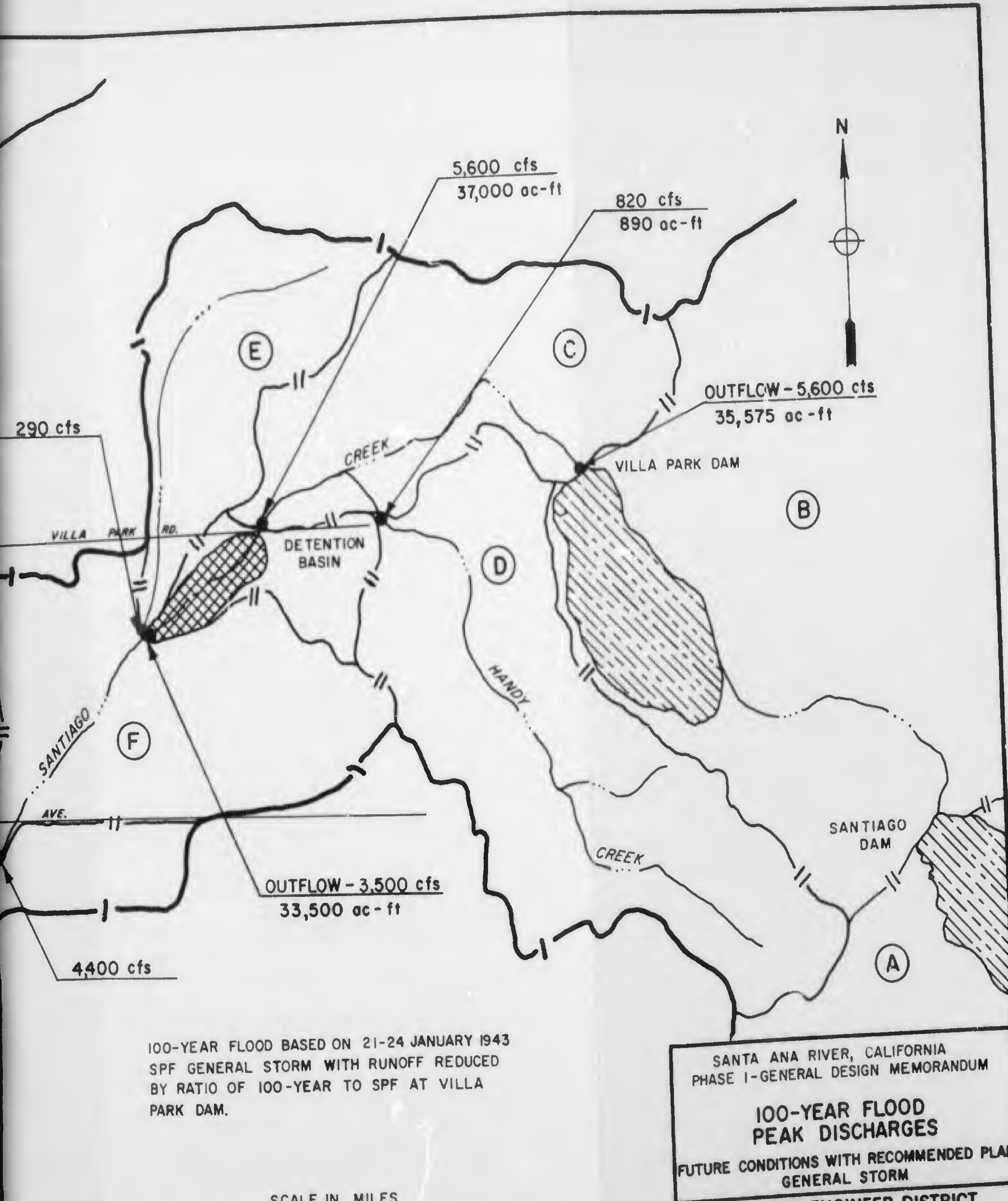
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED

PLATE B-29

2



100-YEAR DISCHARGE
5-DAY VOLUME
SANTA ANA RIVER
290 cfs
FWY
VILLA PARK RD.
DETENT
BASIN
E
G
F
SANTIAGO
SANTA ANA FWY
CHAPMAN AVE.
OUTFLOW
33,500
GARDEN GROVE FWY
H
4,400 cfs
4,800 cfs
5,000 cfs
NEWPORT
100-YEAR FLOOD BASED O
SPF GENERAL STORM W
BY RATIO OF 100-YEAR
PARK DAM.
SCALE IN



100-YEAR FLOOD BASED ON 21-24 JANUARY 1943 SPF GENERAL STORM WITH RUNOFF REDUCED BY RATIO OF 100-YEAR TO SPF AT VILLA PARK DAM.

SCALE IN MILES

1 ½ 0 1

SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

100-YEAR FLOOD
PEAK DISCHARGES

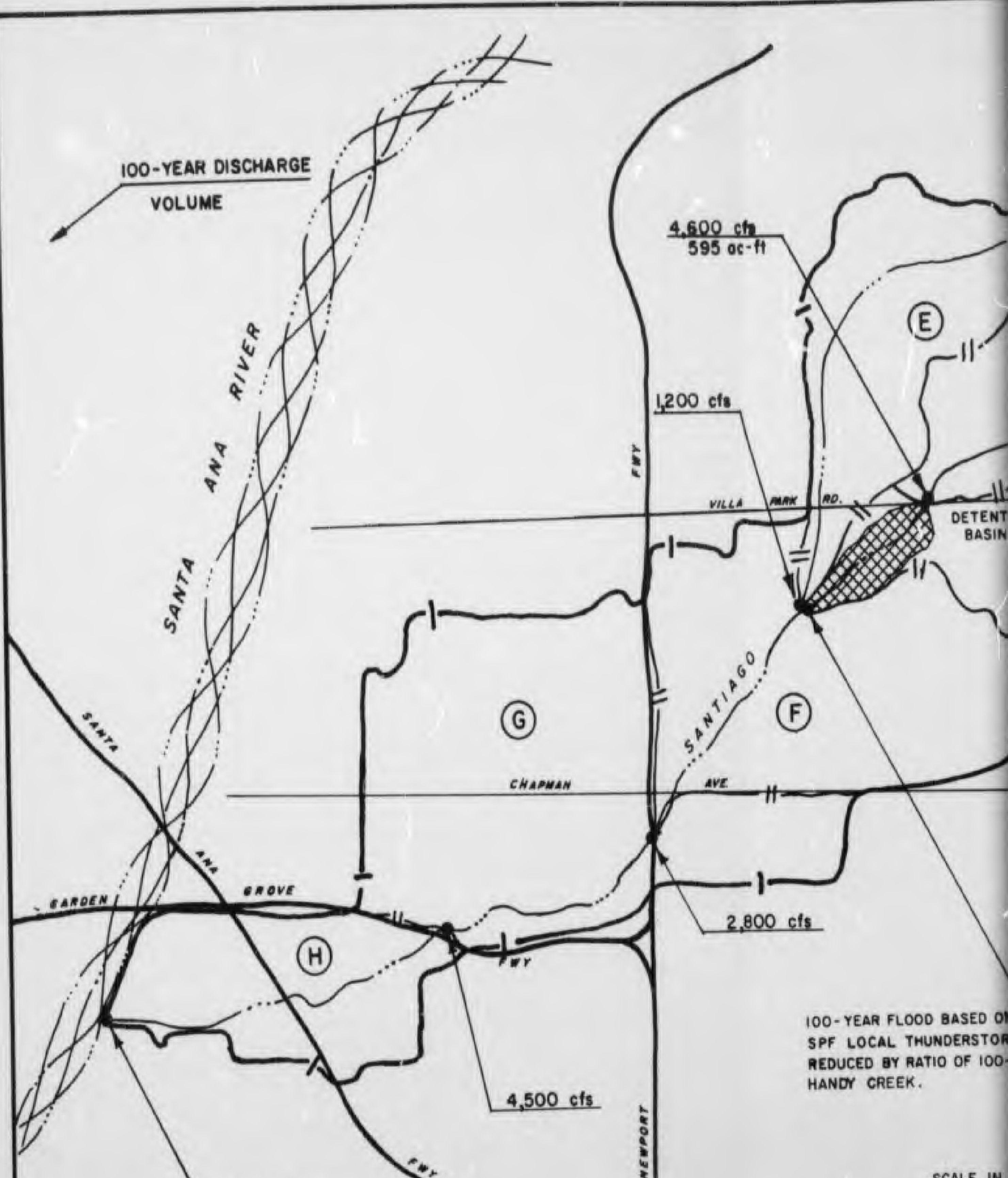
FUTURE CONDITIONS WITH RECOMMENDED PLAN
GENERAL STORM

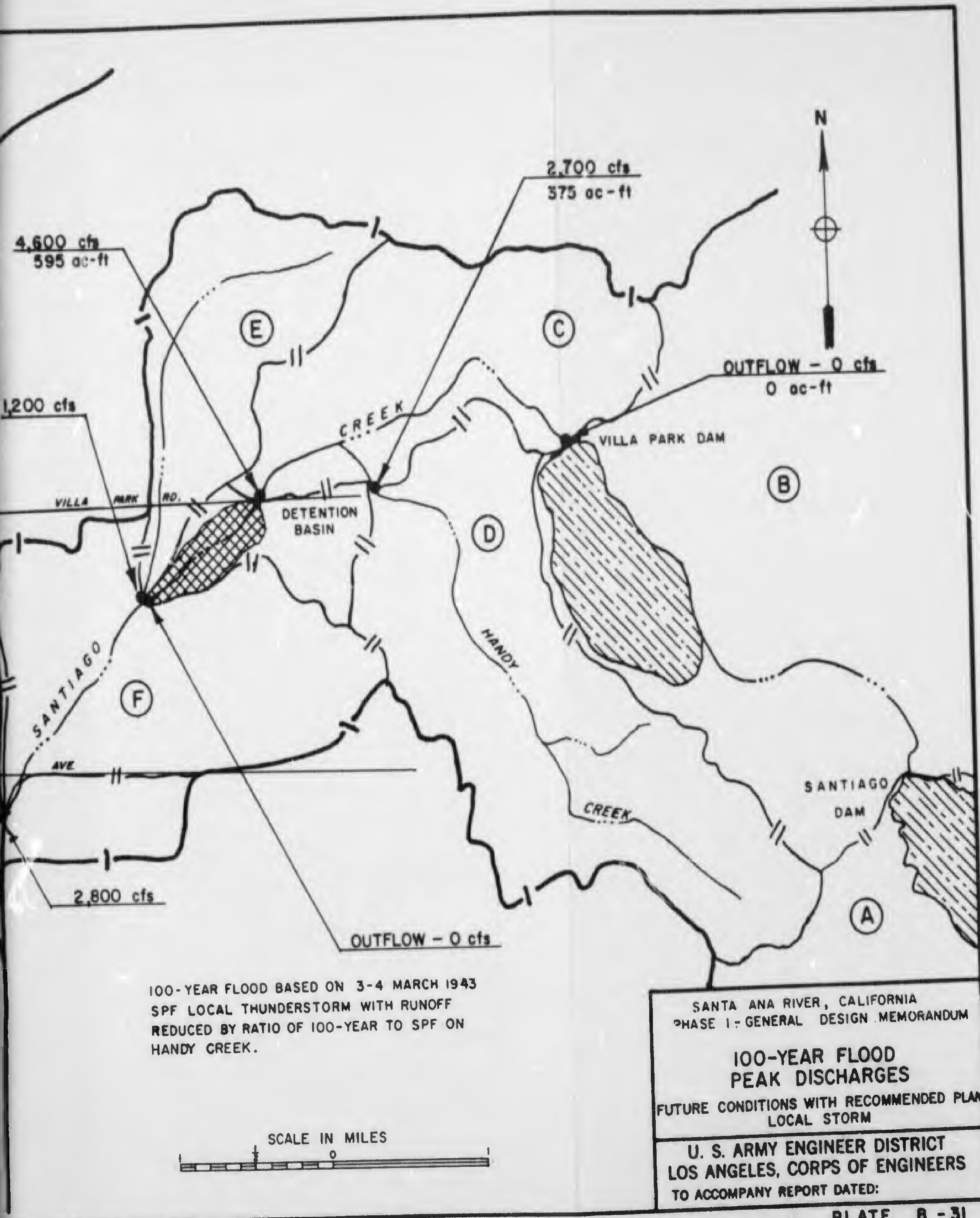
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

TO ACCOMPANY REPORT DATED:

PLATE B-30

2


100-YEAR DISCHARGE
VOLUME
SANTA ANA RIVER
4,600 cfs
595 ac-ft
1,200 cfs
FWY
VILLA PARK RD.
DETENT
BASIN
E
F
G
H
SANTIAGO
CHAPMAN AVE.
SANTA ANA
GARDEN GROVE
FWY
2,800 cfs
4,500 cfs
4,500 cfs
NEWPORT
FWY
100-YEAR FLOOD BASED O
SPF LOCAL THUNDERSTOR
REDUCED BY RATIO OF 100-
HANDY CREEK.
SCALE IN



SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

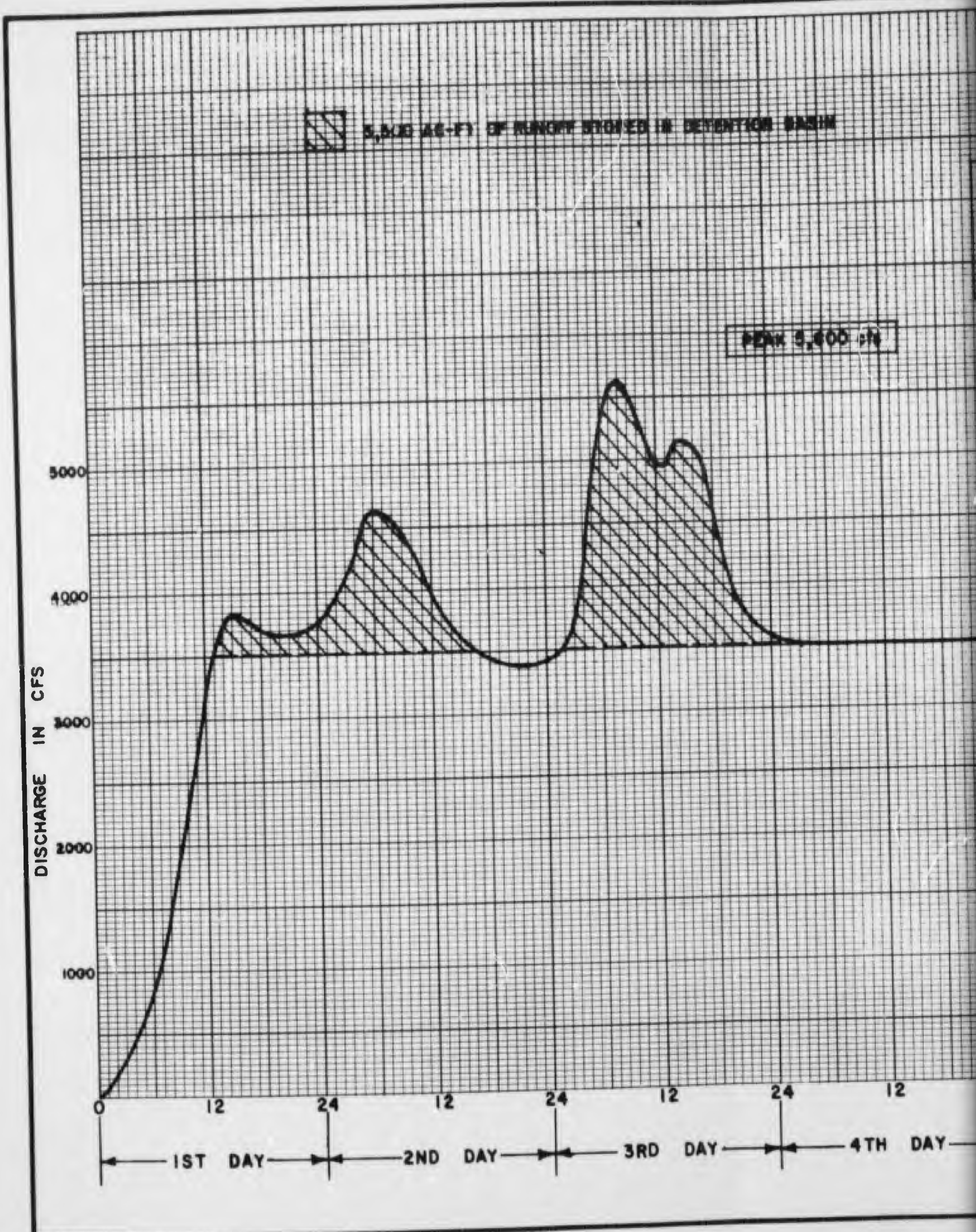
100-YEAR FLOOD
PEAK DISCHARGES

FUTURE CONDITIONS WITH RECOMMENDED PLAN
LOCAL STORM

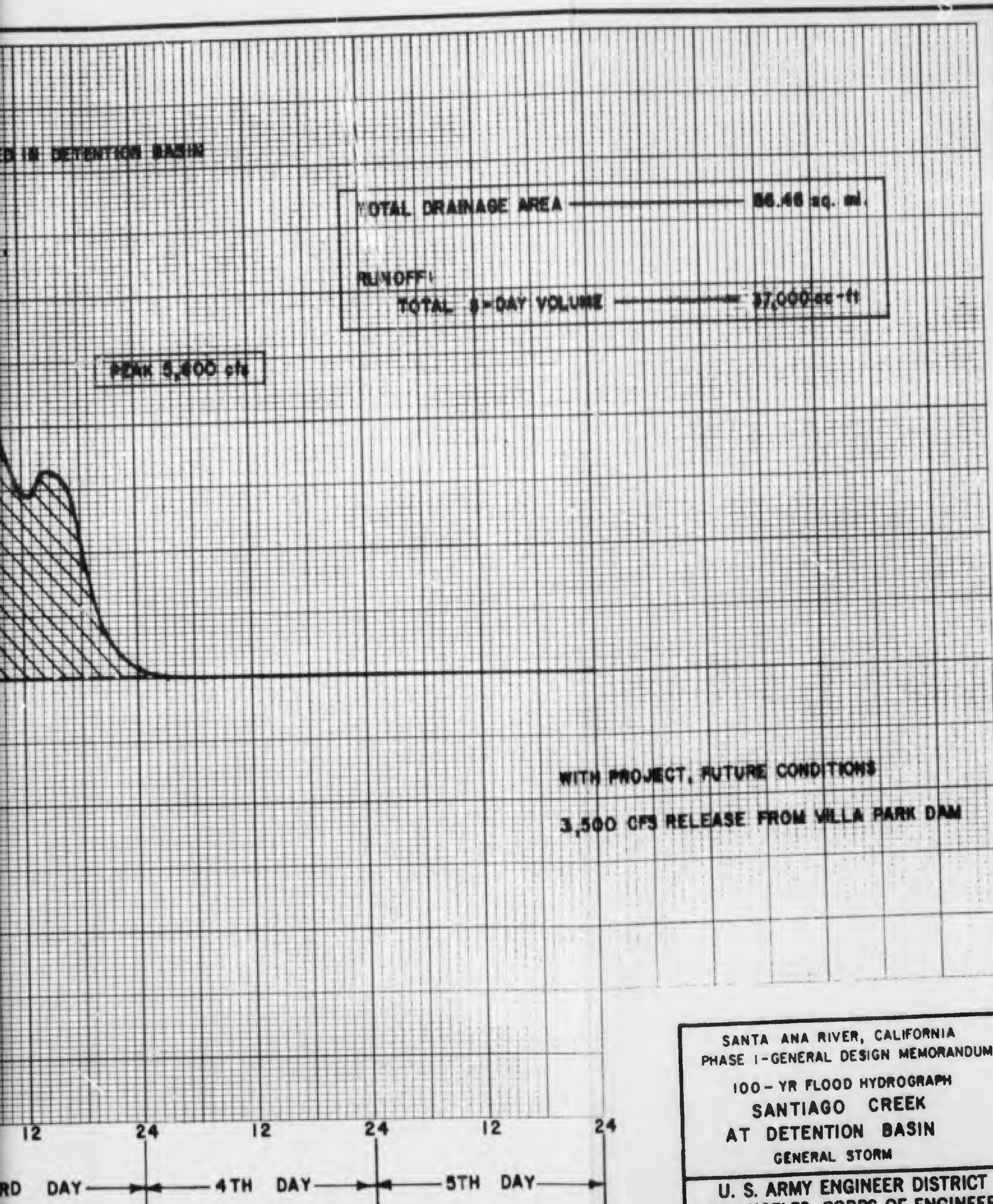
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B - 31

2



DISCHARGE IN CFS
5000
4000
3000
2000
1000
0
12
24
12
24
12
24
12
1ST DAY
2ND DAY
3RD DAY
4TH DAY



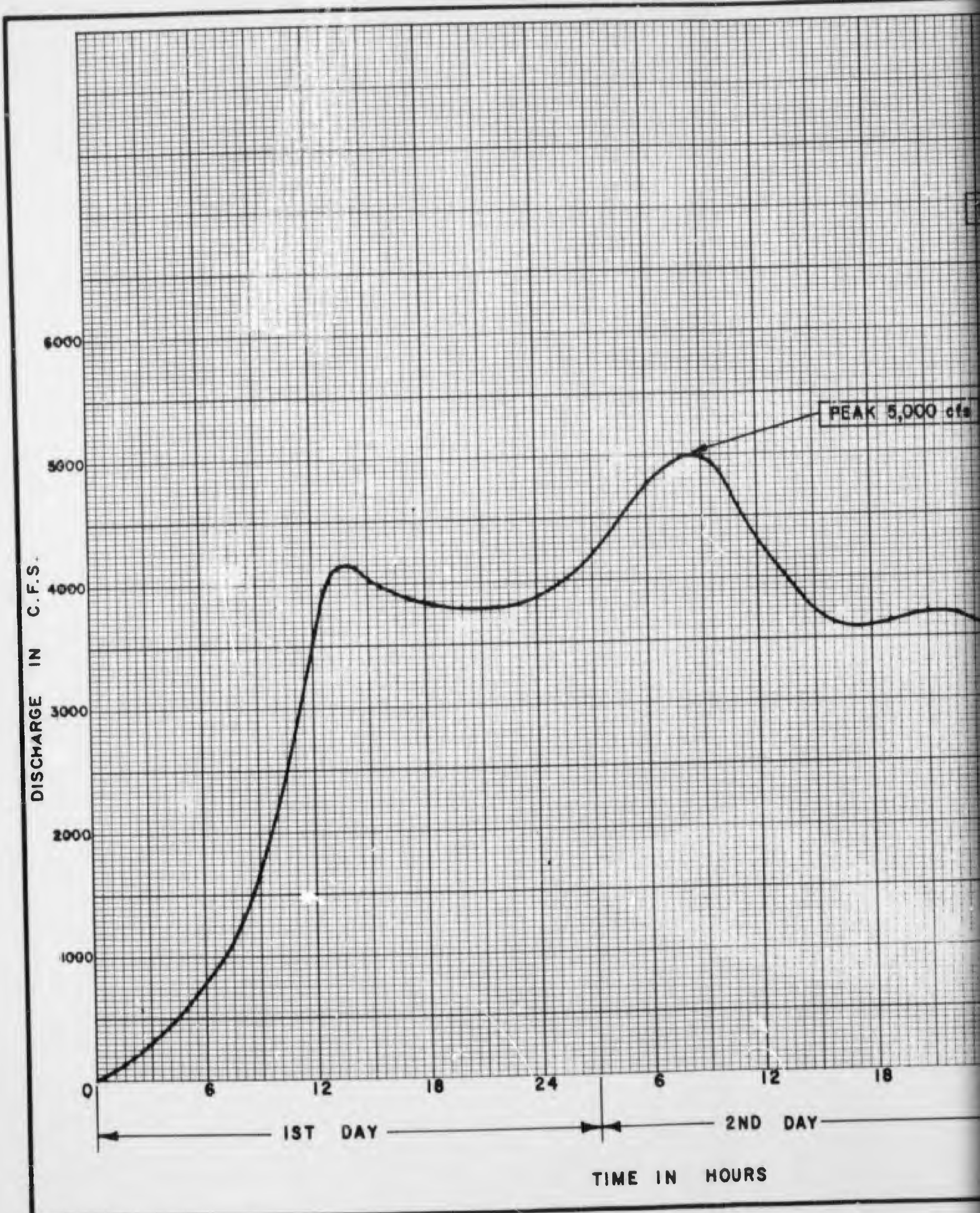
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

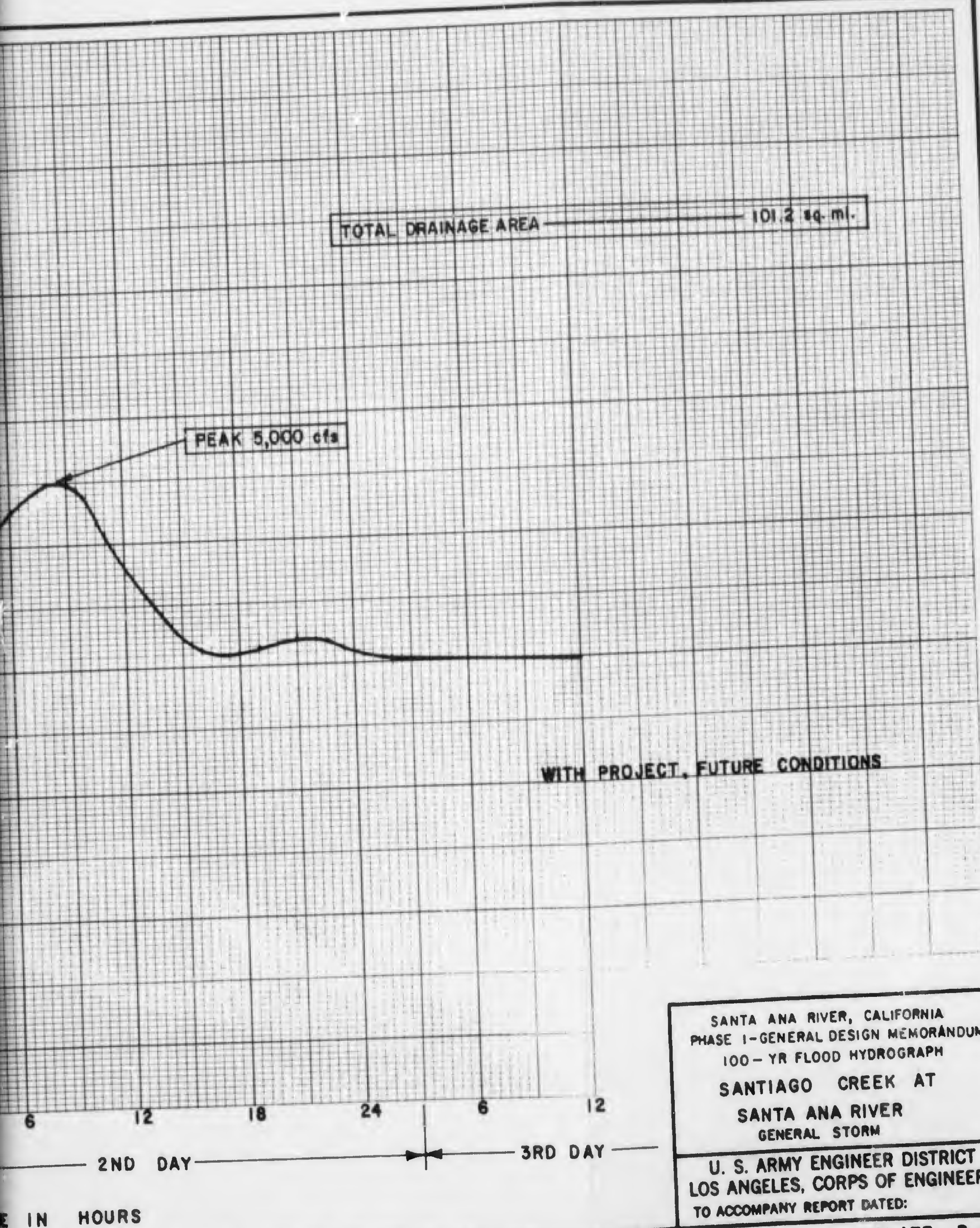
100-YR FLOOD HYDROGRAPH
SANTIAGO CREEK
AT DETENTION BASIN
GENERAL STORM

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-32

2


PEAK 5,000 cfs
6000
5000
4000
3000
2000
1000
0
DISCHARGE IN C.F.S.
6
12
18
24
6
12
18
1ST DAY
2ND DAY
TIME IN HOURS



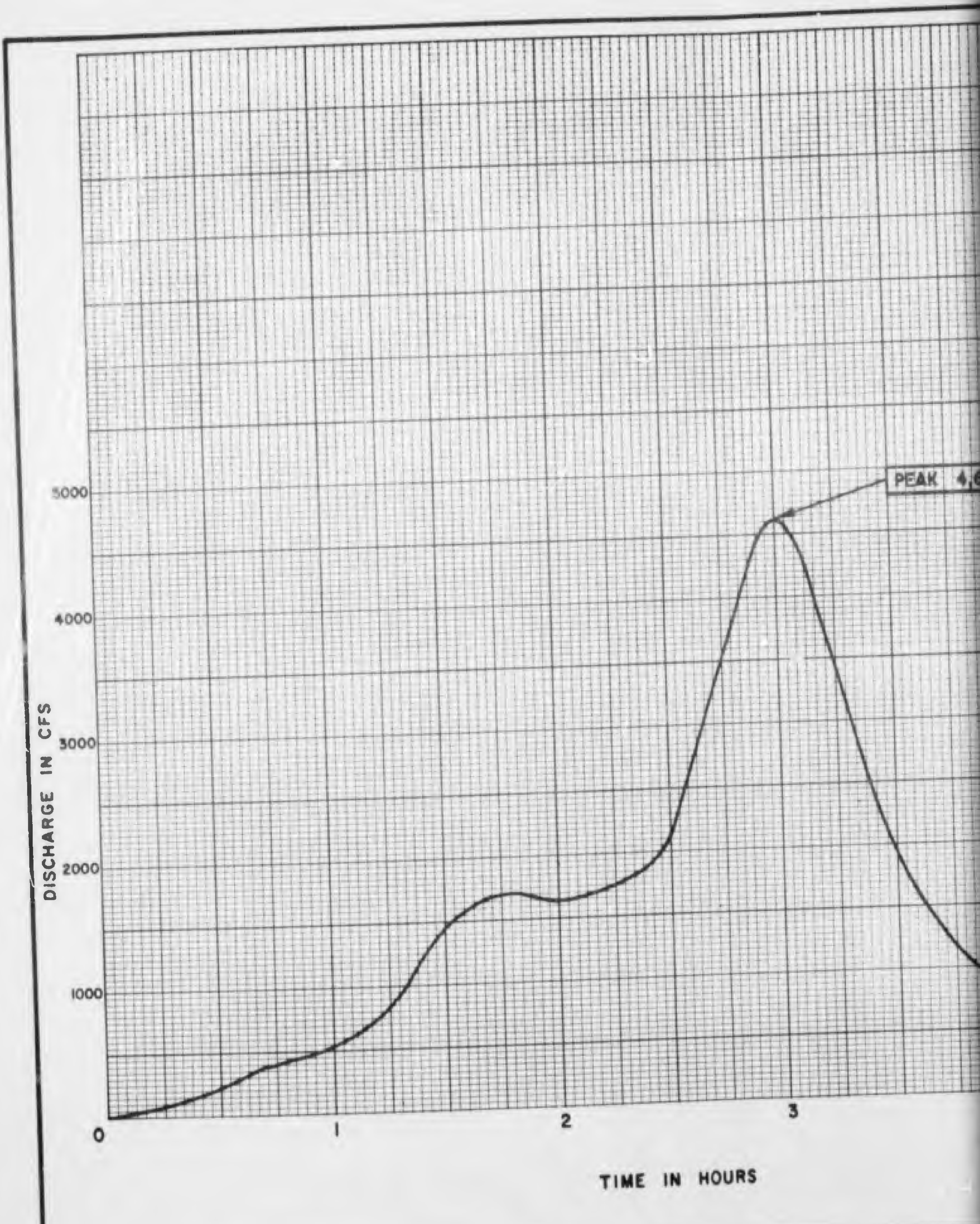
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM
100-YR FLOOD HYDROGRAPH

SANTIAGO CREEK AT
SANTA ANA RIVER
GENERAL STORM

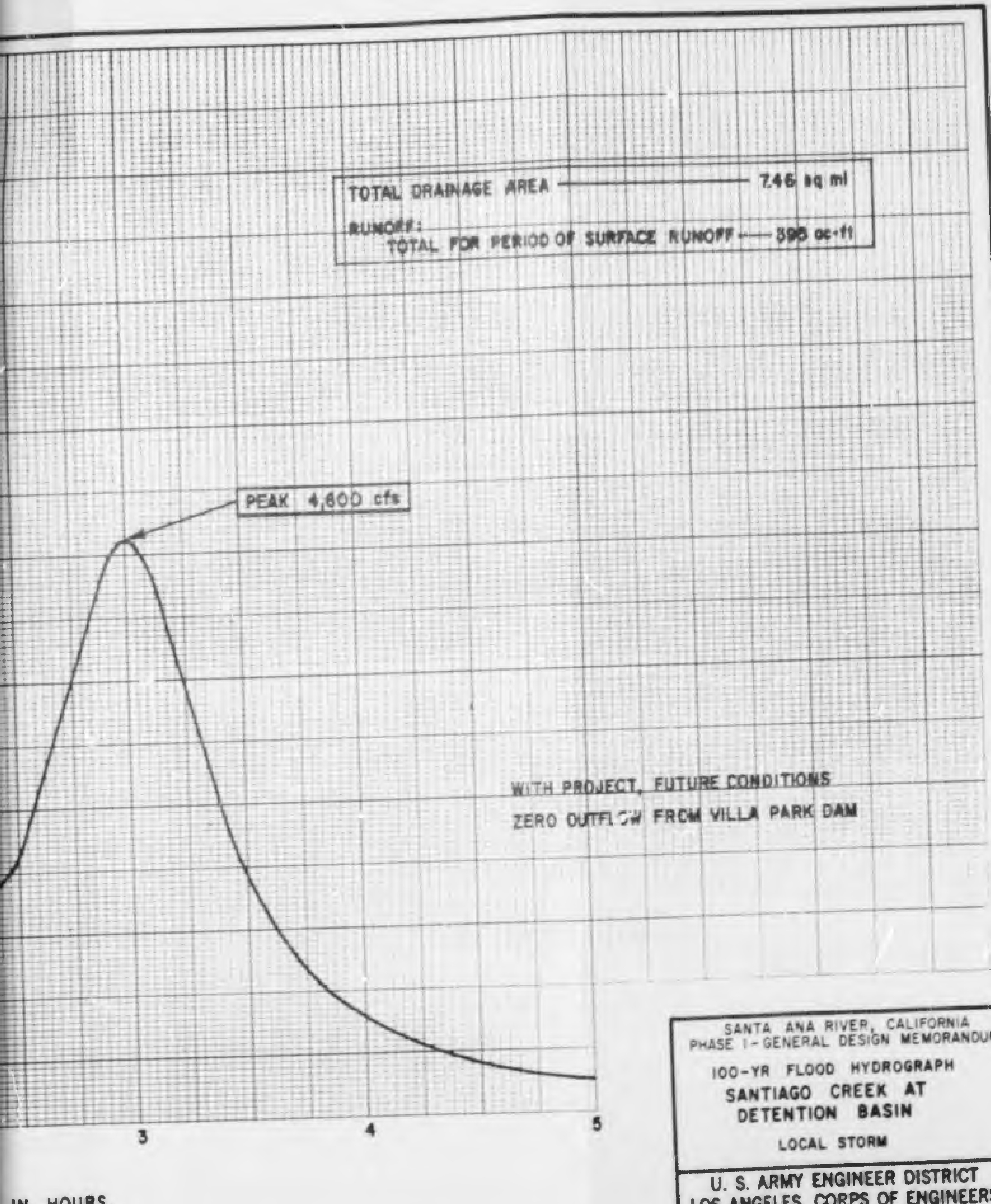
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-33

2



PEAK 4,6
5000
4000
3000
2000
1000
DISCHARGE IN CFS
0
1
2
3
TIME IN HOURS



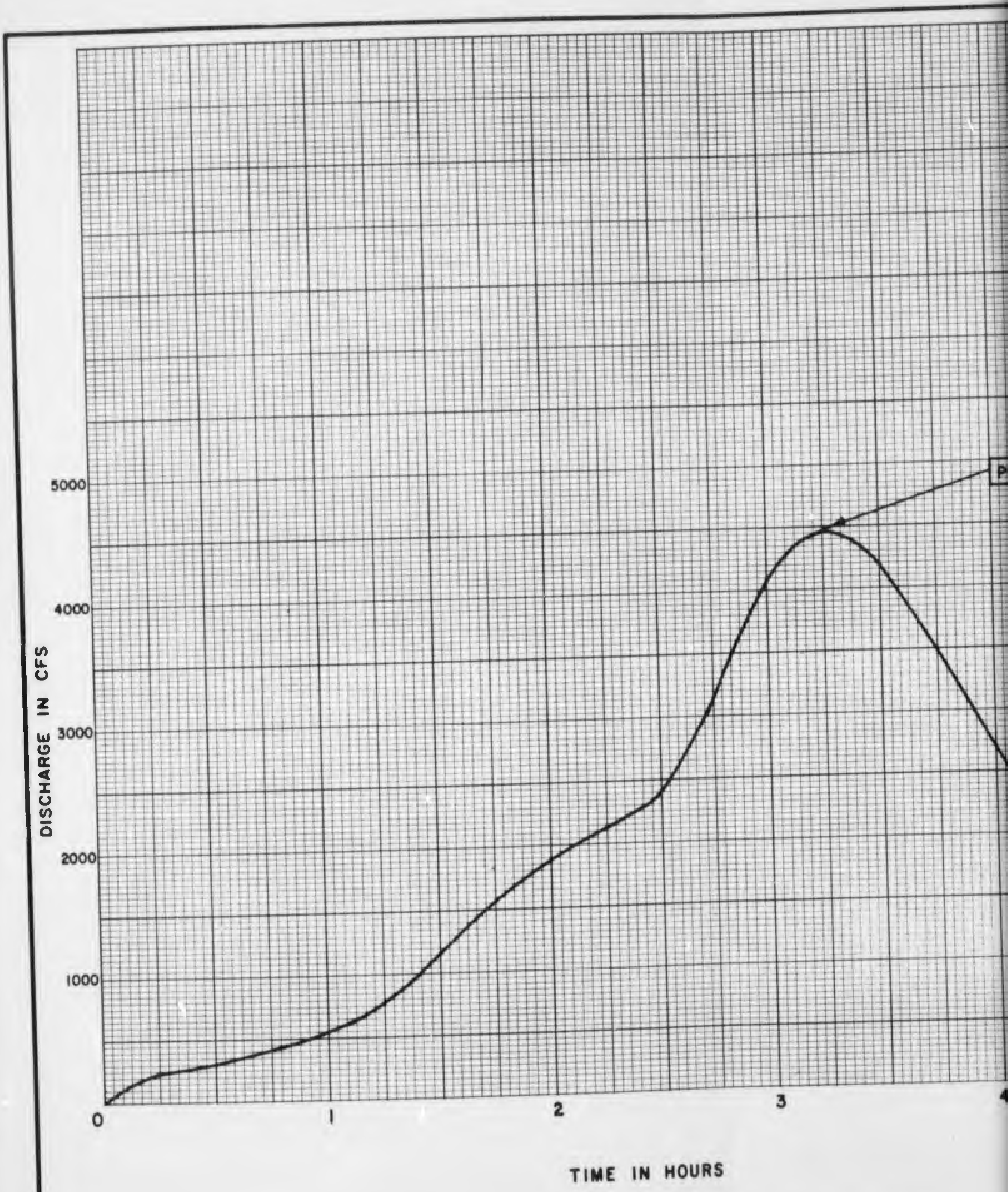
SANTA ANA RIVER, CALIFORNIA
PHASE I - GENERAL DESIGN MEMORANDUM

100-YR FLOOD HYDROGRAPH
SANTIAGO CREEK AT
DETENTION BASIN

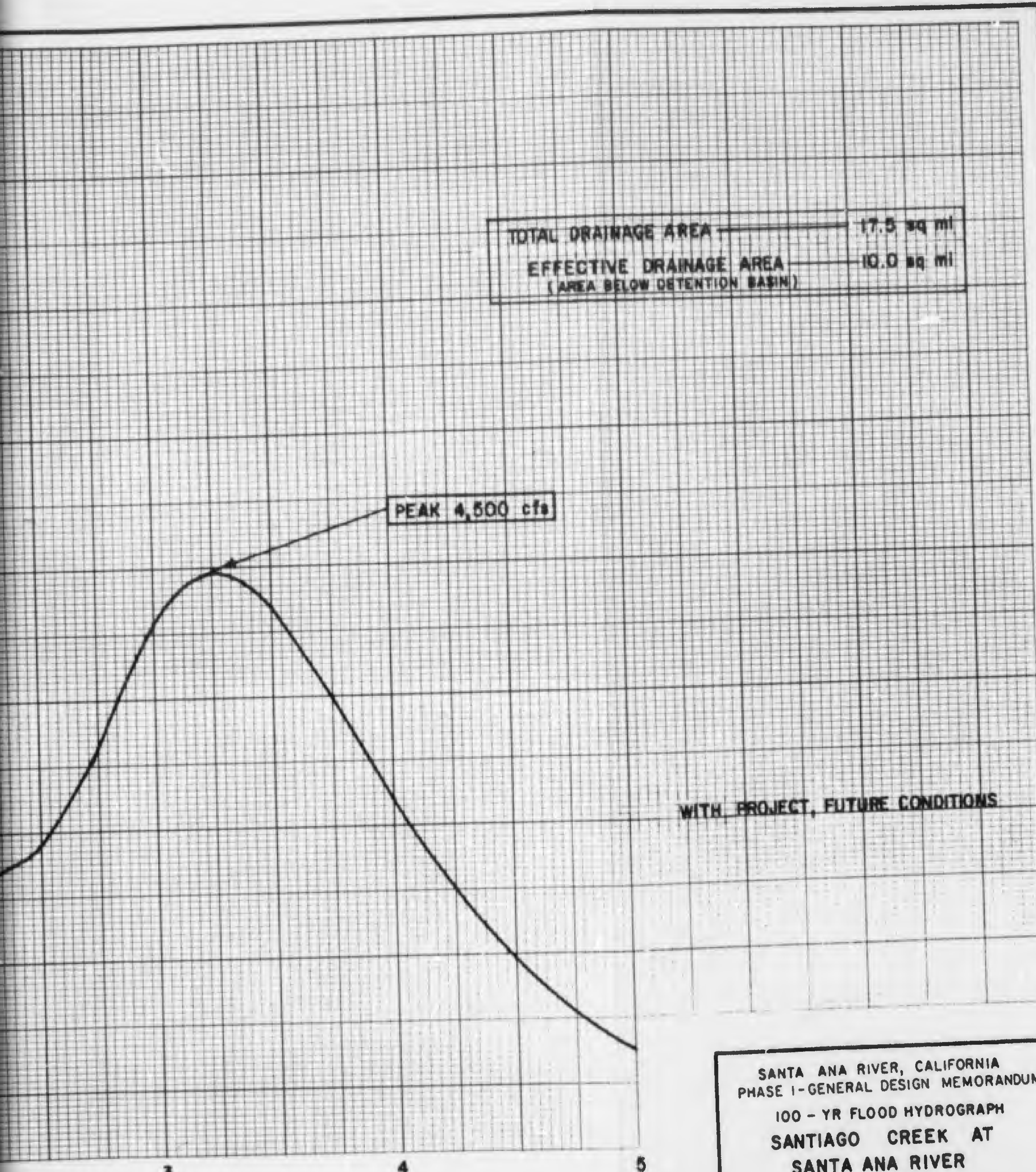
LOCAL STORM

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE B-34



DISCHARGE IN CFS
5000
4000
3000
2000
1000
0
1
2
3
4
TIME IN HOURS



SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

100 - YR FLOOD HYDROGRAPH

SANTIAGO CREEK AT
SANTA ANA RIVER

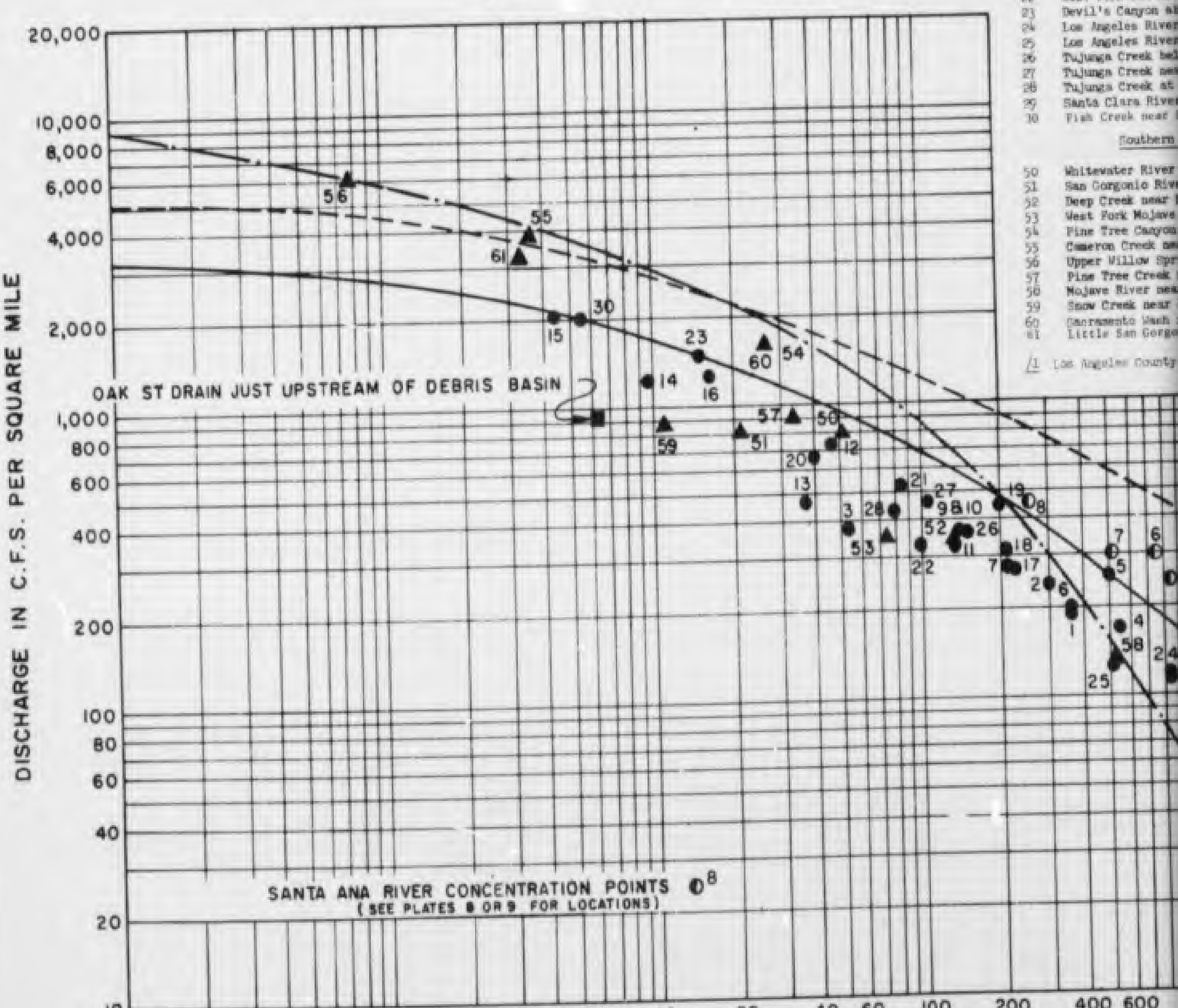
LOCAL STORM

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

TO ACCOMPANY REPORT DATED:

PLATE B - 35

2



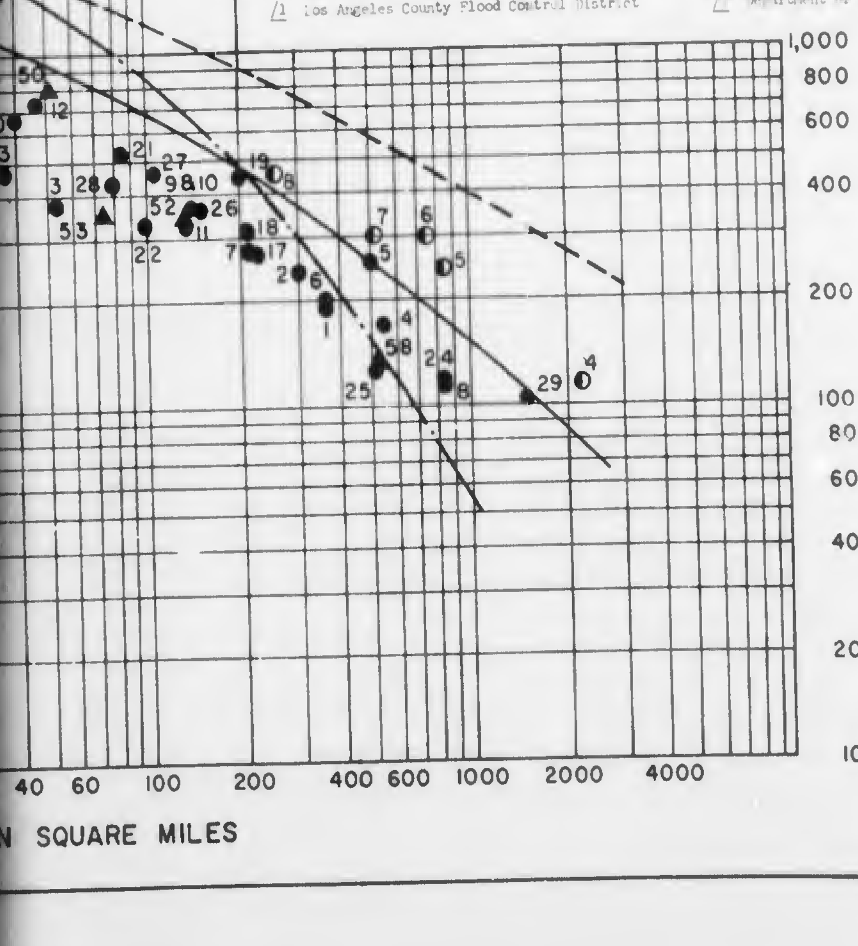
NO.	STREA
	Southe
1	San Diego River ne
2	San Dieguito River
3	Santa Ysabel Creek
4	San Luis Rey River
5	San Luis Rey River
6	San Luis Rey River
7	San Luis Rey River
8	Santa Ana River at
9	Santa Ana River ne
10	.....do........
11	San Jacinto River
12	Lytle Creek near
13	Mill Creek near Y
14	Cucamonga Creek ne
15	Day Creek near Et
16	San Antonio Creek
17	San Gabriel River
18	San Gabriel River
19	San Gabriel River
20	San Gabriel River
21	East Fork San Gab
22	West Fork San Gab
23	Devil's Canyon ab
24	Los Angeles River
25	Los Angeles River
26	Tujunga Creek bel
27	Tujunga Creek nea
28	Tujunga Creek at
29	Santa Clara River
30	Fish Creek near
	Southern
50	Whitewater River
51	San Gorgonio Rive
52	Deep Creek near
53	West Fork Mojave
54	Pine Tree Canyon
55	Cameron Creek ne
56	Upper Willow Spr
57	Pine Tree Creek
58	Mojave River nea
59	Snow Creek near
60	Sacramento Wash
61	Little San Gorgo

/1 Los Angeles County

PERTINENT DATA

NO.	STREAM AND LOCATION	DRAINAGE AREA Square miles	PEAK DISCHARGE Cubic feet per second	DATE	AUTHORITY
	Southern California-Pacific Slope Basins				
1	San Diego River near Santee	377	70,200	27 Jan 1916	USGS WSP 447
2	San Dieguito River near Bernardo	299	72,100	....do.....	USGS WSP 426
3	Santa Ysabel Creek near Mesa Grande	53.9	21,100	....do.....	USGS WSP 426
4	San Luis Rey River at Oceanside	557	95,600	....do.....	USGS WSP 426
5	San Luis Rey River at Bonsall	512	128,000	23 Feb 1891	USGS WSP 447
6	San Luis Rey River near Pala	373	75,300	27 Jan 1916	USGS WSP 426
7	San Luis Rey River near Mesa Grande	209	58,600	....do.....	USGS WSP 426
8	Santa Ana River at Riverside Narrows	858	100,000	2 Mar 1938	USGS WSP 844
9	Santa Ana River near Mentone	144	52,300	....do.....	USGS WSP 844
10	.....do	144	53,700	23 Feb 1891	USGS WSP 447
11	San Jacinto River below North Fork near San Jacinto	141	45,000	16 Feb 1927	USGS WSP 844
12	Lytle Creek near Fontana	47.9	35,900	25 Jan 1969	USGS Calif 1969
13	Mill Creek near Yucaipa	38.1	18,100	2 Mar 1938	USGS Calif. 1969
14	Cucamonga Creek near Upland	10.1	14,100	25 Jan 1969	USGS Calif. 1969
15	Day Creek near Etivanda	4.6	9,450	....do.....	USGS Calif 1969
16	San Antonio Creek near Claremont	16.9	21,400	2 Mar 1938	USGS WSP 844
17	San Gabriel River at Foothill Blvd	230	61,800	....do.....	USGS WSP 844
18	San Gabriel River below Morris Dam	211	65,700	....do.....	USGS Calif. 1963
19	San Gabriel River at San Gabriel Dam	202	90,000	....do.....	USGS WSP 844
20	San Gabriel River at Cogswell Dam	40.4	26,900	....do.....	USGS WSP 844
21	East Fork San Gabriel River near Camp Bonita	88.2	46,000	....do.....	USGS Calif. 1963
22	West Fork San Gabriel River at Camp Rincon	102	34,000	....do.....	USGS Calif. 1963
23	Devil's Canyon above Cogswell Dam	15.4	23,000	....do.....	/1
24	Los Angeles River at Long Beach	832	102,000	25 Jan 1969	USGS Calif. 1969
25	Los Angeles River at Los Angeles	514	67,000	3 Mar 1938	USGS Calif. 1963
26	Tujunga Creek below Hansen Dam	150	54,000	....do.....	USGS Calif. 1963
27	Tujunga Creek near Sunland	106	50,000	....do.....	USGS Calif. 1963
28	Tujunga Creek at Tujunga Dam (inflow)	81.4	35,000	....do.....	USGS Calif. 1969
29	Santa Clara River near Saticoy	1,596	165,000	25 Jan 1969	USGS Calif. 1969
30	Fish Creek near Duarte	6.4	13,000	....do....	USGS Calif. 1969
	Southern California-Interior Basins				
50	Whitewater River above Whitewater	51.4	42,000	2 Mar 1938	USGS WSP 844
51	San Gorgonio River near Banning	21.2	17,000	....do.....	USGS WSP 844
52	Deep Creek near Hesperia	137	46,600	....do.....	USGS WSP 844
53	West Fork Mojave River near Hesperia	74.8	26,100	....do.....	USGS WSP 844
54	Pine Tree Canyon 12 miles north of Mojave	35.0	59,500	12 Aug 1931	/2
55	Cameron Creek near Tehachapi	3.59	13,500	30 Sep 1932	/1
56	Upper Willow Springs Canyon near Mojave	0.81	4,900	....do.....	/1
57	Pine Tree Creek near Mojave	33.5	30,000	23 Aug 1961	USGS Calif. 1963
58	Mojave River near Victorville	530	70,600	2 Mar 1938	USGS Calif. 1963
59	Snow Creek near Palm Springs	11.0	9,500	Feb 1927	/3
60	Sacramento Wash near Needles	27.0	43,000	17 Aug 1939	USGS
61	Little San Gorgonio Cr. near Beaumont	3.23	11,000	25 Feb 1969	USGS Calif. 1969

/1 Los Angeles County Flood Control District /2 Department of Water and Power, Los Angeles /3 Coachella Valley Water District



LEGEND

● RECORDED OR ESTIMATED PEAK DISCHARGE-PACIFIC SLOPE BASINS.

▲ RECORDED OR ESTIMATED PEAK DISCHARGE-INTERIOR BASINS.

– – – – CREAGER ENVELOPING CURVE OF MAXIMUM FLOODS IN THE U. S.

——— C OF E ENVELOPING CURVE OF RECORDED OR ESTIMATED PEAK DISCHARGES FOR SOUTHERN CALIFORNIA COASTAL STREAMS.

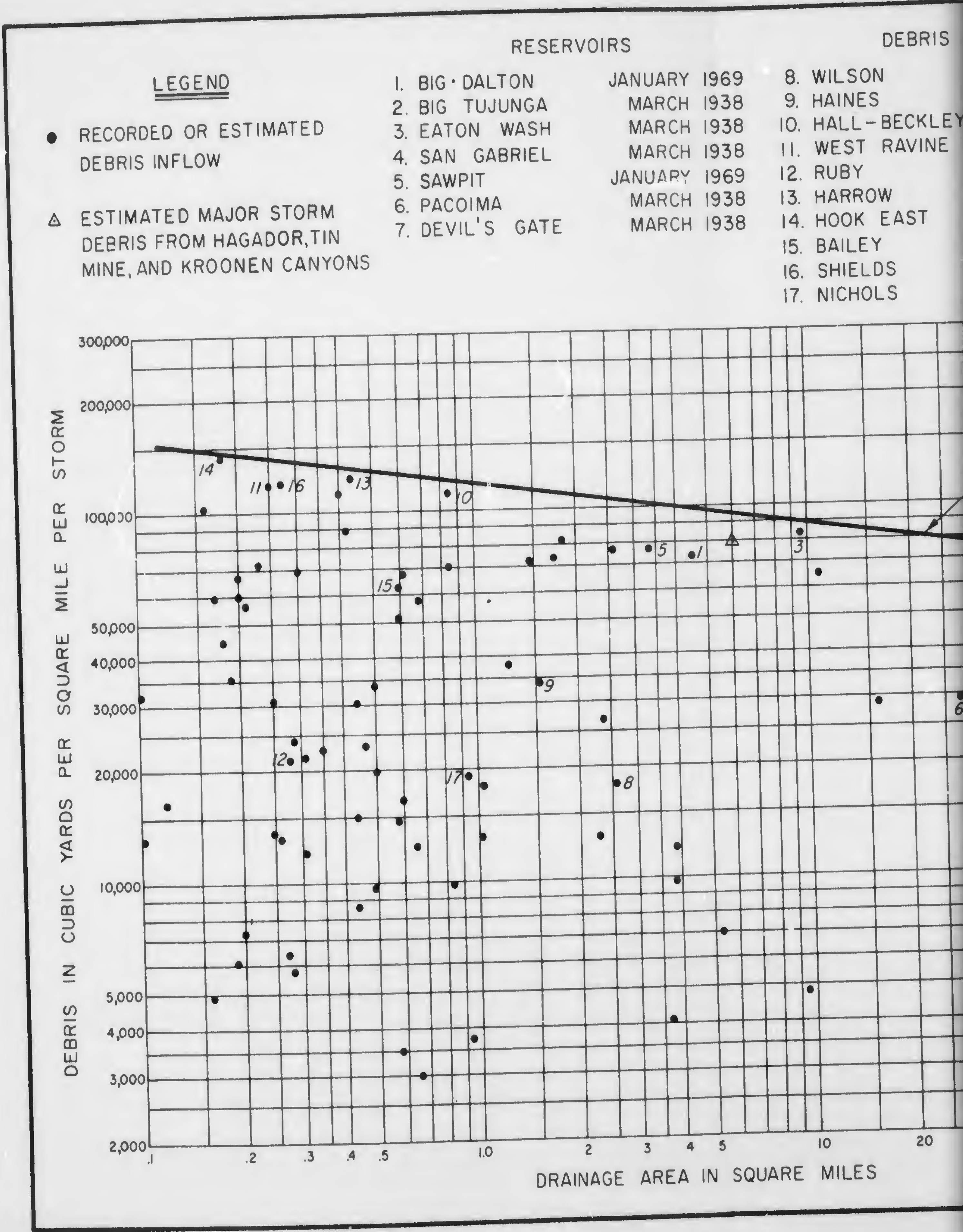
—·— C OF E ENVELOPING CURVE OF RECORDED OR ESTIMATED PEAK DISCHARGES FOR SOUTHERN CALIFORNIA DESERT STREAMS.

SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

ENVELOPING CURVES OF PEAK DISCHARGES STREAMS IN SOUTHERN CALIFORNIA

U.S.ARMY ENGINEER DISTRICT
LOS ANGELES.CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED

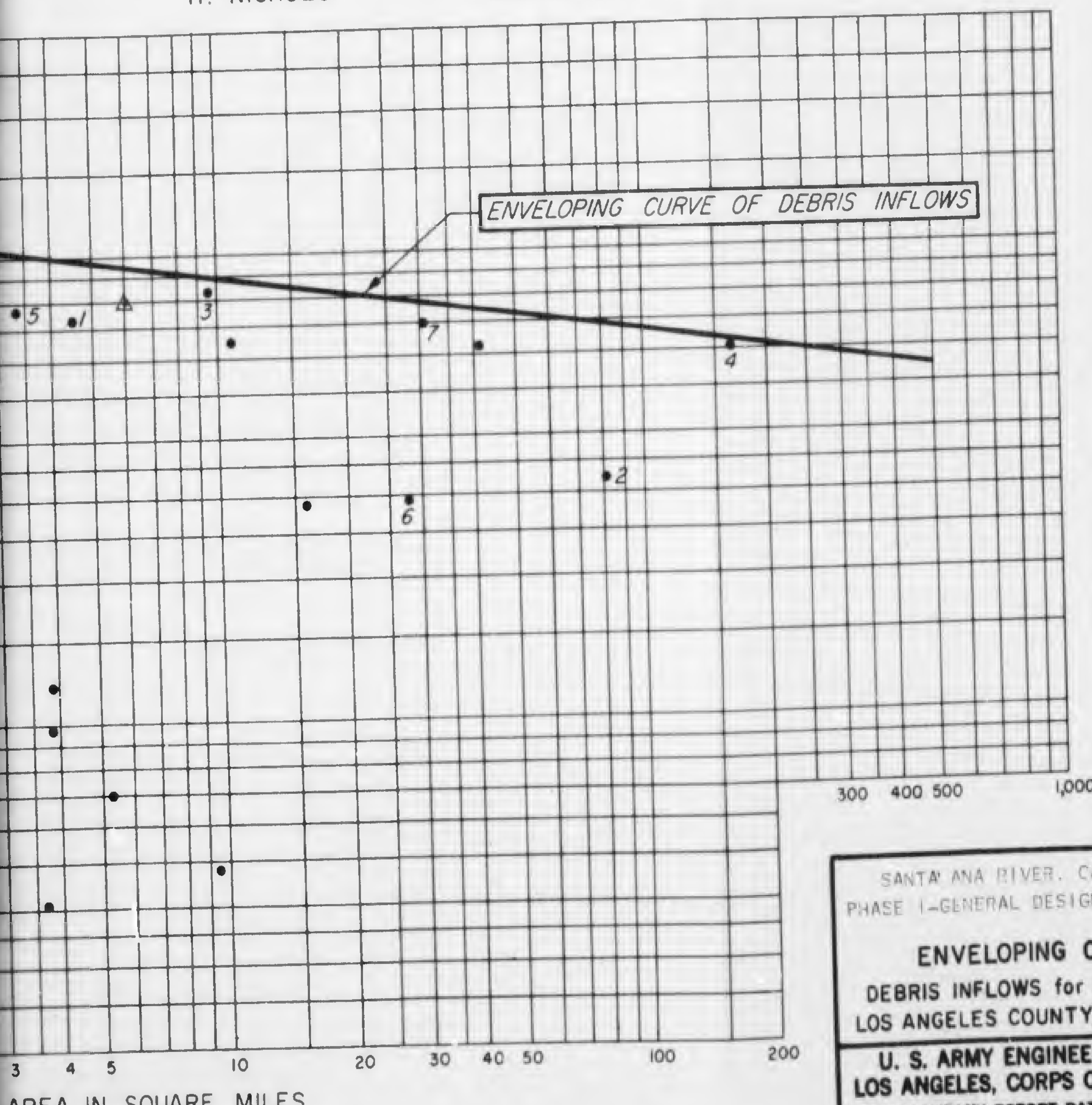
2



RESERVOIRS
DEBRIS
LEGEND
● RECORDED OR ESTIMATED DEBRIS INFLOW
△ ESTIMATED MAJOR STORM DEBRIS FROM HAGADOR, TIN MINE, AND KROONEN CANYONS
1. BIG·DALTON JANUARY 1969
2. BIG TUJUNGA MARCH 1938
3. EATON WASH MARCH 1938
4. SAN GABRIEL MARCH 1938
5. SAWPIT JANUARY 1969
6. PACOIMA MARCH 1938
7. DEVIL'S GATE MARCH 1938
8. WILSON
9. HAINES
10. HALL-BECKLEY
11. WEST RAVINE
12. RUBY
13. HARROW
14. HOOK EAST
15. BAILEY
16. SHIELDS
17. NICHOLS
DEBRIS IN CUBIC YARDS PER SQUARE MILE PER STORM
300,000
200,000
100,000
50,000
40,000
30,000
20,000
10,000
5,000
4,000
3,000
2,000
.1
.2
.3
.4
.5
1.0
2
3
4
5
10
20
DRAINAGE AREA IN SQUARE MILES

DEBRIS BASINS

NUARY 1969	8. WILSON	JANUARY 1969
IARCH 1938	9. HAINES	MARCH 1938
IARCH 1938	10. HALL-BECKLEY	MARCH 1938
IARCH 1938	11. WEST RAVINE	MARCH 1938
NUARY 1969	12. RUBY	JANUARY 1969
IARCH 1938	13. HARROW	JANUARY 1969
IARCH 1938	14. HOOK EAST	JANUARY 1969
	15. BAILEY	JANUARY 1954
	16. SHIELDS	MARCH 1938
	17. NICHOLS	MARCH 1938



SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

ENVELOPING CURVE

DEBRIS INFLOWS for BASINS in LOS ANGELES COUNTY, CALIFORNIA

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

TO ACCOMPANY REPORT DATED:

PLATE B-37

2

C Hydraulics

HYDRAULIC DESIGN

APPENDIX C

U. S. ARMY ENGINEER DISTRICT

LOS ANGELES

CORPS OF ENGINEERS

To accompany Phase I General Design Memorandum

for Santa Ana River, California

SANTA ANA RIVER REPORT
APPENDIX C
HYDRAULIC DESIGN

CONTENTS

PLATES

APPENDIX C

HYDRAULIC DESIGN

1. GENERAL.

a. The following paragraphs describe the important parameters associated with recommended improvements for the main stem of the Santa Ana River, Santiago Creek, Greenville-Banning Channel, Oak Street Drain, and the Prado and Mentone Reservoirs.

2. SANTA ANA RIVER.

a. Santa Ana River Channel. The proposed channel improvements (see pl. F-29 to F-57) begins near Weir Canyon Road, extending through approximately 23 miles of urbanized area to the river mouth. Channel improvement is constrained by existing channel widths, drop structures, bridge deck levels, utilities along the river, existing rights-of-way, and urban developments adjacent to the channel. The channel would be designed to convey 38,000 cfs at the upstream inlet (near Weir Canyon Road) increasing to 42,000 cfs at Santiago Creek; from Santiago Creek down to the confluence structure for Greenville-Banning (just below Victoria Street) the channel would convey 46,000 cfs; from Victoria Street down to the ocean the channel would be designed to convey 47,000 cfs.

b. Water surface profiles were computed for the channel with and without sedimentation. The Manning's equation using the reach method was used to compute the water surface. For water surface without sediment, an "n" value of 0.014 and 0.030 was used to compute flow depths in the concrete and earth bottom channel, respectively. An "n" value if 0.020 was used for the reach where sediment was assumed to be deposited. For riprap protection in the drop structure reach an "n" value of 0.020 was used to determine the design velocity.

c. The reach of concrete channel from River View Golf Course downstream to Adams Avenue would be in rapid flow condition when no sediment is assumed and would not be affected by backwater in the downstream reach from Adams Avenue to the ocean. The governing flow condition occurs when 5 feet of sediment is assumed to be deposited from Adam Avenue downstream to the ocean. The sediment deposition was assumed to taper out from Adams Avenue on upstream to 5th Avenue. When sediment is assumed, backwater flow condition would move upstream to station 505+00 (below Fairview Street), where a hydraulic jump would occur. For either of the above conditions, flow would be within the unstable zone and the channel wall height was adjusted for it. Some of the unstable flow condition could possibly be eliminated during detailed design studies.

d. Base width of the recommended rectangular channel in this reach would vary from 230 to 250 feet. Wall heights would vary from 12.5 to

20.0 feet. A freeboard of 2 feet would be added, in addition to the adjustments mentioned above, to the depth of flow to determine the required channel wall heights. Refer to plates F-48 through F-54 for profiles, plan views, typical sections, and dimensions of the proposed channel.

e. The earth-bottom channel reach extends from River View Golf Course upsteam to Weir Canyon Road. This reach of channel has 13 drop structures; of the 13 drop structures, 9 are existing. Three of the existing drop structures would require modification. The drop structures reduce the invert slope to an average of about 0.0018. Because the distance between the drop structures is about one mile, stabilizers would also be required to stabilize the channel invert between the drop structures. Wall heights for this reach of the channel was adjusted for sediment deposition. A freeboard of 2.5 feet would be added, in addition to the above adjustment, to the depth of flow to determine the required wall height. The base width for this reach of channel would vary from 270 to 320 feet and would be trapezoidal in cross section, with side slopes of 1V on 2H. The side slopes would be riprapped. The levee height would vary from 15 to 18 feet.

f. River View Golf Course, a 3500-foot long green-belt reach, is located between the downstream end of the earth bottom channel and the inlet to the concrete channel. Santiago Creek discharges into this green-belt area (see pls. F-46 and F-47).

g. Design at Channel Mouth. Backwater studies at the mouth of the Santa Ana River channel, were conducted to insure the adequacy of the design section. The Greenville-Banning channel joins the Santa Ana River channel with a confluence structure approximately 1800 feet downstream from Victoria Street. Based on what was experienced during two major storms in January and February of 1969, water surface profiles were computed for two conditions; with and without sediment deposition in the channel invert. About 5 feet of sediment were deposited in the channel near the mouth by the 1969 floods. No breakdown is available on the amount of sediment that was deposited in the channel invert by each of the major storms. However, considering the relatively long periods of rainfall and runoffs that accompanied the first storm it is reasonable to assume, that most of the sediment deposited during this storm.

h. To start backwater computation without sediment deposition, channel invert was set at elevation -6.85 and water-surface at El. 2.54 MHHW (MSL Datum). With sediment deposition channel invert was set at El. -1.85 with depth of flow at critical depth.

i. This reach of channel, from station 13+00 to station 157+00, is an earth bottom channel with rectangular cross section. The vertical walls would be of reinforced concrete inverted tee section. This portion of channel would be earth-bottom because of environment requirements. See plates F-52, F-53, and F-54.

j. The last 1,600 feet of the Huntington Beach channel (see pl. F-54) would be realined to make more room for the Santa Ana River. The Huntington Beach channel would be located farther west of its present location.

k. Toe depths for the levees and the depth of protection for the drop structures and stabilizers were selected from observations of streambed erosion that has occurred in leveed channels similar to the improvement proposed for the Santa Ana River.

l. Interior Drainage Considerations. No significant side drainage problems would be created as a result of the recommended project. Currently, the storm water along the right side of the Santa Ana River from the Pacific Ocean to about Santiago Creek are collected in Orange County storm drains and are carried away from the river, except in three localized areas where the storm runoff is collected and pumped into the river. Along the left side of the river from the Pacific Ocean to Santiago Creek, the Greenville-Banning channel collects the storm runoff and carries it parallel to the river and into the confluence structure with the Santa Ana River. Above Chapman Avenue to the upper end of the project, Orange County has a series of storm drains that collect the storm runoff and empty into the Santa Ana River. Since the recommended plan will not raise the water surface for most reaches of the river, and only slightly (less than 1 foot) in portions of the reach between Katella Avenue and Imperial Highways, the county drains would function as designed. These storm drains in general meet the criteria presented in EM-1110-2-1410 titled "Interior Drainage of Leveed Urban Areas: Hydrology."

m. Implementation of the recommended plan would result in an increase in the design water surface behind Prado Dam from the current 543 feet to 563 feet. As a result, three facilities within the enlarged reservoir area will be effected. They are the California Institution for Women (site 1), the Alcoa Aluminum Plant (site 2), and the Chino Sewage Treatment Plant (site 3). The low points of sites 1, 2, and 3 are 560, 552, and 540 feet, respectively. The corresponding frequency that the reservoir water surface elevation will equal or exceed the low point of each site is 155 year for site 1, 135 year for site 2, and 75 year for site 3. Protection for the three sites will be provided by dikes. For the purpose of this report and to establish an upper limit on the cost, the following analysis were made.

n. Two cases were analyzed to determine the effect on interior drainage after construction of the dikes. The first case considered was centering the standard project flood local thunderstorm over each site with the reservoir water surface elevation below the level of the dikes. The resulting standard project flood peak discharges were 210 cfs for site 1, 120 cfs for site 2, and 70 cfs for site 3. The drainage area sizes are 0.15 square miles for site 1, 0.10 square miles for site 2, and 0.04 square miles for site 3. Pipes with flapgates would be provided at each site to pass the standard project flood peak discharge through the dike. The number and size of pipes required are seven - 36 inch pipes at site 1, four-36 inch pipes at site 2, and two-36 inch pipes at site 3.

o. The second case considered was with the reservoir water surface elevation at or above the elevation of the flapgates. Under this condition, a standard project flood or something approaching the standard project flood would have just occurred. Since it is highly unlikely that any significant rainfall would occur within a day or two following a storm front that resulted in the standard project flood, a 25 year frequency storm was used to determine the volume of water to be ponded until the reservoir water surface would be lowered. The coincidental frequency of this event would be between a 100 year and standard project event. The 25-year volume at site 1, 2, and 3 is 6, 3, and 2 acre-feet, respectively. The dikes would be located and land obtained so that the water could be ponded without flooding any facilities.

p. Sedimentation in the Proposed Santa Ana River Channel. The sediment deposition allowances were made based on information of sediment movement and deposition obtained from the January and February 1969 flood events on the Santa Ana River, and from sediment routings using DuBoys Equations. This routing study indicated that, during peak flows of the standard project flood, little sediment would be deposited in the proposed channel. Most of the sediment would be deposited in the channel bed after the storm has passed and during long periods of controlled releases. The sediment is probably produced within the approximately 6 miles reach (Santa Ana Canyon) of natural stream between Prado Dam and the upstream inlet of the proposed channel at Weir Canyon Road. Additional sediment data would be gathered and used with the HEC-6 sediment program during detailed design studies to determine the adequacy of the design sediment allowance in the downstream channel. Preliminary studies indicated that the HEC-6 sediment program could be utilized to reproduce the existing prototype condition.

q. The estimated average annual rate of sediment contribution to the delta varies from 80,000 to 100,000 cubic yards per year. This estimate is based on sediment contributions from the standard project, 100-year, and 50-year floods, channel width widened to 480 feet, channel depth lowered 5 to 9 feet, and the presence of thirteen drop structures in the upstream reaches. It was assumed that storms occurring more frequently than once every 20 years would have a negligible effect on the sediment contribution to the coast because of the low volume and velocity of the flow and percolation to groundwater. The less frequent standard project flood will carry more sediment to the ocean under recommended plan conditions, because the flood runoff will be confined and controlled (under existing conditions, the entire downstream basin would be flooded and the sediment would be distributed over the floodplain).

Type of Flood	Deposited in delta
Standard Project	6,550,000
100-year	870,000
50-year	490,000

r. The estimated average annual rate of sediment deposition to be removed from the Santa Ana River is 150,000 cubic yards. This is based on past removal of debris from the Santa Ana River by Orange County. Since 1938, 2.5 million cubic yards of sediment have been removed from the lower Santa Ana River, or an average annual amount of about 60,000 cubic yards. The recommended project would about double tne existing width of channel and slightly reduce the slope. To account for these channel modifications, the historical average annual amount was increased by a factor of 2.5 to 150,000 cubic yards. This material will deposit in the lower 4 miles of the project.

s. Based on the past 42 years, the frequency of removal of excess sediment from the channel will be on the average of once every 10 years. One possible disposal site for this material would be gravel pits near the river. The combined capacity of these pits is about 12 million cubic yards. About 7 million cubic yards of this space may be utilized to dispose of channel excavation during construction of the project, leaving 5 million cubic yards for possible disposal of the material removed from the channel. Other possible sites for disposal would be to developers to pad up building sites and disposal of the material on the beach.

3. SANTIAGO CREEK.

a. The Santiago Creek proposed improvements (see pls. F-24 to F-28) consist of a regulating reservoir, located in Villa Park City, and 6,000 feet of channel improvement upstream from the confluence with the Santa Ana River. The reach of the existing stream between the proposed improvements is capable of conveying the design discharge, and further flood protective works are not considered necessary in this reach. The regulating reservoir would include: (a) an inlet structure to convey flows from Santiago Creek; (b) the reservoir itself; and (c) a gated outlet structure with an outlet channel that would meet the existing Santiago Creek channel.

b. Inlet structure. The inlet structure would be a baffled apron as described in the U.S.B.R. Engineering Monograph No. 25; Hydraulic Design of Stilling Basins and Energy Dissipators. The inlet structure would be designed for discharges up to 5,600 cfs, which is the combined 100-year peak discharge (general storm) from Santiago Creek and Handy Wash.

c. Regulating reservoir area. The existing gravel pits, located between Villa Park Road and Prospect Road, would be utilized to serve as the regulating reservoir. Some excavation would be necessary to provide the required capacity of 3,300 ac. ft. between elevation 280.0 and 298.0.

d. Outlet structure. The outlet structure would have three gates to control the outflow from the general storm, to a maximum of 3500 cfs. The outflow from local storms would be controlled by ponding to a maximum of 500 cfs.

e. Channel improvements. About 6,000 feet of channel upstream from the confluence with the Santa Ana River would be improved. The side slopes of the channel in this reach are presently protected with gravel and wire mesh, but additional protection would be required to prevent damage during the design discharge of 5,000 cfs. Therefore, the side slopes and invert of the channel would be provided with a 18"-thick blanket of riprap.

4. GREENVILLE-BANNING CHANNEL.

a. The recommended improvements (see pls. F-55 to F-57) for the Greenville-Banning Channel would require reconstruction of the channel from just below California Street to the confluence structure on the Santa Ana River, downstream of Victoria Street. The channel would be of rectangular concrete cross section. Design discharge in the channel would range from about 3,300 cubic feet per second upstream from Fairview channel, a tributary, to 4,400 cubic feet per second at the confluence structure. Channel base width would vary from 50 to 60 feet and wall height would range from 13.5 to 17 feet.

b. Wall heights for the channel were adjusted for the combination of backwater conditions that would give the highest backwater flow depths. The backwater conditions include: (a) different combination of discharges in the confluence structure for Santa Ana River and Greenville-Banning channel; and (b) with and without sediment deposition in both channels.

5. OAK STREET DRAIN AND DEBRIS BASIN.

a. Debris Basin. The existing debris basin (see pl. F-11) constructed by the Riverside Flood Control District in 1979, would be modified to accommodate the PMF of 16,000 cfs with 3 feet of freeboard. The present debris basin consists of a compacted earth embankment, an excavated basin, a rectangular concrete broadcrested spillway with stilling basin, and a pool drain.

(1) Debris storage capacity. The basin would provide about 253 acre-feet of natural storage, based on an assumed debris-basin slope of one-half the average slope of the existing slope, projected upstream from the spillway crest. The estimated debris-storage requirement is 305 acre-feet. The excess debris volume of 52 acre-feet would be transported into the proposed downstream channel.

(2) Spillway and embankment. The existing spillway would pass the spillway design flood of 16,000 cfs with the maximum water surface at about the top of the dam embankment. The existing embankment has a maximum height of about 30 feet.

(3) The existing spillway, 120 feet in width and 103 feet in length, would be extended 1,000 feet downstream, transitioning from a base width of 120 feet to 20 feet with each wall converging on a 1:20 ratio. The top of walls for the extended spillway section would be set to convey the spillway design flow with 3 feet of freeboard for a distance of about 300 feet downstream from the toe of the embankment before the PMF overtops the spillway walls. About 3.8 feet freeboard is available within the existing spillway at spillway design discharge. At the design discharge of 5,000 cfs (SPF) the depth of flow would range from 3.8 feet at spillway crest to 8.3 feet at the downstream end of the transition (Sta. 162+00). Velocities would range from 11 feet per second to 31 feet per second.

(4) Sediment discharge of 52 acre-feet into the channel, after the debris basin capacity is exceeded, would occur during the receeding limb of the hydrograph. This amount of sediment discharging into the channel would not significantly affect the wall height that is designed to convey the SPF peak discharge. A comprehensive sediment inflow-outflow analysis would be conducted during the detailed design stage of the project.

b. Channel. An entrenched rectangular concrete-lined channel, (see pls. F-11 to F-14), with base width ranging from 20 feet to 24 feet and wall heights ranging from 8.0 feet to 13 feet would extend 2.9 miles from the debris basin spillway structure to its terminus at Temescal Canyon. The channel would be designed to convey the SPF design discharge of 5,000 cfs to 9,200 cfs. Depths of flow would vary from 6.2 feet to 9.7 feet. Flow velocities would vary from 35 feet per second to 45 feet per second.

c. Confluence structures would be provided at the junction with the future Lincoln Diversion channel, to be constructed by local interests, and at the existing confluence with Mangular channel.

d. Between Sta. 46+00 and Sta. 33+00 would be 1300 feet of covered channel. The 24 feet width reinforced concrete box section is designed to convey the SPF peak discharge flow in opening channel condition with a 2 foot minimum freeboard. Reference was made to ETL 1110-2-215, EM 1110-2-2902 and EM 1110-2-1410 that would pertain to the design of conduits, culverts and pipes.

6. PRADO AND MENTONE DAM RESERVOIR.

a. General. Enlarging Prado Dam and Reservoir to control the standard project flood with maximum release of 30,000 cubic feet per second and with a dam near Mentone and East Highlands would require the acquisition of about 1,670 acres between elevations 556 and 566.

Hydraulic model studies would be required prior to completion of detailed design for Prado Dam to verify design assumptions and theoretical analysis for the outlet works and the spillway.

b. The main features of the proposed plan for Prado Dam and Reservoir consist of the following:

(1) Raise Prado Dam 30 feet to elevation 596. (See plate F-16.)

(2) Construct new outlet works to more than double the existing outlet capacity. (See plates F-19 and F-20.)

(3) Raise and widen the spillway. (See plates F-17 and F-21.)

(4) Construct a containing levee on the south side of the proposed reservoir approximately along the Santa Fe railroad. The top of the levee would be at elevation 596. The levee would have a service road on the top, a revetted face on 2:1 side slopes, and would be a little more than 2 miles long.

(5) Construct ring levees to protect the Corona wastewater treatment plant and the Alcoa Aluminum Plant on Rincon Street in Corona.

(6) Modify the interchange between the Riverside Freeway and State Highway Route 71.

(7) Develop a recreational system consisting of an information center, an overlook area, wildlife areas, camping areas, a trailer camp, parks, day-use areas, three fishing lakes, picnic areas, agricultural buffer zones, and recreational trails.

c. The Prado Dam spillway design flood routing was based on the reservoir net capacity, the spillway discharge curve, and 15,000 cfs maximum flow passing through each of the two outlet conduits. The starting water-surface elevation for the spillway design (PMF) flood routing is the elevation in the reservoir five days after the beginning of the standard project flood, elevation 548.0 feet. The resulting maximum water-surface elevation would be 584.9 feet, and the corresponding peak spillway outflow would be 575,000 cfs. Thus, the peak inflow of 700,000 cfs would be reduced to a combined outflow of 605,000 cfs including that through the two 25 foot diameter outlet conduits. The time-inflow-outflow reservoir water-surface relationships for the spillway design-flood routing are shown on plate C-1.

d. The spillway discharge curve was computed by using variable discharge coefficient 'C' in the formula: $Q = CLH^{3/2}$ where 'L' is the crest length in feet and 'H' the head measured between spillway crest elevation and reservoir water-surface levee. The discharge coefficient was obtained from hydraulic design criteria sheet 122-1, low ogee crest with approach depth effects.

e. Outlet works. The preliminary structural evaluation requires the existing outlet works to be relocated and the present conduit plugged. This was done for several reasons. With increased weight of the enlarged embankment, failure of the existing conduit would be a real possibility if it remained in operation. In addition, the present control tower would have to be rebuilt to remain usable with the increased reservoir water surface elevations, thus necessitating the construction of two towers; one for the new outlet works and one for the old. In addition to the cost factor, operational problems could result from having to operate two separate control towers.

f. The Mentone damsite is located in the upper San Bernardino Valley just downstream from the Santa Ana River's junction with Mill Creek and Plunge Creek. (See pl. F-2.) The primary function of Mentone Reservoir would be to collect floodwaters from Big Bear Lake, the upper Santa Ana River, Mill Creek, and Plunge Creek. The waters would be detained 4 or 5 days until the high water level at Prado Reservoir had passed and would then be released slowly until the reservoir is emptied. Mentone Dam would be a horseshoe-shaped earthfill dam resting on a broad gravel bed area.

g. Construction of Mentone Dam would be combined with the improvement and extension of the Mill Creek Levee. The existing levee would be raised an average of 6 feet. In addition, the levee would be extended downstream by 1.2 miles and would terminate in the Mentone Reservoir area. The raising and lengthening of the Mill Creek Levee would prevent floodflows up to and including the standard project flood from breaking out of Mill Creek and by passing the dam. The levee is not designed to contain the probable maximum flood. Because of the large debris load carried by the floodflows from Mill Creek and the Santa Ana River, 10 feet of freeboard was added to the Mill Creek levee extension. During a probable maximum flood event, floodflows, in all probability, would breakout of Mill Creek where the levee begins and bypass the dam.

h. A groin-field will be constructed in conjunction with the Mill Creek Levee extension. The groin field will serve two purposes. The first is to prevent floodflow from the Santa Ana River from directly impinging on the levee. The second is to direct floodflows and the debris that they carry further away from the spillway approach area to prevent any potential blockage of the spillway. There are 8 groins ranging in length from 1600 ft to 2800 ft. The spacing and alignment of the groins were set so that impinging flows would have to cross at least 2 groins before reaching the levee. The length of the groins were set to direct the floodflows and debris from smaller flood events to form a channel along the tips of the groins, keeping the flows from impinging on the levee and carrying debris away from the spillway aproach. For large flood events, (those approaching the 100-year flood) flows will breakout of existing natural low flow channels and spread out over the entire alluvial fan. Debris carried by the larger events would be spread out over the entire fan, with a large portion being carried into the reservoir area. The design of the Mill Creek Levee extension and the groin field are considered adequate to prevent breaching of the levee and blockage of the spillway. However, because of the many unknowns involving floodflow and debris movement on alluvial fans, further studies will be required in the next study phase to finalize the design.

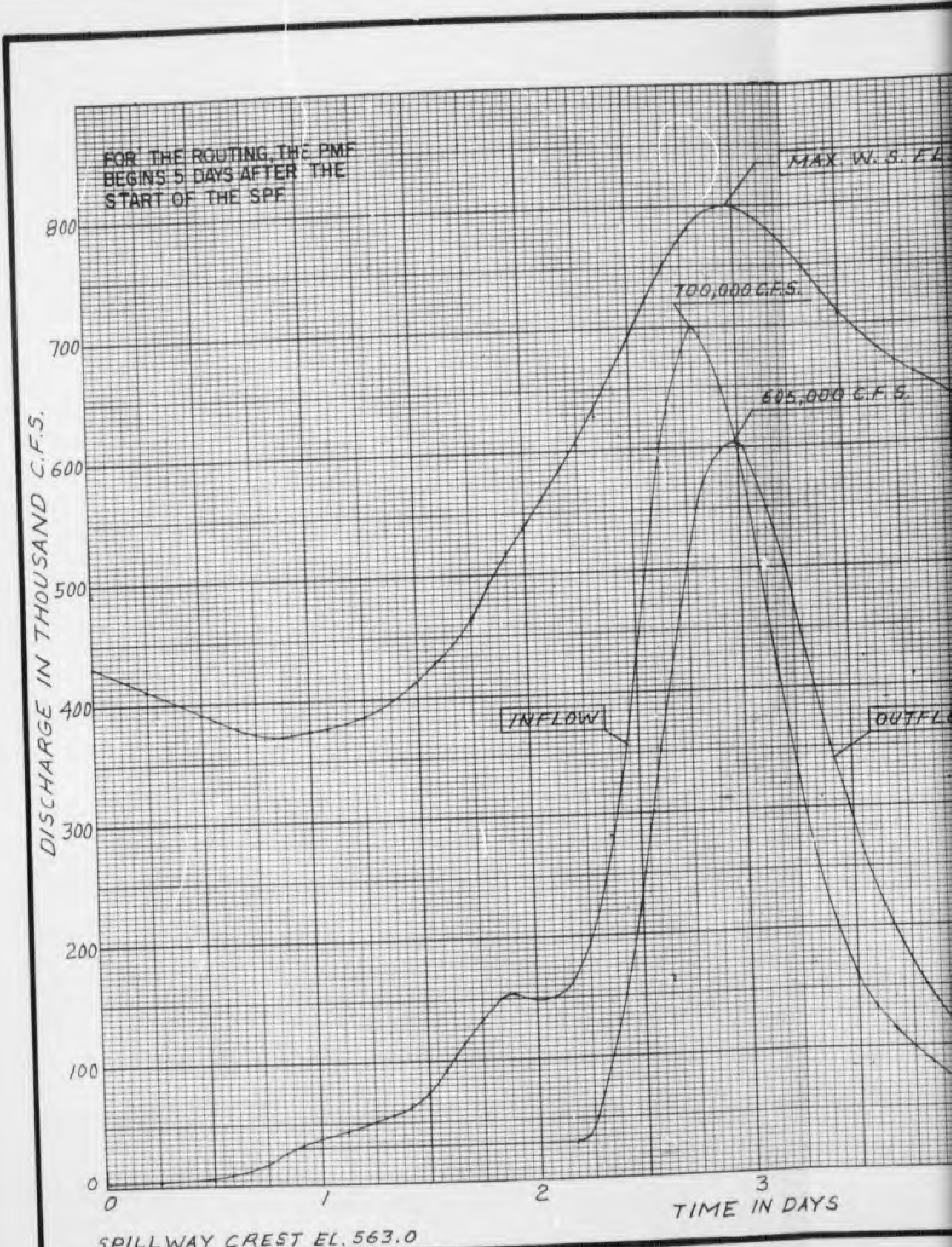
i. The reservoir would control the standard project flood, and the spillway would pass the maximum probable flood. The dam would rise about 243 feet above the streambed. Top of dam would be at elevation 1,573.5. Spillway crest would be at elevation 1548.5. Most of the 4,300 acres required for rights-of-way for the dam and reservoir are currently owned by various governmental agencies. The outlet work would be designed to control a maximum release of 6,000 cubic feet per second. (See plates F-7 and F-8.)

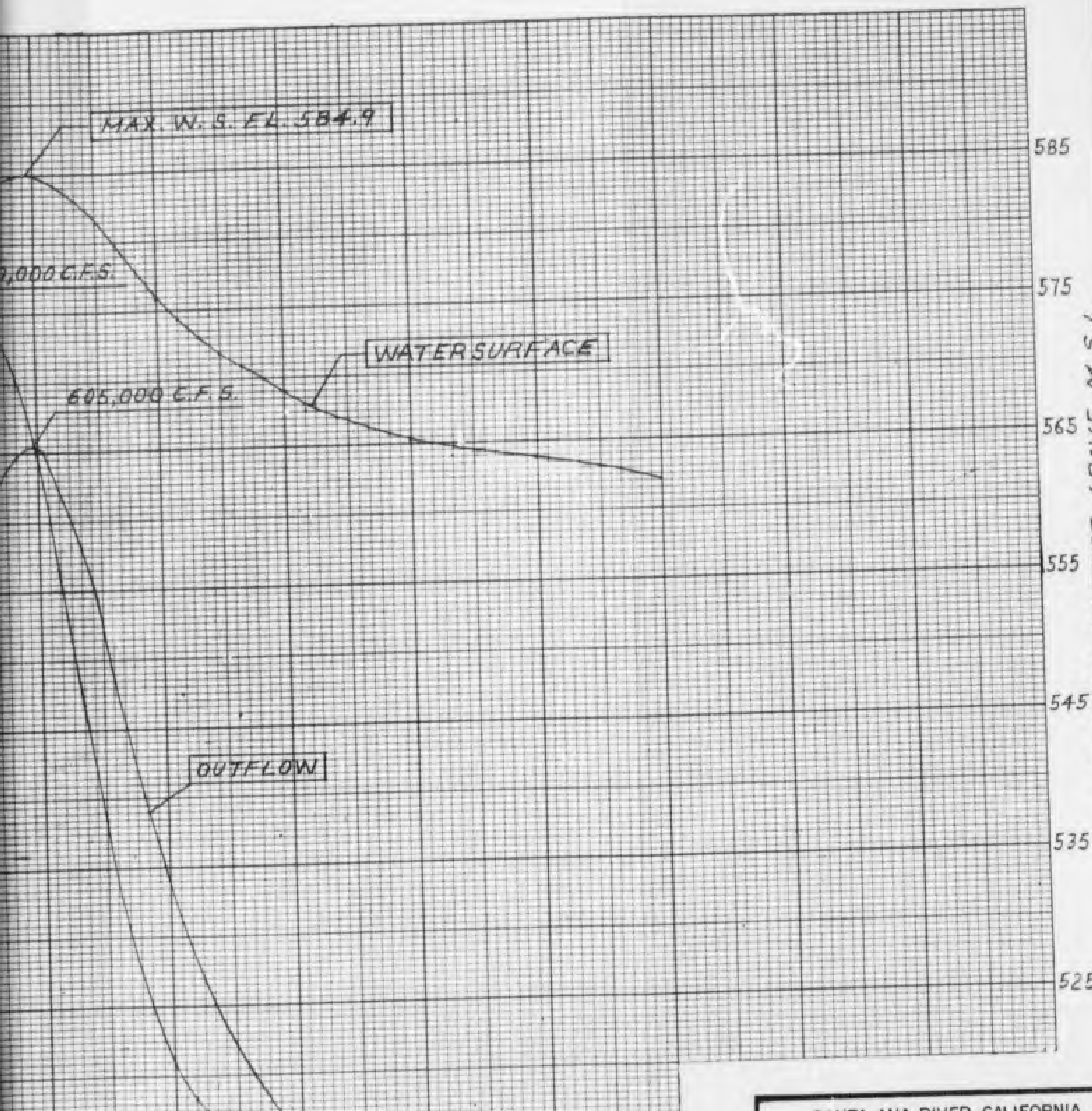
j. The Mentone Dam spillway design-flood routing was based on the reservoir net capacity, the spillway discharge curve and with a 5000 cfs outflow discharging through the 14-foot diameter outlet conduit. The starting water-surface elevation for the spillway design flood routing is the elevation in the reservoir five days after the beginning of the standard project flood, elevation 1542.1 feet. The resulting maximum water-surface elevation would be 1564.6 feet, and the corresponding peak spillway outflow would be 254,000 cfs. Thus, the peak inflow of 266,000 cfs would be reduced to a combined outflow of 259,000 cfs. The time-inflow-outflow reservoir water-surface relationships for the spillway design-flood routing are shown on plate C-2. The spillway discharge curve was developed same as in the Prado Dam design.

k. The Mentone Reservoir would have an estimated gross storage capacity of 188,000 acre-feet for flood control including 37,000 acre-feet for debris storage. Plate C-3 shows the area-capacity curve.

l. Construction of the reservoir near Mentone would allow a reduction of about 3,500 acres at Prado Reservoir. Mentone Dam would also provide standard project flood protection to the reach of the Santa Ana River from Mentone Dam to Prado Reservoir.

m. Spillway Outlet Structure. The plan would consist of: (1) a concrete transition, going from rectangular to a trapezoidal cross section; (2) 300 feet of grouted stone section with grouted stone apron, sloping down to 12 feet; and (3) 100 feet of dump stone section with apron, sloping down to 25 feet. The dump stone section and sloping apron would be 12-feet thick and would consist of derrick stone. A grouted stone wrap-around would be provided on the embankment at the end to protect against eddy action. The toe of the wrap-around would be about 15 feet below the present streambed. The outlet structure's terminal velocity would be about 47 feet per second.


FOR THE ROUTING, THE PMF BEGINS 5 DAYS AFTER THE START OF THE SPF
MAX. W. S. EL.
700,000 C.F.S.
605,000 C.F.S.
INFLOW
OUTFL
DISCHARGE IN THOUSAND C.F.S.
800
700
600
500
400
300
200
100
0
0
1
2
3
TIME IN DAYS
SPILLWAY CREST EL. 563.0



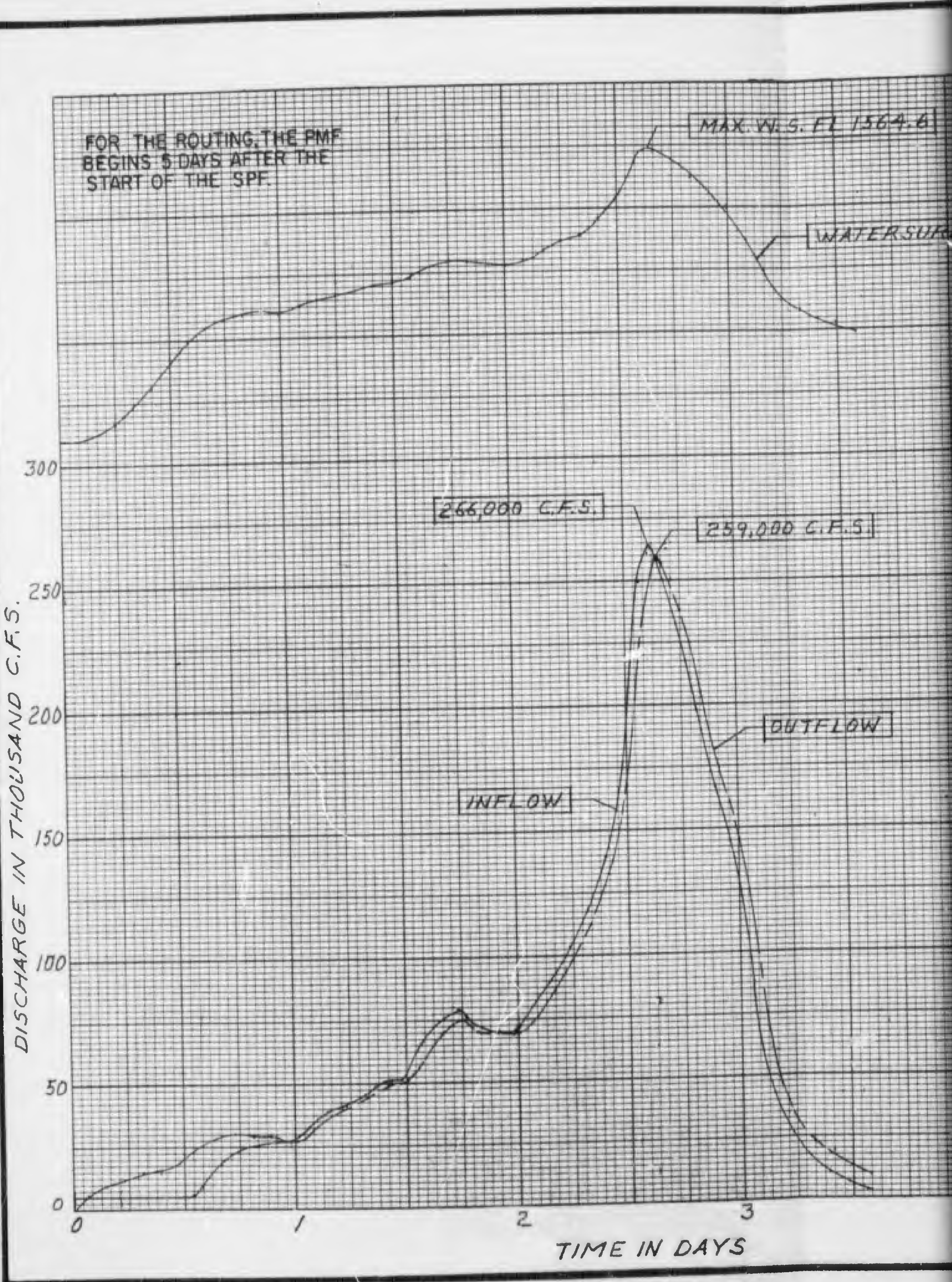
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

PROBABLE MAXIMUM FLOOD ROUTING SANTA ANA RIVER AT PRADO DAM

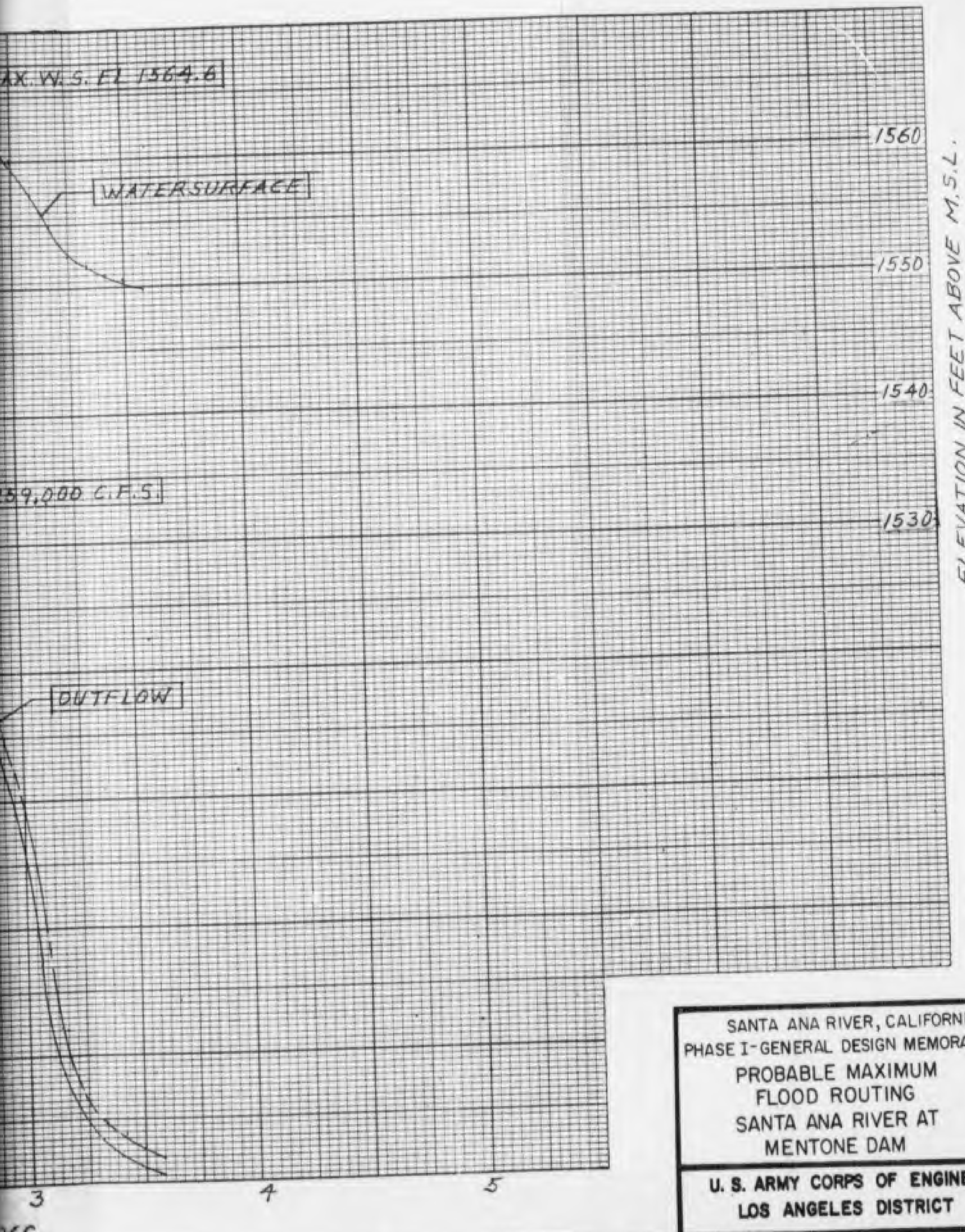
U. S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE C-1

2



FOR THE ROUTING, THE PMF BEGINS 5 DAYS AFTER THE START OF THE SPF.
MAX. W.S. EL 1564.6
WATERSUR
266,000 C.F.S.
259,000 C.F.S.
OUTFLOW
INFLOW
DISCHARGE IN THOUSAND C.F.S.
300
250
200
150
100
50
0
0
1
2
3
TIME IN DAYS



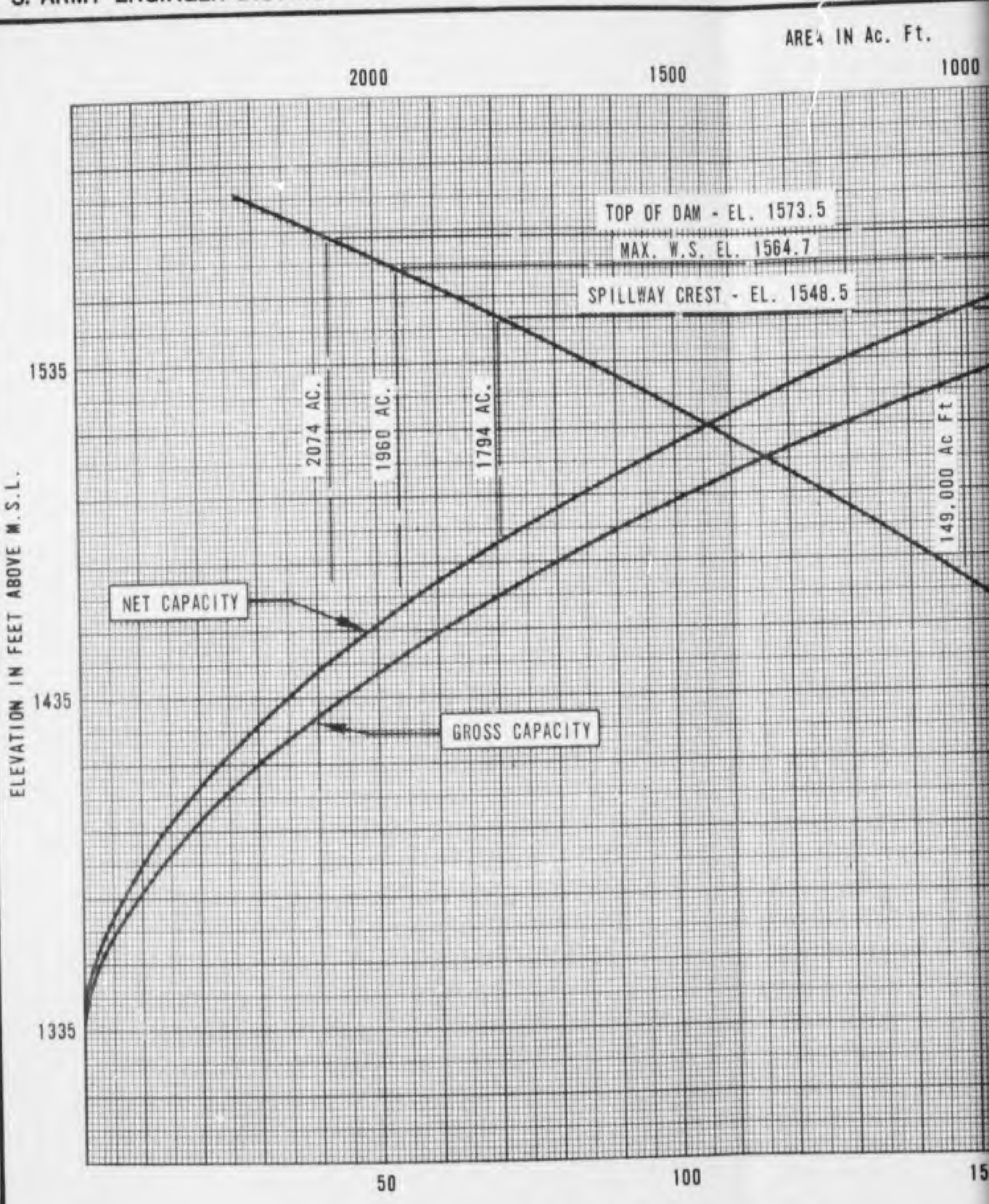
SANTA ANA RIVER, CALIFORNIA
PHASE I-GENERAL DESIGN MEMORANDUM

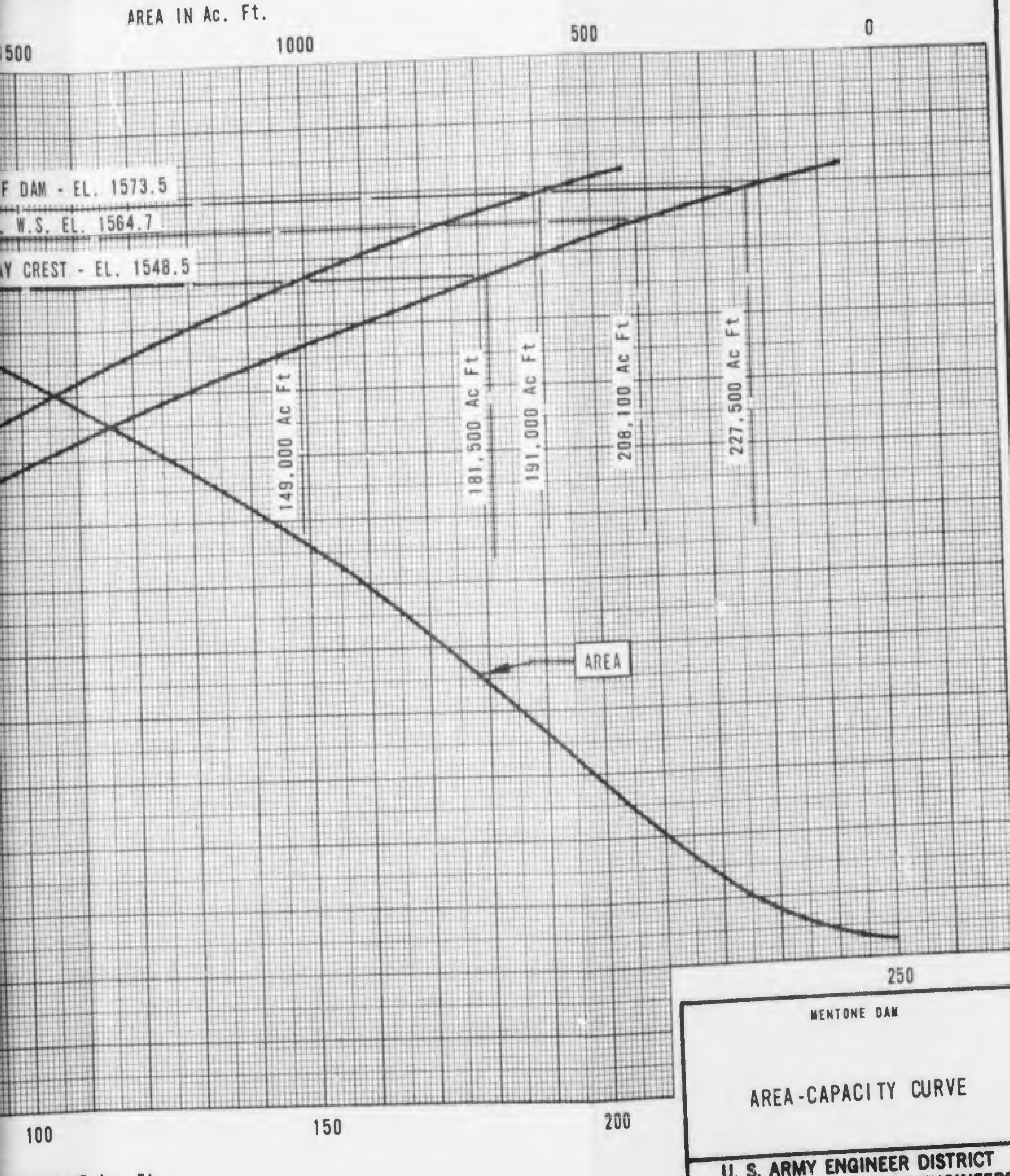
PROBABLE MAXIMUM
FLOOD ROUTING
SANTA ANA RIVER AT
MENTONE DAM

U. S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE C-2

U. S. ARMY ENGINEER DISTRICT





MENTONE DAM

AREA-CAPACITY CURVE

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

2

D Geology & Soils

APPENDIX D
SANTA ANA RIVER IMPROVEMENT
GEOLOGY, MATERIALS AND SOILS

APPENDIX D
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PLATES

Plates (Cont'd)

APPENDIX D
SANTA ANA RIVER IMPROVEMENT
GEOLOGY, MATERIALS AND SOILS

I. INTRODUCTION

Purpose and Scope

1.01 Limited geologic, soils and materials investigations have been conducted to determine the extent, distribution and physical properties of the rock and soils at the site of the proposed Mentone Dam, Prado Dam modifications and channel improvements along the Santa Ana River, Santiago Creek and Oak Street Drain. The preliminary investigations were conducted to obtain information on the foundation conditions, construction materials and ground water conditions in order to provide a technical basis for the phase I level conceptual design of the proposed improvements. The original studies were made in 1974 and updated in 1980. This appendix describes the geotechnical explorations, testing, seismicity, foundation conditions, preliminary design values, foundation treatments, embankment sections, and construction considerations. For this study conservative assumptions have been made in the absence of sufficient, detailed information that influence the design of a safe structure. In this context, a conceptual worst case for Metnone Dam was included as a comparative test for the benefit-cost ratio determination. As a result the associated cost estimate serves as an upper limit which may be modified downward in phase II, when detailed investigations and studies would be conducted to verify the basis of design used in this appendix.

Description of Project Features

1.02 The proposed project features consist of construction of Mentone Dam and appurtenant structures, modification to the existing Prado Dam, Santa Ana River channel improvements, Oak Street Drain improvements and Santiago Creek improvements. The project features are discussed in some detail in the following sections on each feature.

II. PRADO DAM

Project Description

2.01 For complete description of the project see the main report. Generally the proposed project requires enlarging Prado Dam and Reservoir to control the standard project flood, with a maximum release of 40,000 cubic feet per second, and would require, the acquisition of about 1,670 acres of land between elevations 556 and 566. The main features of the proposed plan for Prado Dam and Reservoir area are as follows:

a. Raise Prado Dam 30 or 43 feet to elevation 596 or 609, respectively. (See plate D-23.)

b. Stabilize the upstream toe foundation area for dynamic loading (see plate D-23).

c. Construct a new outlet works to more than double the existing outlet capacity.

d. Raise and widen the spillway.

e. Construct a containing levee on the south side of the proposed reservoir approximately along the Santa Fe railroad. The top of the levee would be at elevation 596 or 609. The levee would have a service road on top, revetted upstream and side slopes of 1V on 3H upstream and 1V on 2.5H downstream.

f. Construct ring levees to protect the Corona waste water treatment plant and the Alcoa Aluminum Plannt on Rincon Street in Corona.

g. Modify the interchange between the Riverside Freeway and State Highway Route 71.

h. Develop a recreational system consisting of an information center, an overlook area, wildlife areas, camping areas, a trailer camp, parks, day-use areas, three fishing lakes, picnic areas, agricultural buffer zones, and recreational trails.

Geology

2.02 Most of the geologic information was obtained from publications issued by other agencies. However, the Corps made local field investigations in the late 1930's for the design of the dam, in 1971 for possible spillway modification, and in 1972, 1974, and 1975 for additional studies of the dam and its foundation. These investigations consisted of (1) mapping the various geologic formations, and (2) subsurface exploration to determine the nature and extent of the soil and bedrock materials. A simplified map of the regional geology is shown on plate D-13; site geology is presented on plate D-14, and a more complete regional description is available on California State Division of Mines Geologic Map of California-Santa Ana Sheet, 1966.

REGIONAL GEOLOGY

2.03 Prado Dam is located at the tip of the Chino Hills (also known as the Eastern Puente Hills) in the head of Santa Ana Canyon. These hills are composed of Tertiary, Miocene, and lower Pliocene age (10 to 25 million years old) sediments called the Puente formation. The sediments consist mostly of friable sandstones with hard siltstone and shale interbeds and scattered lenses of conglomerate. The Chino Hills and the Puente Hills to the northeast are a structural unit that has been uplifted between the Whitter fault zone, which is near the southwest margin, and the Chino fault zone, which forms the northeast margin. Uplift of the region occurred during the past 2 to 3 million years (Quaternary time) and deformed the Puente formation with extensive warping and faulting. The warping generally trends northwest and southeast, parallel with the hills. Several very pronounced folds known as the Mahala anticline, Arena Blanca anticline, and the Arena Blanca syncline project through the Chino Hills near Prado Dam. Between the Whittier and Chino faults numerous minor faults exist, trending in two general directions, that is, northwest southeast or parallel to the main faults and the northeast southwest normal to the main faults.

SITE GEOLOGY

2.04 Prado Dam, spillway and outlet works are founded on the same Puente formation exposed in the Chino Hills, except the streambed portion of the dam, which is founded on alluvium. The foundation is composed of the formation's upper member known as the Sycamore Canyon. This member is locally characterized by white friable sandstones interbedded with brown very fine sandstones, conglomerates, and siltstones. The white sandstones contain very little matrix and are lightly cemented; in places they appear similar to a packed sand. When dry, the sandstones range in hardness from moderately soft to moderately hard, but lose much of their coherency when wet. The grain size ranges from fine to coarse, occasionally into the conglomerates, with little silt. The conglomerate lenses are numerous in the hills adjacent to the right abutment, but decrease markedly under the dam and spillway. The clasts are well rounded pebbles of hard granites and metamorphics derived from rocks in the San Gabriel and San Bernardino Mountains to the north. The interbeds of very fine sandstones and siltstones are moderately hard to hard and very competent. The brown coloring is due to a clay matrix that has bound together the larger silt and sand grains. Generally, the siltstones are less permeable than the sandstones and conglomerates. Under the dam, the sediments strike near parallel with the dam and dip 65 to 70 degrees upstream. This attitude is favorable in that the lower permeability siltstone layers form barriers to water seepage through the bedrock.

ALLUVIUM

2.05 Alluvium is present in two ages, Recent and older. Older alluvium is prevalent in irregular thicknesses throughout the existing spillway approach, in the terraces adjoining it, and as a capping on the ridges adjacent to the right abutment on the dam. The older alluvium is composed of poorly consolidated sands and gravels and irregular bodies of silt; boulders well over 12 inches in diameter are common. The sands are loose and extremely permeable and the silts are generally dense and tight. Recent alluvium is present in the Santa Ana River channel and the reservoir with minor amounts in the larger neighboring washes. Under the dam, the alluvium reaches a maximum thickness of about 80 feet and consists of sands and gravels with some silts and clays. The materials generally become coarser with depth.

Seismicity

2.06 Earthquakes in California have been frequent and occasionally destructive and are expected to continue. The prominent San Andreas fault can be considered as a boundary line in which the land west of it is drifting north relative to the east side. This drift builds up stresses throughout the region that are eventually relieved by movement along the San Andreas and other faults. The regional stress accumulation does not appear to be equally distributed among the faults. Besides the San Andreas faualt, Southern California is sliced by other major northwest-southeast trending faults such as the San Jacinto, Whittier-Elsinore, and Newport-Inglewood. Innumerable smaller faults exist among them, most of which are considerably less active or apparently not active.

2.07 The Whittier-Elsinore fault passes about 2 miles south of Prado Dam. Crossing the reservoir about a mile east of the dam is a fault called the Chino fault, which forms the eastern toe of the Puente Hills to the north and possibly joins the Whittier-Elsinore fault south of the dam. An even smaller fault is the Aliso Canyon, which trends parallel with the Whittier fault crossing the Santa Ana River about one-half mile downstream from the dam. The Scully Hill fault branches from the Whittier fault and follows the Santa Ana Canyon east to merge with the Aliso Canyon fault south of Prado Dam. Faulting under the existing and modified dam and its appurtenance are currently being studied.

2.08 Prado Dam is located in a highly seismic area and future earthquakes may be expected because of the activity of the major faults. To date, the strongest shock experienced by the dam was the 1971 San Fernando event. Between 1910 and 1974, 13 events of magnitudes 5.0 to 6.8 (Richter Scale) have originated within a 50-mile radius of the damsite, see plate D-15. Between 1934 and 1961, 15 earthquakes ranging from magnitude 4.0 to 5.0 have originated within 20 miles of the dam. A great possibility exists that the dam will experience an earthquake from the San Andreas fault and especially from the San Jacinto fault, 30 and 25 miles, respectively, further northeast. A magnitude 6+ earthquake occurs on the San Jacinto every 5 to 20 years and the fault is believed to be capable of generating a magnitude 7+ earthquake. Regional design earthquakes for the evaluation of the foundation and design of the modification to Prado Dam will be selected based on a magnitude 7+ on the San Jacinto, or 8+ on the San Andreas fault. Estimates of maximum ground acceleration occurring at Prado Dam from the events would be 0.25 g on the San Jacinto and 0.30g on the San Andreas.

2.09 A study was initiated (FY 80) to locate and determine the recency and magnitude of faulting along the Chino Fault system immediately east of Prado Dam. The recency and magnitude of faulting will be used in determination of the parameters for a local design earthquake to evaluate the dynamic stability of the embankment modification and foundation.

2.10 The possibility of an earthquake being generated by impoundment of water in Prado Reservoir has been considered. At the present state of the art, little is known on whether earthquakes occurring near a reservoir might be caused by increased fluid pressure, by crustal loading, or by both. It is known, however, that quakes have been reported in association with the filling of some reservoirs, and that damaging earthquakes of relatively large magnitudes have occurred mostly near large reservoirs. A large reservoir is defined as one with a volume of at least one million acre-feet, usually impounded behind a dam 300 feet or greater in height. Reservoirs that experienced earthquakes generally did so over a period of months as the reservoir filled, with the greatest tremor occurring about the time the reservoir was full to possibly 2 years later. Because Prado Dam is considerably less than 300 feet high and has a reservoir capacity of less than 225,000 acre-feet and relatively short pool storage duration, a major earthquake produced by the reservoir filling is unlikely. Should the reservoir become full, it would be drained as soon as practicable, thereby reducing the effect of the water on the foundation of the reservoir.

2.11 Because both Prado Dam and the site of the proposed Mentone Dam are located in an area of high seismicity traversed by active major faults, a board of eminent consultants in the fields of seismology and earthquake engineering was selected to advise to Los Angeles District. The board advised the Corps regarding the technical feasibility of the proposed concepts for design and construction of a flood control dam at the Mentone site and for modification of the existing Prado Dam. The Board of Consultants consisting of: Dr. Bruce A. Bolt, Professor of Seismology, University of California, Berkeley; Dr. Nathan M. Newmark, Professor of Civil Engineering, University of Illinois; and Dr. H. Bolton Seed, Professor of Civil Engineering, University of California, Berkeley, was convened in December 1973 to review available information conditions in the area of the Mentone Dam the Board of Consultants recommended that the seismic safety of Prado Dam and its appurtenant works be reviewed for earthquake motions associated with the Whittier and Chino fault systems as well as those associated with the San Jacinto and San Andreas fault systems. This recommendation led to the investigations of the dam to review the results of the investigations that had been made at both Prado Dam and the Mentone site subsequent to the December 1973 meeting. During this period following the December 1973 meeting, Dr. Clarence R. Allen, Professor of Geology and Geophysics, California Institute of Technology, was added to the board. The board's findings on the design concepts for both Prado Dam and the proposed Mentone Dam are presented in appropriate sections of this appendix. The formal reports of the board for both the December 1973 and January 1975 meetings are also included as attachments to this appendix.

Construction Data of Existing Embankment

GENERAL

2.12 The embankment is a rolled earthfill structure with a maximum height of 106 feet above streambed and a total volume of 3,900,000 cubic yards. The dam is composed of four zones: (1) a relatively thin upstream pervious zone, (2) an upstream random zone, (3) a central impervious core, and (4) a downstream pervious zone. Conservative downstream slopes were used to utilize the large volume of materials from the required spillway excavation. Underseepage is controlled by a steel sheet pile cutoff driven to refusal along the bedrock profile at the center of the dam.

FOUNDATION TREATMENT

2.13 During foundation stripping, layers of silt and clay were encountered, varying in thickness from a fraction of an inch to 14 inches. As stripping progressed, thicker layers were encountered near the left abutment, which required deeper excavation than specified over a large area. The area required an excavation some 20 feet deeper than the exploration trench (elev. 448). It extended from beneath the upstream toe to approximately 240 feet downstream from the axis and was limited between stations 16+75 and 22+00. Stripping in the remaining streambed areas underneath the embankment extended to elevation 454. Plate D-16 shows the extent of stripping and the depth of sheet pile along the axis of the dam.

EMBANKMENT MATERIALS

2.14 Materials used for the core zone were obtained from a borrow area upstream from the left abutment. The materials are well graded sandy clays and sandy silts. Pervious materials, obtained from the spillway excavation, are well-graded gravelly silty sands. Random materials, obtained from other required excavations and a designated borrow area northeast of the spillway, fall between the gradings of the above materials. Plate D-17 shows average gradations of the various embankment materials and plates D-18 and D-19 are plasticity charts for the random and core zones. A summary of the compactive effort for each zone and the compaction results obtained from construction records are presented in Table I.

Table I

Embankment Compaction and Density

Zone	Material	Compactive effort	Average dry density (pcf)	Average moisture (%)
1	U.S. pervious	4 passes of drum roller	130.1	9.3
2	Random	8 passes of sheepsfoot	124.5	10.9
3	Core	9 passes of sheepsfoot	118.6	12.9
4	D.S. pervious	4-6 passes of drum roller	127.4	9.8

All materials were placed in 6-inch layers, before compaction.

EMBANKMENT CONSTRUCTION

2.15 The embankment was constructed in three phases to facilitate diversion of water. The initial phase was the construction of about two-thirds of embankment length from station 8+00 (pl. D-16) to the left abutment up to elevation 525. After completion of the outlet works, the second phase or closure section was constructed meeting the initial phase top elevation. During the second phase the cofferdam was incorporated in the upstream portion of the embankment. The final phase completed the embankment construction.

Recent Investigation

GENERAL

2.16 After it was determined in 1969 that the spillway size was inadequate, preliminary field investigations of the existing spillway and of a proposed auxiliary site were made from December 1970 to February 1971. During 1972, explorations were initiated to obtain preliminary foundation data necessary for the modification of the embankment. These investigations disclosed the need for further evaluation of the foundation to address its response when subjected to seismic loading conditions. Subsequent investigations were conducted during 1974 and 1975. The locations of these borings are shown on plate D-20. In January and February 1975 and November 1979 investigations were conducted for sources of borrow material and for the foundation of the proposed dike east of the spillway. Locations of boring and trenches of borrow areas and dike foundation are shown on plate D-22. Testing for these latter studies has not been completed at this time. No field studies have yet been conducted for the outlet works modifications.

SPILLWAY

2.17 Explorations were oriented toward the alternatives of widening the existing spillway or constructing a new auxiliary spillway through the hills immediately west of the dam. Two 6-inch and one 4-inch diameter core holes were drilled along a line 200 feet upstream from the existing spillway crest. In addition, shallow refractive seismic surveys and backhoe trenches were made between the core holes and on the terrace to the east. Unconfined compression tests were made on selected cores from the borings. At the auxiliary spillway site, one 4-inch diameter hole and one 6-inch diameter hole were drilled in the crest area, and the local region was mapped. Logs of the core holes are shown on plate D-21.

EMBANKMENT AND FOUNDATION

2.18 Forty-two holes, 16 inches in diameter, were drilled during 1972 (TH72-1 through 15) and 1975 TH75-31 through 34, TH75-49 through 71) with a bucket-type power auger. The 1972 investigation focused on embankment materials and the foundation upstream from the dam toe. The 1975 investigation explored the foundation upstream and downstream from the dam toes, through the upstream berm, and through the downstream toe. In all borings penetration tests were conducted, and disturbed samples were obtained for mechanical analysis. Revert was used to support the test holes in loose zones and below water table.

2.19 Thirty-nine holes, 5-inches in diameter, were drilled during 1974, 1975, and 1976, with a Failing 1500 drill under three separate contracts. These investigations focused on obtaining undisturbed samples 3 inches in diameter and standard penetration test data. Drilling mud was used in all borings. The investigations are summarized in Table II.

Table II

Drilling Contracts

Period of Exploration	Test Hole No.
July - August 1974	9F - 16F
March - May 1975	17F - 35F
October 1975 - January 1976	37F - 48F

2.20 The July 1974 investigation was initiated to obtain undisturbed samples of foundation materials beneath and upstream from the dam toe for density determination and material classification. The March 1975 investigation was for the purpose of obtaining undisturbed samples for dynamic laboratory tests. The samples were taken from the embankment and from the foundation upstream from the dam and below the embankment near the upstream and downstream toes. During these explorations, the standard penetration tests were conducted when undisturbed sampling was not attempted. The October 1975 investigation was for the purpose of supplementing the standard penetration test data of the foundation beneath the central portion of the dam and for obtaining undisturbed samples at selected intervals to determine the density.

2.21 All undisturbed samples taken were 3 inches in diameter. Samples taken for density determination were obtained by a Hvorslev fixed piston sampler or Pitcher sampler; test samples for dynamic testing were obtained by an Osterberg piston sampler or a Pitcher sampler. In general, the Pitcher sampler was used to sample compacted embankment materials, foundation materials below depths of about 30 feet, or materials containing gravel. Hydraulic pressure was used to push the samplers, and the average drive pressure for each half-foot of advance was noted, to have a record of the driving resistance.

2.22 Geophysical investigations, consisting of surface-refraction seismic and crosshole surveys, were conducted during April 1974. Cross-hole surveys were made at eight locations from which preliminary P and S-wave velocities were determined. Surface refractive surveys were conducted at seven locations, including a line about a mile east of the dam, in an attempt to locate the Chino fault. Plate D-20 shows a plan of the seismic investigation. The results of the geophysical investigation are presented in US Army Engineer Waterways Experiment Station, Miscellaneous Paper 5-75-6, dated June 1975.

Results of Investigations

GENERAL

2.23 The results of the foundation and embankment investigation are presented in detail in the report titled "Prado Foundation Analysis," dated July 1976. The following describes the information gained from the foundation and embankment explorations, the laboratory and field testing, and the geophysical surveys.

SPILLWAY

2.24 Most of the spillway is founded on bedrock consisting of sandstone strata with some siltstone and conglomerat interbeds. In the approach channel, irregular thicknesses of older alluvium overlie the bedrock. The alluvium becomes even thicker under the terrace to the east. The alluvium generally consists of silts overlying loose sand and gravel with occasional cobbles to 12-inch diameter. The sandstones and conglomerates range from fine to coarse grained and are weakly to moderately well cemented. Generally, they are moderately hard when dry, but lose coherency and soften when wet. The siltstones comprise less than 50 percent of the strata around the spillway crest. They are relatively hard when dry and do not significantly lose coherency when wet. Unconfined compression tests on five sandstone and siltstone core samples indicated the sandstone had strengths ranging from 1.00 to 3.17 tons per square foot and the siltstones 14.4 to 19.5 tons per square foot. The auxiliary spillway would be founded on weakly cemented sandstones with some harder siltstones in the lower half of the alinement. No tests were made on the core samples, but visual inspection indicated the sandstones in the crest area are weaker than those in the existing spillway.

EMBANKMENT FOUNDATION

2.25 The investigations revealed the presence of a loose saturated sand layer between elevation 465 and 445 or about 10 to 30 feet below the ground surface. This layer is approximately 10 feet thick and underlies a portion of the upstream half of the dam. The comprehensive field and laboratory study, is presented in the report titled "Prado Foundation Analysis," dated July 1976. Evaluation of the field and laboratory data obtained to date indicates that the layer of sand is seismically stable under normal operating condition. However, should the dam be raised in height, the foundation conditions would be reanalyzed, in light of the current state-of-the-art, to evaluate the impact of the modified embankment. Additional field, laboratory and design studies are currently being conducted as part of the Prado Interim Project Report to evaluate the stability of the foundation material under the larger embankment (maximum) section being considered within this report. Any interim construction at Prado Dam will be structurally compatible with the overall Santa Ana River Project. Remedial construction may be carried out independently of this study and be initiated as soon as practicable.

EMBANKMENT

2.26 The results of recent investigations, and particularly the available construction control data, indicate that the existing embankment materials are dense and mostly coarse grained. These materials would yield minimal post construction settlement resulting from the load of the additional fill required for proposed modifications.

OUTLET WORKS

2.27 Although no exploration has been conducted, it is anticipated the outlet works would be excavated through sandstone and siltstone bedrock similar to that exposed in the spillway. These materials, though relatively soft, are competent and appear to be unbroken by faulting.

BORROW AREAS

2.28 Materials suitable for the enlargement of Prado Dam and for any required foundation modification would be available in sufficient quantity within the reservoir area east of the spillway, see plate D-22, and from required excavation from widening the spillway.

Technical Feasibility

EMBANKMENT FOUNDATION

2.29 The liquefaction potential of the loose layer in the foundation of Prado Dam can be corrected by controlling underseepage, and by densifying and increasing the confinement of the layer, see plate D-23. Corrective measures would be compatible with the recommended project modifications. The existing embankment and abutments provide satisfactory foundation conditions for the proposed modifications.

EMBANKMENT MODIFICATIONS

2.30 The proposed modifications require raising the dam crest 30 to 40 feet. This would involve placing about 1,300,000 to 2,100,000 cubic yards of embankment material on the crest and the downstream face of the dam. Preparation for fill would involve stripping the crest surface and excavating the downstream slope protection. Bond with the existing dam embankment would be accomplished by scarifying, moistening, and compacting. The cross section of the dam embankment and other proposed embankments would consist of materials obtained from the reservoir basin and required excavations. A conceptual section of the proposed enlargement is shown on plate D-23.

SPILLWAY MODIFICATIONS

2.31 In 1969, it was determined the spillway size was inadequate for a storm flood of greater than 70-year frequency. An estimate was made that peak inflow from a spillway flood (MPF) would be about 700,000 cubic feet per second and that an additional 1,200 feet of spillway crest was needed to pass this flow with the existing dam. The plan also considered constructing an alternate spillway around the right abutment in lieu of the widening and also the possibility of adding gates to the existing spillway. The investigations indicate that widening the spillway would be feasible. However, to found the proposed gates uniformly on bedrock, the alinement should be located near the existing crest. Gates designed for a 3-tons-per-square-foot loading should be stable. If an auxiliary spillway is planned, it would have to be fully lined because of the relatively soft bedrock. Also, the allowable loading for this structure should be no more than 2 tons per square foot.

LEVEE

2.32 The results of the foundation exploration for the proposed levee east of the relocated left spillway wall (pl. D-22), based on visual observation of materials sampled and blow counts of the penetration tests, indicate that the materials are uniformly dense. The foundation treatment would consist of excavating the top 5 feet of the foundation area and excavation of a 10-feet deep exploration trench. The proposed embankment would have a random cross section with an internal drain as shown on plate D-23.

Board of Consultants

2.33 At the January 1975 meeting, the Board of Consultants stated that the evidence available to them at that time did not suggest any major soils, geologic, or seismic problems that would make the proposed modifications to Prado Dam infeasible. They recommended further geologic and geophysical studies to verify this tentative conclusion as well as to permit the assignment of a realistic design earthquake. The board stated that the necessity for avoiding liquefaction of the materials underlying the present dam was of particular concern, and expressed the opinion that this problem could probably be controlled by remedial measures similar to those considered, or by appropriate modifications of such measures. The board recommended that further studies be undertaken of the relative reliability of the several measures proposed. Relative to the question of the possible triggering of earthquakes as a result of water impoundment behind the dam, the board indicated that the maximum depth of water to be stored behind Prado Dam and its short storage time does not appear to be consistent with those few cases where earthquakes have apparently been triggered by reservoirs. The complete report of the Board of Consultants for the January 1975 meeting is presented in attachment No. 2.

Construction Materials

2.34 Soils for the construction of the embankments would be obtained from within the reservoir area. Proposed borrow areas for embankment material are shown on plate D-22. Sound and durable soils with medium high shear strength and low consolidation characteristics for the construction of the outer shells and the transition zone would be obtained from designated borrow areas in the reservoir. Sources of material for the core similar to those of the existing embankment could also be obtained from the basin.

2.35 Other construction materials are available from commercial sources. Concrete aggregates, gravel and sand can be obtained commercially within a 5-mile radius. Rock for stone protection can be obtained from quarries within a 30-mile radius. Portland cement is available from plants within a 30-mile radius.

III. MENTONE DAM

Project Description

3.01 The Mentone damsite is located in the upper San Bernardino Valley just downstream from the Santa Ana River's junction with Mill Creek. (See plate D-24.) The primary function of Mentone Reservoir would be to collect floodwaters from Big Bear Lake, the upper Santa Ana River, Mill Creek. The waters would be detained 4 or 5 days until the high water level at Prado Reservoir had passed and would then be released slowly until the reservoir emptied.

3.02 Mentone Dam would be a horseshoe-shaped earth and rock fill embankment founded on alluvial materials. Construction of the dam would be combined with the improvement and strengthening of the Mill Creek levee. The levee would be lengthened about 1.2 miles, extending the levee into the reservoir area. To prevent an accumulation of debris near the spillway entrance, the levee is alined to direct floodflows and debris into the reservoir and away from the spillway.

3.03 The dam would rise about 230 feet above the streambed. Top of the dam would be at elevation 1,573.5. Spillway crest would be at elevation 1,548.5. The Mentone Reservoir would have an estimated gross storage capacity of 181,500 acre-feet for flood control including 37,000 acre-feet for debris storage.

3.04 Because of the proximity of the proposed embankment to the highly seismic San Andreas fault and considering the comments by the Board of Consultants in December 1973 on the foundation materials (see attachment No. 1), preliminary investigatons were conducted to explore the in-situ characteristics of the foundation soils in order to establish the technical feasibility of the project.

Topography

3.05 The upper San Bernardino Valley is 5 to 6 miles wide and is surrounded by the high and rugged San Bernardino Mountains on the north and eastside, and relatively low Crafton Hills on the southeast, and the also low "Badlands" on the south. Elevations range from approximately 1,300 feet at the site, to 11,502 feet on San Gorgonio Mountain 18 miles east, and to 3,543 feet on Zanja Peak in the Crafton Hills. The proposed embankment would span the westerly flowing Santa Ana River near the river's junction with Plunge Creek and Mill Creek; all originate in the San Bernardino Mountains. The Santa Ana Wash is about 1 mile wide at the damsite and has a gradient of about 100 feet per mile. The present channel is located on the south side of the wash, an area with many boulders, and has a cut bank 15 to 30 feet high. The remainder of the wash is covered with about 1/2 foot of soil and well established brush. Because the embankment would be totally on the flood plain, the abutment slopes would be long and gentle. The right abutment would be founded on terrace of older alluvium between the river wash and the mountain toe, which slopes about 70 feet per 1,000 feet. The left abutment would extend upstream from the left side of the spillway.

Geology

REGIONAL GEOLOGY

3.06 The region is composed basically of crystalline rocks and alluvial sediments derived from them. (See plate D-13.) The San Bernardino Mountains are made up of several varieties of igneous and metamorphic rocks; mostly quartz monzonite, quartz, diorite, and some schists and gneiss, all at least 65 million years old (Cretaceous age). The Crafton Hills are also composed of these schists and mylonites and other igneous and metamorphic rocks over 570 million years old (Pre-Cambrian age). The Badlands are composed of continental sediments about 10 million years old (Pliocene age). Materials eroded from these higher areas have coalesced to form the San Bernardino Valley floor. The combined fans of the Santa Ana River and Mill Creek at the damsite are the largest and most distinct in the valley. The San Bernardino Valley contains several alluvial units of Pliocene to Recent age. The oldest of these units is the consolidated Potato Sandstone located in the Mill Creek area. Progressively younger, the other units range through older alluvium and "plain and bench" deposits around the valley rim to younger alluvium with river channel deposits across the valley floor. In general, the recent alluvium is underlain by older alluvial deposits.

3.07 Similar conditions exist in other major ranges in Southern Californa. They are composed of similar cyrstalline rocks with detritus from them deposited along the toes. The most comparable fans are those along the south toe of the San Gabriel Mountains, especially west of San Bernardino. Near the apexes of the fans, the alluvium is very coarse with little stratification becoming progressively finer and more layered downstream. Since the tectonic environment is relatively the same in all these mountains, the sequence and distribution of older alluvial units is also similar to those emcompassing the Mentone damsite.

SITE GEOLOGY

3.08 The project area contains both crystalline rocks and alluvium primarily separated by the San Andreas fault along the toe of the mountains (see plate D-25). The dam would be located no closer than 0.3 miles south of the fault's most recent trace so that the embankment and its appurtenances would be founded totally on alluvium. The depth of alluvium under the site is unknown. In the Mill Creek area between the Crafton Hills and Mentone, well data indicate bedrock is a little over 100-foot deep, but drops off at the Mentone barrier about 2 miles upstream from the main portion of the dam. However, a review of available data indicates the alluvium lies on an irregular crystalline bedrock surface at least several hundred feet deep. Three observation wells numbered 581, 585, and 586 were drilled at the site in 1974, San Bernardino Valley Municipal Water District, to monitor ground water levels in response to nearby percolation ponds. The wells were drilled to depths of 240, 260 and 350 feet. It was reported that bedrock was not encountered in the drilling. However, a hard, tight and well-cemented layer, which may be weathered bedrock, was encountered below

235 feet in the 350-foot well. The alluvium is made up of two known age groups, recent and older. Recent alluvium is apparently more than 100 feet deep and is underlain by the older, possibly Pliocene age alluvium. The recent alluvium would form the foundation for the left abutment and streambed portions of the embankment. This alluvium consists of unweathered and unconsolidated materials, which are generally coarse sandy gravel. A large portion of the material is in the cobble and boulder sizes to 6-foot diameter with the sand portion generally well graded. These materials occur in layers of varying thickness and length with discontinuous small sand lenses and some concentrations of cobbles and boulders having occasional point to point contact. Although the younger alluvium contains little fines and the sand lenses tend to be low to medium dense, near vertical exposures are stable where exposed in trenches, pits, or natural channels. Gravel pits approximately 2 miles downstream from the site have near vertical walls over 100 feet high, with essentially no caving. The older alluvium becomes exposed at the ground surface north of the Santa Ana Wash and would be the foundation for the right abutment. The top of the older alluvium, in the vicinity of the wash, is characterized by a clay and gravel stratum and the remainder underneath is composed of highly weathered reddish-brown but consolidated clay, silt, sand, gravel, and boulders. Thick horizons of sandy and clayey material occur, separated by beds of poorly to moderately sorted sand and gravel. The older alluvium was deposited before and during the mid-Pleistocene time of major structural activity in southern California. As a result this alluvium is warped and cut by faults. However, no evidence of shears or offsets in the recent alluvium was observed in the exploratory exploration trench walls at the damsite nor in the gravel pits 2 miles downstream.

GROUNDWATER

3.09 Groundwater information available in the project area indicates that the groundwater conditions are somewhat complex. Because of the coarse, recent alluvium with high permeability rates, the region is highly sensitive to recharge. Groundwater contours generally follow the ground surface contours, see plate D-25A. Faults and barriers appear to affect them only slightly except at major boundries such as the San Andreas fault. The contours are relatively steep at the head of the Santa Ana Wash down to about the Mentone barrier. Downstream of the barrier to roughly 2 miles downstream of the dam site, the contours flatten slightly and then flatten further into an artesian area caused by the San Jacinto fault.

3.10 In general, the Southern California weather is typical of semi-arid regions. There are no definitive recurrence patterns of wet and dry cycles or periods of specific durations. The average precipitation per year in Los Angeles is approximately 15.06 inches and averages 16.54 inches in the Mentone area. The wettest year of record occurred in the 1883-84 winter, with 38.18 inches of precipitation in Los Angeles. The driest year of record in Los Angeles occurred in the 1960-61 season with only 4.85 inches of precipitation. One of the wettest periods occurred during 3 consecutive years 1977-80 with a total of 70.06 inches for the 3 years.

3.11 The groundwater levels in the Mentone basin reflect the wet and dry periods. Six key wells in the project area are examples of this. Three irrigation wells, numbered 3600604, 3601586 and 3600680 have been monitored since 1938 and the 3 observation wells since 1974, see Plate D-25A for locations. A review of the groundwater levels from these wells indicate that in the period 1938 to 1977, the water table was greater than 75 feet deep, even after the winter of 1938-39, see Plate D-25D. In early 1980, during the period when more than 70 inches of rain fell, water levels rose to within 10 feet of the ground surface, see Plate D-25B and 25C. Although the data for these wells was not available beyond June 1980, more recent levels in other wells within the basin indicate the water table peaked in May-June and appears to have declined several feet during the next few months. To summarize it can be stated that normally, groundwater at the Mentone Dam site is deeper than 100 feet and for extremely wet seasons, the water table rises nearly to the ground surface.

3.12 Withdrawls from the groundwater basin are causing only slight subsidence in the valley, particularly in the San Bernardino-Colton area about 10 miles west of the site. Because of the high recharge and light pumping at the site, subsidence there has been minimal. According to USGS data, subsidence at the site was estimated to be about 0.02 feet per year between 1944 and 1956. This subsidence rate would pose no adverse impact to the structures being considered at this site.

SEISMICITY

3.13 A review of the geology and seismicity of the region shows conclusively that the Mentone dam site is in a zone of high seismic hazard. The San Andreas fault is a dividing line, which separates two major plates of the earth's crust. The plate to the west is known as the Pacific plate and the plate to the east of the San Andreas is the North American plate. These plates, and others forming the land areas on the surface of the earth are in motion, slowly slipping past and towards one another. The slip rate along the San Andreas fault is estimated between 1/2 and 2 inches per year. This slip causes regional shear and compressional strain build-up, which is relieved by occasional sudden movements along the fault or along major associated faults, such as the San Jacinto, Newpoert-Inglewood, and San Fernando. Movement on the San Andreas was responsible for the Fort Tejon earthquake (estimated magnitude 8.3) in 1857 and the disastrous San Francisco earthquake (magnitude 8.3) in 1906. In these events the ground was displaced approximately 21 feet during the San Francisco earthquake and reported up to 30 feet along the Fort Tejon event. The vertical displacement was in the order of 5 feet. The length of ground rupture was about 200 miles and, in the Fort Tejon event, may have extended into the San Bernardino area. Because of the estimated 1/2 to 2-inch-per-year slip rate, other geologists have determined that a recurrence of an 8+

earthquake is possible somewhere on the San Andreas every 100 to 200 years. In the 20th century, movement along the fault has been more active in northern and central California than in the southern reach that includes San Bernardino. Most seismic activity in this reach has occurred along its major branches, such as the San Jacinto fault, possibly relieving some stress on the San Andreas. Since at least the turn of the century, a magnitude 6+ earthquake has occurred on the San Jacinto every 5 to 20 years. In July 1923 a magnitude 6.3 earthquake occurred on the San Jacinto fault near Box Springs Mountains about 10 miles southwest of the Mentone site. The last significant event on the fault was a magnitude 6.4 earthquake near Borrego Mountain in April 1968. The seismicity of the area in recent years is shown in table III and on plate D-26. Between 1974 and 1980 only 5 events greater than magnitude 4 occurred within a 50 mile radius of the site and these were less than magnitude 5; only one event was centered near the San Andreas fault.

Table III

Summary of local earthquakes between 1934 and 1974

Magnitude	Radius from site (miles)	Total number within radius
2.0 to 2.9	10	141
3.0 to 3.9	10	59
4.0 to 4.9	20	23
5.0 to 5.9	50	16
6.0 to 6.9	100	8
7.0+	150	1

3.14 It can be expected that a major 8+ earthquake will occur on the San Andreas fault during the life of the Mentone Dam. The most recent trace of the fault, which is the southern branch located about 0.3 miles beyond the north end of the proposed dam, is the most likely site of future rupture developing horizontal ground displacements of up to 20 feet. Future movements can be expected to occur within a few feet of the single main trace. Although displacement would be primarily horizontal and right lateral, vertical displacement of several feet could also occur. Lesser faults branching from the San Andreas fault or subsidiary to it are not positively known to exist under the proposed dam but are inferred from groundwater data in which barriers in the older, deeper alluvium have formed. The faults dissect the valley floor in two general directions, parallel with the San Andreas and near normal to it, (see plate D-25.) Although none of the faults appear to underlie the damsite, the existence of faulting there is a real possibility. Since seismicity has been comparatively low at the site during the past 7,000 to 10,000 years, none of the splinter faults are known to extend into the recent alluvium overlying them.

3.15 The possibility of an earthquake being generated by temporary impoundment of water in the Mentone Reservoir has been considered. In the present state-of-the-art, little is known whether earthquakes occurring near a reservoir might be caused by increased fluid pressure, by crustal loading, or by both. It is known that quakes have been reported in association with the filling of some smaller reservoirs and lower dams. However, the damaging earthquakes of relatively large magnitudes have occurred near large reservoirs. A large reservoir is defined as one with a volume of at least 1,000,000 acre-feet, usually impounded behind a dam 300 feet or greater in height. Those reservoirs that have experienced earthquakes generally did so over a period of months as the reservoir filled, with the greatest tremors occurring about the time the reservoir was full to possibly 2 years later. Because Mentone Dam would be a flood control structure with a reservoir capacity of only 181,500 acre-feet, it is unlikely that a major earthquake would be produced by the reservoir filling. Also, should the reservoir become full, current plans are to drain it in about a month's time, thereby quickly reducing the effect of the water on the reservoir's foundation.

Investigations

GENERAL

3.16 The field investigations for this feasibility report consisted of foundation explorations and geophysical surveys. The following describes the type and extent of the investigations.

FOUNDATION INVESTIGATIONS

3.17 Preliminary reconnaissance at the project site and review of investigations by others disclosed a large excavation for concrete aggregate in the streambed of the Santa Ana River downstream from the proposed dam site. The excavation is several acres in area and approximately 100 feet deep. The materials appear to be uniformly dense and stand nearly vertical for the entire depth of the excavation. Logs of the three observation wells, drilled by others in 1974, to depths ranging from 240 to 350 feet, disclosed that the streambed alluvium is very pervious and extends to variable depths of at least 240 feet. A review of a report by others of a materials investigation in the streambed near the foundation of the proposed dam showed that two pits excavated in November 1973 by clam shell, to depths of 30 to 80 feet, encountered recent alluvium classifying mostly as sandy gravel with large amounts of cobbles and boulders.

3.18 Based on the visual observations made possible by these large-scale excavations and on the well data, it was decided to limit the scope of this phase I level investigation to determining the gradation, in-place density, relative compaction, and specific gravity of the upper 50 feet of the foundation. It was tentatively assumed that the material below the 50-foot depth would be as dense or denser based on the knowledge of their deposition. Later, when sand lenses were exposed in the trench excavations, additional studies were made to determine the extent, gradation, and density of the sand lenses.

Field and Laboratory Tests, Streambed Alluvium

3.19 To determine the gradation and in-place density of materials in the foundation of the proposed embankment, excavations were made in May and June 1974 at three representative locations of the streambed. For this feasibility study it was determined that 3 trenches, TT-1 through TT-3, excavated approximately 4000 feet apart along the alinement would be adequate to sufficiently evaluate the types of materials on which the embankment would be founded. This was based on the district's experience that the materials within alluvial cones, at the base of the San Bernardino Mountains are relatively consistent in terms of geology, depositional environment, soil stratification and range in material gradations. See plate D-27 for the Plan of Exploration and plates D-28 through D-30 for soil logs. Excavations were made first by excavating to approximately a 25-foot depth with a D-9 dozer. Large-scale in-place density test were conducted at approximately 5-foot depth increments. A steel ring, 8 inches high and 4 feet in diameter, was used to obtain in-place densities of the alluvial materials. Materials were excavated to a depth of about 18 inches. Approximately 1,270 pounds of material were excavated for each density test. A plastic sheet was then placed in the hole and over the ring and filled with water to determine the volume. The moisture content of the total sample was obtained for each test to determine the in-place dry density.

3.20 A shaft was excavated by clam shell below the bottom of each trench to a final depth of about 50 feet. At 5-foot depth increments in each shaft, an 8-foot diameter casing was installed, equipment and personnel lowered into the shaft, and an in-place density test was taken. The equipment and method of taking densities were the same as those described previously. The average diameter of the shafts was about 12 feet.

3.21 The minus 6-inch material obtained from each field density test was used to determine the maximum density in accordance with the State of California Department of Water Resources (CDWR), maximum vibrated density test method. In this test the plus 6-inch material was removed from the sample and replaced with an equal weight of plus 3 to minus 6-inch material. A sample of approximately 500 pounds was placed in a 27-inch-diameter by 30-inch-high mold. A calculated amount of water to almost saturate the sample was added during the placement, the surface leveled, and a surface plate placed prior to placing a 2-psi surcharge. The mold was then vibrated at approximately 6,000 vibrations per minute for exactly 15 minutes. Sample thickness was measured before and after the test.

3.22 A few tests were also conducted in accordance with the above method on minus 3-inch material from the above test to correlate the results of the test to the standard maximum density test using the same materials. The minus 3-inch material from each field density test was used for detemination of the maximum and minimum density in accordance with Appendix XII, EM 1110-2-1906, Laboratory Soils Testing, using the 0.5 cubic-foot mold, vibrated for 8 minutes at 3,600 cycles per minute with a 26 psi surcharge.

3.23 During the excavation of the trenches and shafts, the inspector visually classified the materials encountered and collected disturbed samples for laboratory classification tests. All materials from in-place density tests were graded and classified. The total sample of minus 6-inch material was graded using large vibrating screens.

Field and Laboratory Tests, Sand Lenses

3.24 When the exposed materials on the sides of the three trenches were examined, it was disclosed that several discontinuous lenses of clean, relatively loose sands existed. It was decided to increase the investigation to include the determination of in-place density, relative density, gradation, moisture content, and the extent of these layers of sand and some gravelly sands. In excavating for these layers of materials, it was disclosed that their extent is quite limited in thickness, length, and width. The majority of the layers exposed on each side of the trenches could not be traced sufficiently into the side of the trench to be able to test for in-place density.

3.25 The in-place density of the sand layers was determined by the standard sand cone test and/or by excavating a larger pit up to 1.5 cubic feet and using plastic sheet and water to determine its volume.

3.26 The laboratory maximum-minimum density tests were conducted in accordance with Appendix XII, EM 1110-2-1906, Laboratory Soils Testing, using the 0.1-cubic-foot mold. The maximum particle size was less than 1-1/2 inches.

3.27 The materials excavated during the density test were visually classified, and samples were collected for laboratory classification tests. All materials from in-place density test were graded and classified.

Crosshole Seismic Survey

3.28 Two sites, designated G-1 and G-2, were selected in the vicinity of test trenches TT-1 and TT-2, respectively, see plate D-27. The purpose of the investigation was to determine the P and S-wave velocities, as a function of depth below the ground surface. Three holes were drilled along a straight line at each of the two sites. The end hole was used as the shot hole; the other two holes, located 20 and 120 feet from the shot hole, were used as receiver holes in which geophones were placed. To keep the borings from caving, it was necessary to insert plastic casing. The test holes were drilled to depths of 92 to 107 feet in May 1974 with a Becker 180 drill rig. Each hole was surveyed to measure the deviation and drift.

3.29 Crosshole surveys were conducted using the Dresser SIE refraction unit modified to accept calibrated triaxis geophones. The geophones were placed in two boreholes at the same elevation as the energy source, which was placed in the third hole. The energy source for the crosshole survey at site G-1 was an electric blasting cap with a 25-gram dynamite booster. At site G-2 two tests were made. The source of energy for one test was explosives and for the other, a borehole vibrator. The crosshole surveys conducted with borehole vibratory source used the same geophones and seismic unit that were used in the conduct of the other crosshole surveys. The borehole vibratory source was a 25-pound electromagnetic vibrator mounted on aluminum tubing that extended down the borehole and attached to a geophone (sender). Another geophone (receiver) attached to the aluminum box member was placed in the borehole 20 feet away at the same elevation as the sender. Prior to recording data, a manual frequency sweep from 20-400 Hz was used to select the best propagation frequency for that particular combination of layer thickness, layer velocity, material type, and mechanical coupling. Having selected a frequency, the electrical input to the vibrator was gated with a toneburst generator allowing a period of no vibration followed by several cycles of vibration. The resulting oscillatory wave train was recorded from each geophone. The two geophones were then moved to the next elevation to be tested and the above procedure was repeated until all desired elevations had been tested.

Refractive Seismic Survey

3.30 Seismic surveys, designated G-1, G-2, and G-3 were conducted in the vicinity of test trenches TT-1, TT-2, and TT-3, respectively, see plate D-27. The purpose of the investigation was to determine zones of true velocities in the foundation of the proposed dam and to correlate velocities with exposed materials in the test trenches. A refraction seismic line 1,450 feet long was conducted at each site.

Results of Investigations

GENERAL

3.31 Complete results of the investigations, laboratory and field tests are available in the Los Angeles District's Foundation and Materials Branch files. Complete results of the geophysical investigation is available in the report titled; "Geophysical Investigation, Prado Dam and Mentone Damsite" Miscellaneous Paper 5-75-6 dated June 1975 by US Army Engineer Waterways Experiment Station. The following is a summary of the results.

FOUNDATION INVESTIGATION

Field and Laboratory Tests, Streambed Alluvium

3.32 The minus 6-inch materials typically are sandy gravels with 3 percent or less fines, 40 percent sand and 57 percent plus No. 4 size. The material contains varying amounts of cobbles and boulders. It is estimated that approximately one-third of the alluvial material is larger than 3 inches. Close inspection of the sides of the trenches and of the shafts excavated in the bottom of the trenches indicates that some point to point contact of the cobbles and boulders exist. There was no evidence of nesting of cobbles or of open graded gravels. A total of 33 gradations and in-place density test were taken in the three test trenches. The results are shown on plates D-31 through D-33. A summary of the data is shown in table IV.

3.33 Logs of materials along various locations on the sides of the three test trenches are shown on plates D-28 through D-30. The gradations of the materials from each density test are presented on plates D-31 through D-33.

Table IV

In-Place Dry Density

	In-place dry density (pcf)
Upper quartile	138
Median	133
Lower quartile	128

3.34 A maximum density test (CDWR) was conducted on each material from the 33 density tests. The results are shown on plates D-31 through D-33. A summary of these data is shown in table V.

Table V

Laboratory Max. Dry Density

	Lab. max. dry density (pcf)
Upper quartile	140
Median	134
Lower quartile	131

The above densities were determined in accordance with the CDWR vibrated density.

3.35 A total of 18 maximum-minimum density tests (ASTM) were conducted on representative materials from the 33 in-place density tests. A summary of the data is shown on table VI.

Table VI

Minus 3" Laboratory Max Dry Density

	Lab. max. dry density (pcf)
Upper quartile	139
Median	137
Lower quartile	130

The above densities were determined in accordance with Appendix XII of EM 1110-2-1906 using the 0.5-cubic-foot mold on minus 3-inch material.

3.36 The relative compaction of the materials at the site of the 33 tests computed using the CDWR test data as maximum is presented in plates D-31 and D-33. A summary of the data is shown in table VII.

Table VII

Relative Compaction

	Relative compaction percent of maximum
Upper quartile	100
Median	98
Lower quartile	95

The relative density of the same materials at the site of the 16 tests, using the maximum-minimum test on minus 6-inch material, are summarized in table VIII.

Table VIII

Relative Density

	Relative density percent
Upper quartile	116
Median	71
Lower quartile	62

Field and Laboratory Tests, Sand Lenses

3.37 The investigation for the sand layers disclosed that their extent was limited in thickness, length, and width. It is estimated that approximately 50 percent of the layers exposed on the sides of the trenches were less than 2 feet thick and less than 5 feet wide.

3.38 A total of 36 in-place density tests and gradations were taken in the top 25 feet of the three test trenches. The results are shown on plates D-34 through D-36. A summary of these data is presented in table IX.

Table IX
In-Placo Density

	In-place dry density (pcf)
Upper quartile	130
Median	103
Lower quartile	99

3.39 The results of 36 maximum-minimum density tests and computed relative densities are presented on plates D-34 through D-36. A summary of these data is presented in table X.

Table X
Relative Density

	Relative density percent
Upper quartile	69
Median	60
Lower quartile	40

Crosshole Seismic Survey

3.40 For site G-1, near TT-1, the true P and S-wave velocities are shown alongside each geophone position on plates D-37 and D-38. The P-wave velocities generally indicate an increase with depth ranging from a low of 1,578 fps near the surface to 2,885 fps at a depth of approximately 95 feet (elevation 1,290). The S-wave velocities also show a general increase with depth ranging from 707 fps at a depth of 30 feet (elevation 1,356) to 1,618 fps at a depth of approximately 95 feet (elevation 1,290).

3.41 For site G-2, near TT-2, the true P- and S-wave velocities are shown on plates D-39 and D-40. The P and S-wave velocities indicate an increase with depth except near the bottom of the holes at approximately elevation 1,340. The S-wave velocities obtained from the two types of crosshole tests were averaged to obtain the average true velocity at each test elevation. The average S-wave velocities range from 509 fps at a depth of 20 feet (elevation 1,410) to 993 fps at a depth of 70 feet (elevation 1,360); where it decreased to 815 fps at a depth of 80 feet (elevations 1,350 and 1,340). The P-wave velocities in general increase with depth from a low of 960 fps at a depth of 20 feet (elevation 1410) to a high of 1920 fps at a depth of 80 feet (elevation 1350.

Refractive Seismic Survey

3.42 Subsurface profiles construced from results of the surface refraction seismic survey are shown on plates D-41 through D-43 for sites G-1, G-2, and G-3. For site G-1, the profile for the P-wave shows three true-velocity zones. The near surface zone indicates a velocity of 2,000 fps and extends to depths ranging from 13 to 26 feet. The second zone has a velocity of 3,200 fps and extends from below the near surface zone to depths ranging from 134 to 146 feet. The third zone has a velocity of 7,200 fps and extends to an undetermined depth, probably in excess of 400 feet.

3.43 For site G-2, the profile for the P-wave shows four true-velocity zones. The near surface zone indicates a velocity of 2,000 fps to depths ranging from 11 to 14 feet. The second zone has a velocity of 3,500 fps and extends from below the near surface zone to depths ranging from 84 to 112 feet. The third zone has a velocity of 5,150 fps and extends to depths ranging from 232 to 241 feet. The fourth zone has a velocity of 9,250 fps.

3.44 For site G-3, the profile for the P-wave shows three true-velocity zones. The near surface zone indicates a velocity of 2,000 fps and extends to depths ranging from 9 to 18 feet. The second zone has a velocity of 3,000 fps to depths ranging from 160 to 173 feet. The third zone has a velocity of 7,600 fps and extends to an undetermined depth, probably deeper than 400 feet.

Geophysical Survey Summary

3.45 The geophysical survey results indicate that the velocity zones under sites TT-1 and TT-3 are almost identical. The 2,000 to 3,000 fps P-wave velocity materials extending to 134 and 173 foot depths are possibly recent alluvium of similar character as exposed and tested in the excavations. The 7,200 to 7,650 fps P-wave velocity materials under the upper zone could be the more dense older alluvium. However, no positive crystalline bedrock surface could be identified at either site. At site TT-2, the upper zone is similar to that at the other two sites to a depth of 112 feet. The 5,150-fps P-wave velocity below this upper zone is indicative of the ground water table, which is in line with a measured depth of 125 feet in an observation well nearby. The 9,250 fps P-wave zone below a depth of 241 feet could correspond to the bedrock. If this is bedrock and none was encountered at sites TT-1 and TT-3 to at least 400-foot depths, it appears the bedrock surface is irregular or faulted under the proposed dam.

Evaluation of Dam Site Feasibility

GENERAL

3.46 The design will consider a normal groundwater condition and various measures to keep the near surface foundation material from becoming saturated by the reservoir. However, to demonstrate technical feasibility this evaluation conservatively assumes complete foundation saturation resulting from the water table being at the ground surface. For that condition it becomes apparent that the density and gradation of the foundation materials have a major impact in the embankment stability assessment. To demonstrate technical feasibility for this study the district considered the two following alternatives:

a. Perform additional field testing to verify the in-situ relative density, permeability and gradation to show they are sufficient to nullify the concern about potential liquefaction, or

b. Demonstrate with existing data that the foundation materials are relatively uniform, adequately dense and pervious enough to safely support the proposed dam under strong earthquake motions that may develop at the site.

3.47 A review of site specific information combined with other available data, technical literature, and case histories indicated that the second alternative would be adequate and efficient. The following discussion presents the data and rationale to support the adequacy of the foundation materials at the Mentone Dam site for the above groundwater conditions.

EMBANKMENT FOUNDATION

Gradation

3.48 A review of the gradation data of the streambed materials from the three trenches tested during the investigations conducted in May and June 1974, indicate the materials do not vary significantly from trench to trench, see figures 1, 2, 3, and 4. Because of the distance between the test trenches, up to 5000 feet apart, and the similarity of the materials from trench to trench, the gradations obtained in the three test trenches are considered representative of the streambed alluvium to a depth of 50 feet within the embankment foundation. Gravel pits, located approximately a mile downstream of the site, expose the alluvium to depths up to 100 feet. The alluvial materials at the gravel pits and the proposed dam site are similar in terms of gradation and stratification to known depths of 50 feet. The similarity is based on visual inspection, the materials originating from the same source and the materials were deposited in the same depositional environment. It is, therefore, reasonable to assume the foundation conditions at the proposed dam site are comparable to the gravel pits for a depth of at least 100 feet. No conditions were encountered in the test trenches or the existing pits which would indicate anything other than consistency of foundation conditions.

3.49 An evaluation of additional gradation data on materials with similar depositional environments, from district projects located at Cucamonga Creek, Santa Fe Dam, San Antonio Dam and Deer Creek, indicate that the alluvium of the Mentone Dam site is not unique. The average gradations of materials from these various project sites are plotted in figure 5. The gradations indicate that materials deposited under intermittent, high velocity stream flows are in general very coarse and consistent from alluvial cone to alluvial cone.

3.50 Therefore, it can be concluded that the foundation materials at the Mentone Dam site, to depths of more than 50 feet and perhaps 100 feet consist of relatively coarse grained sandy gravels and gravelly sands containing large amounts of cobbles and boulders, up to 57 percent by weight, with a maximum size of about 36-inches.

Density

3.51 The in-situ density data obtained in the three test trenches up to depths of 52 feet are summarized in figure 6. The results indicate that the foundation materials, in general, become denser with depth. The average densities are 123.7 pcf from 0 to 10 feet, 130.8 pcf from 10 to 35 feet and 137.7 pcf from 35 to 52 feet. The density vs depth plot, figure 6, indicates the densities in the foundation are, in general, uniform and increase with depth in each trench. The densities are representative of the streambed alluvium to a depth of 50 feet within the embankment foundation. It is therefore, reasonable and conservative to assume that the average densities would be at least 138 pcf within the embankment foundation below depths of 50 feet.

3.52 These in-situ density data indicate that foundation materials are at an average relative compaction and relative density of 98 and 70 percent, respectively. Even though state-of-the-art techniques were used to determine these values, there is some uncertainty in the results of the laboratory maximum and minimum density. This is due to the sensitivity of obtaining the minimum density and the difficulty of overcoming bulking of the sample in the mold during the maximum density tests. Improved precedures to determine maximum and minimum densities are now available and will be used in the design phase to determine in-situ conditions.

3.53 To obtain additional information on the density of the alluvial materials at Mentone site, density data was obtained from district construction projects having materials with similar gradations. The density data are summarized in figure 7 and indicate that coarse grained materials can be compacted to an average density of about 138 pcf based on actual construction experience with similar materials. The average in-situ density (137.3 pcf) below a depth of 35 feet indicates that the foundation materials are probably as dense as present construction methods can compact them. This construction information further varifies the field and laboratory test results and supports the conclusion that alluvial outwash materials of this type are naturally dense.

Liquefaction Potential

3.54 A review of the data on gradation and density of the foundation materials at the Mentone Dam site indicates that the materials are coarser and denser than alluvial deposits of the kind that literature indicates in the past have shown evidence of liquefaction. Even if there were a tendency for excess pore pressure development, there should be less concern about catastrophic liquefaction based on the following:

a. The foundation materials are coarse grained cohesionless soils with relatively high permeabilities. The U.S. Geological survey has estimated from percolation studies the permeability of the alluvium at 190 feet/day. Calculated permeabilities, which seem to be high, using a relationship developed by Justin, Hinds and Craeger, see figure 8, ranged from 110 feet/day to 4800 feet/day. Field permeability tests, using a constant head and falling head stand pipe procedure, were conducted by this district in foundation materials with similar gradations at Santa Fe and San Antonio Dams and indicated permeabilities of 9 to 390 feet/day. The average measured field permeability at Santa Fe and San Antonio Dams was 29 feet/day and 76 feet/day, respectively. Because of the relative coarseness and relatively high permeabilities, the foundation materials would be subjected to partial drainage under seismic loading. This partial drainage would increase the dynamic strength of the soil over the undrained dynamic strength.

b. Coarse grained relatively dense cohesionless soils tend to develop less strains (Wong 1971), (Banerjee, Seed and Chan 1979) than the kind of alluvial deposits that have historically shown evidence of liquefaction. The foundation materials at Mentone site, under dynamic loading, may at the very worst, have the potential to develop a condition of initial liquefaction with limited strains. "Initial liquefaction with limited strain potential," as described by Seed et. al. 1975, "denotes a condition in which cyclic stress applications develop a condition of initial liquefaction and subsequent cyclic stress applications cause limited strains to develop either because of the remaining resistance of the soil to deformation or because the soil dilates, the pore pressure drops, and the soil stabilizes under the applied loads". Tests conducted on Oroville gravels (Wong et. al., 1974 and Banerjee et. al. 1979) indicate limiting axial strains, on samples at 60 to 84 percent relative density, ranged from ± 2.5 percent to over ± 10 percent.

c. Previous strain history from earthquakes dynamically loading the foundation materials at the site may have increased the resistance of the foundation materials to liquefaction. Studies conducted by Lee and Focht (1975) indicated that previous strain histories led to an increase in cyclic strength for dense sand by a factor of at least 1.5.

3.55 Due to the above three reasons, there are few known instances of liquefaction-type phenomenon or large deformations recorded in gravelly soils. A study reported by Wong et. al., 1974, during the Alaskan earthquake of 1964, see figure 9, on movements of bridge foundations in the same areas indicate large movements of foundation founded in sandy soils with much smaller movements for foundations founded on gravelly soils. A plot of case history data for stress ratio's causing conditions of liquefaction and non liquefaction in the field and maximum ground acceleration is shown on figure 10. The data indicate that the gravels did not liquefy under loading conditions which caused liquefaction in sands. The one possible exception was at Valdez during the Alaskan earthquake of 1964, where sands and gravels liquefied. The depositional environment (glacial soils were deposited through water) and nonuniformity of material (alluvium consisted of sand, silt and gravel in lenses and layers) and layers of silts and sands were the probable cause of liquefaction (Seed, 1968) not the gravel.

3.56 In summary, historical and recent test data (Wong et. al. 1974 and Banerjee et. al. 1979) indicate that coarse grained, cohesionless, relatively dense foundation materials of the type at the Mentone Dam site would not undergo catastrophic liquefaction. The coarse grained materials would probably develop limiting dynamic strains on the order of ± 3 to 5 percent.

FOUNDATION TREATMENT

3.57 Should detailed investigations, to be conducted during the design stage, indicate that the foundation materials are even marginal, in terms of grain size or density, they would be improved by removal or compaction, see plate 48. Available site specific density data indicate the materials are very dense below a depth of 35 feet. But were this not the case it would be economically feasible to remove material down to at least 50 feet. Additionally marginal pockets, remaining below that depth, would be compacted for at least another 50 feet by dynamic compaction procedures (Leonards et. al. 1980). Removal and compaction of loose materials would assure a dense foundation beyond depths to which liquefaction has historically been known to occur or it is reasonable to postulate it could occur.

ASSUMED FOUNDATION LIQUEFACTION

3.58 To study the impact of complete liquefaction of the foundation materials to a depth of 100 feet, see figure 11, on the stability of the embankment, a static slope stability analysis was conducted (see figs. 12 and 13). The analysis indicates that the minimum factor of safety would be 1.2. The analysis is grossly conservative because there could not be complete loss of strength in the materials nor could they liquefy to the 100 foot extent shown in figure 11.

FOUNDATION FEASIBILITY SUMMARY

3.59 The foundation alluvium at the Mentone Dam site provides a suitable foundation on which to build the embankment for a flood control dam. Catastrophic failure with release of water can not occur as the result of liquefaction of the foundation under seismic loading because of the following reasons:

a. The foundation materials are relatively coarse and dense.

b. The foundation materials are subject to partial drainage during dynamic loading.

c. The foundation materials are only susceptible to initial liquefaction and have limited strain potential (3 to 5 percent).

d. Historical data do no indicate a liquefaction potential for alluvial materials similar to the foundation materials at Mentone dam site.

e. Assuming liquefaction of the foundation to a depth of 100 feet, with no strength in the liquified zone, results in a calculated static slope stability factor of safety of at least 1.2.

f. The foundation materials, should questionable areas be discovered during design or construction phases, can and would be improved by densification to a depth of about 100 feet.

g. The concept for the embankment cross section is very conservative and the rockfill portion of the embankment would preclude a piping or rupture failure due to large flows of water through the structure. The rockfill also provides resistance to cracking, erosion from overtopping, loss of freeboard, liquefaction of the downstream slope, and improves the overall stability of the embankment.

Consultants Findings

1975 BOARD

3.60 At the January 1975 meeting, the board expressed the opinion that an earth-rockfill dam and appurtenant works could be constructed on the proposed site in such a way that the consequences of a major earthquake, such as a 8+ magnitude associated with the adjacent San Andreas fault, would present no significant hazard. The board's opinion was predicated on the conditions that the dam would not be used for water storage except on the rare occasions when major floodflows occur and that complete drawdown of the reservoir would be rapid. The board recommended that the dam and its appurtenant works be designed to resist the lateral and vertical forces appropriate for a major earthquake (M 8+) on the San Andreas fault using the most recent dynamic analysis techniques for simulating strong shaking. The board concluded that because "the normal groundwater table is below 150 feet at this site, the likelihood of adverse ground effects in strong shaking is quite small". Furthermore, the board concluded; "In view of the inevitable uncertainly associated with the stability of the foundation soils should they become saturated, it would seem desirable to explore the possibility of eliminating this problem by preventing them from becoming saturated as a result of water storage. The foundation materials will be dry throghout most of the life of the project and a high degree of saturation may well be avoidable if storage of water is limited to relatively brief periods of time." The board indicated the need to consider in the design the possibility of branch or splinter faulting with permanent ground displacements up to 3 feet, either horizontally or vertically in the foundation of the dam. Regarding the possible occurrence of earthquakes associated with the reservoirs loading, the board expressed the view that, as far as the triggering mechanics of a large reservoir are understood, the necessary conditions would not even be approached at the Mentone facility if there were no permanent water storage behind the dam. The complete report of the Board of Consultants for the January 1975 meeting is presented in attachment No. 2.

1980 CONSULTANT INPUT ON CHANGED GROUNDWATER CONDITIONS

3.61 Since the convening of the 1975 Board of Consultants, groundwater, which was thought to remain at depths greater than 100 feet, has over the last 3 years risen to within 10 feet of the ground surface near the site of the proposed embankment. It now becomes apparent that groundwater may saturate the foundation soils periodically during the life of the project. Dr. Seed, the geotechnical consultant of the 1975 board, was asked in November 1980, in view of this changed condition, if he considered the site still suitable for design of a flood control dam. The reply to this inquiry is included in attachment No. 3. In part Dr. Seed stated: "In view of the high natural water levels now being attained and saturation being developed to within a few feet of the ground surface in the region near the dam, it appears desirable to reconsider the original concept of designing the dam in such a way that the foundation would be only partially saturated. This will require a re-evaluation of in-situ densities of the foundation soils. In a partially saturated condition, the densities of these soils, provided they were reasonably dense, were not a major criterion in evaluating their seismic stability but if the design is to be made for the foundation soils in a saturated condition, an accurate and reliable determination of their degree of densification will become of major importance". Furthermore he recommended "a comprehensive re-evaluation of all existing data pertaining to the determination of density and relative density of the foundation soils, including such indicators of density as in-situ shear wave velocity measurements." He concluded that "there is good reason to expect that the conduct of these tests will show the foundation conditions to be adequate to safely support the proposed dam even under the strong earthquake motions which may develop at this site. However, I believe that additional documentation to check this preliminary opinion is desirable."

3.62 Additional extensive field investigations and studies during the design phase to confirm the relative density of the foundation. A comprehensive re-evaluation of all existing data and literature pertinent to the foundation material will also be made.

1980 CONSULTANT INPUT ON SEISMICITY

3.63 Due to possible changes in the "state-of-the-art" or the level of knowlede about the San Andreas fault, since the 1975 Baord findings, the district asked Dr. Allen and Dr. Bolt in December 1980 to review their findings. Specifically, they were asked if there was a need to re-establish or revise; (1) the design parameters for the potential maximum event on the San Andreas fault; (2) amount of displacement on subsidiary or splinter faults; and (3) potential for triggering earthquakes as a result of reservoir filling. Through personal communication they indicated that there are no significant changes from the 1975 Board findings. Dr. Allen's written reply to this inquiry is included in Attachement No. 4.

Seismic Design Considerations

GENERAL

3.64 As stated previously a board of eminent consultants was selected during the initial planning stage of this project. Available members of this board would be reconvened during the design phase of Mentone Dam. In addition, the services of experts in the field of dynamic response would be retained to review work as it is planned and accomplished. It is anticipated that the investigation and design for Mentone Dam would require higher than usual efforts, considerable expertise and special considerations due to (1) the relative coarseness of the foundation and proposed embankment materials, and (2) the location of the dam in a highly seismic area.

3.65 The basic design approach presently envisioned for this project would be submitted to the board of consultants during the initial stages of design. The design would be accomplished in phases. A determination would be made at the end of each phase, to decide whether additional studies are required. Should any of the studies indicate the embankment and foundation are seismically unstable, results and recommendations would be presented to the board of consultants.

SEISMIC DESIGN CRITERIA

3.66 Tentatively, it is estimated that the peak horizontal acceleration, from a magnitude 8+ earthquake, will be 0.75g with a duration of strong shaking of 60 seconds. The intensity of vertical shaking may be as great as the horizontal intensity. The design of the dam will consider the possibility of the proposed embankment being ruptured by movements along subsidiary faults thay may underlie the dam. The amount of subsidiary rupture is not expected to exceed 3 feet at the bedrock surface, conservatively, it is assumed that the magnitude of this rupture will be transmitted through the alluvium to the ground surface. Future studies on the seismic and the foundation conditions under the embankment will refine the design earthquake parameters to be used.

MATERIAL PROPERTIES

3.67 During the design stage of the project, complete laboratory and sub-surface geotechnical investigations would be conducted to obtain adequate data necessary to evaluate the geology, foundation and to design the embankment.

3.68 The foundation investigation would include in-situ density, field test fills in which actual foundation materials are compacted with heavy vibratory rollers, field permeability, mass gradation tests, void ratio and critical void ratio deteminations, P and S wave velocity measurements. The data would be used to assess the response of the foundation materials under dynamic loadings.

3.69 The investigations for the embankment material would include the determination of all static properties (gradation, permeability, density, strengths etc.). In addition dynamic strengths and response properties would be obtained. The static and dynamic performance of the material would be analyzed using varying combinations of material densities and corresponding strengths.

ANALYSIS

Level of Risk

3.70 Since Mentone Dam would be operated as a flood control structure without a permanent pool, the combined annual maximum earthquake/flood pool loading condition would have a recurrence interval on order of about 10^{-5}. However, given the proximity of the project to the San Andreas fault system, the embankment will be designed and evaluated to preclude catastrophic and uncontrolled releases of water.

Transient Seepage Study

3.71 Since the dam would be operated as a flood control structure, without permanent pool, floodflow impoundments would partially saturate the foundation and embankment materials under normal groundwater conditions. To determine to what extent partial saturation would occur, a finite element program would be used to evaluate the transient seepage condition for various flood pool elevations. The computer program would be used to evaluate the effectiveness of the slurry trench in reducing the saturated zone in the near surface foundation materials.

Pore Pressure Dissipation

3.72 The embankment foundation materials consist predominantly of coarse grained sandy gravels and gravelly sands with cobbles and boulders. Because of the coarseness of the foundation materials, pore pressure dissipation may have a significant effect on the liquefaction potential of the soil. Data from the previous studies, consisting of the material properties, combined with earthquake/flood pool loading and transient seepage conditions would be used in the study to evaluate pore pressure buildup and dissipation under seismic loading.

Detailed Liquefaction Analyses

3.73 Because of the relative coarseness of the foundation materials, the current analytical approach for assessing liquefaction would not be easily applied. Extensions to the current state-of-the-art would probably be required with regard to: (1) Laboratory testing of coarser materials and (2) simultaneous computation of excess pore pressure development and dissipation, possibly in conjunction with use of nonlinear soil properties. In any event, the analyses deemed most appropriate at the time of the studies would be conducted.

Seiche Evaluation

3.74 Seiche is not considered to be a problem at Mentone in that this will be a flood control project, which during the life of the embankment would have a huge amount of freeboard available. However, to address the impact of an extreme case of this phenomenon on the project the following discussion is presented.

3.75 The word tsunami is generally associated with long water waves, having periods ranging from 5 to 60 minutes and longer, generated impulsively by local disturbances such as tectonic displacements associated with earthquakes, volcanic eruptions, submerged land slides and coastal landslides in bays and reservoirs. Fresh water tsunamis or seiches, caused by landslide in reservoirs and bays, have been simulated in laboratory tests by allowing a block to fall vertically as well as by allowing it to slide along a submerged inclined plane and by the rotation of rigid blocks. Based on model studies, it has been found that the wave energy liberated ranges between a fraction of 1 percent and about 2 percent of the net potential energy of the dropping or sliding mass (N.M. Newmark, E. Rosenblueth).

3.76 Based on the tectonics of the Mentone Dam site, it is virtually impossible to generate a seiche, however, for academic purposes it is assumed that the known tectonics are reversed so that the embankment is pushed upstream forcing the generation of a seiche when the flood pool is at spillway crest. It has been found that a seiche breaks at a height equal to 78 percent of the depth of water below the wave (H.M. Morris). Due to the shallow conditions existing on the upstream end of Mentone basin, fault slippage in the downstream direction would cause negligible wave action in the reservoir. Therefore, for this analysis, a seiche behind Mentone Dam is assumed to be generated by a sudden movement of the embankment in the upstream direction (against known tectonics). For this study it has been assumed that a fault slippage of up to 30 feet would occur through the dam embankment. The velocity of fault rupture moving the dam into the reservoir, is assumed to be on the order of 2 km/sec. Since the potential energy released during fault rupture is transferred to the reservoir through the dam embankment, optimum transfer of energy would occur at the velocity of foundation movement activating, but not exceeding, the base shear strength of the dam. The net potential energy of the moving embankment may be calculated as the kinetic energy of the embankment mass moving at the peak velocity. Based on momentum considerations, assuming an embankment unit weight of 145 pcf and base shear friction angle equal to 45 degrees, the peak embankment movement velocity was computed to be on the order of 25 fps. The total kinetic energy of the embankment mass is given by the equation:

$$K.E. = \frac{1}{2} mv^2$$

Where m is the mass of the embankment and v is the peak embankment movement velocity of 25 fps.

3.77 Incorporating a safety factor of 1.5, 3 percent of the total kinetic energy of the embankment mass is assumed to be transformed into seiche wave energy. The seiche generated is assumed to be a solitary wave produced by a single disturbance and propagated essentially unaltered form. The energy, E, necessary to generate the solitary wave is related to the depth, D, of the reservoir bottom below the undisturbed pool level, and the unit weight of water, γ . The energy of the solitary wave is given approximately by the equation: (H.M. Morris)

$$E = \frac{8\gamma (HD)^{3/2}}{3\sqrt{3}}$$

3.78 Equating 3 percent of the total kinetic energy of the moving embankment and the energy necessary to generate a solitary wave, the resulting wave height is given by the equation:

$$H = \left(\frac{3\sqrt{3}\,K.E.}{8}\right)^{2/3} \frac{1}{D}$$

3.79 Due to the mildly sloped conditions existing along the upstream shore of the flood pool, the seiche would be dissipated over a broad area and wave reflection effects should not be significant. The returning wave run-up may be assumed to be like, in effect, a sudden increase in run-off. To protect against transient run-up of water along the upstream face of Mentone Dam during movement of the embankment into the flood pool, the slope protection of the dam is extended up to the paved crest of the dam and down over the downstream face.

Embankment Sections

PROPOSED EMBANKMENT SECTION

3.80 The proposed embankment section is shown on plate D-47. The section would be designed to withstand 3 foot lateral and vertical displacements and forces associated with a major earthquake (M8+) on the San Andreas fault as recommended by the board of consultants. Freeboard requirements would be designed to satisfy seismic considerations. The proposed embankment section has a large crest width, relatively flat slopes, generous transition zones to accomodate large displacements, in excess of those recommended by the consultants, high strength materials, an inclined core zone which would assure that a larger portion of the embankment remains dry, a large rock zone on the downstream side and an upstream impervious blanket tied into a slurry trench at the upstream toe. This would be an excellent seismic resistant structure which would preclude catastrophic release of water.

3.81 The upstream impervious blanket and slurry trench would be designed to control any near surface underseepage from saturating the foundation during flood pool storage. The thickness of the blanket and width of the trench would be selected to withstand permanent ground displacements up to 3 feet.

3.82 The rock zone would consist of plus 3-inch material screened from the streambed alluvium. The materials would be well graded between 3 and 36 inches. The rock zone materials would be compatible with the pervious materials based on preliminary data. During subsequent studies, should the rock zone and pervious materials prove not to be compatible a transition zone would be added. The massive rock zone would be capable of surviving a fault rupture beneath the embankment or major displacements of the upstream slope, would allow and be non erodible to through seepage and preclude a catastrophic release of water.

ULTIMATE EMBANKMENT SECTION

General

3.83 To demonstrate the economic feasibility of the Mentone project, even under the most severe and unforeseen embankment loadings and the most adverse foundation conditions imaginable, the ultimate embankment section has been developed, see plate D-48. The improbable extreme loading conditions, which could occur separately or in combination, consist of the following:

a. A maximum 8.5 magnitude earthquake on the San Andreas fault with an associated fault rupture south of the existing trace through the dam causing a lateral displacement of up to 30 feet and vertical displacements up to 10 feet.

b. A standard project flood, filling the reservoir to spillway crest.

c. A seiche overtopping the dam.

d. Groundwater at the ground surface with complete saturation of the foundation.

Vertical and Lateral Displacements

3.84 The width of the transition zones would be 30 feet, that width would accommodate lateral fault displacements up to 30 feet distributed over a zone of shear several feet wide. All features of the embankment would be designed to accommodate up to 10 feet of vertical displacements.

Foundation Improvements

3.85 Experience with seismic evaluation of dams indicates that the areas of the foundation susceptible to liquefaction is, in general, located in the free field, beneath the upstream and downstream toes of the embankment and areas near the embankment centerline. The foundation materials for the ultimate section would be removed and recompacted to a depth of 50 feet beneath the upstream and downstream toes. In addition the foundation would be densified to a depth up to 50 feet by dynamic compaction methods.

3.86 Due to the assumption of groundwater at the ground surface and the densification of the foundation materials, the bentonite slurry trench is no longer required, and liquefaction is not a potential problem.

Seiche Overtopping

3.87 A seiche was assumed to overtop the embankment. To protect against embankment erosion, a 5-foot thick riprap layer would be placed from the spillway crest elevation to the paved crest of the dam on the upstream slope and from the crest to the rock zone on the downstream slope.

Slope Flattening

3.88 The upstream slope has been flattened to insure stability under combined vertical and horizontal accelerations at levels of about 1.0g.

Downstream Rock Zone

3.89 The downstream rock zone would be capable of adjusting to a fault rupture developing 30 feet of lateral displacement beneath the embankment or major displacements of the upstream slope without allowing a catastrophic release of water.

Cost Estimate

3.09 The estimated total cost for the "Ultimate Embankment Section" is $436,806,000, an increase of $50,485,000 over the cost estimate presented in the main report. The cost estimate reflects the increase in embankment quantities and length of the outlet conduit. The unit prices are based on October 1979 price levels. Using the same operation and maintenance costs on the main report cost estimate, the resulting benefit-cost ratio for the "Ultimate Embankment Section" would be about 1.1 (calculated on a last added element at 7-1/8% interest).

Construction Materials

EMBANKMENT

Shell

3.91 Materials for the construction of the embankment shells would be available in sufficient quantities within the reservoir area. Sound and durable materials with high shear strength and low consolidation characteristics for the construction of the pervious zones would be obtained from required excavation and designated borrow areas upstream from the dam site. Material for the rock zone would be produced from the streambed alluvium by screening the plus 3-inch material.

Core

3.92 Approximately ten million cubic yards of core materials are required for the construction of the dam. The core materials would consist of plastic sandy silts and clays. Materials in the immediate vicinity of the dam site would not meet core material requirements.

3.93 Materials meeting core material requirements are available in sufficient quantity in Prado Basin upstream of Prado Dam. Prado Basin is located approximately 30 miles from the damsite. The 30 mile distance was used as the maximum radius from the damsite within which to locate potential core borrow areas. Potential borrow areas were located and identified by using soil survey maps published by the Department of Agriculture. The potential areas were visually inspected to note cultural features not indicated on the maps. Factors considered in selecting the potential borrow areas consisted of he following: (a) transportation systems capable of moving large quantities of materials; (b) haul distances from the damsite; (c) cultural features; and (d) site drainage. The pertinent data of the potential borrow areas studied are summarized on plate D-44.

3.94 Based upon the quantities available, transportation systems available near the borrow areas and lack of any significant developments, potential borrow area No. 4 located in Prado Basin and borrow area No. 1 located approximately 14 miles southeast of the damsite are considered as likely sources of core materials. See plates D-45 and D-46. The selection of the borrow area would be based on economic, environmental and future developmental considerations. Quantitative and qualitative studies would be required during subsequent studies to delineate the borrow areas.

Slope Protection

3.95 Rock for stone revetment could be obtained from the basin excavation. Basin materials would be processed to obtain the required rock for slope protection.

CONCRETE

3.96 Suitable construction materials are available from required excavation and commercial sources. Concrete aggregates could be obtained from basin excavation. Portland cement is available from plants within a 20-mile radius.

IV. LOWER SANTA ANA RIVER

Project Description

4.01 The lower reach of the Santa Ana River is proposed to be a concrete rectangular channel transitioning into a soft-bottomed channel with either concrete T-wall or reinforced earth sidewalls. The Santa Ana River channel would be parallel, on the left, by the Greenville-Banning Channel. The Greenville-Banning Channel would be a paved rectangular section. For a complete project description see the main report.

4.02 The lower reach of the Santa Ana River channel would be a concrete section above station 142+50, with vertical walls about 20 feet high. Downstream of station 142+50 the channel would be soft-bottomed with vertical walls about 20 feet high. The Greenville-Banning channel would also have vertical walls about 20 feet high, and would be located 40 feet to the left of the Santa Ana River channel above station 88+00. The Greenville-Banning Channel would then transition into a confluence with the Santa Ana River Channel at station 77+00. Approximately 450 feet of the transition would have a common wall between the Greenville-Banning Channel and the Santa Ana River Channel. The sidewalls in the soft-bottomed section would be protected from scour by armor stone located below the invert. An under seepage cutoff would be a required element of the wall section because, the proposed invert is at approximately the same elevation with the surrounding ground level within this reach.

Topography

4.03 The lower end of the Santa Ana River enters the Pacific Ocean between Huntington Beach and Newport Beach through a coastal lowland known as the Santa Ana Gap. The gap is an alluvial valley about 2-1/2 miles wide, bounded on either side by highland areas known as the Huntington Beach and Newport Mesas. The mesas range in elevation from 50 to 85 feet higher than adjoining areas in the gap.

Regional Geology

4.04 The gap is located in an area which once was part of a major marine basin. Throughout much of the Tertiary and Quarternary periods, sedimentation occurred within this basin and several thousand feet of marine deposits were laid down. The gap is also located in an area where large scale faulting and folding have and are continuing to occur. As a result of these tectonic forces. the once deeply buried sediments have been deformed and uplifted so that early Pleistocene formations are exposed in the mesas on either side of the gap. The gap itself was created near the end of the Pleistocene when a major decline in sea level occurred. The ancestral Santa Ana River eroded a valley, the Santa Ana Gap, about 200-feet deep in response to the changing base level. After the last of the ice age glaciers melted and the sea level began to rise, the river began to aggrade depositing coarse alluvium

(Talbert Aquifer). As the rate of sea level rise slowed, the sediments deposited in the gap became finer grained. These relatively impervious silts, clays and organic deposits effectively confined the very permeable sands and gravels below.

Faulting and Seismicity

4.05 The Newport-Inglewood structural zone is the predominant structural/tectonic feature to cross the Santa Ana Gap. The zone is approximately 4 miles wide near the mouth of the river. It is characterized by northwest trending parallel faults and folds. The location of several named branches of the fault under the gap are based upon oil well data, ground water barriers in the older sediments and surface geologic mapping in the mesas on either side of the gap. The zone is seismically active as evidenced by the 1933 Long Beach earthquake, as well as subsequent macroseismic activity. "Disruption of the ground surface, not necessarily along known faults, will probably occur during any future local shock of the magnitude and duration of the Long Beach earthquake. The extensive cracking of the ground in the vicinity of the mouth of the Santa Ana River...may actually represent a major cause of damage during future shocks."[1]

Groundwater

HYDROGEOLOGY

4.06 A typical geologic cross-section in the gap along the river would depict three units which control the ground water movement and affect salt water intrusion. These units are: (1) a surficial fine grained unit, (2) the Talbert sands and gravels, and (3) older Pleistocene formations. At the bottom of the section are the various Pleistocene water bearing formations containing individual aquifers. These formations have been tilted and extensively faulted so that barriers to hydrologic continuity have been created and the saline ocean water cannot directly intrude them. These formations are separated from the base of the Talbert aquifer by an erosional unconformity. Continuity exists between the Talbert and some of the underlying older aquifers. Overlying the Talbert sands and gravels are the fine grained recent sediments consisting of silt, clay, peat and stringers of sand. These deposits are generally more than 50 feet thick and except in isolated areas near the ocean, these fine sediments confine the Talbert aquifer within the gap.

[1]California Division of Mines and Geology, Special Report 114, a Review of the Geology and Earthquake History of the Newport-Inglewood Structural Zone, Southern California, 1974.

SEA WATER INTRUSION

4.07 The Talbert and older Pleistocene aquifers are very productive and have yielded great quantities of water since the early 1900's. As early as 1930 the mining of the ground water had lowered pressure levels in the shallow aquifers to below sea level and sea water intrusion began. See Plate D-1 for a piezometric profile of the Talbert aquifer in 1963. Since the Talbert aquifer was in continuity with the ocean it was the first to experience the effects. By 1960 the intrusion had also begun to affect certain water bearing zones below the Talbert.

4.08 The problem of sea water intrusion in the Santa Ana Gap was studied in detail by the California Department of Water Resources. The results and recommendations of the study were presented in Bulletin 147-1, Santa Ana Gap Salinity Barrier, Orange County. One of the suggested plans to control intrusion was to create an injection ridge along Ellis Avenue, near station 250+00. The plan was later implemented by establishing series of injection wells that have been successful in reversing the gradient and halting the intrusion of saline water. Plate D-3 shows a recent piezometric profile and contour map of the gap. As a part of injection program, the Orange County Water District monitors a series of wells, see Plate D-3, on a weekly, monthly and bi-annual basis. In addition to the piezometric levels in the aquifer, various water quality parameters are measured and the results published in a Talbert Barrier Performance Report.

Project Effects on Sea Water Intrusion

4.09 In 1979, ten shallow soil borings were drilled with a hollow stem auger between Hamilton Avenue and the Pacific Coast Highway along the existing Santa Ana River channel. The generalized information from these borings in addition to data from previously drilled observation wells and shallow soils investigations conducted along the Santa Ana River channel are presented in cross-section on Plate D-3. The top of the Talbert aquifer is positively identified in the deeper observation wells at an elevation between -40 and -70 feet MSL. Downstream from well M-10 the top of the aquifer is not positively known. However, it may daylight at MSL near station 30+00. Regardless, excavations for a soft bottom channel in this reach would not effect the quality of the ground water because the Talbert aquifer is already in hydrologic continuity with the ocean and the injection program upstream maintains a positive gradient which would not allow landward movement of degraded water.

Recent Field Investigation

4.10 The exploration of the proposed channel improvements consisted of drilling 10 test holes, TH79-1 through 10, and 22 test trenchs, TT79-1 through 22. The test holes were drilled to a depth of 40 feet with a continuous hollow stem auger, during August 1979. The test trenches were excavated to an approximate depth of 10 feet with a backhoe during July 1979. The locations of the test holes and test trenches are shown on plates D-4, D-5, and D-6. The materials encountered were visually classified and disturbed and some undisturbed samples of representative material types were obtained for detailed laboratory testing. Soil samples were obtained at intervals of 5 feet or at more frequent intervals if the soil type changed. Drive samples were obtained to determine in-situ densities and standard penetration testing was conducted within all the test holes.

Previous Field Investigation

4.11 Prior to the recent investigation, a literature search was conducted for geotechnical information on the subject area. Seven existing studies were located.

4.12 Two of these studies dealt with the levees within the subject reach. The first is titled Dike Stability Investigation, Santa Ana River, San Diego Freeway to Fifth Street, Orange County, California and was prepared for the Orange County Flood Control District by Woodward-McNeil and Associates. The test borings found in that study have been designated TB73-1, 2, 3, 4, and 5. The second report is titled Geotechnical Investigation, Santa Ana River Channel Improvements, Pacific Coast Highway to Garfield Avenue, Orange County, California and was prepared for the Orange County Environmental Management Agency by Woodward-Clyde Consultants. The test boring found in that study have been designated TB76-1, 2, 3, and 4.

4.13 The remaining five studies were conducted at specific bridge improvement locations. Each study was conducted by the Orange County Environmental Management Agency. The reports discuss the foundation conditions at the Hamilton-Victoria bridge, the Adams Avenue bridge, the Slater-Segerstrom bridge, the McFadden Avenue bridge, and at the 17th Street bridge. The information from each of these studies has been used to supplement the recent Corps exploration.

Laboratory Tests

4.14 Mechanical analysis, Atterberg limits, moisture content determination, compaction, consolidation, permeability, unconfined compression, unconsolidated undrained triaxial, consolidated undrained triaxial, and consolidated drained triaxial tests have been conducted on representative disturbed and undisturbed samples in accordance with EM-1110-2-1906. The soil classification is in accordance with criteria provided by the Unified Soil Classification System. The results of the detailed laboratory tests and analysis are summarized in Table XI.

Analysis of Data

FOUNDATION

Streambed Conditions

4.15 An evaluation of the data collected from the investigations and laboratory tests indicates that, generally, the streambed materials within the limits of Lower Santa Ana River may be divided into two distinct areas as follows:

a. Foundation Conditions from Victoria-Hamilton Bridge to the Pacific Ocean. The materials encountered within this reach (see Plates D-4 and D-5) are predominately poorly graded sands or non-plastic silty sands. The materials are loose to dense with moisture contents ranging from dry to saturated. There is some surface sandy clay material, which appears to be fill. The plasticity index of the sandy clay ranges from 28 to 46 and the liquid limit range from 47 to 79. Ground water was encountered at depths varying from 9 to 20 feet below the ground surface. The logs of borings conducted in this reach are shown on plates D-4 and D-5.

b. Invert Materials from Victoria-Hamilton Bridge to 17th Street Bridge. Generally, the materials encountered within this reach (see plate D-6) are a non-plastic sand or silty sand, with up to 40 percent fines. There are occasional pockets of sandy silts or sandy clays with as much as 90 percent fines. The plasticity index of the silty sands and clays ranges from 9 to 33, and the liquid limit ranges from 31 to 60. The materials are loose to medium dense with moisture contents ranging from dry to completely saturated. Ground water was encountered from 1 to 10 feet below ground surface. The logs of borings conducted in this reach are shown on plate D-6.

PRELIMINARY DESIGN VALUES

4.16 Representative values are tentatively selected for preliminary design based on results of field and laboratory tests conducted on the representative remolded and undisturbed materials.

Densities

4.17 The selection of density values, for the compacted fill material, was based upon the results of standard ASTM 698 compaction test on representative materials. The moist unit weight was assumed to be at 95 percent maximum density and optimum moisture content. The mean density values of the in-situ materials were determined by undisturbed sampling (and were also determined from the moisture content of the materials below the ground water level and their known specific gravity of 2.74).

Strength

4.18 The preliminary strength parameters of the compacted fill were based upon the results of "R" triaxial shear and direct shear tests on material remolded to 95 percent of maximum density. Both the effective and total strength were determined. The mean shear strength of the in-situ materials was based on "R" triaxial shear tests on undisturbed samples.

Consolidation and Permeability

4.19 The selection of consolidation and permeability values was based on the results of consolidation and permeability studies on undisturbed samples of the in-situ materials, and by extrapolating information from the remolded tests in accordance with EM 1110-2-1913, Chap. 3. Preliminary representative design values for the Lower Santa Ana River Project area shown in table XI.

Table XI

Preliminary Design Values

Victoria-Hamilton Bridge to the Pacific Ocean	Compacted Fill	In-situ Material
Dry weight, (pcf)	110	100
Moist weight, (pcf)	126	118
Saturated weight, (pcf)	132	127
Angle of internal friction		
S-type, (degrees)	34	36
R-type, (degrees)	--	30*
Permeability, (fpd)	0.1	10
Equivalent fluid weight, (pcf)		
active,	40	--
passive	400	--
Bearing Capacity (psf)	2,500	2,500

* Failure in strain, as defined at 15 percent.

Table XI (Continued)

17th Street Bridge to Victoria-Hamilton Bridge	Compacted Fill
Dry weight, (pcf)	110
Moist weight, (pcf)	126
Saturated weight, (pcf)	132
Angle of internal friction S-type, (degrees)	35
Permeability, (fpd)	0.1
Equivalent fluid weight active, (pcf)	40

Design Applications

CHANNEL DESIGN

4.20 The channel within the lowest reach of the Santa Ana River, from approximately the Victoria-Hamilton Bridge to the Pacific Ocean would be soft bottomed with vertical walls. The two technically feasible proposals are to construct the vertical walls as either a T-wall section, or as a reinforced earth section.

T-wall

4.21 The T-wall section, as shown on figure 14, would be supported by the foundation materials through the spread footing of the structure. The footing would be located 5 feet below the finished invert. A buried stone protection will extend at a 1:1.5 slope, beyond the toe of the T-wall, to a depth of at least 20 feet beneath the base of the structure. The stone protection would inhibit scour or movement of supportive foundation materials. In order to lengthen the seepage path beneath the T-wall section, and thus, reduce the exit gradient, a 15-foot deep concrete or bentonite slurry trench cutoff is proposed. The cutoff would extend from the base of the T-wall to a depth of 15 feet. The concrete or bentonite cutoff would be placed at the riverside of the T-wall in order to reduce uplift. An alternative to the two phased foundation preparation discussed above, would be to eliminate the concrete or bentonite cutoff and replace the buried stone with a Prepakt-type fabric formed grout. The grouted form would be placed with an average thickness of 4 inches. Bedding material would not be required beneath the grout.

Reinforced Earthwall

4.22 The reinforced earthwall section is shown on figure 15. The wall would extend at least 5 feet below the channel invert to prevent localized low-flow scour. The foundation preparation would include a starter wall for the first course of reinforced panels, and the complete wall would be supported by the integral backfill. The scour and seepage controls would be as described in the preceeding paragraph. Coarse-grained materials available from invert excavation along the Lower Santa Ana River would be used for construction backfill for the wall. The facing panels or skin of the wall would be approximately 4-feet square and would consist of precast concrete to resist corrosion and abrasion. Filter fabric or gravel would be used to prevent the migration of backfill materials, through the skin joints. The reinforcing strips or ties, which stabilize the backfill and retain the skin, would consist of aluminum-magnesium alloy to inhibit corrosion. The horizontal embedment of the ties would approximately equal to the height of the wall.

CHANNEL EXCAVATION

4.23 The proposed sub-invert improvements would be constructed by open cut. The cutoff trench excavation would be held open by bentonite slurry as work progresses. Temporary slopes would not be steeper than 1V:1H. The invert materials to be excavated between the Victoria-Hamilton Bridge and the 5th Street bridge may be used as beachfill or as backfill for the reinforced earth channel wall. See paragraph 4.24 for quality and selection criteria of backfill. See paragraph 4.30 for beachfill criteria. Excess excavated materials may be used as a fill in the Santiago Creek phase of this project.

COMPACTED BACKFILL

4.24 Structural backfill would consist of select material from the required project excavation. Select material would consist of sand or silty sand from invert and levee reconstruction. The materials used as backfill for the reinforced earth structure would be granular and nonplastic in order to develop friction between the reinforcing strips and backfill. The reinforced earth backfill materials would be sand obtained primarily from the invert excavation and would have a percentage of fines not to exceed 15 percent by weight passing the 200 sieve. Backfill materials for other structures would be sand obtained from both the invert excavation and the silty sands from the existings levee. The material would be placed in 1 foot thick lifts and compacted to not less than 95 percent of maximum density (ASTM 698) and within 2 percent of optimum moisture.

SUBDRAIN SYSTEM

4.25 The channel invert between station 142+50 to station 528+00 would be paved. A clay cap that lies within a depth of 40 feet under the channel, may cause perched water to collect under the paved invert. To reduce uplift, a subdrainage system will be placed under the paved invert between station 142+50 to station 528+00.

Construction Considerations

DEWATERING

4.26 The excavation for the proposed improvements below the Victoria-Hamilton Bridge would be in permeable alluvium. Finished grade for the channel, within this reach, would be below the water table and extensive dewatering would be required during construction. Dewatering may be accomplished by combining dikes to control tidal inundation with a well-point system to draw the local water table down.

INVERT REPLACEMENT

4.27 The winter storms of 1979-1980 removed much of the Santa Ana River bottom materials, within the study area. Between 17th Street and the San Diego Freeway, the river bottom has been eroded to below the finished grade for the proposed project. The invert may be rebuilt by utilizing the excess sands and silty sands obtained from the proposed required excavation channel.

BEACHFILL MATERIALS

4.28 The excess sand materials excavated from the invert of the lower Santa Ana River were evaluated for use as a potential beach fill source. The logs shown on plate D-6 present the invert materials as of July 1979. The winter storms of 1979-1980 have significantly changed the invert profile. For this report, it is assumed that the general composition of excess materials are the same as shown on the logs (plate D-6). Only the location and quantity of materials have changed from what is shown.

4.29 The grain size data from the soils exploration along the Santa Ana River were utilized in determining potential beach fill properties and sources. Huntington and Newport Beach materials were also sampled, and tested to determine the grain size composition of existing beach materials, in order to establish grain size criterion for beach fill replacement.

4.30 The Environmental Protection Agency (EPA) has established a guideline for artificially placed beach fill material. The EPA states that either (a) the material have less than 10 percent of its weight passing the number 200 sieve and no less than 90 percent of its weight passing the number 4 sieve, or (b) that the replacement material be comparable with existing materials.

4.31 The beach sand samples collected in May of 1979 indicate that the materials are sand-silty sands (SP-SM) with about 9 percent passing the number 200 sieve and 96 percent passing the number 4 sieve.

4.32 Currently the only definable source of beach fill type materials comparable in size to those existing on the beach would be between Adams Avenue and the San Diego Freeway, where the invert is now approximately 8 feet higher than the final construction grade. In the other areas studied, the current channel invert is below the proposed channel invert.

V. SANTIAGO CREEK

Project Description

5.01 For geotechnical consideration the proposed improvements along Santiago Creek may be considered as two distinct elements. For a complete project description, see the main report. The first element would be the improvements to Santiago Creek between the Bond Street Gravel Pit and the Santa Ana River. The creek would be developed in two separate reaches as an armored trapezoidal section with a depth of about 11 feet, and the center reach of the creek remaining undeveloped. The second element would be the improvements within the Bond Street Gravel Pit at the upper reach of the project. The gravel pit would serve as a flood detention basin, with an inlet at Villa Park Road, and a gated outlet structure at Prospect Avenue.

Topography, Geology, and Faulting

5.02 Santiago Creek is a major tributary to the Santa Ana River. Its headwaters drain the western slopes of the Santa Ana Mountains. The flow is now somewhat regulated by Santiago and Villa Park dams several miles upstream from the project. However, the competence of Santiago Creek is attested to by the thick accumulations of relatively coarse alluvium in the fan on which the project is located. At the upstream end of the project, deep gravel pits have been excavated in cobbley gravelly sands with progressively finer sediments encountered in borings downstream. The depth of the alluvium in the project area is on the order of several hurdred feet.

5.03 Just upstream from the project, the active creek channel is bounded by terrace deposits resulting from older stream activity. Flanking these older terraces are complex associations of Tertiary sediments, and on the south side of the creek Tertiary volcanics.

5.04 Geologic mapping by the USGS[1] and the CDMG[2] show the volcanics to be extensively faulted. However, no recently active faulting in the volcanics is implied. Approximately 2 miles from the upstream end of the project, a possible concealed extension of the Norwalk fault cuts across Santiago Creek. This fault is believed to have experienced Quaternary movement[3]. The closest recognized active faults to the project are the Whittier and Newport-Inglewood. Both faults are approximatley 8 miles from the project and along with major events on the San Andreas could generate an earthquake which would cause significant ground shaking at the site.

[1]U.S. Geologic Survey, Geologic Map of the Northern Santa Ana Mountains, Map O.M. 154, 1954.

[2]California Division of Mines and Geology, Geologic Map of Orange County, California, Preliminary Report 15, 1973.

[3]California Division of Mines and Geology, Fault Map of California, Map No. 1, 1975.

5.05 The ground water conditions directly under Santiago Creek are not known. However, monitored wells nearby indicate that the depth to water increases upstream with a minimum depth of about 50 feet near the Santa Ana River confluence. Ground water depths at the upper end of the project may exceed 200 feet. Small bodies of perched water may be present locally especially downstream and during the wet season. Ground water should not be a problem during construction.

Recent Field Investigation

5.06 The exploration for the proposed improvements consisted of drilling 7 test holes, TH79-1 through TH79-7, along Santiago Creek, and sampling the walls of the Bond Street Gravel Pit at 5 locations, TL79-8 through TL79-12. The test holes were drilled to 30 feet with a bucket-type power auger, during October 1979. The locations of the test holes are shown on plate D-7. The areas sampled along the walls of the Bond Street Gravel Pit, are shown on plate D-8. The materials encountered were visually classified, and disturbed samples of representative material types were obtained at intervals of 5 feet or at more frequent intervals if the soil type changed for detailed laboratory testing. Standard penetration testing was conducted within all the test holes.

Previous Field Investigation

5.07 Several geotechnical studies of the subject area were located and utilized in this design. One study, deals with the material types encountered along Santiago Creek, within the City of Santa Ana. The report is entitled, Report and Preliminary Geotechnical Investigation at Three Alternate Reservoir and Pump Station Sites, Northeast Portion of the City of Santa Ana, and was prepared for the Santa Ana Department of Public Works by Evans, Goffman, and McCormack.

5.08 Three studies were previously done in the area of the gravel pits. The reports are titled, Stability and Seepage Evaluation, Proposed Hewes Avenue Gravel Pit, which was prepared for the Orange County Flood Control District by Converse, Davis and Associates; Soils and Geologic Investigation, Santiago Creek Greenbelt Corridor, which Mourseth, Howe, Lockwood and Associates prepared for the Orange County and Control District; and Preliminary Engineering Geology of the Bond Avenue and Hewes Avenue Gravel Pits, which was prepared by the Orange County Environmental Management Agency (OCEMA).

5.09 The information from these studies was used to supplement the recent Corps explorations. The locations and logs of the OCEMA exploration are shown on plates D-8 and D-9.

Laboratory Test

5.10 Mechanical analysis, Atterberg limits, moisture content determination, and compaction tests have been conducted on representative samples in accordance with EM 1110-2-1906. The soil classification is in accordance with criteria provided by the Unified Soil Classification System. The results of the laboratory tests and analyses are summarized in Table XII.

Analysis of Data

Foundation Conditions

5.11 An evaluation of the data collected by the investigations and laboratory test indicates the following conditions:

a. Conditions from the Bond Street Pit Area to the Santa Ana River. Generally, the invert materials within this reach (see plate D-7) are a non-plastic silty sands. The materials are medium dense, with moisture contents ranging from dry to moist. Ground water was encountered at a depth of 27 feet at the mouth of the Creek. The logs of borings are presented on plate D-7.

b. Conditions within the Bond Street Gravel Pit Area. The materials in this area consist generally of medium dense to dense sandy gravels and gravelly sands interbedded with some sandy clays and silts. The material variations appear as strata in the pit walls. The pit walls are generally cut at approximately 1V:1H slopes. The waste materials within the pit area are the fine grained silty sands washed from the pit's sand and gravel productions. These waste materials are located toward the center at the pit and are confined by native gravels having cut slopes of 1V:1H.

Preliminary Design Values

5.12 Representative values are tentatively selected for preliminary design based on results of field and laboratory tests on the representative disturbed materials.

5.13 The selection of density values for the compacted fill material, was based upon the results of standard ASTM 698 compaction tests. The moist unit weight was assumed to be at 95 percent of maximum density and optimum moisture content. The density values of the in-situ materials were determined by interpreting values from the blow count tests, and from density tests by others. Based upon general material type the relative densities of the in-situ creek and pit materials were determined to be approximately 70 percent. The preliminary strength parameters of the materials were selected using criteria provided in EM 1110-2-1913, Chapter 3, paragraph 8. Based on the gradation of the material and assuming that the in-situ material would be at a minimum of 70 percent relative density, mean values were selected to represent the angles of internal friction for the Santiago Creek and Bond Street Pit wall materials. Permeabilities were selected based upon the D_{10} of the materials and upon permeability tests conducted by others.

Table XII

Preliminary Design Values

Bond Street Pit area to the Santa Ana River	Compacted Fill	In-situ Material	
Dry weight, (pcf)	115	120	
Moist weight, (pcf)	126	130	
Saturated weight, (pcf)	134	138	
Angle of internal friction	S-type, (degrees)	35	35
Permeability, (fpd)	0.1	10	
Equivalent fluid weight	active, (pcf)	40	40
Bond Street Pit area			
Dry weight, (pcf)	115	125	
Moist weight, (pcf)	125	131	
Saturated weight, (pcf)	135	141	
Angle of internal friction	S-type, (degrees)	32	37
Permeability, (fpd)	0.1	10	

Design Applications

CHANNEL DESIGN

5.14 The channel, from the Bond Street Pit area to the Santa Ana river would be a trapezoidal section with riprap placed on bedding material. The channel slopes would be cut to 1V:2.5H. The riprap would extend to below the depth of scour.

PIT DESIGN

5.15 The Bond Street Gravel Pit would be designed to serve as a gated flood retention basin. Fill would be placed against the walls of the pit to a slope of 1V:2.25H. The inlet, located at Villa Park Road, would be a paved approach. The spillway would be built on compacted fill. A rock blanket would protect the end of the spillway. Separate rock blankets would protect the inlet approach, and the end of the spillway. Another rock blanket would protect the materials to the side of the spillway. The approach to the gated outlet structure at Prospect Avenue would also be protected by a rock blanket.

CHANNEL EXCAVATION

5.16 The proposed channel would be constructed by open cut. No temporary slopes would be steeper than 1V:1H. Excess excavated materials may be used as slope fill within the Bond Street Gravel Pit. Excavated materials greater than 9-inches would be raked to the channel invert for use as bottom armor.

COMPACTED BACKFILL

5.17 Structural backfill would be of select silty sand material from the required project excavation. Backfill material would be placed in 1-foot lifts and compacted to not less than 95 percent of maximum density (ASTM 698) and within 2 percent of optimum moisture.

5.18 Slope backfill for the Bond Street Pit would be of material from the required project excavation. Sandy silts to sandy gravels would be placed in not greater than 18-inch lifts and compacted to at not less than 95 percent of maximum density (ASTM 698) and within 2 percent of optimum moisture.

ROCK PROTECTION

5.19 The specific gravity of the riprap would not be less than 2.60. Filter blankets would be required beneath all riprap sections.

VI. OAK STREET DRAIN

Project Description

6.01 The proposed improvements along the Oak Street alinement would include a rectangular concrete channel that would extend from the existing Oak Street debris basin downstream to just below the Santa Fe Railroad alinement. The wall height of the channel would vary from 10 to 14 feet. A proposed collection system would divert water from Lincoln Avenue to the Oak Street channel. The diversion channel would have 10-foot high walls. For a complete project description see the main report.

Topography, Geology and Faulting

6.02 The project is located on an alluvial fan at the base of the Santa Ana Mountains south of the City of Corona. Below the existing debris basin, the topography uniformly slopes toward the north with a gradient which varies from 120 to 150 feet per mile. The topography at the debris basin is more complex due to the effects of the Chino and Elsinore fault system.

6.03 The Elsinore fault is one of the major northwest trending faults in southern California. In the vicinity of the Oak Street Drain, the fault exists as a 3/4-mile wide zone composed of several named branches along with numerous minor faults. It passes within 2,000-feet upstream from the debris basin. The Elsinore fault zone separates the early Mesozoic rocks which make up the bulk of the Santa Ana Mountains from a strip of Cretaceous and Tertiary sediments at the base of the mountains. The debris basin is located in this zone of younger sediments. A hill just west of the left abutment is composed of the Silverado Formation.

6.04 An irregular shaped subdued hill southeast of the right abutment has been mapped as older alluvium and terrace deposits. However, its irregular topography and relationship to the Main Street and Chino faults suggest that it may be also composed of Tertiary age sediments. The Chino fault is located for the most part by inference because it is concealed by alluvium. It is exposed and mapped in the Chino Hills 5 miles to the northwest. Based primarily upon topographic evidence, a projection of the Chino fault passes within 200 feet of the right abutment of the debris basin and may indeed be present in the foundation. Limited shallow refractive seismic investigations conducted by Converse Davis Dixon Associates[1] and a shallow trench excavated by

[1] Converse Davis Dixon Associates. Geotechnical Feasibility/Siting Study, Potential Debris Structures, Oak Street Channel Watershed, Corona, California. Report for Alderman, Swift and Lewis, November 17, 1977.

the same firm[2] across the suspected tract did not reveal any signs of faulting.

6.05 Although both the Chino and Elsinore faults are recognized as major southern California faults their degree of recent activity is questionable. Based primarily upon topographic evidence, both faults are considered to have moved during the late Quaternary. However, there are no known instances where Recent sediments have been disrupted.

6.06 A 1972 study by Langenkamp and Combs[3] of the microseismicity of the Elsinore fault zone led the researchers to the conclusion that: (1) the micro-earthquake activity is least in the vicinity of the Corona and increases to the south; (2) the Elsinore fault is not tectonically as active as the San Andreas of San Jacinto faults, and (3) the first motions of events indicate a complex geometry of the movement which is primarily dip-slip in contrast to the strike-slip motion on the San Jacinto and San Andreas. Therefore, even though the maximum earthquake for either the Chino or Elsinore exceed magnitude 6, based upon fault length criteria, the probability of occurrence in the site vicinity is low.

6.07 The existing debris basin is founded on recent and older alluvium. The foundation is essentially a dense silty sand with a greater percentage of gravel and cobbles toward the westside and more silt and clay toward the east. In 1978 four holes were drilled along the dam axis to a maximum depth of 40 feet. Ground water was not encountered except for some perched water at a depth of 36 feet, elevation 980. Data from nearby wells also indicate that the depth to water at the debris basin exceeds 40 feet and increases toward the valley center. Ground water is not expected to pose problems in the construction of channel improvements downstream.

Recent Field Investigation

6.08 The exploration of the proposed channel improvements consisted of excavating 8 test trenches, TT79-1 through TT79-8. The trenches were excavated to an approximate depth of 15 feet with a backhoe, during November 1979. The locations of the test trenches are shown on plates D-10 and D-11. The materials encountered were visually classified and disturbed samples of representative material types were obtained for detailed laboratory testing. Material samples were obtained at intervals of 5 feet or at more frequent intervals if the soil type changed.

[2]Converse Davis Dixon Associates. Design Memorandum No. 1, Field and Laboratory Data, Proposed Oak Street Channel Debris Control Facility. June 12, 1978.

[3]Langenkamp, David and Combs, Jim. Microearthquake Study of the Elsinore Fault Zone, Southern California, in Seismelogical Society of American Builders.

Previous Field Investigation

6.09 Geotechnical studies by other agencies, of the subject area, were utilized as supplemental elements of analysis for this report. A study by Alderman, Swiff and Lewis Consulting Engineers for the City of Corona, titled, Sixth and Tenth Street Bridges over Oak Street Channel, was used to supplement information about the material types along the proposed alinement. The logs are shown on plate D-12 and the exploration locations are shown on plate D-10.

6.10 A second study, entitled, Proposed Oak Street Channel Debris Control Facility, was prepared by Converse, Davis, Dixon Geotechnical Consultants, for the City of Corona. The study discusses geotechnical design considerations of the debris structure.

Laboratory Tests

6.11 Mechanical analysis, Atterberg limits, moisture content determination, and compaction tests have been conducted on representative samples in accordance with EM 1110-2-1906. The soil classifiction is in accordance with criteria provided by the Unified Soil Classification System. The results of laboratory classification tests and analysis are summarized on plates D-10 to D-12.

Analysis of Data

MATERIAL CONDITIONS

6.12 An evaluation of the data collected from the explorations and laboratory tests indicates that the materials encountered along the Oak Street drainage channel (see plates D-10, D-11, and D-12) consist of non-plastic silty sands or silty gravels. There are occasional pockets of clayey sand in the upper reach of the channel. The plasticity index of the clayey materials ranges from 6 to 14, and the liquid limit ranges from 22 to 32. The materials are medium dense to dense with moistures from dry to moist. Ground water was not encountered.

Preliminary Design Values

6.13 Design values are tentatively selected for preliminary design based on results of field and laboratory tests conducted on the similar types of materials. The preliminary design values are presented in table XIII.

DENSITY

6.14 The selection of density values for compacted fill material, was based upon the results of standard ASTM 698 compaction tests on representatives materials. The moist unit weight was assumed to be at 95 percent of maximum density and optimum moisture content. The density values of the in-situ material, as determined by others, were consistent with densities encountered during the recent Corps field observations.

STRENGTH

6.15 The preliminary strength parameters of the materials were established with criteria provided within EM 1110-2-1913, Chapter 3, paragraph 8. Based upon the gradation of the materials and assuming that the in-situ material would be at a minimum of 70 percent relative density, a mean value was selected to represent the angle of internal friction for the Oak Street drain channel materials.

PERMEABILITY

6.16 Permeabilities were determined upon the D_{10} of the materials and from test results conducted on similar types of materials.

Table XIII

Preliminary Design Values

Oak Street Drainage Channel	Compacted Fill	In-situ Material
Dry weight, (pcf)	115	120
Moist weight, (pcf)	126	130
Saturated weight, (pcf)	134	138
Angle at internal friction S-type, (degrees)	35	35
Permeability, (fpd)	0.1	0.1
Equivalent fluid weight active, (pcf)	40	--

Design Applications

CHANNEL DESIGN

6.17 The Oak Street channel and the Lincoln Avenue diversion channel would be constructed as either an L-wall or U-wall rectangular section. A subdrain system is not required for the channel, since the structure is not expected to intercept groundwater.

CHANNEL EXCAVATION

6.18 The proposed channel would be constructed by open cut. No temporary slopes would be steeper than 1V on 1H. Control of ground water is not expected to be a construction consideration.

COMPACTED BACKFILL

6.19 Structural backfill would be select silty sand material from the required project excavation. Backfill material would be placed in 1 foot-thick lifts and compacted to not less than 95 percent of maximum density (ASTM 698) and within 2 percent of optimum moisture.

Existing Oak Street Debris Basin

6.20 The existing debris basin located at the head of the proposed channel improvement, was given a cursory review. The embankment appears to have been designed with adequate geotechnical consideration given to seepage, static embankment stability, and construction material control. Should the basin be found to be hydraulically adequate, additional investigation of the Chino fault and its effects upon the embankment would be made.

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NATHAN M. NEWMARK | 1705 S. PLEASANT ST., URBANA, ILLINOIS

27 December 1973

Col. John V. Foley
District Engineer
Los Angeles District, Corps of Engineers
300 N. Los Angeles Street
P.O. Box 2711
Los Angeles, California 90053

Dear Colonel Foley:

The undersigned Board of Consultants, having met with representatives of the Los Angeles District, the South Pacific Division, and the Office of the Chief of Engineers, at your office on 11 and 12 December 1973, and after consideration of the information presented to us concerning the Mentone Dam flood control project, respond to your questions with the following recommendations and suggestions.

1. The close proximity of the proposed site to the south trace of the San Andreas fault does not in itself rule out the construction of a safe earth-rockfill dam on the site.

   From the historical and geological record, displacements on the San Andreas fault are of strike-slip type with rupture confined, in even a major earthquake, to a narrow zone with only minor vertical offsets.

   Because horizontal offsets on the San Andreas of up to 20 ft must be expected in a major earthquake, the Board recommends an alignment for the embankment which does not cross the south fault trace.

2. With the use of recently developed techniques of analysis and design it is feasible to construct a properly engineered earth or rockfill dam and appurtenant structures that can safely resist transient

vibratory motions of the high intensity associated with primarily strike-slip motions arising from proximity to major faults. These motions are not significantly greater adjacent to the fault than they would be several to ten or more miles away.

3. However it should be recognized that the characteristics of the foundation soils on which an embankment dam is constructed have a major influence on the seismic stability of the embankment and the reliability of the control works associated with the dam. Currently there is very little information available on the stability of the foundation soils at the Mentone site under earthquake loading conditions. Furthermore, while there is a good likelihood that they will prove to be adequately stable for the expected seismic environment at the site, there is also some probability that the generation of increases in pore water pressures in these soils during earthquake shaking could lead to undesirably large deformations of the embankment. Accordingly, before a decision on the technical feasibility of the project can be reached, it is essential to explore in reasonable detail the in-situ characteristics of the foundation soils.

4. While this will present special problems at this site because of the presence of cobbles and boulders, in view of the importance of the project and the chances of establishing its technical feasibility, the Board believes that it would be justified at this stage to expend about 200,000 dollars on a comprehensive exploration of the characteristics of the foundation soils.

5. In addition, part of the information needed for a final evaluation of the site is exploration of the geological structures which underlie the proposed dam. Geophysical profiling and more detailed geological mapping should be aimed at more precise definition and classification of water barriers and inferred faults shown on available geological maps.

6. Since the Mentone Dam is part of a flood control system that acts together with other facilities, it is essential that the safety of the other critical parts of the system be considered in the design. Hence the Board recommends that the safety of the Prado Dam, outlet works, spillway, and downstream channelization be reviewed for earthquake motions associated with the Whittier and Chino fault systems, as well as those associated with the San Jacinto and San Andreas fault systems.

7. Because of the triggering effects of sustained excess hydrostatic pressure on fault motions, the design requirements to limit the risk of failure to acceptable levels are substantially higher for a dam that is required to store water for continued periods of time rather than for a limited period to provide flood control. The Board would not recommend the construction of a dam at the Mentone site to provide a long term reservoir unless further consideration is given to the potential consequences of the water impoundment.

Respectfully submitted,

Bruce A. Bolt

Bruce A. Bolt

Nathan M. Newmark

Nathan M. Newmark

H. Bolton Seed

H. Bolton Seed

NATHAN M. NEWMARK | 1705 S. PLEASANT ST., URBANA, ILLINOIS

11 March 1975

Colonel John V. Foley, C.E.
District Engineer
Los Angeles District, Corps of Engineers
P. O. Box 2711
Los Angeles, California 90053

Dear Colonel Foley:

The undersigned Board of Consultants met with representatives of the Los Angeles District at your office on 31 January 1975 and reviewed the investigations conducteu to date concerning the Mentone Dam and Prado Dam flood control projects. At the conclusion of the meeting a number of questions were addressed to the Board concerning these projects, and our responses are presented in the following pages. It should be noted that the Board was presented with only qualitative descriptions concerning the project and did not have an opportunity to review all of your studies in depth. The comments below should be considered in that light.

A. Proposed Mentone Dam

Question 1. Based on the information which was presented to you at the first meeting of the Board on 11 and 12 December 1973 and the subsequent data obtained by the Los Angeles District during the past year on the characteristics of the foundation soils on which the Mentone Dam embankment and appurtenant structures would be constructed, presented at this meeting, does the Board consider the plan for a flood control dam at the Mentone site as presently conceived to be technically feasible?

Response: As indicated by the Board in December 1973 there are a number of different aspects which must be considered in making a recommendation. In the ensuing year some additional information has been obtained by the Corps of Engineers, in part in response to the Board's earlier suggestion that the in-situ characteristics of the foundation materials be explored in reasonable detail. Other aspects of background knowledge necessarily remain the same, including the regional structural geology and the historical earthquake record.

While, as is often the case in such site evaluations, the information available is sparse in certain respects, the Board feels that it can respond with reasonable assurance to the main questions of technical feasibility.

(a) The proposed dam and structures are closely adjacent to the south trace of the San Andreas fault. The proposed embankment, however, does not cross the fault trace.

In our opinion this site may be subject to earthquakes (up to magnitude 8+) associated with significant horizontal offsets along the San Andreas fault trace at any time. As a consequence the dam may be subjected to very substantial strong ground motion. Horizontal accelerations may occur exceeding 0.5g and shaking continue for 60 or more seconds. Secondly, the fling along the fault might produce a seiche of significant amplitude in any body of water in the reservoir at the time. Thirdly, "subsidiary" faulting may occur under the embankment associated with the major displacement on the main San Andreas fault trace. Offsets of up to

2 or 3 feet, either horizontally or vertically, may occur under the dam in such an earthquake.

(b) It is the Board's opinion that an earth-rockfill dam can be constructed on the site in such a way that the consequences of a major earthquake, described above, will present no significant hazard. Our conclusion is predicated on the following conditions. First, the dam will not be used for water storage except in the rare occasions (perhaps one per century) when major flood-flows occur. Complete draw-down of the reservoir will be rapid, not exceeding a few weeks. Under these conditions the joint probability of an almost simultaneous major earthquake and major flood can be estimated as extremely small.

(c) Nevertheless, the Board recommends that the dam design be developed so as to resist the lateral and vertical forces appropriate for a major earthquake on the San Andreas fault. This design should be based on the most recent dynamic analyses techniques for simulating strong shaking and also should permit permanent ground displacements of the scale mentioned above without critical damage to the embankment.

(d) The Board reiterates that there still is available only little information on the behavior of the foundation alluvium (principally sandy gravels with cobbles and boulders) under earthquake loading conditions. The Board feels, however, that because the normal water table is below 150 feet depth at this site, the likelihood of adverse ground effects in strong shaking is quite small. The Corps should investigate the hydrological changes in the alluvium under planned conditions of temporary water storage.

Question 2. Does the Board have any specific comments or recommendations regarding the present concepts proposed for foundation treatment, the embankment cross-section, and the spillway and outlet works structures?

Response: Major problems associated with insuring the seismic stability of the proposed Mentone Dam are associated with the nature of the foundation materials and the possibility of branch faulting in the foundation of the Dam.

The foundation materials consist of extremely variable sandy gravels with many cobbles and boulders and occasional sand lenses. There are no techniques available for reliably determining the character of such a material although the excellent investigation already completed indicates it to be reasonably dense and only vulnerable to strength loss under extremely strong shaking. Nevertheless, strong shaking is possible at the Mentone site. In view of the inevitable uncertainty associated with the stability of the foundation soils should they become saturated, it would seem desirable to explore the possibility of eliminating this problem by preventing them from becoming saturated as a result of water storage. The foundation materials will be dry throughout most of the life of the project and a high degree of saturation may well be avoidable if storage of water is limited to relatively brief periods of time. This possibility should be explored in detail.

A related study should also be initiated to determine the statistical probability of various earthquake and flood combinations which may occur in the life of the dam.

It should be recognized that, although the dam itself is not located on the San Andreas fault, there is a possibility of branch or splinter faults developing close to the major fault, and therefore in the foundation of the dam; the cross-section should be designed for this possibility.

Embankment stability should be ensured by requiring a very high degree of compaction; special care will be required in the seismic design of spillway and outlet works structures.

Question 3. Recognizing that additional geophysical profiling and detailed mapping are required to provide more precise definition and classification of water barriers and inferred faults shown on available geological maps, is it the opinion of the Board that the proposed plan for retaining flood flows in the reservoir would be satisfactory from the standpoint of possible triggering of earthquakes?

Response: In recent years special checks for related seismicity have been made at many hundreds of moderate to large reservoirs around the world. The conclusion has been that there are only perhaps ten to fifteen cases of earthquakes that can be associated with the reservoir loading. The only well-documented cases are for reservoirs behind large dams, i.e., in excess of 300 feet high, which hold water permanently.

So far as the triggering mechanism of a large reservoir is understood, the necessary conditions would not even be approached at the

Mentone facility if permanent storage of water behind the dam is avoided. The embankment and reservoir capacity are of insufficient size to fall into the category where concern for triggering might be needed. Two other factors essentially rule out hazard from reservoir triggering in this case. First, any peak water load on the crust would be sustained only for a few days at most. Secondly, no reservoir-induced earthquake is known with a magnitude greater than 6.5. The Board is recommending that the design of this facility take into account a magnitude 8+ earthquake associated with the adjacent San Andreas fault.

B. Prado Dam

Question 1. Based on the information presented at this meeting on the geology, faulting and seismicity of the Prado Dam area and the field investigations and analyses of the Prado Dam embankment and foundation conducted by the Los Angeles District during the past year, is it the opinion of the Board that the apparent high potential for liquefaction of certain strata of the dam foundation can be controlled adequately by the presently conceived remedial plans subject to their verification and further development as a result of the detailed study being undertaken by the Los Angeles District?

Response: The Board believes that the apparent high liquefaction potential of certain strata in the foundation of Prado Dam can probably be adequately controlled by remedial measures similar to those proposed, or by appropriate modifications of such measures.

Question 2. Does the Board foresee any problems in the soils, geologic or seismic areas which would render the present concepts for the proposed project modifications of Prado Dam and its appurtenant works technically infeasible?

Response: Evidence available to the Board at this time does not suggest any major soils, geologic or seismic problems that would make this project infeasible. However, further geologic and geophysical studies should be carried out to verify this tentative conclusion, as well as to permit the assignment of a realistic design earthquake. In particular, all available published and unpublished studies on the configuration and possible Quaternary activity of the Whittier and Chino faults should be summarized, and gaps in knowledge identified. The argument for the absence of faulting through the dam itself would be considerably strengthened if continuity of the axis of the Arena Blanca syncline could be demonstrated; perhaps exposures in shallow trenches near the east abutment, bucket-auger observation holes, or shallow drill-holes yielding oriented cores could quickly establish this. A very detailed, large-scale geologic map of the immediate area of the dam does not seem to exist, and this should certainly be a minimal requirement for the Prado -- or any other -- dam.

Question 3. Does the Board have any specific comments or recommendations regarding the present concepts proposed for the embankment cross-section, spillway and outlet works structures?

Response: The general cautions described elsewhere in this letter regarding the provision to resist dynamic motions, fault slip, and possible liquefaction, are applicable also to the Prado Dam. Of particular concern is the necessity for avoiding liquefaction of the materials underlying the present dam. Further studies of the relative reliability of the several measures proposed should be undertaken.

It is also recommended that studies be undertaken to locate more precisely possible fault systems that may cross or intersect the site, and that provision be made for the relative motion that might be caused by subsidiary faulting in the embankment, or in the spillway and outlet works.

With regard to the embankment cross-section, it appears likely that a larger stabilizing buttress will be required than that indicated on the tentative cross-sections for proposed remedial measures.

Question 4. Recognizing that additional geophysical profiling and detailed mapping are required to provide more precise definition of faults shown on available geological maps, is it the opinion of the Board that the proposed plan for retaining flood flows in the reservoir would be satisfactory from the standpoint of possible triggering of earthquakes?

Response: As is the case with the proposed Mentone Dam, the maximum depth of water to be stored behind the Prado Dam, as well as its short storage time, do not appear to be consistent with those few cases where earthquakes have apparently been triggered by reservoirs. Furthermore, the maximum credible earthquake to be specified for the nearby faults is likely to be at least as large as the largest reservoir-induced earthquakes that have ever been observed.

Respectfully submitted,

Clarence R. Allen

Bruce A. Bolt

Nathan M. Newmark

H. Bolton Seed

H. Bolton Seed, Inc.

623 CROSSRIDGE TERRACE, ORINDA, CALIFORNIA 94563 (415) 254-3036

November 25, 1980

Mr. L. Lauro
Department of the Army
Los Angeles District, Corps of Engineers
Attn: F and M Branch, Room 6627
P.O. Box 2711
Los Angeles, CA 90053

Dear Mr. Lauro,

Following our meeting on November 20, 1980 concerning the seismic design of the Mentone Dam, I am summarizing below my conclusions concerning this project based on a review of the water level data which you provided and reviewed at the meeting:

1. When the feasibility of designing and constructing a safe dam at this site was discussed by the Consulting Board in 1975, it was suggested that this could be accomplished if the design could be made in such a way that the foundation soils for the dam would not become saturated, even during periods of flood water storage. At that time water levels were typically about 150 to 200 ft. below the ground surface at the proposed site. Due to the higher than normal rainfall in 1978, 79 and 80, the ground water levels at the site have now risen to within about 10 to 20 ft. of the ground surface and this changed condition must be considered in evaluating the safety of the proposed dam.

2. In view of the high natural water levels now being attained and saturation being developed to within a few feet of the ground surface in the region near the dam, it appears desirable to reconsider the original concept of designing the dam in such a way that the foundation would be only partially saturated. This will require a re-evaluation of in-situ densities of the foundation soils. In a partially saturated condition, the densities of these soils, provided they were reasonably dense, were not a major criterion in evaluating their seismic stability but if the design is to be made for the foundation soils in a saturated condition, an accurate and reliable determination of their degree of densification will become of major importance.

3. In view of apparent uncertainties concerning some aspects of past determinations of the in-place relative density and other in-situ characteristics, I would recommend:

(a) A comprehensive re-evaluation of all existing data pertaining to the determination of density and relative density of the foundation soils, including such indicators of density as in-situ shear wave velocity measurements.

(b) The conduct of field compaction tests in which actual foundation soils are compacted with heavy vibratory rollers using 12 inch lifts to determine the highest density to which the soil can be placed. Such a procedure may produce higher densities than laboratory compaction tests on scalped samples and provide a better indication of the present condition of the foundation alluvium than can be obtained from comparisons with laboratory compaction data.

(c) The conduct of studies to provide information on the characteristics of the foundation soils throughout the full depth of the foundation alluvium. In situ shear wave velocity determinations may be the only way in which this can be done effectively.

There is good reason to expect that the conduct of these tests will show the foundation conditions to be adequate to safely support the proposed dam even under the strong earthquake motions which may develop at this site. However, I believe that additional documentation to check this preliminary opinion is desirable.

Sincerely yours,

H. Bolton Seed

H. Bolton Seed

CLARENCE R. ALLEN
700 SOUTH LAKE AVENUE, APT. 322
PASADENA, CALIFORNIA 91106

2 January 1981

Mr. Norman Arno
Chief, Engineering Division
Los Angeles District, Corps of Engineers
P. O. Box 2711
Los Angeles, California 90053

Dear Mr. Arno:

In your letter of 12 December 1980, you asked me to review some of our earlier recommendations concerning the seismic design parameters for the proposed Mentone Dam of the Santa Ana River Project. In particular, you asked that I reconsider certain questions in the light of technical knowledge that may have been gained since the time of our Consulting Board report of 11 March 1975.

Let me discuss in order the various items that you mention:

(1) Earthquake magnitude.--We suggested in the 1975 report a magnitude 8+ design earthquake centered on the nearby San Andreas fault, and I see no reason to modify this recommendation. Geologic studies by Dr. Kerry Sieh, carried out subsequent to 1975, suggest that earthquakes of this magnitude level have occurred about every 150 years along the San Andreas fault near Valyermo, 50 miles northwest of the Mentone site. Although there is some debate as to whether the same recurrence interval characterizes the fault near Mentone, because the fault system is fraying-out southeastward, we must conservatively assume a similar degree of activity in the two areas. In any case, a great earthquake centered very close to the damsite is a likely event during the life of the dam and should be considered in the engineering planning. Although we use the term "8+", the ground motion at the site will not be predictably different whether the actual magnitude (M_s) be 8.0, 8.5, or some other nearby figure, owing to the great length of faulting that necessarily will be associated with such a large event, the complexity of the fault-rupture process, and the nature of the resulting ground motion.

(2) Acceleration and duration.--Within the past five years, we have learned a great deal about the nature of strong ground motion during earthquakes. However, this is somewhat outside of my field of expertise, and other members of the Board of Consultants are more qualified than myself to advise you on this subject.

(3) *Fault displacement*.--Surface fault displacements of up to 20 feet on the main trace of the San Andreas fault were specified in the 1975 report, and I see no reason to modify this recommendation. The main trace of the fault does not, of course, directly cross any of the structures of the proposed dam. Historic experience has shown time and again that fault breaks tend to repeat faithfully along earlier breaks, and it is exceedingly unlikely -- virtually incredible -- that the main trace of the San Andreas fault would shift during a future earthquake to a new location beneath the dam, the nearest point of which is about 0.3 miles from the presently active main fault trace.

(4) *Subsidiary faulting*.--The Board recommended in 1975 that as much as 2 to 3 feet of displacement, in any direction, be considered credible on subsidiary faults that might pass beneath the dam itself. This recommendation was based on the observation that minor subsidiary fractures have indeed sometimes occurred adjacent to major fault breaks, and because the very recent stream gravels at the site would probably conceal any branch or subsidiary faults that might in fact be present. Furthermore, various ground-water barriers in the area have been attributed to faulting. Were the area of the dam to be excavated to bedrock, it is likely that some subsidiary faults would be observed, although the probability of any specific one of these breaking in association with a given major earthquake on the nearby San Andreas fault is low.

It is my judgment that, assuming a major slip on the adjacent segment of the San Andreas fault every 150 years, a subsidiary fault somewhere beneath the dam might break with significant displacement during every 20th such event, in a statistical sense, giving a recurrence interval for subsidiary faulting of 3,000 years. At any given point beneath the dam, the recurrence interval would of course be much greater, and I think it would be unduly conservative to assume that every localized structure of the dam (e.g., a spillway gate) would have to be designed on the assumption of a significant fault displacement directly through it. Nevertheless, subsidiary faulting during the life of the dam, somewhere along its length, is by no means incredible.

The basis of our judgment of 2 to 3 feet of maximum offset on subsidiary faults was determined by the types and frequencies of subsidiary faults that have been observed during historic earthquakes on faults similar to the San Andreas. This is certainly not the maximum that has every been observed; a subsidiary fault displacement of 35 feet was observed locally at the time of the great 1897 Assam earthquake (probably a thrust-fault event), and 2.5 feet of vertical and 4 feet of horizontal displacement took place in 1906 on a branch fault about 0.6 miles west of the main San Andreas trace in Marin County. Displacements this large are, however, relatively rare, and our suggested 3 feet of maximum displacement seems adequately conservative in view of (1) the very limited area of the Mentone Dam as compared to the vast area within which localized subsidiary faulting might statistically occur during the next great earthquake on this segment of the San Andreas fault, (2) the dominantly strike-slip nature of the San Andreas fault, (3) the fact that the average distance from the fault of the dam (including its most critical segments) is perhaps 2 miles, and (3) the exceedingly unlikely occurrence of a major earthquake during the very infrequent periods when the dam is impounding water. Were it not for this last factor, a more appropriately conservative displacement figure might be 3 feet of vertical and 5 feet of horizontal displacement.

One geologic map of the area (Dutcher and Garrett, 1963) shows two faults trending toward the dam, although not actually continuing beneath it. These faults, "K" and "L", are, however, "postulated on the basis . . . of somewhat inconclusive evidence" relating primarily to ground-water effects, and I do not consider them to be sufficiently well documented to call for specific engineering planning other than inclusion under the overall umbrella of 2-3 feet of subsidiary faulting already specified. If subsequent investigations show these faults to break young alluvium at specific localities, then further consideration will be called for. Such investigations should be undertaken.

(5) Bedrock or ground-surface faulting?--You specifically asked whether the specified amount of subsidiary faulting should be assumed to occur at the ground surface or on the buried bedrock surface. Our assumption was that this should be considered a ground-surface displacement, at the base of the dam. We recognize that the alluvium is relatively thick at the damsite, but thick alluvium in itself does not necessarily dissipate fault displacement imposed by underlying bedrock dislocation.

(6) Reservoir-induced earthquakes.--I note one statement in the 1975 report that requires modification: On page 5 we stated, with regard to reservoir-induced earthquakes, that "the only well-documented cases are for reservoirs behind large dams, i.e., in excess of 300 feet high, which hold water permanently." This statement still applies for relatively large reservoir-induced events -- those in excess of magnitude 5.7 -- but there are now several well-documented cases of induced low-level (non-damaging) seismicity in shallow reservoirs. For reasons stated in the 1975 report, however, I continue to recommend no special consideration for reservoir-induced earthquakes at the Mentone Dam, primarily because a very large naturally-occurring design earthquake has been specified anyway.

I should emphasize that in reviewing our 1975 conclusions, I have not made an extensive literature search of geologic, hydrologic, or geophysical work that might have been done in the damsite area since that time. Nor have I talked at length to engineering geologists who have recently been working in this area. I assume that your own geologic staff will advise the Board of any such studies, and I of course stand ready to modify my conclusions if any relevant new work should come to light.

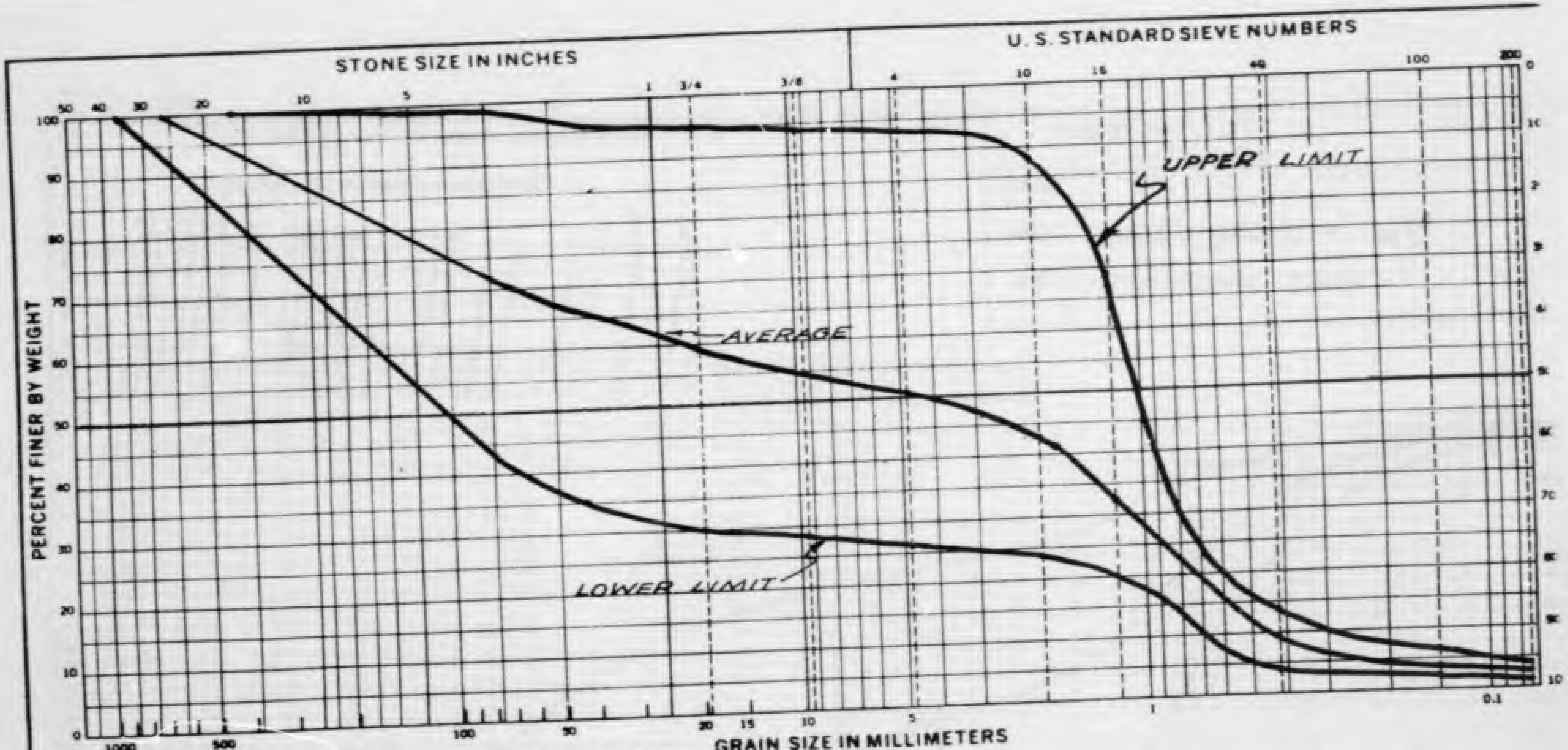
Dr. Bruce Bolt and I have talked over the telephone about most of the items covered in this letter, but, in the interest of time, we are submitting separate letter reports. If you wish, we can at a later time formulate a joint statement.

Very truly yours,

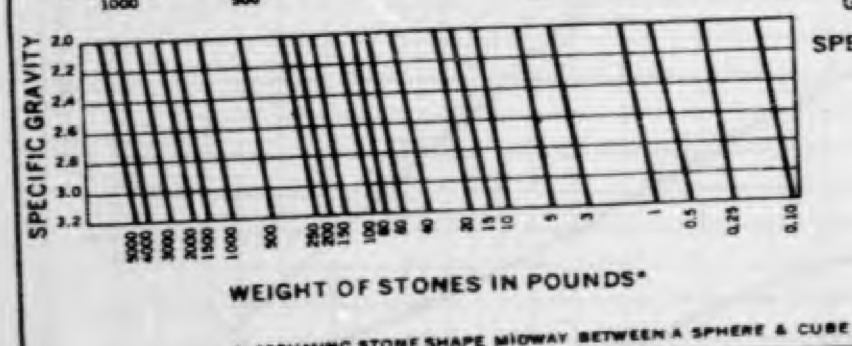
CR Allen

Clarence R. Allen

cc: Dr. Bruce Bolt
Dr. Nathan Newmark
Dr. H. Bolton Seed



SPECIFIC GRAVITY OF STONES............



PROJECT MENTONE DAM

AREA TEST TRENCH #1

DATE ....

GRADATION, TT-1

ENG FORM 4056
APR 67

FIG 1

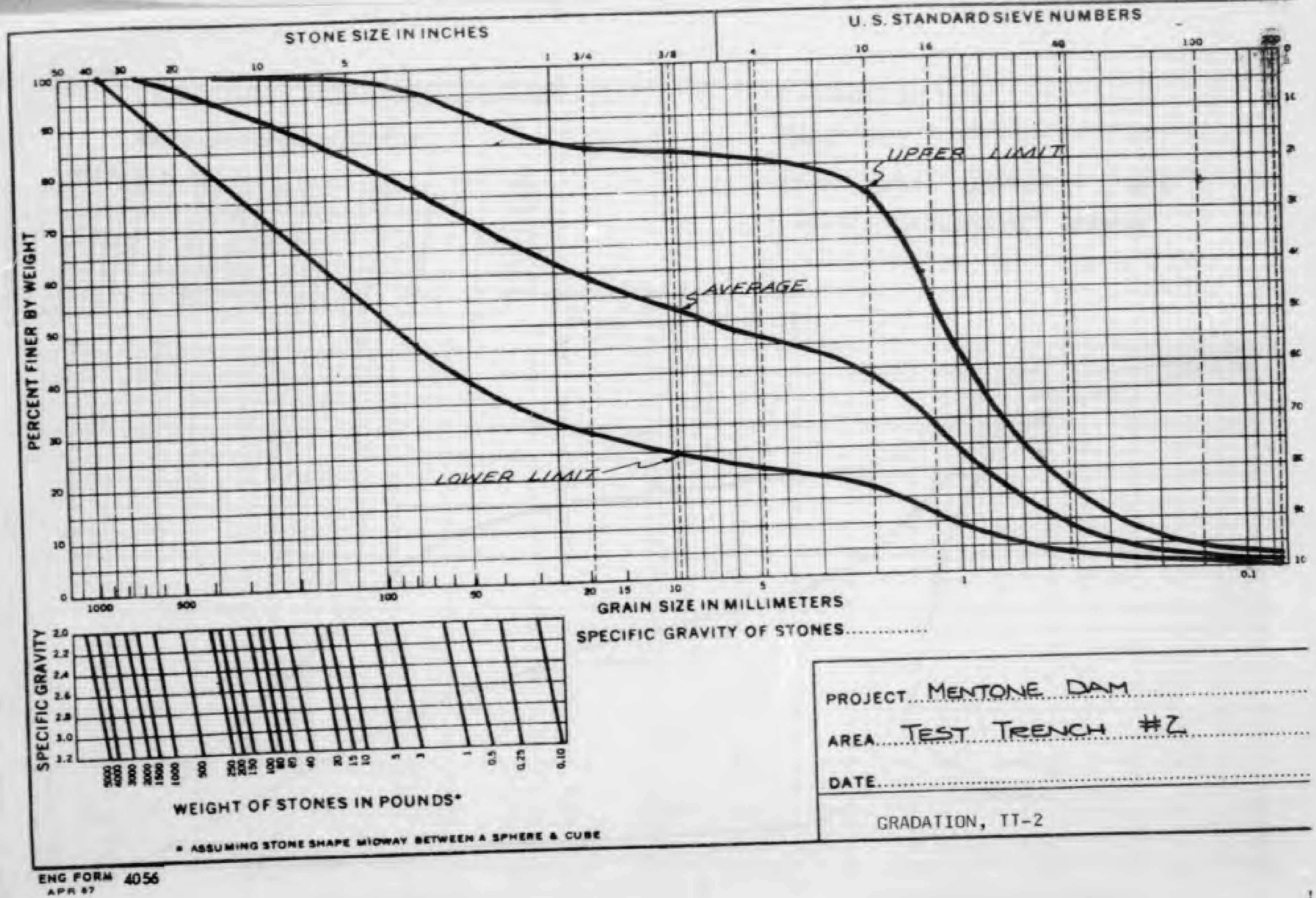
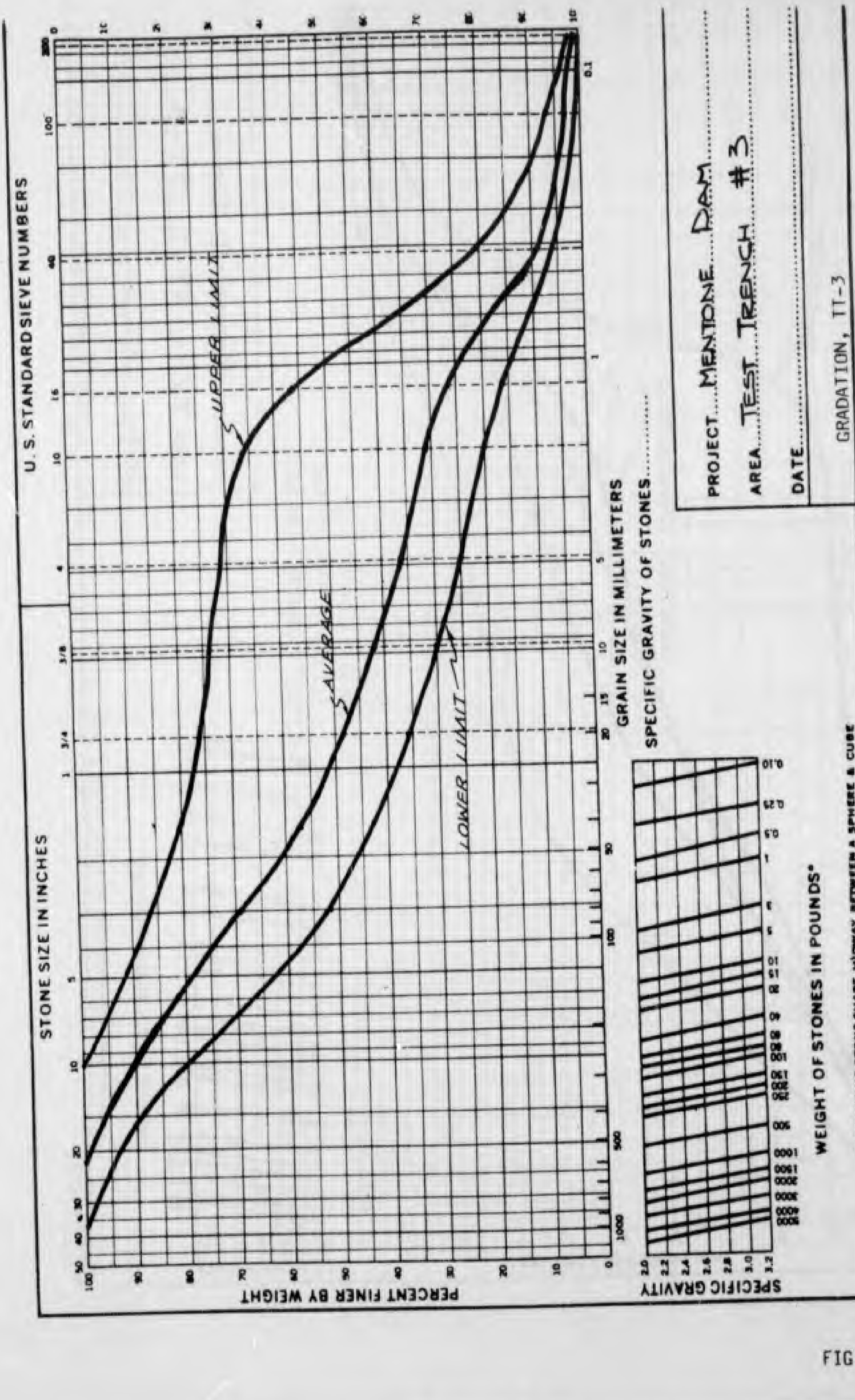

STONE SIZE IN INCHES
U. S. STANDARD SIEVE NUMBERS
PERCENT FINER BY WEIGHT
UPPER LIMIT
AVERAGE
LOWER LIMIT
GRAIN SIZE IN MILLIMETERS
SPECIFIC GRAVITY OF STONES............
SPECIFIC GRAVITY
WEIGHT OF STONES IN POUNDS*
* ASSUMING STONE SHAPE MIDWAY BETWEEN A SPHERE & CUBE
PROJECT MENTONE DAM
AREA TEST TRENCH #2
DATE
GRADATION, TT-2
ENG FORM 4056
APR 67

FIG 2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STONE SIZE IN INCHES
PERCENT FINER BY WEIGHT
UPPER LIMIT
AVERAGE
LOWER LIMIT
GRAIN SIZE IN MILLIMETERS
SPECIFIC GRAVITY OF STONES
SPECIFIC GRAVITY
WEIGHT OF STONES IN POUNDS*
* ASSUMING STONE SHAPE MIDWAY BETWEEN A SPHERE & CUBE
PROJECT MENTONE DAM
AREA TEST TRENCH #3
DATE
GRADATION, TT-3
ENG FORM 4056
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FIG 3

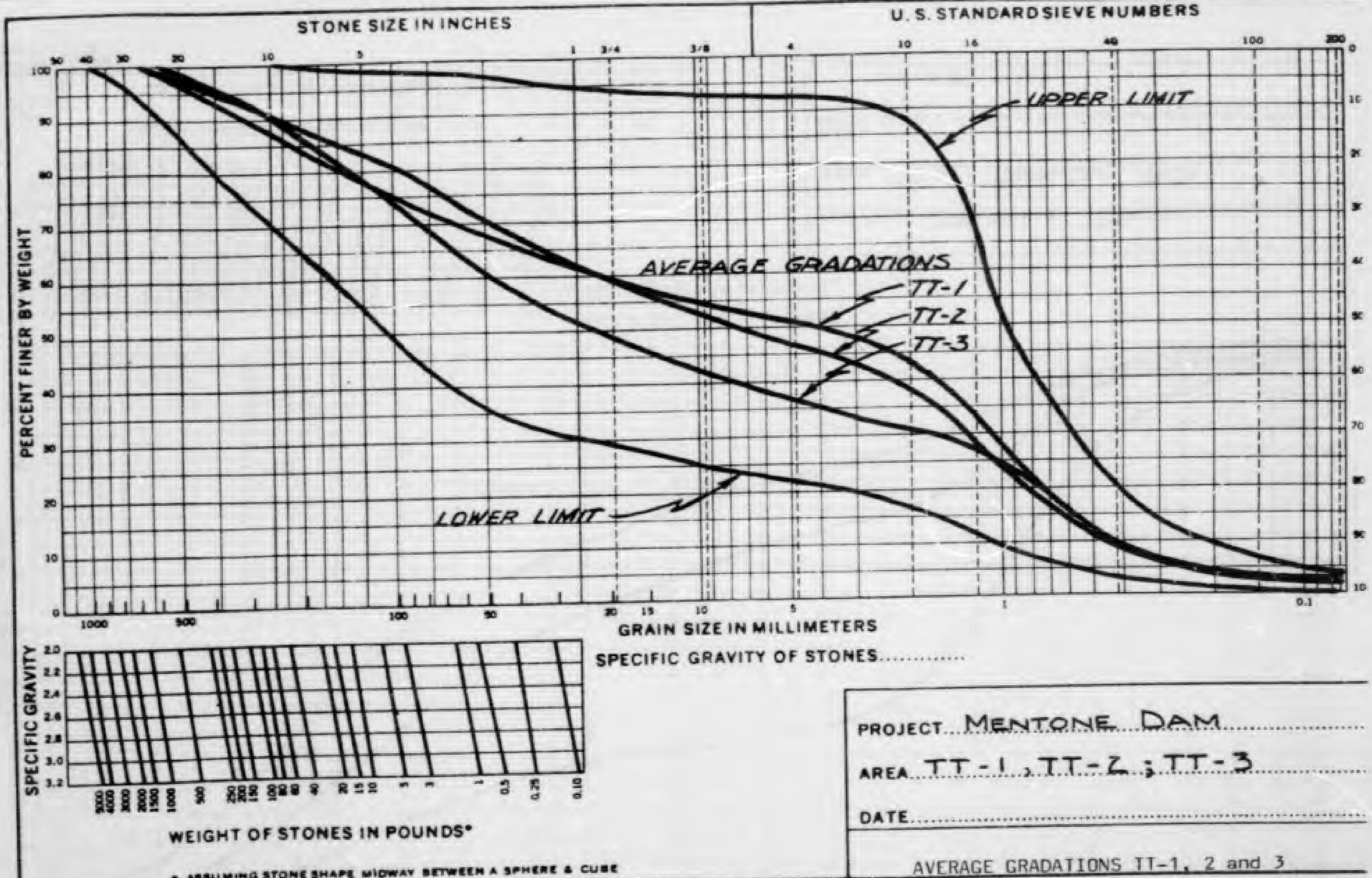

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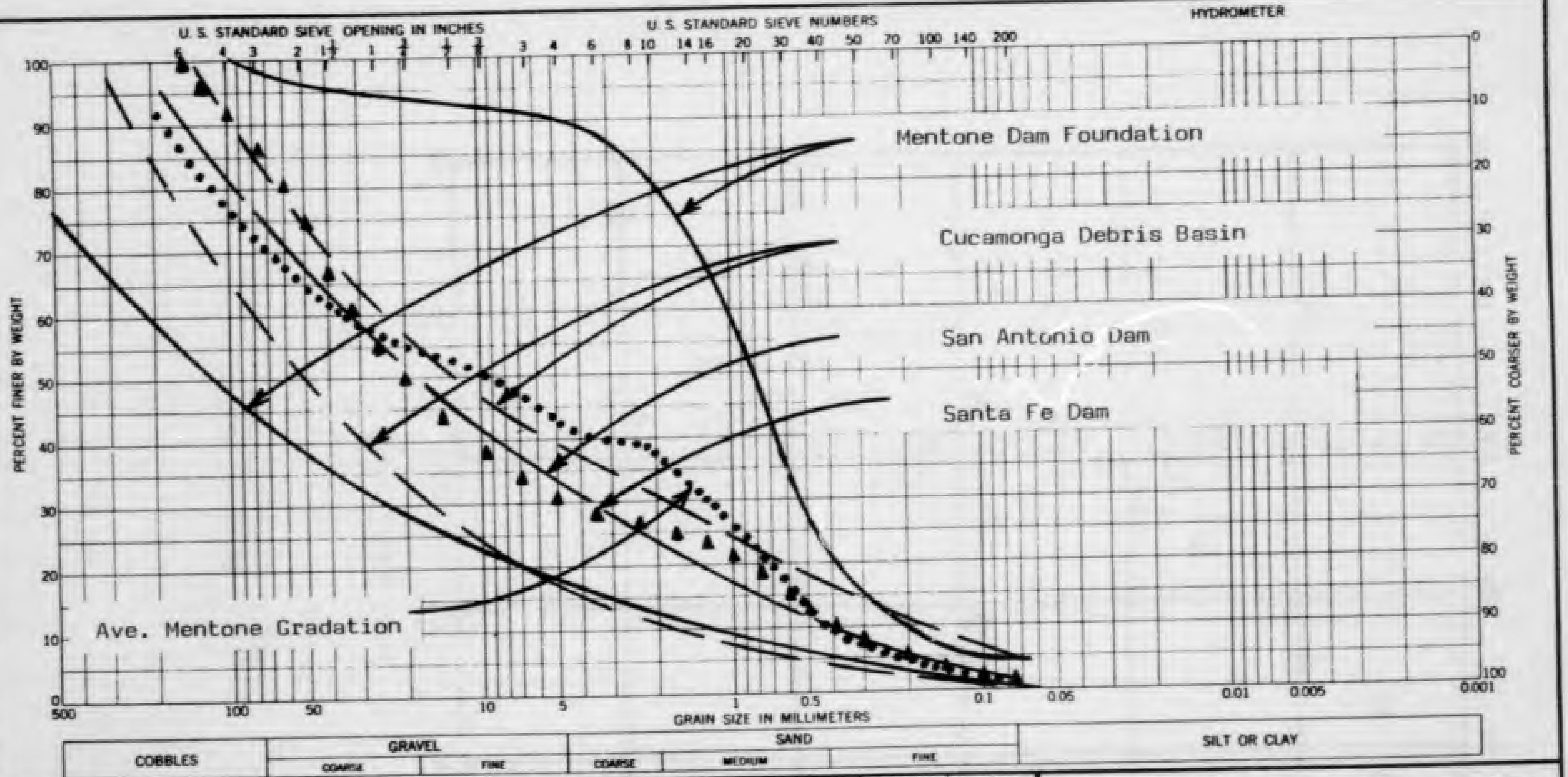
FIG 4



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project Mentone Dam Material Comparison

Area

Boring No.

Date

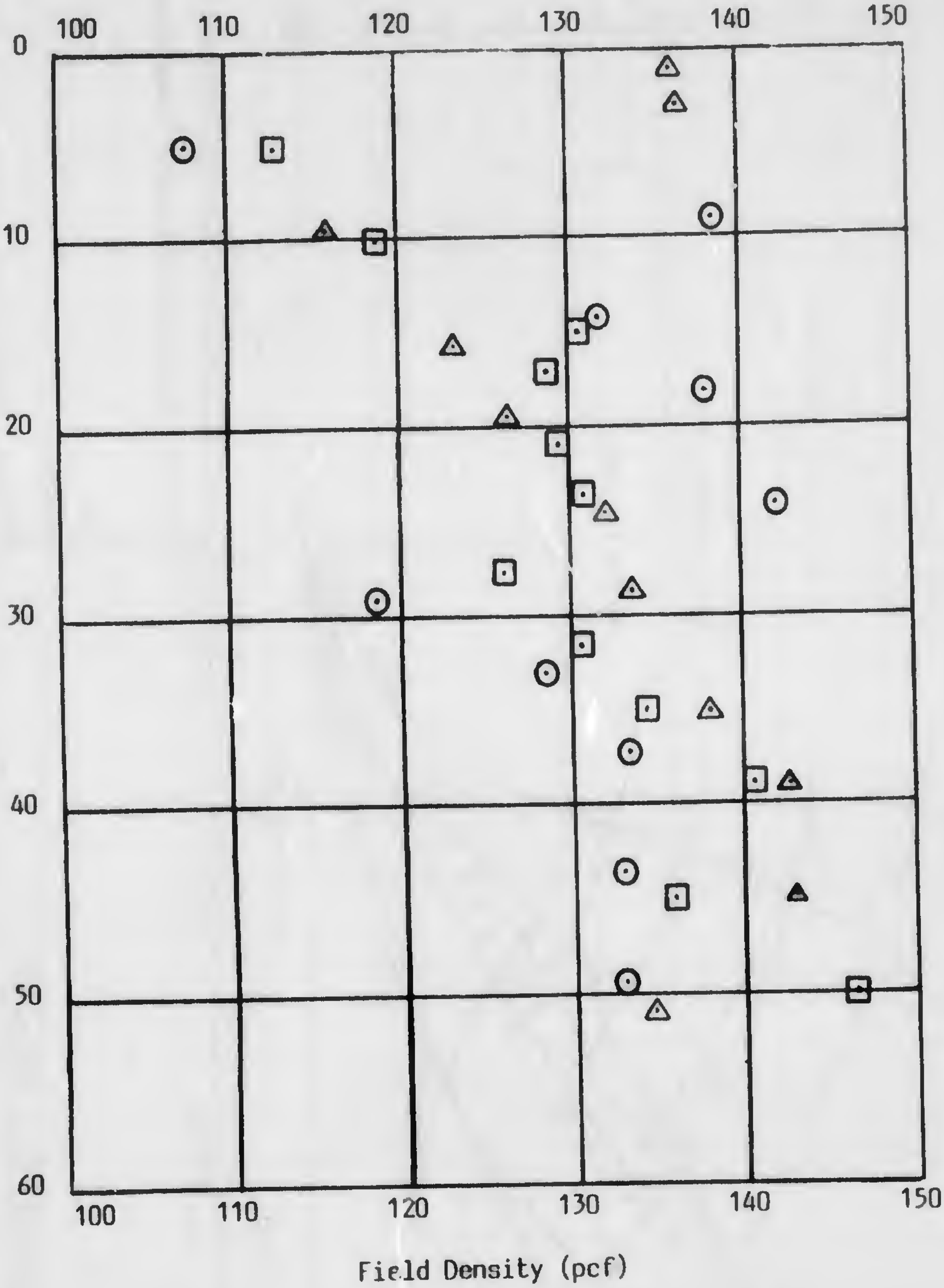
GRADATION CURVES

ENG FORM 1 MAY 63 2087

GRADATIONS OF VARIOUS PROJECTS

FIG 5

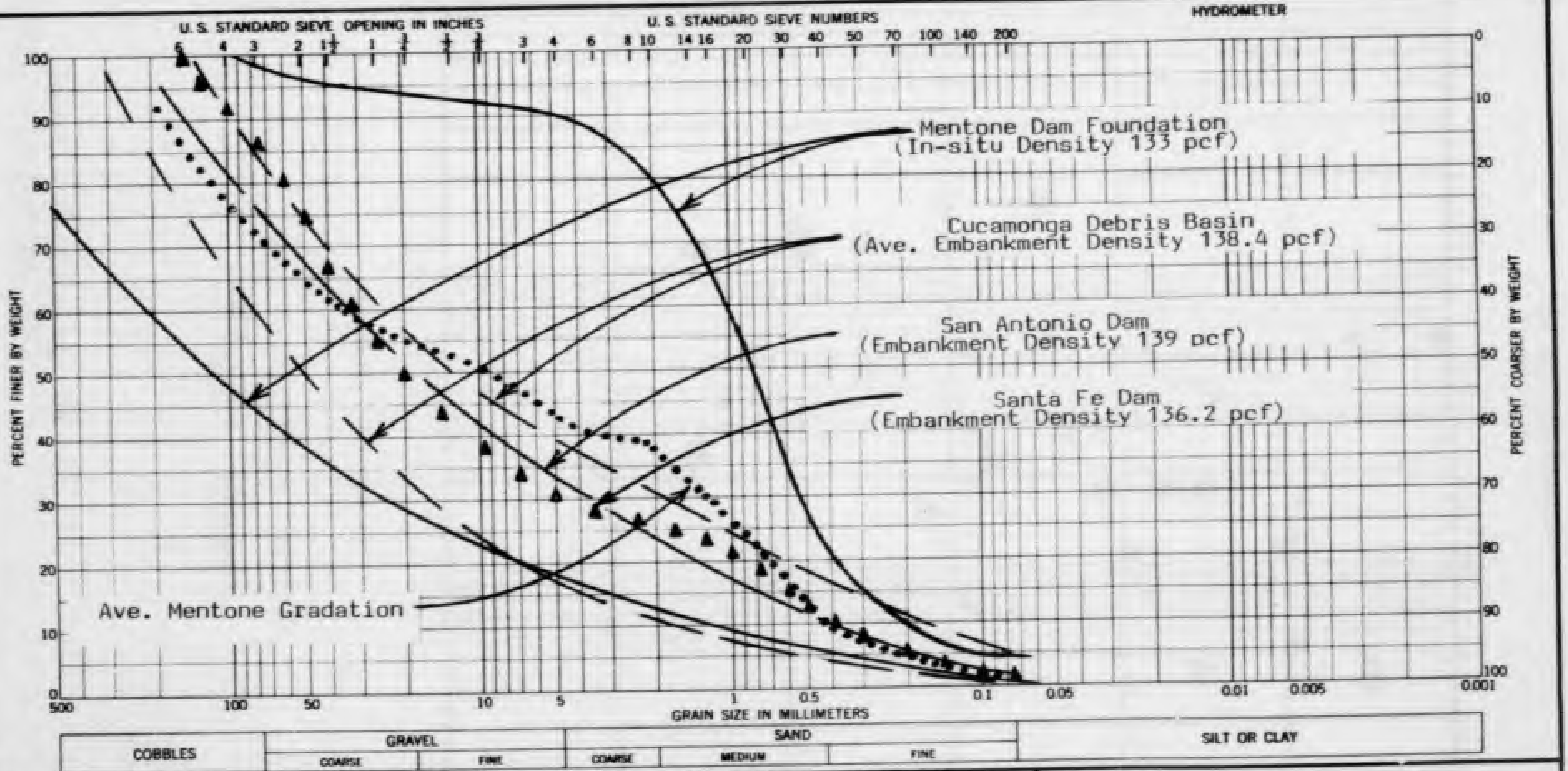
IN-SITU DENSITY VS DEPTH



LEGEND
- ⊙ TT - 1
- ⊡ TT - 2
- △ TT - 3

NOTE: Below 10ft Depth
γ(AVG) = 133 pcf

FIG 6



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project Mentone Dam Material Comparison

Area

Boring No.

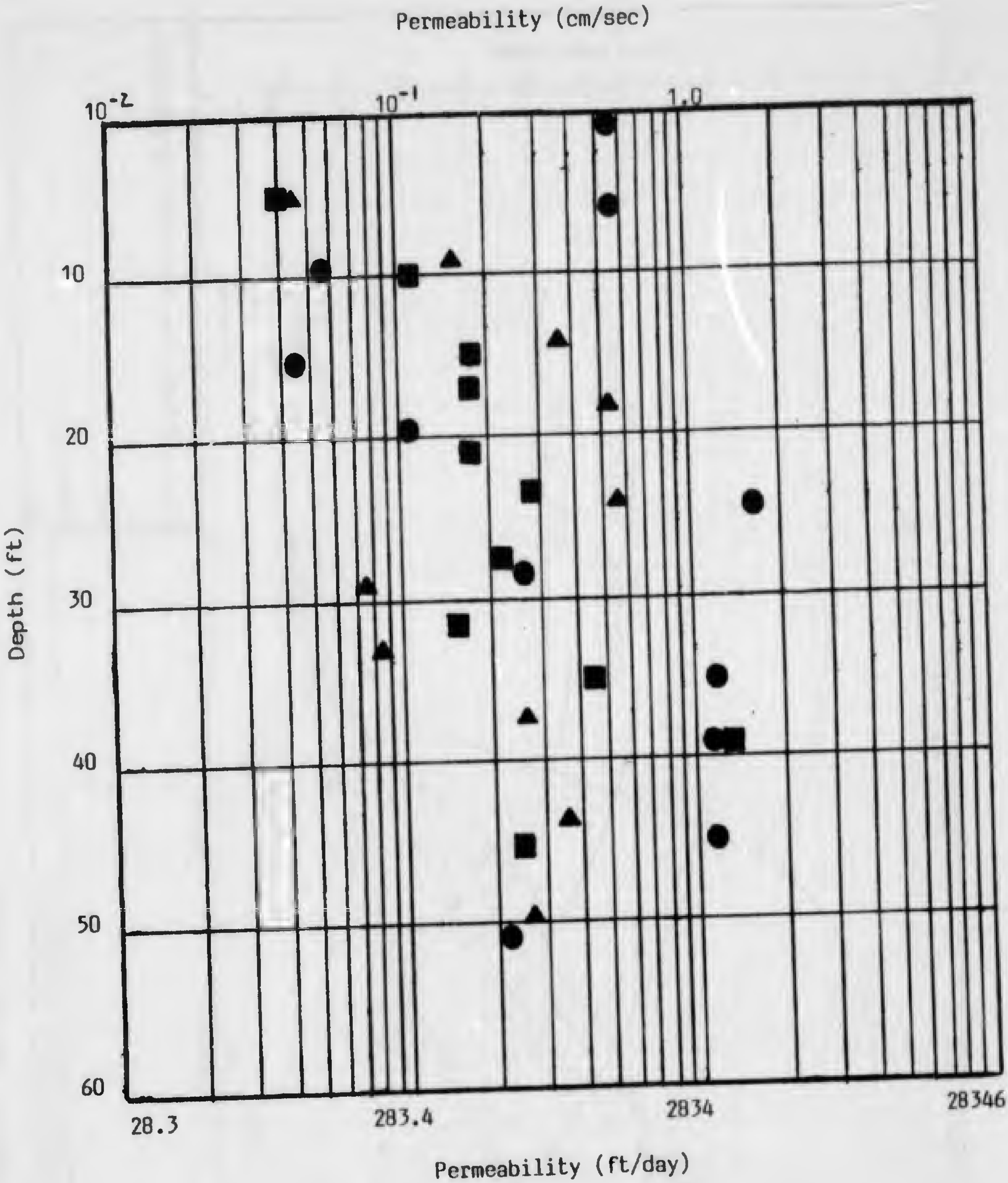
Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

DENSITY AND GRADATIONS OF VARIOUS PROJECTS

FIG 7



CALCULATED PERMEABILITY VS DEPTH

$$K \text{ (cm/sec)} = 77(D_{20})^{2.32}$$

▲ = TT - 1
■ = TT - 2
● = TT - 3

REF Justin, Hinds & Craeger
1947, Engineering for Dams, Vol III

FIG 8

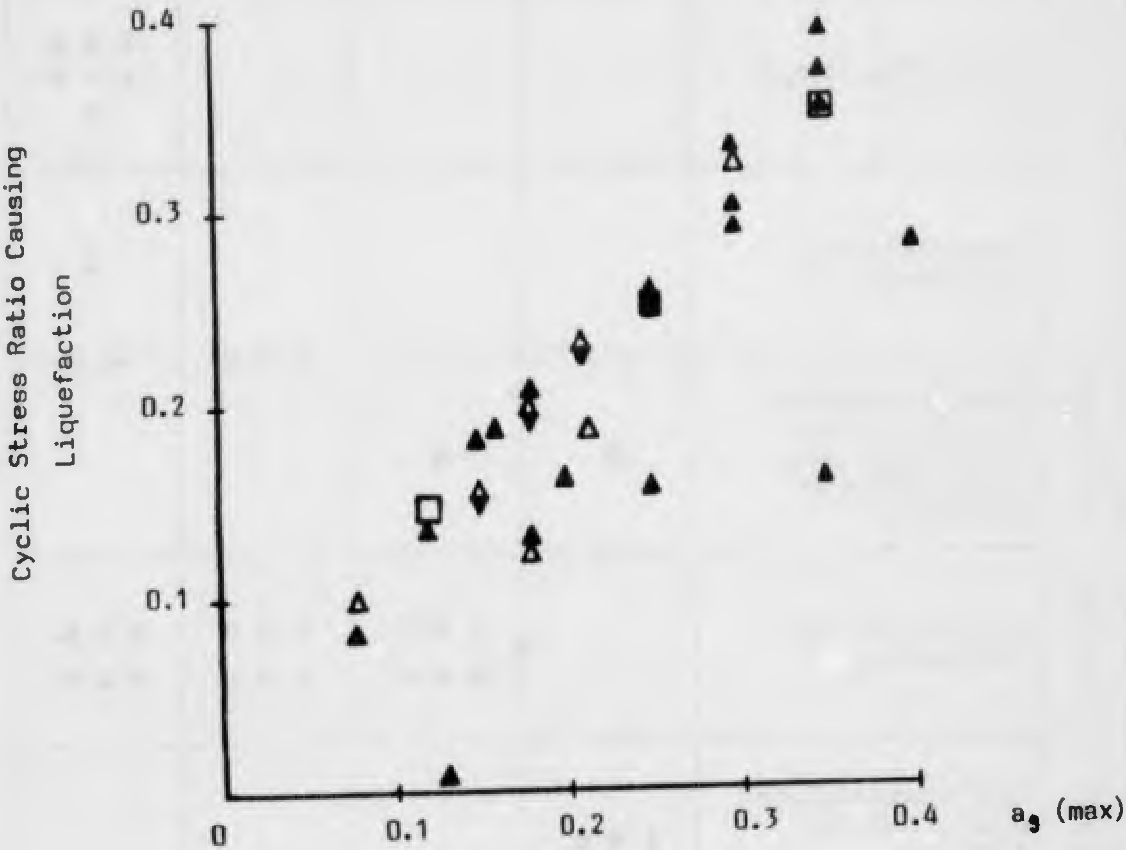
FOUNDATION SUPPORT CONDITIONS	FOUNDATION DISPLACEMENTS			
	Severe	Moderate	Minor	Nil
Founded directly on bedrock				●●●●●●●●
Piling to bedrock through cohesionless soils			●●●	●●●
Founded on bedrock at one end of bridge, directly or via piles; piling embedded in cohesionless soils over remaining length	●	●		
Piling embedded in gravels and gravelly sands		●●●●●●●●	●●●●●●	●●●●●●
Piling embedded in saturated medium to dense sands and silts (20 < N < 40 approx)	●●●			
Piling driven into medium to dense sand and silts (N > 20) through saturated loose to medium-dense sands and silts (N < 20)	●●●●●●●●			
Piling embedded in saturated loose to medium-dense sands and silts (N < 20)	●●●●●●●●●●	●	●●	

Note: Number of cases classified was limited by availability of data to 60 from a total of approximately 120 bridges on the three highways.

BRIDGE FOUNDATION MOVEMENTS

FIG 9

PLOT OF CASE HISTORY DATA
FOR STRESS RATIO CAUSING LIQUEFACTION IN THE
FIELD AND MAXIMUM GROUND SURFACE
ACCELERATION



LEGEND

Δ = No liquefaction of sand, sandy silt & silty sand

□ = No liquefaction of gravel & sand

▲ = Liquefaction of sand, sandy silt & silty sand

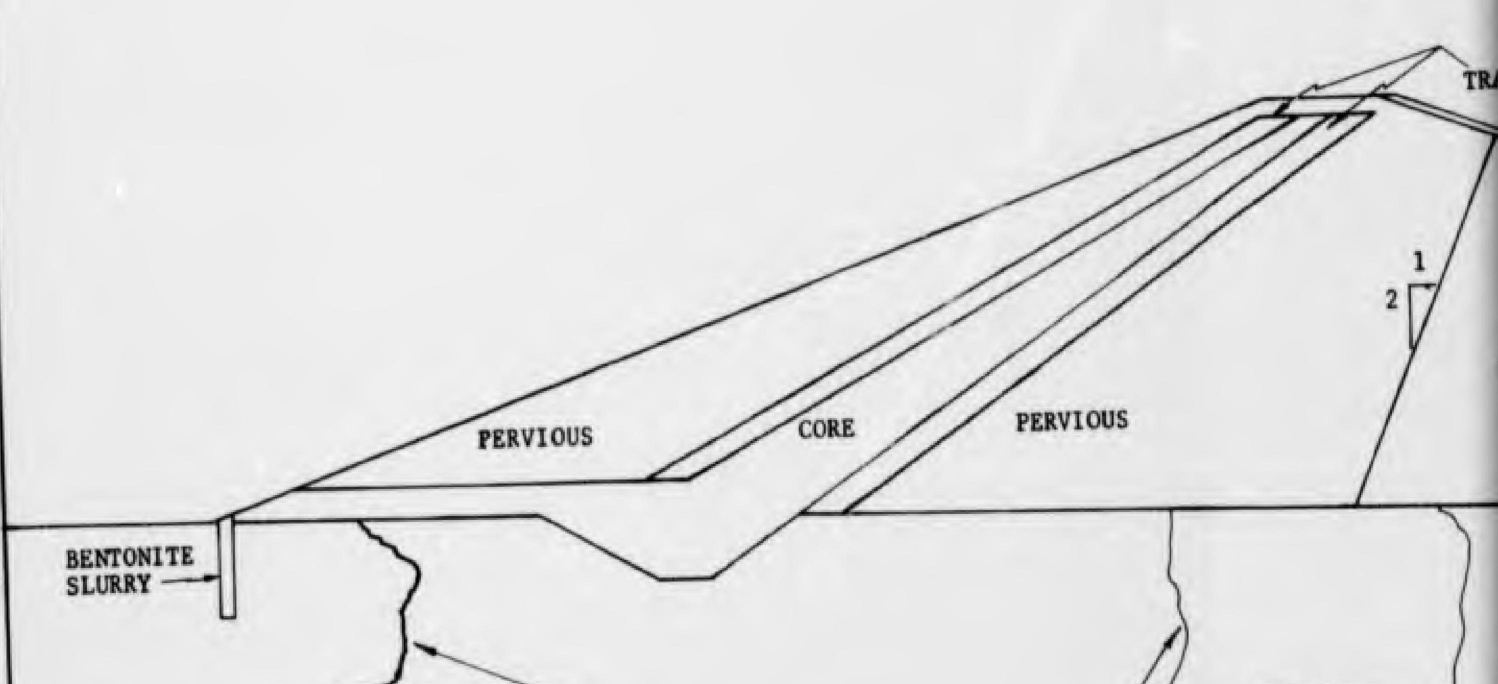
■ = Liquefaction of gravel & sand

◆ = Both cases of liqefaction and Non-liquefaction have been recorded

REF Liquefaction Problems in Geotechnical Engineering
ASCE National Convention Sept 27-Oct 1, 1976
Page 70

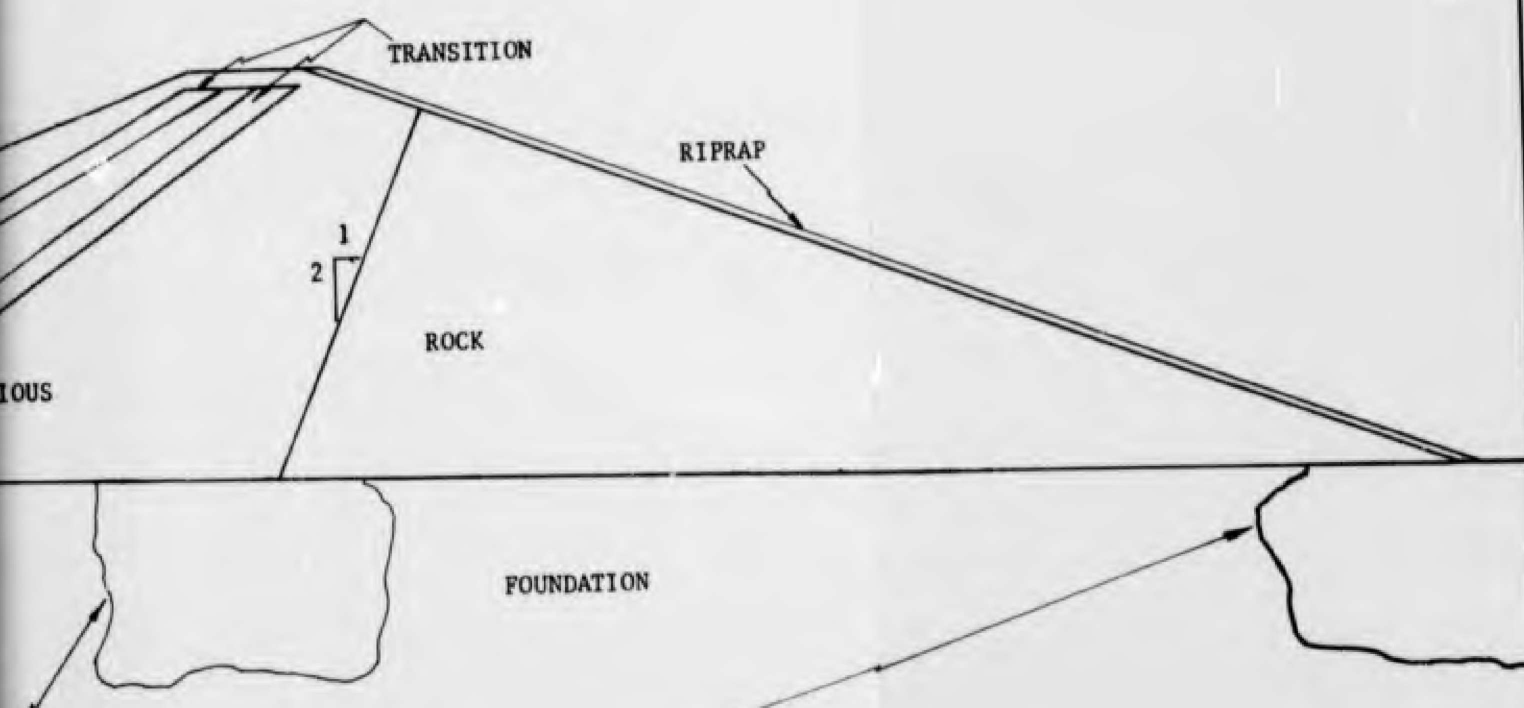
FIG 10

MENTONE DAM



TR
1
2
PERVIOUS
CORE
PERVIOUS
BENTONITE SLURRY
ZONES OF POTENTIAL LIQUEFACTION
BASED ON PREVIOUS STUDIES

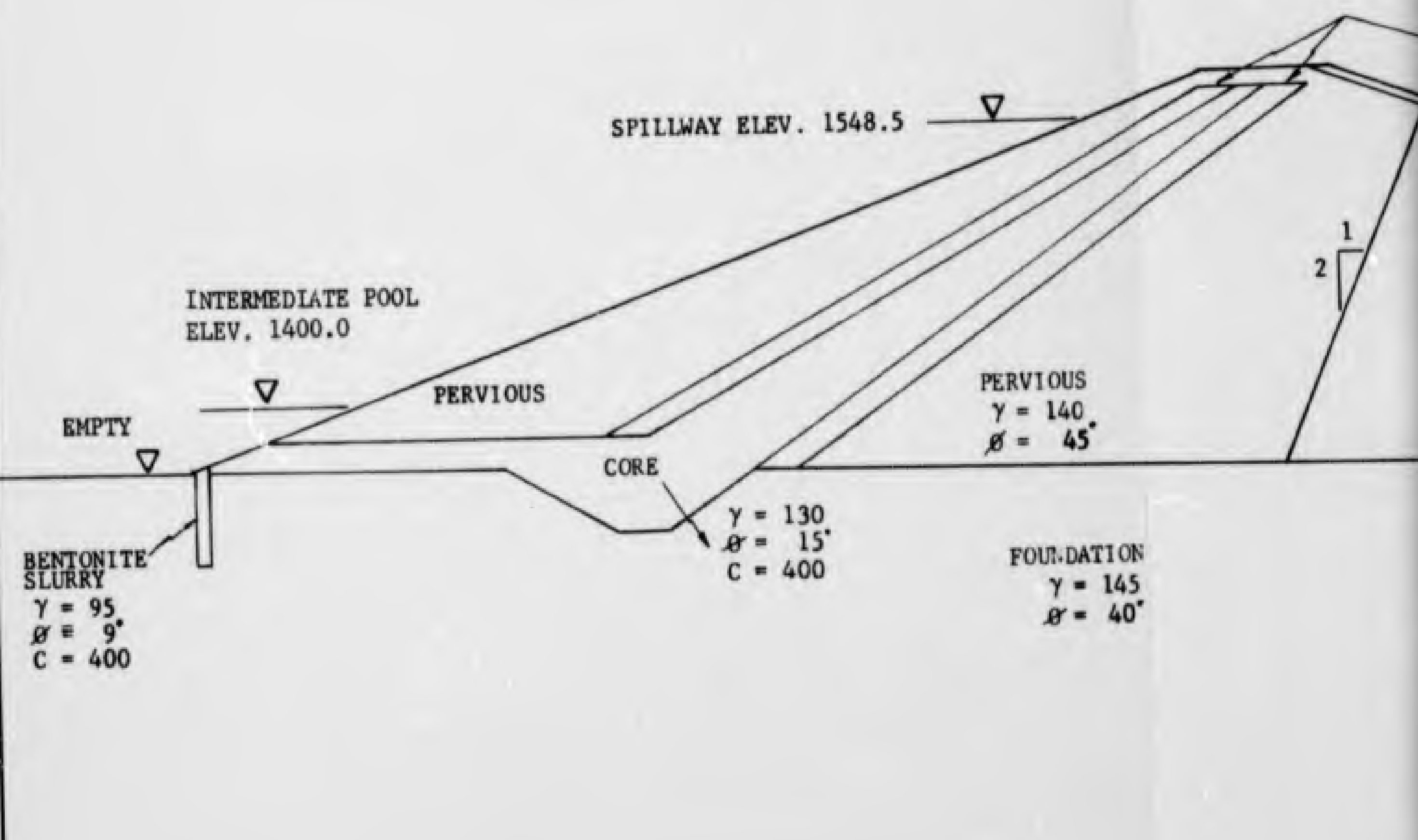
MENTONE DAM



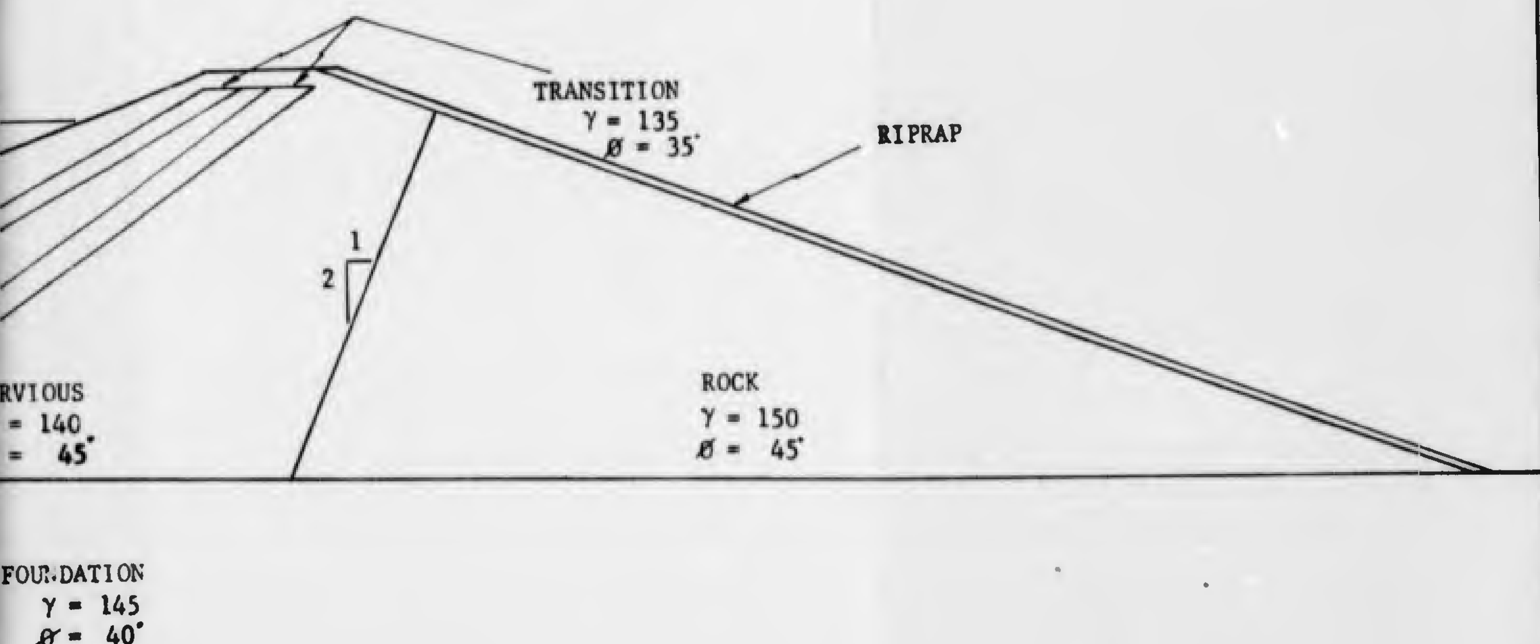
ZONES OF POTENTIAL FOUNDATION LIQUEFACTION

FIG. 11

MENTONE DAM

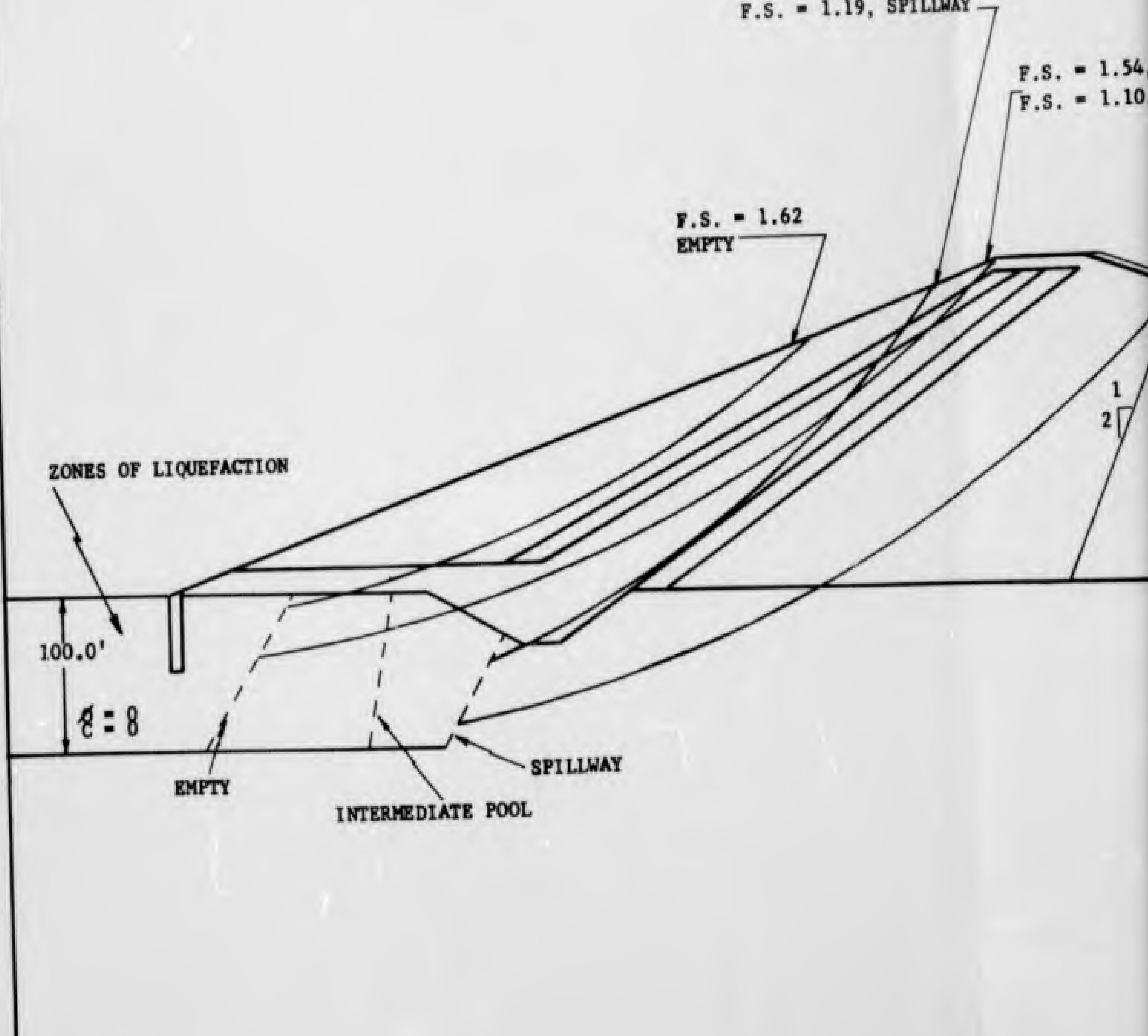

SPILLWAY ELEV. 1548.5
INTERMEDIATE POOL
ELEV. 1400.0
EMPTY
PERVIOUS
PERVIOUS
γ = 140
ø = 45°
CORE
γ = 130
ø = 15°
C = 400
BENTONITE SLURRY
γ = 95
ø = 9°
C = 400
FOUNDATION
γ = 145
ø = 40°
1
2

MENTONE DAM

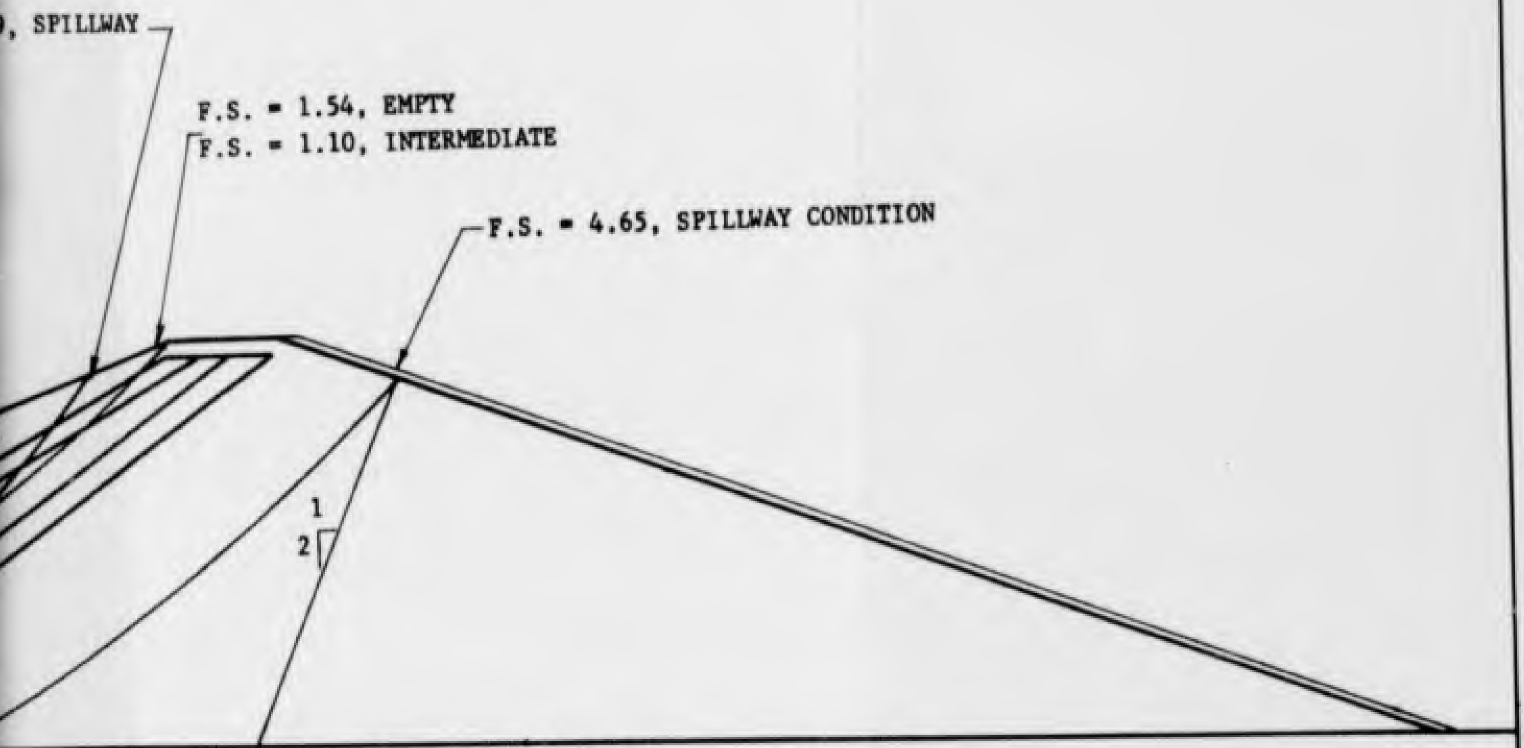


STABILITY DESIGN VALUES

FIG. 12

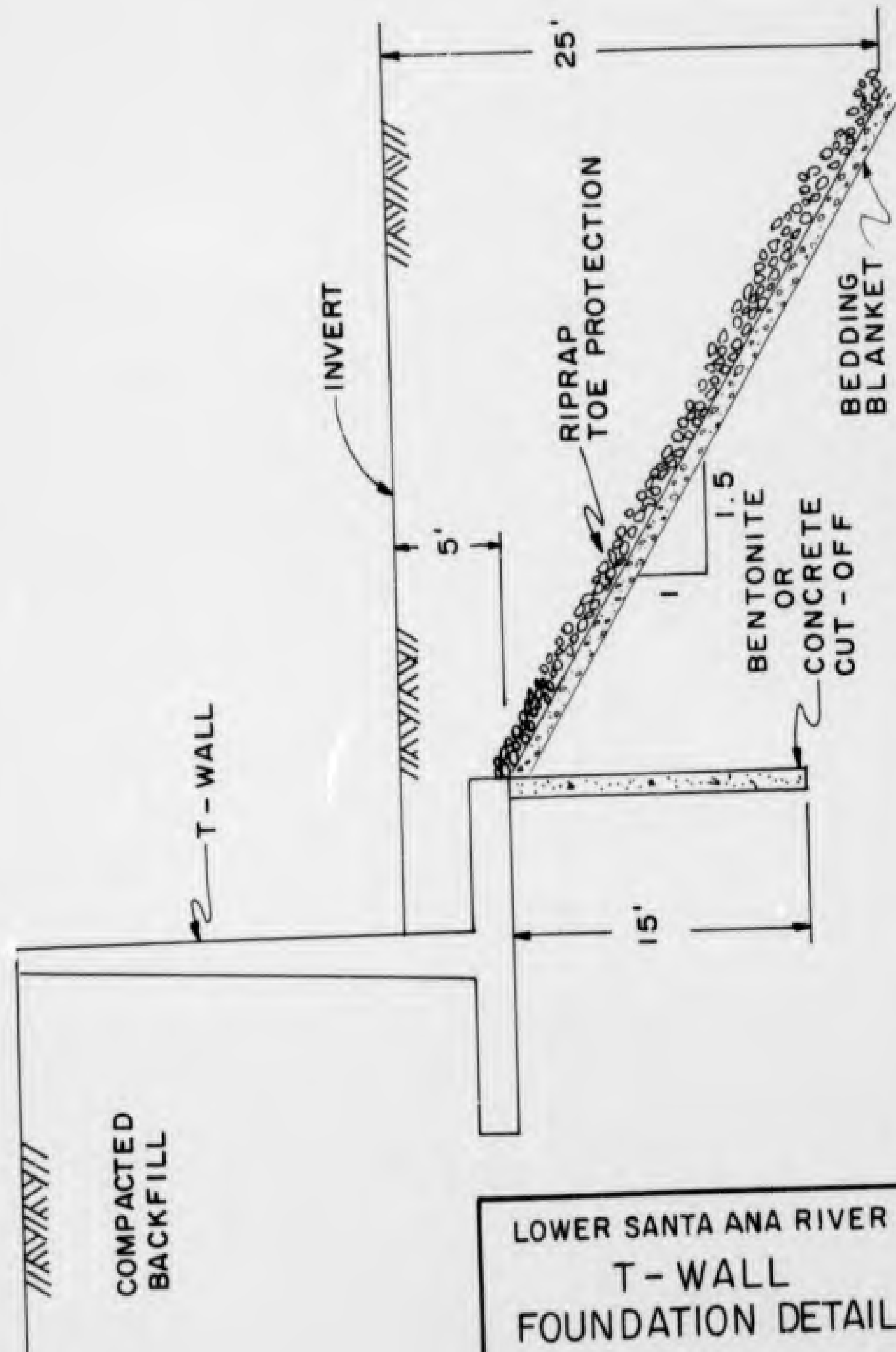


F.S. = 1.19, SPILLWAY
F.S. = 1.54,
F.S. = 1.10,
F.S. = 1.62
EMPTY
1
2
ZONES OF LIQUEFACTION
100.0'
EMPTY
INTERMEDIATE POOL
SPILLWAY



FACTORS OF SAFETY

FIG. 13



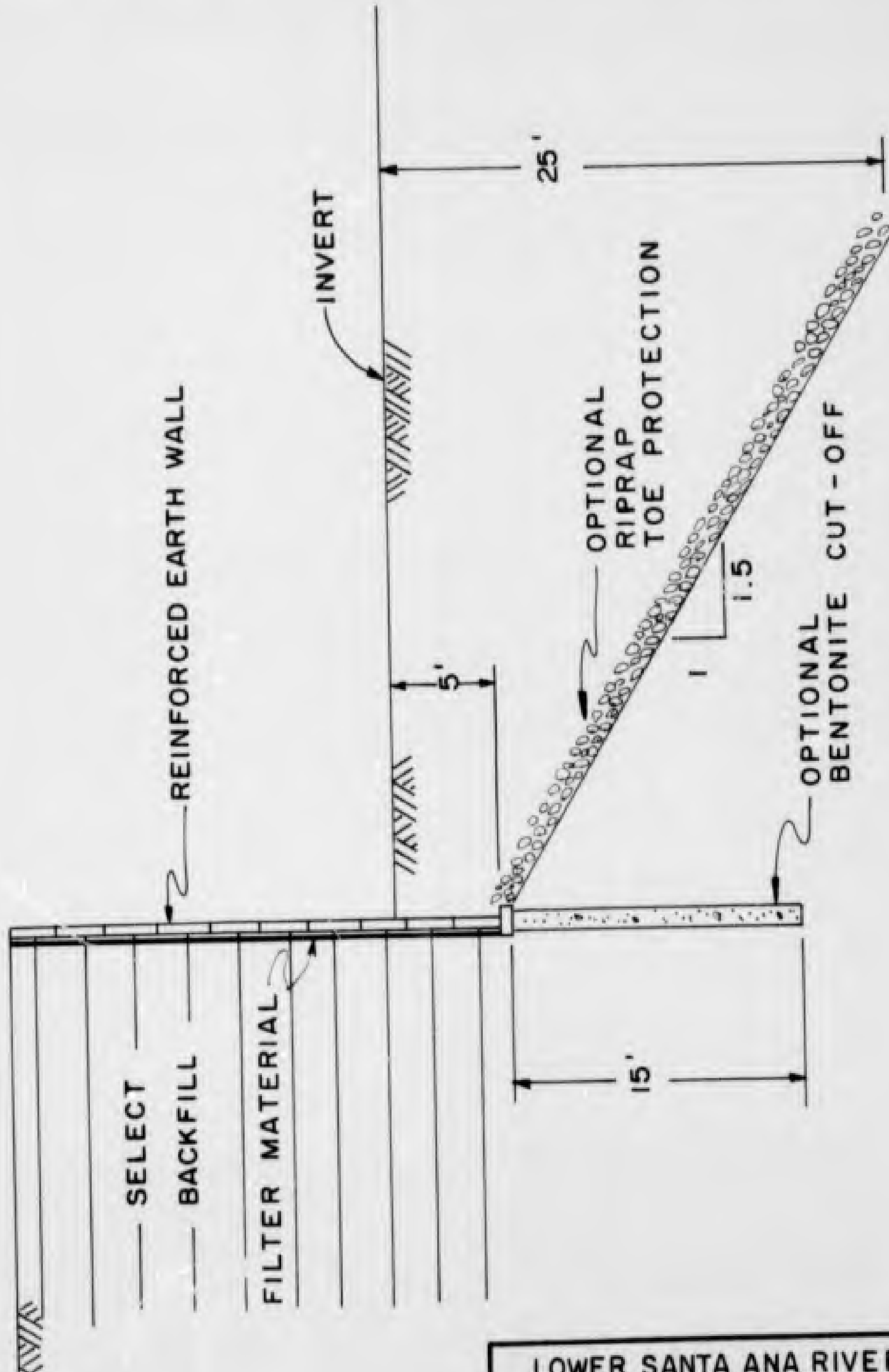
LOWER SANTA ANA RIVER

T-WALL
FOUNDATION DETAIL

NO SCALE

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

FIGURE 14



LOWER SANTA ANA RIVER

REINFORCED EARTH WALL
FOUNDATION DETAIL

NO SCALE

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

FIGURE 15

KEY

LEGEND

RECOMMENDED PLAN

CHANNEL IMPROVEMENT

DAM AND RESERVOIR

EXISTING

FLOOD CONTROL DAM

CHANNEL

LEVEE

BOUNDARY OF SANTA ANA RIVER DRAINAGE AREA

5 0 5

SCALE IN MILES

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
SANTA ANA RIVER BASIN
LOCATION MAP

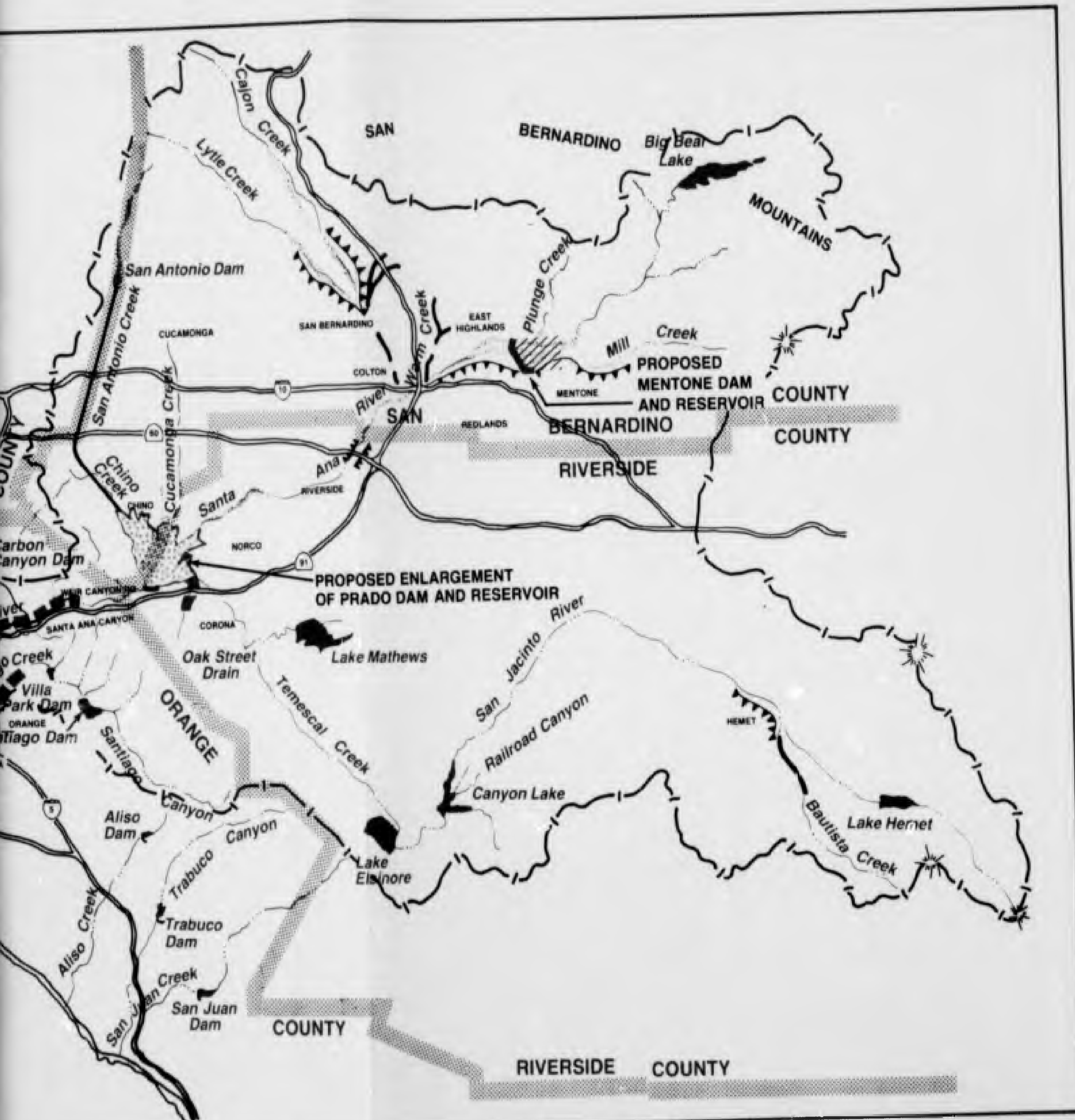

SAN
BERNARDINO
MOUNTAINS
Big Bear Lake
Cajon Creek
Lytle Creek
San Antonio Dam
San Antonio Creek
CUCAMONGA
Cucamonga Creek
SAN BERNARDINO
EAST HIGHLANDS
Warm Creek
Plunge Creek
Mill Creek
PROPOSED MENTONE DAM AND RESERVOIR
MENTONE
COLTON
10
SAN BERNARDINO COUNTY
COUNTY
REDLANDS
60
RIVERSIDE
Santa Ana River
Chino Creek
CHINO
NORCO
91
Carbon Canyon Dam
PROPOSED ENLARGEMENT OF PRADO DAM AND RESERVOIR
WEIR CANYON
SANTA ANA CANYON
CORONA
Oak Street Drain
Lake Mathews
Villa Park Dam
ORANGE
Santiago Dam
Santiago Canyon
ORANGE
Temescal Creek
San Jacinto River
Railroad Canyon
Canyon Lake
HEMET
Lake Hemet
Bautista Creek
5
Aliso Dam
Trabuco Canyon
Lake Elsinore
Aliso Creek
Trabuco Dam
San Juan Creek
San Juan Dam
COUNTY
RIVERSIDE COUNTY

PLATE D-1

OAK STREET DEBRIS BASIN
LOCAL GEOLOGY

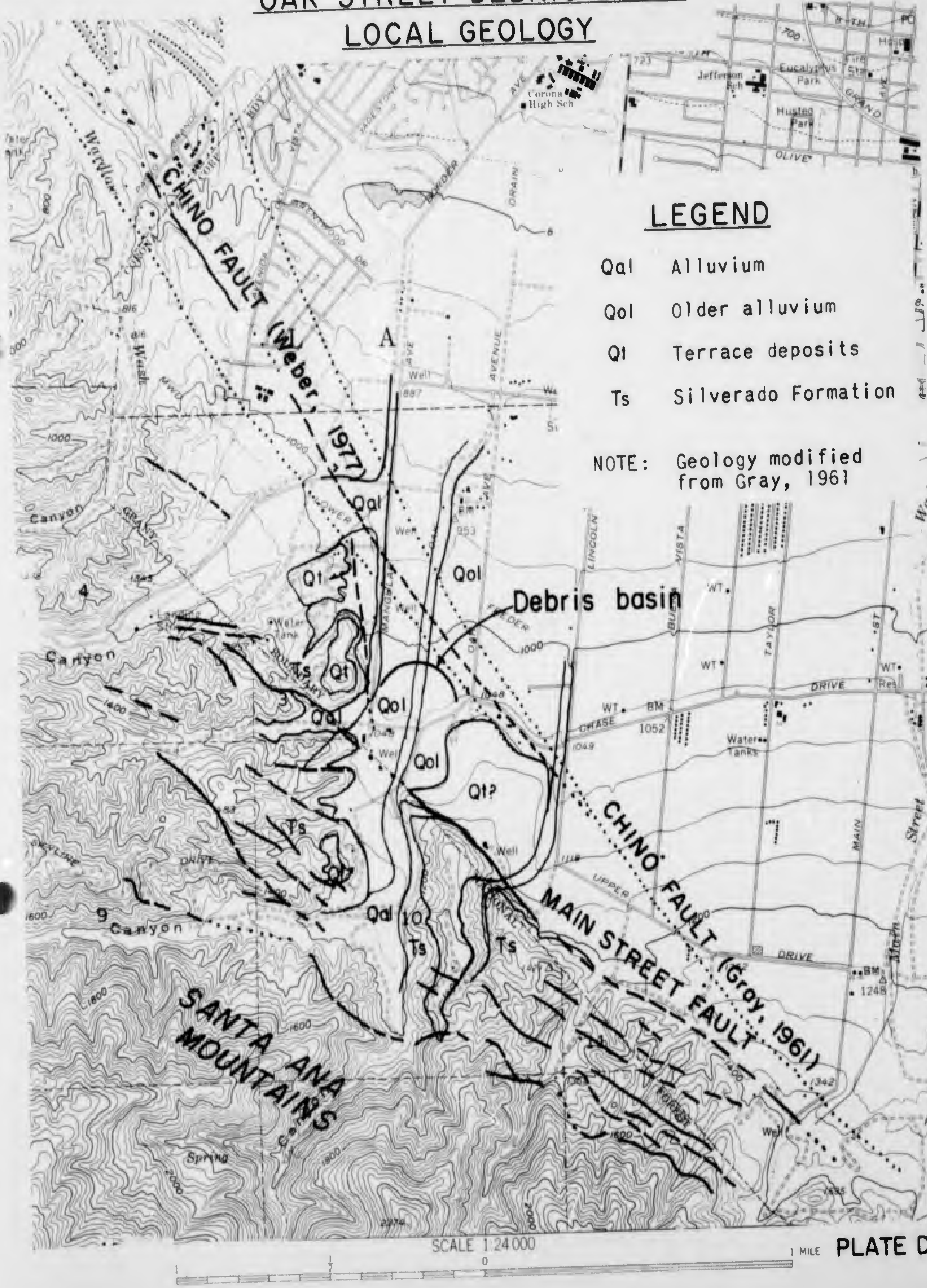
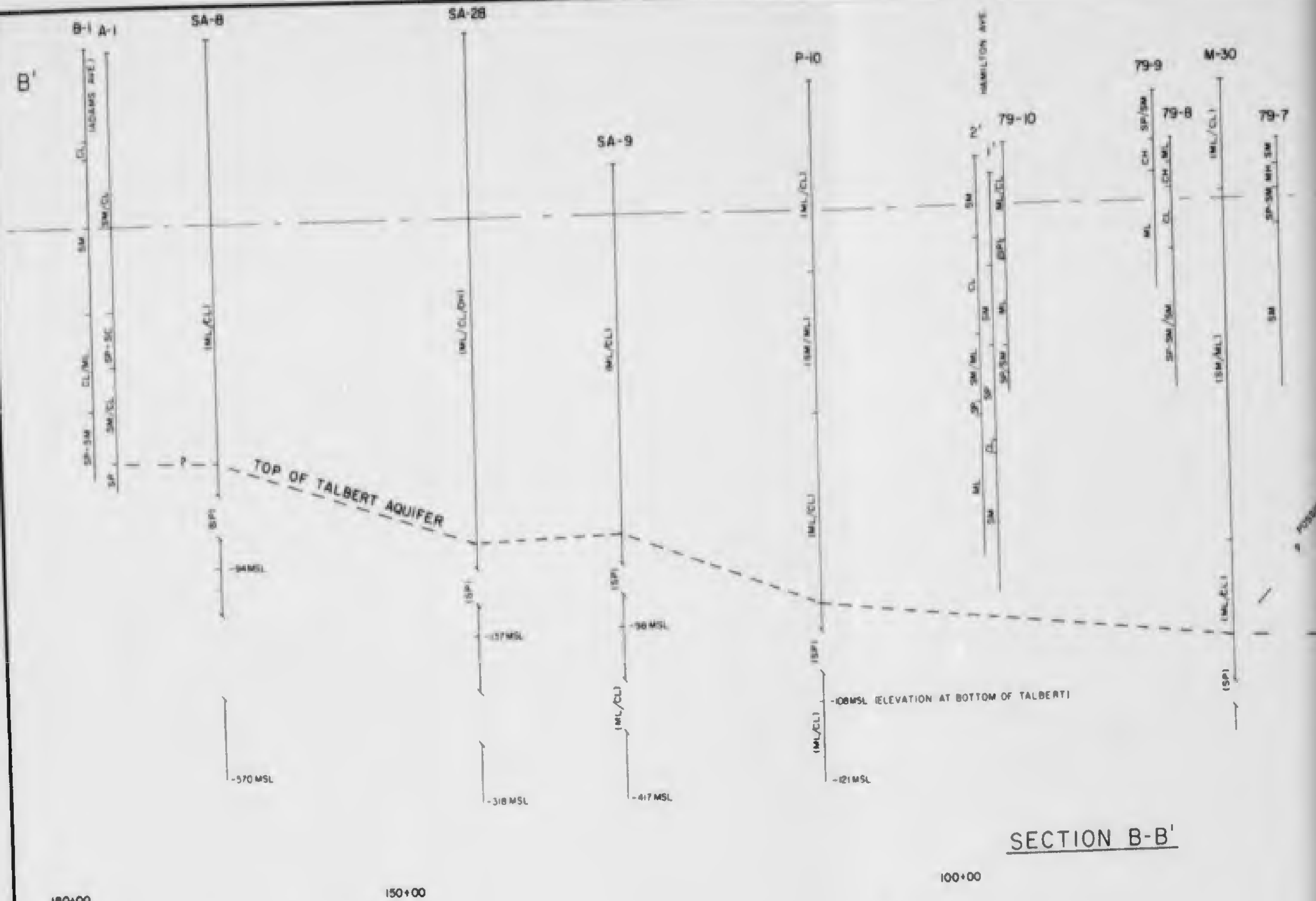
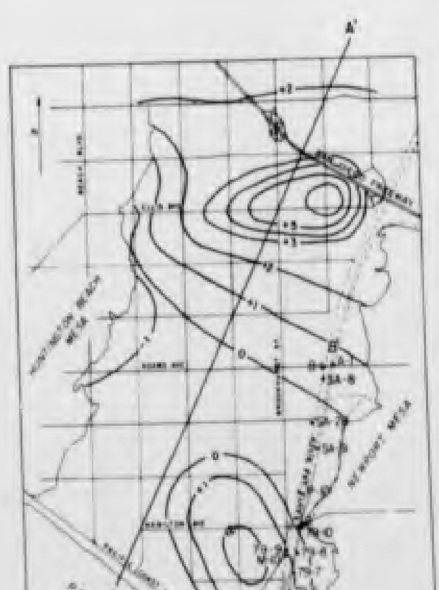


PLATE D-2

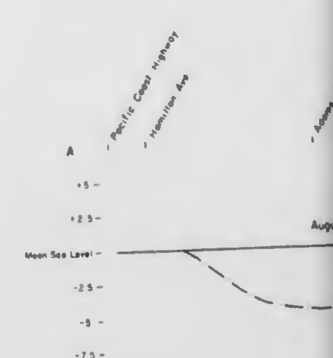


SECTION B-B'

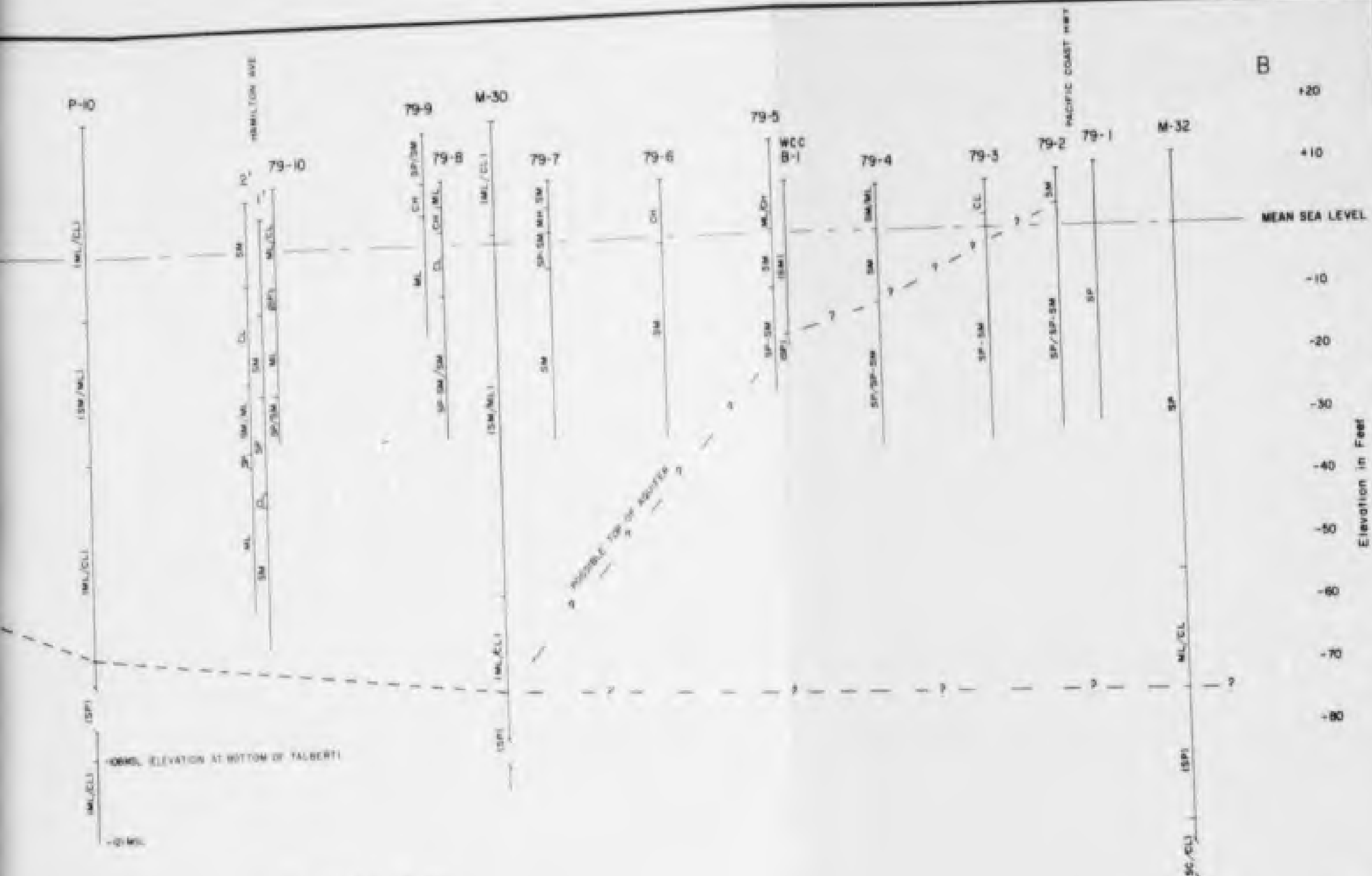
GROUNDWATER MONITORING LOCATIONS



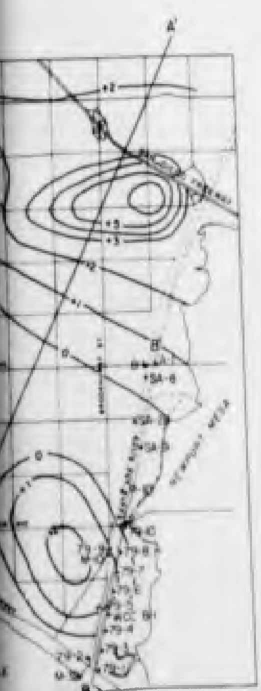
TALBERT AQUIFER WATER LEVEL
AUGUST 1979
LOCATION OF WELLS AND BORINGS



PIEZOMETR
TALBERT



SECTION B-B'



AQUIFER WATER LEVEL
AUGUST 1979
OF WELLS AND BORINGS

PIEZOMETRIC PROFILES
TALBERT AQUIFER
A-A'

NOTES

1. Soils borings 79-1 thru 79-10 by U.S. Army Corps of Engineers, 1979.
2. Soils boring WCC B-1 by Woodward Clyde Consultants.
3. Exploratory wells M-30, P-10, SA-9, SA-28 and SA-8 were drilled for the California State Department of Water Resources.
4. Well M-32 was logged by Glenn A. Brown and Associates, 1971.
5. Soils borings 1', 2', A-1 and B-1 were engineering investigations for bridge crossings at Hamilton and Adams Avenues.
6. Soils are classified by the Unified Soils Classification System.
7. Where soils symbols are in parentheses, (SP), the classification is based upon a visual description and not laboratory analysis.
8. Symbols separated by a slash, SM/ML, indicate interbedded soils or a composite description.
9. Groundwater contours and monitoring locations provided by the Orange County Water District.

REVISIONS

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

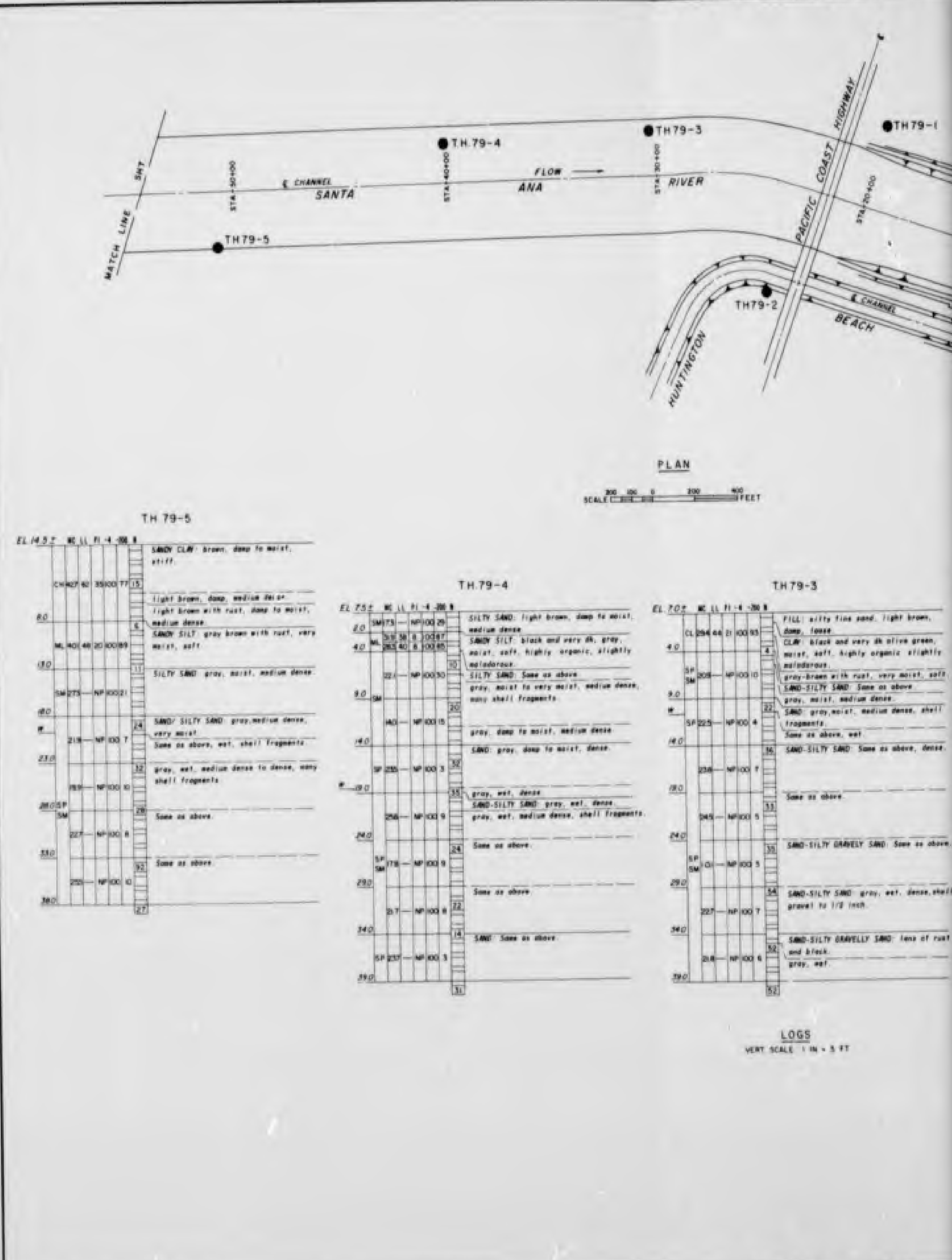
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

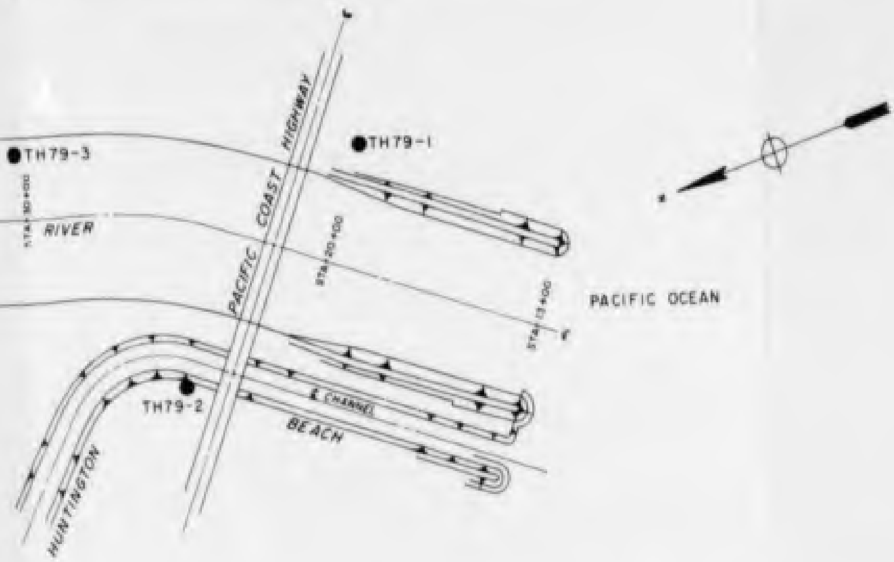
LOWER SANTA ANA RIVER
SEA WATER INTRUSION

DESIGNED BY: DWL
DRAWN BY: DWL
CHECKED BY:
SUBMITTED BY:
APPROVAL RECOMMENDED:
APPROVED:
SPEC. NO. DACW 09-
DISTRICT FILE NO.
DATE:
SHEET OF SHEETS

PLATE D-3

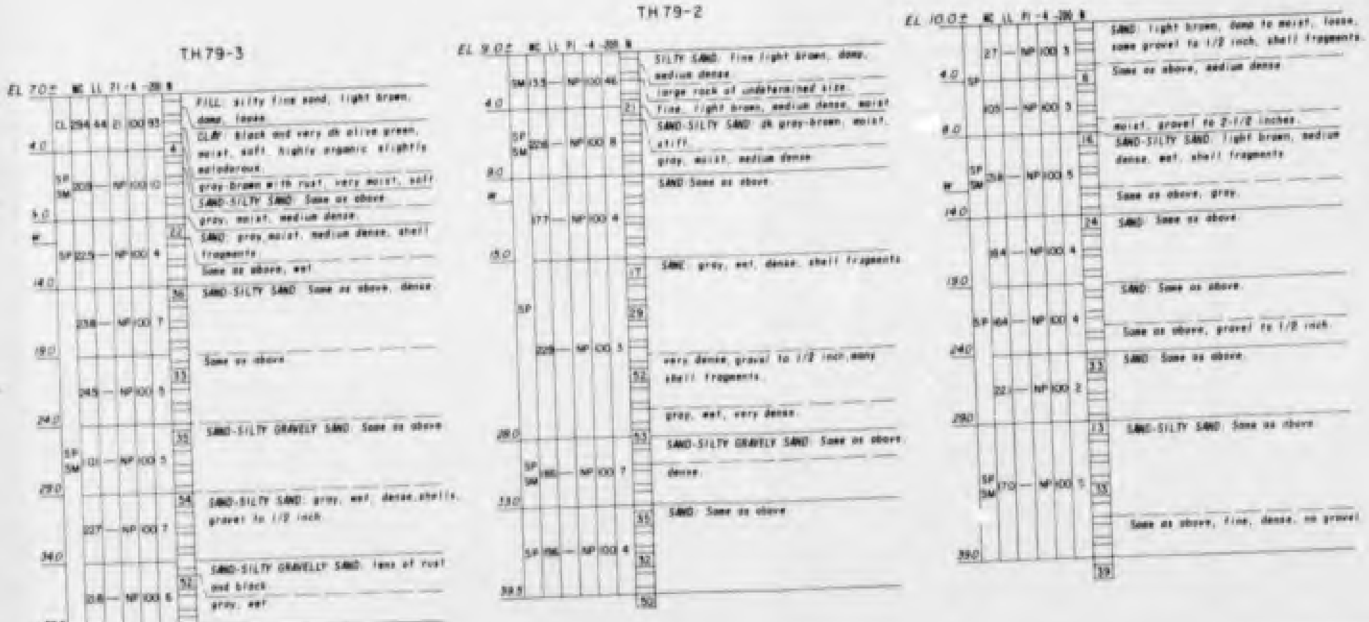
2


MATCH LINE SHT
STA 50+00
STA 40+00
STA 30+00
STA 20+00
℄ CHANNEL
SANTA
ANA
RIVER
FLOW
TH79-5
T.H. 79-4
TH79-3
TH79-1
TH79-2
PACIFIC COAST HIGHWAY
HUNTINGTON
BEACH
℄ CHANNEL
PLAN
SCALE 200 100 0 200 400 FEET
TH 79-5
EL 14.5±
SANDY CLAY: brown, damp to moist, stiff.
light brown, damp, medium dense.
light brown with rust, damp to moist, medium dense.
SANDY SILT: gray brown with rust, very moist, soft.
SILTY SAND: gray, moist, medium dense.
SAND/ SILTY SAND: gray, medium dense, very moist.
Same as above, wet, shell fragments.
gray, wet, medium dense to dense, many shell fragments.
Same as above.
Same as above.
TH 79-4
EL 7.5±
SILTY SAND: light brown, damp to moist, medium dense.
SANDY SILT: black and very dk. gray, moist, soft, highly organic, slightly malodorous.
SILTY SAND: Same as above.
gray, moist to very moist, medium dense, many shell fragments.
gray, damp to moist, medium dense.
SAND: gray, damp to moist, dense.
gray, wet, dense.
SAND-SILTY SAND: gray, wet, dense.
gray, wet, medium dense, shell fragments.
Same as above.
Same as above.
SAND: Same as above.
TH 79-3
EL 7.0±
FILL: silty fine sand, light brown, damp, loose.
CLAY: black and very dk. olive green, moist, soft, highly organic, slightly malodorous.
gray-brown with rust, very moist, soft.
SAND-SILTY SAND: Same as above.
gray, moist, medium dense.
SAND: gray, moist, medium dense, shell fragments.
Same as above, wet.
SAND-SILTY SAND: Same as above, dense.
Same as above.
SAND-SILTY GRAVELY SAND: Same as above.
SAND-SILTY SAND: gray, wet, dense, shells, gravel to 1/2 inch.
SAND-SILTY GRAVELLY SAND: lens of rust and black.
gray, wet.
LOGS
VERT SCALE 1 IN = 5 FT



PLAN

SCALE 200 100 0 100 400 FEET



LOGS
VERT SCALE: 1 IN = 5 FT

NOTES
1. See plate D10 for legend and soil classification.
2. Test holes drilled in August 1979 using a hollow stem auger.

VERT SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

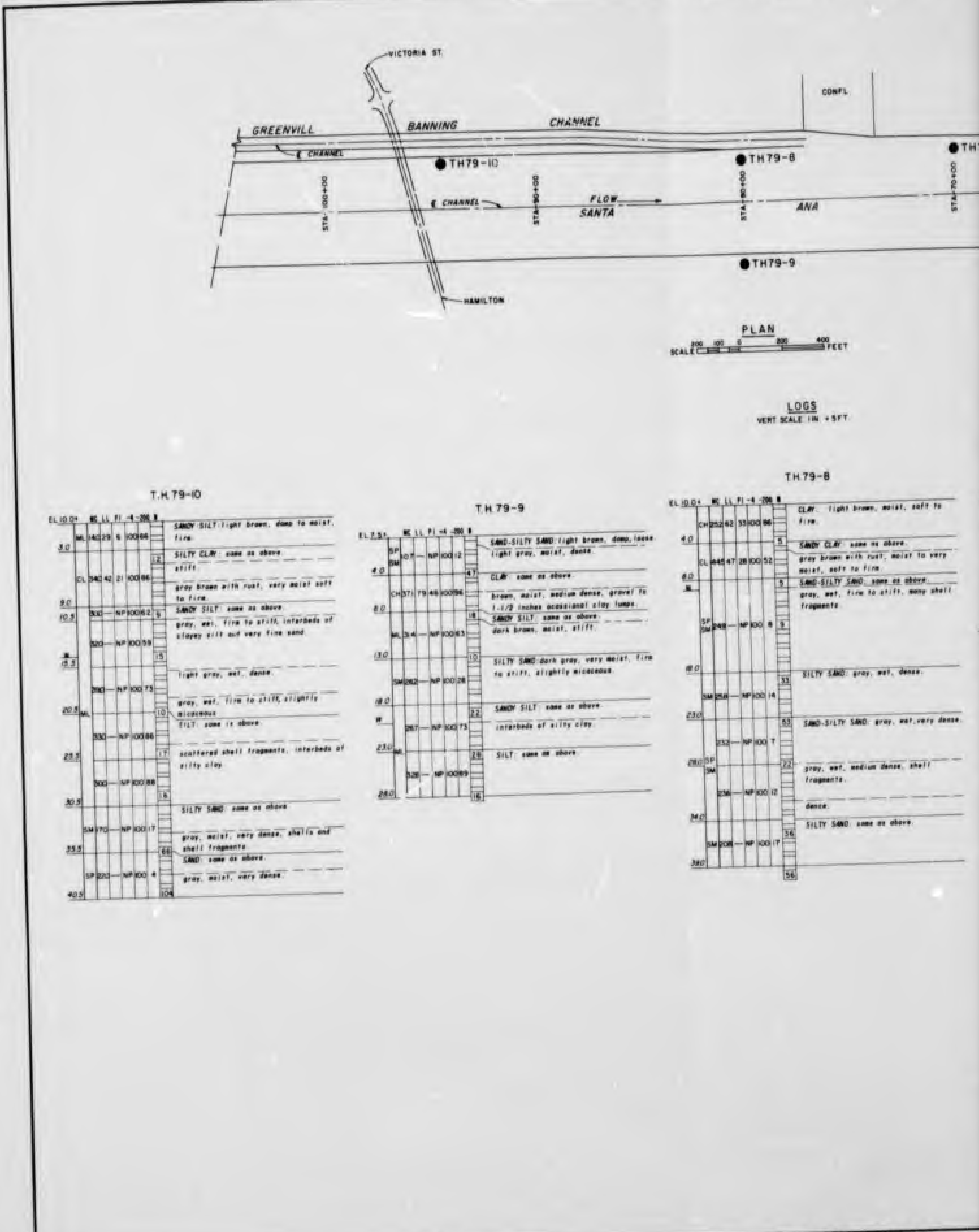
DESIGNED BY:
DRAWN BY: J. J. B.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE 2 GENERAL DESIGN MEMORANDUM
LOWER SANTA ANA RIVER
FOUNDATION INVESTIGATION
TEST HOLES TH 79-1 TO TH 79-5

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

PLATE D-4

2


VICTORIA ST.
CONFL
GREENVILL
BANNING
CHANNEL
TH79-10
TH79-8
FLOW
SANTA
ANA
TH79-9
HAMILTON
PLAN
LOGS
T.H. 79-10
T.H. 79-9
T.H. 79-8

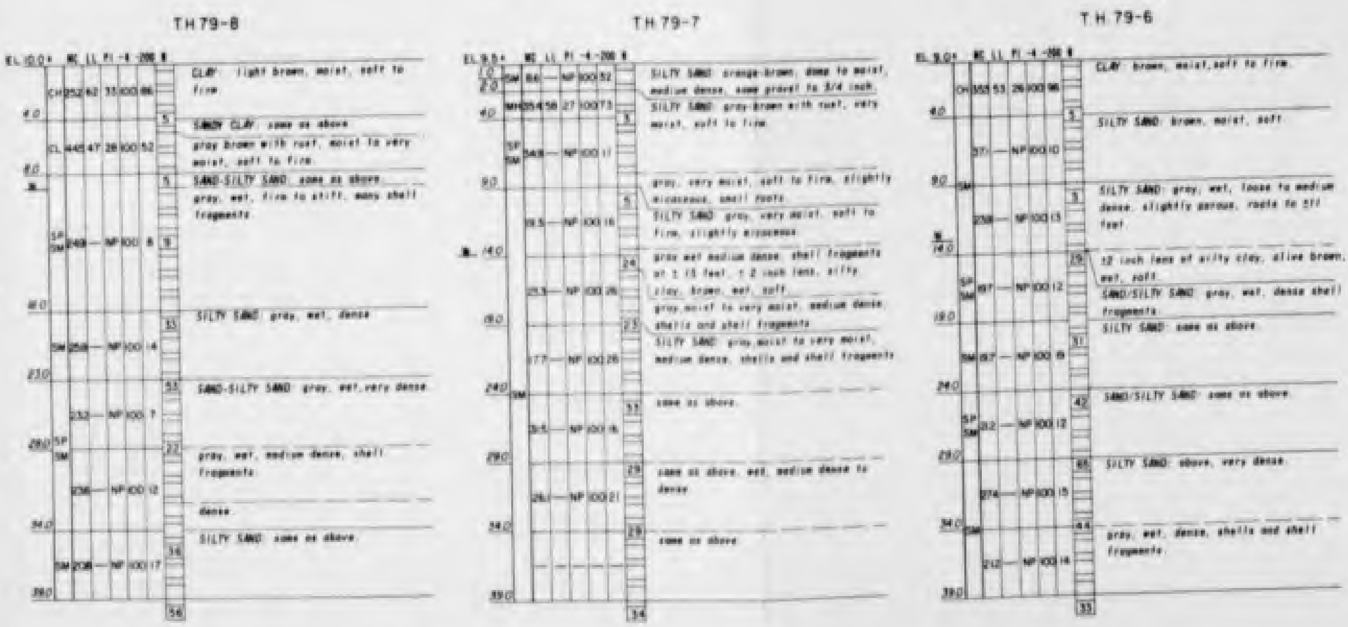
VALUE ENGINEERING PAYS

PLAN

SCALE 200 100 0 200 400 FEET

LOGS

VERT SCALE 1IN = 5FT



NOTES

1. See Plate D-10 for Legend and Soil Classification
2. Test Holes drilled in August 1979 using a hollow stem auger.

VERT SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY:

DRAWN BY: J.O.B.

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

LOWER SANTA ANA RIVER
FOUNDATION INVESTIGATION

TEST HOLES TH79-6 TO TH 79-10

SUBMITTED BY:

DATE APPROVED:

SPEC. NO. DACW09-___ B-___

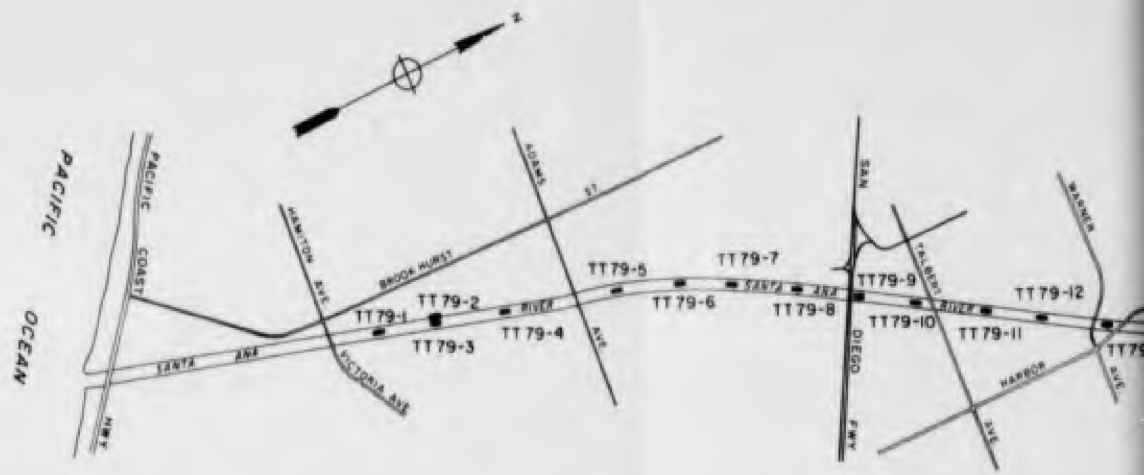
DISTRICT FILE NO.

SHEET

PLATE D-5

SAFETY PAYS

2



PLAN

SCALE 1000 0 1000 2000 3000 FEET

T.T. 79-1

Depth		MC	LL	PI	-4	-200	Description
4.0	SM	11		NP	98	19	SILTY SAND, light brown, damp, vegetation
7.0	ML	39	33	9	100	74	SANDY SILT, gray, wet, some cohesion, ground water encountered
10.0	CL	38	36	15	100	79	SANDY CLAY, black, some cohesion organic odor trenching stopped due to caving

T.T. 79-2

Depth		MC	LL	PI	-4	-200	Description
	SP	14		NP	98	1	SAND, gray, moist, vegetation
4.0		26		NP	100	2	gray, moist to wet

water stands at 3 feet

T.T. 79-3

Depth		MC	LL	PI	-4	-200	Description
2.0	SP SM			NP	99	8	SAND-SILTY SAND, gray, moist, vegetation, some cohesion
4.0	SM			NP	100	43	SILTY SAND, gray, wet, some clay

T.T. 79-6

Depth		MC	LL	PI	-4	-200	Description
	SM	11		NP	98	34	SILTY SAND, gray, moist
7.0		16		NP	100	41	brown, moist
10.0	SP SM	20		NP	100	10	SAND-SILTY SAND, brown, wet

T.T. 79-7

Depth		MC	LL	PI	-4	-200	Description
4.0	SM	7		NP	91	13	SILTY SAND, gray, moist, few cobbles to 12 inch max.
	SP	10		NP	100	2	SAND, gray, moist
10.0		16		NP	99	2	same as above

T.T. 79-8

Depth		MC	LL	PI	-4	-200	Description
4.0	SP SM	8		NP	90	8	SAND-SILTY SAND, gray, moist, occasional
	SP	6		NP	100	2	SAND, gray, moist
10.0		16		NP	95	1	same as above

T.T. 79-11

Depth		MC	LL	PI	-4	-200	Description
3.0	SP SM	4		NP	95	7	SAND-SILTY SAND, brown-gray, damp
4.0	SM	22		NP	100	35	SILTY SAND, black, moist

ground water encountered

T.T. 79-12

Depth		MC	LL	PI	-4	-200	Description
	SP	13		NP	99	2	SAND, gray, moist.
3.5				NP	88	1	gray, moist, gravel to 3/4 inch

T.T. 79-13

Depth		MC	LL	PI	-4	-200	Description
2.0	SP	2		NP	96	1	SAND, gray, humid
3.0	SM			NP	93	11	SAND-SILTY SAND, gray, moist

ground water encountered

T.T. 79-16

Depth		MC	LL	PI	-4	-200	Description
2.5	SP	4		NP	95	1	SAND, gray, damp, occasional gravel to 3/4 inch vegetation

ground water encountered at 2.5 feet

T.T. 79-17

Depth		MC	LL	PI	-4	-200	Description
1.0	SP	6		NP	98	1	SAND, gray, moist, vegetation

T.T. 79-18

Depth		MC	LL	PI	-4	-200	Description
1.0	SP	6		NP	97	1	SAND, gray, moist, vegetation

ground water encountered

T.T. 79-20

Depth		MC	LL	PI	-4	-200	Description
1.0	SP	5		NP	81	0	SAND, gray, moist, gravel to 1 inch

ground water encountered

T.T. 79-21

Depth		MC	LL	PI	-4	-200	Description
1.0	SP	1		NP	61	1	GRAVELLY SAND, gray, humid, vegetation, gravel to 2 inches

ground water encountered

T.T. 79-22

Depth		MC	LL	PI	-4	-200	Description
2.0	SP	4		NP	59	3	GRAVELLY SAND, gray, moist, organic odor, vegetation, occasion boulders to 24 inch
	CL	32	44	22	100	86	SANDY CLAY, brown, wet, cohesive.
		26	31	13	100	57	brown, wet, cohesive
10.0		27	31	12	100	56	brown, wet, cohesive, running water

ground water encountered.

PLAN

SCALE 1000 0 1000 2000 3000 FEET

T.T. 79-3

Depth	Class	MC	LL	PI	-4	-200	Description
2.0	SP SM			NP	99	8	SAND-SILTY SAND, gray, moist, vegetation, some cohesion
4.0	SM			NP	100	43	SILTY SAND, gray, wet, some clay

T.T. 79-4

Depth	Class	MC	LL	PI	-4	-200	Description
	ML	6		NP	100	84	SANDY SILT, gray, moist, vegetation
7.0	ML	14		NP	100	81	gray, moist
10.0	CH	45	60	30	100	91	SANDY CLAY, gray, wet

T.T. 79-5

Depth	Class	MC	LL	PI	-4	-200	Description
4.0	SW SM	30		NP	79	11	GRAVELLY SAND - SILTY GRAVELLY SAND, gray, moist, vegetation
7.0	SM	22		NP	100	46	SILTY SAND, gray, moist
10.0	ML			NP	100	69	SANDY SILT, gray, moist, some cohesion

T.T. 79-8

Depth	Class	MC	LL	PI	-4	-200	Description
4.0	SP SM	8		NP	90	8	SAND-SILTY SAND, gray, moist, occasional
	SP	6		NP	100	2	SAND, gray, moist
10.0	SP	16		NP	95	1	same as above

T.T. 79-9

Depth	Class	MC	LL	PI	-4	-200	Description
4.0	SW	3		NP	89	2	SAND, gray, humid
	SP	14		NP	94	3	SAND, gray, moist
10.0	SP	16		NP	96	4	ground water encountered

T.T. 79-10

Depth	Class	MC	LL	PI	-4	-200	Description
4.0	SP SM	3		NP	95	5	SAND-SILTY SAND, gray, humid, some gravel to 1/2-inch

T.T. 79-13

Depth	Class	MC	LL	PI	-4	-200	Description
2.0	SP	2		NP	96	1	SAND, gray, humid
3.0	SM			NP	93	11	SAND-SILTY SAND, gray, moist

ground water encountered

T.T. 79-14

Depth	Class	MC	LL	PI	-4	-200	Description
2.0	SP	4		NP	98	0	SAND, gray, humid; gray, moist

ground water encountered at 2 feet

T.T. 79-15

Depth	Class	MC	LL	PI	-4	-200	Description
1.0	SP	8		NP	100	1	SAND, gray, moist, vegetation

ground water encountered

NOTES:
1. See Plate D10 for Legend and Soil Classi ication.
2. Test Trenches were Excavated with a bac hoe in July 1979
3. Although the surface of all Test Trenches was originally at the channel invert, the Winter 1980 floods recut the invert as shown.

TT 79-18

Depth	Class	MC	LL	PI	-4	-200	Description
1.0	SP	6		NP	97	1	SAND, gray, moist, vegetation

ground water encountered

TT 79-19

Depth	Class	MC	LL	PI	-4	-200	Description
1.0	SP	6		NP	98	2	SAND, gray, moist, vegetation, occational gravel to 3/4 inch

ground water encountered

TT 79-22

Depth	Class	MC	LL	PI	-4	-200	Description
2.0	SP	4		NP	59	3	GRAVELLY SAND, gray, moist, organic odor, vegetation, occasion boulders to 24 inch
	CL	32	44	22	100	86	SANDY CLAY, brown, wet, cohesive
	CL	26	31	13	100	57	brown, wet, cohesive
10.0	CL	27	31	12	100	58	brown, wet, cohesive, running water

ground water encountered

VERT SCALE 5 0 5 10 15 FEET

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

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

DESIGNED BY:
DRAWN BY: LBA
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

LOWER SANTA ANA RIVER
FOUNDATION INVESTIGATION
TEST TRENCHES TT79-1 TO TT79-22

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

PLATE D-6

PLAN

SCALE 1000 0 1000 2000 FEET

(SCALE 1" = 1000')

TH 79-1

EL 112±

Depth		MC	LL	PI	-4	-200	N	Description
		5		NP	76	19	R	SILTY GRAVELLY SAND, Brown, Damp, medium dense, some pea-size gravel.
							24	
	SM	12		NP	92	30	7	SILTY SAND, Brown, moist, loose, chunks of clay, angular gravel to 1", cohesion.
9.0	SP-SM	6		NP	94	7	14	SAND-SILTY SAND, Light brown, damp, medium-dense, angular gravel to 1", no cohesion.
							18	
13.5	CL	21	39	21	100	79	27	SANDY CLAY, Brown, moist, very stiff, gravel to 1", cohesion.
16.5	SM	17		NP	100	49	20	SILTY SAND, Tan, moist, medium dense, gravel to 1/2", cohesion.
21.0							18	SANDY CLAY, Dark brown, moist to damp, stiff, rounded gravel to 1" max. (<5%), cohesion.
	CL	21	32	14	100	82	9	
25.5							17	SANDY CLAY-SANDY SILT, Gray brown, wet cohesion.
28.2	ML-CL	22	26	5	100	50		Hit ground water at 28'-0"
30.0	SM	23		NP	100	56		SILTY SAND, Gray brown, wet, cohesion.

TH 79-3

EL 145±

Depth		MC	LL	PI	-4	-200	N	Description
	SM	1		NP	92	28		SILTY GRAVELLY SAND, Tan dry, pea gravel to 1", medium dense.
3.0							17	SILTY SANDY GRAVEL, Tan, dry, gravel to 3".
	GM	2		NP	56	15	R	
9.0							R	GRAVELLY SAND-SILTY GRAVELLY SAND, Brown, damp, medium dense gravel to 1"
10.5	SM	2		NP	60	7		SILTY SAND, Brown moist, medium dense, pea gravel.
		10		NP	94	36	10	
	SM						19	GRAVELLY SILTY SAND, Brown, moist, medium dense, subrounded gravel to 1".
		9		NP	87	41	16	
19.5							15	SANDY CLAY-SANDY SILT, Brown, moist, stiff, subrounded gravel to 1".
	CL-ML	11	23	6	99	62		
24.0							12	SANDY CLAY, Brown, moist, stiff, subrounded gravel to 1".
	CL	11	24	9	98	54	12	
28.5								SANDY CLAY-SANDY SILT, Brown, moist, stiff, subrounded gravel to 1".
30.0	CL-ML	10	23	7	98	52	12	

TH 79-

EL 200±

Depth		MC	LL	PI	-4	-200	N	Description
	SC	5	23	6	76	26	R	GRAVELLY … cobbles …
4.5	SM	16		NP	100	33	R	SILTY SA… to 9"
9.0	SC	10	22	5	72	27	R	GRAVELLY … and boul…
13.5	SM	6		NP	82	17	R	SILTY GR… to 10"
16.5	SM	4		NP	52	10	R	SANDY GR… damp, …
19.5 / 20.0	SM	2		NP	13	3		SANDY GR… boulders

TH 79-2

EL 120±

Depth		MC	LL	PI	-4	-200	N	Description
	SP-SM	2		NP	72	11	R	GRAVELLY SAND-SILTY GRAVELLY SAND, Tan, dry, subrounded gravel to 2", no cohesion.
3.0	SW-SM	1		NP	77	11	R	GRAVELLY SAND-SILTY GRAVELLY SAND, Light brown, dry to damp, subrounded gravel to 2", no cohesion.
6.0	SC	22	34	17	96	49	16	CLAYEY SAND, Dark brown, wet, medium dense, subrounded pea gravel, cohesion.
10.5	CL	18	35	18	99	75	14	SANDY CLAY- Darkbrown, moist, stiff, medium dense, very little gravel.
							14	
15.0	SM	18		NP	100	50	17	SILTY SAND, Brown, moist, stiff, some subrounded gravel to 1", some cohesion.
18.5		14		NP	100	63	18	SANDY SILT, Brown, moist, stiff, some gravel to 3". (May be caving).
	ML	21		NP	100	67	5	Brown, moist to wet, medium consistency, large chunks.
25.5		24		NP	99	22	10	SILTY SAND, Brown,wet, medium dense.
30.0	SM	24		NP	95	16	25	

TH 79-4

EL 160±

Depth		MC	LL	PI	-4	-200	N	Description
	SC	4	24	7	77	28	R	GRAVELLY CLAYEY SAND, Brown, damp, cobbles to 4", some cohesion.
4.5	SP-SM	2		NP	75	8	R	GRAVELLY SAND-SILTY GRAVELLY SAND, Brown, damp, cobbles to 6", no cohesion.
7.5	SM	3		NP	85	25	R	GRAVELLY SILTY SAND, Light brown, damp, loose, cobbles to 5", some cohesion.
							5	
12.0		13	26	5	100	65	10	SANDY CLAY-SANDY SILT Light brown, moist, stiff, cohesion.
	CL-ML	14	23	5	99	56		Brown, moist, gravel to 1 1/2", cohesion.
21.0	SM	8	22	7	74	26		GRAVELLY SILTY SAND, Brown, damp, cobbles to 7", no cohesion.
24.0		5		NP	66	9		GRAVELLY SAND-SILTY GRAVELLY SAND, Brown, damp, cobbles to 6", no cohesion.
28.5	SP-SM	5		NP	51	8		Brown, damp, cobbles to 7", no cohesion, caving started at 29'-6".

EL 235±

4.5

9.0

13.5

15.0

VERT SCAL

VALUE ENGINEERING PAYS

TH 79-3

SILTY GRAVELLY SAND, Tan, dry, pea gravel to 1", medium dense.

SILTY SANDY GRAVEL, Tan, dry, gravel to 3".

GRAVELLY SAND-SILTY GRAVELLY SAND, Brown, damp, medium dense, gravel to 1".

SILTY SAND, Brown moist, medium dense, pea gravel.

GRAVELLY SILTY SAND, Brown, moist, medium dense, subrounded gravel to 1".

SANDY CLAY-SANDY SILT, Brown, moist, stiff, subrounded gravel to 1".

SANDY CLAY, Brown, moist, stiff, subrounded gravel to 1".

SANDY CLAY-SANDY SILT, Brown, moist, stiff, subrounded gravel to 1".

TH 79-5

EL. 200±

Depth	Class	MC	LL	PI	-4	-200
	SC	5	23	6	76	26
4.5	SM	16		NP	100	33
9.0	SC	10	22	5	72	27
13.5	SM	6		NP	62	17
16.5	GW-GM	4		NP	52	10
19.5	GM	2		NP	13	3
21.0						

GRAVELLY CLAYEY SAND, Brown, damp to dry, cobbles and boulders to 12".

SILTY SAND, Gray brown, moist, cobbles to 9".

GRAVELLY CLAYEY SAND, Brown, damp cobbles and boulders to 13", some cohesion.

SILTY GRAVELLY SAND, Brown, damp cobbles to 10".

SANDY GRAVEL-SILTY SANDY GRAVEL, Brown, damp, cobbles to 9".

SANDY GRAVEL, Brown, damp, cobbles and boulders to 14".

TH 79-7

EL. 255±

Depth	Class	MC	LL	PI	-4	-200
	SM	5		NP	62	21
4.5		5		NP	49	10
	GW-GM	6	23	7	42	7
13.5	SW-SM	—		NP	57	5
18.0						

SILTY GRAVELLY SAND, Brown, dry to damp, cobbles to 8", some cohesion.

SANDY GRAVEL-SILTY SANDY GRAVEL, Dark brown, damp cobbles and boulders to 12", some cohesion.

Dark brown, damp, cobbles and boulders to 17", some cohesion.

GRAVELLY SAND-SILTY GRAVELLY SAND, Brown, moist to wet, cobbles to 5".

caving at 18'-0".

TH 79-4

Class	MC	LL	PI	-4	-200
SC	4	24	7	77	28
SW-SM	2		NP	75	8
SM	3		NP	85	25
	13	26	5	100	65
CL-ML	14	23	5	99	56
SM	8	22	7	74	14
	5		NP	66	9
SW-SM	3		NP	51	8

GRAVELLY CLAYEY SAND, Brown, damp, cobbles to 4", some cohesion.

GRAVELLY SAND-SILTY GRAVELLY SAND, Brown, damp, cobbles to 6", no cohesion.

GRAVELLY SILTY SAND, Light brown, damp, loose, cobbles to 9", some cohesion.

SANDY CLAY-SANDY SILT Light brown, moist, stiff, cohesion.

Brown, moist, gravel to 1 1/2", cohesion.

GRAVELLY SILTY SAND, Brown, damp, cobbles to 7", no cohesion.

GRAVELLY SAND-SILTY GRAVELLY SAND, Brown, damp, cobbles to 6", no cohesion.

Brown, damp, cobbles to 7", no cohesion, caving started at 29'-6".

TH 79-6

EL. 235±

Depth	Class	MC	LL	PI	-4	-200
	SP	2		NP	60	4
4.5	SM	2		NP	69	14
9.0	GP-GM	3		NP	44	6
13.5	SW-SM	4		NP	50	7
15.0						

GRAVELLY SAND, Tan, dry, cobbles and boulders to 14".

SILTY GRAVELLY SAND, Brown, dry to damp, cobbles and boulders to 12", some cohesion.

SANDY GRAVEL-SILTY SANDY GRAVEL, Brown, damp, cobbles and boulders to 12".

GRAVELLY SAND-SILTY GRAVELLY SAND, brown, damp, cobbles and boulders to 14", bit nested cobbles at 15'-0".

NOTES:

1. See Plate D-10 for Legend and Soil Classification.
2. Test Holes were drilled in October 1979 using a bucket auger.

VERT SCALE 0 5 10 15 20 FEET

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTIAGO CREEK
FOUNDATION INVESTIGATION
TEST HOLES TH 79-1 TO TH 79-7

DESIGNED BY:
DRAWN BY: YBA MLB
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET

PLATE D-7

SAFETY PAYS

2

PLAN

SCALE 500 0 500 1000 1500 FEET

TL 79-8

EL 310.0±	LOG	MC	LL	PI	-4	-200	N	DESCRIPTION
	GM	–		NP	49	15		SILTY SANDY GRAVEL, Tan, rounded cobbles to 8" dense
10.0	GP	–		NP	43	2		SANDY GRAVEL, Tan, cobbles to 4", gravel to 1", medium-dense
20.0								

TL 79-10

EL 320.0±	LOG	MC	LL	PI	-4	-200	N	DESCRIPTION
	GP-GM	-		NP	55	12		SANDY GRAVEL-SILTY SANDY GRAVEL, Gravel to 3", medium dense
10.0	GP-GM	-		NP	36	7		SANDY GRAVEL-SILTY SANDY GRAVEL, Tan, dense cobbles to 6", gravel to 3"
40.0								

TL 79-12

EL 335.0±	LOG	MC	LL	PI	-4	-200	N	DESCRIPTION
	CL	-	26	8	99	62		SANDY CLAY, Red brown, stiff, some gravel to 1"
5.0	GP-GM	-		NP	49	8		SANDY GRAVEL-SILTY SANDY GRAVEL, Tan, cobbles to 9", gravel to 3"
25.0								

NOTES:
1. See Plate D0 for legend and Soil Classification.
2. Test logs sampled from pit walls in October, 1979.
3. See plate D9 for Test Boring logs by others.

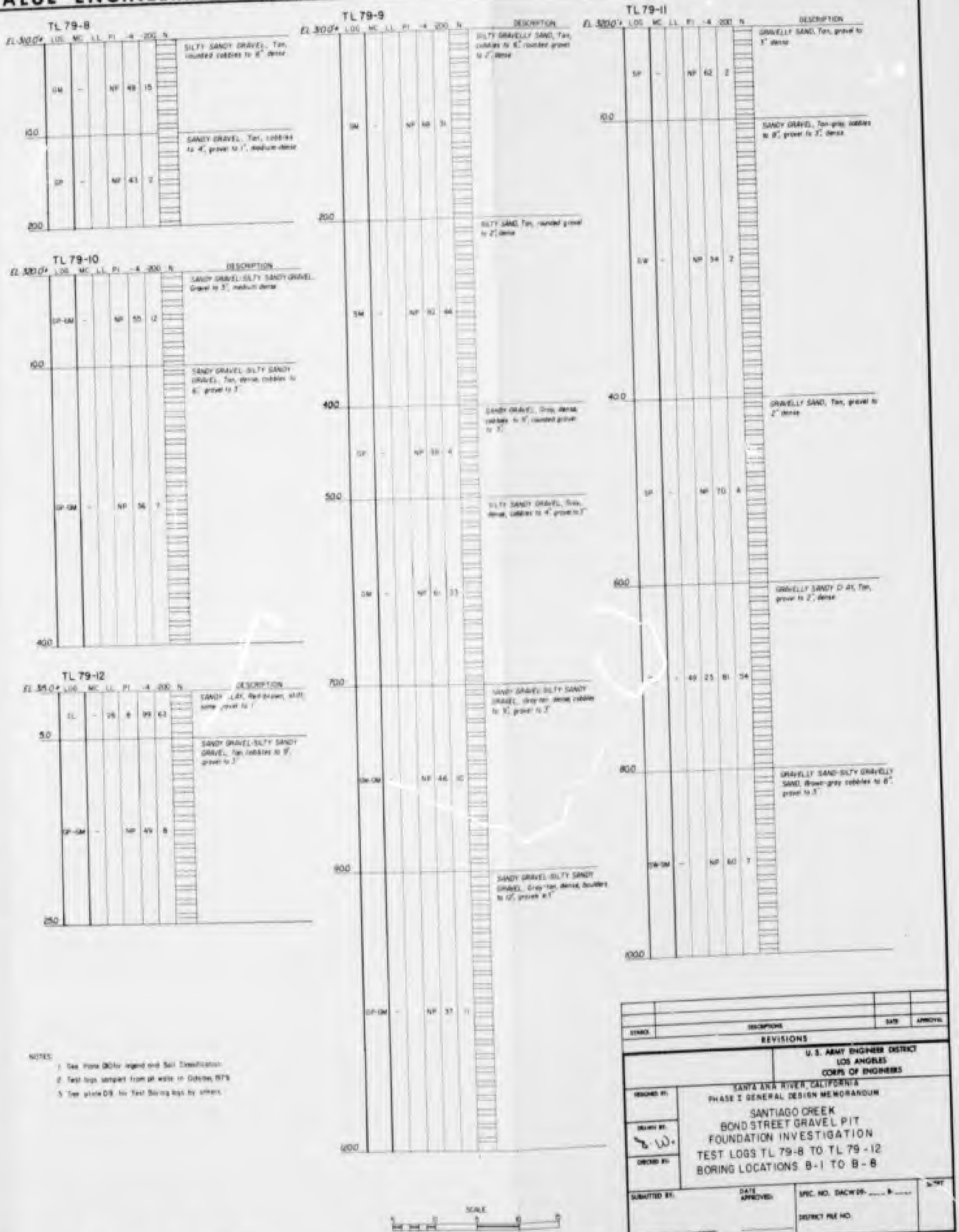

TL 79-8
TL 79-9
TL 79-10
TL 79-11
TL 79-12
LOG
MC
LL
PI
-4
200
N
DESCRIPTION
NOTES
SCALE
REVISIONS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTIAGO CREEK
BOND STREET GRAVEL PIT
FOUNDATION INVESTIGATION
TEST LOGS TL 79-8 TO TL 79-12
BORING LOCATIONS B-1 TO B-8

PLATE D-8

2

B-1

Lt. Brn. Silty Very Fine Sand-Loose-Dry
5' Lt. Brn. Silty Very Fine Sand - Loose-Slightly Damp
10' Lt. Brn. Silty Very Fine Sand - Loose-Damp (with tr. clay)
14'
15' Lt. Brn. Clayey, Silty Very Fine Sand - Loose-Damp
20' Lt. Brn. Clayey, Silty Very Fine Sand - Loose-Wet
22½' Med. Brn., Very, Very Fine Sandy, Silty Clay - Loose Very Wet
25' Med. Brn. Very, Very Fine Sandy, Silty Clay - Loose Very Wet
27½' Med. Brn. Very Fine Sandy Silty Clay - Loose-Very Wet
30' Med. Reddish Brn., Very Fine Sandy, Silty Clay - Loose Very Wet
35' Med. Brn., Very Fine Sand, Clayey Silt Loose - Extremely Wet

B-2

Gravelly Sand up to 9" Dia.-(up to 9" Dia. Cobbles Encountered) Firm - Slightly Damp
5'
10' Brn., Silty Sand & Gravel with Cobbles up to 9" Dia. Firm-Damp
15'
18' Brn. Silty Sand & Gravel - Firm-Wet
20' Brn. Silty Sand & Gravel - Firm-Very Wet
25'
27' Reddish Brn. Silty, Clayey Sand & Gravel - Firm Extremely Wet
28' Bottom of boring, large boulders on top of clay & water

B-3

Reddish Brn. Silty Sand & Gravel - Firm - Slightly Moist
5'
10'
15' Bottom of Hole

B-6

Reddish Brn. Silty Sand & Gravel with Cobbles - Firm - Slightly Moist
5'
10'
15' Reddish Brn. Clayey, Silty, Sand & Gravel with Cobbles - Firm-Wet
20' Reddish Brn., Clayey Silty Sand & Gravel with Cobbles - Firm-Very Wet
25' Bottom of Hole Large Cobbles also Perched Water

B-7

Reddish Brn. Silty Sand, Gravel - Cobbles Firm-Dry
5' Bottom of Hole Cobbles too big - tried 5 locations in Area - Could not go deeper than 5 feet

B-8

Med. Brn., Clayey, Very Fine Silty Sand - Firm - Moist
5'
10'
15'
20'
25'
30' Med. Brn. Clayey Fine Sand Silt (with gravel) Firm - Moist
Med. Brn. Clayey, Very Fine Silty Sand - Firm - Moist
35'
35' - 50'
Med. Brn., Clayey Fine Sandy Silt (with gravel) Firm - Moist
50'

B-3

Reddish Brn.
Silty Sand & Gravel - Firm - Slightly Moist.

5'

10'

15' Bottom of Hole

B-4

Brn. gravel & Cobbles up to 1" in Dia. (Small amount of Silty Sand) - Firm & Dry

5'

8' Bottom of Hole Boulders too large to get through

B-5

Reddish Brn. Silty Sand, Gravel & Cobbles Firm - Slightly Moist

5'

10' Bottom of Hole Boulders too large to Auger through

B-8

Med. Brn., Clayey, Very Fine Silty Sand - Firm - Moist

5'

10'

15'

20'

25'

30' Med. Brn. Clayey Fine Sand Silt (with gravel) Firm - Moist

Med. Brn. Clayey, Very Fine Silty Sand - Firm - Moist

35'

35' - 50'
Med. Brn., Clayey Fine Sandy Silt (with gravel) Firm - Moist

50'

NOTES:
1. See Plate D10 for Legend and Soil Classification.
2. Borings drilled in January 1976 by others.
3. See Plate D8 for Boring Locations.
4. These logs by others are presented only as a supplement to Corps logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

SANTA ANA RIVER, CALIFORNIA
PHASE I, GENERAL DESIGN MEMORANDUM
SANTIAGO CREEK
BOND STREET GRAVEL PIT
FOUNDATION INVESTIGATION BY OTHERS
BORINGS B-1 TO B-8

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- ___ B- ___
DISTRICT FILE NO.
SHEET

PLATE D-9

2

PLAN

SCALE 1" = 200' — 200 100 0 200 400 FEET

T.T. 79-8

EL. 5705		MC	LL	PI	-4	-200	
3.0'	GW-GM	3	-	NP	41	7	SANDY GRAVEL - SILTY SANDY GRAVEL: light brown damp to dry, cohesive, dense, gravel to 4"
	GW	4	-	NP	39	4	SANDY GRAVEL: brown, damp, medium dense to dense, gravel to 3"
9.0'		5	-	NP	43	2	SAME AS ABOVE
12.0'	GP	5	-	NP	28	3	SAME AS ABOVE, rock to 6"
15.0'	GW	5	-	NP	27	3	SAME AS ABOVE, rock to 3"

T.T. 79-1

EL. 5692		MC	LL	PI	-4	-200	
2.0'	GP	3	-	NP	44	4	SANDY GRAVEL: brown, damp, gravel to 2"
3.5'	SM	8	-	NP	81	25	GRAVELLY SILTY SAND: brown, damp, cohesive, gravel to 2"
6.0'	GP-GM	4	-	NP	42	6	SANDY GRAVEL-SILTY SANDY GRAVEL: brown, damp, gravel to 2"
9.0'	SM	12	-	NP	94	38	SILTY SAND: brown, damp, cohesive, gravel to 3/4"
12.0'	GP	5	-	NP	41	5	SANDY GRAVEL: brown, damp, moist, gravel to 2"
	GP-GM	4	-	NP	37	5	SANDY GRAVEL-SILTY SANDY GRAVEL: brown, damp to moist, gravel to 6"
17.0'		5	-	NP	41	11	

T.T. 79-2

EL. 6271		MC	LL	PI	-4	-200	
2.5'	SM	3	-	NP	88	27	SILTY SAND: light brown, dry to damp, cohesive, gravel to 3/4"
3.0'	SW	1	-	NP	86	1	SAND: light brown, damp, cohesive gravel to 3/4"
8.0'	SP-SM	3	-	NP	71	6	GRAVELLY SAND-SILTY GRAVELLY SAND: brown, moist gravel to 3/4"
10.0'	SP	11	-	NP	65	2	GRAVELLY SAND: brown, moist to wet, gravel to 1 1/2"

EL. 6652		MC	LL	P[illegible]
3.0'	SM	11	-	[illegible]
6.0'	SM	7	-	[illegible]
9.0'	SM	12	-	[illegible]
12.0'	SM	6	-	[illegible]
15.0'	SM	10	-	[illegible]

NOTES:
1. SEE PLATES [illegible] FOR LOGS OF TEST BORINGS
2. TEST TRENCHES WERE EXCAVATED WITH [illegible]
IN NOVEMBER 1979.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS	GRAVELS	Clean gravels	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS	Clean sands	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly-graded sands, gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS	Low liquid limit	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
		High liquid limit	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
Highly organic soils			Pt	Peat and other highly organic soils.

NOTES:

1. Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.
2. All sieve sizes on this chart are U. S. Standard.
3. The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus no. 200 sieve material is silt if the liquid limit and plasticity index plot below the "A" line on the plasticity chart (Table VI, Military Standard 619B), and is clay if the liquid limit and plasticity index plot above the "A" line on the chart.
4. For a complete description of the Unified Soil Classification System, see "Military Standard 619B" dated 20 March 1976

LEGEND

T.H.	LOCATION AND NUMBER OF TEST HOLE.
T.B.	LOCATION AND NUMBER OF TEST BORING BY OTHERS.
T.T.	LOCATION AND NUMBER OF TEST TRENCH
MC	FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT.
LL	LIQUID LIMIT.
PI	PLASTICITY INDEX (LIQUID LIMIT MINUS PLASTIC LIMIT).
NP	NONPLASTIC
-4	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 4 SIEVE.
-200	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 200 SIEVE.
N	NUMBER OF BLOWS OF A 140-POUND DROPHAMMER FALLING 30 INCHES REQUIRED TO DRIVE A SAMPLING SPOON ONE FOOT. OUTSIDE DIAMETER OF SPOON IS 2 INCHES. INSIDE DIAMETER IS 1-3/8 INCHES. PROCEDURE IS CALLED STANDARD PENETRATION TEST.
W	DEPTH TO WATER
T.B.	LOCATION AND NUMBER OF TEST BORING BY OTHERS.

T.T. 79-2

EL 627±

		MC	LL	PI	-4	-200	
2.5'	SM	3	-	NP	98	27	SILTY SAND light brown, dry to damp, cohesive, gravel to 3/4"
5.0'	SW	1	-	NP	86	1	SAND light brown, damp, cohesive, gravel to 3/4"
8.0'	SP-SM	3	-	NP	71	6	GRAVELLY SAND-SILTY GRAVELLY SAND brown, moist, gravel to 3/4".
10.0'	SP	11	-	NP	65	2	GRAVELLY SAND brown, moist to wet, gravel to 1 1/2".

T.T. 79-3

EL 665±

		MC	LL	PI	-4	-200	
3.0'	SM	11	-	NP	88	47	SILTY SAND brown, damp, cohesive, gravel to 1".
6.0'	GM	7	-	NP	54	18	SILTY SANDY GRAVEL same as above, damp to moist, gravel to 2"
9.0'	SM	12	-	NP	89	44	SILTY SAND same as above, gravel to 1"
12.0'	GM	8	-	NP	56	16	SILTY SANDY GRAVEL same as above, gravel to 2"
15.0'	SM	12	-	NP	64	18	SILTY SAND same as above, gravel to 2"

NOTES:
1. SEE PLATE D-9 FOR LOGS OF TEST BORINGS BY OTHERS
2. TEST TRENCHES WERE EXCAVATED WITH A BACKHOE IN NOVEMBER 1979.

VERT SCALE FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

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

DESIGNED BY:
DRAWN BY: JY
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
OAK STREET DRAIN
FOUNDATION INVESTIGATION
TEST TRENCHES TT 79-1, 2, 3, 8
TEST BORING LOCATIONS TB 78-1 TO TB 78-5

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

PLATE D-10

2

T.T. 79-4

EL 760±		MC	LL	PI	-4	-200	
2.0'	GM	6	–	NP	49	15	SILTY SANDY GRAVEL brown, damp to dry, gravel to 2"
5.0'	SM	9	–	NP	79	32	SILTY GRAVELLY SAND brown, damp, cohesive gravel to 3/4"
8.0'	GW	6	–	NP	30	4	SANDY GRAVEL brown, damp to moist, gravel to 3"
9.0'	SM	19	–	NP	89	37	SILTY SAND brown, moist, cohesive, gravel to 1"
12.0'	GW	5	–	NP	27	3	SANDY GRAVEL brown, moist, cobbles to 6" max well graded
15.0'	GP	5	–	NP	29	3	Same as above, poorly graded, 75% boulders, cobbles, and gravel

T.T. 79-5

EL 794±		MC	LL	PI	-4	-200	
3.0'	SC SM	6	24	6	58	23	CLAYEY-SILTY SAND light brown, damp, cohesive, med dense, gravel to 2"
6.0'	SM	9	–	NP	77	29	GRAVELLY SILTY SAND brown, damp, a little cohesion, med dense, gravel to 2"
10.5'	SW SM	5	–	NP	58	11	SAND-SILTY SAND brown, damp, little cohesion, med dense, gravel to 2"

Below this depth the side wall of the excavation caved in

T.T. 79-6

EL [illegible]		MC	LL	PI	-4	-200	
3.0'	SC	7	27	11	87	35	CLAYEY SAND light brown, damp, cohesive, dense to med dense, gravel to 1"
6.0'	SP SM	6		NP	65	12	GRAVELLY SAND-SILTY GRAVELLY SAND brown, damp to moist, very little cohesion, med dense, gravel to 2"
9.0'	CL ML	12	22	7	98	65	SANDY CLAY-SANDY SILT dark brown, damp to moist, cohesive, dense to med dense gravel to 1"
12.0'	SC SM	11	23	6	97	48	CLAYEY SILTY SAND dark brown, moist, cohesive, dense to med dense, gravel to 1"
15.0'	SC	12	32	14	88	41	CLAYEY SAND dark brown, moist, cohesive, dense to med dense, gravel to 1"

EL 929±		MC
3.0'	GM	7
6.0'	SM	10
10.0'	SM SC	8
13.0'	GP GM	6
15.0'	GW GM	7

VALUE ENGINEERING PAYS

NOTE:
1. SEE PLATE D0 FOR LEGEND AND SOIL CLASSIFICATION
2. TEST TRENCHES EXCAVATED WITH A BACKHOE IN NOVEMBER 1979.

T.T. 79-6

EL.822±		MC	LL	PI	-4	-200	
3.0'	SC	7	27	11	87	35	CLAYEY SAND: light brown, damp, cohesive, dense to med. dense, gravel to 1"
6.0'	SP-SM	6		NP	65	12	GRAVELLY SAND-SILTY GRAVELLY SAND: brown, damp to moist, very little cohesion, med. dense, gravel to 3"
9.0'	CL-ML	12	22	7	98	65	SANDY CLAY-SANDY SILT: dark brown, damp to moist, cohesive, dense to med. dense, gravel to 1"
12.0'	SC-SM	11	23	6	97	48	CLAYEY SILTY SAND: dark brown, moist, cohesive, dense to med. dense, gravel to 1"
15.0'	SC	12	32	14	88	41	CLAYEY SAND: dark brown, moist, cohesive, dense to med. dense, gravel to 1"

T.T. 79-7

EL.929±		MC	LL	PI	-4	-200	
3.0'	GM	7	-	NP	49	17	SILTY SANDY GRAVEL: light brown, damp, cohesive, med. dense to dense, gravel to 8"
6.0'	SM	10	-	NP	84	46	GRAVELLY SILTY SAND: brown, damp, cohesive, med. dense to dense, gravel to 3/4"
10.0'	SM-SC	6	22	5	68	36	GRAVELLY SILTY SAND-GRAVELLY CLAYEY SAND: brown, damp to moist, cohesive, med. dense to dense, gravel to 2"
13.0'	GP-GM	6	-	NP	34	7	SANDY GRAVEL-SILTY SANDY GRAVEL: brown, moist, med. dense, gravel to 6"
15.0'	GW-GM	7		NP	42	6	same as above

VERT. SCALE FEET

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

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

DESIGNED BY:
DRAWN BY: 17
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
OAK STREET DRAIN
FOUNDATION INVESTIGATION
TEST TRENCHES TT 79-4 TO TT79-7

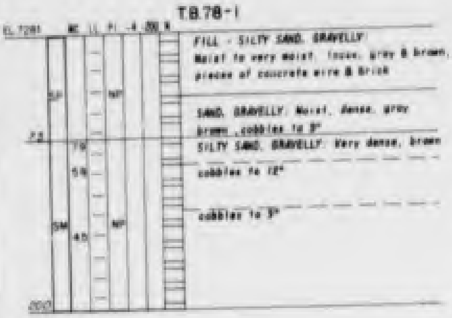
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW 09-___ B-___
SHEET
DISTRICT FILE NO.

PLATE D-11

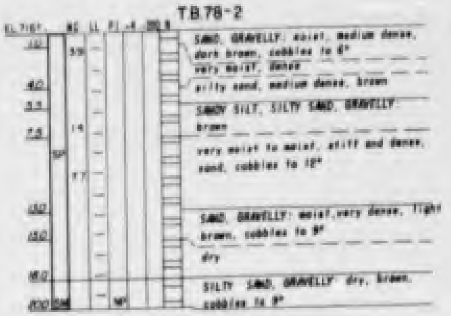
SAFETY PAYS

2

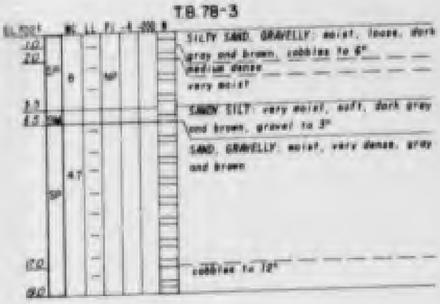
T.B.78-1

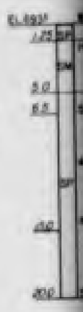


T.B.78-2

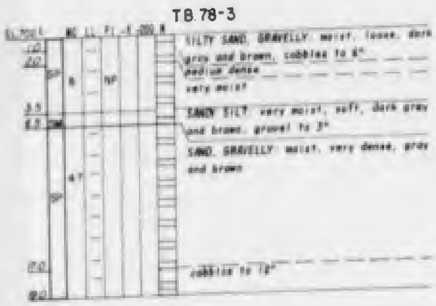


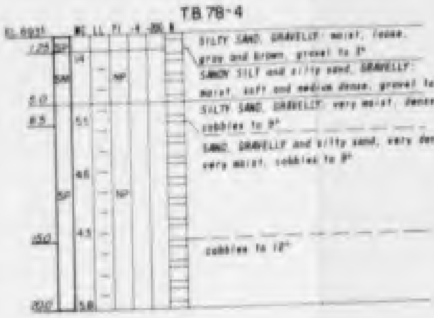
T.B.78-3

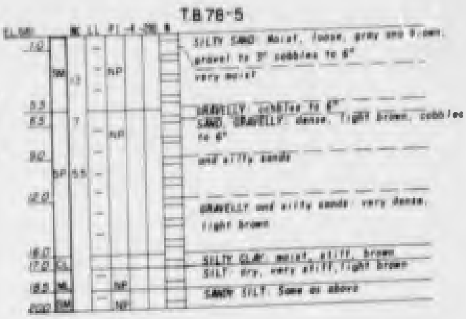




VERT. SCALE







NOTES.
1. See Plate D10 for Legend and Soil Classification.
2. Test Borings were drilled by others using a bucket auger in March 1978.
3. See Plate D10 for location of borings.

VERT. SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

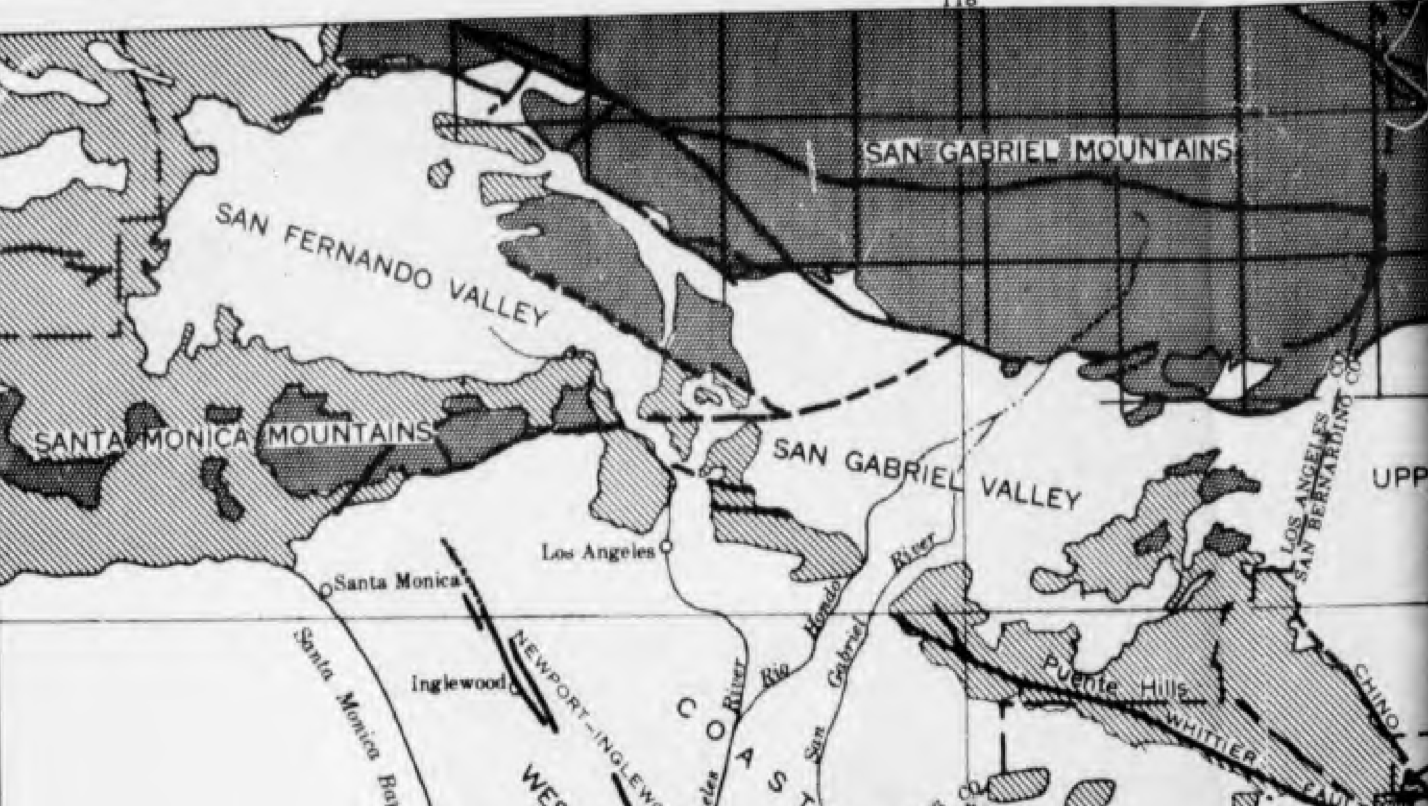
DESIGNED BY:
DRAWN BY: NLS
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
OAK STREET DRAIN
FOUNDATION INVESTIGATION BY OTHERS
TEST BORINGS TB 78-1 TO TB 78-5

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

PLATE D-12

2



Base map and fault pattern largely after Geologic Map of California (Jenkins, 1938)

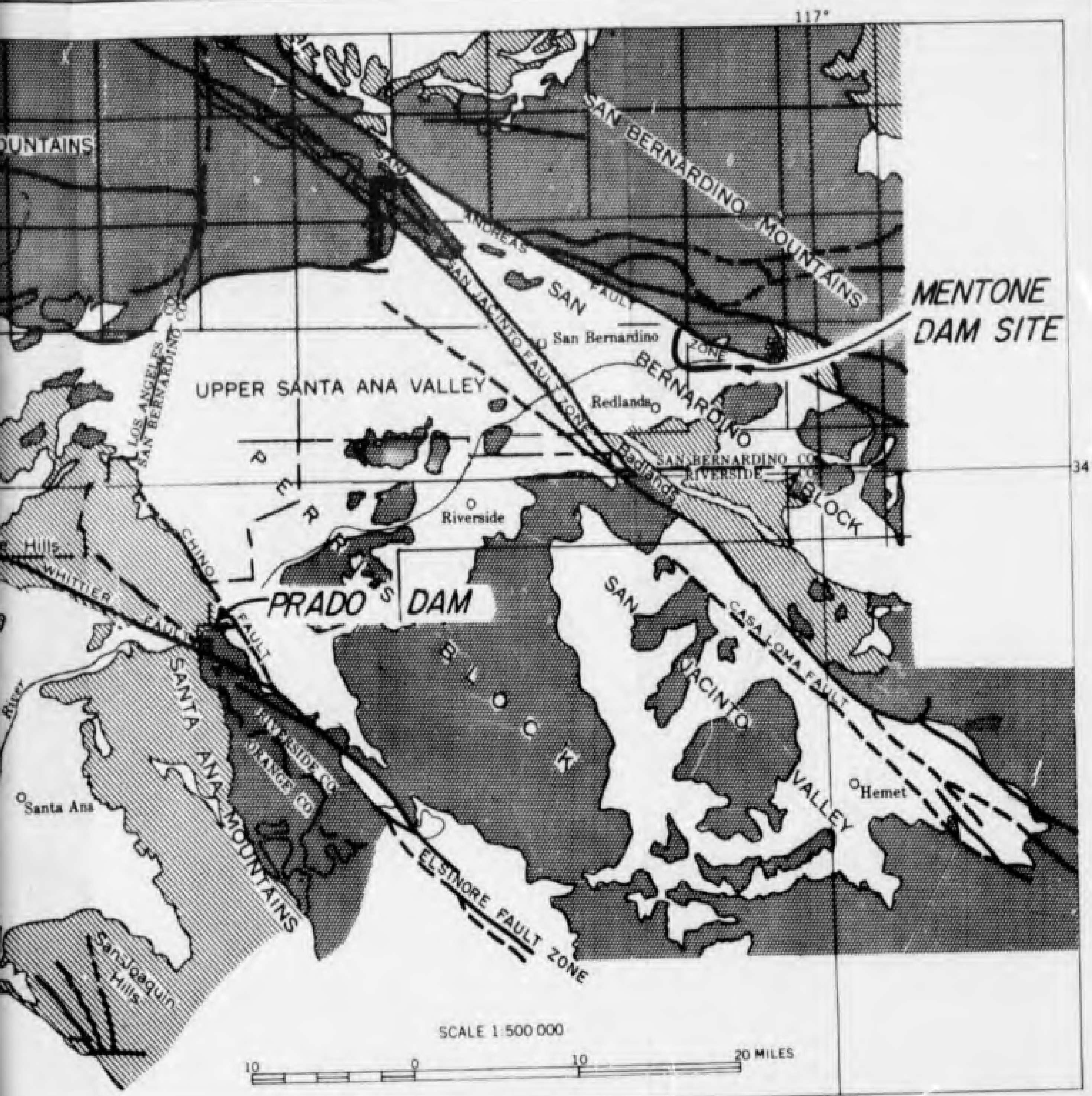
EXPLANATION

Alluvium and associated deposits of Recent or Pleistocene age

Crystalline and metamorphic rocks of Jurassic or greater age; some volcanic rocks of Tertiary age

Sedimentary ro[...] largely of T[...] part of Cre[...] age

CORPS OF ENGINEERS



EXPLANATION

...ic rocks ...e; some ...y age

Sedimentary rocks of marine origin; largely of Tertiary age but in part of Cretaceous or Triassic age

Contact

Fault
Dashed where inferred; dotted where concealed

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM
REGIONAL GEOLOGY

U. S. ARMY ENGINEER DISTRICT

TO ACCOMPANY REPORT DATED

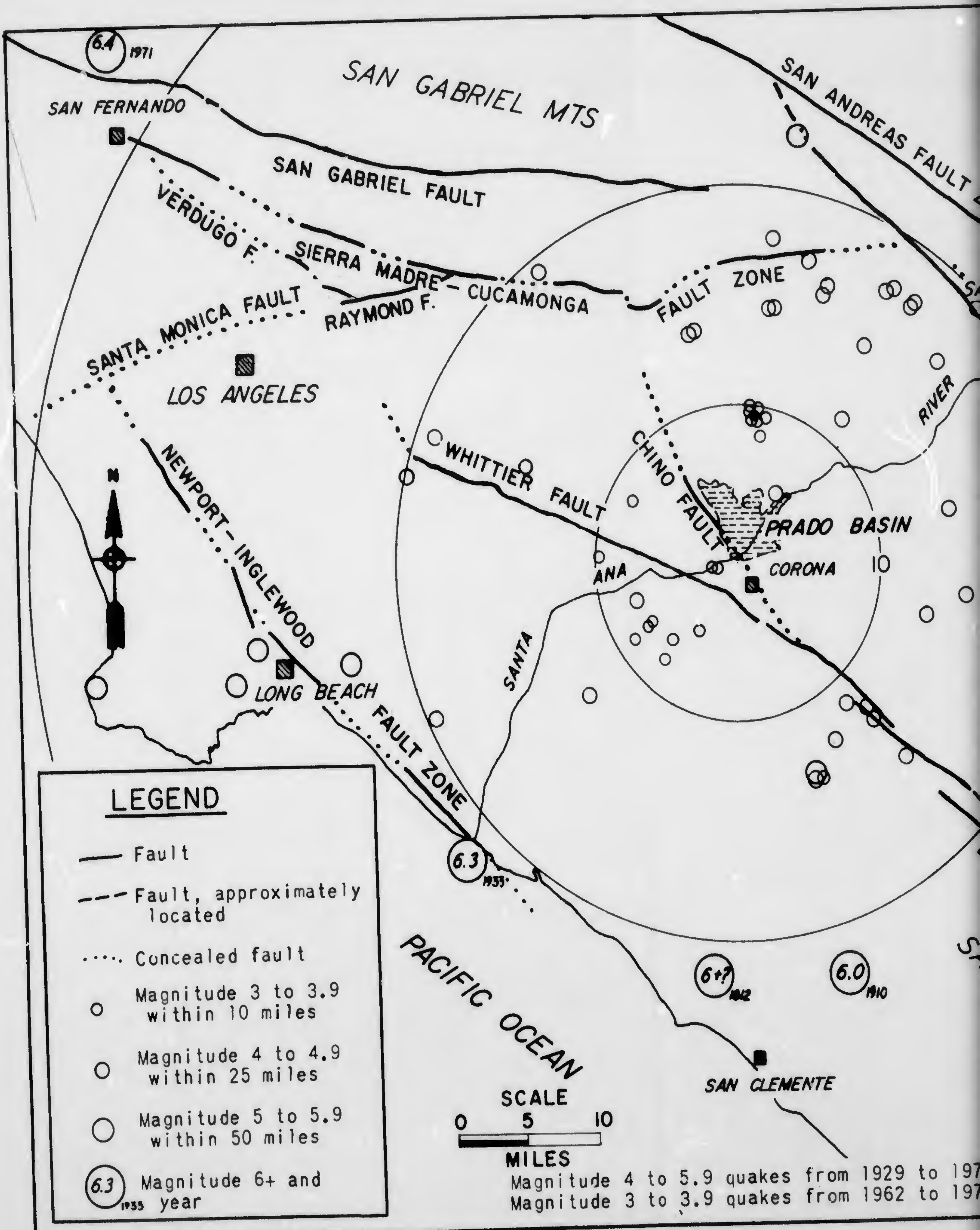
PLATE D-13

2

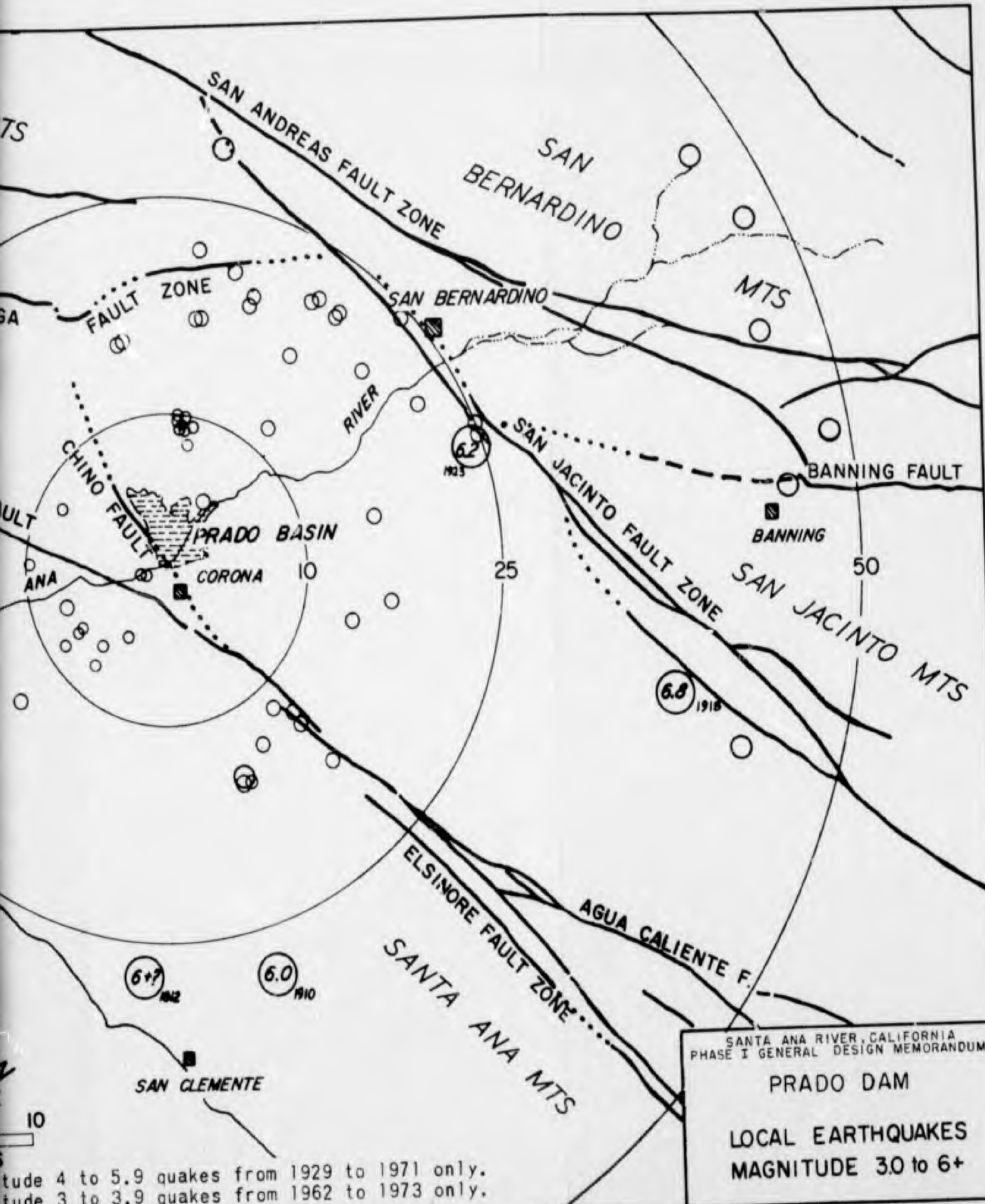

CHINO CREEK
SANTA ANA RIVER
CHINO FAULT
AXIS ARENA BLANCA SYNCLINE
PRADO DAM
ALISO CANYON FAULT
Qal
Qoal
Tpsc


SANTA ANA RIVER
CHINO FAULT
PRADO DAM
Qal
Qoal
Tpsc
Surface Refractive Line
LEGEND
Qal RECENT ALLUVIUM
Qoal OLDER ALLUVIUM
Tpsc PUENTE FORMATION - SYCAMORE CYN. MEMBER
APPROXIMATE CONTACT BETWEEN FORMATIONS
STRIKE AND DIP OF STRATA
FAULT, QUERIED WHERE LOCATION INFERRED
DIP OF FAULT
SCALE
400 200 0 400 800 1200 FEET
REVISIONS
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
PRADO DAM
LOCAL GEOLOGY
DWL
DWL
VFM

PLATE D-14



6.4 1971
SAN GABRIEL MTS
SAN ANDREAS FAULT
SAN FERNANDO
SAN GABRIEL FAULT
VERDUGO F.
SIERRA MADRE – CUCAMONGA
FAULT ZONE
SANTA MONICA FAULT
RAYMOND F.
LOS ANGELES
RIVER
CHINO FAULT
WHITTIER FAULT
PRADO BASIN
CORONA
10
ANA
SANTA
NEWPORT – INGLEWOOD
LONG BEACH
FAULT ZONE
N
6.3 1933
PACIFIC OCEAN
6+? 1812
6.0 1910
SAN CLEMENTE
SCALE
0 5 10
MILES
Magnitude 4 to 5.9 quakes from 1929 to 197
Magnitude 3 to 3.9 quakes from 1962 to 197
LEGEND
Fault
Fault, approximately located
Concealed fault
Magnitude 3 to 3.9 within 10 miles
Magnitude 4 to 4.9 within 25 miles
Magnitude 5 to 5.9 within 50 miles
6.3 1935 Magnitude 6+ and year



SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

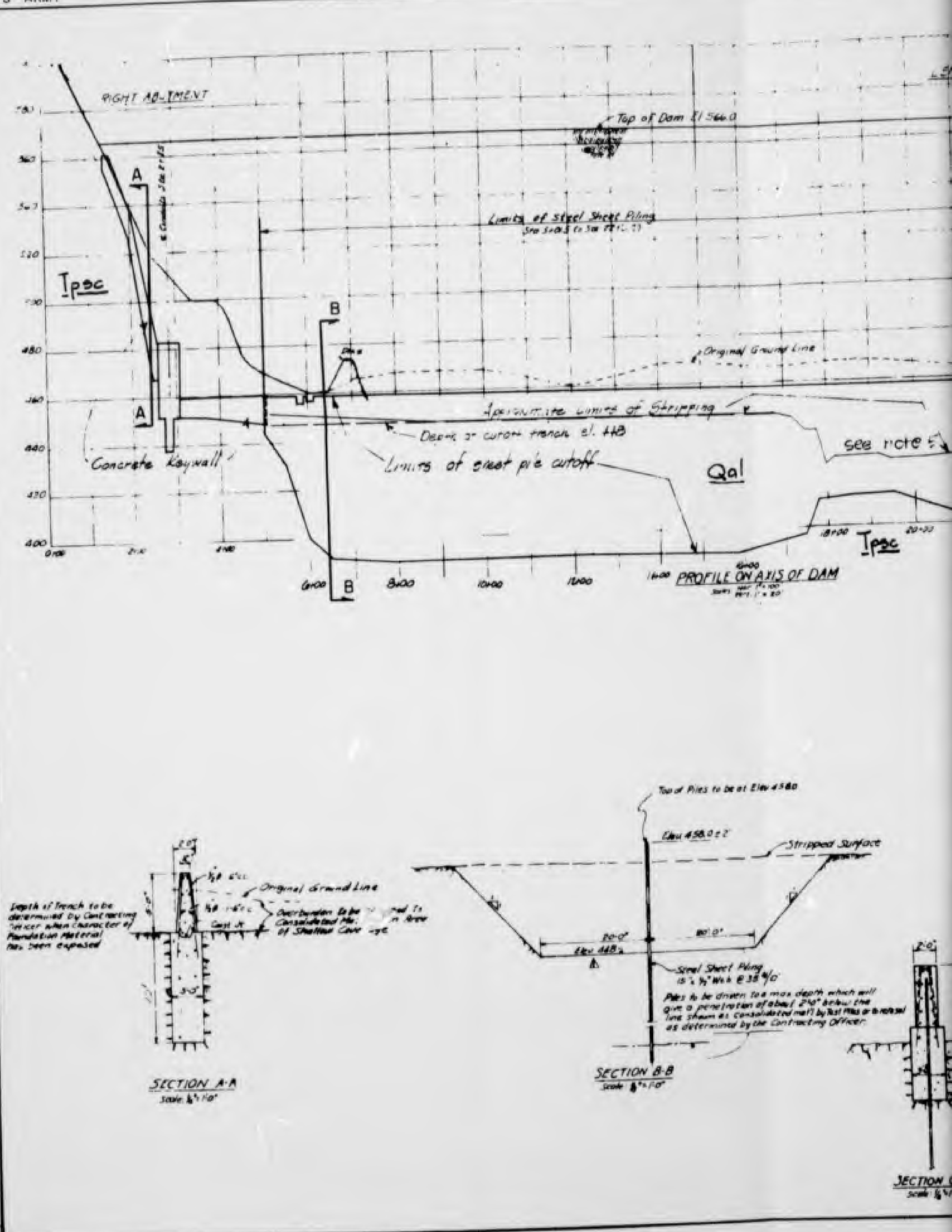
PRADO DAM

LOCAL EARTHQUAKES
MAGNITUDE 3.0 to 6+

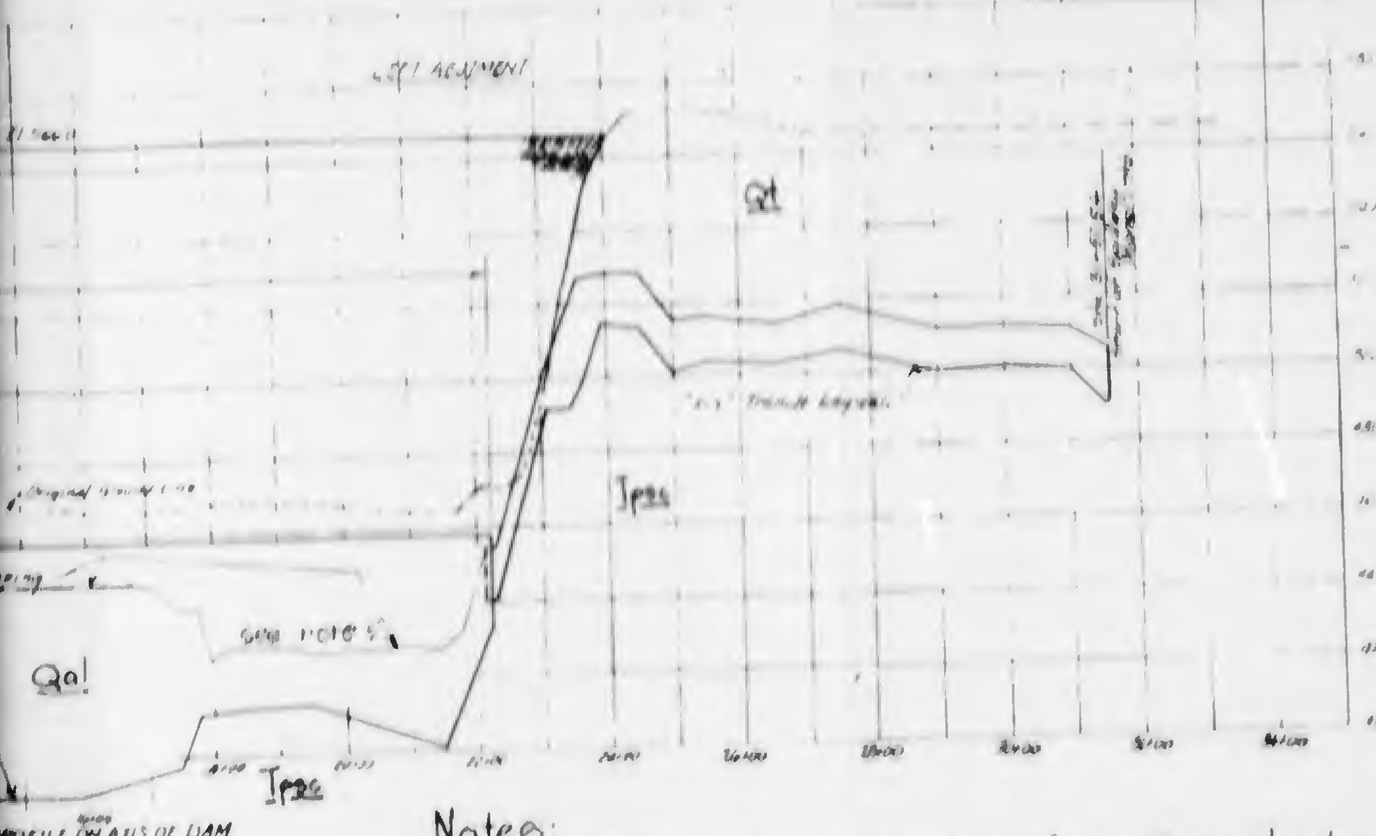
PLATE D-15

2

U S ARMY

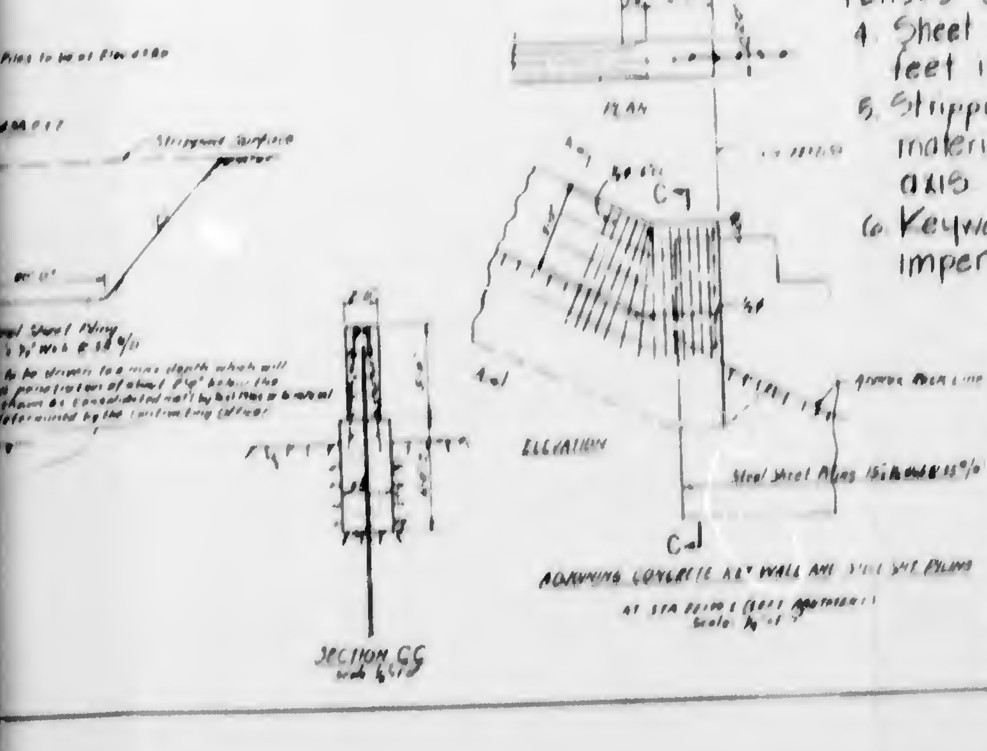


RIGHT ABUTMENT
Top of Dam El 564.0
Limits of Steel Sheet Piling
Tpsc
Original Ground Line
Approximate Limits of Stripping
Depth of cutoff trench el. 448
Concrete Keywall
Limits of sheet pile cutoff
Qal
see note 5
PROFILE ON AXIS OF DAM
Top of Piles to be at Elev 458.0
Elev 458.0 ± 2
Stripped Surface
Original Ground Line
Depth of Trench to be determined by Contracting Officer when character of Foundation Material has been exposed
Overburden to be removed to Consolidated Mat. in Area of Shallow Cave
20'-0"
Elev 448
Steel Sheet Piling
Piles to be driven to a max depth which will give a penetration of about 2'-0" below the line shown as Consolidated mat'l by Test Piles or to refusal as determined by the Contracting Officer
SECTION A-A
SECTION B-B
SECTION



Notes:

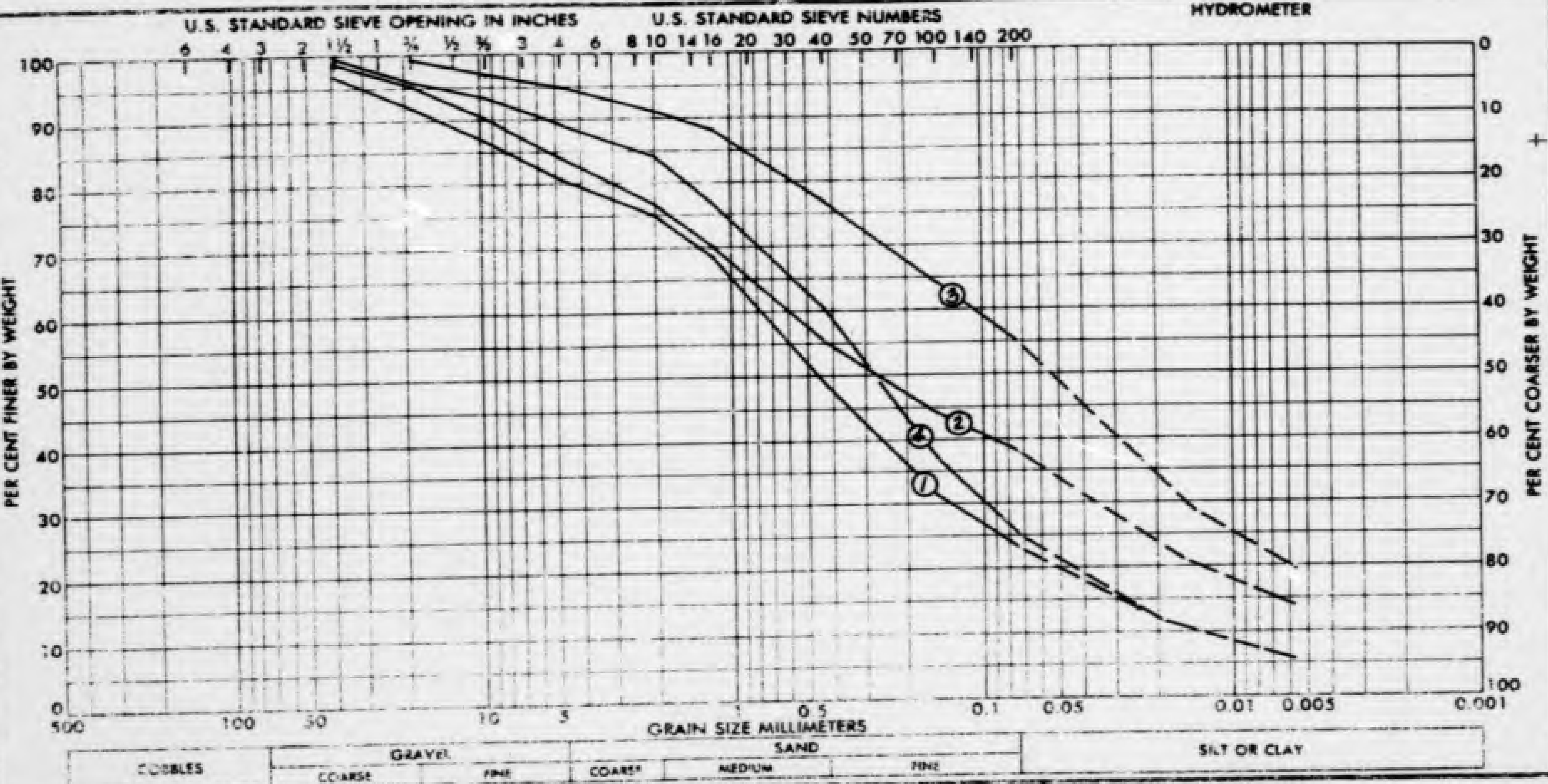
1. Qal = Recent alluvium consists of gravelly sand + clay lenses
e. Qt = Terrace alluvium, consists of loose to dense silts, sands, gravel, and cobbles.
3. Tpsc = Puente formation; Sycamore Canyon member (Lower Pliocene); consists of slight to moderate cemented sandstone w/ siltstone interbeds and scattered lenses of conglomerate.
4. Sheet piling was driven from 2 to 3 feet into bedrock.
5. Stripping line indicates depth of materials removed upstream of the axis.
6. Keywall trench backfilled w/ select impervious material.



SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM

AS CONSTRUCTED
FOUNDATION TREATMENT



ZONE	CLASSIFICATION	LL	PI
① U.S. Pervious	SM, Silty Sand		NP
② Random	SC, Clayey Sand	27	12
③ Core	CL, Sandy Clay	30	14
④ D.S. Pervious	SM, Silty Sand		NP

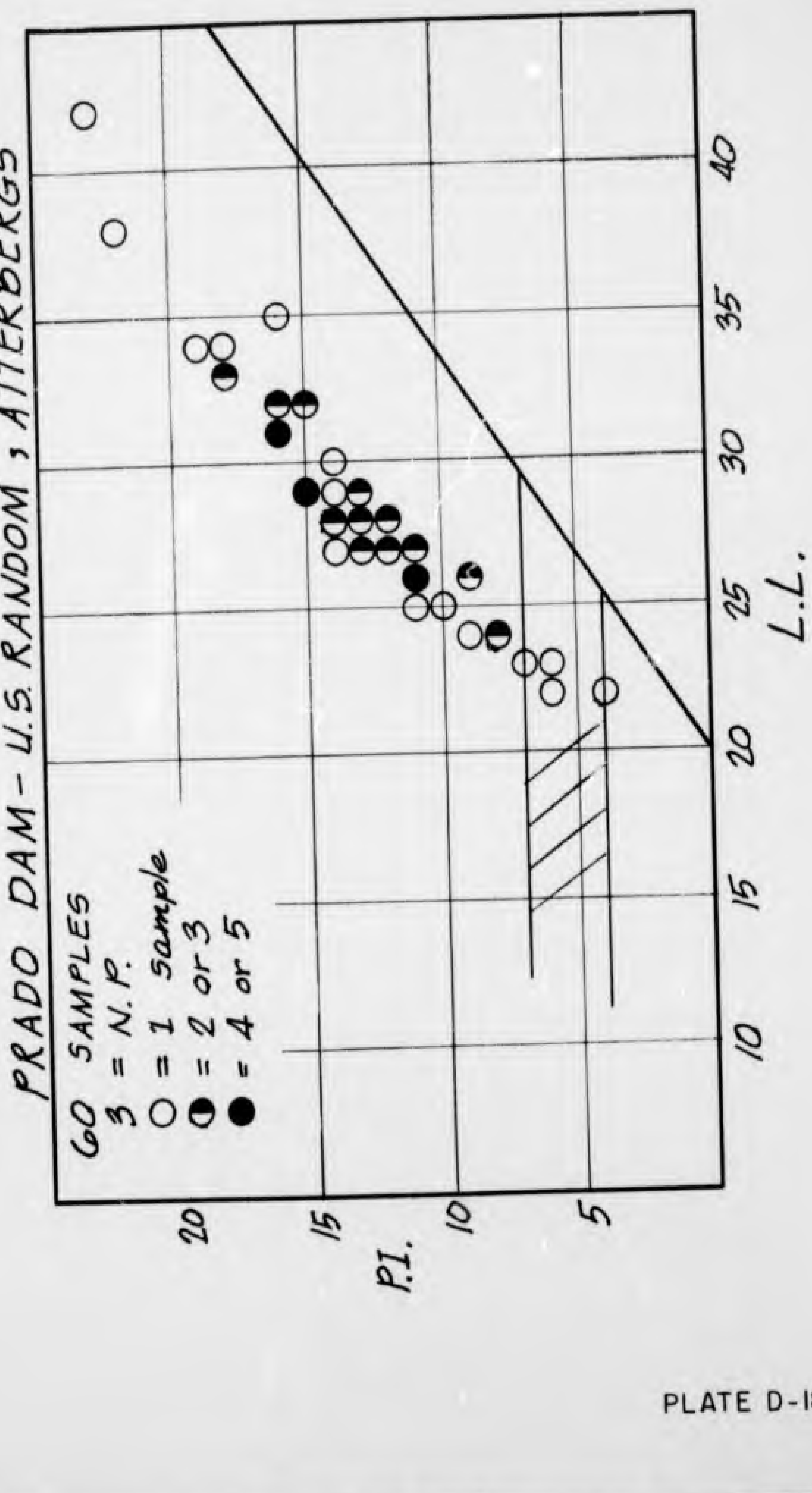
PROJECT PRADO DAM

Average gradations of embankment materials

DATE January 1974

GRADATION CURVES

ENG FORM 2087 1 MAY 63 — REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.


PRADO DAM - U.S. RANDOM, ATTERBERGS
60 SAMPLES
3 = N.P.
○ = 1 sample
◐ = 2 or 3
● = 4 or 5
P.I.
20
15
10
5
10
15
20
25
30
35
40
L.L.

PRADO DAM - CORE , ATTERBERGS

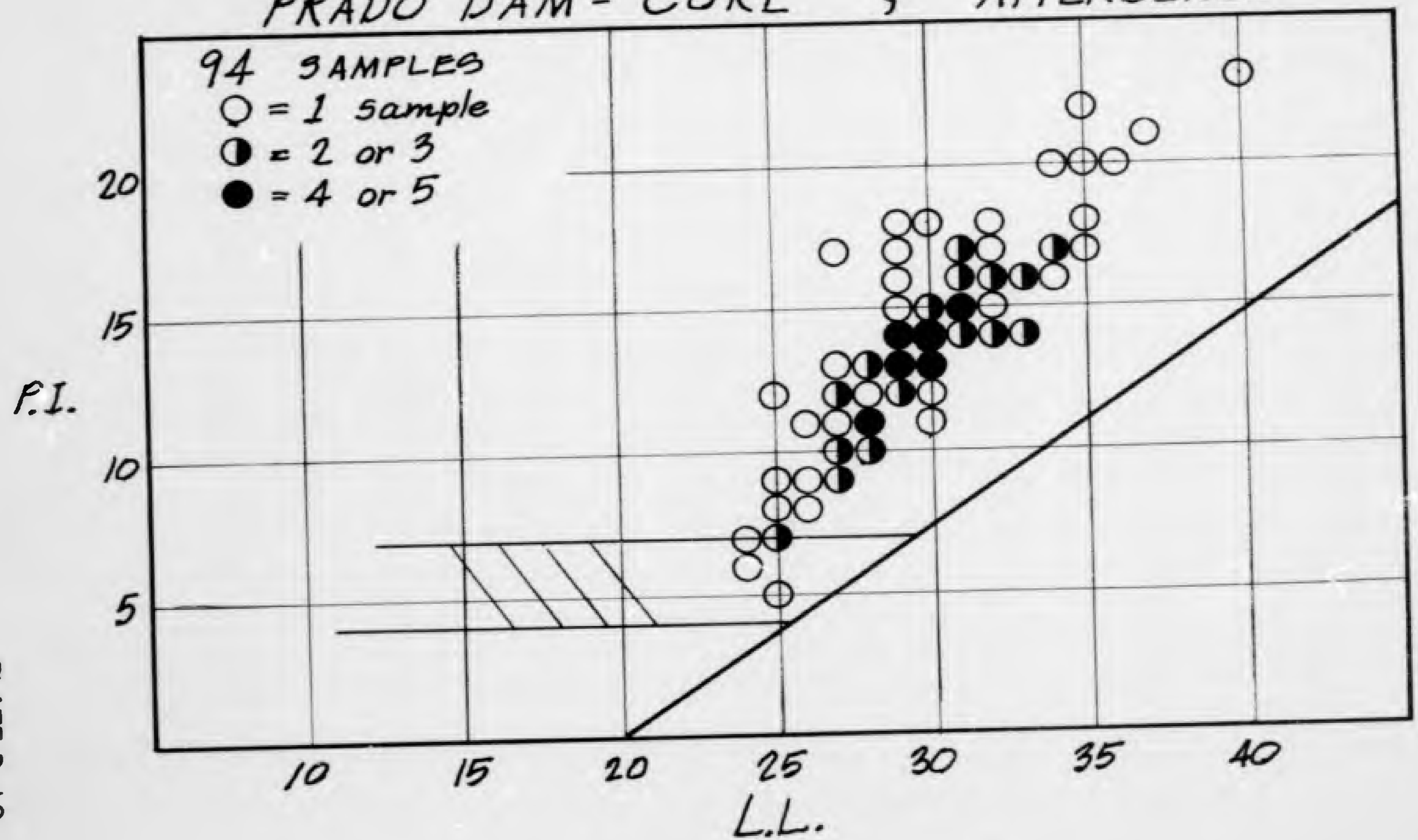


PLATE D-19.

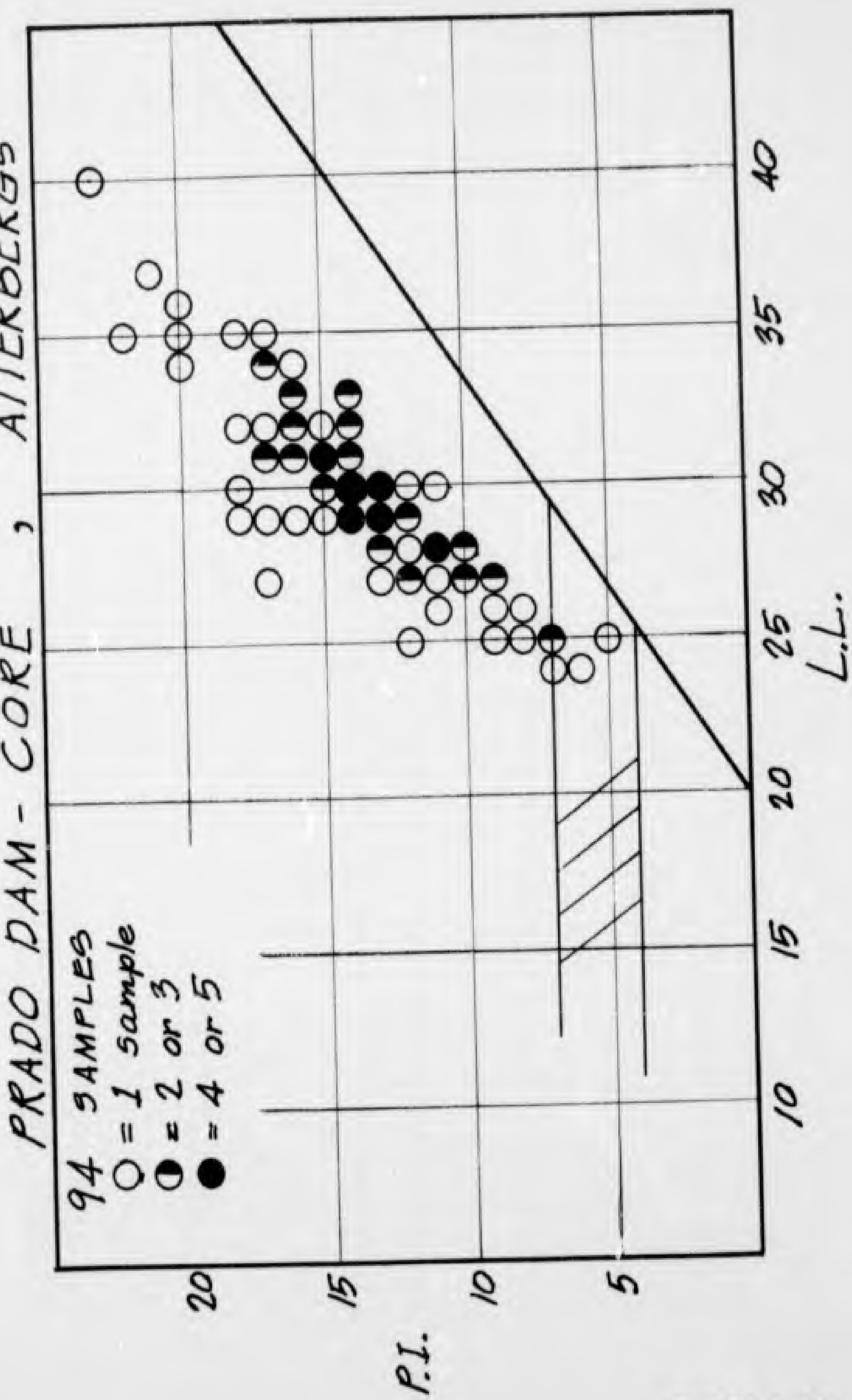
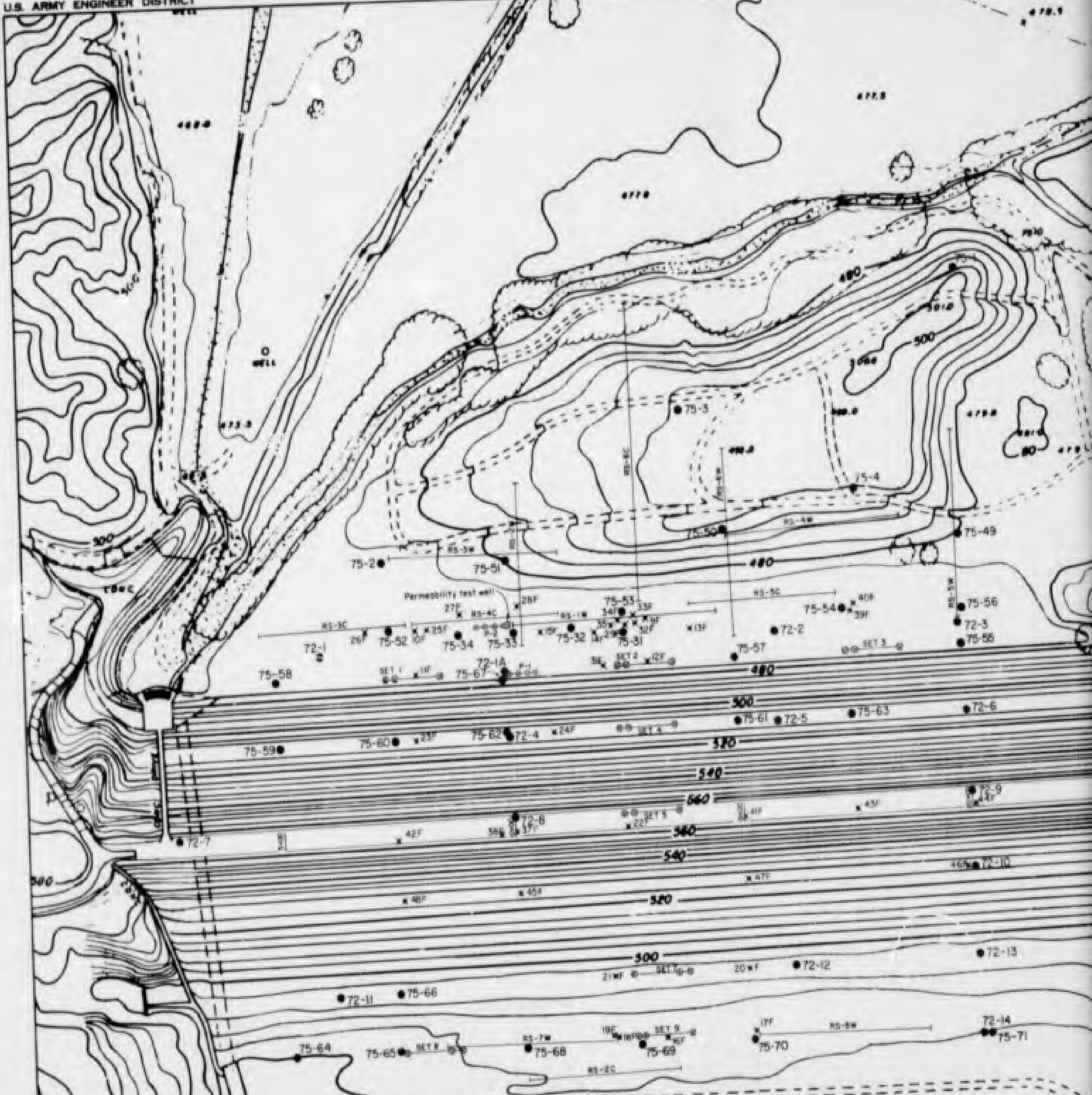

PRADO DAM - CORE, ATTERBERGS
94 SAMPLES
○ = 1 sample
◐ = 2 or 3
● = 4 or 5
P.I.
L.L.
20
15
10
5
10
15
20
25
30
35
40

PLATE D-19

U.S. ARMY ENGINEER DISTRICT



CORPS OF ENGINEERS

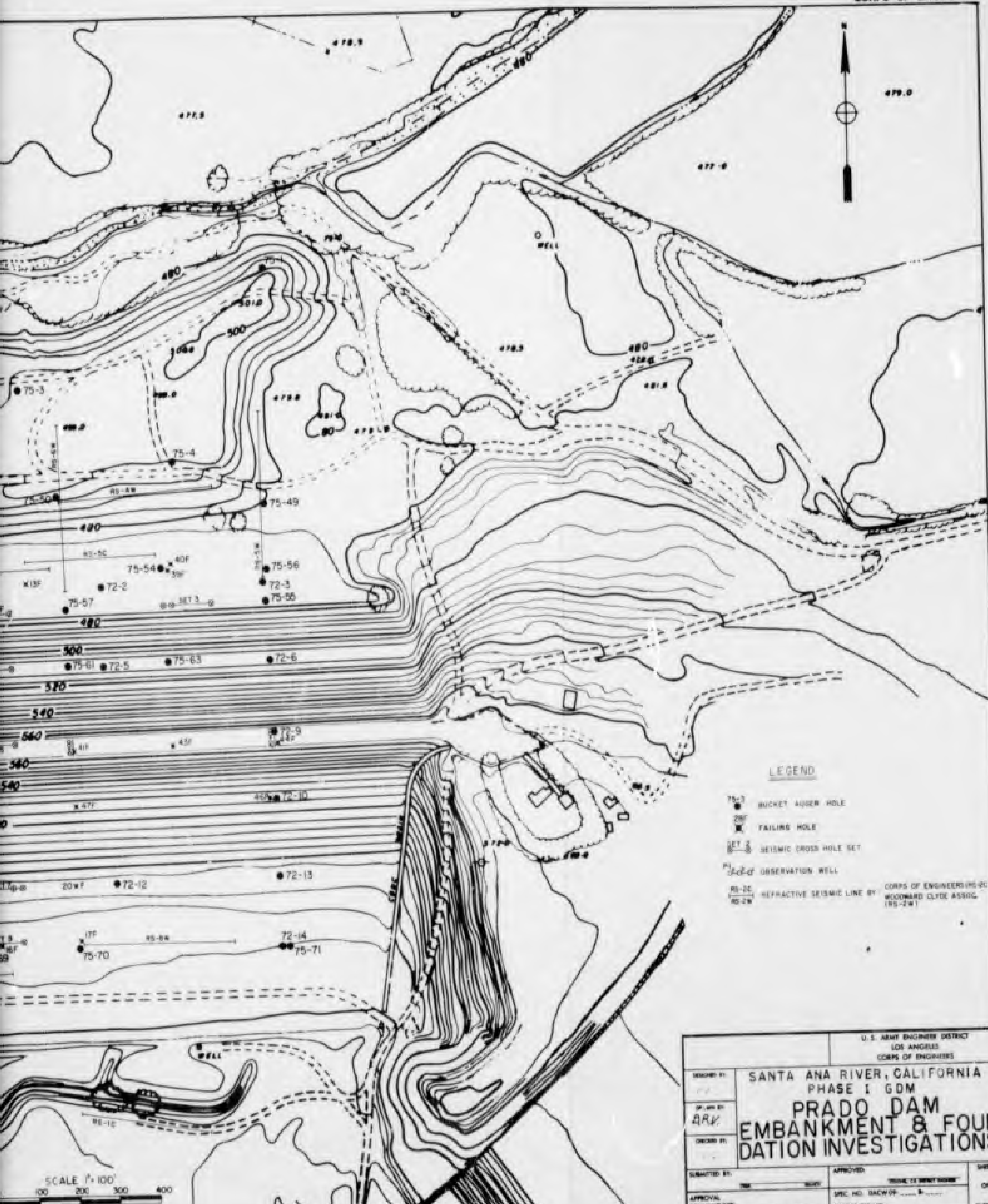
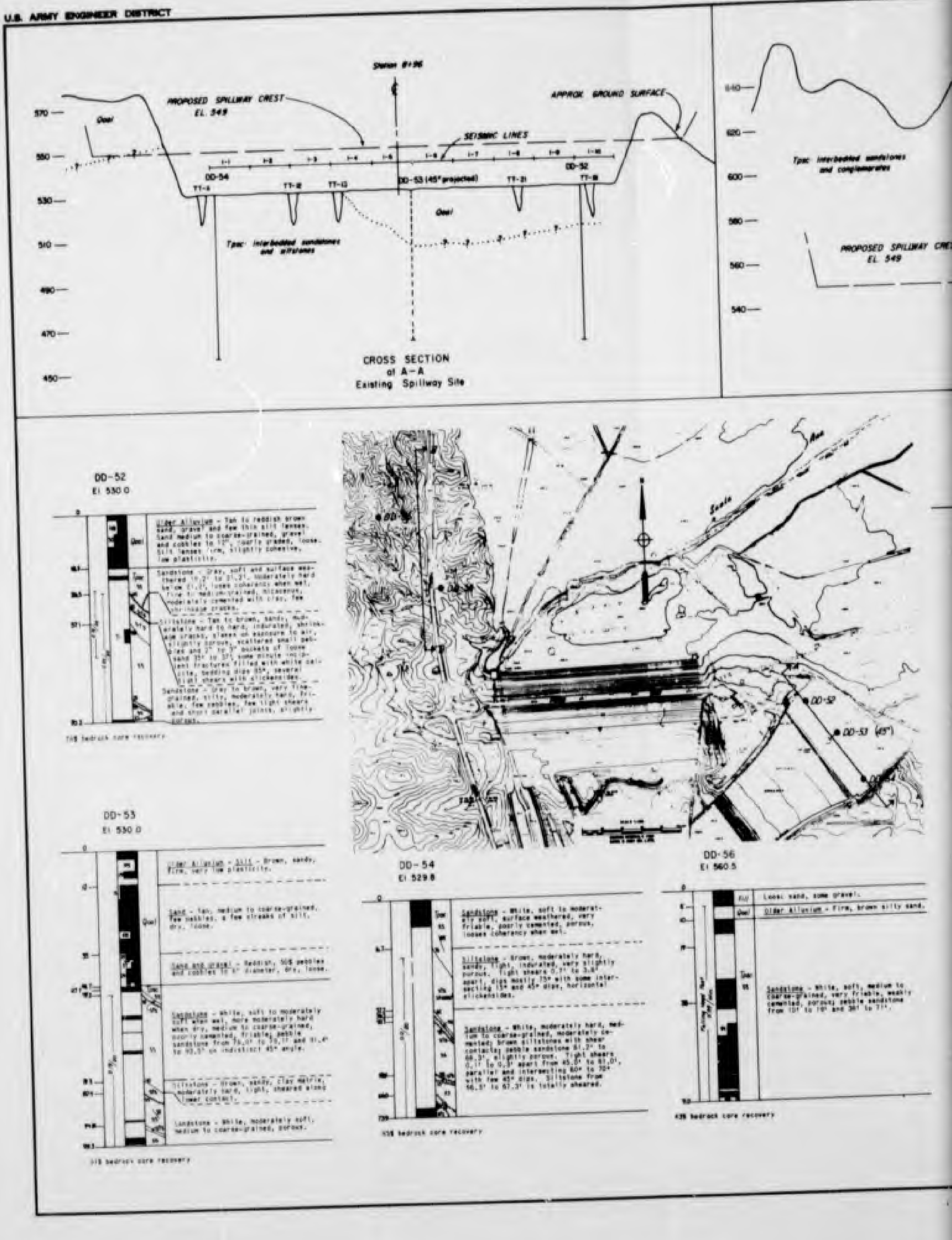
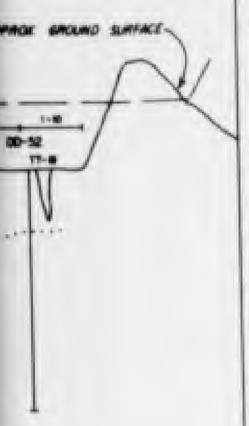
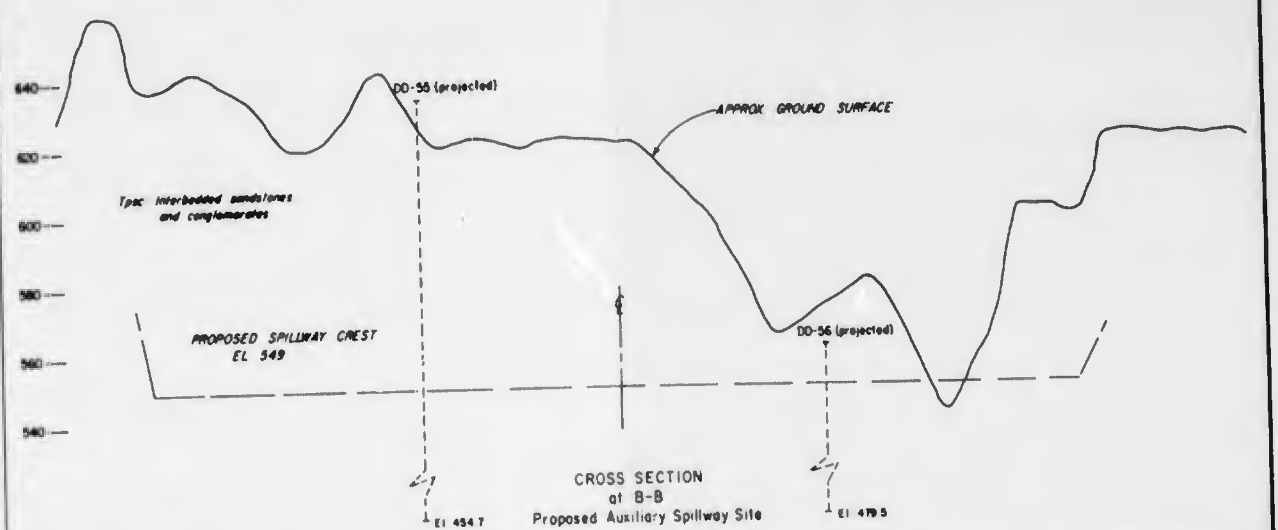


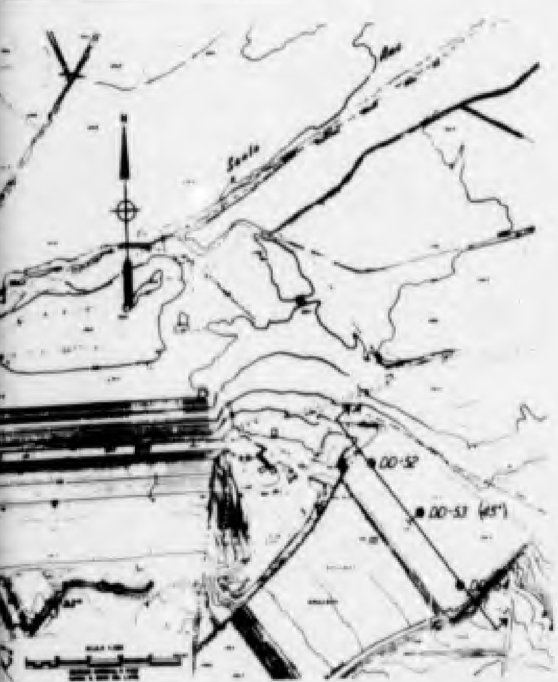
PLATE D-20

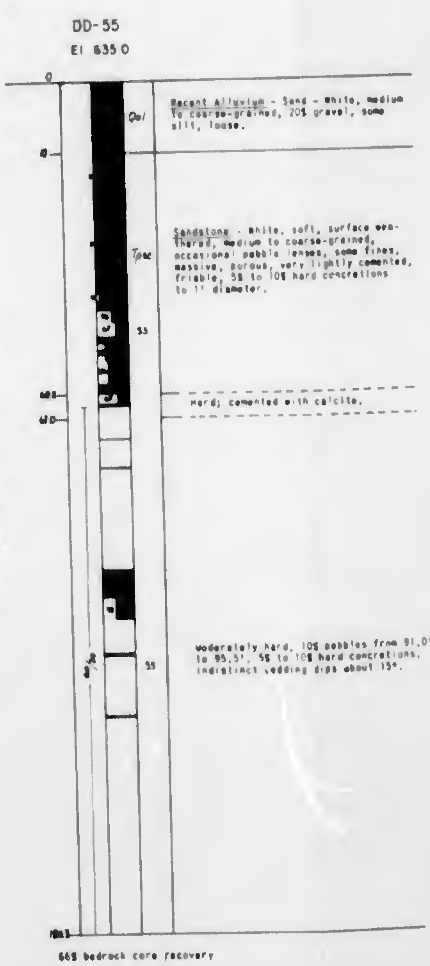


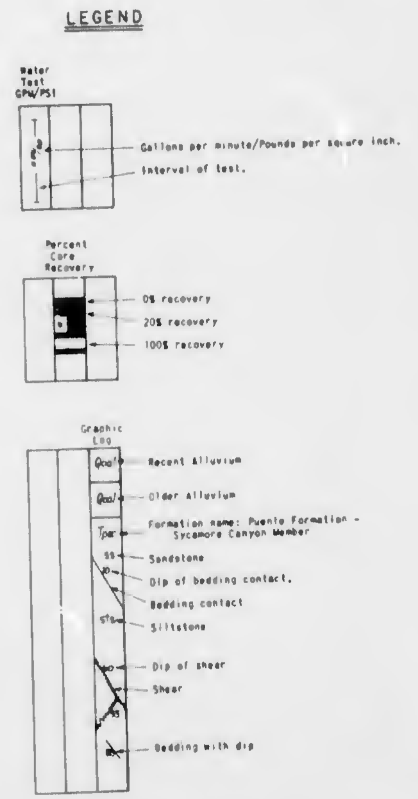
U.S. ARMY ENGINEER DISTRICT
Station 8+96
PROPOSED SPILLWAY CREST
EL 549
APPROX GROUND SURFACE
SEISMIC LINES
Qoal
Tpsc interbedded sandstones and siltstones
DD-54
DD-53 (45° projected)
DD-52
TT-11
TT-12
TT-13
TT-21
TT-19
570
550
530
510
490
470
450
CROSS SECTION
at A-A
Existing Spillway Site
640
620
600
580
560
540
Tpsc interbedded sandstones and conglomerates
PROPOSED SPILLWAY CRE
EL 549
DD-52
El 530.0
DD-53
El 530.0
DD-54
El 529.8
DD-56
El 560.5
DD-52
DD-53 (45°)

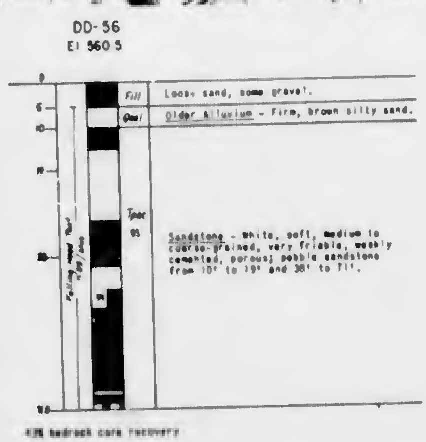












SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

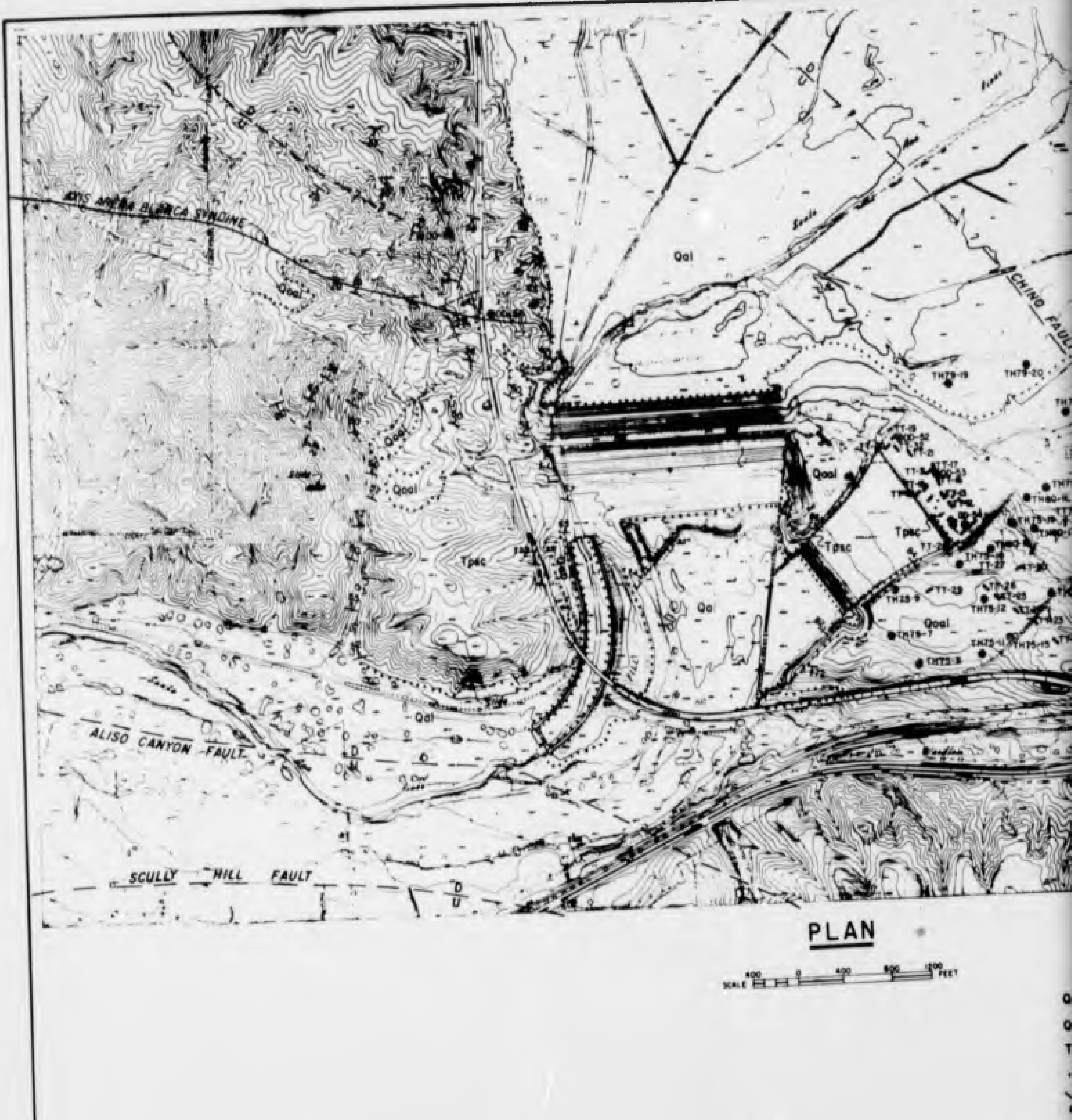
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM
SPILLWAY INVESTIGATION
PLAN SECTION AND DRILL LOGS

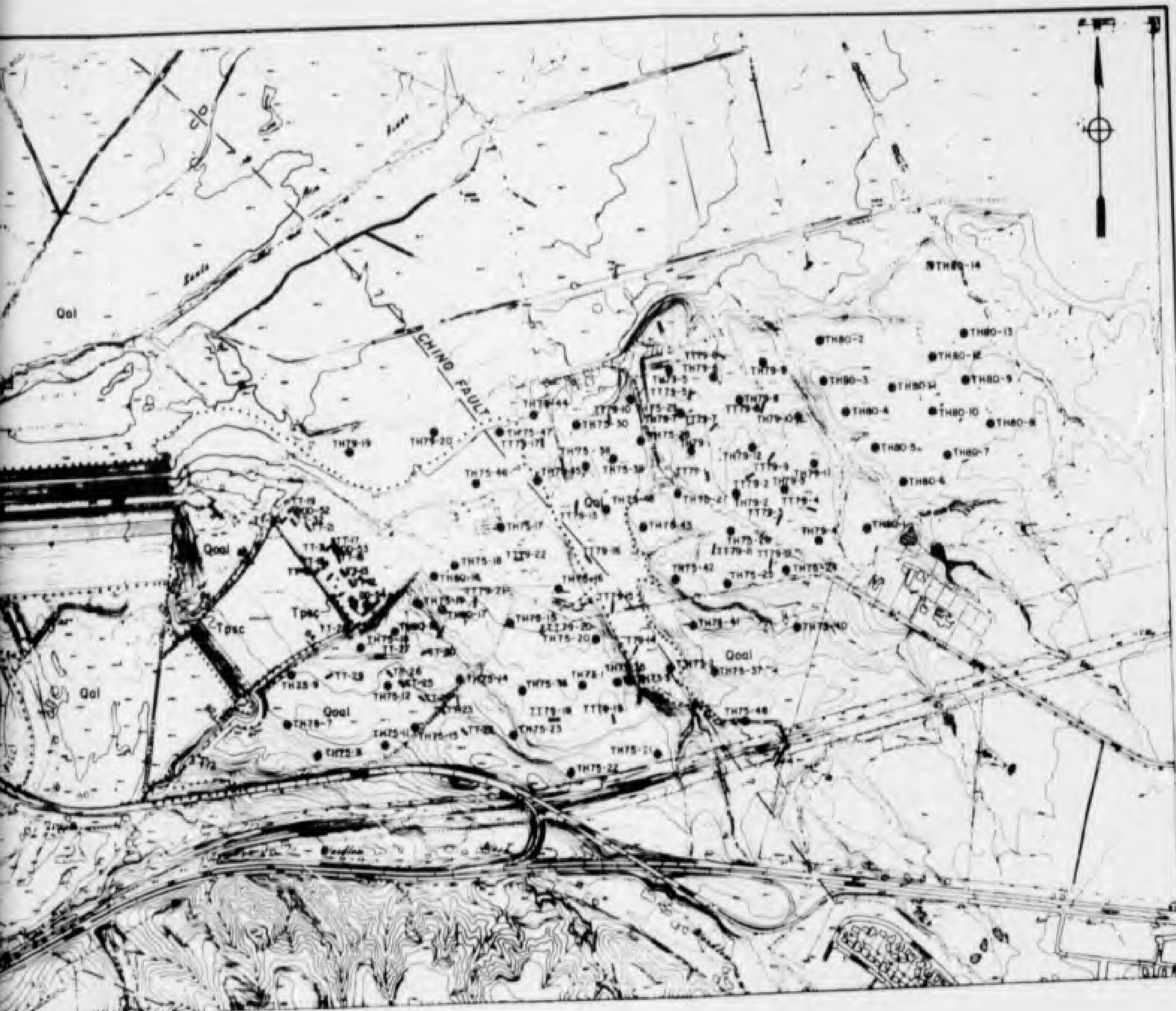
DESIGNED BY: C. W. O.
DRAWN BY: D W L.
CHECKED BY:
SUBMITTED BY:
APPROVED:
APPROVAL RECOMMENDED:
SPEC. NO. DACW09-____ B-____
DISTRICT FILE NO.
SHEET OF SHEETS

2



PLAN

SCALE 400 0 400 800 1200 FEET



PLAN

SCALE 400 0 400 800 1200 FEET

LEGEND

Qal RECENT ALLUVIUM

Qoal OLDER ALLUVIUM

Tpsc PUENTE FORMATION - SYCAMORE CANYON MEMBER

APPROXIMATE CONTACT BETWEEN FORMATIONS

FAULT WITH DIRECTION OF MOVEMENT, QUERIED WHERE LOCATION APPROXIMATE

STRIKE AND DIP OF FAULT

30 STRIKE AND DIP OF BEDROCK STRATA

DD-53 DIAMOND CORE HOLE

TH 73-1 BUCKET AUGER HOLE

TT-23 TEST TRENCH

ELEVATION DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

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

DESIGNED BY:

DRAWN BY:

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM

GEOLOGY AND BORROW
EXPLORATION PLAN

SUBMITTED BY:

DATE:

SPEC. NO.

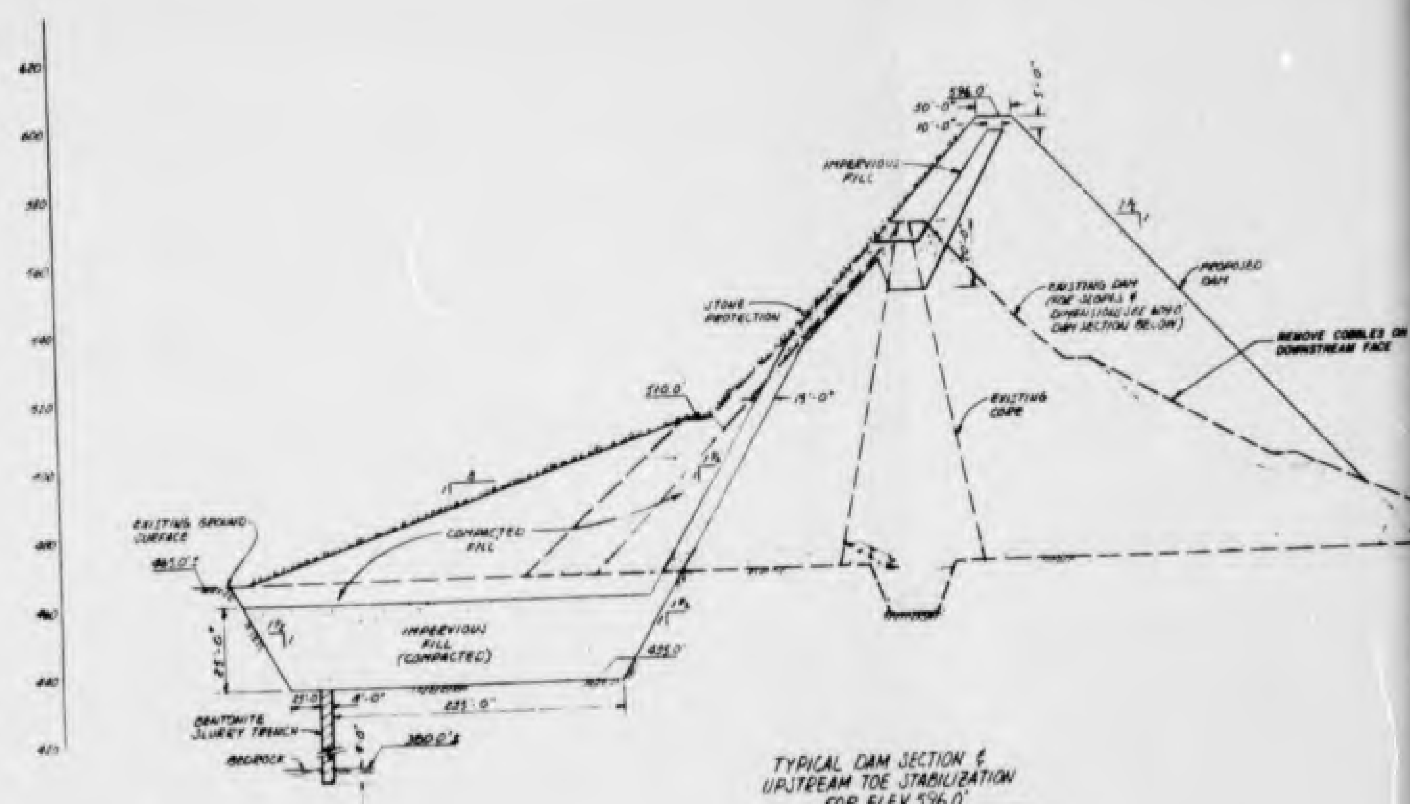
DRAWING NUMBER

DISTRICT FILE NO.

SHEET

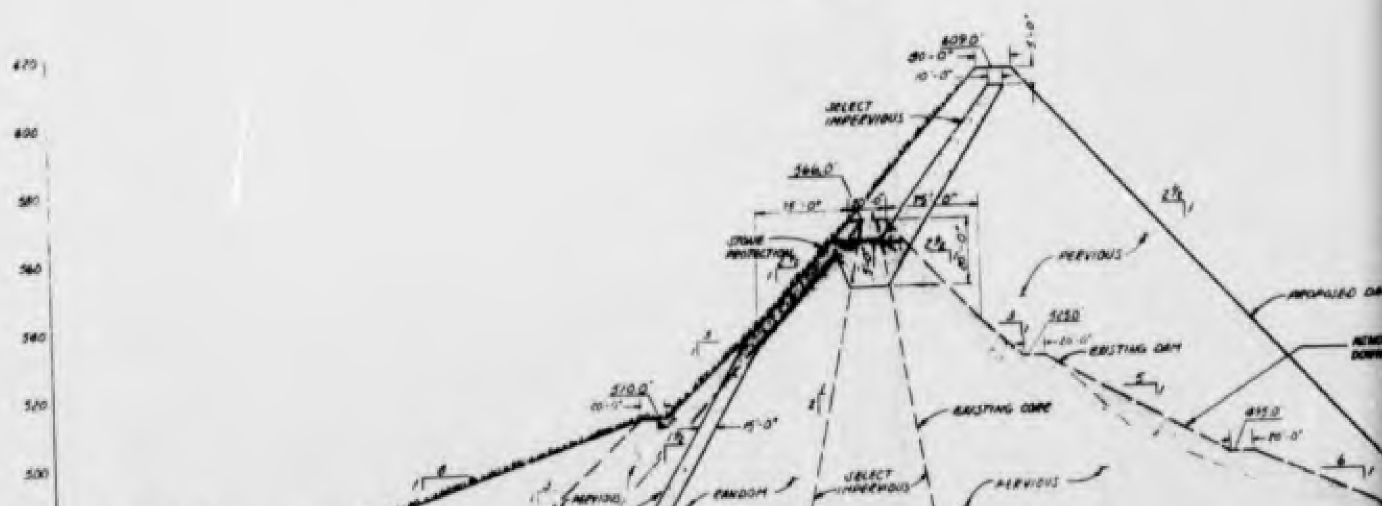
PLATE-D-22

2



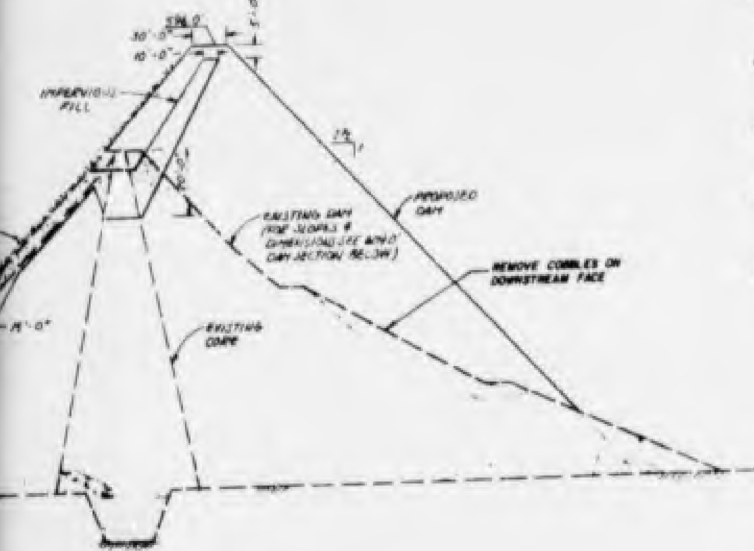
TYPICAL DAM SECTION & UPSTREAM TOE STABILIZATION FOR ELEV 596.0'

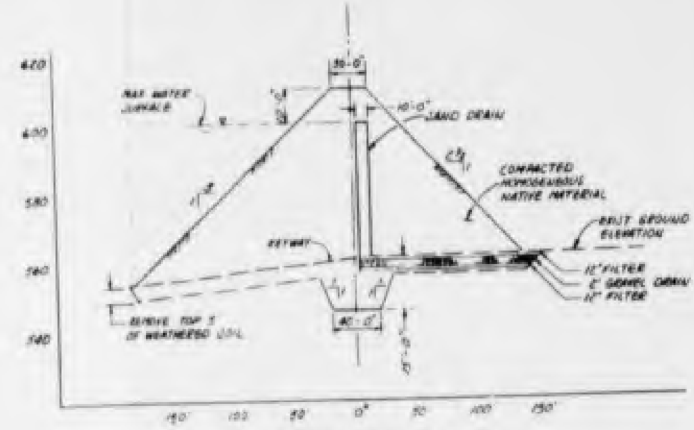
SCALE: HORIZ. 1" = 50'-0"
VERT. 1" = 20'-0"



TYPICAL DAM SECTION & UPSTREAM TOE STABILIZATION FOR ELEV 609.0'

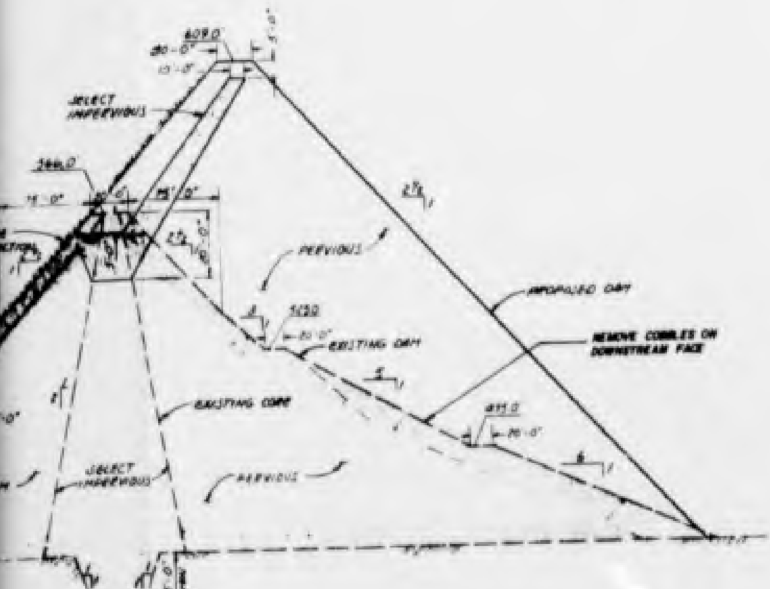
SCALE: HORIZ. 1" = 50'-0"
VERT. 1" = 20'-0"





TYPICAL DIKE SECTION
SCALE HORIZ 1"=50'-0"
VERT 1"=20'-0"

TYPICAL DAM SECTION &
UPSTREAM TOE STABILIZATION
FOR ELEV 596.0'
SCALE HORIZ 1"=50'-0"
VERT 1"=20'-0"



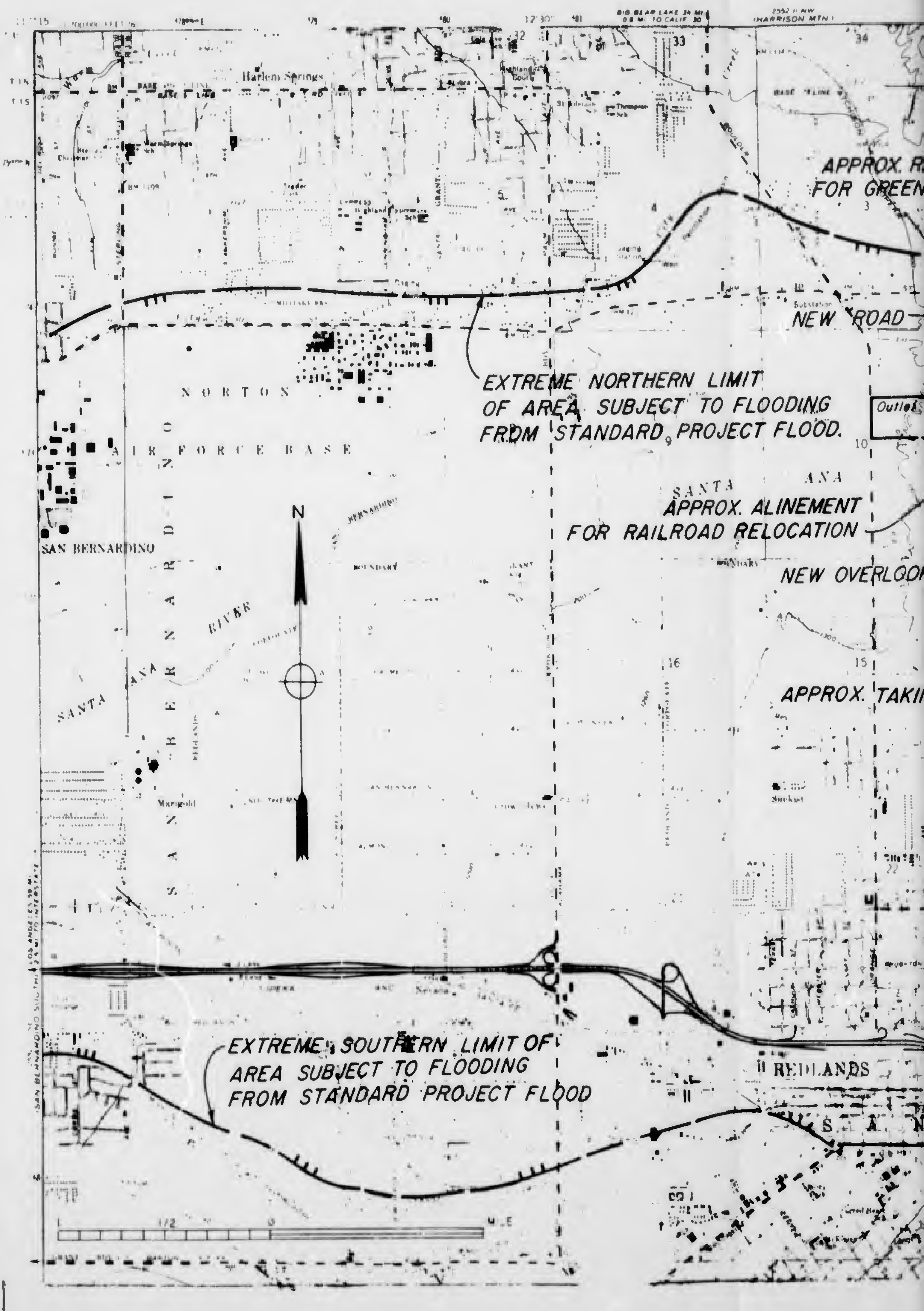
TYPICAL DAM SECTION &
UPSTREAM TOE STABILIZATION
FOR ELEV 609.0'
SCALE HORIZ 1"=50'-0"
VERT 1"=20'-0"

SYMBOL	DESCRIPTION	DATE	APPROVAL
REVISIONS			

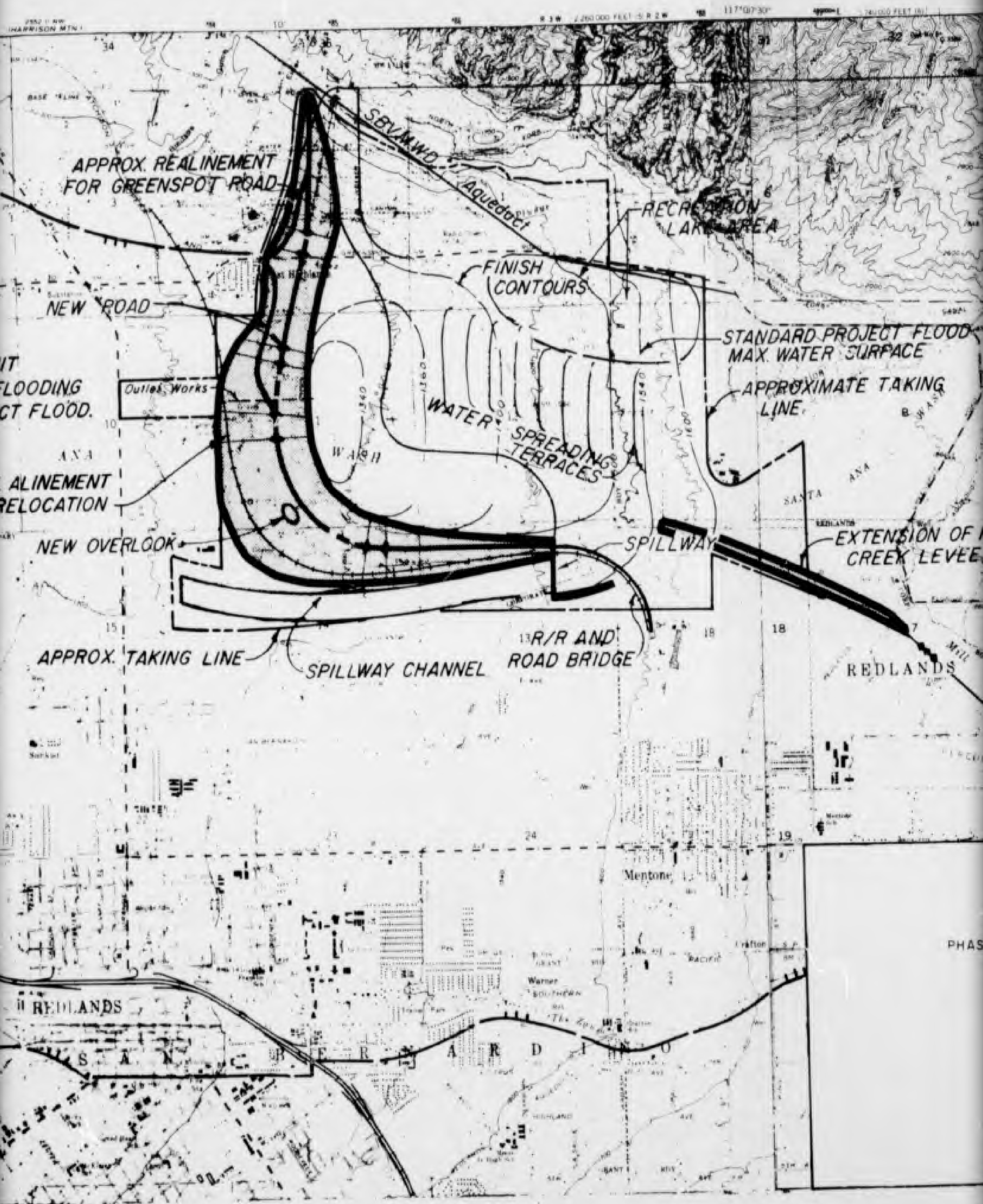
PSOMAS AND ASSOCIATES IRVINE, CALIFORNIA	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY: JRW RLD DRAWN BY: BH DG CHECKED BY: RHB	SANTA ANA RIVER, CALIFORNIA PHASE I GENERAL DESIGN MEMORANDUM PRADO DAM ELEVATIONS 596.0' AND 609.0' TYPICAL DAM AND DIKE SECTIONS
SUBMITTED BY: DATE APPROVED:	SPEC. NO. DACW09-___-B-___ DISTRICT FILE NO.

PLATE D-23

2



APPROX. R
FOR GREEN
NEW ROAD
EXTREME NORTHERN LIMIT
OF AREA SUBJECT TO FLOODING
FROM STANDARD PROJECT FLOOD.
Outlet
NORTON
AIR FORCE BASE
SANTA ANA
APPROX. ALINEMENT
FOR RAILROAD RELOCATION
SAN BERNARDINO
NEW OVERLOO
SANTA ANA RIVER
SAN BERNARDINO
APPROX. TAKI
Harlem Springs
N
EXTREME SOUTHERN LIMIT OF
AREA SUBJECT TO FLOODING
FROM STANDARD PROJECT FLOOD
REDLANDS
1/2 0 MILE



APPROX. REALINEMENT FOR GREENSPOT ROAD
SBVMWD Aqueduct
RECREATION LAKE AREA
FINISH CONTOURS
NEW ROAD
STANDARD PROJECT FLOOD MAX. WATER SURFACE
APPROXIMATE TAKING LINE
Outlet Works
WATER SPREADING TERRACES
ALINEMENT RELOCATION
NEW OVERLOOK
SPILLWAY
EXTENSION OF CREEK LEVEE
APPROX. TAKING LINE
SPILLWAY CHANNEL
R/R AND ROAD BRIDGE
REDLANDS
Mentone
SAN BERNARDINO
PHAS

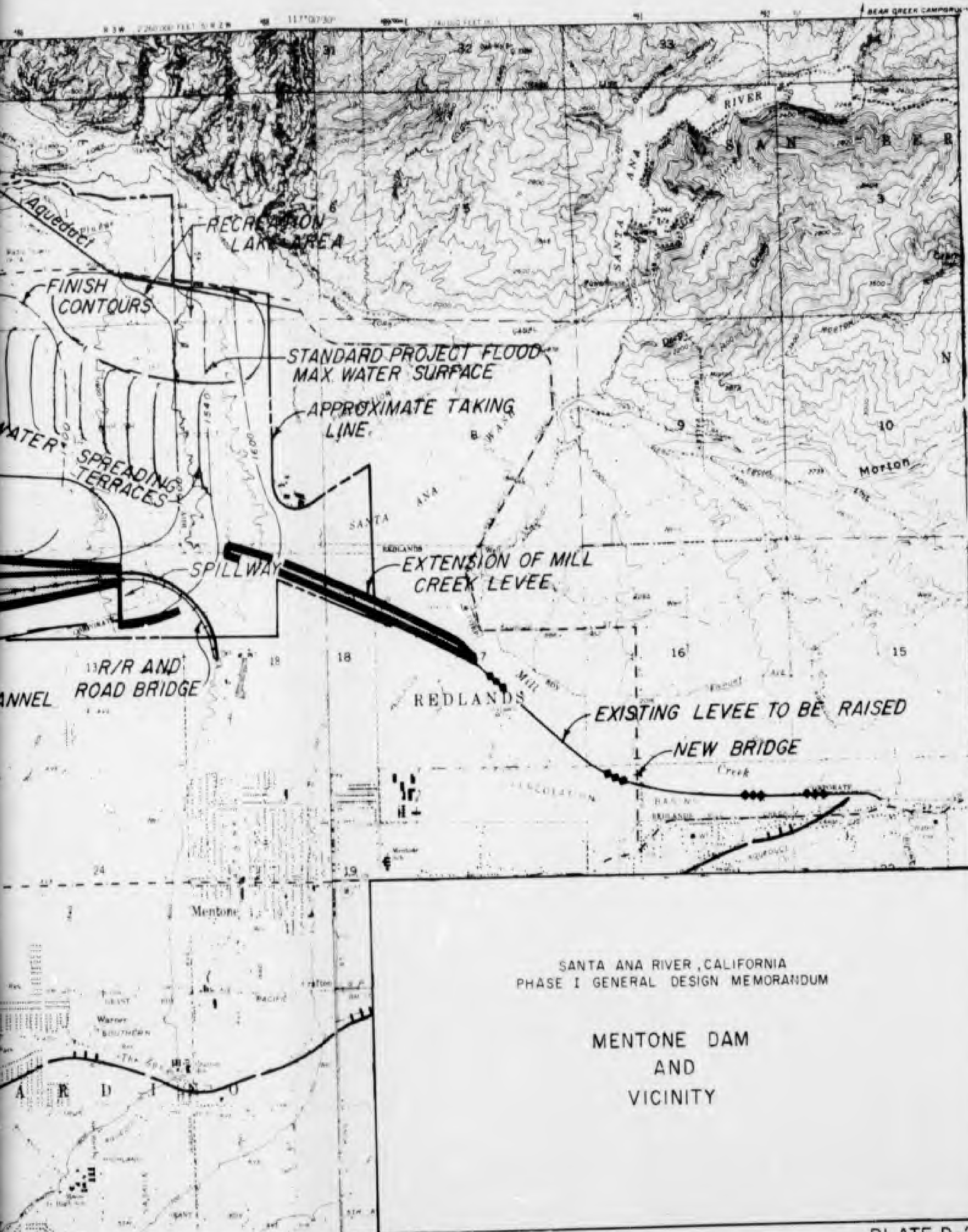

RECREATION LAKE AREA
FINISH CONTOURS
STANDARD PROJECT FLOOD MAX. WATER SURFACE
APPROXIMATE TAKING LINE
SPREADING TERRACES
SPILLWAY
EXTENSION OF MILL CREEK LEVEE
R/R AND ROAD BRIDGE
EXISTING LEVEE TO BE RAISED
NEW BRIDGE
REDLANDS
Mentone
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
MENTONE DAM
AND
VICINITY

PLATE D-24

U. S. ARMY ENGINEER DISTRICT

CORPS OF ENGINEERS

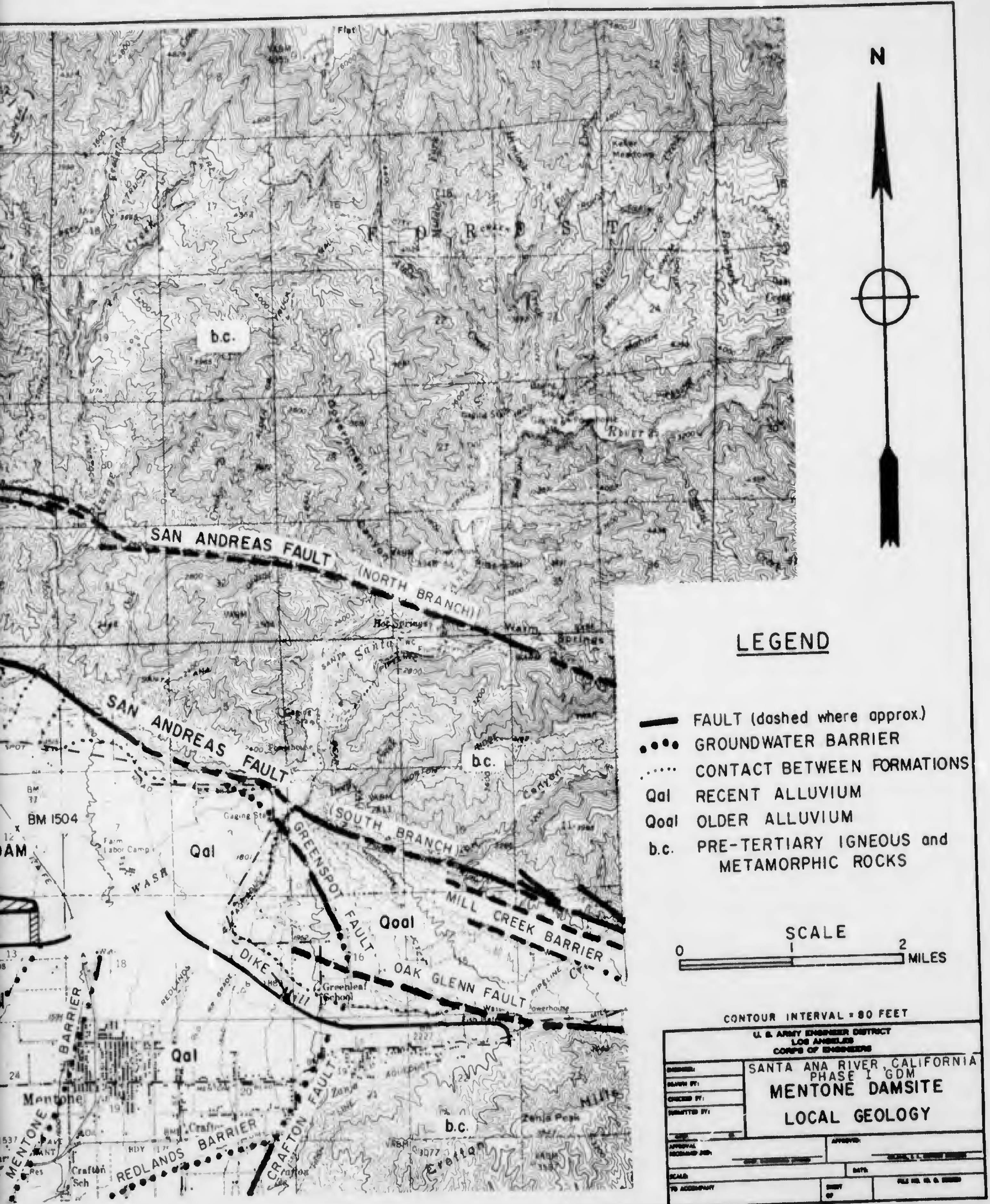
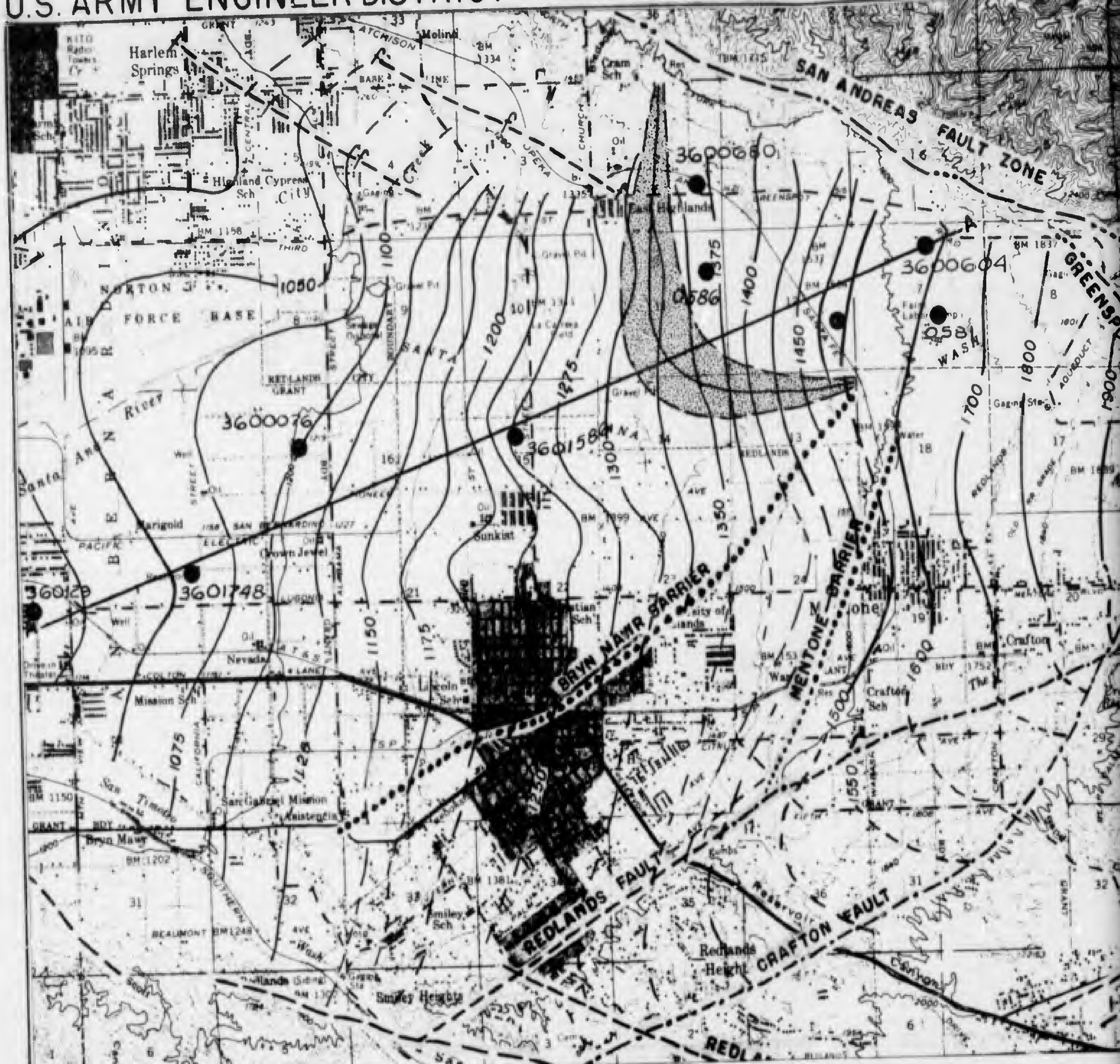


PLATE D-25

U.S. ARMY ENGINEER DISTRICT

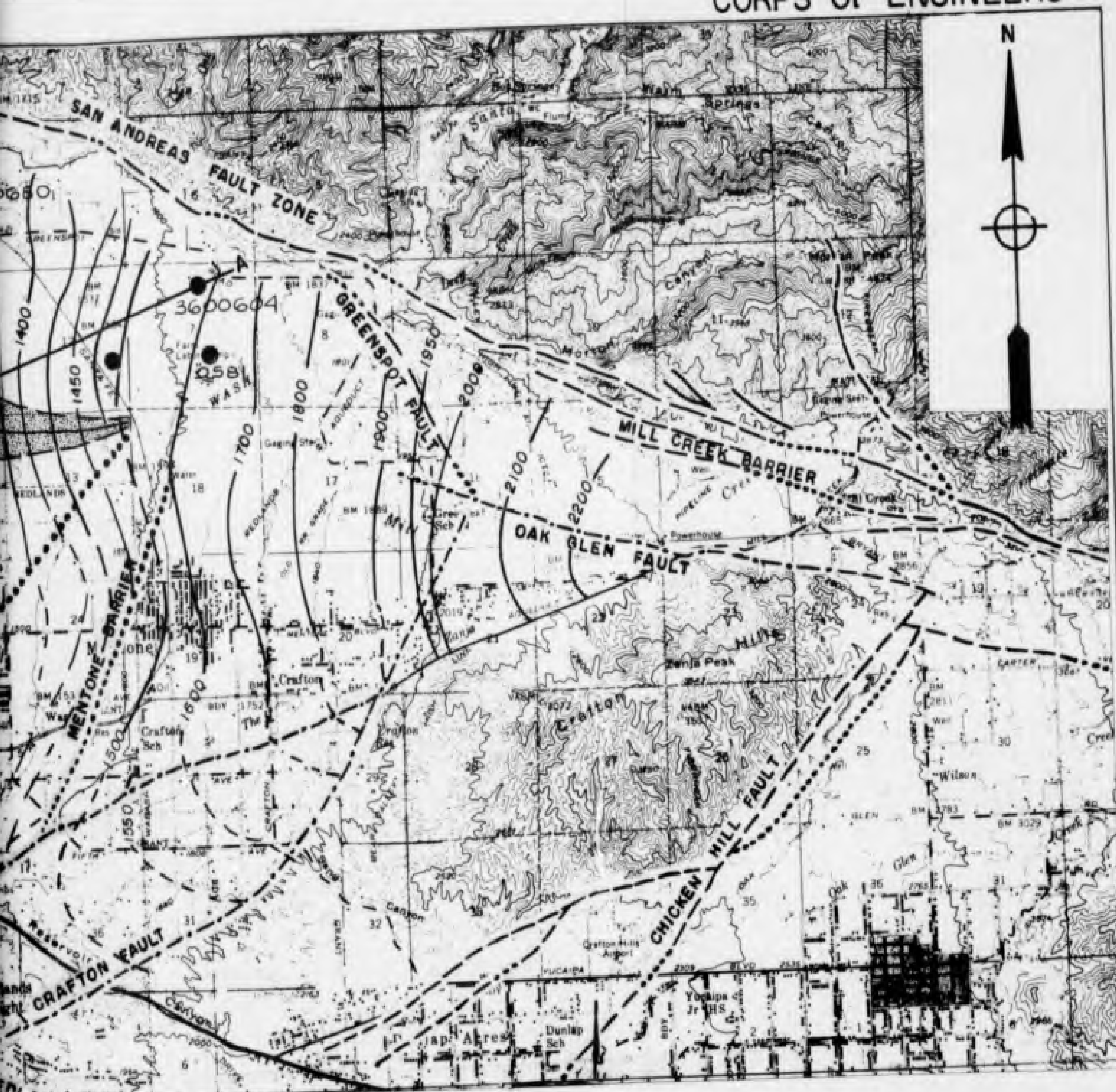


SCALE 2000 1000 0 2000 4000 6000 FEET

NOTES:

1. See plate D-25E for section A-A
2. Well numbers and data for conto from San Bernerdino Valley Mun Water District.
3. Groundwater contours from levels

CORPS OF ENGINEERS



NOTES:

1. See plate D-25E for section A-A.
2. Well numbers and data for contours from San Bernerdino Valley Municipal Water District.
3. Groundwater contours from levels in March 1980.

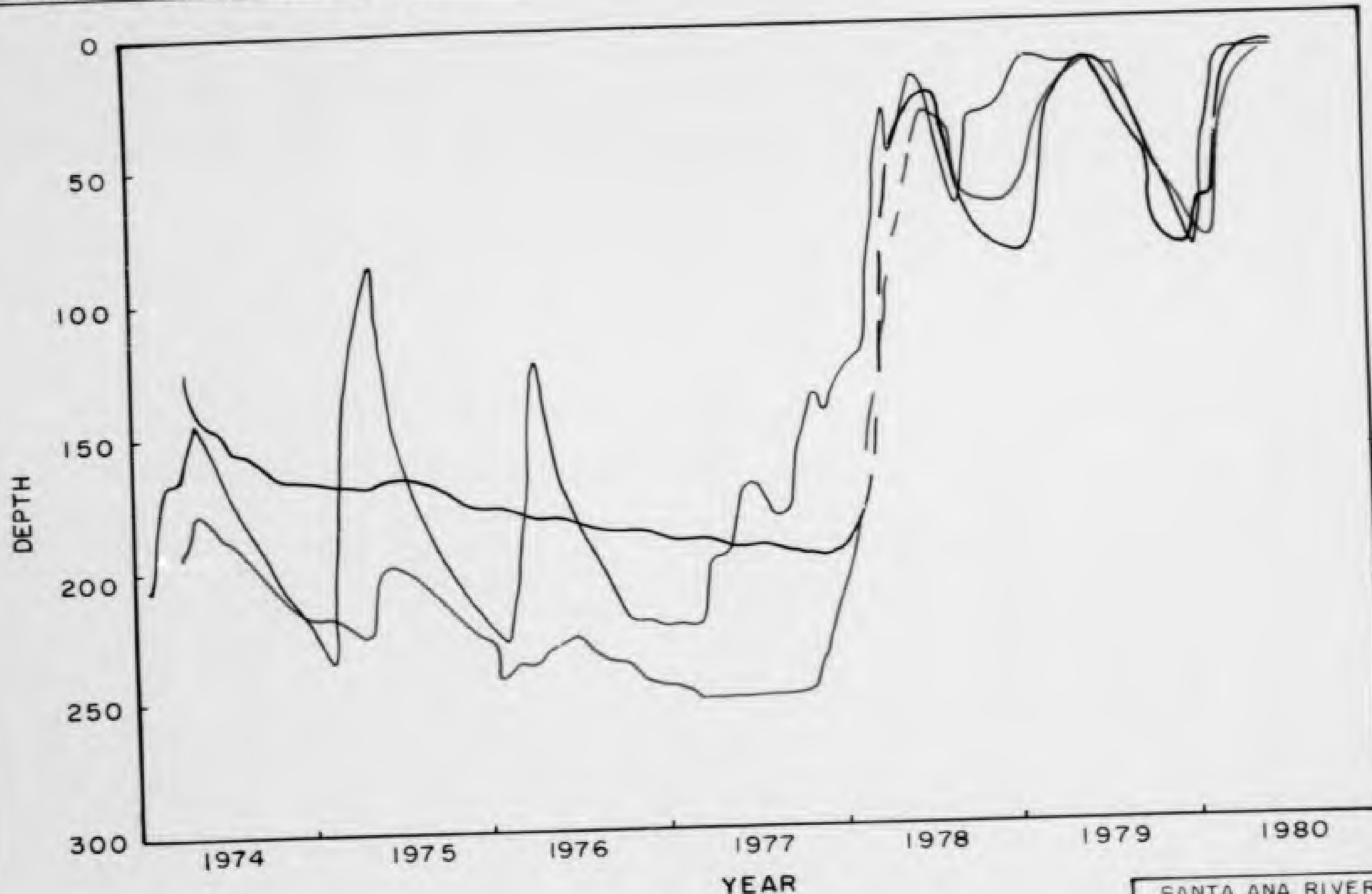
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAM
KEY WATER WELLS
GROUNDWATER CONTOURS
SECTION A-A

PLATE D-25A

U.S. ARMY ENGINEER DISTRICT

CORPS OF ENGINEERS



NOTES:
1.) SEE PLATE D-25A FOR LOCATION OF WELLS
2.) WELLS WERE DRILLED IN 1974.

WELL NUMBER

0581 0585 0586

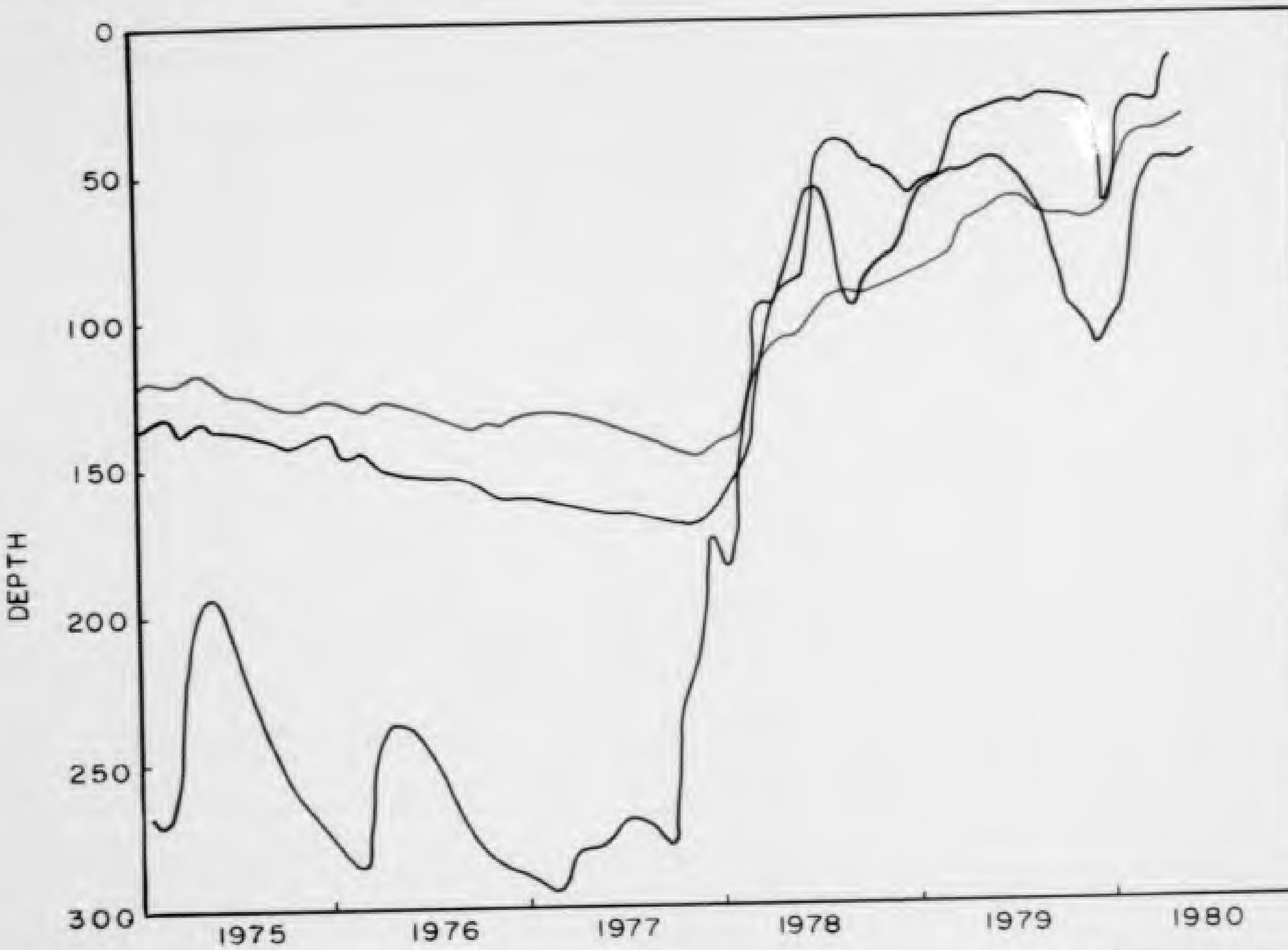
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
WATER LEVELS
1974 - 1980
OBSERVATION WELLS

PLATE D-25B

U.S. ARMY ENGINEER DISTRICT

CORPS OF ENGINEERS



NOTES:
1.) SEE PLATE D-25A FOR LOCATION OF WELLS
2.) SEE PLATE D-25D FOR WATER LEVELS OF THESE WELLS PRIOR TO 1975

WELL NUMBER

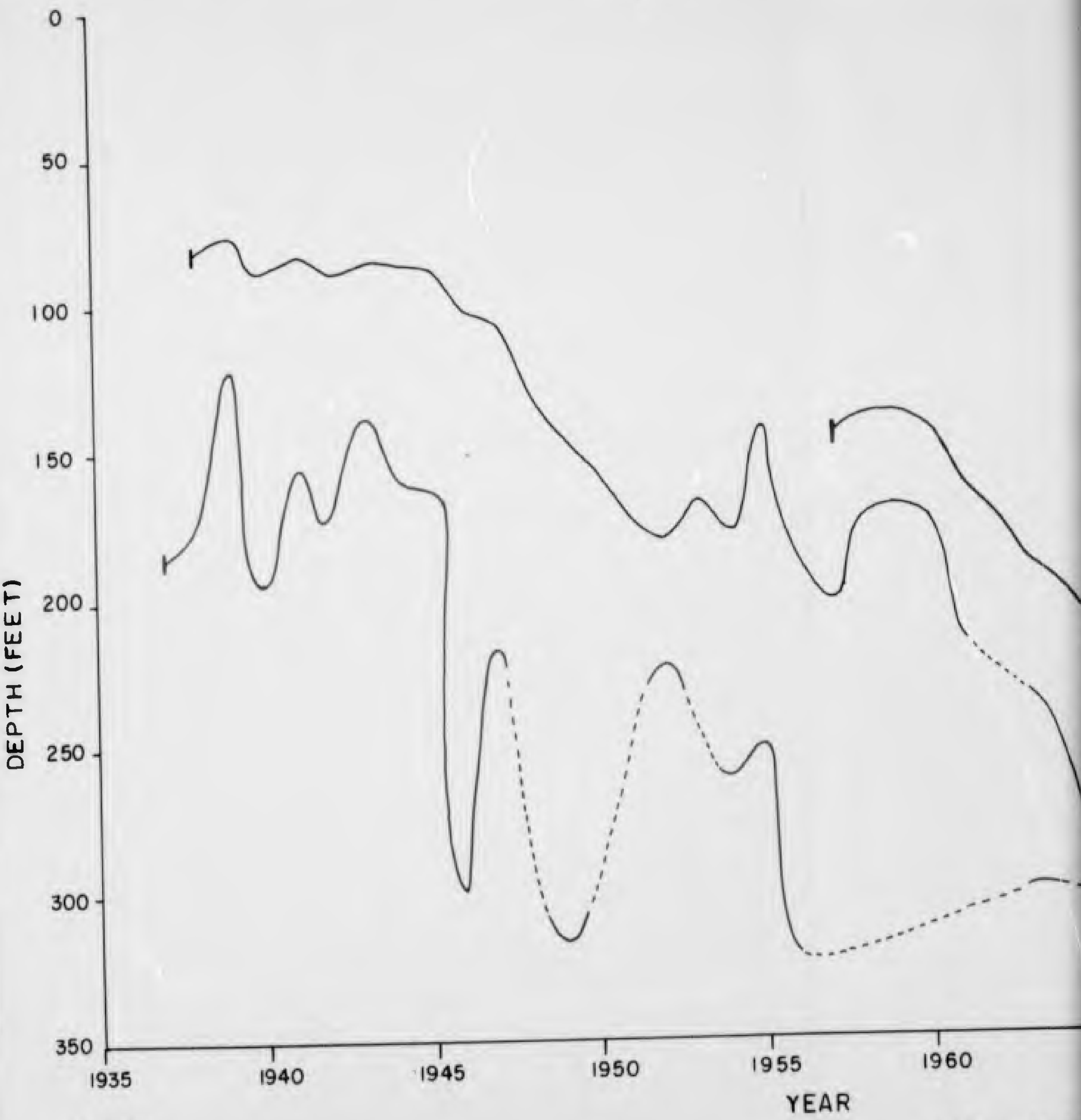
3600604 3601586 3600680

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
WATER LEVELS
1974 - 1980
IRRIGATION WELLS

PLATE D-25C

U. S. ARMY ENGINEER DISTRICT



NOTES:

1. Curves made from highest ground water levels for each water year. Levels generally were highest during the spring.

2. Data from 1946 through 1952 and 1956 through 1972 for well 3600604 is meager.

WELL NUM

3601586 360060

WELL NUMBER

3601586 3600604 3600680

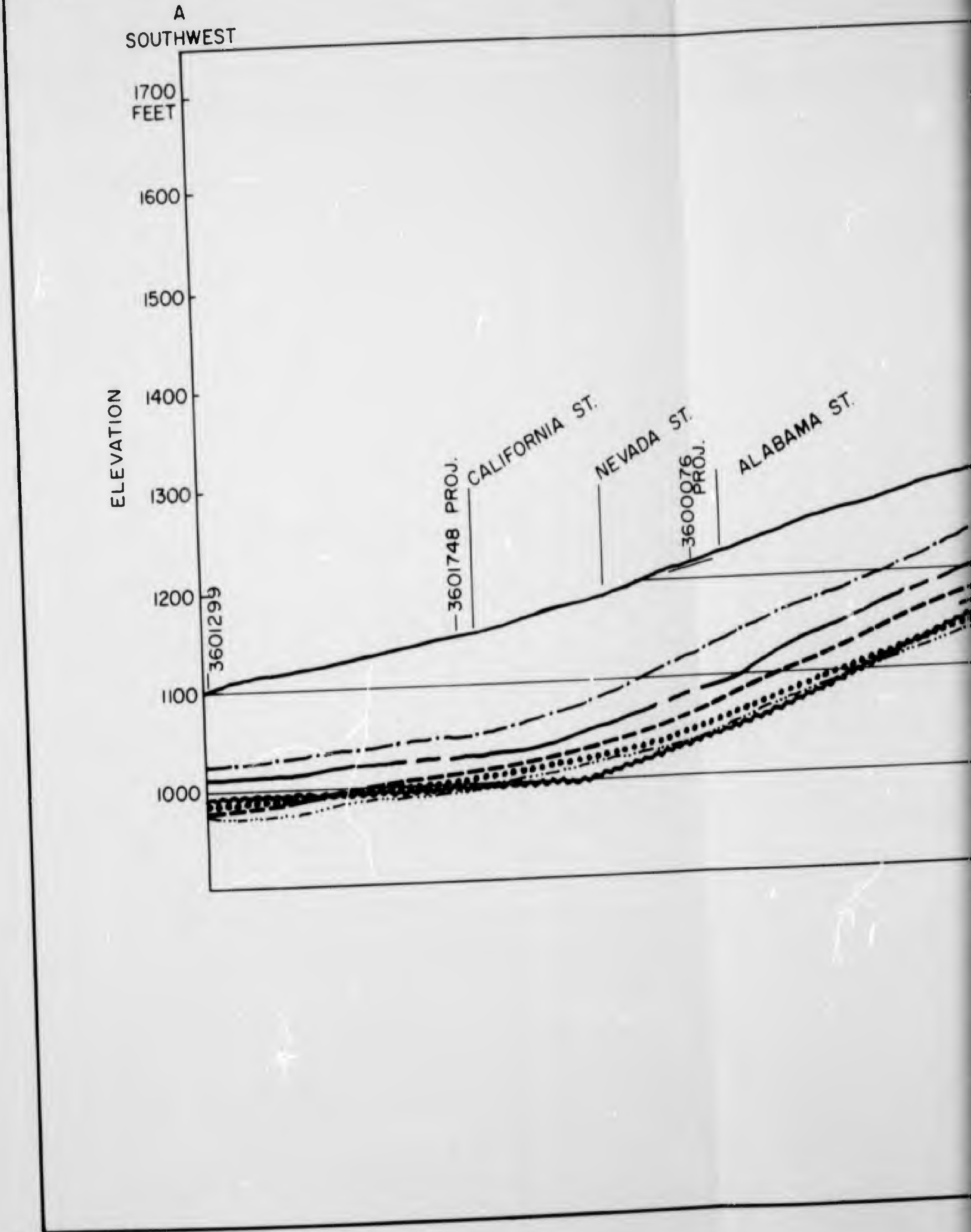
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAM
WATER LEVELS
1938 - 1980
IRRIGATION WELLS

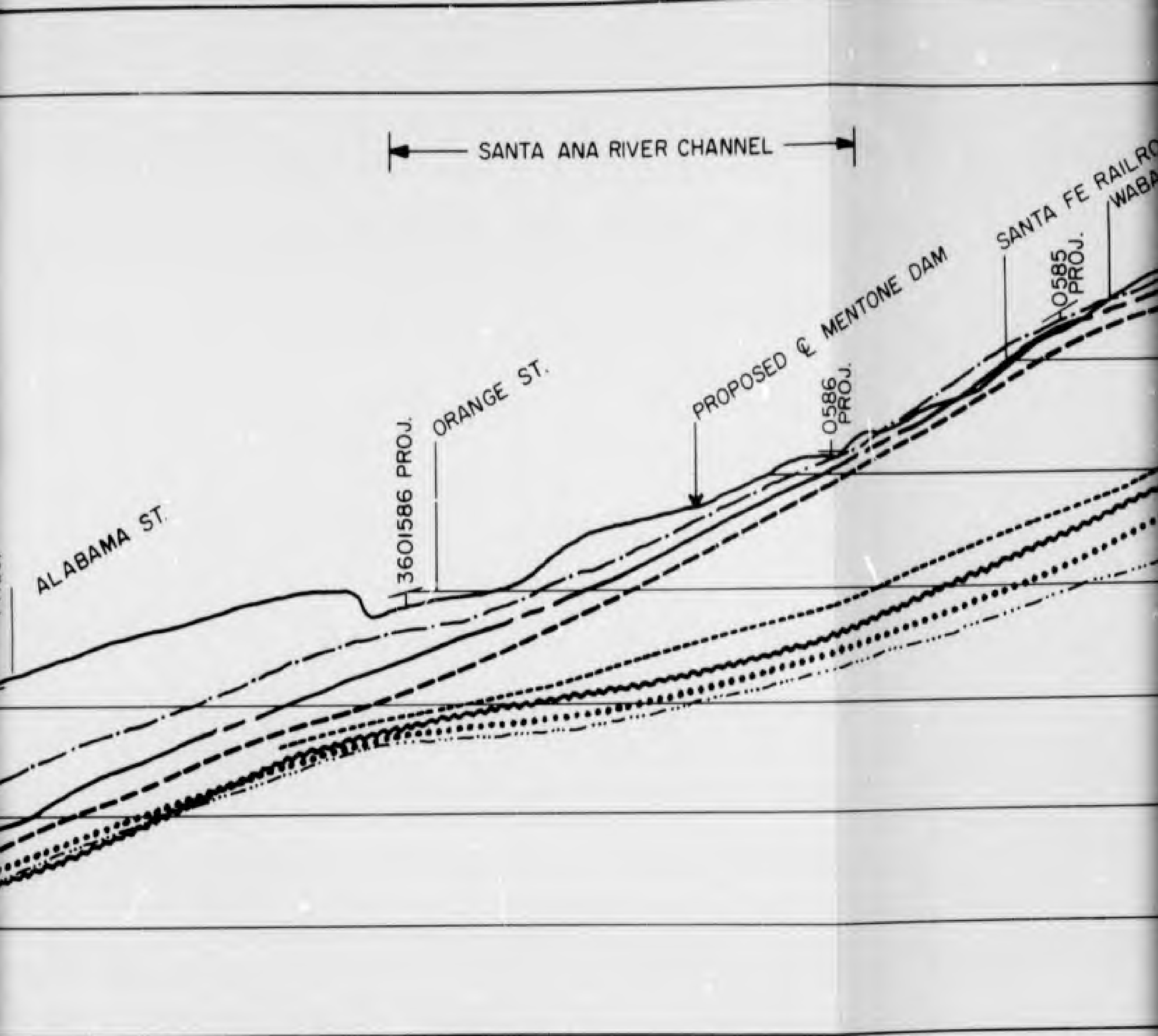
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE D-25D

2

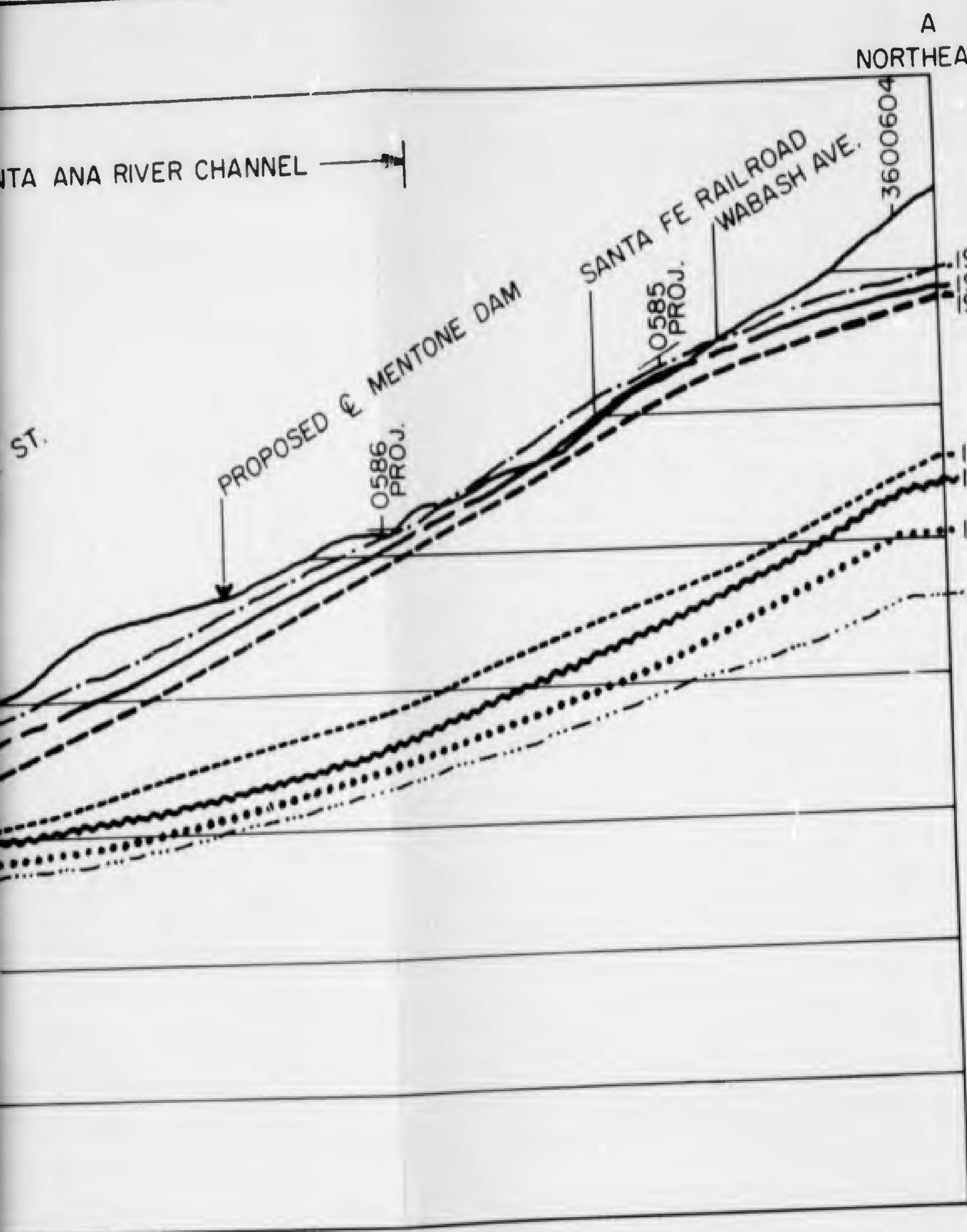


A
SOUTHWEST
1700
FEET
1600
1500
1400
1300
1200
1100
1000
ELEVATION
3601299
3601748 PROJ.
CALIFORNIA ST.
NEVADA ST.
3600076 PROJ.
ALABAMA ST.



NOTE: See plate D-25A for location of section.

2



NOTE: See plate D-25A for location of section.

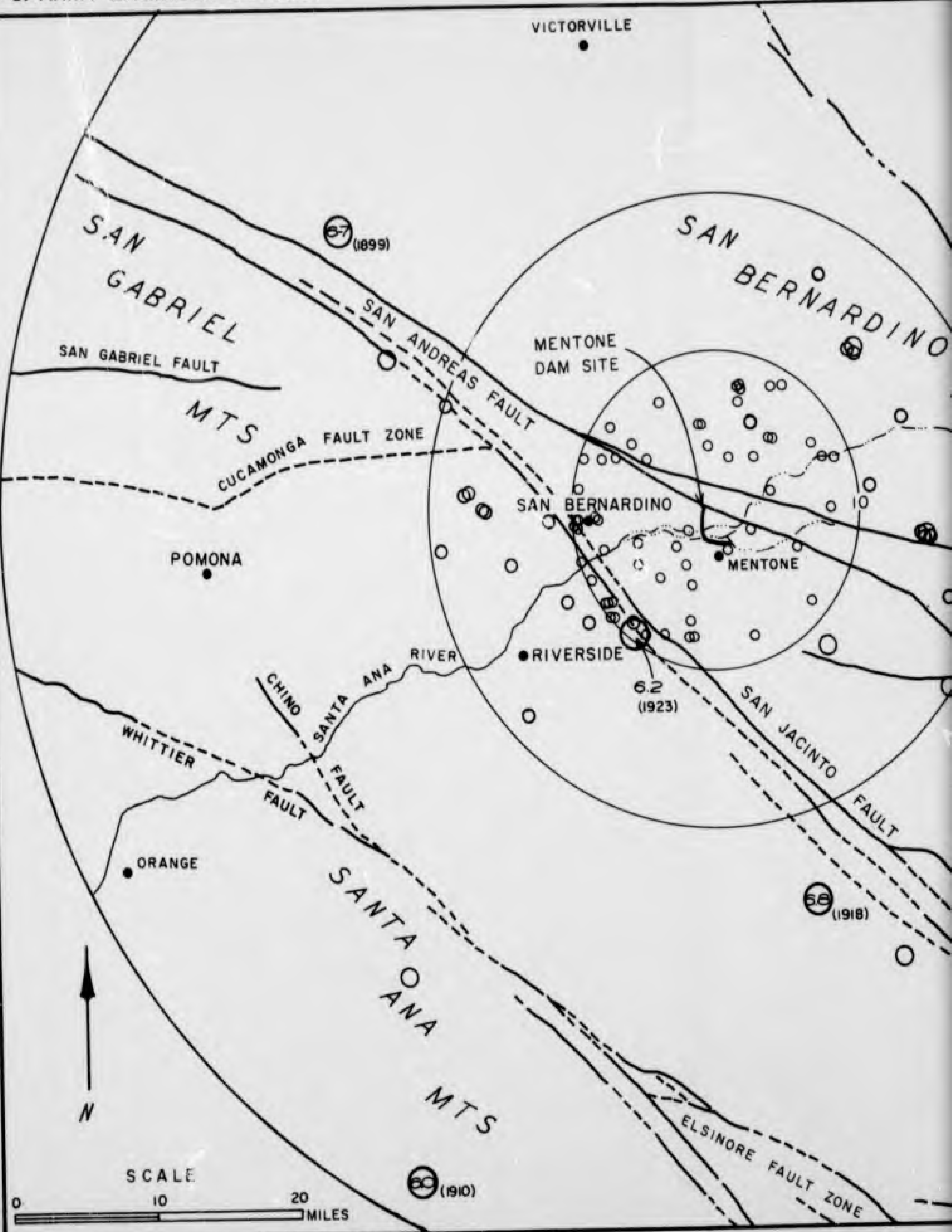
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
GEN. HIGHEST GRNDWTR LEVELS
1974 - 1980
SECTION A - A

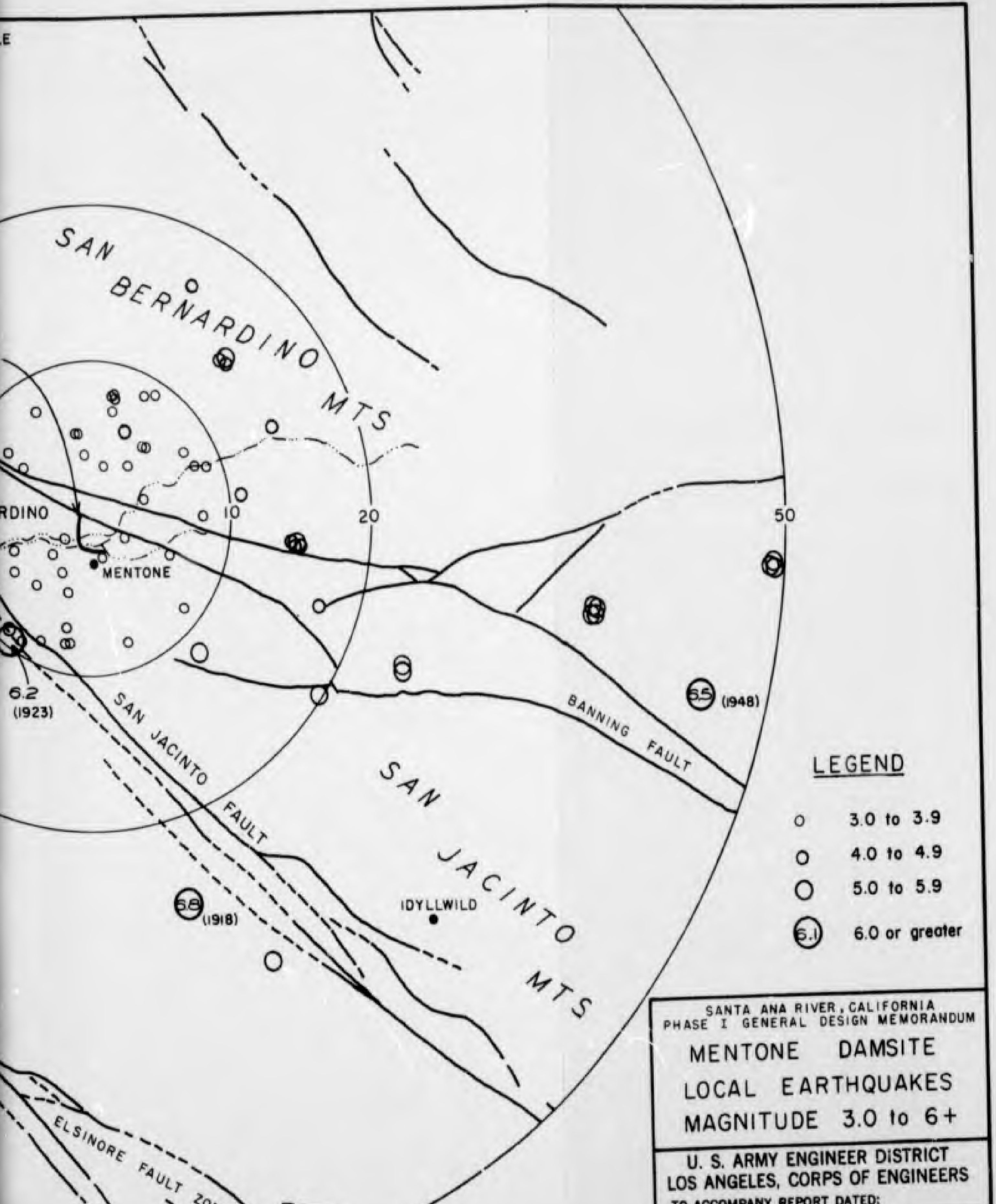
PLATE D-25E

2

U. S. ARMY ENGINEER DISTRICT



CORPS OF ENGINEERS



SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
LOCAL EARTHQUAKES
MAGNITUDE 3.0 to 6+

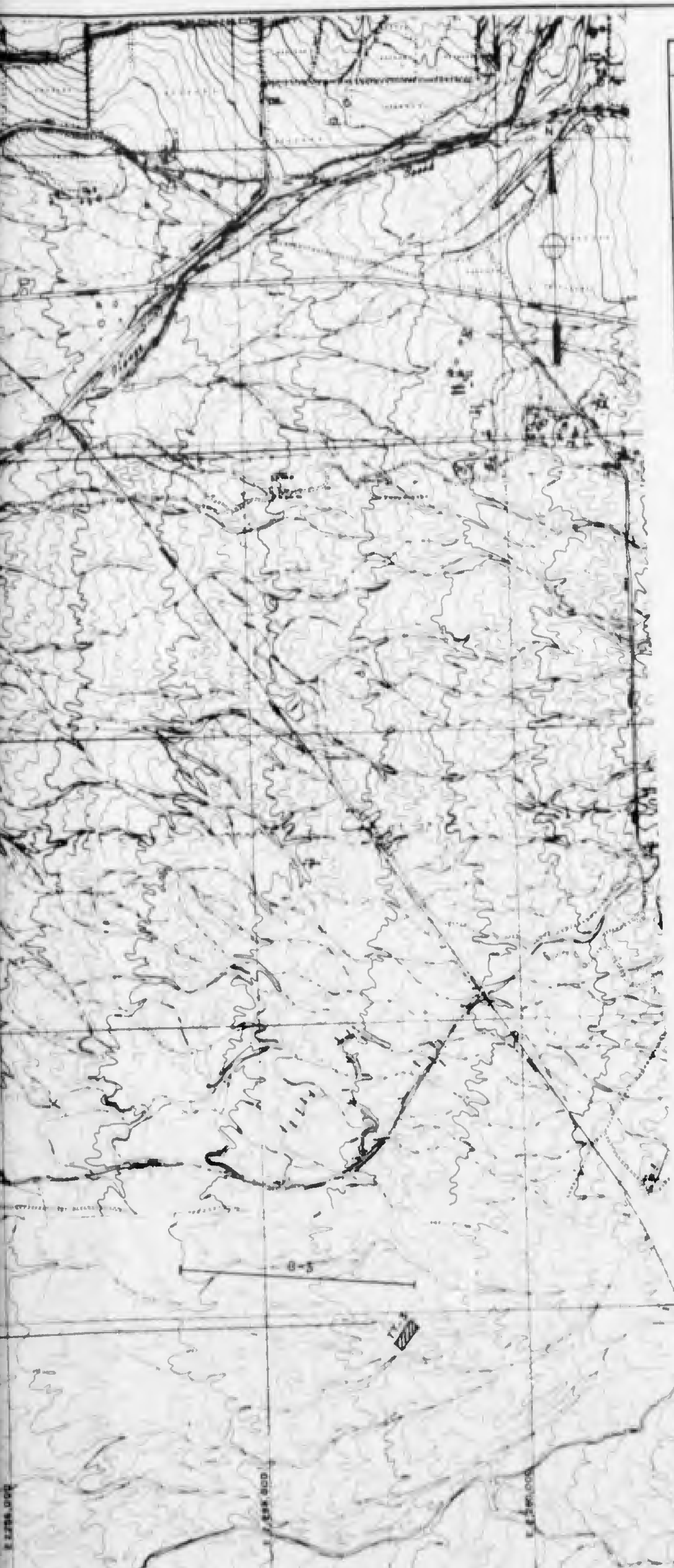
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

2

PLATE D-26

U.S. ARMY ENGINEER DISTRICT

CORPS OF ENGINEERS



UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS [illegible]	GRAVELS [illegible]	Clean gravels	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS [illegible]	Clean sands	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly-graded sands, gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS [illegible]	SILTS AND CLAYS	Low liquid limit	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts, with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
		High liquid limit	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			Pt	Peat and other highly organic soils

NOTES:

1. Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.

2. All sieve sizes on this chart are U. S. Standard.

3. The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus no. 200 sieve material is silt if the liquid limit and plasticity index plot below the "A" line on the plasticity chart (Table VI, Military Standard 619A), and is clay if the liquid limit and plasticity index plot above the "A" line on the chart.

4. For a complete description of the Unified Soil Classification System, see "Military Standard 619A" dated 20 March 1962

LEGEND

TT-1	LOCATION AND NUMBER OF TEST TRENCH AND PIT
MC	FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT.
LL	LIQUID LIMIT.
PI	PLASTICITY INDEX (LIQUID LIMIT MINUS PLASTIC LIMIT).
NP	NONPLASTIC
-4	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 4 SIEVE
-200	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 200 SIEVE.
G-2 A	SEISMIC SHOT HOLE
B C	SEISMIC GEOPHONE HOLES.

NOTES:
1. TT-1 through TT-3 where excavated during May - June 1974 with a D-9 dozer and a clam-shell.
2. The logs of test trenches are shown on plates D-28 to D-30
3. No ground water was encountered in the test trenches and pits

SCALE 1 IN = 400 FT

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
APPROVAL RECOMMENDED:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE

PLAN OF EXPLORATIONS

APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.

SHEET OF SHEETS

PLATE D-27

2

U.S. ARMY ENGINEER DISTRICT

TT-1

Depth	LOG	MC	LL	PI	-4	-200	UW	% COMP	Description
0' – 5'	SP SM	1			80	6			SAND - SILTY SAND grey-brown, loose, gravel pockets, cobbles and boulders to 18" max.
5' – 6.5'	SP	2			93	2	[illegible]	100%	SAND brown, dense, gravel to 2" max.
6.5' – 9'	SP SM	1			100	5	[illegible]		SAND - SILTY SAND brown, dense, gravel to 2" max.
9' – 13.5'	SP	2			46	1	[illegible]	90%	SANDY GRAVEL light brown, dense, 20% cobbles and boulders to 30" max.
		1			43	2			
		5			40	2	[illegible]	97%	
13.5' – 18'	SP				57	2			GRAVELLY SAND light brown, dense, small boulders.
18' – 19.5'	GP	4			37	2	[illegible]	98%	SANDY GRAVEL light brown, dense, small cobbles
19.5' – 24'	SP SM	3			57	5			SAND - SILTY SAND brown, very dense, 40% cobbles and boulders to 60" max. One 10' boulder
24' – 25.5'	GP	5			26	3	[illegible]	100%	SANDY GRAVEL brown, open gravel, dense
25.5' – 29'	GP GM	5			53	5			SANDY GRAVEL - SILTY SANDY GRAVEL brown, dense, 30% cobbles and boulders to 30" max.
29' – 30.5'	SP	6			77	3	118.7	99%	GRAVELLY SAND brown, very dense.
30.5' – 33'	SP SM	5			79	7			GRAVELLY SAND - SILTY GRAVELLY SAND brown, very dense, cobbles and boulders to 24" max.
33' – 37'	SP	4			62	5	[illegible]	100%	GRAVELLY SAND brown, dense cobbles and boulders to 24" max.
		5			71	3			
37' – 38.5'	SP	5			44	2	[illegible]	99%	SANDY GRAVEL dense
38.5' – 42'	SP	4			73	4			GRAVELLY SAND brown, dense, cobble and boulders to 60" max.
42' – 43.5'	SP	7			56	4			GRAVELLY SAND brown, dense
43.5' – 45'	GP	6			42	3	[illegible]	40%	SANDY GRAVEL brown, dense
45' – 49.5'	GP GM	6			54	5			SANDY GRAVEL - SILTY SANDY GRAVEL brown, dense, cobbles and boulders to 36" max.
49.5' – 51'	GP	9			41	4	[illegible]	99%	SANDY GRAVEL brown, dense, cobbles are clay coated.

TT-1A

Depth	LOG	MC	LL	PI	-4	-200	N	Description
0' – 7'	SP		V I	S	U A	L		SAND grey-brown, loose, gravel and sand pockets, cobbles and boulders to 18" max.
7' – 11'	SP	3			92	2		GRAVELLY SAND brown dense, 30% cobbles and boulders to 14" max.
11' – 23'	GP	3			28	1		SANDY GRAVEL brown open gravel.
		2			51	2		Brown dense, 15% cobbles to 10" max.
		4			44	2		
		9			46	1		Wet, 25% cobbles and boulders to 14" max.
		6			39	1		Moist 30% cobbles and boulders to 36" max.
23' – 28'	SP	6			57	2		GRAVELLY SAND brown, very dense, 40% cobbles and boulders to 36" max.
		10			65	2		

TT-1B

Depth	LOG	MC	LL	PI	-4	-200	UW	Description
0' – 5'		V	I	S	U A	L		
5' – 7'	SP				30	1		GRAVELLY SAND tan, medium dense, cobbles and boulders to 18" max.
7' – 14'	GP				34	1		SANDY GRAVEL light brown, dense, 40% cobbles and boulders to 18" max.
					42	2		20% cobbles and boulders to 30" max open gravel at 12'.
14' – 16'	SP				80	3		SAND light brown, dense, 15% cobbles to 10" max.
16' – 19'	GP				32	1		SANDY GRAVEL light brown, dense, 20% cobbles to 10" max.
19' – 20'	SP				96	2	[illegible]	SAND light brown, dense.
20' – 28'	GP				40	1		SANDY GRAVEL light brown, dense, 40% cobbles and boulders to 48" max.
					53	4		

TT-1C

Depth	LOG	MC	LL	PI	-4
0' – 6'		V	I S	U	A
6' – 8.5'	SP				[illegible]
8.5' – 10'	SP				[illegible]
10' – 13.5'	GP				[illegible]
					[illegible]
13.5' – 15'	SP				[illegible]
15' – 20'	GP				[illegible]
					[illegible]
20' – 24'	GP				[illegible]
24' – 28'	SP				[illegible]

TT-1D

Depth	LOG	MC	LL	PI
0' – 5'		V I	S	U
5' – 8'	GP			
8' – 9.5'	SP			
9.5' – 12'	GP			
12' – 13'	SP			
13' – 15'	GP			
15' – 17.5'	SP			
17.5' – 29'	GP			

CORPS OF ENGINEERS

SAND grey-brown, loose, gravel and sand pockets, cobbles and boulders to 18" max

GRAVELLY SAND brown dense, 30% cobbles and boulders to 14" max

SANDY GRAVEL brown open gravel

Brown dense, 15% cobbles to 10" max

Wet, 25% cobbles and boulders to 14" max

Moist 30% cobbles and boulders to 36" max

GRAVELLY SAND brown, very dense, 40% cobbles and boulders to 36" max

TT-1C

Depth	LOG	MC	LL	PI	-4	-200	UN	Description
0'		V	I	S	U A	L		
6'	GP				44	1		SANDY GRAVEL light brown, dense, 20% cobbles and boulders to 24" max
8.5'	SP				52	1		GRAVELLY SAND light brown, dense
10'	GP				44	2		SANDY GRAVEL light brown, lenses of open gravel, cobbles to 8" max
					46	2		30% cobbles and boulders to 24" max
13.5'	SP				100	3		SAND brown, dense
15'	SP				51	1		GRAVELLY SAND light brown, cobbles to 7" max
					66	2		25% cobbles and boulders to 36" max
20'	GP				48	3		SANDY GRAVEL light brown, dense, 25% cobbles and boulders to 24" max
24'	SP				68	4		GRAVELLY SAND light brown, dense, 25% cobbles and boulders to 24" max
28'								

-200 UN

L

GRAVELLY SAND tan, medium dense, cobbles and boulders to 18" max

SANDY GRAVEL light brown, dense, 40% cobbles and boulders to 18" max

20% cobbles and boulders to 30" max open gravel at 12"

SAND light brown, dense, 15% cobbles to 10" max.

SANDY GRAVEL light brown, dense, 20% cobbles to 10" max

SAND light brown, dense

SANDY GRAVEL light brown, dense, 40% cobbles and boulders to 48" max

For location of TT-1 See Plate D-27

TT-1D

Depth	LOG	MC	LL	PI	-4	-200	UN	Description
0'		V	I	S	U A	L		
5'	GP				49	2		SANDY GRAVEL light brown, dense, 70% cobbles and boulders to 30" max
8'	SP				95	2		SAND light brown, dense
9.5'	GP				32	1		SANDY GRAVEL light brown, dense 40% cobbles and boulders to 18" max 1-36" boulder
12'	SP				75	1		GRAVELLY SAND light brown, dense
13'	GP				37	1		SAND GRAVEL light brown, dense, 40% cobbles and boulders to 14" max
15'	SP				77	1		GRAVELLY SAND light brown, dense gravel to 2" max
17.5'	GP				20	1		SANDY GRAVEL light brown, dense, cobbles to 7" max
					28	1		
					36	4		20% cobbles and boulders to 24" max
					38	2		
29'								

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY:

DRAWN BY: R L A

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE

SOILS LOGS TT-1

SUBMITTED BY:

APPROVED:

APPROVAL RECOMMENDED:

SPEC. NO. DACW09-____ B-____

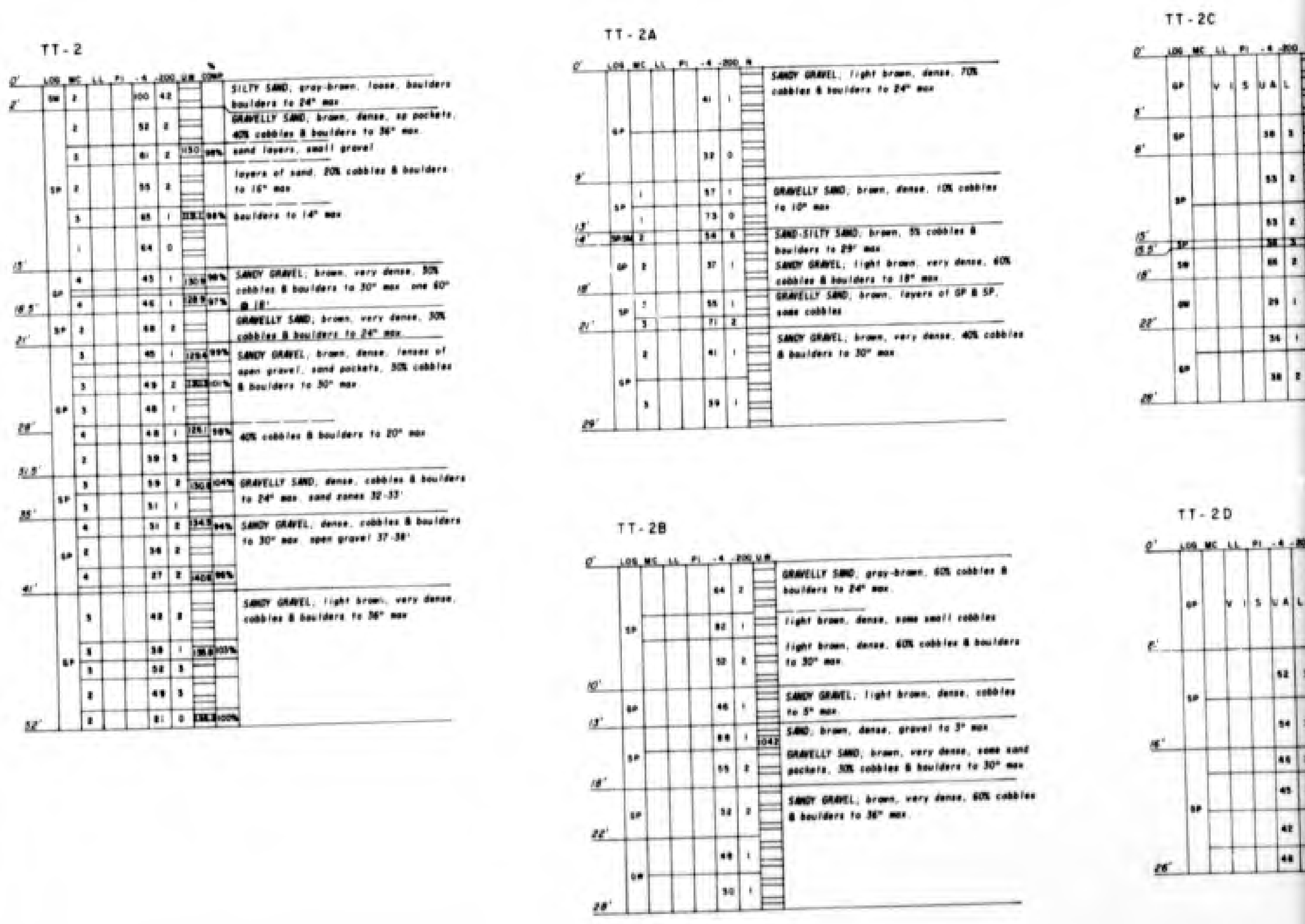
DISTRICT FILE NO.

SHEET OF SHEETS

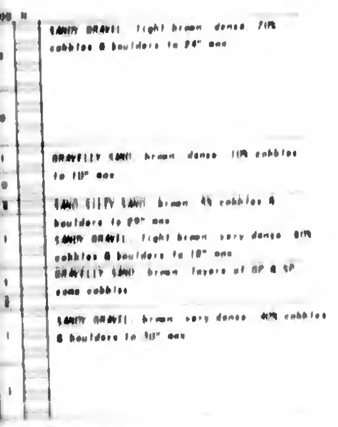
PLATE D-28

2

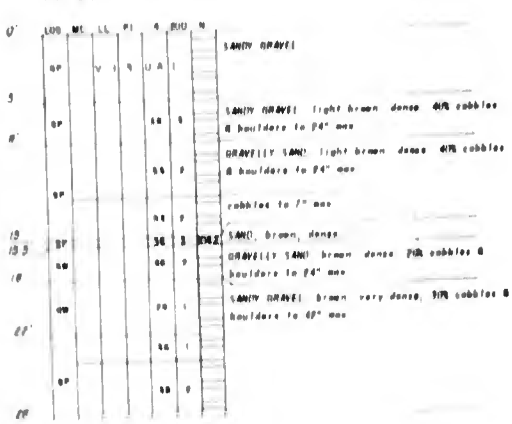
U.S. ARMY ENGINEER DISTRICT



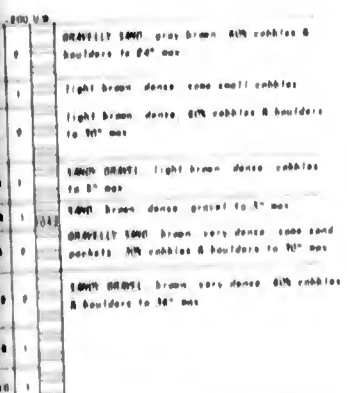
CORPS OF ENGINEERS



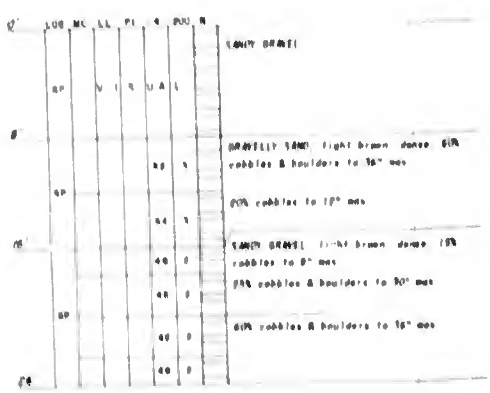
TT 2C



For location of TT-2 See Plate D-27



TT 2D



SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE

SOILS LOGS TT-2

DESIGNED BY:

DRAWN BY: R L A

CHECKED BY:

SUBMITTED BY:

APPROVED:

APPROVAL RECOMMENDED:

SPEC. NO. DACW09-

DISTRICT FILE NO.

SHEET OF SHEETS

PLATE D-29

2

TT-3

Depth	LOG	MC	LL	PI	-4	-200	UW	% COMP	Description
0'									
1'	SP	0			85	2			SAND, gray-brown, loose
2.5'	GP	1			29	1	[illegible]	96%	SANDY GRAVEL, brown, dense, cobbles & boulders to 18" max
4'	SP	1			66	1			GRAVELLY SAND, brown, dense, cobbles & boulders to 18" max
	GP	2			47	2			SANDY GRAVEL, light brown, dense, 20% cobbles & boulders to 18" max
7.5'		3			38	1	136.8	94%	
	SP	2			65	2			GRAVELLY SAND, light brown, dense, some sand zones, 15% cobbles to 10" max
11'		5			71	2	135.9	99%	
	GP	2			47	0			SANDY GRAVEL, brown, dense, thin layers of open gravel, cobbles to 12" max
		2			42	2			
		5			45	2	123.5	95%	40% cobbles & boulders to 30" max
		2			54	2			
		3			59	1	126.6	95%	sand layers
		5			38	0			30% cobbles & boulders to 30" max
		5			27	1	132.1	95%	
		2			55	1			30% cobbles & boulders to 36" max
30'		3			59	1	133.7	100%	
31'	SM	18			100	21			SILTY SAND, brown, medium dense
	GP	10			29	1			SANDY GRAVEL, light brown, very dense, 30% cobbles & boulders to 30" max, open gravel & cobbles to 12" max
		3			26	1	138.2	96%	38-39' very tight
39'		2			50	2			
	GP	2			28	1	142.6	100%	SANDY GRAVEL, light brown, very dense, 30% cobbles to 8" max
43'		4			49	2			
45'	SP-SM	5			78	5			SAND-SILTY SAND, brown, cobbles to 6" max
	GP	3			25	1	145.1	100%	SANDY GRAVEL, light brown, very dense, 30% cobbles & boulders to 20" max
		3			45	2			60% cobbles & boulders to 50" max
52'		4			56	1	[illegible]	98%	

TT-3A

Depth	LOG	MC	LL	PI	-4	-200	N	Description
0'								
3'	SP				56	2		GRAVELLY SAND, light brown, medium dense, 30% cobbles & boulders to 24" max
	GP				46	2		SANDY GRAVEL, 30% cobbles & boulders to 24" max
					44	0		70% cobbles & boulders to 48" max
					39	1		40% cobbles & boulders to 14" max
17'					42	1		30% cobbles & boulders to 24" max
19'	SW				67	1		GRAVELLY SAND, brown, dense, gravel to 2" max
23'	SP				53	2		fine grained, 20% cobbles to 12" max
24'	SM				92	34	111.4	SILTY SAND, brown, dense
27'	SP				54	2		GRAVELLY SAND, brown, dense, 20% cobbles & boulders to 16" max
30'	GP				41	2		SANDY GRAVEL, grey-brown, dense, cobbles to 10" max

TT-3B

Depth	LOG	MC	LL	PI	-4	-200	N	Description
0'								
	SP				52	5		GRAVELLY SAND; grey, loose to medium dense, 60% cobbles & boulders to 36" max
8'					54	5		
	GP				49	2		SANDY GRAVEL, brown, dense, 30% cobbles & boulders to 36" max
					50	1		15% cobbles & boulders to 16" max
17'					45	1		
17.5'	SP				97	2		SAND, brown, dense
21'	GP				39	3		SANDY GRAVEL; brown, medium dense, cobbles to 6" max
21.5'	SP-SM				75	11		SAND-SILTY SAND, brown, dense
23'	SP				60	1		GRAVELLY SAND, grey-brown, dense, cobbles to 5" max
24'	SP				100	3	116.7	SAND, brown, dense
26'	GP				49	1		SANDY GRAVEL; grey-brown, dense, 25% cobbles & boulders to 24" max

TT

TT-3A

LOG	MC	LL	PI	-4	-200	N	Description
SP				56	2		GRAVELLY SAND, light brown, medium dense, 30% cobbles & boulders to 24" max
GP				46	2		SANDY GRAVEL, 30% cobbles & boulders to 24" max
				44	0		70% cobbles & boulders to 48" max
				59	1		40% cobbles & boulders to 14" max
				42	1		30% cobbles & boulders to 24" max
SW				67	1		GRAVELLY SAND, brown, dense, gravel to 2" max
SP				53	2		Fine grained, 20% cobbles to 12" max
SM				92	34	11.4	SILTY SAND, brown, dense
SP				54	2		GRAVELLY SAND, brown, dense, 20% cobbles & boulders to 16" max
GP				41	2		SANDY GRAVEL, gray-brown, dense, cobbles to 10" max

TT-3C

Depth	LOG	MC	LL	PI	-4	-200	N	Description
0' – 7'	GP		V I	S	U A	L		SANDY GRAVEL
7' – 10'	SP				52	3		GRAVELLY SAND, light brown, medium dense, 30% cobbles & boulders to 30" max
10' – 12'	GW				27	2		SANDY GRAVEL, brown, open lenses, 10% cobbles & boulders to 18" max
12' – 14'	SW				79	2		GRAVELLY SAND, brown, dense, gravel to 2" max
14' – 18'	SP				55	3		GRAVELLY SAND, brown, dense, gravel & several cobbles to 10" max
18' – 21'	GW				33	2		SANDY GRAVEL, brown, very dense, 40% cobbles & boulders to 24" max, open gravel pockets
21' – 22'	SP				71	2		GRAVELLY SAND, brown, dense, gravel to 2" max
22' – 26'	GP				45	1		SANDY GRAVEL, brown, very dense, 30% cobbles & boulders to 30" max

TT-3B

LOG	MC	LL	PI	-4	-200	N	Description
SP				52	3		GRAVELLY SAND, gray, loose to medium dense, 60% cobbles & boulders to 36" max
				54	3		
GP				49	2		SANDY GRAVEL, brown, dense, 30% cobbles & boulders to 36" max
				50	1		15% cobbles & boulders to 16" max
				45	1		
SP				97	2		SAND, brown, dense
GP				59	3		SANDY GRAVEL, brown, medium dense, cobbles to 6" max
SM				75	11		SAND-SILTY SAND, brown, dense
SP				60	1		GRAVELLY SAND, gray-brown, dense, cobbles to 5" max
SP				100	3	16.2	SAND, brown, dense
GP				49	1		SANDY GRAVEL, gray-brown, dense, 25% cobbles & boulders to 24" max

For location of TT-3 See Plate D-27

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY:

DRAWN BY: R L A

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE 1 GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE

SOILS LOGS TT-3

SUBMITTED BY: CHIEF, ... BRANCH

APPROVAL RECOMMENDED: CHIEF, ENGINEERING DIVISION

APPROVED: COLONEL, CE, DISTRICT ENGINEER

SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET OF SHEETS

U. S. ARMY ENGINEER DISTRICT

TEST TRENCH #1

DEPTH	PERCENT PASSING: GRAVEL (IN)					SAND (#)					FINES	FIELD	
	MAX	3	1½	¾	⅜	4	10	16	40	100	200	MC (%)	DENS (PC…
5.0'	18"	99	96	95	94	93	88[A]	67	8[B]	3	2	2.2	107 (cut off)
9.0'	16"	65	60	55	52	46	42[A]	30	4[B]	2	1	1.7	138 (cut off)
14.0'	30"	55	51	47	44	40	35[A]	24	4[B]	2	2	2.8	131 (cut off)
18.0'		92	75	55	43	37	31	25	8	3	2	3.8	13 (cut off)
24.0'	36"	43	35	30	28	26	24	20	10	4	3	5.0	14 (cut off)
29.0'	30"	91	86	83	80	77	61	43	14	5	3	5.5	11 (cut off)
33.0'	C	80	76	70	66	62	51	39	14	5	3	3.8	12 (cut off)
37.0'		57	53	50	48	44	37	29	10	3	2	3.3	13 (cut off)
43.5		71	61	53	47	42	31	22	9	5	3	6.1	13 (cut off)
49.5'		70	59	53	47	41	31	24	14	7	4	6.2	13 (cut off)

*DWR TEST METHOD: PERFORMED ON -6" MATERIAL, LARGE MOLD - 2'-3½" D…
15 MIN AT 6000 CPS WITH A 2 PSI SURCHARGE

A - #8 SIEVE USED

B - #30 SIEVE USED

C - COULD NOT REMOVE 20" BOULDER

TRENCH #1

PASSING → SAND (#)					FINES	FIELD		DWR DENSITY METHOD *	
4	10	16	40	100	200	MC (%)	DENSITY (PCF)	MAX DENSITY (PCF)	REL COMP (%)
93	88[A]	67	8[B]	3	2	2.2	107.8	102.9	104.8
46	42[A]	30	4[B]	2	1	1.7	138.4	136.5	101.4
40	35[A]	24	4[B]	2	2	2.8	131.9	135.7	97.2
37	31	25	8	3	2	3.8	138.0	140.4	98.3
26	24	20	10	4	3	5.0	142.1	142.4	99.8
77	61	43	14	5	3	5.5	118.7	133.9	88.6
62	51	39	14	5	3	3.8	128.5	128.5	100.0
44	37	29	10	3	2	3.3	133.3	135.3	98.5
42	31	22	9	5	3	6.1	132.8	131.4	101.0
41	31	24	14	7	4	6.2	132.9	134.0	99.1

MATERIAL, LARGE MOLD - 2'-3½" DIAMETER, VIBRATED FOR
A 2 PSI SURCHARGE

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
FOUNDATION EVALUATION-
TEST RESULTS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

TEST TRENCH #2

DEPTH	PERCENT PASSING — GRAVEL (IN)					PERCENT PASSING — SAND (#)					FINES	FIELD	
	MAX	3	1½	¾	⅜	4	10	16	40	100	200	MC (%)	DENSI… (PCF)
5.0'	36"	90	87	84	83	81	74	52	16	4	2	2.5	113.
10.0	16"	94	89	83	74	65	55 [A]	36	13	1	1	2.7	119.
15.0'	30"	86	67	57	50	43	37	29	10	3	1	4.0	130
17.0'	30"	80	70	62	54	46	38	29	9	2	1	3.6	128
21.0'	24"	93	70	59	51	45	41	29	10	2	1	3.1	129
23.5'	30"	89	80	69	59	49	35	25	8	2	2	2.7	130
27.5'	20"	71	61	56	52	48	41	29	4	1	1	4.2	126
31.5'	24"	96	85	75	67	59	50	39	13	3	2	3.1	130
35.0'	30"	60	51	43	37	31	25	21	8	3	2	3.6	134
39.0'	30"	67	52	40	33	27	22	17	7	3	2	3.6	14
45.0'	14"	65	55	47	42	38	31	25	8	2	1	2.9	13
50.0'	36"	47	36	29	25	21	17	12	3	1	0	1.7	14

*DWR TEST METHOD: PERFORMED ON -6" MATERIAL, LARGE MOLD-DIAME… 15 MIN AT 6000 CPS WITH A 2 PSI SURCHARGE

A - #8 SIEVE USED

TRENCH #2

PASSING						FIELD		OWR DENSITY METHOD *	
SAND (#)					FINES				
	10	16	40	100	200	MC (%)	DENSITY (PCF)	MAX DENSITY (PCF)	REL COMP (%)
1	74	52	16	4	2	2.5	113.0	114.8	98.4
5	55^	36	13	1	1	2.7	119.1	121.6	97.9
3	37	29	10	3	1	4.0	130.7	133.8	98.0
6	38	29	9	2	1	3.6	128.9	133.3	96.7
5	41	29	10	2	1	3.1	129.4	131.0	98.7
9	35	25	8	2	2	2.7	130.9	130.1	100.6
8	41	29	4	1	1	4.2	126.1	128.9	97.8
59	50	39	13	3	2	3.1	130.8	125.9	103.9
31	25	21	8	3	2	3.6	134.3	142.8	94.0
27	22	17	7	3	2	3.6	140.6	146.1	96.3
38	31	25	8	2	1	2.9	135.8	131.9	102.9
21	17	12	3	1	0	1.7	146.5	133.9	109.4

MATERIAL, LARGE MOLD - DIAMETER 2'-3½"; VIBRATED FOR
... A 2 PSI SURCHARGE

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
FOUNDATION EVALUATION-
TEST RESULTS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

TEST TRENCH #3

DEPTH	PERCENT PASSING — GRAVEL (IN)					SAND (#)					FINES	FIELD	
	MAX	3	1½	¾	⅜	4	10	16	40	100	200	MC (%)	DENSITY (PCF)
1.0'	18"	63	48	40	34	29	24	19	6	1	1	1.3	136.
6.0'	18"	59	49	43	38	34	27	20	/	7	1	3.0	136.
9.5'	10"	84	80	76	74	71	66	56	17	3	2	4.7	115.
15.5'	12"	64	58	53	49	45	42	38	21	6	2	4.5	123.
19.5'	30'	70	58	50	44	39	38	34	12	3	1	3.3	126.
24.5'	30"	68	54	43	34	27	20	16	6	1	1	3.1	132.
28.5'	36"	87	77	62	48	39	30	25	9	2	1	3.0	133.
35.0'	30"	52	43	36	32	26	22	18	7	2	1	3.1	138.
39.0'	12"	82	60	44	34	28	22	19	9	2	1	2.3	142.
46.0'	20"	61	48	37	30	25	21	17	6	1	1	3.3	143
50.5'	30"	70	56	47	41	36	31	26	[illegible]	2	1	4.2	134.

* DWR TEST METHOD: PERFORMED ON -6" MATERIAL, LARGE MOLD-DIA
15 MIN AT 6000 CPS WITH A 2 PSI SURCHARGE

RENCH #-3

	PASSING → SAND (#)				FINES	FIELD		DWR DENSITY METHOD *	
	10	16	40	100	200	MC (%)	DENSITY (PCF)	MAX DENSITY (PCF)	REL. COMP. (%)
	24	19	6	1	1	1.3	136.4	141.4	96.5
	27	20	/	7	1	3.0	136.5	144.8	94.3
	66	56	17	3	2	4.7	115.9	117.4	98.7
	42	38	21	6	2	4.5	123.3	133.2	93.4
1	38	34	12	3	1	3.3	126.6	136.7	92.6
1	20	16	6	1	1	3.1	132.1	141.7	93.2
9	30	25	9	2	1	3.0	133.7	134.0	99.8
6	22	18	7	2	1	3.1	138.2	144.5	95.6
8	22	19	9	2	1	2.3	142.8	143.3	99.7
5	21	17	6	1	1	3.3	143.1	136.7	104.7
6	31	26	8	2	1	4.2	134.6	137.9	97.6

MATERIAL, LARGE MOLD-DIAMETER 2'-3½", VIBRATED FOR
...I A 2 PSI SURCHARGE

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
FOUNDATION EVALUATION-
TEST RESULTS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

SAND LENS
TEST TRENCH #1

DEPTH	PERCENT PASSING — GRAVEL (IN)			SAND (#)					FINES	FIELD		DATA RELATI…
	1½	3/4	3/8	4	10	16	40	100	200	MC (%)	DENSITY (PCF)	MAX (PCF)
3.7				99	99	98	81	35	16	0.8	91.8	110.2
6.6	98	91	79	67	53	35	9	2	1	1.6	121.4	125.2
6.6			99	98	94	80	26	5	2	2.4	105.6	113.0
7.0					97	82	26	4	2	3.2	98.9	111.6
7.6		99	98	97	94	79	24	4	2	2.0	96.8	112.7
8.2					97	82	26	4	2	3.0	98.9	111.6
8.6		99	98	97	94	79	24	4	2	2.4	111.1	114.3
9.3					97	82	26	4	2	2.8	98.5	111.6
11.1					97	82	26	4	2	3.0	102.7	111.6
14.7			99	98	94	80	27	4	2	4.3	101.7	110.2
16.6		99	96	93	87	74	25	5	2	1.3	112.2	116.4
19.7				99	97	87	31	7	4	2.3	112.5	110.5
20.6				99	99	92	35	7	3	3.8	108.4	106.6

LENS
TRENCH #1

...ING #)		FINES	FIELD		DATA RELATIVE	FOR DENSITY	
40	100	200	MC (%)	DENSITY (PCF)	MAX (PCF)	MIN (PCF)	REL. (%)
81	35	16	0.8	91.8	110.2	86.5	28
9	2	1	1.6	121.4	125.2	107.9	81
26	5	2	2.4	105.6	113.0	94.3	69
26	4	2	3.2	98.9	111.6	92.6	39
24	4	2	2.0	96.8	112.7	93.0	24
26	4	2	3.0	98.9	111.6	92.6	37
24	4	2	2.4	111.1	114.3	94.9	86
26	4	2	2.8	98.5	111.6	92.6	37
26	4	2	3.0	102.7	111.6	92.6	58
27	4	2	4.3	101.7	110.2	89.3	63
25	5	2	1.3	112.2	116.4	98.5	81
31	7	4	2.3	112.5	110.5	89.5	108
35	7	3	3.8	108.4	106.6	85.8	107

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
FOUNDATION EVALUATION-
TEST RESULTS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

SAND LENS
TEST TRENCH #2

DEPTH	PERCENT PASSING → GRAVEL (IN)			SAND (#)					FINES	FIELD		D REL
	1½	3/4	3/8	4	10	16	40	100	200	MC (%)	DENSITY (PCF)	MAX (PC
3.0			99	99	97	93	77	43	23	1.2	97.5	←
3.6					98	96	79	42	26	5.0	88.8	←
4.6			99	98	94	82	21	3	1	2.1	102.8	110
6.6	94	82	70	59	46	36	13	4	2	1.5	120.7	13
7.0			99	99	97	93	47	6	2	2.4	99.8	11
7.7			99	99	97	93	47	6	2	2.4	98.1	11
15.0					99	95	41	8	4	3.5	104.2	10
16.6	98	95	88	80	67	51	16	4	2	2.2	117.8	12
16.6	99	94	90	84	68	53	16	4	2	2.3	119.9	1
23.6		96	91	85	75	64	25	5	2	3.8	105.6	12
25.0				99	98	94	38	3	1	4.8	99.4	1
25.6					98	94	45	7	2	3.9	104.2	11

-ND LENS
T TRENCH #2

PASSING →				FINES	FIELD		DATA FOR RELATIVE DENSITY		
D (#)									
0	16	40	100	200	MC (%)	DENSITY (PCF)	MAX (PCF)	MIN (PCF)	REL (%)
7	93	77	43	23	1.2	97.5	← NONE →		
8	96	79	42	26	5.0	88.8	← NONE →		
94	82	21	3	1	2.1	102.8	110.7	93.1	60
46	36	13	4	2	1.5	120.7	132.9	111.7	50
97	93	47	6	2	2.4	99.8	112.0	92.5	43
97	93	47	6	2	2.4	98.1	112.0	92.5	35
99	95	41	8	4	3.5	104.2	108.2	88.2	85
67	51	16	4	2	2.2	117.8	122.7	103.0	78
68	53	16	4	2	2.3	119.9	121.9	103.4	91
75	64	25	5	2	3.8	105.6	120.1	101.7	25
98	94	38	3	1	4.8	99.4	106.9	87.7	67
98	94	45	7	2	3.9	104.2	111.8	92.3	69

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE
FOUNDATION EVALUATION-
TEST RESULTS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

U. S. ARMY ENGINEER DISTRICT

SAND LENS
TEST TRENCH #3

DEPTH	← PERCENT PASSING → GRAVEL (IN)			SAND (#)					FINES	FIELD		DATA RELATI
	1½	¾	⅜	4	10	16	40	100	200	MC (%)	DENSITY (PCF)	MAX (PCF)
7.6			99	97	91	73	12	1	0	1.6	102.0	107.7
7.6			98	97	86	65	12	1	0	1.7	104.0	113.4
10.6		99	98	96	91	84	52	22	11	8.9	103.5	117.5
12.6					98	93	49	8	3	4.2	96.5	110.5
12.6			99	99	98	96	47	5	1	3.9	98.7	106.5
13.6		99	99	98	95	86	32	5	2	3.2	100.4	112.8
16.6	97	92	83	73	66	58	30	8	3	2.3	118.2	129.7
18.6		97	91	86	79	69	23	3	1	2.9	110.2	119.9
19.6				99	97	92	56	9	2	4.5	98.5	107.7
22.6	99	94	89	81	70	55	14	1	0	1.2	112.2	121.5
22.7					98	94	56	13	4	2.4	99.8	113.1
23.5	99	96	94	91	88	69	21	2	1	2.2	116.2	115.3
23.7					98	94	56	13	4	3.4	103.6	113.1

LENS
TRENCH #3

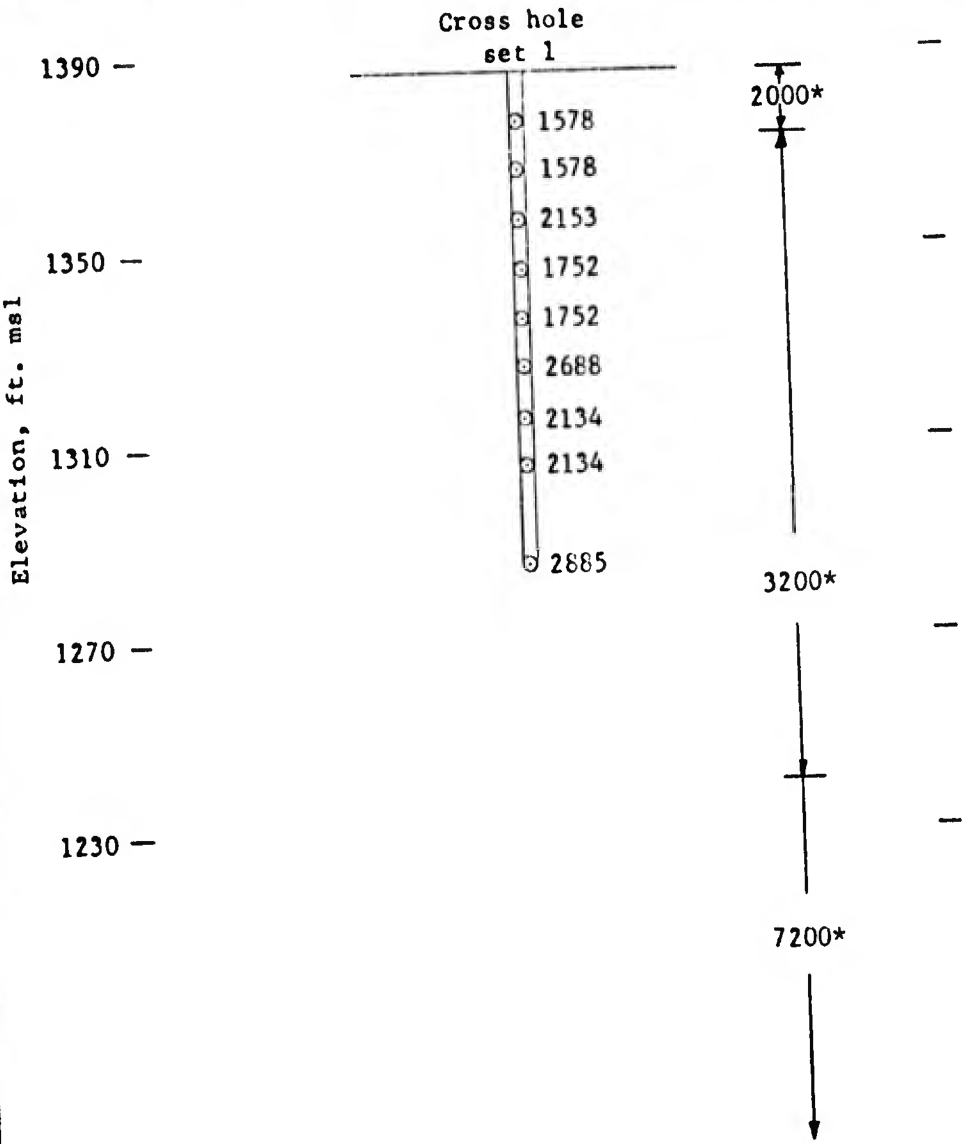
…NG (…#)		FINES	FIELD		DATA FOR RELATIVE DENSITY		
40	100	200	MC (%)	DENSITY (PCF)	MAX (PCF)	MIN (PCF)	REL (%)
12	1	0	1.6	102.0	107.7	93.9	66
12	1	0	1.7	104.0	113.4	97.1	48
52	22	11	8.9	103.5	117.5	95.2	44
49	8	3	4.2	96.5	110.5	90.1	37
47	5	1	3.9	98.7	106.5	87.3	66
32	5	2	3.2	100.4	112.8	93.7	40
30	8	3	2.3	118.2	129.7	110.5	47
23	3	1	2.9	110.2	119.9	100.2	58
56	9	2	4.5	98.5	107.7	86.1	65
14	1	0	1.2	112.2	121.5	103.9	55
56	13	4	2.4	99.8	113.1	92.1	42
21	2	1	2.2	116.2	115.3	97.8	104
56	13	4	3.4	103.6	113.1	92.1	61

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAMSITE FOUNDATION EVALUATION-TEST RESULTS

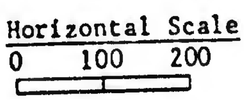
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

2



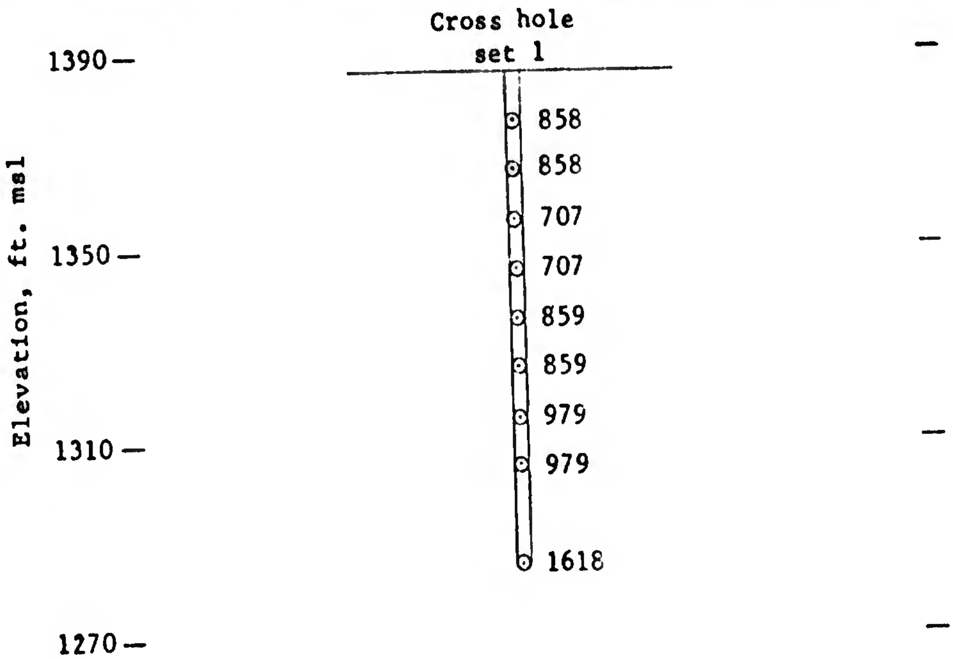
Legend:
⊙ Location of geophones

Note: All velocities are fps. No asterisk denotes cross hole velocities.
* Denotes velocities from surface refraction seismic tests.



CROSS HOLE GEOPHONE PLACEMENT
AND TRUE P-WAVE VELOCITIES
Mentone Damsite, Site G-1

PLATE D-37

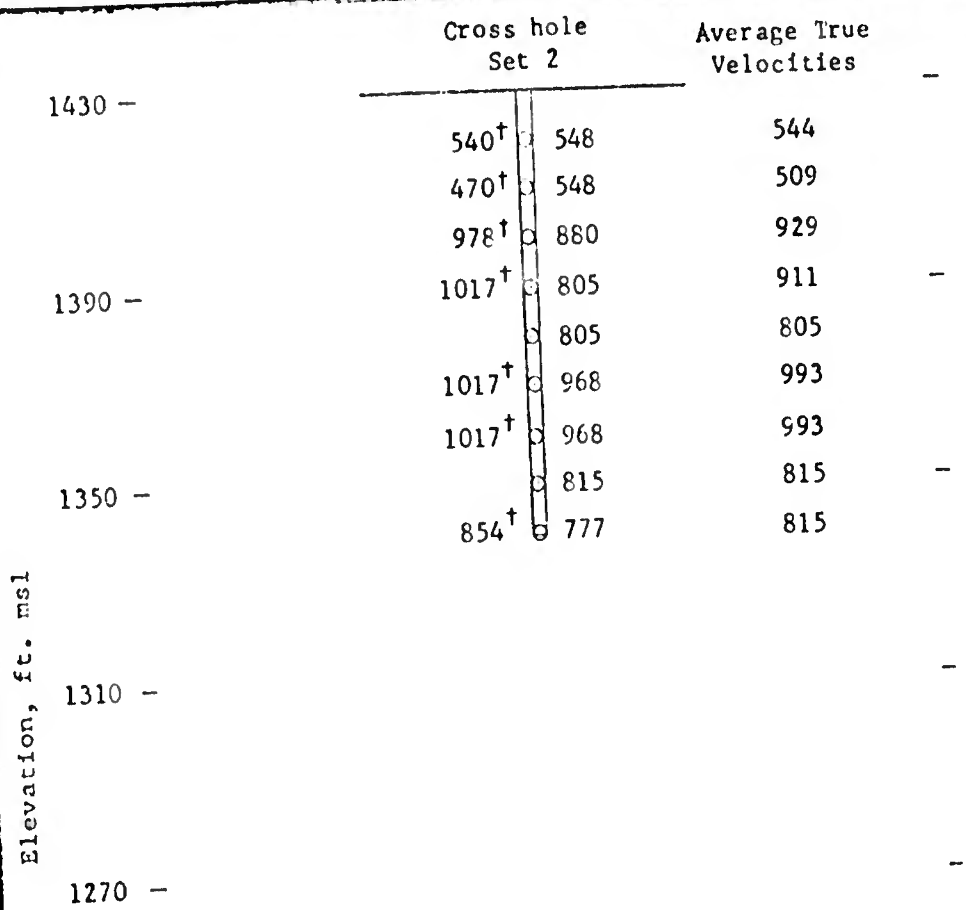


Legend:
⊙ Location of geophones

Note: All velocities are fps.

CROSS HOLE GEOPHONE PLACEMENT
AND TRUE S-WAVE VELOCITIES
Mentone Damsite, Site G-1

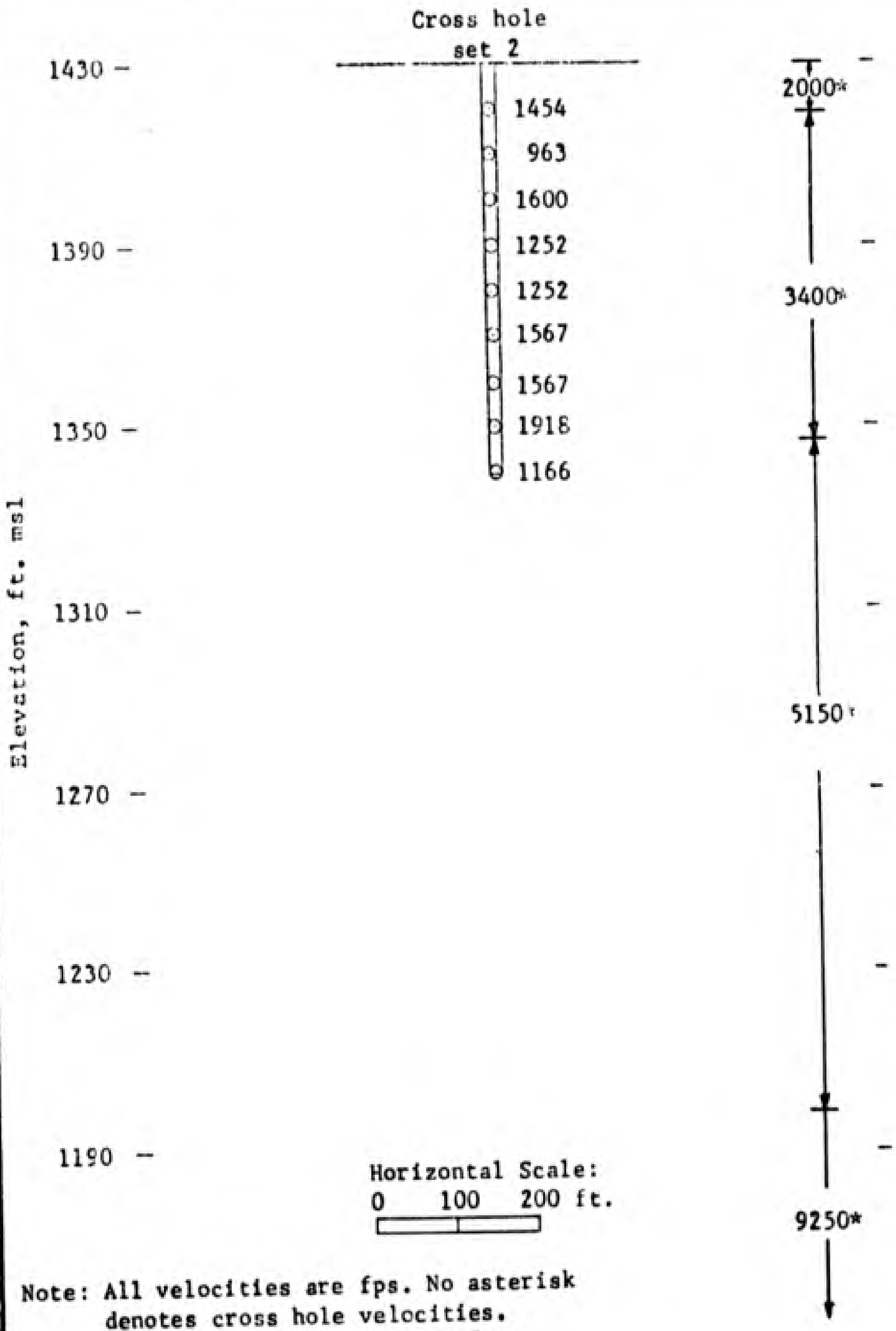
PLATE D-38



Legend:
o Location of geophones

Note: All velocities are fps. No cross denotes cross hole velocities.
† Denotes velocities from borehole vibratory source.

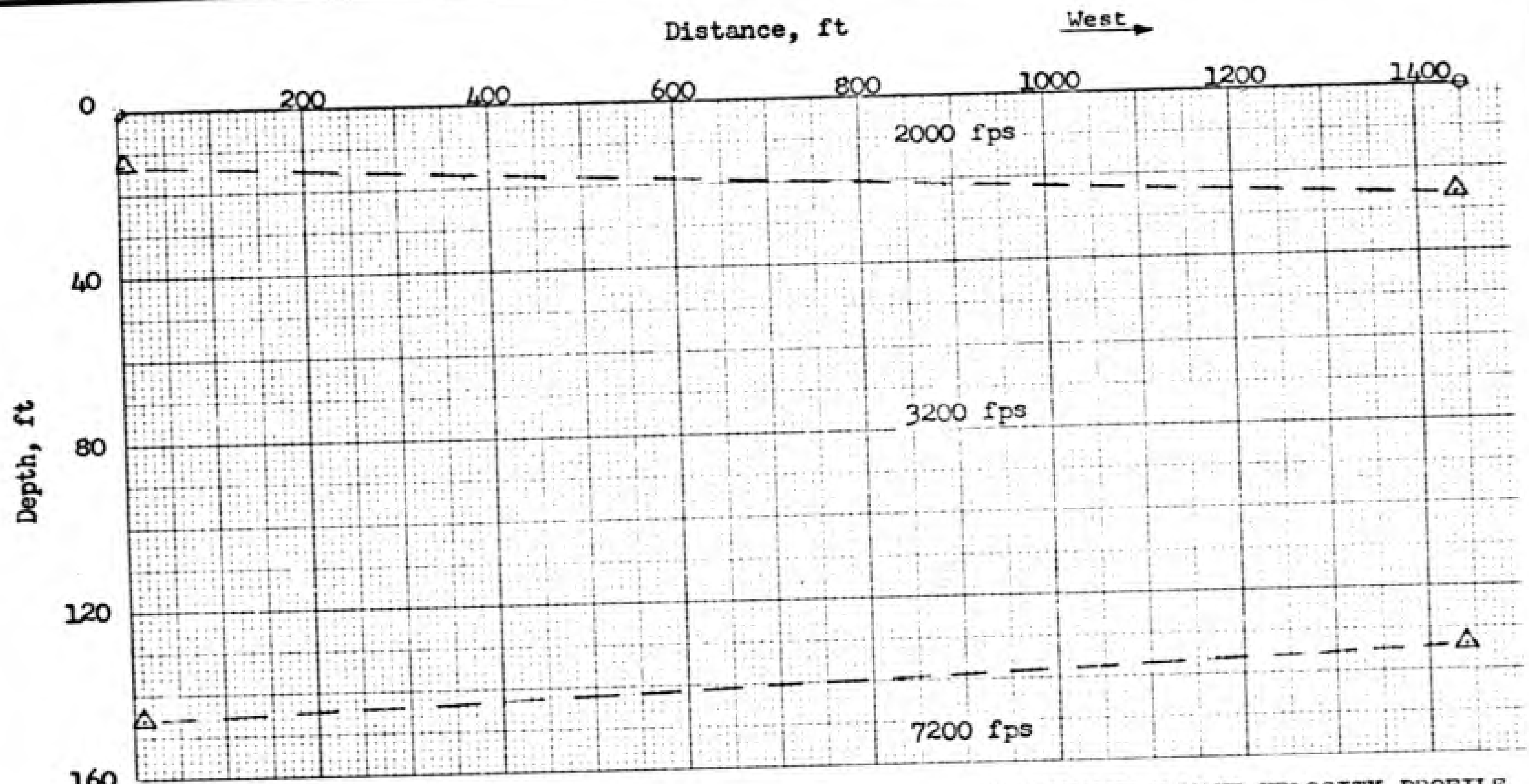
CROSS HOLE GEOPHONE PLACEMENT AND TRUE S-WAVE VELOCITIES
Mentone Damsite, Site G-2



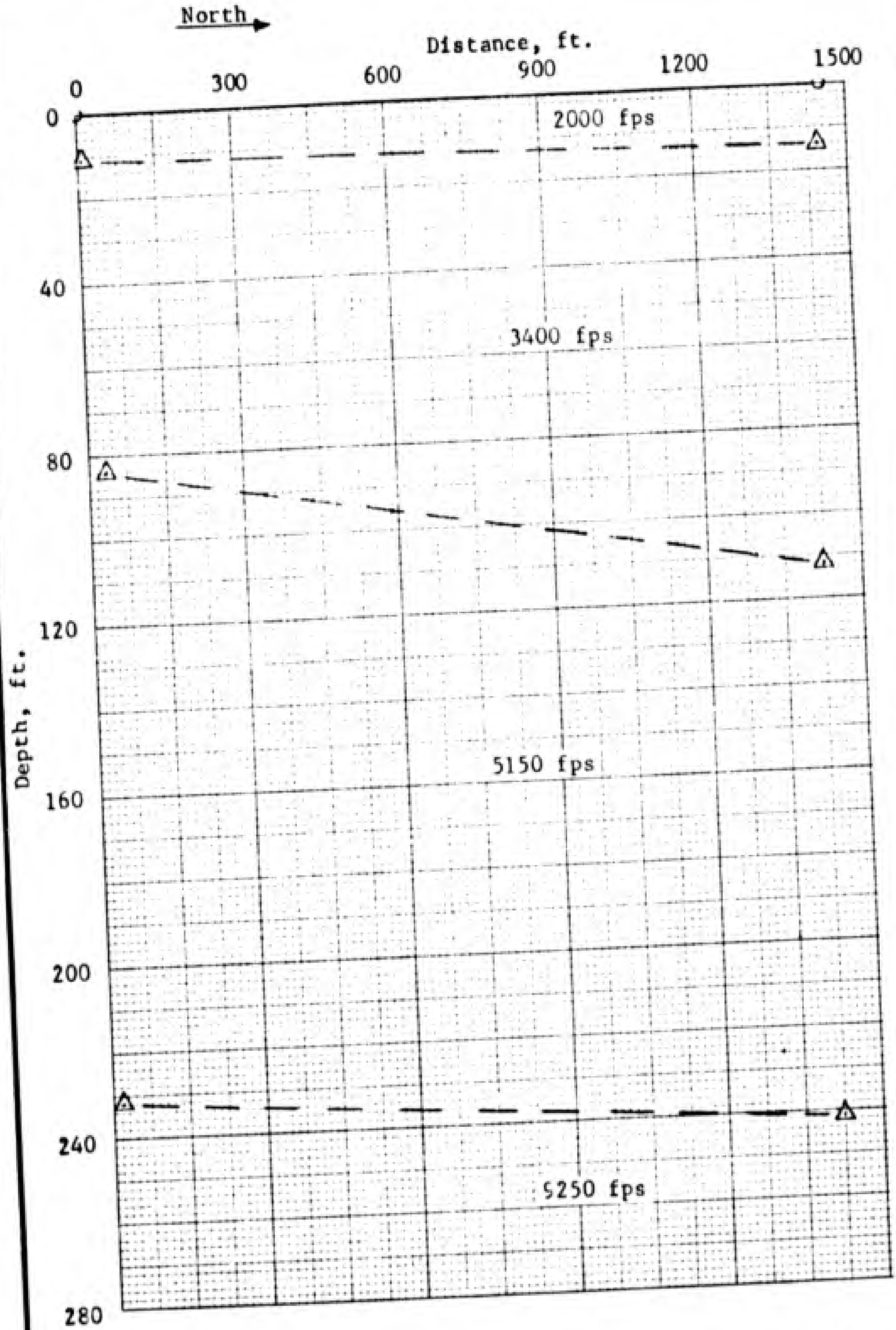
Note: All velocities are fps. No asterisk denotes cross hole velocities.
* Denotes velocities from surface refraction seismic tests.

Legend:
⊙ Location of geophones

CROSS HOLE GEOPHONE PLACEMENT
AND TRUE P-WAVE VELOCITIES
Mentone Damsite, Site G-2

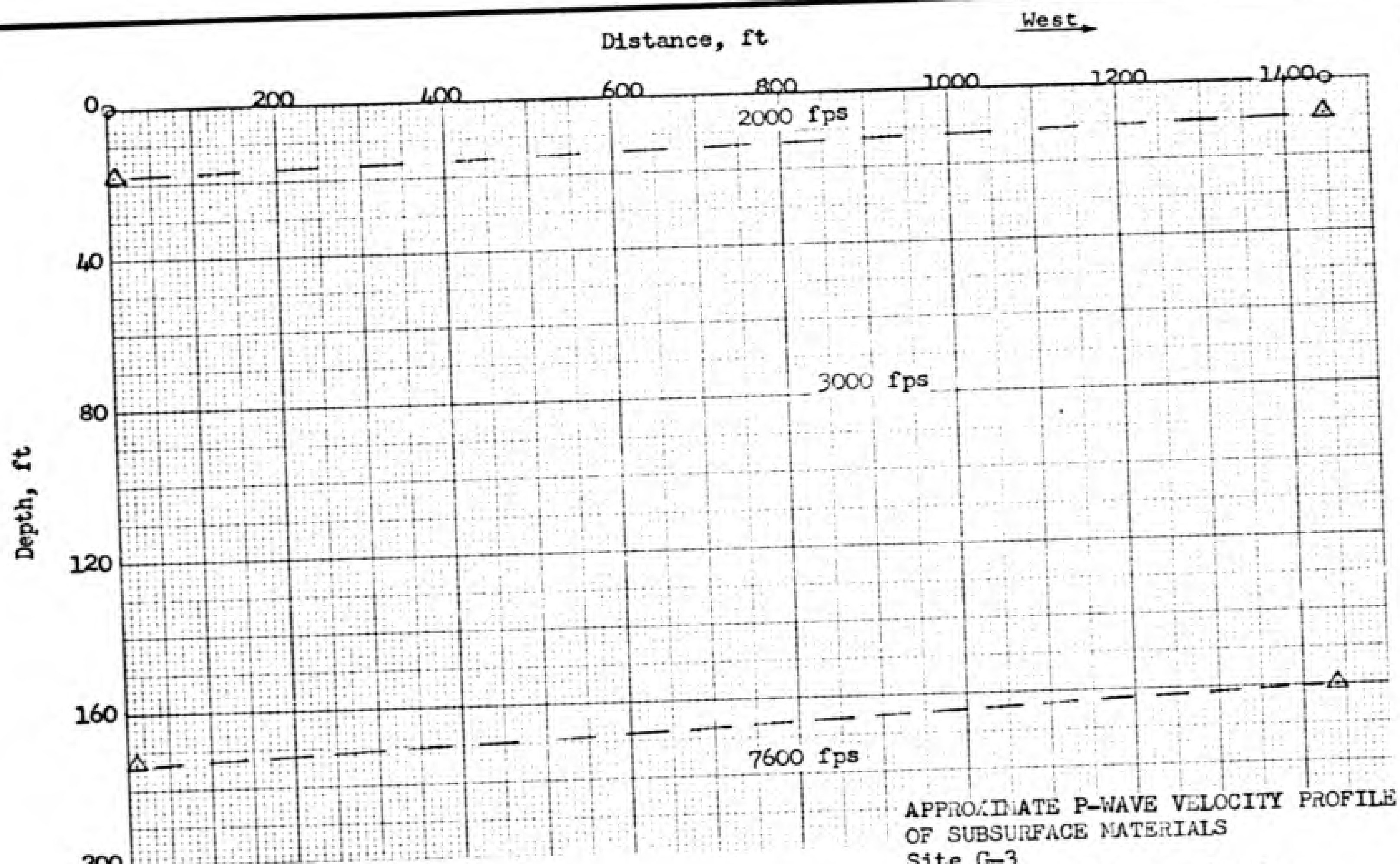


APPROXIMATE P-WAVE VELOCITY PROFILE
OF SUBSURFACE MATERIALS
Site G-1
Mentone Damsite, California



APPROXIMATE P-WAVE VELOCITY PROFILE
OF SUBSURFACE MATERIALS
Site G-2
Mentone Damsite, California

PLATE D-42



APPROXIMATE P-WAVE VELOCITY PROFILE
OF SUBSURFACE MATERIALS
Site G-3
Mentone Damsite, California

POTENTIAL BORROW AREAS AND LOCATIONS

	LOCATION: BORROW SITE	DEPTH INCHES	USCS	200 SIEVE	LIQUID LIMIT	PLASTIC INDEX	PERMEABILITY	DISTANCE AZIMUTH	ROAD DIST
1	EL CASCO QUADRANGLE W 1/2 SEC. 16, SEC. 17, SEC. 20, SEC. 21, T.3S., R.2W., GRANT BOUNDRY & ALESSANDRO SOIL SURVEY MAP #46	0 - 60 ALSO 0 - 60	SM CH, MH	40 - 50 90 - 100	- 50 - 70	- 25 - 40	2.0 - 6.3 0.06 - 0.2	13.8 MI	EST 16.4 MI
2	YUCAIPA QUADRANGLE NE1/4 SEC. 17, W1/2 SEC. 16, NW1/4 SEC. 15, W1/2 SEC. 14, T. 1S., R.2W. MILL CREEK & NEWPORT AVE. SOIL SURVEY MAP SHEET #10	0 - 23 23 - 60	SM SC, ML CL	35 - 45 45 - 55	- 25 - 35	- 5 - 15	2.0 - 6.3 0.2 - 0.63	5.0 MI	EST 6.2 MI
3	YUCAIPA QUADRANGLE MID. SEC. 29, S1/2 SEC. 32, T. 1S., R.2W. CITRUS AVE. & CRAFTON AVE. SOIL SURVEY MAP SHEET #16	0 - 23 22 - 60	SM SC, ML CL	35 - 45 45 - 55	- 25 - 35	- 5 - 15	2.0 - 6.3 0.2 - 0.63	5.2 MI	EST. 10.6 MI
4	PRADO DAM PRADO BASIN T.2S., R.7W., CORONA FWY & BOUND SOIL SURVEY MAP SHEET #11 & 12	0 - 27 27 - 36	ML, CL ML, CL	10 - 80 60 - 75	20 - 35 15 - 25	5 - 10 5 - 10	0.63 - 2.0 < 0.06	30.0 MI	EST. 34 MI
5	REDLANDS QUADRANGLE NW1/4, SEC 31, NE1/4 SEC. 36, SOIL SURVEY MAP SHEET #9	0 - 23 23 - 60	SM SC, ML, CL	35 - 45 45 - 55	- 25 - 35	- 5 - 15	2.0 - 6.3 0.2 - 0.63	4.6 MI	EST. 10.1 MI
6	RIVERSIDE EAST QUADRANGLE W1/2, SEC. 36, T.2S., R.4W. IRONWOOD AVE. & PIGEON PASS SOIL SURVEY MAP #28	0 - 23 23 - 60 60 - 74	SM SC, ML, CL SC	35 - 45 45 - 50 35 - 45	- 25 - 35 20 - 30	- 5 - 15 5 - 15	2.0 - 6.3 .2 - .63 2.0 - 6.3	11.4 MI	EST. 13.2 MI
7	SUNNYMEAD QUADRANGLE E1/2 SEC. 12, T.3S., R.4W. S1/2 SEC. 7, T. 3S., R.3W. ALESSANDRO BLVD & INDIAN ST. SOIL SURVEY MAP #44	0 - 10 10 - 28 28 <	SM SC, CL HARDPAN	35 - 50 45 - 60	- 25 - 40	- 10 - 20	2.0 - 6.3 0.2 - 0.63	12.3 MI	EST. 21.4 MI
8	SUNNYMEAD QUADRANGLE NE1/4, SEC. 13, T.3S., R.3W. ALESSANDRO BLVD &PERRIS BLVD SOIL SURVEY MAP #44	0 - 10 10 - 28 < 28	SM SC, CL HARDPAN	35 - 50 45 - 60	- 25 - 40	- 10 - 20	2.0 - 6.3 0.2 - 0.63	13.4 MI	EST. 26.1 MI
9	RIVERSIDE EAST QUADRANGLE N1/2, SEC 14 & NE1/4, SEC. 13, T.3S., R.4W., CACTUS AVE. & ESCONDIDO FWY SOIL SURVEY MAP #43 & 44	0 - 10 10 - 28 28 <	SM SC, CL HARDPAN	35 - 50 45 - 60 -	- 25 - 40 -	- 10 - 20 -	2.0 - 6.3 0.2 - 6.3 -	13.8 MI	EST. 35.5
10	RIVERSIDE EAST QUADRANGLE SW1/4, SEC. 12, T.3S., R.4W., ALESSANDRO BLVD & FREDRICK ST. SOIL SURVEY MAP #44	0 - 10 10 - 28 28 <	SM SC, CL HARDPAN	35 - 50 45 - 60 -	- 25 - 40 -	- 10 - 20 -	2.0 - 6.3 0.2 - 6.3 -	13.3 MI	EST. 25.4
11	RIVERSIDE EAST QUADRANGLE NE1/4, SEC 11, NW1/4 SEC.12, SW1/4, SEC. 1, T.3S., R.4W., FREDRICK ST. & COTTONWOOD SOIL SURVEY MAP #27 & 28	1 - 10 10 - 28 28 <	SM SC, CL HARDPAN	35 - 50 45 - 60 -	- 25 - 40 -	- 10 - 20 -	2.0 - 6.3 0.2 - 6.3 -	13.1 MI	EST. 25.2
12	RIVERSIDE EAST QUADRANGLE SE1/4, SEC.3, T.3S., R.4W., HWY 60 & ESCONDIDO FWY SOIL SURVEY MAP #27	0 - 10 10 - 28 28 <	SM SC, CL HARDPAN	35 - 50 45 - 60 -	- 25 - 40 -	- 10 - 20 -	2.0 - 6.3 0.2 - 6.3 -	12.6 MI	EST. 20.7
13	RIVERSIDE EAST QUADRANGLE N1/2, SEC 2, T.3S., R.4W., FWY 60 & DAY ST SOIL SURVEY MAP #2	0 - 10 10 - 28 28 <	SM SC, CL HARDPAN	35 - 50 45 - 60 -	- 25 - 40 -	- 10 - 20 -	2.0 - 6.3 0.2 - 6.3 -	12.3 MI	EST 21.1
14	RIVERSIDE WEST QUADRANGLE SE1/4, SEC.24, T.2S., R.6W., NE1/4, SEC. #1, T.2S., R.6W., LIMONITE AVE. SOIL SURVEY MAP #10	0 - 19 19 - 26 26 - 37 37 - 63	SM SC, CL HARDPAN CL	35 - 50 70 - 90 - 65 - 75	- 55 - 75 - 20 - 30	- 20 - 45 - 10 - 20	2.0 - 6.3 < .06 - 0.2 - .63	18.2 MI	EST 23.2
15	RIVERSIDE WEST QUADRANGLE T.2S., R.6W., SOUTH OF FLABOB AIRPORT. SOIL SURVEY MAP #11	0 - 60	SM	40 - 50	-	-	2.0 - 6.3	16.3 MI	EST. 2
16	RIVERSIDE WEST QUADRANGLE SE1/4, SEC.23, T.2S., R.6W., LIMONITE AVE & VAN BUREN SOIL SURVEY MAP #10	0 - 19 19 - 26 26 - 37 37 - 62	SM CH, MH HARDPAN CL	35 - 50 70 - 90 - 65 - 75	- 55 - 75 - 20 - 30	- 20 - 45 - 10 - 20	2.0 - 6.3 < .06 - .02 - .63	16.2 MI	E 26.
17	RIVERSIDE WEST QUADRANGLE SE1/4, SEC 14, T.2S., R.6W., JURUPA RD. & PEDLEY RD. SOIL SURVEY MAP #10	0 - 19 19 - 26 26 - 37 37 - 62	SM CH, MH HARDPAN CL	35 - 50 70 - 90 - 65 - 75	- 55 - 75 - 20 - 30	- 20 - 45 - 10 - 20	2.0 - 6.3 < .06 - 0.2 - .63	19.0 MI	E 25.
18	EL CASCO QUADRANGLE MIDDLE, SEC. 31, T.2S., R.1W., FWY 10 & CANYON RD SOIL SURVEY MAP #31	0 - 23 23 - 60 60 - 74	SM SC, ML, CL SC	35 - 45 45 - 55 35 - 45	- 25 - 35 20 - 30	- 5 - 15 5 - 15	2.0 - 6.3 0.2 - 0.63 2.0 - 6.3	13.1 MI	E 17

POTENTIAL BORROW AREAS AND LOCATIONS

EVE	LIQUID LIMIT	PLASTIC INDEX	PERMEABILITY	DISTANCE AZIMUTH	ROAD DISTANCE	DEVELOPMENT	EST. AREA CU. YDS.	ADVANTAGES & DISADVANTANGES
0 00	- 50 - 70	- 25 - 40	2 0 - 0 3 0 06 - 0 2	13 8 MI	EST 18 4 MI	NO DEVELOPMENT	16 203 300	MAJOR BORROW AREA FLAT LAND GOOD PRIMARY AND SECONDARY HAUL ROADS EXCELLANT MATERIAL WITH DRAINAGE INTO DUCK PONDS AND STREAMS
5 5	- 25 - 35	- 5 - 15	2 0 - 6 3 0 2 - 0 63	5 0 MI	EST 6 2 MI	SOME HOUSES AND FARM LAND ROADS AND AQUEDUCT	5,744 400	GOOD HAUL ROADS SLOPES INTO CREEK TO PROVIDE FOR DRAINAGE AREA HAS SOME POCKETS OF UNSUITABLE MATERIAL
5 5	- 25 - 35	- 5 - 15	2 0 - 6 3 0 2 - 0 63	5 2 MI	EST 10 6 MI	SOME HOUSING AND FARM LAND ROADS AND STREAMS	6 111 110	GOOD HAUL ROAD SLOPES INTO VALLEY AREA DRAINAGE CHANNEL MAYBE REQUIRED TO PREVENT FLOODING AREA HAS SOME POCKETS OF UNSUITABLE MATERIAL
0 5	20 - 35 15 - 25	5 - 10 5 - 10	0 63 - 2 0 < 0 06	30 8 MI	EST 34 MI	HOUSING FARMLAND OTHER DEVELOPMENTS	17 000 000	GOOD PRIMARY AND SECONDARY HAUL ROADS FLAT LAND DEVELOPED IN SOME AREA ROADS AND STREETS RAILROAD LOCATED IN AREA
35 55	- 25 - 35	- 5 - 15	2 0 - 6 3 0 2 - 0 63	4 8 MI	EST 10 1 MI	SOME HOUSING FARMLAND OPEN FIELDS AND ROADS	5 100,000	GOOD PRIMARY AND SECONDARY HAUL ROADS FLAT LAND WITH SOME HOUSING DEVELOPMENTS HEAVY EQUIP CAN WORK IN AREA
45 35 45	- 25 - 35 20 - 30	- 5 - 15 5 - 15	2 0 - 6 3 2 - 63 2 0 - 6 3	11 4 MI	EST 13 2 MI	NO DEVELOPMENT	2 450 300	GOOD HAUL ROAD CAN DRAIN INTO INTERMITTENT STREAMS OR DIRECTLY INTO RESERVOIR SLOPE OF LAND IS MINIMAL
50 80	- 25 - 40	- 10 - 20	2 0 - 6 3 0 2 - 0 63	12 3 MI	EST 21 4 MI	HOUSING AND APARTMENT IN AREA FLAT LAND	2 110 000	GOOD HAUL ROADS DRAINAGE WILL BE INTO INTERMITTENT STREAM WHICH RUNS THROUGH THE AREA HOUSING AND APT PLUS SCHOOL ARE IN THE AREA
50 60	- 25 - 40	- 10 - 20	2 0 - 6 3 0 2 - 0 63	13 4 MI	EST 26 1 MI	SOME HOUSING ROADS AND PIPELINE	1 190 000	GOOD HAUL ROADS AND DRAINAGE HOUSING DEVELOPMENTS ON EAST SIDE AND PIPELINE RUNNING THROUGH MIDDLE OF AREA
50 60	- 25 - 40 -	- 10 - 20 -	2 0 - 6 3 0 2 - 6 3 -	13 8 MI	EST 25 5 MI	3 STORE ON FAR WEST SIDE PIPELINE RUNNING THROUGH MIDDLE OF AREA	1 348 140	GOOD HAUL ROADS LARGE OPEN AREA DRAINAGE WILL BE RUN INTO INTERMITTENT STREAM WHICH RUNS THROUGH TO PROPERTY PIPELINE WILL BE A PROBLEM
50 80	- 25 - 40 -	- 10 - 20 -	2 0 - 6 3 0 2 - 6 3 -	13 3 MI	EST 25 4 MI	OPEN FIELD NO HOUSING	500 740	GOOD HAUL ROADS AND HEAVY EQUIPMENT CAN WORK EFFECTIVELY IN AREA NEED TO SLOPE INTO NORTHWEST CORNER TO INTER-MITTENT STREAM FOR DRAINAGE SMALL QUANTITY
50 80	- 25 - 40 -	- 10 - 20 -	2 0 - 6 3 0 2 - 6 3 -	13 1 MI	EST 25 2 MI	NO DEVELOPMENT	1 840 800	CAN USE FREEWAY AS MAJOR HAUL ROAD OR CORNER STREETS DRAINAGE IS GOOD HAVING SEVERAL INTERMITTENT STREAMS TO USE AREA OPEN EXCEPT FOR SCHOOL LOCATED NEAR STREET
50 80	- 25 - 40 -	- 10 - 20 -	2 0 - 6 3 0 2 - 6 3 -	12 8 MI	EST 20 7 MI	NO DEVELOPMENT	700 000	CAN USE FREEWAYS AS MAJOR HAUL ROADS DRAINAGE IS ALSO VERY GOOD AREA IS OPEN NO HOUSING QUANTITY IS SMALL
50 80	- 25 - 40 -	- 10 - 20 -	2 0 - 6 3 0 2 - 6 3 -	12 3 MI	EST 21 1 MI	SOME HOUSING OPEN SLOPE AREA	1 000 000	CAN USE MAJOR FWY AS HUAL ROAD HAS GOOD DRAINAGE INTO INTERMITTENT STREAMS THERE IS A SLIGHT SLOPE TO FWY SO DRAINAGE WOULD HAVE TO BE TOWARD STREAMS
50 90 75	- 55 - 75 - 20 - 30	- 20 - 45 - 10 - 20	2 0 - 6 3 < 06 - 0 2 - 63	10 2 MI	EST 23 2 MI	SOME HOUSING AQUEDUCT ROADS AND DIRT ROADS THROUGH AREA	1 173 330	GOOD HAUL ROADS DRAINAGE PROBLEM AQUELUCT AND ROADS POSE CONSTRUCTION PROBLEMS RAILROAD TRACKS NEAR AREA
50	-	-	2 0 - 6 3	16 3 MI	EST 21 1 MI	NO DEVELOPMENT	2 583 330	GOOD HAUL ROADS WITH DRAINAGE INTO RIVER NO DEVELOPMENT RAILROAD TRACKS NEAR AREA
50 90 75	- 55 - 75 - 20 - 30	- 20 - 45 - 10 - 20	2 0 - 6 3 < 06 - 02 - 63	19 2 MI	EST 20 2 MI	FARMLAND WITH HOUSING AND ROADS RAILROAD N&S	1 001 518	GOOD HAUL ROADS AND HEAVY EQUIPMENT CAN WORK EFFECTIVELY IN AREA BUT NEED TO SLOPE AND MAKE SHORT CHANNEL TO STREAM FOR DRAINAGE
50 90 75	- 55 - 75 - 20 - 30	- 20 - 45 - 10 - 20	2 0 - 6 3 < 06 - 0 2 - 63	19 0 MI	EST 25 1 MI	FARMLAND AND PASTURES WITH SOME HOUSING	992 592	GOOD HAUL ROADS AND DRAINAGE BUT TOO MUCH HOUSING DEVEL-OPMENT FOR HEAVY EQUIPMENT TO WORK AROUND ALSO NOISE CONTROL PROBLEM
45 55 45	- 25 - 35 20 - 30	- 5 - 15 5 - 15	2 0 - 6 3 0 2 - 0 63 2 0 - 6 3	13 1 MI	EST 17 5 MI	GARDENS AND REST AREA SOME HOUSING ROADS AND RR	1 210 120	CAN USE FWY AS MAJOR HAUL ROAD INTERMITTENT STREAMS PROVIDE GOOD DRAINAGE ALSO USE RAILROAD FOR HAULING MATERIAL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAM
POTENTIAL CORE BORROW AREAS

DESIGNED BY:
DRAWN BY: RT
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- ___ - B- ___
DISTRICT FILE NO.
SHEET

PLATE D-44

2


POTENTIAL
BORROW
AREA

VALUE ENGINEERING PAYS

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAM
POTENTIAL CORE BORROW
PRADO BASIN

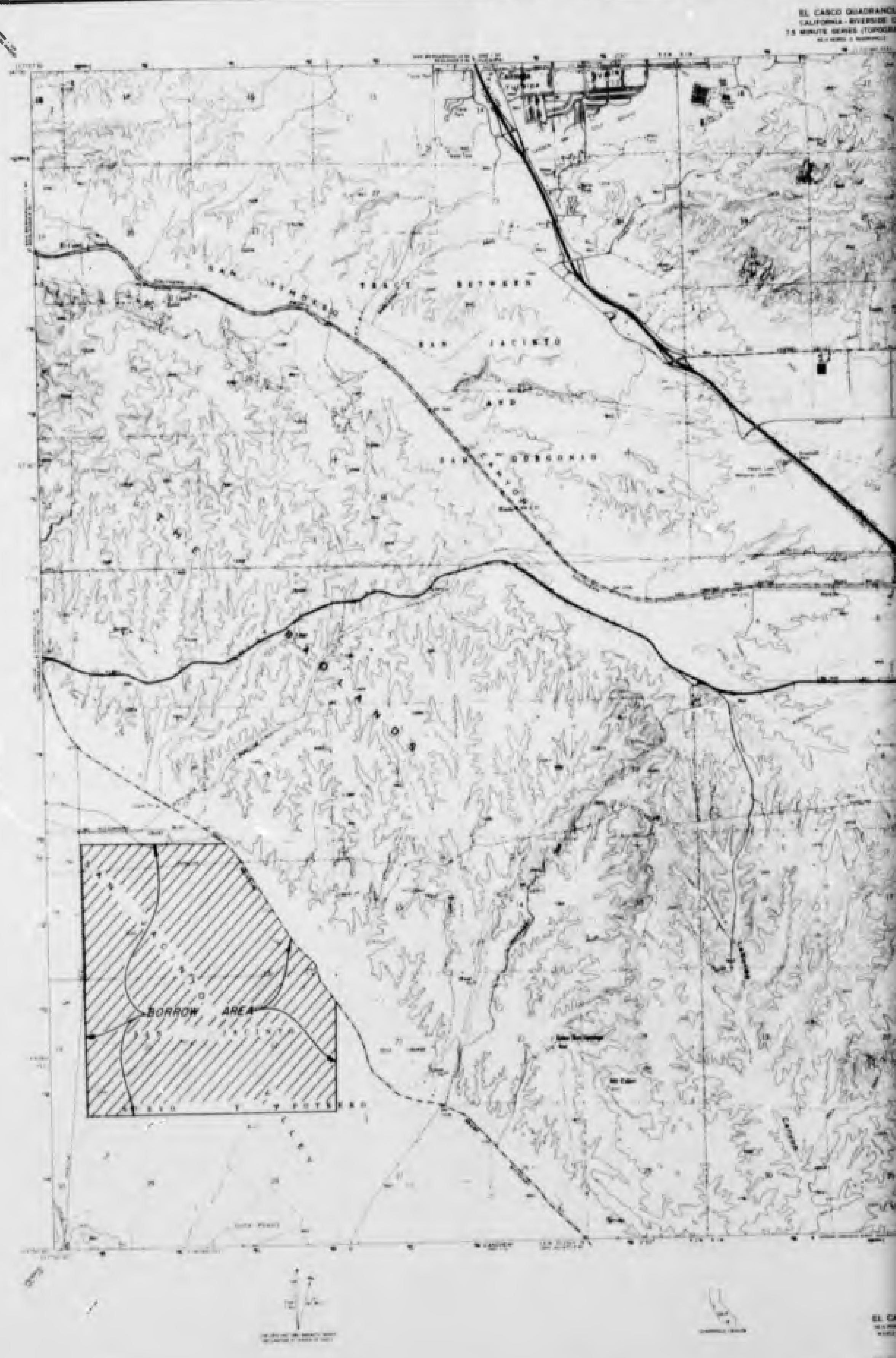
DESIGNED BY:
DRAWN BY: RA
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- ___ B- ___
DISTRICT FILE NO.
SHEET

PLATE D-45

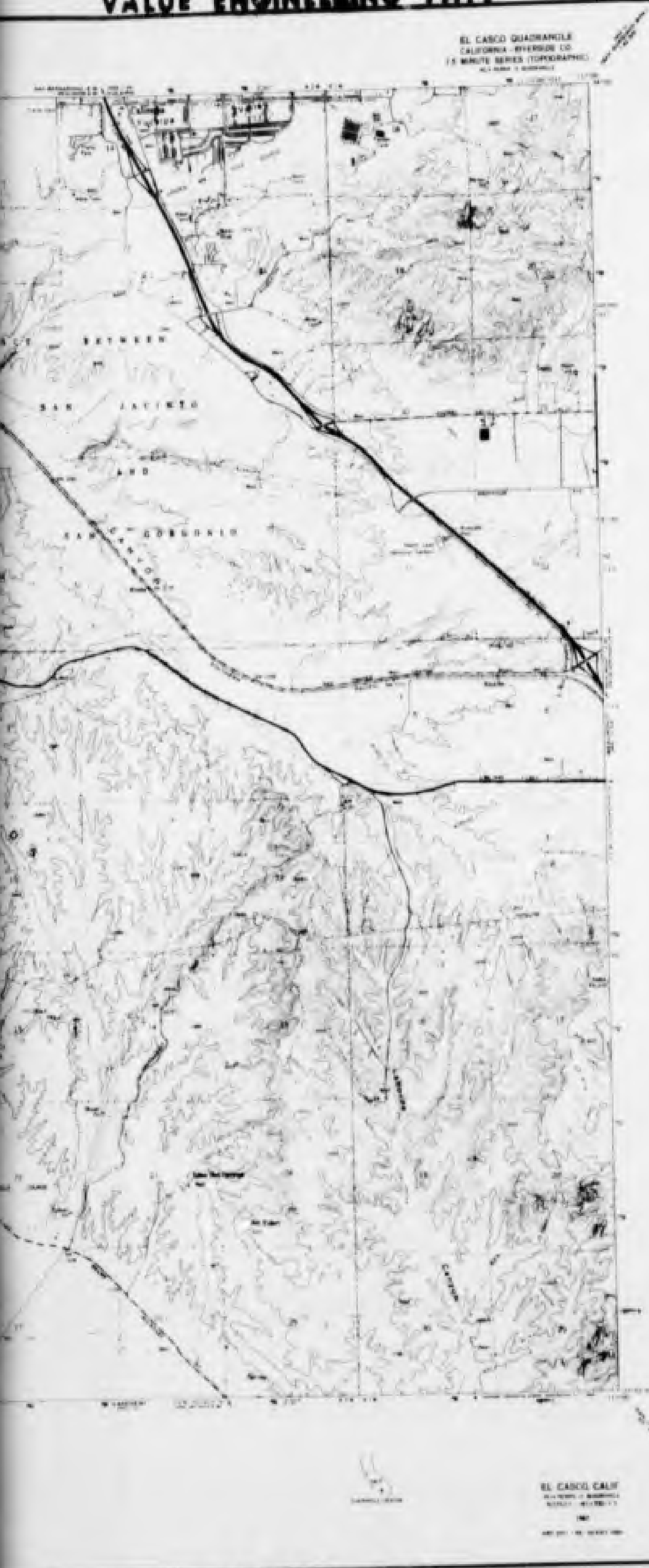
SAFETY PAYS

2

VALUE ENGINEERING PAYS



SAFETY PAYS



REVISIONS

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

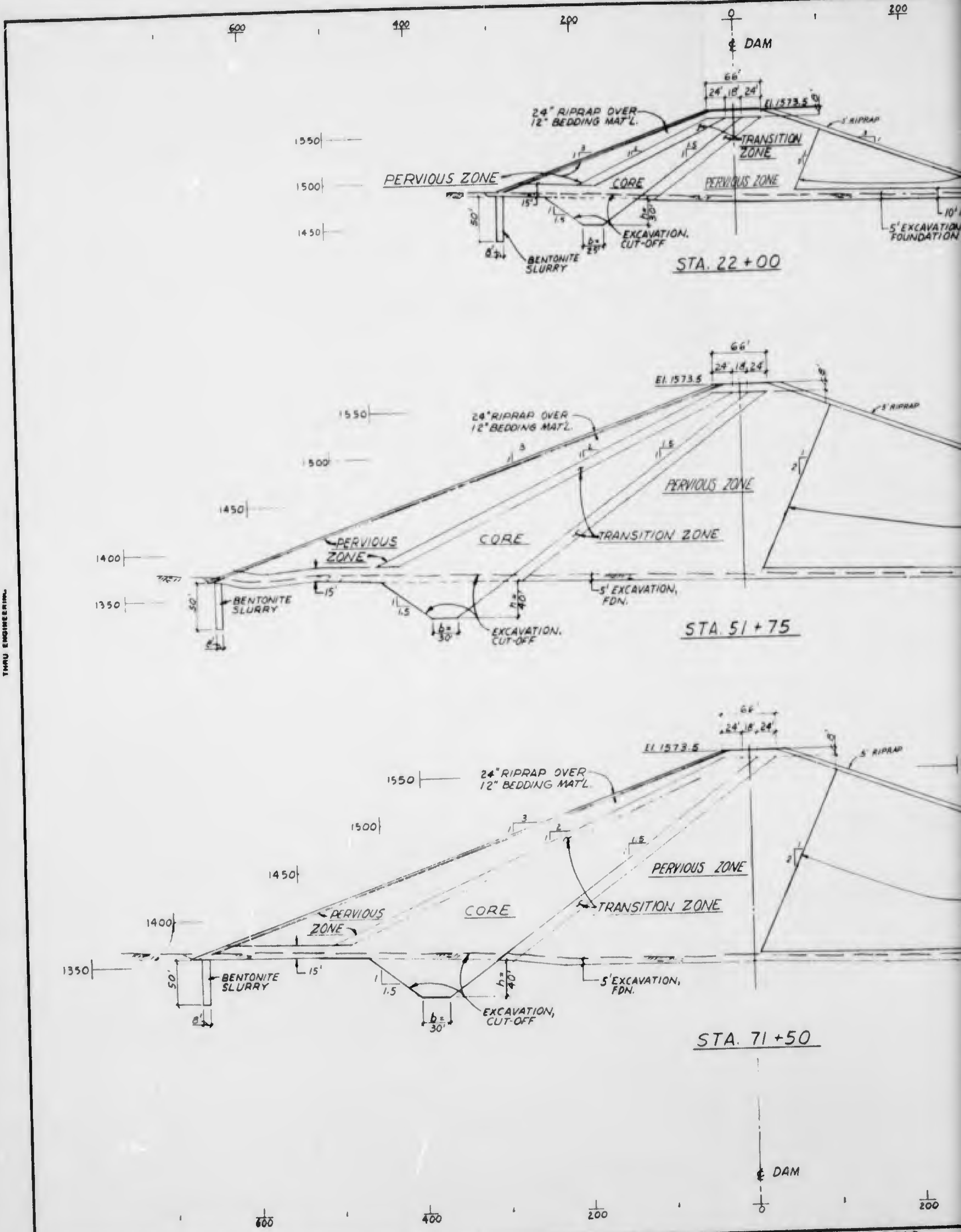
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAM
POTENTIAL CORE BORROW
SAN JACINTO VALLEY

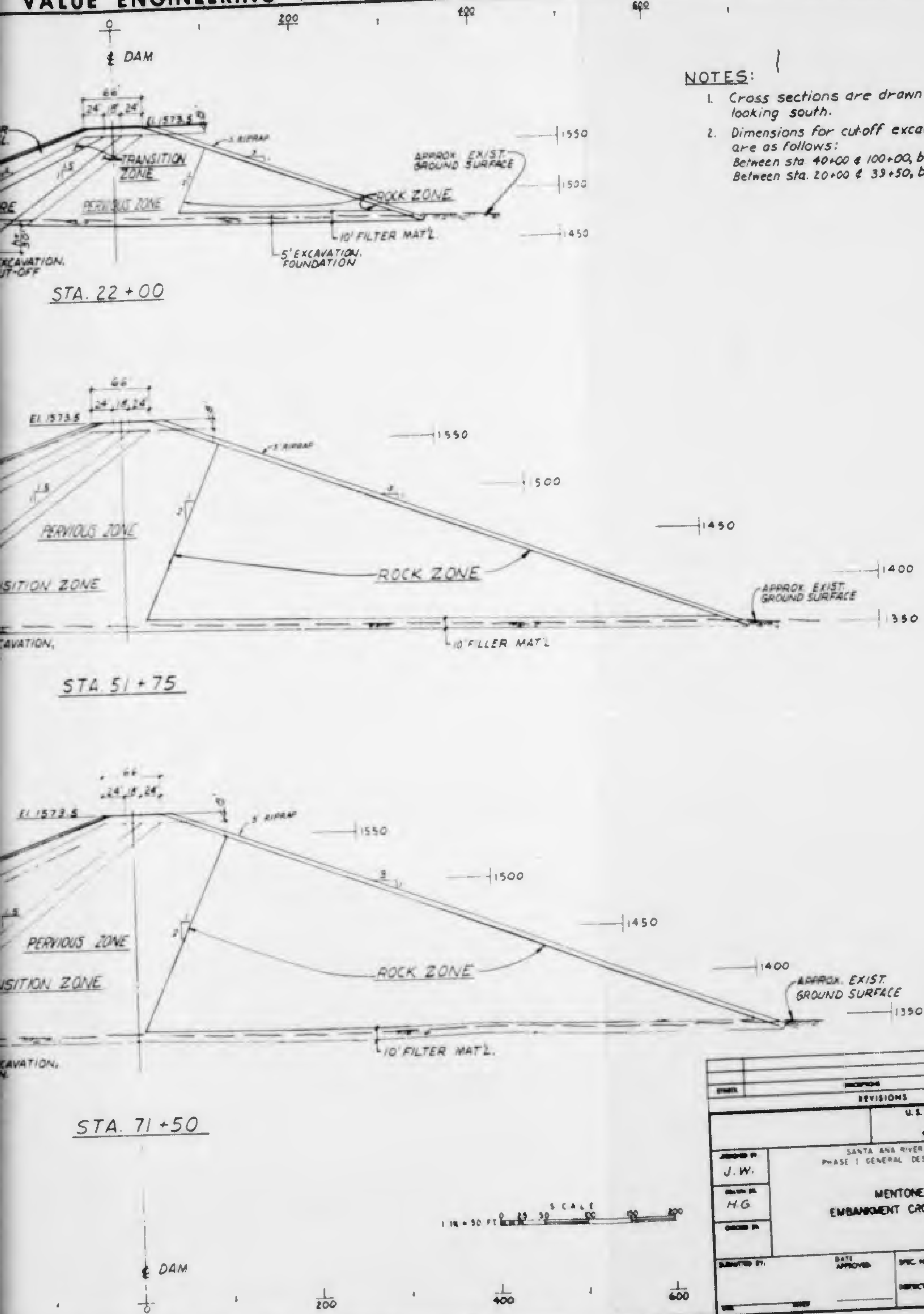
R.A.

PLATE D-46

2



THRU ENGINEERING



NOTES:

1. Cross sections are drawn looking south.
2. Dimensions for cut-off excavation are as follows:
   Between sta. 40+00 & 100+00, b=30', h=40'
   Between sta. 20+00 & 39+50, b=25', h=30'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

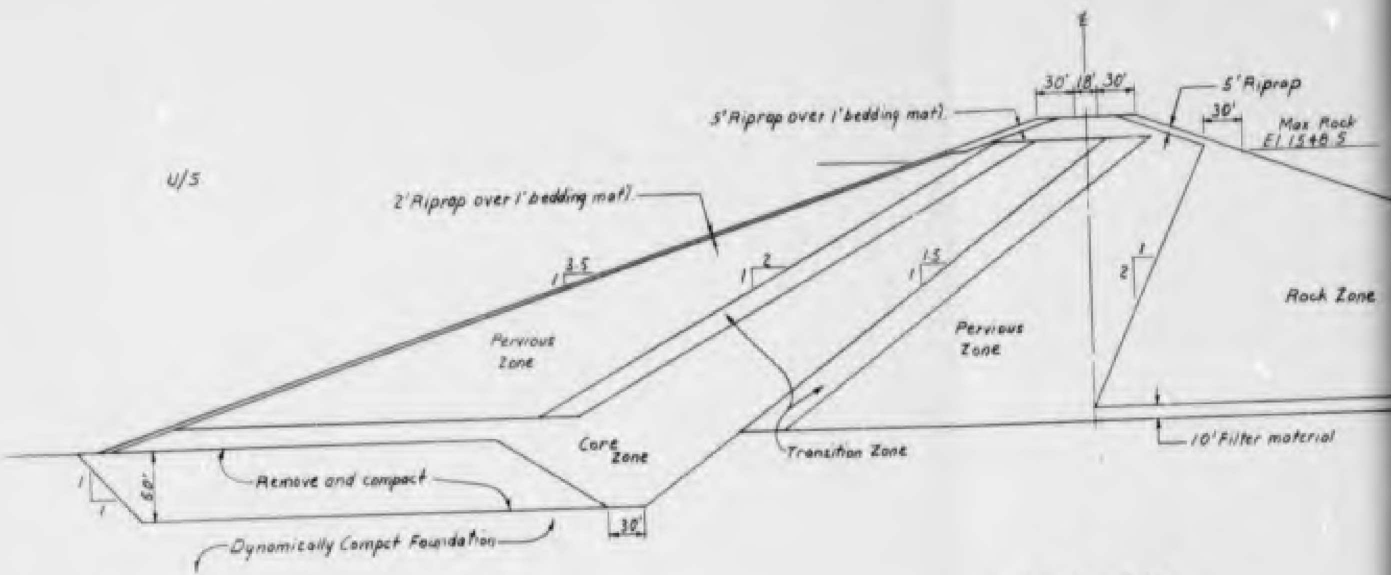
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAM
EMBANKMENT CROSS SECTIONS

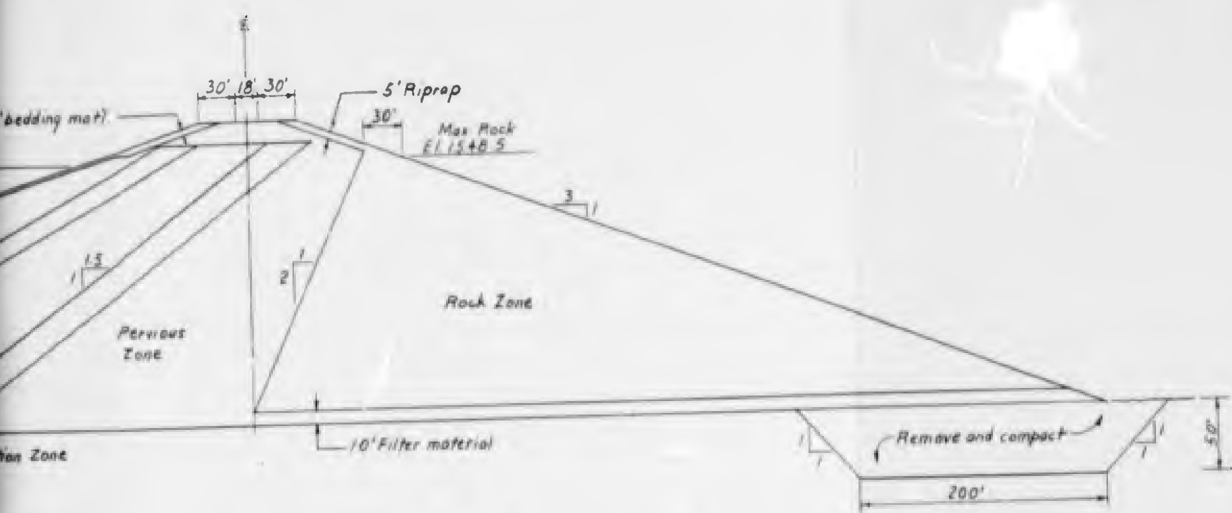
DESIGNED BY: J.W.
DRAWN BY: H.G.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-___-B-___
DISTRICT FILE NO.
SHEET OF

2



CROSS SECTION

SCALE: 1 IN = 50 FT



CROSS SECTION
SCALE: 1 IN = 50 FT

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY:

DRAWN BY: J.Y.

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

MENTONE DAM
ULTIMATE EMBANKMENT SECTION

SUBMITTED BY:

DATE APPROVED:

SPEC. NO. DACW 09- ___ B- ___

DISTRICT FILE NO.

SHEET

50 0 50 100 150

PLATE D-48

2

E Real Estate

APPENDIX E

REAL ESTATE

Contents

Plates

Supplement

APPENDIX E

REAL ESTATE

I. INTRODUCTION

The right-of-way requirements for the Santa Ana River, Phase I GDM are broadly set forth as follows:

The All River Plan

This plan, in main, proposes to (1) Raise the Prado Reservoir 10 feet from elevation 556 feet to elevation 566 feet; (2) Upgrade the existing title of the basin from easement to fee; and (3) To construct a second dam 34 miles upstream near the community of Mentone.

The (NED) Plan

This is an alternate plan which, in main, proposes to (1) Raise the Prado Reservoir 26 feet from elevation 556 feet to elevation 582 feet; and (2) Upgrade the existing basin title from easement to fee.

Note. Both plans include the rights-of-way required for two tributaries--the the Oak Street Drain in the City of Corona which flows into the Prado Basin at the southeast corner and Santiago Creek which flows westerly to the river approximately 18 miles downstream in north Santa Ana.

The Marsh Area-A Wildlife Habitat Adjunct

Over and above the real estate requirements for the above two concepts, the acquisition of lowlands on the east side of the river between Pacific Coast Highway in Newport Beach and Victoria Avenue in Costa Mesa is contemplated as a wildlife habitat for certain endangered species such as the Least Tern. These lowlands which are known as the Marsh Area is a part of the West Newport oil field and is involved with over 100 oil and gas wells.

It should be noted that the river channel improvement adjacent to the Marsh Area would require a 200 foot strip for the widening of the right-of-way. Consequently, the Marsh Area proposed for the wildlife habitat is actually the remainder located between the 200 foot strip and the bluffs to the east. Because of the oil field various acquisition alternatives are possible but only the following ones are considered at this time:

(1) Acquisition of exclusive rights including the oil and gas interests.

(2) Acquisition of the fee subject to the remaining economic life of that portion of the oil field affected.

(3) Acquisition of exclusive rights on Areas 1, 2, 3, and 5, and a 600 foot corridor (Area 4A) abutting the river from Area 4 which connects Areas 1 and 5. See plate E-1.

II. ASSUMPTIONS AND LIMITATIONS

General:

The foregoing right of way estimates are based upon gross real estate appraisals of the fee title estates unless otherwise indicated. The acreages are approximate but commensurate with the mapping provided by the project engineer. The value estimates are made as of 1 March 1980.

The All River Plan

Reach A (Prado Dam and Reservoir)

The Corona National Golf Club, a private golf course, is located partly within the existing basin and partly within the expansion area for elevation 566 feet. It has been assumed that land only will be acquired and that the owners will desire to remain and operate the course under a recreational agreement.

The California Institution for Women, a major state prison will remain in place protected by appropriate flood proofing. It is located on Chino Corona Road in San Bernardino County south of Pine Avenue.

No dairies are being acquired on a partial basis since dairy owners would not be able to comply with space requirements needed to avoid water quality control and pollution problems.

It has been assumed that the Yorba Slaughter Adobe, a registered monument, can and will be relocated within a reasonable distance, hence, money has been included for this purpose.

It has been assumed that the Sunkist Growers, Inc. owners of the land on the southside of Rincon Street at Main Street in Corona will be allowed to continue to dispose of their waste water on this property from their lemon processing plant located elsewhere as the waste water cannot be discharged directly into a stream or the Prado Basin without additional treatment.

Reach B (Upgrade Existing Basin Title)

With respect to the upgrade of the existing basin, it has been assumed that existing oil wells, golf courses, City of Corona Airport, sewage treatment plant and settling ponds, and various water wells will be allowed to remain and operate in place subject to flooding from time to time.

Reach C (Oak Street Drain)

It has been assumed that all necessary rights of way now held by either the City of Corona or the County of Riverside will be made available at no cost for the project.

It has also been assumed that project right of way will be located along the rear property line of a new MacDonald's Hamburger outlet at the southwest corner of Lincoln Ave and "D" Street so as to create the least possible interruption of business. It is further assumed that the building is not affected but the vehicular circulation and/or parking may be affected during construction of the covered drain across the rear of this property.

Reach D (Mentone Dam and Basin)

The following assumptions have been made:

1. That the abandonment of the Atchison, Topeka and Santa Fe Railroad which has been commenced will be completed within the forseeable future. It is assumed that the improvements will be removed and salvaged by Santa Fe prior to construction of the project.

2. That the three radio towers in the basin area belonging to station KCAL may be relocated without undue difficulty to a point outside the basin.

3. That Greenspot Road, a major local road, will not be severed by the proposed dam but will be partially rerouted over Santa Ana Canyon Road to Weaver Street thence around and over the north end of the dam and thence southerly to Santa Ana Canyon Road and back to Greenspot.

4. That that portion of the spreading ground ponds and structures affected and belonging to the San Bernardino Valley Water Conservation District can be relocated. The District opposes interruption of their activities on the basis that they have prior rights by virtue of a Congressional Act dated 1 February 1909.

Reach E (Santa Ana Canyon)

It has been assumed that the project incurs no expense with respect to the lands and improvements of the County of Orange Featherly Regional Park.

Reach F (Santa Ana River Main Stem)

The following assumptions have been made:

(1) That the power poles located within the right of way north and south of Gisler Avenue, Costa Mesa will remain in place and; further, if it is necessary to remove and relocate all or part of the poles; the cost will be a construction item and not a real estate item.

(2) That construction will affect two greens of the River View Golf Course which is north of 17th Street in the City of Santa Ana. One green is on the west of the river and one on the east side at the south end of the course. Further, that the access between the west and east portions of the golf course in the form of two small bridges across the river is unimpaired by the project.

(3) That the right of way required along the east side of the river between Victoria Street, Costa Mesa and Pacific Coast Highway in Newport Beach is a 200 foot strip.

Reach G (Santiago Creek)

It has been assumed that the remaining economic life of the Con Rock sand and gravel pit located just southerly of the Garden Grove Freeway is approximately six years; and further, that prior to the time of real property acquisitons, the major existing improvements will have been removed by the owners.

The NED Plan

Reach H (Prado Dam and Reservoir)

The Corona National Golf Club, a private golf course, is located partly within the existing basin and partly within the expansion area for elevation 582 feet. It has been assumed that land only will be acquired and that owners will desire to remain and operate the course under a recreational agreement.

The California Institution for Women a major state prison, is located on Chino Corona Road south of Pine Avenue in the County of San Bernardino. It has been assumed that the prison cannot be floodproofed and must be acquired.

No dairies are being acquired on a partial basis since dairy owners would not be able to comply with space requirements needed to avoid water quality control and pollution problems.

It has been assumed that the Sunkist Growers, Inc owners of the land on the southside of the Rincon street at Main street in Corona will be allowed to continue to dispose of their waste water on this property from their lemon processing plant located elsewhere as the waste water cannot be discharged directly into a stream or the Prado Basin without additional treatment.

It is assumed that the aluminum plant on Rincon Street east of Smith will be acquired. This is a substantial property owned by the Aluminum Company of America.

The State of California property located on the east side of Bluff Road north of River Road is improved with three water wells which furnish water to the California Rehabilitation Center and the U.S. Naval

Reservation in Norco, California. It has been assumed that adequate substitute water sources are available for these institutions.

For purposes of this estimate it has been assumed that any islands of land created by the 582 foot contour lines will be acquired.

Reaches B, C, E, F, and H are assummed to apply to the NED plan as they do to the All-River Plan.

III. RIGHT-OF-WAY COSTS

SUMMARY:

The All River PLAN

Reach		Acres	Amount
(A)	Prado Dam and Reservoir	1,461	$ 60,000,000
( )	Upgrade Existing Basin Title	3,542	31,300,000
(C)	Oak Street Drain	7	930,000
(D)	Mentone Dam & Basin	3,110	21,500,000
(E)	Santa Ana Canyon	1,500	13,000,000
(F)	Santa Ana River to Ocean	85	6,040,000
(G)	Santiago Creek	184	3,500,000
	Totals	9,889	$136,270,000

The NED PLAN

Reach	Acres	Amount
(A) Prado Dam and Reservoir	4,290	$302,700,000
(B) Upgrade Existing Basin Title	3,542	31,300,000
(C) Oak Street Drain	7	930,000
(E) Santa Ana Canyon	1,500	13,000,000
(F) Santa Ana River to Ocean	85	6,040,000
(G) Santiago Creek	184	3,500,000
Totals	9,608	$357,470,000
		R.O. $358,000,000

The Marsh Lands Adjunct

(Wildlife Habitat)

Alternative (1)	Area	Acres	Amount
	1	72	$ 7,700,000
	2	10	313,000
	3	15	720,000
	4	150	67,300,000
	5	79	3,700,000
	Totals	326	$79,733,000
	Less 200' widening for Santa Ana River channel	35	1,452,500
Alternative (1) TOTALS		291	$78,280,500
			R.O. $78,300,000

The Marsh Lands Adjunct (Cont'd)

(Wildlife Habitat)

Alternative (2)	Area	Acres	Amount
	1	72	$ 3,120,000
	2	10	313,000
	3	15	720,000
	4	150	8,400,000
	5	79	3,700,000
	Totals	326	$16,253,000
Less 200' for widening the) Santa Ana River Channel)		35	$ 1,452,500
Alternative (2) TOTALS		291	$14,800,500
			R.O $14,800,000

Alternative 3	Area	Acres	Amount
	1	72	$ 7,700,000
	2	10	313,000
	3	15	720,000
	4A	33	4,100,000
	5	79	3,700,000
	Totals	209	$16,533,000
Less 200' for widening the) Santa Ana River Channel)		35	$ 1,452,500
Alternative (3) TOTALS		174	$15,080,500
			R.O. $15,100,000

REACH (A)

(Prado Dam and Reservoir, Elevation 566')

R/W COST ESTIMATE

LAND:

Type	Acres	(R.O.)	Unit Cost Amount (R.O.)	Total
Agricultural	29	1,230	$16,500	$20,300,000
Industrial	19	122	19,750	2,410,000
Recreational	1	3	66,650	200,000
Residential	94	106	29,150	3,090,000
Totals	143	1,461	$17,800 (AV)	$26,000,000

IMPROVEMENTS:

Type	Number of Building Units	Amount
Agricultural	23 dairies, 6 misc farm units including a horse ranch, a calf ranch, a cheese factory, a hay loft, a feed lot, a fertilizer plant	$18,000,000
Residential	46 houses (does not include agricultural residences)	3,500,000
	Total	$21,500,000

ADMINISTRATIVE CHARGES:

Acquisition Costs, 143 owners @ $5,000 =	$ 715,000	
Relocation Costs (per P.L.. 91-646)		
46 houses @ $15,000 =	690,000	
29 farm units @ $25,000 =	725,000	
Total	$2,130,000	
District Overhead (R.O.)	370,000	
		$ 2,500,000
	Total	$50,000,000
CONTINGENCIES @ 20%		10,000,000
TOTAL OF REACH (A)		$60,000,000

REACH (B)

(Upgrade Existing Basin Title to Fee)

R/W COST ESTIMATE

LAND

Type	Ownerships	Acres	Unit Cost (R.O.)	TOTAL Amount (R.O.)
Agricultural		1,825	$ 11,200	$20,400,000
Industrial		15	20,000	300,000
Riverbottom		1,702	2,000	3,400,000
	Totals	3,542	$6,800,000+(AV)	$24,100,000

ADMINISTRATIVE CHARGES

Acquisition Costs	3,542 acs @ $480	$1,700,000	
District Overhead		300,000	
			$ 2,000,000
CONTINGENCIES @ 20%			5,200,000
TOTAL OF REACH B			$31,300,000

REACH (C)

(Oak Street Drain)

R/W COST ESTIMATE

LAND

Type	Owners	Acres	Indicated Unit Cost	Total Amount (R.O.)
Agricultural	12	1.29	$20,500	$ 26,500
Residential	3	0.34	$24,250	8,300
Commercial	7	3.32	$87,500	290,500
Industrial	2	1.69	$37,100	62,700
Total	24	6.64	$58,450	$388,000
		7.0 R.O.		

IMPROVEMENTS

Type	Description	Amount
Commercial	Part take on (1) Mobile Home Park, (2) Fast food restaurant and (3) Church	$198,000

ACQUISITION CHARGES

Acquisition Costs: 24 owners @ $5,000	$120,000	
Relocations (per P.L. 91-646) 4 Mobile homes @ $10,000 =	40,000	
Total	$160,000	
District Overhead	29,000	
		$189,000
Total		$775,000
CONTINGENCIES @ 20%		155,000
TOTAL OF REACH (C)		$930,000

REACH (D)

(Mentone Dam and Basin)

R/W COST ESTIMATE

LAND

Type	Owners	Acres	Unit Cost/AC (R.O.)	Total Amount (R.O.)
Agricultural	14	465	$ 6,000	$ 2,800,000
Flood Plain	14	2,260	1,300	$ 3,000,000
Residential	38	385	$17,600	$ 6,800,000
Totals	66	3,110	$ 4,000	$12,600,000

IMPROVEMENTS

Type	Description		Amount
Agricultural	15 Single Family Houses	$652,500	
	Miscellaneous chicken and rabbit houses,	2,500	
	1 Metal Storage Building and 1 office	191,000	$ 846,000
Flood Plain	Industrial improvements includes 1 - office building and misc concrete slabs and outbuildings	$ 70,500	
	6 Single Family House	250,000	320,500
	Relocation of facilities in lieu of acquisition:		
	(1) 3 KCAL radio towers	100,000	
	(2) Water Conservation District	586,000	
	(3) Relocate Greenspot Road*	none	$ 686,000
Residential	18 Single Family Houses		$2,347,500
			$4,200,000

ACQUISITION CHARGES

Acquisition Costs, 66 owners @ $5,000 =	$330,000	
Relocations per P.L. 91-646)		
39 houses @ $15,000 =	$585,000	
4 businesses @ 10,000 =	40,000	
Total	$955,000	
District Overhead R.O.	175,000	$ 1,130,000
Total Including Land		$17,930,000
CONTINGENCIES @ 20% R.O.		3,570,000
TOTAL OF REACH (D)		$ 21,500,000

* Improvements will be replaced by project construction and not by real estate.

REACH (E)

(Santa Ana Canyon)

R/W COST ESTIMATE

LAND:

Type	Acres	Unit Cost	Total Amount (R.O.)
Upper Bench Land (Citrus or Dry Farming)	662	$13,000	$ 8,610,000
Flood Plain between Bench and Riverbed	272	5,000	1,360,000
Wash/Riverbed	566	1,000	570,000
	1,500	$ 7,030 (AV)	$10,540,000

IMPROVEMENTS:

Type	Number and Type of Structures	
Citrus or Dry farming	5 houses, 1 quonset, 1 aluminum storage bldg., 6 steel tanks, 10 irrigation wells, 2 orchard fans, 151,600 SF of chicken pens, misc. small sheds, etc.	Nil*

ADMINISTRATIVE CHARGES:

Acquisition costs, 22 owners @ $5,000 =		$110,000	
Relocation Costs (per P.L. 91-646)	5 houses @ $15,000 =	$75,000	
	1 chicken ranch	10,000	
District overhead		35,000	$ 230,000
	Total		$10,770,000
CONTINGENCIES @20%			2,150,000
TOTAL OF REACH (E)			$12,920,000
		R.O.	$13,000,000

* The lands are valued on a higher use basis.

REACH (F)

(24 miles downstream from Yorba Linda)

R/W COST ESTIMATE

LAND:

	Location	Owners	Acres	Unit Cost (R.O.)	Total Amount (R.O.)
(1)	Yorba Linda to Victoria Avenue	13	41	$17,500	$ 718,000
(2)	Victoria Avenue to Pac. Coast Hwy	6	35	$41,500	1,452,000
(3)	Beach and Highway Frontage	2	9	96,000	864,000
	Total	21	85	$35,700 (R.O.)	$3,034,000

IMPROVEMENTS:

	Location	Description	Amount
(2)	See above	Relocate 9 oil and gas wells and abandon old holes	$1,872,000
(3)	See above	Adjustment of Fencing for Least Tern area and adjustment beach maintenance area included in (3) under land	
		Land and Improvements Total	$4,906,000

ADMINISTRATIVE CHARGES:

Acquisition Costs	21 owners @ $5,000 =	105,000.	
Relocation Pursuant to 91-646		None	
District Overhead		19 000	
			124,000
Total			$5,030,000
CONTINGENCY @ 20%			$1,010,000
TOTAL OF REACH (F)			$6,040,000
	R.O.		$6,040,000

REACH (G)
(Santiago Creek)

R/W COST ESTIMATE

LAND

Type	Owners	Acres	Unit Cost (R.O.)	Total Amount (R.O.)
Speculative Residential	15	9	13,350	$ 120,000
Industrial	10	175	15,000	2,625,000
Total	25	184		$2,745,000

IMPROVEMENTS:

See Assumptions and Limitations — None

ADMINISTRATIVE:

Acquisitions Costs: 25 owners @ 000 =	$125,000	
Relocation Costs	None	
District Overhead	22,800	
		$ 147,800
	Total	$2,892,800
CONTINGENCIES: @ 20%		$578,560
TOTAL REACH (G)		$3,471,360
	R.O.	$3,500,000

REACH (H)
(Raise Dam 24')
R/W COST ESTIMATE

LAND

Type	Ownerships	Acres	Unit Costs (R.O.)	Total Amount (R.O.)
Agricultural	93	2,975	$16,500	$49,088,000
Commercial	15	29	45,550	1,321,000
Industrial	48	390	23,350	9,107,000
Recreational	1	3	61,600	185,000
Residential	760	775	35,270	27,334,000
Special Purpose	2	118	16,650	1,965,000
Roads, Railroads, Streambeds		735		
Totals	919	5,025	$20,750 (AV)	$89,000,000

IMPROVEMENTS

Type	Number of Building Units	Amount
Agricultural	63 dairies, 28 misc farm units including 3 horse ranches 3 calf ranches, 14 ranchetts, and 8 others such as a cheese factory, two fertilizer plants, two hay lofts, a feed supply company, and an auction yard.	$ 70,000,000
Commercial	4 misc units including 1 garage, 1 coffee shop, 1 market, 1 service station	$ 1,600,000
Industrial	7 misc type improvements including 3 water wells, 1 defunct dairy, 2 factories, 1 animal shelter, etc.	$ 14,400,000
Residential	589 houses	$ 33,900,000
Special Purpose	California State Womens Prison and a State Historical Monument	$ 25,100,000
Land and Improvements	Total	$145,000,000

ADMINISTRATIVE CHARGES

Acquisition Cost 919 owners @ $5,000 =	$ 4,595,000	
Relocation Costs (per P.L. 91-646)		
589 houses @ $15,000 =	8,835,000	
91 farm units @ $20,000 =	1,820,000	
3 industrial units =	150,000	
1 special purpose =	50,000	
	$15,450,000	
District Overhead R.O.	2,800,000	$ 18,250,000
		$252,250,000
CONTINGENCIES @ 20%		50,450,000
TOTAL REACH (H)		$302,700,000

R/W Cost Estimate--Cont'd

For Raising Prado to Elev. 582 instead of 580 there would an additional 6 dairies affected, 70 additional homes and several more business establishments.

The additional real estate cost would be $3[illegible],500,000 and additional land requirement 520 acres

For Prado Reservoir raised to elevation 582 feet, the principal elements are.

Dairies affected	69--
Homes affected 589 + 70 =	650--
Incremental Land requirement	5,545
Plus existing reservoir area	9,741
New reservoir area total	15,286
Total Real Estate Cost on additional land (5545 acres)	$336,200,000

The Marsh Lands Adjunct
R/W COST ESTIMATES

Item	Number of owners	Extent of Units	Unit Value(R.O.)	Total Amount (R.O.)
		AREA 1 (Alternatives 1 and 3)		
LAND	4	72 Acres	$35,420	$2,550,000
MINERALS	2	10 Oil & Gas Wells	$377,000	3,770,000
ADMIN COSTS:				
Acquisition Costs, 6 owners @ $10,000 =			$60,000	
District O.H.			11,000	
				71,000
			Total	$6,391,000
CONTINGENCIES @ 20%				1,278,200
TOTAL AREA 1				7,669,200
			R.O.	7,700,000
		AREA 2 (All Alternatives)		
LAND	3	10 Acres	$22,500	$ 225,000
MINERALS				None
ADMINS COSTS:				
Acquisition Costs: 3 owners @ $10,000 =			$30,000	
District Overhead			5,500	
				35,500
			Total	$ 260,500
CONTINGENCIES @ 20%				52,100
TOTAL AREA 3				$ 312,600
			R.O.	$ 313,000
		AREA 3 (All Alternatives)		
LAND	1	15 Acres	$39,050	$ 586,000
OIL AND GAS				None
ADMIN COSTS:				
Acquisition Costs: 1 owner @ $10,000 =			$10,000	
District Overhead			1,800	
				11,800
			Total	$597,800
CONTINGENCIES @ 20%				119,600
TOTAL AREA 3				$717,400
			R.O.	$720,000

The Marsh Lands Adjunct (Cont)

Item	Number of Owners	Extent of Units	Unit Value (R.O.)	Total Amount (R.O.)
		AREA 4 (Alternative 1)		
LAND	1	150 Acres	$ 46,610	$ 6,992,000
OIL AND GAS	1	105 oil and gas wells	$467,695	49,108,000
ADMIN COSTS:				
Acquisition Costs:		2 owners @ $10,000 =	$20,000	
District Overhead			3,600	23,600
			Total	$56,123,600
CONTINGENCIES @ 20%				11,224,700
				$67,348,300
TOTAL AREA 4			R.O.	$67,300,000

Item	Number of Owners	Extent of Units	Unit Value (R.O.)	Total Amount (R.O.)
		AREA 4A (Alternative 3)		
LAND	1	33 Acres	$46,600	$1,538,000
OIL AND GAS	1	9 Wells (relocate in lieu of purchase)	$204,000	1,836,000
ADMIN COSTS:				
Acquisition Costs:		2 owners @ $10,000 =	$20,000	
District Overhead			3,600	23,600
			Total	$3,397,600
CONTINGENCIES @ 20%				697,500
				$4,077,100
TOTAL AREA 4A			R.O.	$4,100,000

Item	Number of Owners	Extent of Units	Unit Value (R.O.)	Total Amount (R.O.)
		AREA 5 (All Alternatives)		
LAND	2	79 Acres	$38,730	$3,060,000
OIL AND GAS				None
ADMIN COSTS:				
Acquisition Costs:		2 owners @ $ 10,000 =	$20,000	
District Overhead			3,600	23,600
			Total	$3,083,600
CONTINGENCIES @ 20%				616,700
				$3,700,300
TOTAL AREA 5			R.O.	$3,700.000

The Marsh Lands Adjunct (Cont)

Item	Number of Owners	Extent of Units		Unit Value (R.O.)	Total Amount (R.O.)
		AREA 1 (Alternatives 2)			
LAND	4	72 Acres		$35,410	$2,550,000
OIL AND GAS					None
ADMIN COSTS:					
Acquisition Costs:	4 owners @ $10,000 =			$40,000	
District overhead				7,300	
					47,300
				Total	$2,597,300
CONTINGENCIES @ 20%					519,500
TOTAL AREA 1					3,116,800
				R.O.	$3,120,000
		AREA 4 (Alternative 2)			
LAND	1	150 Acres		$46,610	$6,9[illegible]2,000
OIL AND GAS					None
ADMIN COSTS:					
Acquisition Costs:	1 owner			$10,000	
District Overhead				1,800	
					11,800
				Total	$7,003,800
CONTINGENCIES @ 20%					$1,400,800
TOTAL AREA 4					$8,404,600
				R.O.	$8,400,000

RECOMMENDATIONS

The District recommends:

1. That Congress authorize an advanced land acquisition fund for the Santa Ana River Project specifically.

2. That the fund be established at 7.5% of the total estimated real estate cost of the authorized project.

3. That the Division and District levels of the Corps administer the land acquisition fund on a case by case basis with dollar limitations on approvals reserved to Division and OCE levels only.

The purposes of the above recommendations are: (1) to minimize hardship problems of property owners within projects who may want to sell their properties to the government but are unable to do so because of lack of project funding; (2) to purchase those properties which are likely to escalate substantially because they are ripe for subdivision or improvement with new buildings, etc.

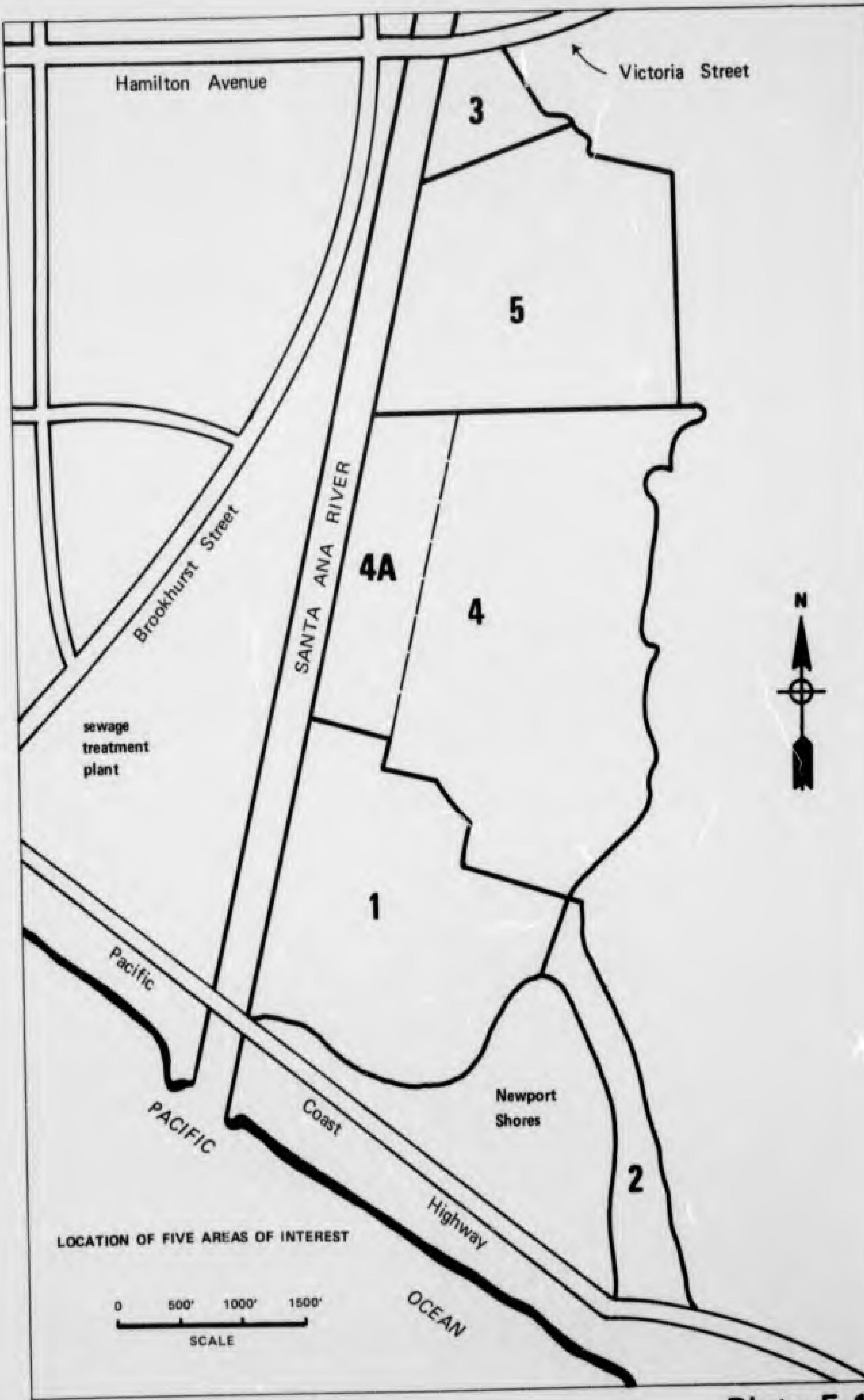

Hamilton Avenue
Victoria Street
3
5
4A
4
SANTA ANA RIVER
Brookhurst Street
sewage treatment plant
1
Pacific Coast Highway
PACIFIC OCEAN
Newport Shores
2
N
LOCATION OF FIVE AREAS OF INTEREST
0 500' 1000' 1500'
SCALE

Plate E-1

C-102

CHINO
CORONA FREEWAY
CONSTRUCT DIKE TO
RAISE PRADO DAM TO
ELEVATION
RAISE SPILLWAY TO
SANTA
ANA
RIVER
1
2
4
6
7
14
15
16
17
26
27
28
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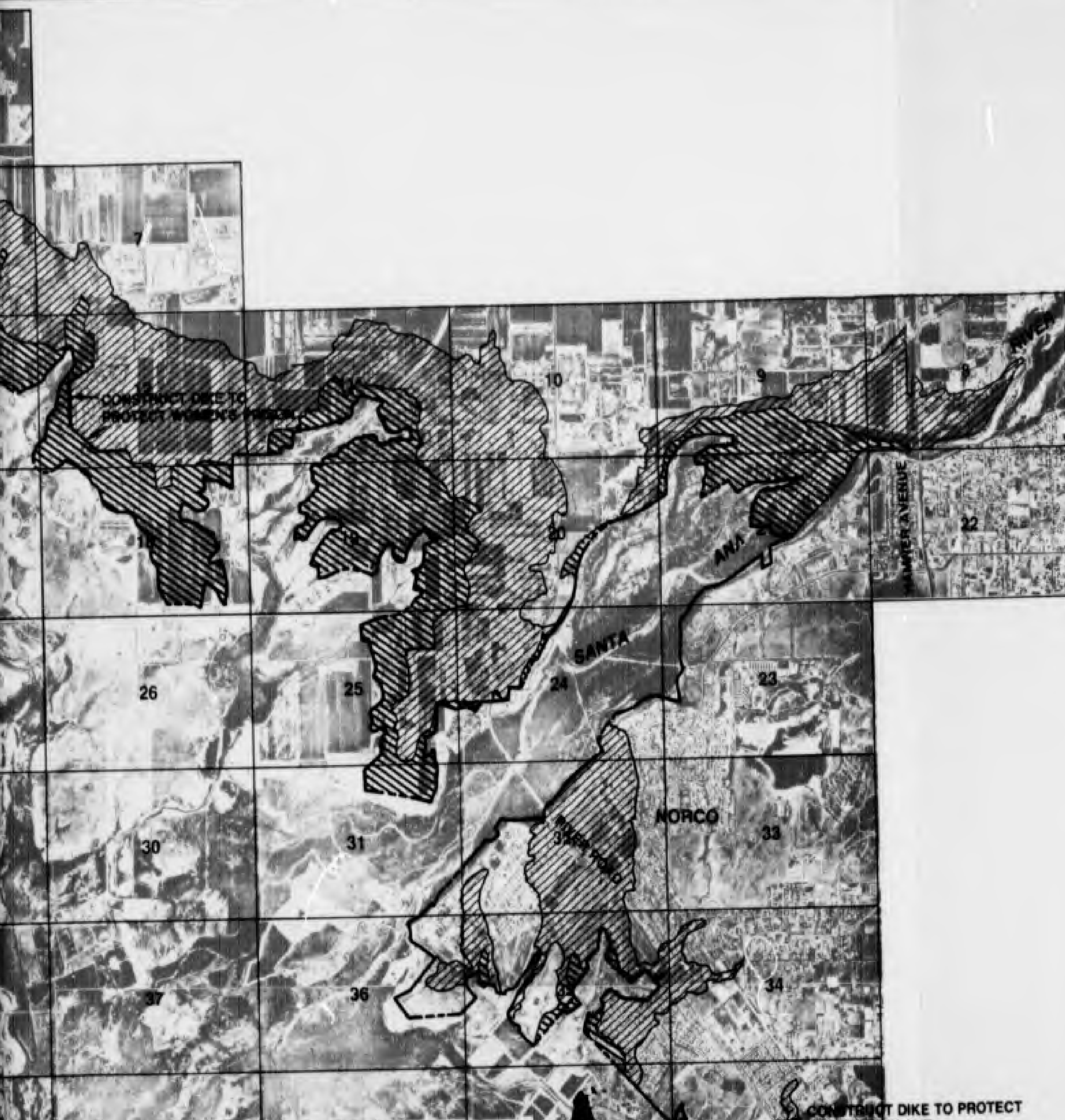
37
39
40
47
48



CONSTRUCT DIKE TO PROTECT WOMEN'S PRISON
SANTA
ANA
RIVER
NORCO
CONSTRUCT DIKE TO PROTECT ALUMINUM PLANT
TEMESCAL WASH
CONSTRUCT DIKE TO PROTECT SEWAGE PLANT
CONSTRUCT RETAINING WALL AND DIKE TO PROTECT FREEWAY
RAISE SPILLWAY TO ELEVATION 563'
FREEWAY
CORONA
7
10
9
22
23
24
25
26
30
31
33
34
36
37
40
41
47

KEY

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566 ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580 ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582 ELEVATION)
AQUISITION LIMITS

SCALE

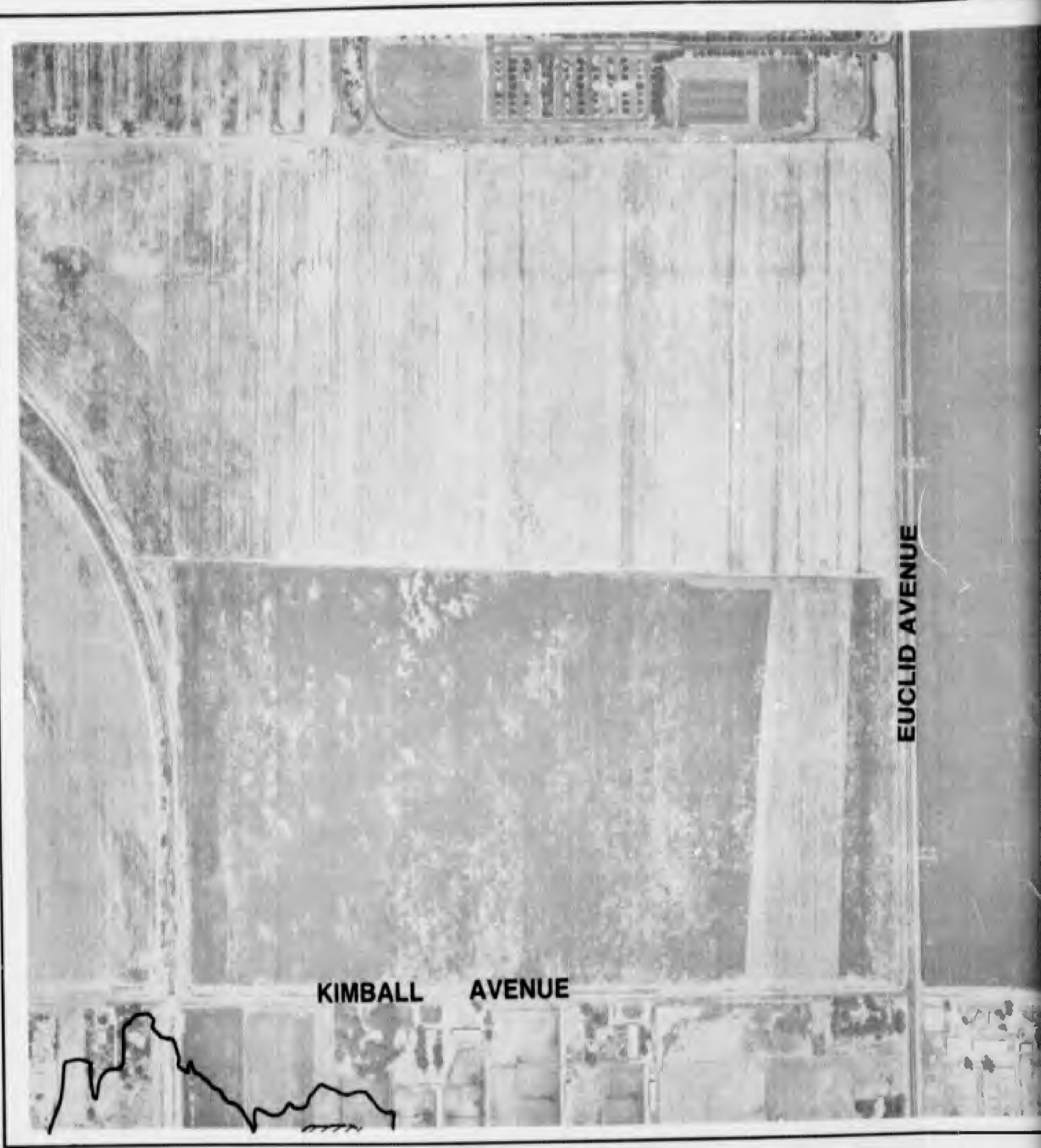
0 5000'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO DAM AND RESERVOIR

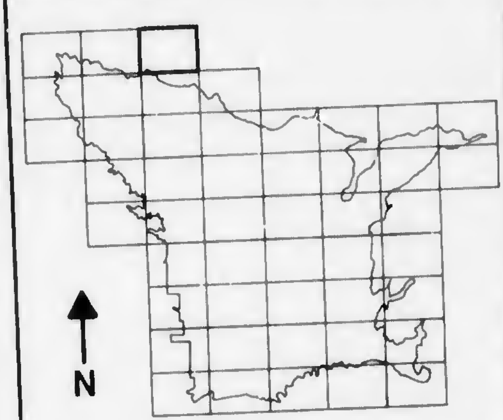
EXPANSION PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-168
PLATE 1


KIMBALL AVENUE
EUCLID AVENUE

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

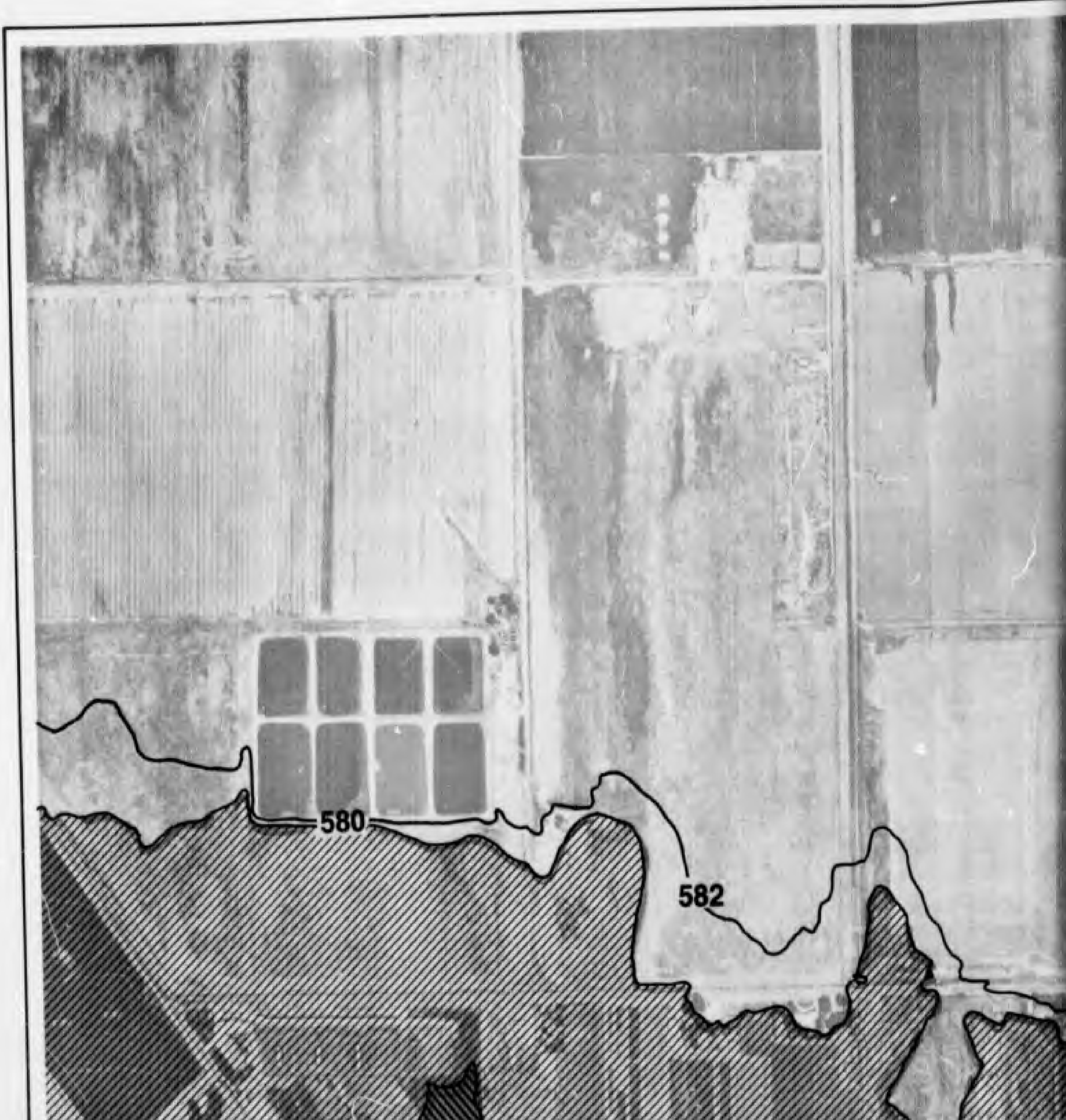
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

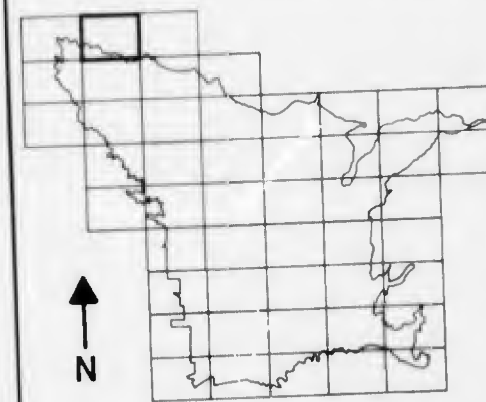
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-125



580
582

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6 (566′ ELEVATION) AQUISITION LIMITS

ALTERNATIVE 5 (580′ ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582′ ELEVATION) AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

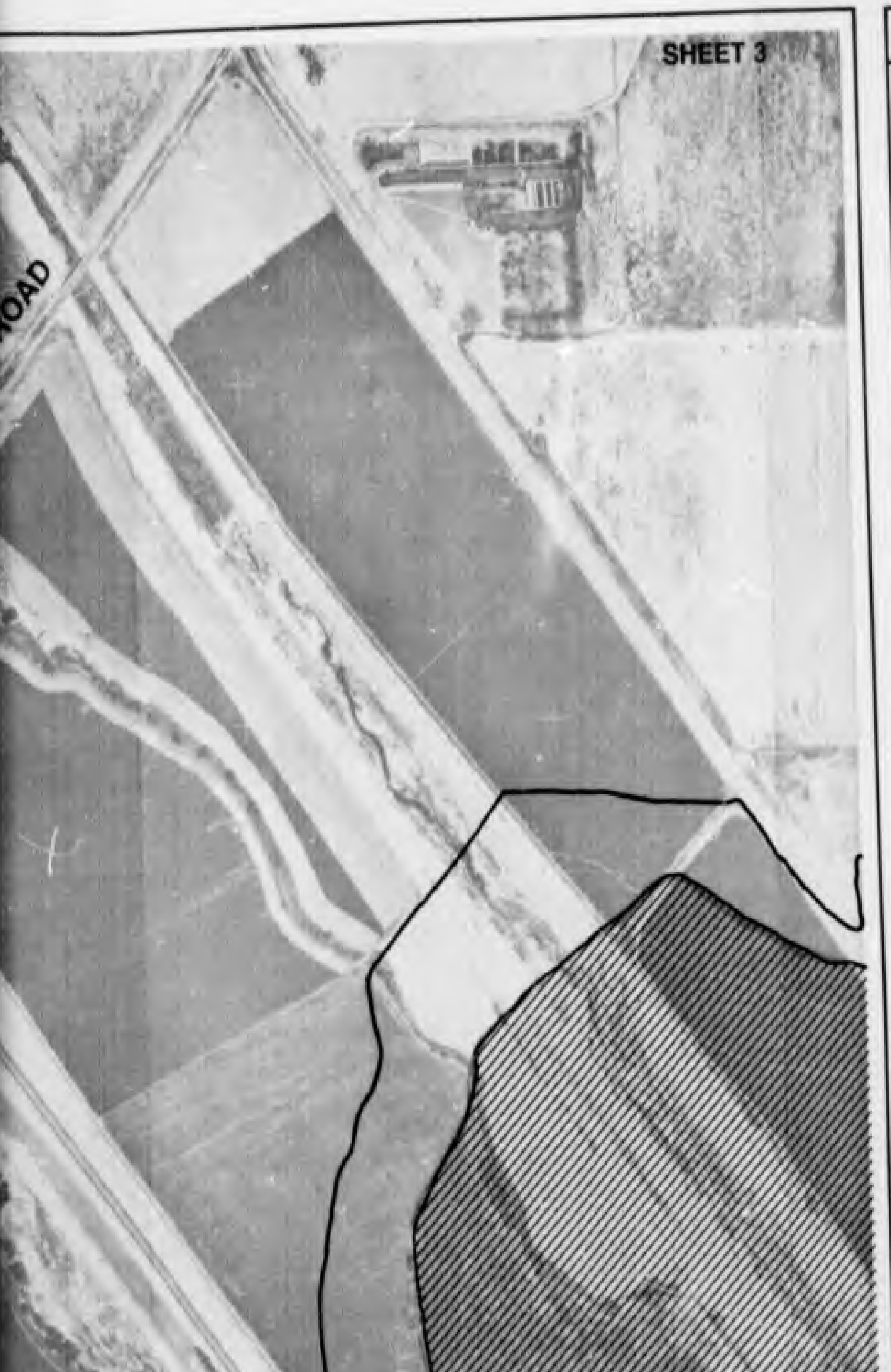
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-123

2


CORONA FREEWAY
(ROUTE 71)
LOS SERRANOS ROAD



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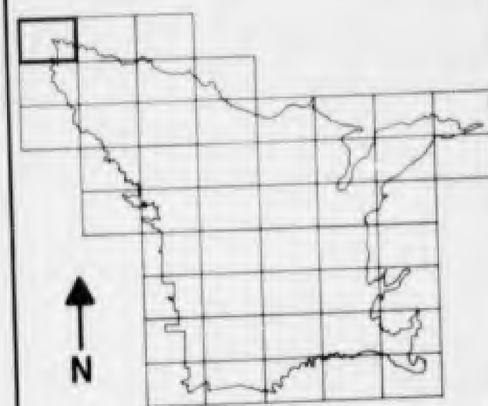


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6 (566' ELEVATION) AQUISITION LIMITS

ALTERNATIVE 5 (580' ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582' ELEVATION) AQUISITION LIMITS

SCALE

500' 0 500'

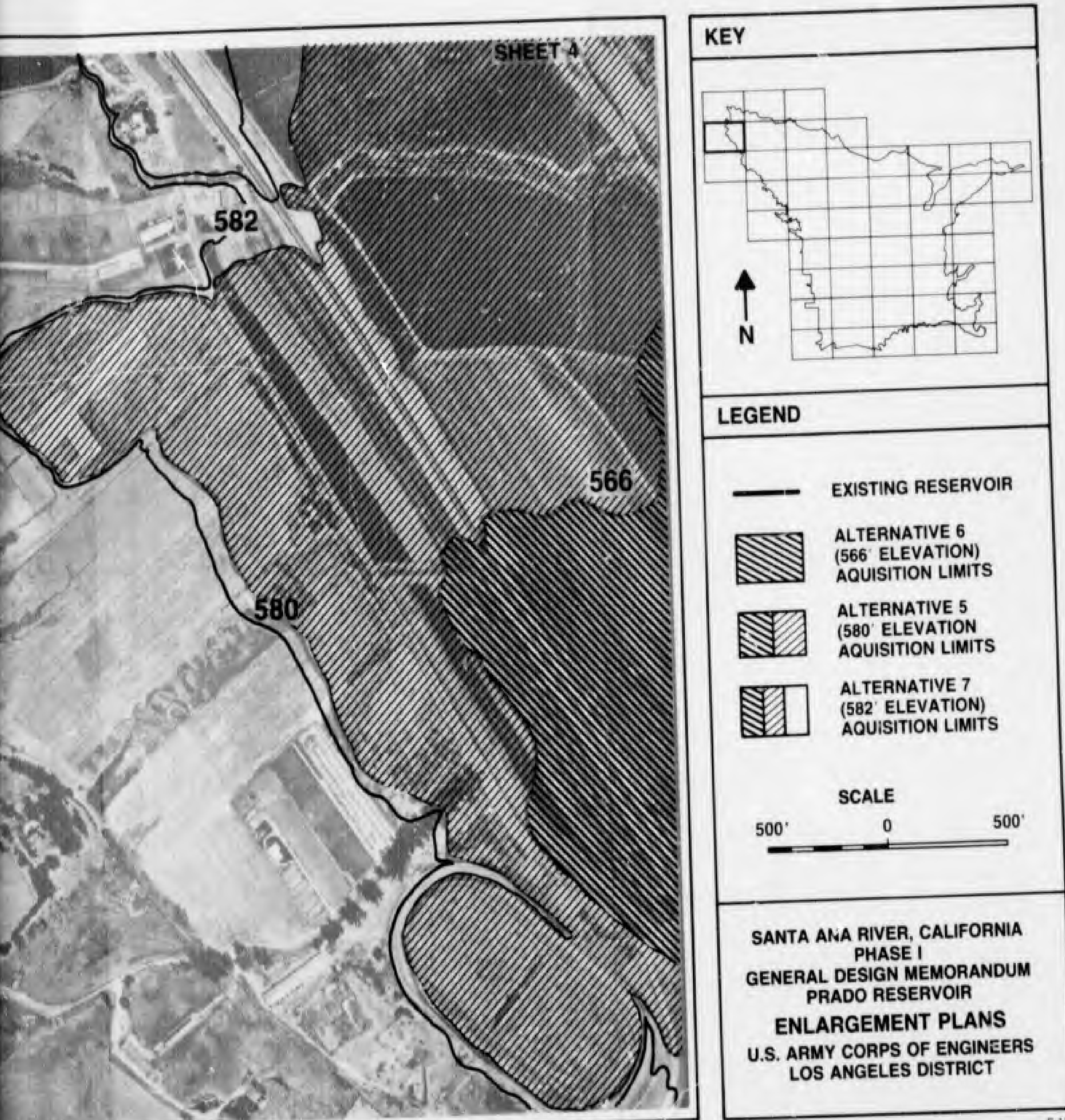
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-122

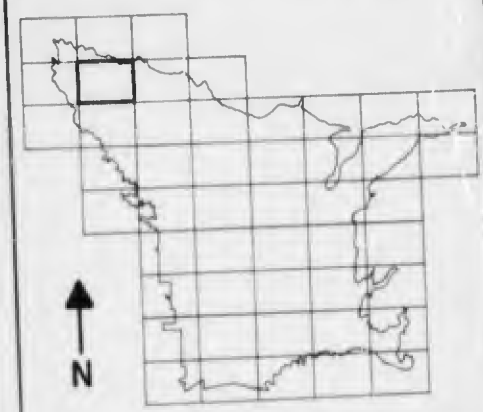
2


SHEET 4
582
566
580
KEY
N
LEGEND
EXISTING RESERVOIR
ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS
ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS
ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS
SCALE
500' 0 500'
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR
ENLARGEMENT PLANS
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-112


CHINO CREEK
CENTRAL AVENUE

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

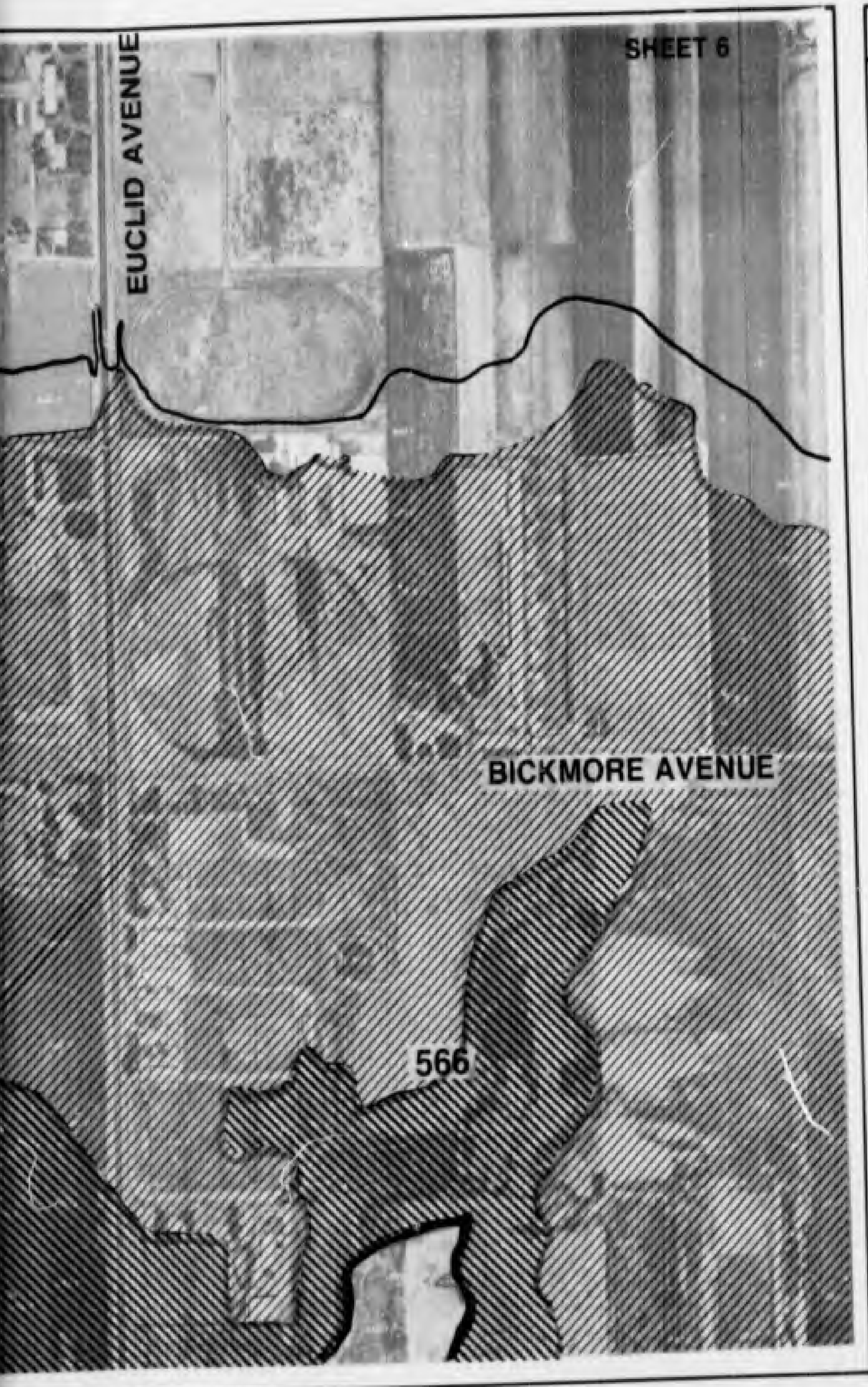
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-115

2


EUCLID AVENUE
566



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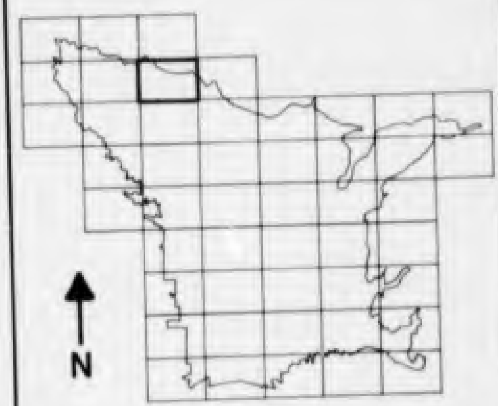


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6 (566' ELEVATION) AQUISITION LIMITS

ALTERNATIVE 5 (580' ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582' ELEVATION) AQUISITION LIMITS

SCALE

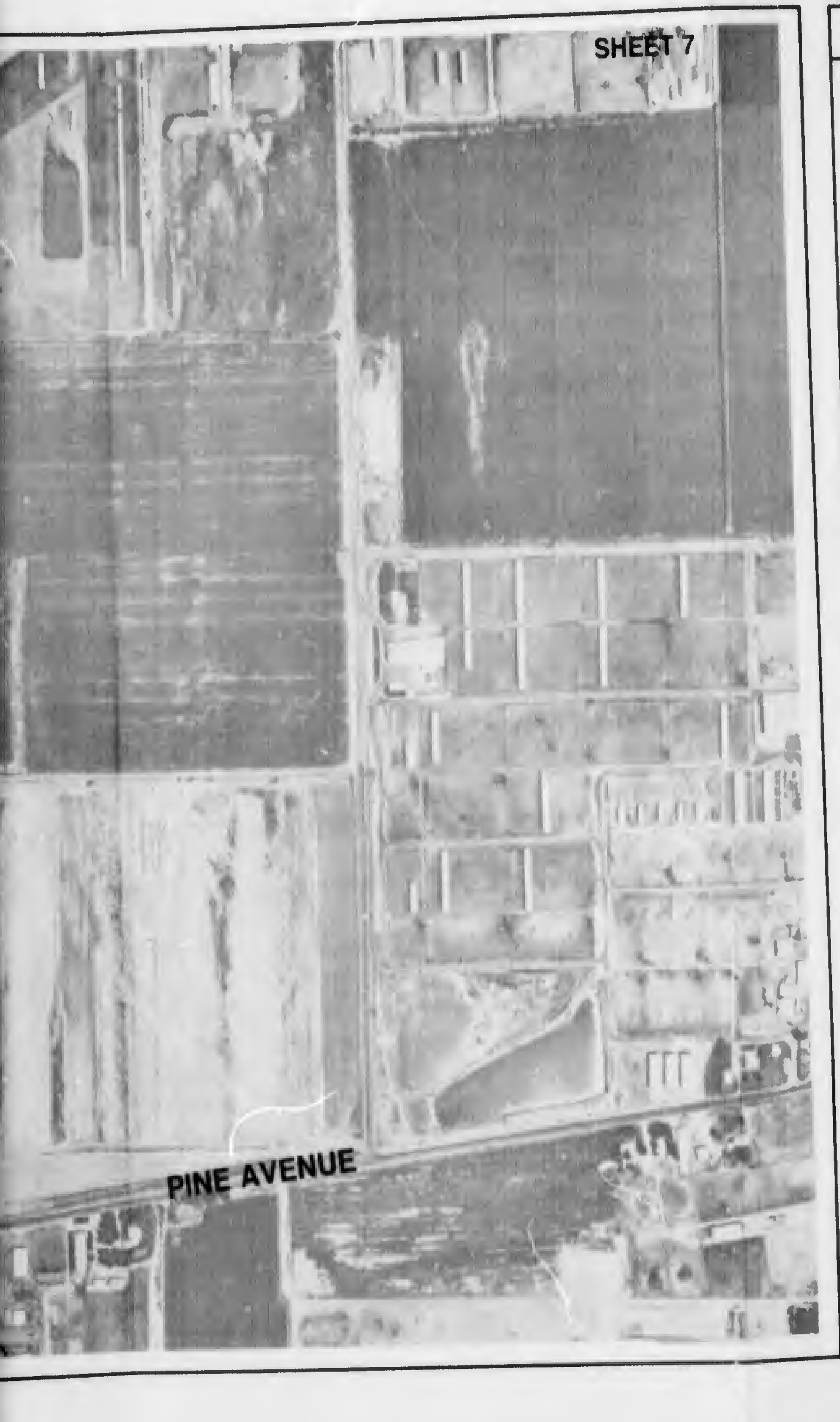
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-142



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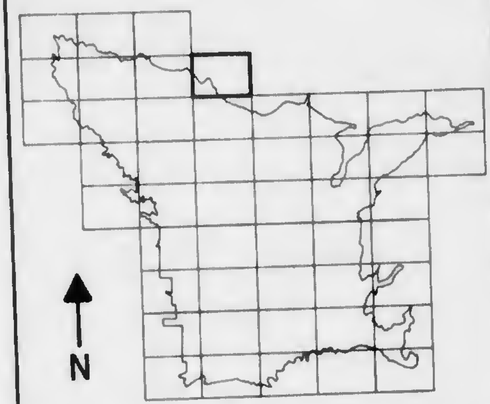


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6 (566' ELEVATION) AQUISITION LIMITS

ALTERNATIVE 5 (580' ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582' ELEVATION) AQUISITION LIMITS

SCALE

500' 0 500'

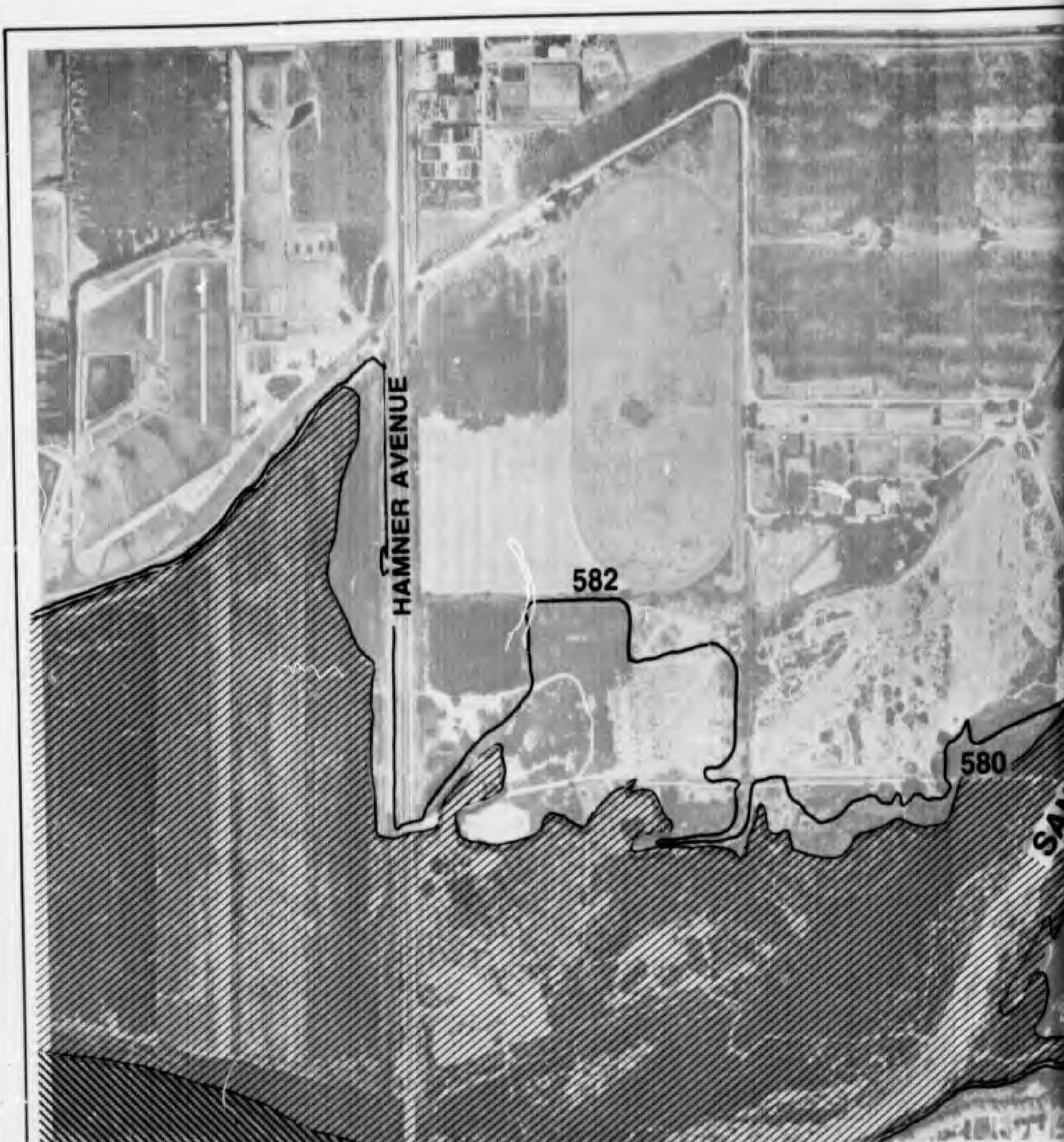
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

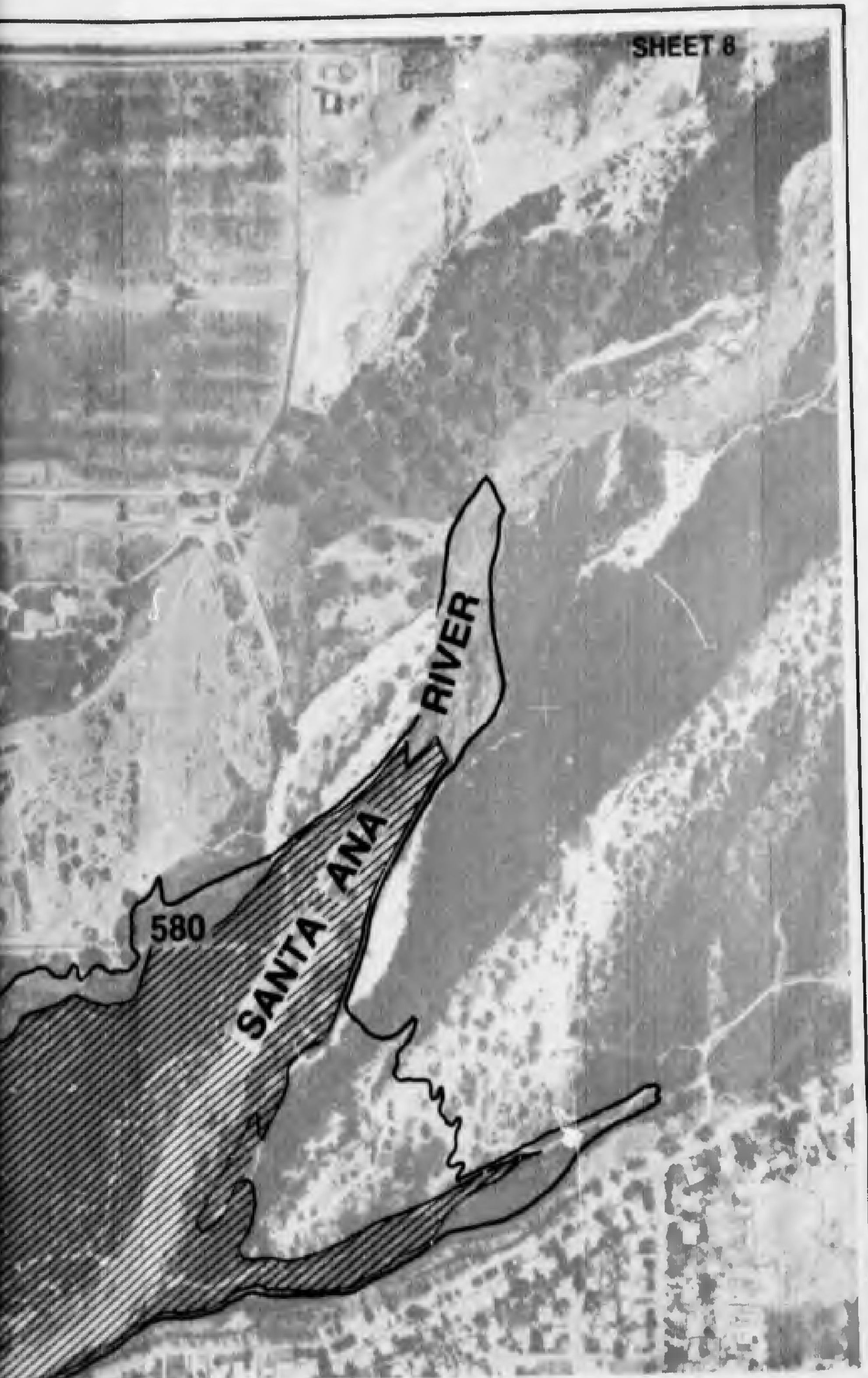
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-114

2


HAMNER AVENUE
582
580



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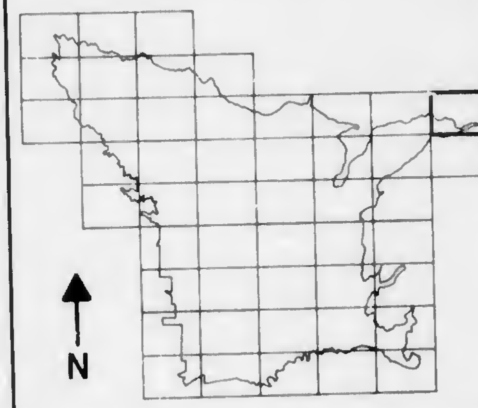


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566′ ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

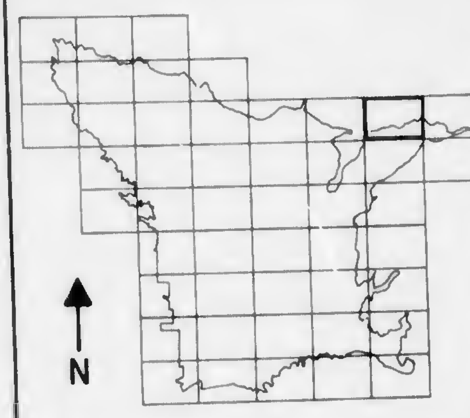
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-110

2


566
582
580

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

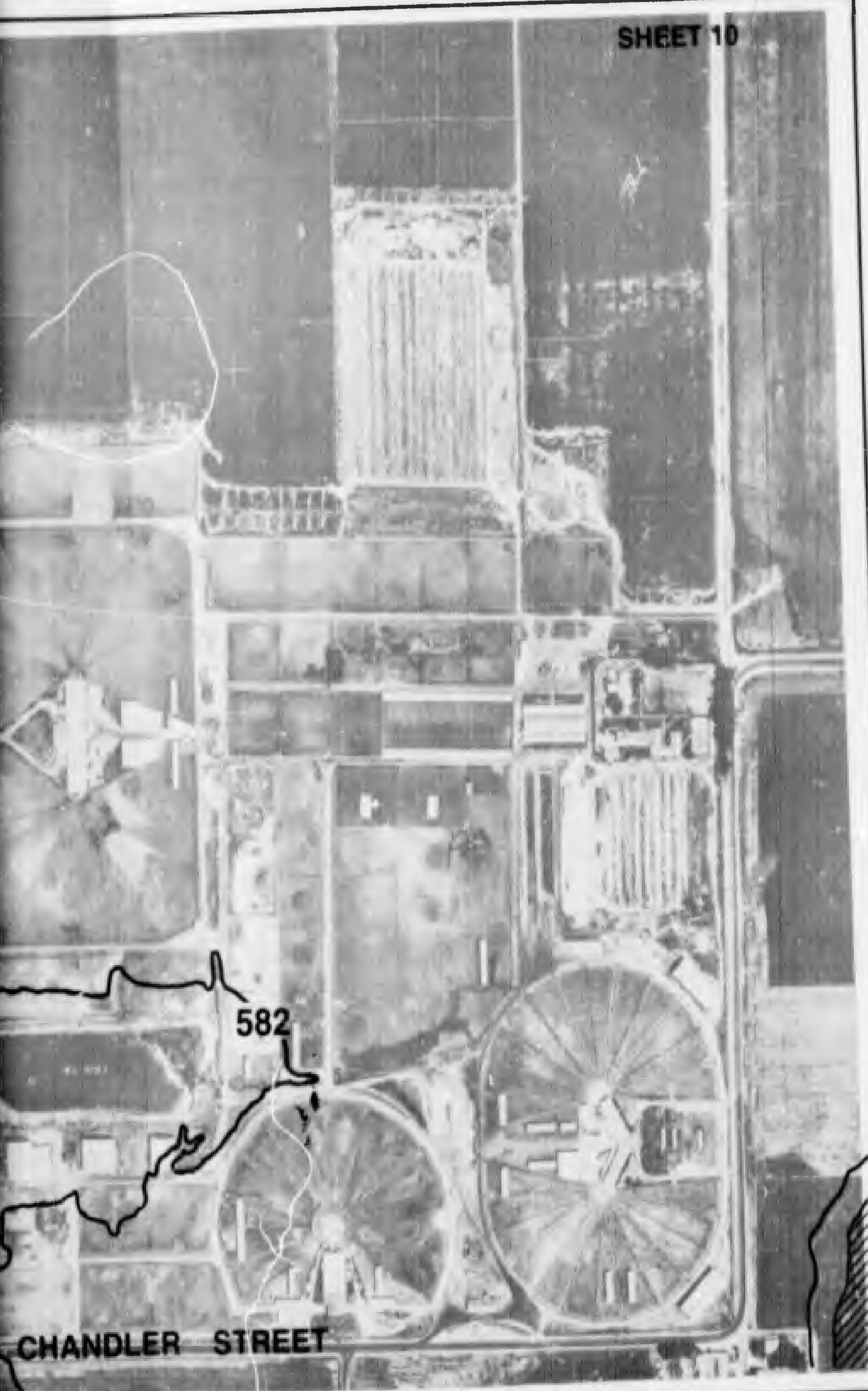
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-109

2


580
ARCHIBALD AVENUE
58
CHANDLER STR



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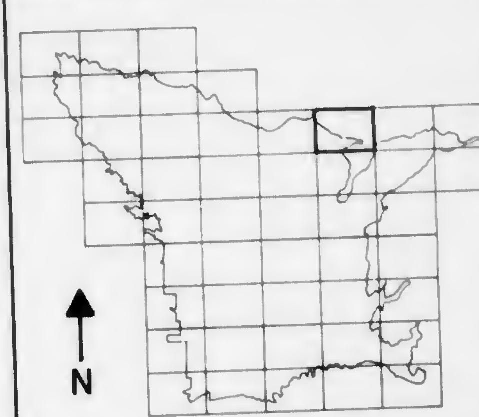


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566′ ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

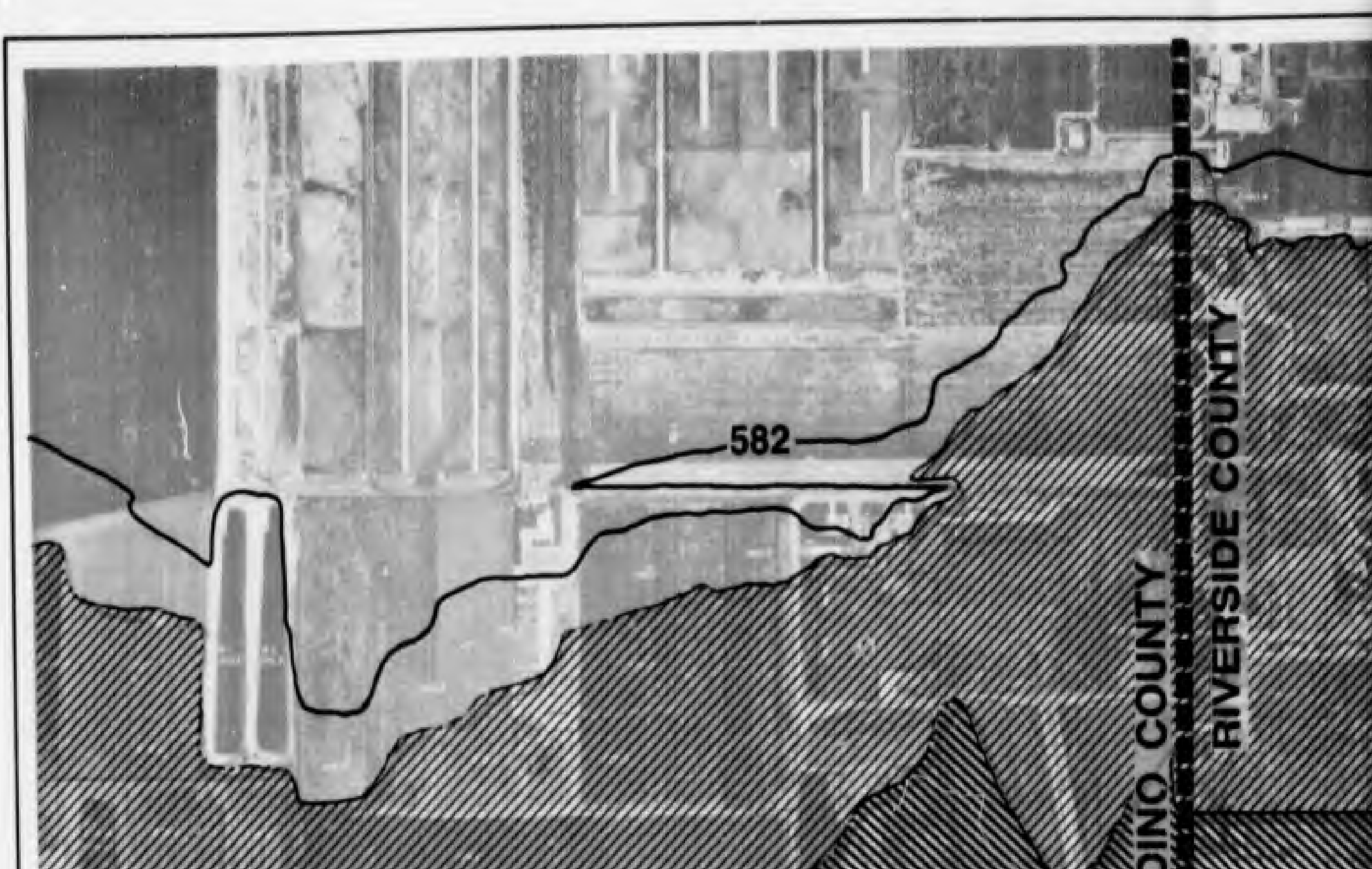
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

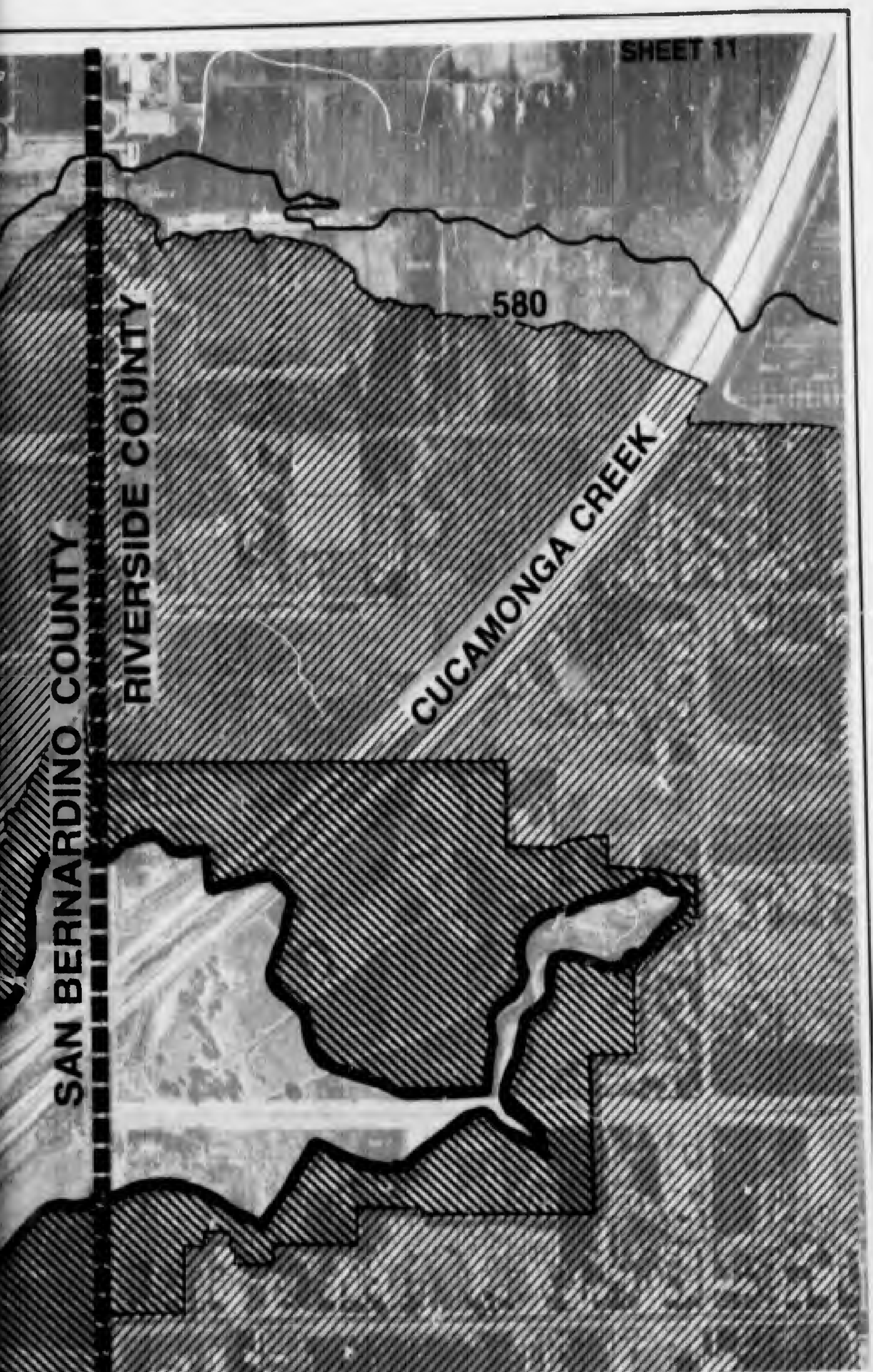
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-143

2


582
SAN BERNARDINO COUNTY
RIVERSIDE COUNTY



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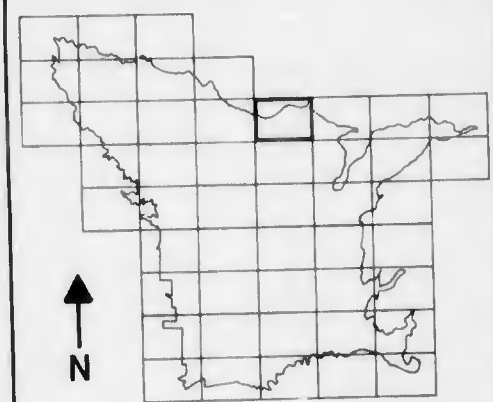


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

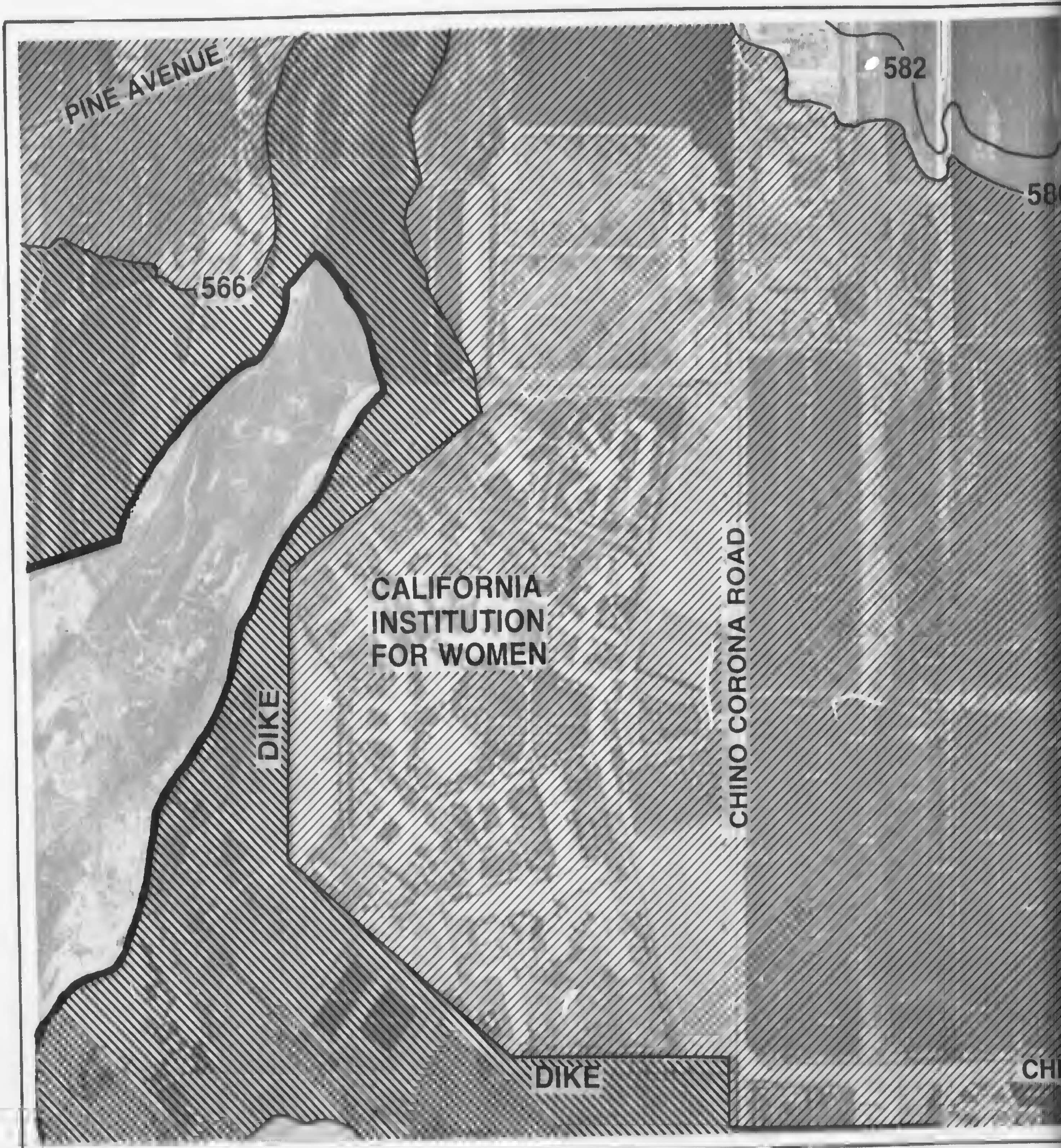
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

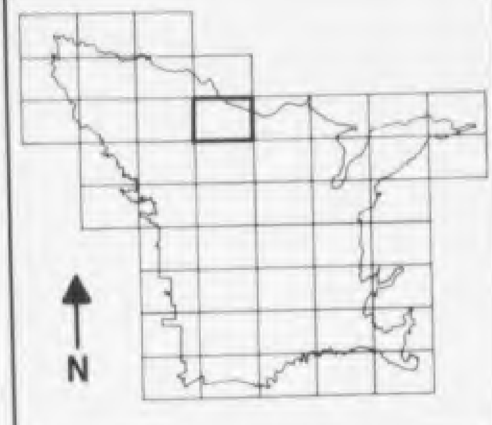
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-144


PINE AVENUE
582
566
CALIFORNIA
INSTITUTION
FOR WOMEN
DIKE
CHINO CORONA ROAD
DIKE

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

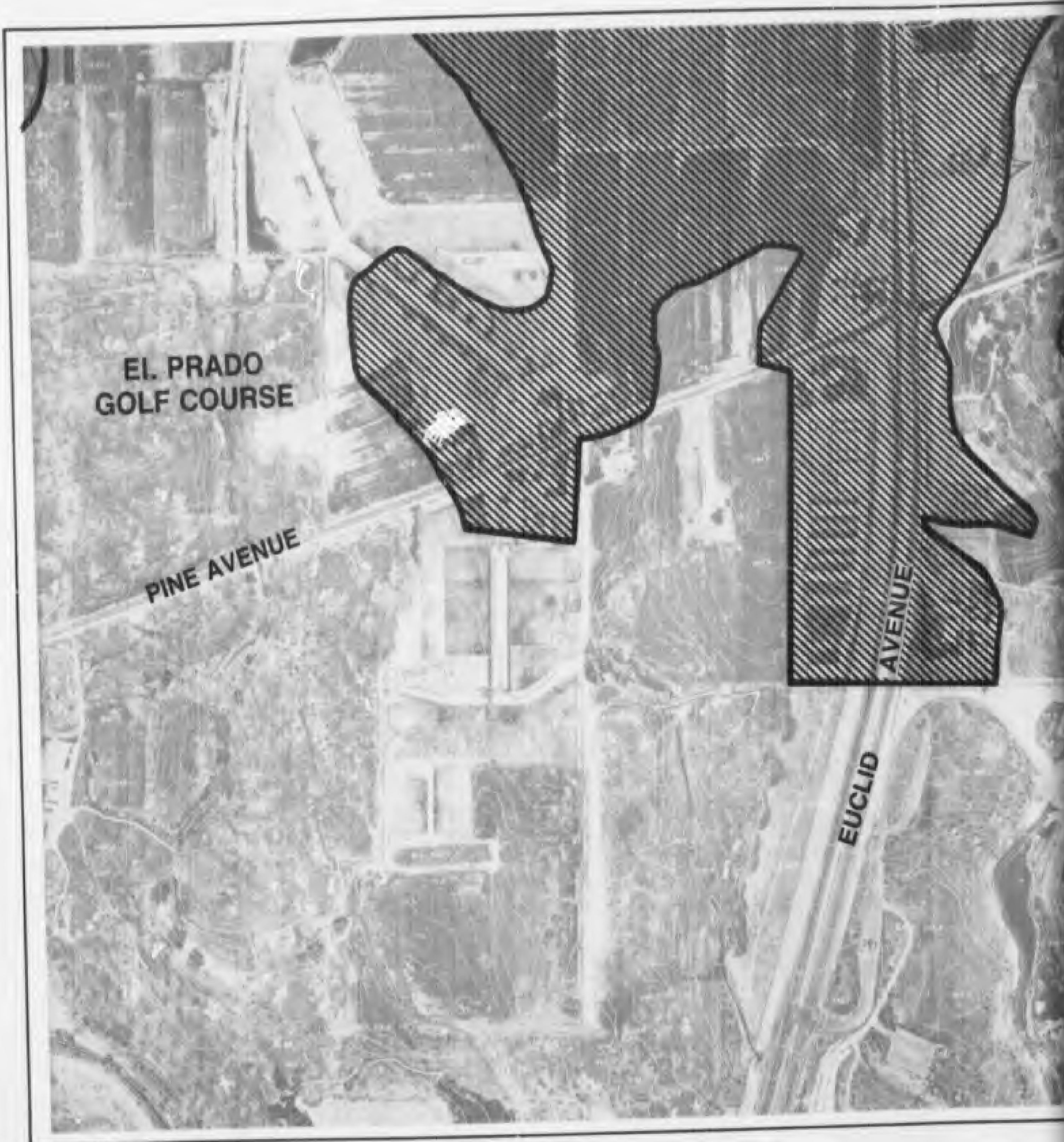
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

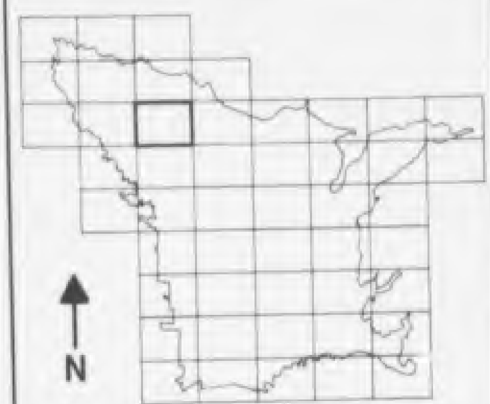
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-121

2


EL PRADO
GOLF COURSE
PINE AVENUE
EUCLID
AVENUE

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

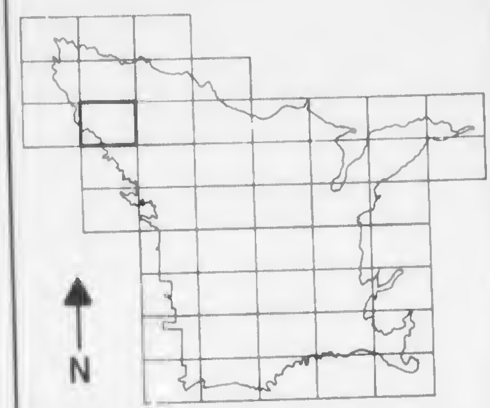
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-119

2


CORONA FREEWAY
(ROUTE 71)

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566′ ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

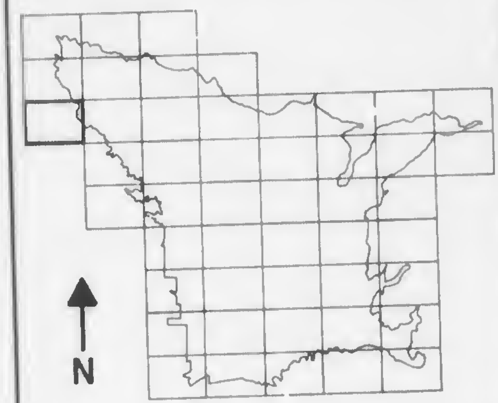
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-145

2

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

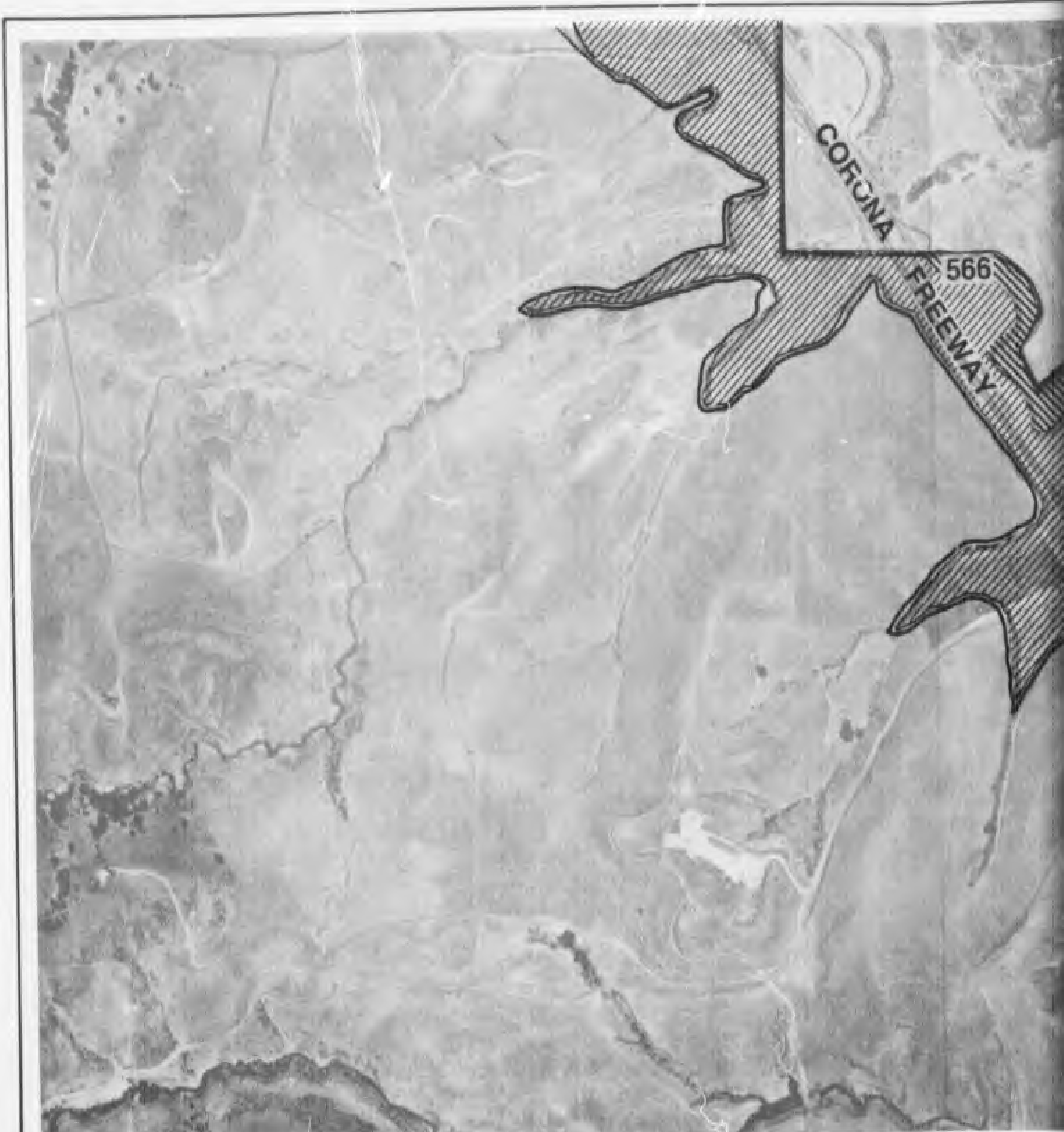
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

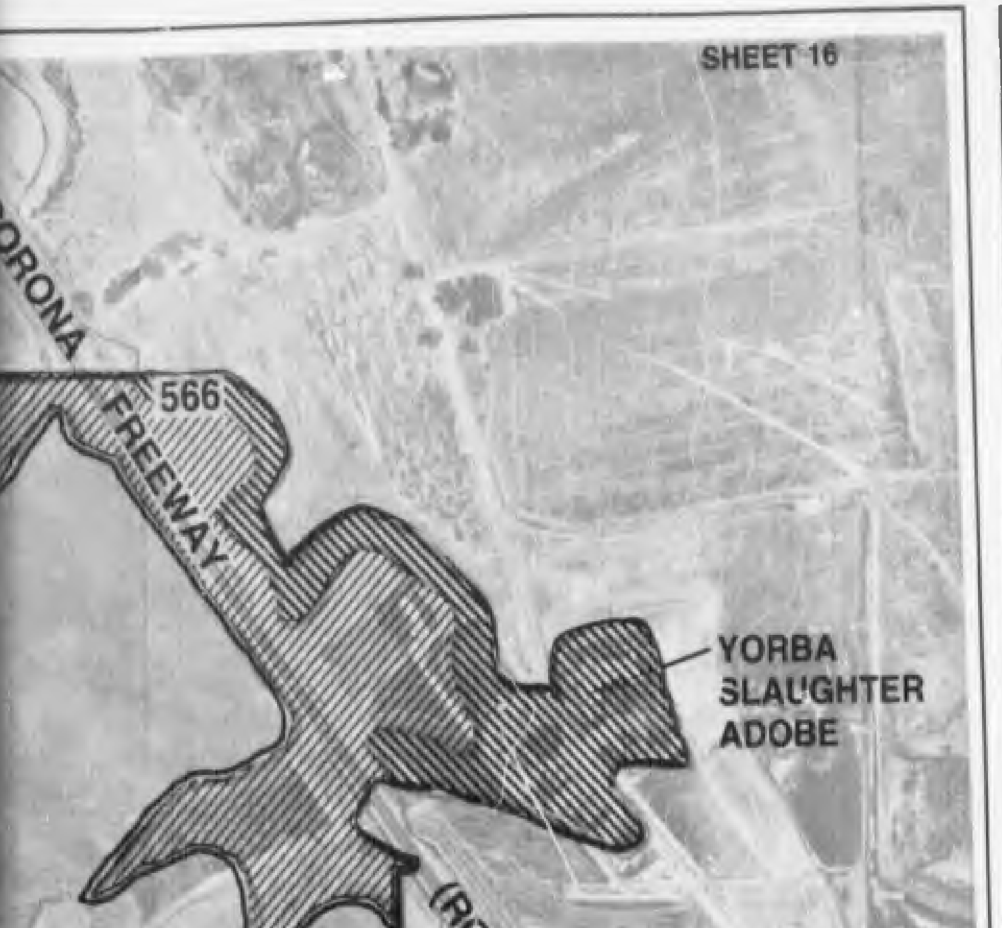
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-146

2



CORONA
FREEWAY
566



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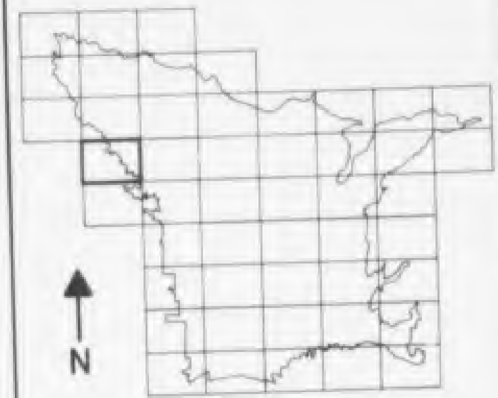


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6 (566′ ELEVATION) AQUISITION LIMITS

ALTERNATIVE 5 (580′ ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582′ ELEVATION) AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

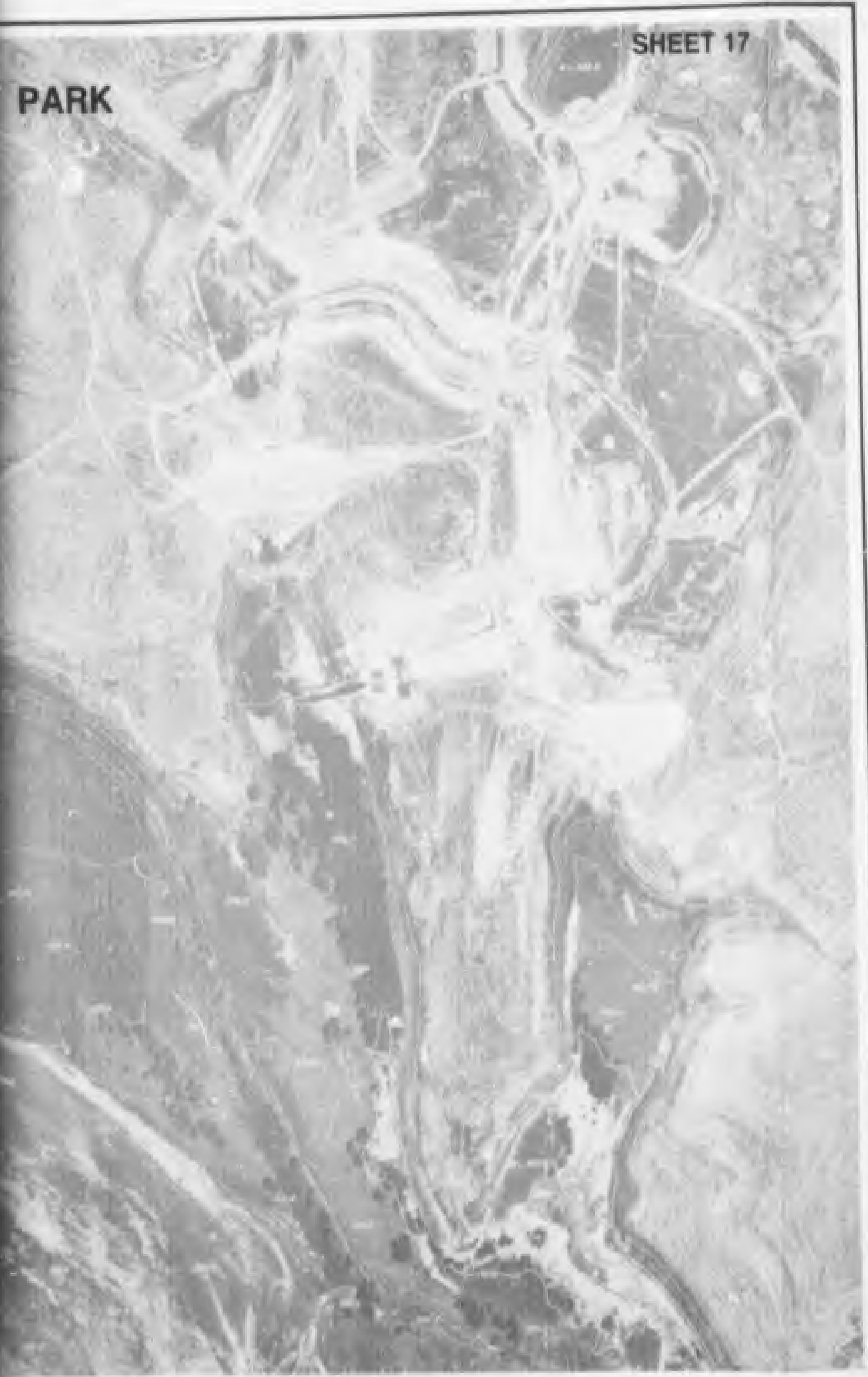
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-147

2


PARK
CHINO
CREEK
AVENUE
EUCLID



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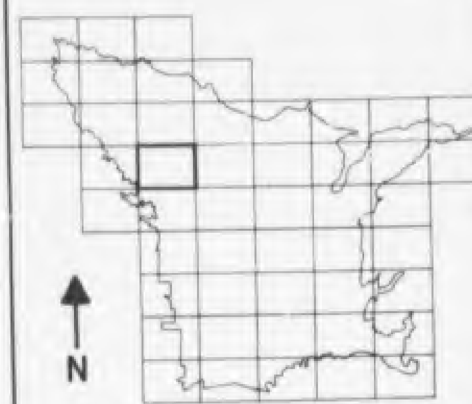


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

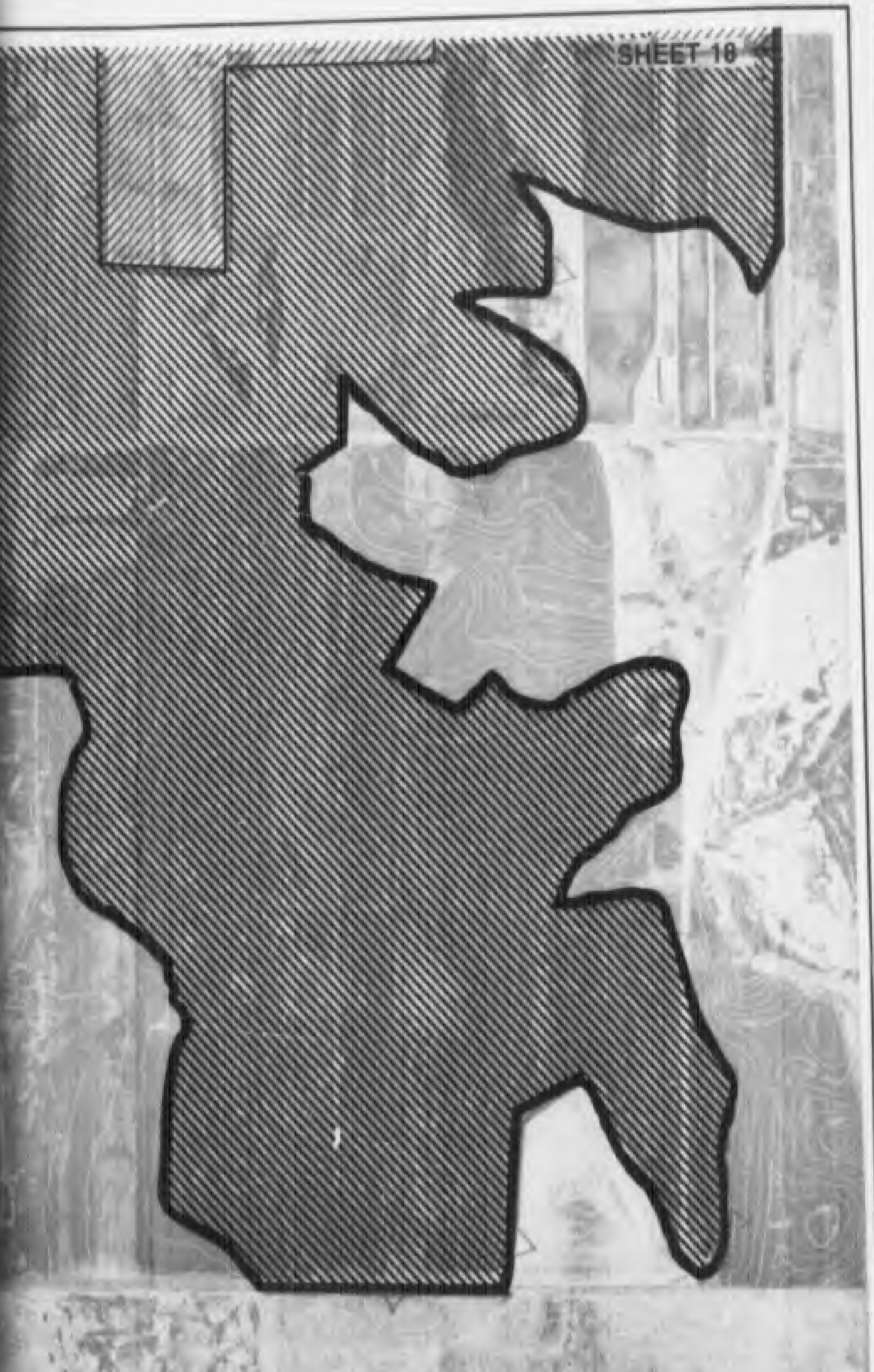
SCALE
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

2



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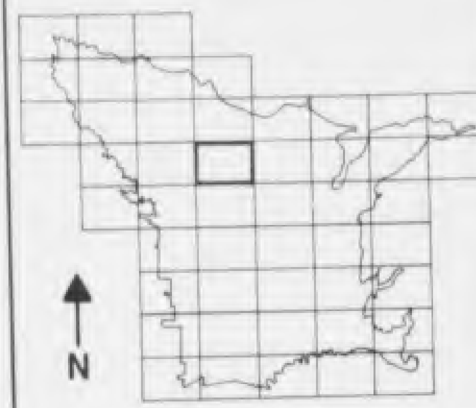


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

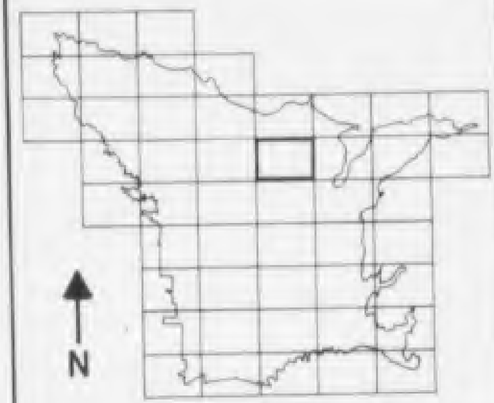
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-148

2


SAN BERNARDINO CO.
RIVERSIDE COUNTY
HELLMAN AVENUE

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566′ ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

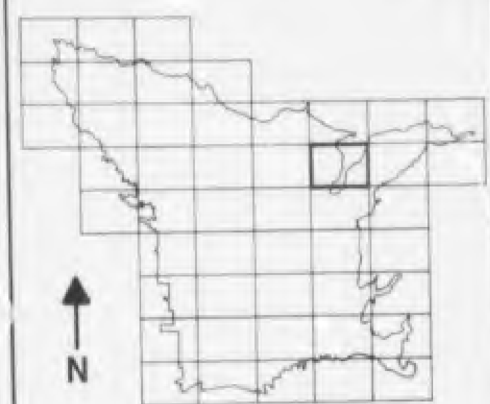
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-141

2

566
580
582
ARCHIBALD AVENUE

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6 (566' ELEVATION) AQUISITION LIMITS

ALTERNATIVE 5 (580' ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582' ELEVATION) AQUISITION LIMITS

SCALE

500' 0 500'

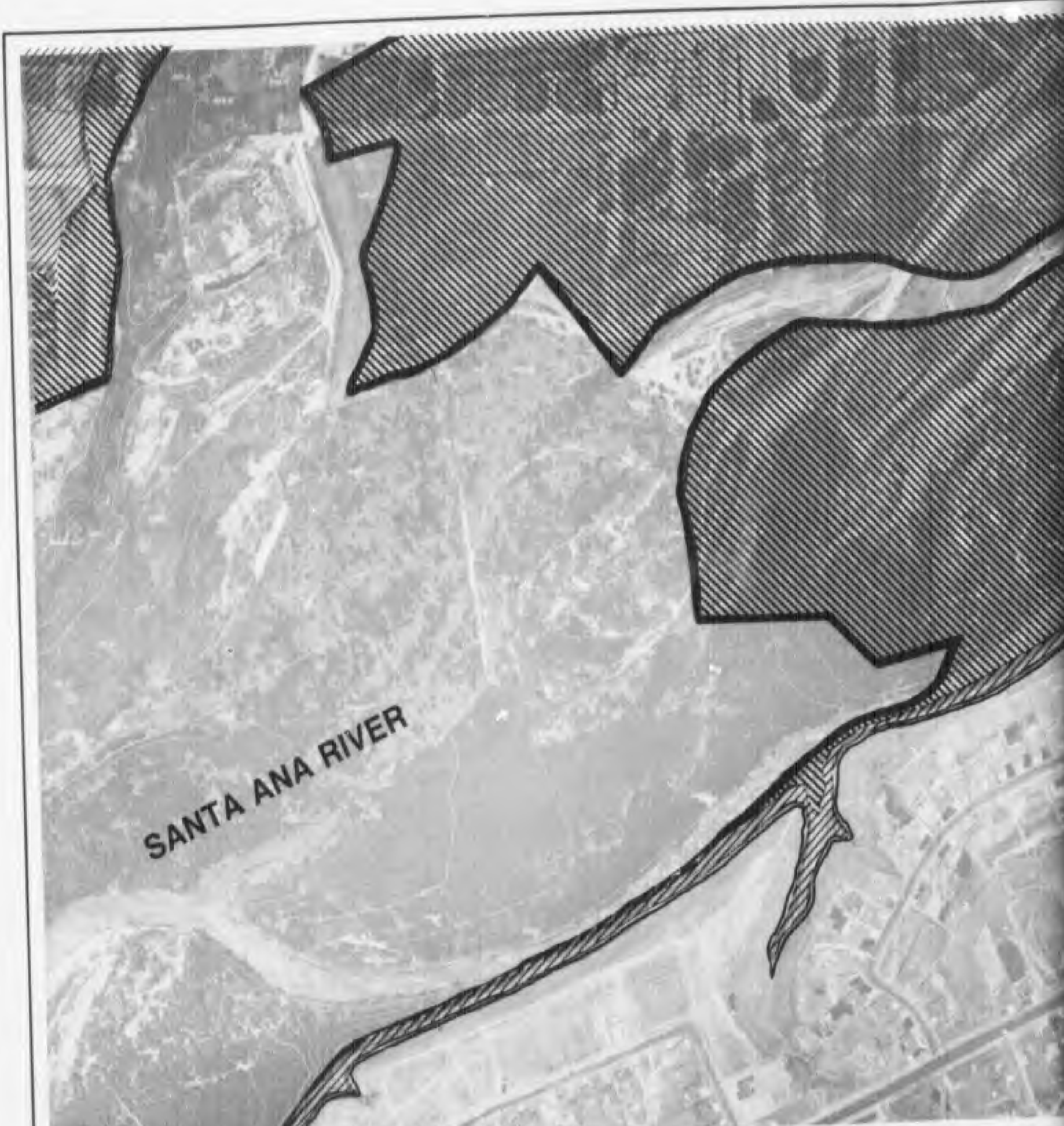
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

5-131

2


SANTA ANA RIVER

KEY

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566′ ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

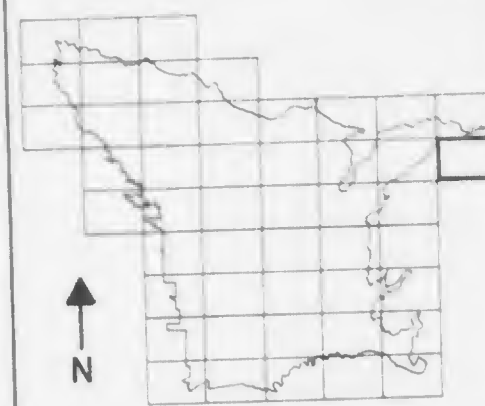
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-101

2


HAMNER AVENUE
SIXTH

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S 102

2


CALIFORNIA REHABILITATIO
CENTER

KEY

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(58?' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

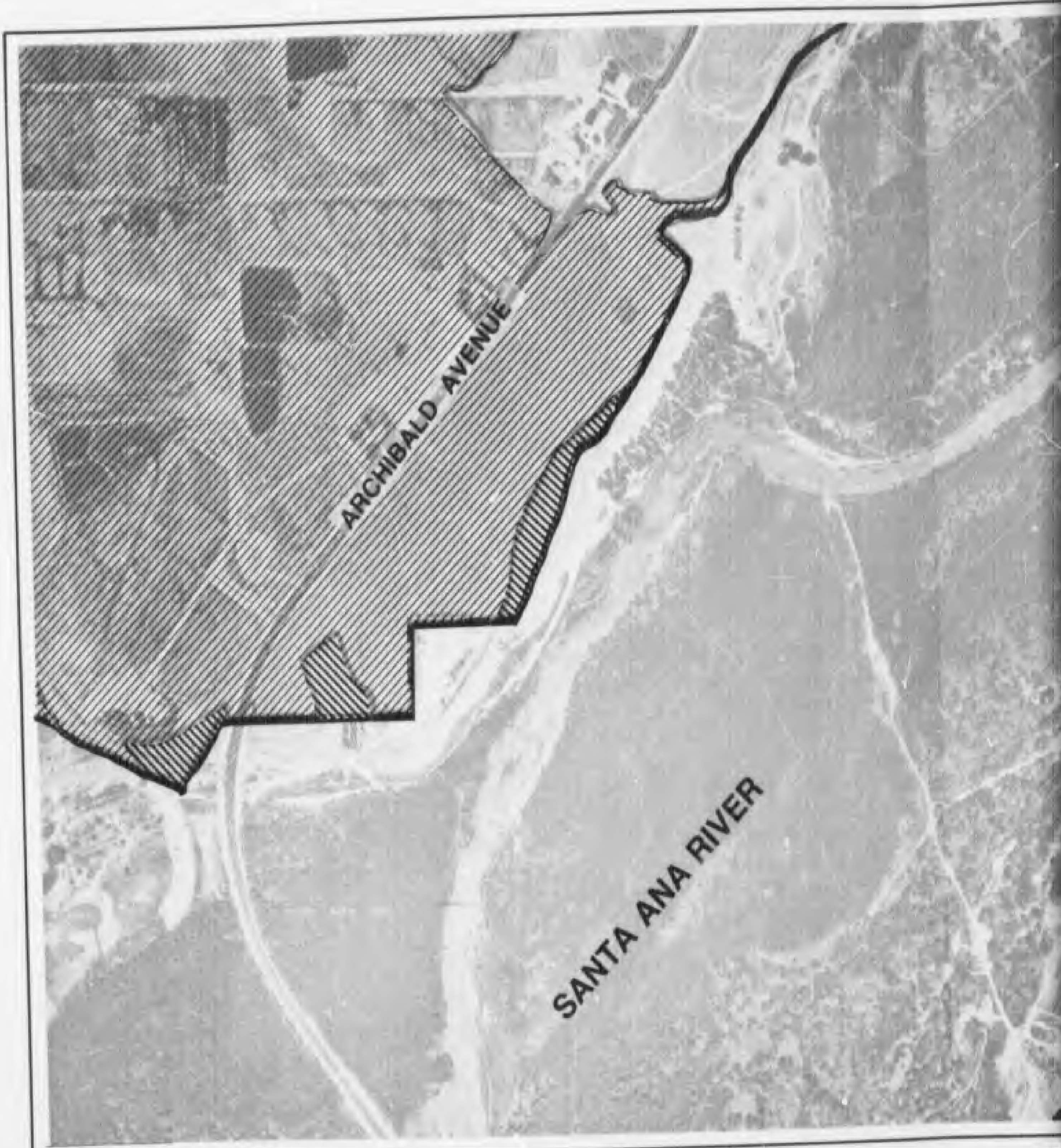
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

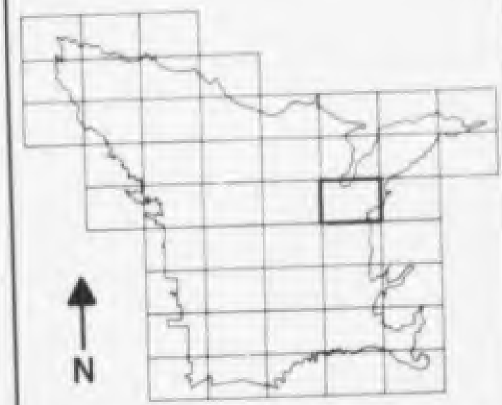
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-129

2


ARCHIBALD AVENUE
SANTA ANA RIVER

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

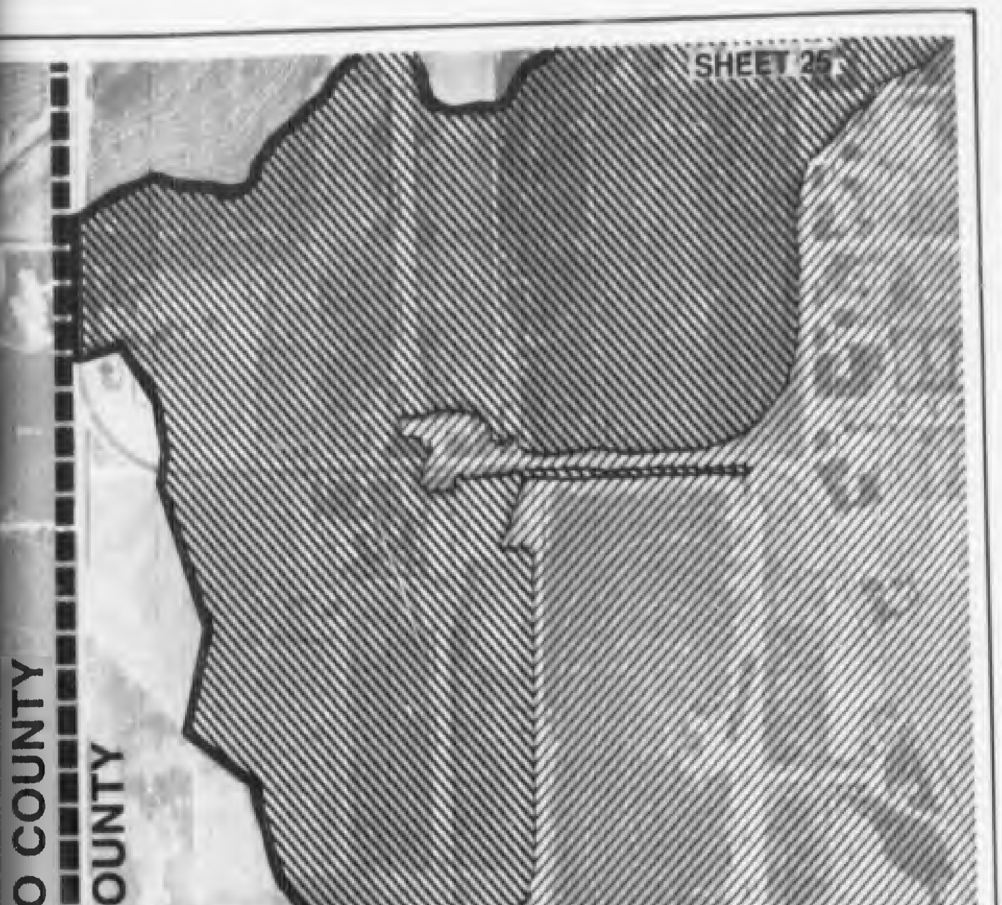
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-103


550
SAN BERNARDINO COUNTY
RIVERSIDE COUNTY



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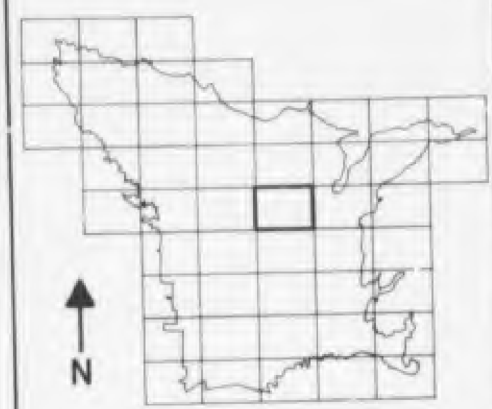


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

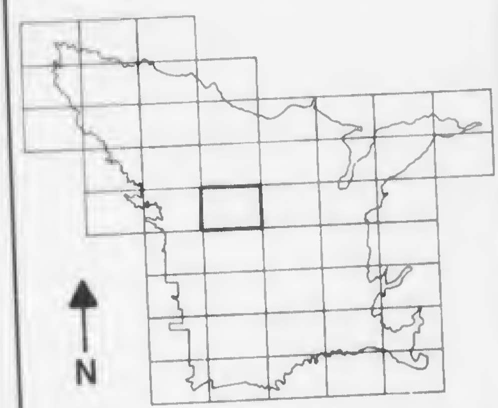
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-126

2


500
SAN BERNARDINO COU
RIVERSIDE COUNTY

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566′ ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-107

2


EUCLID AVENUE
CORONA FREEWAY
500

KEY

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

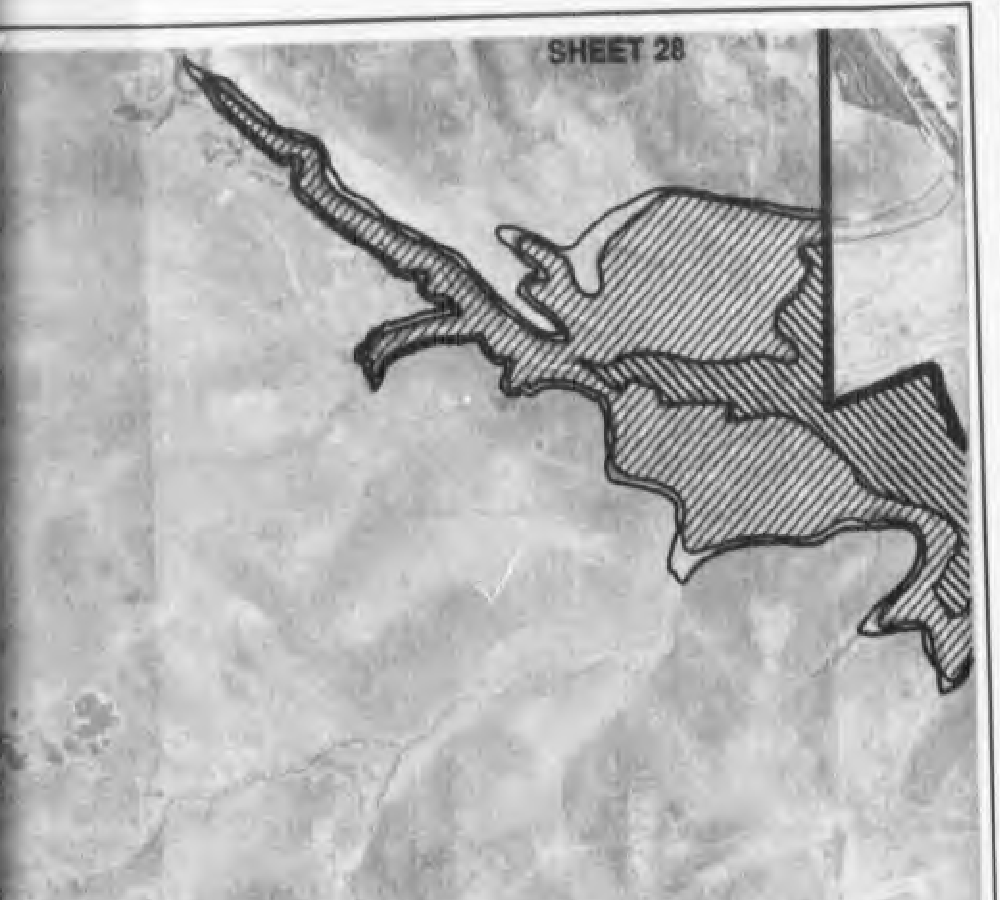
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-132



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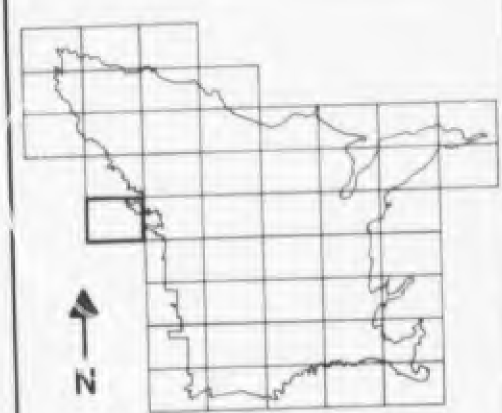


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

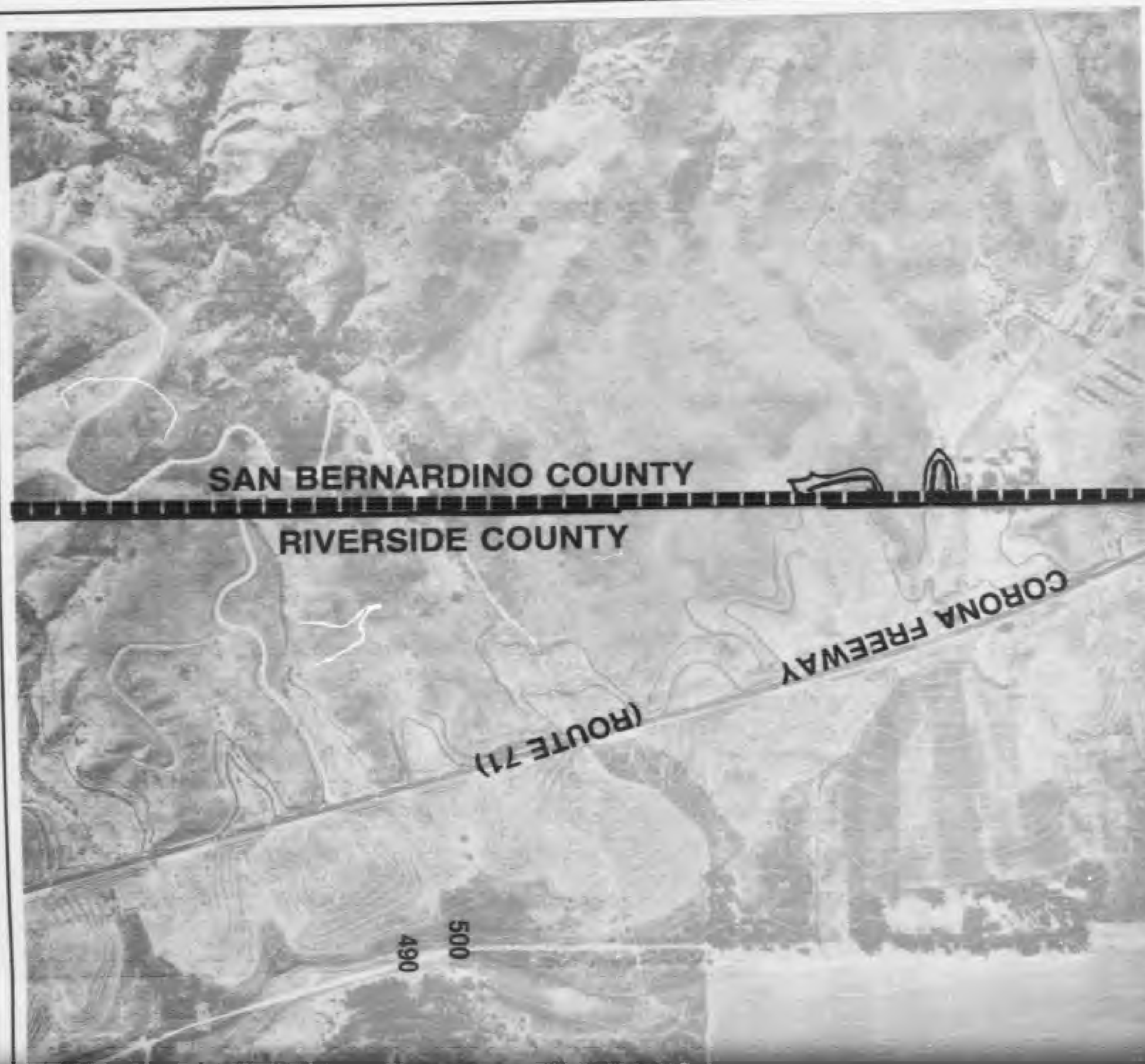
SCALE
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

2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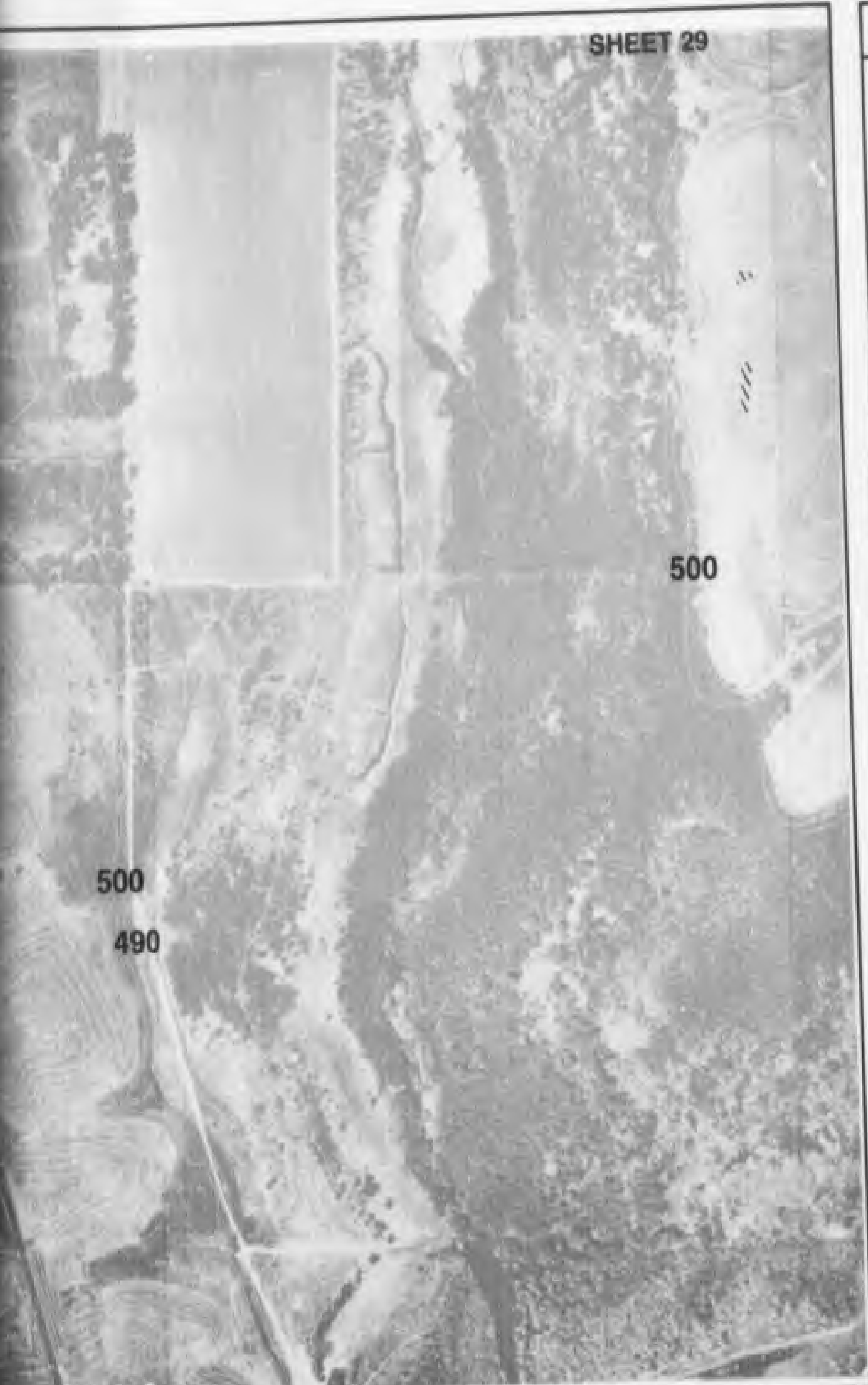
RIVERSIDE COUNTY
CORONA FREEWAY
(ROUTE 71)
500
490



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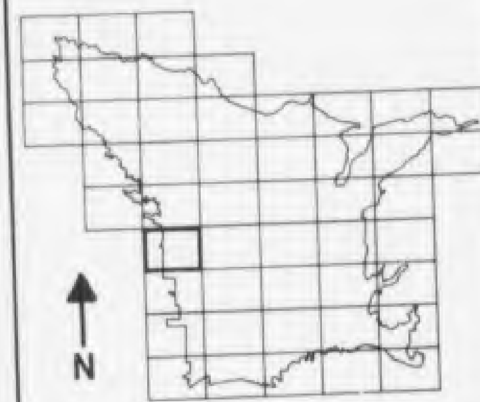

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
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AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

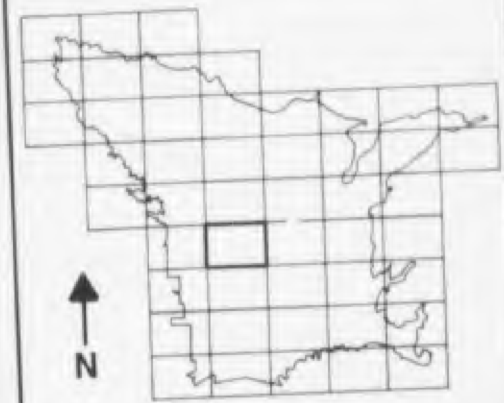
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-120

2

500
500
500
490

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
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AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

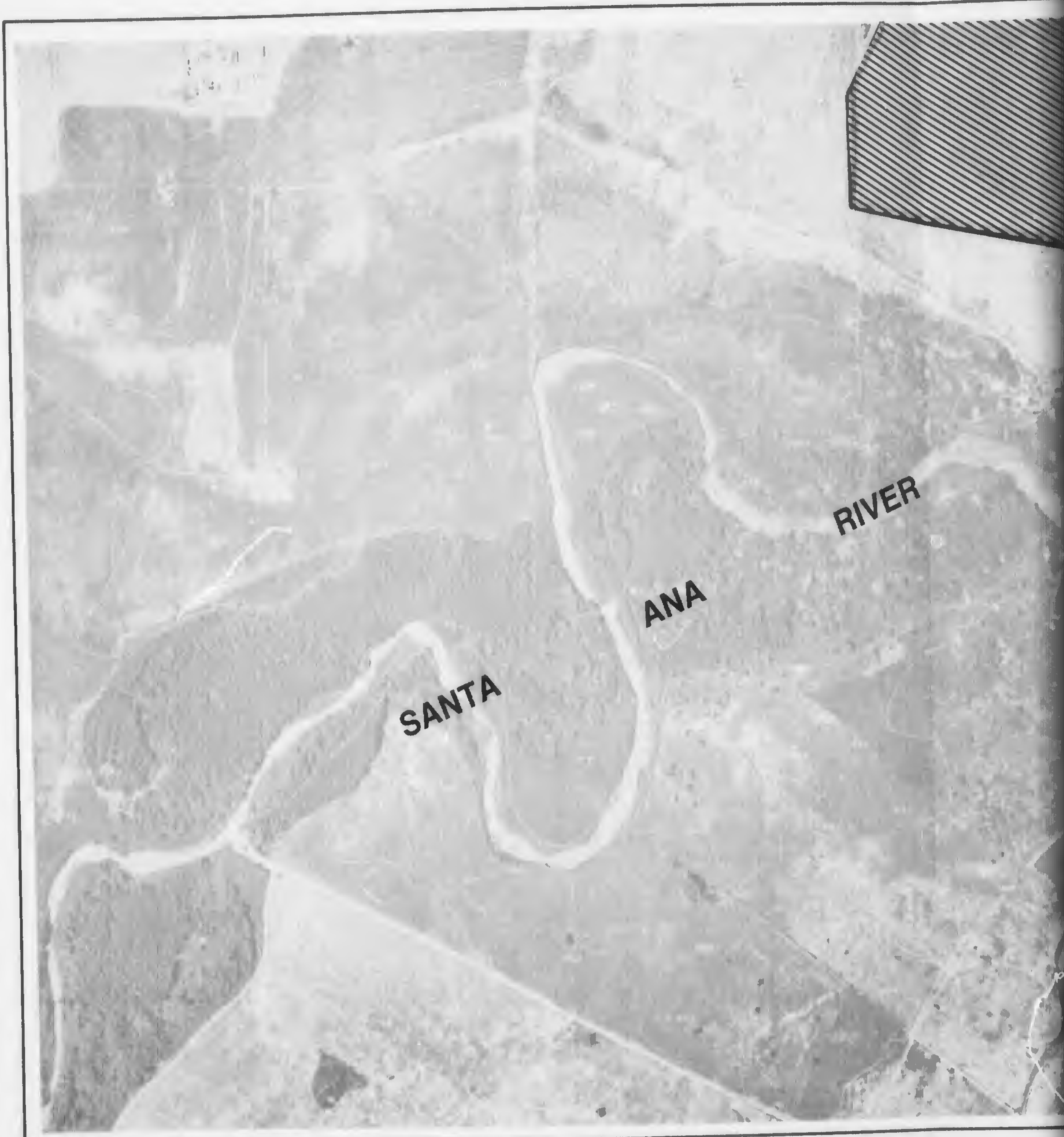
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

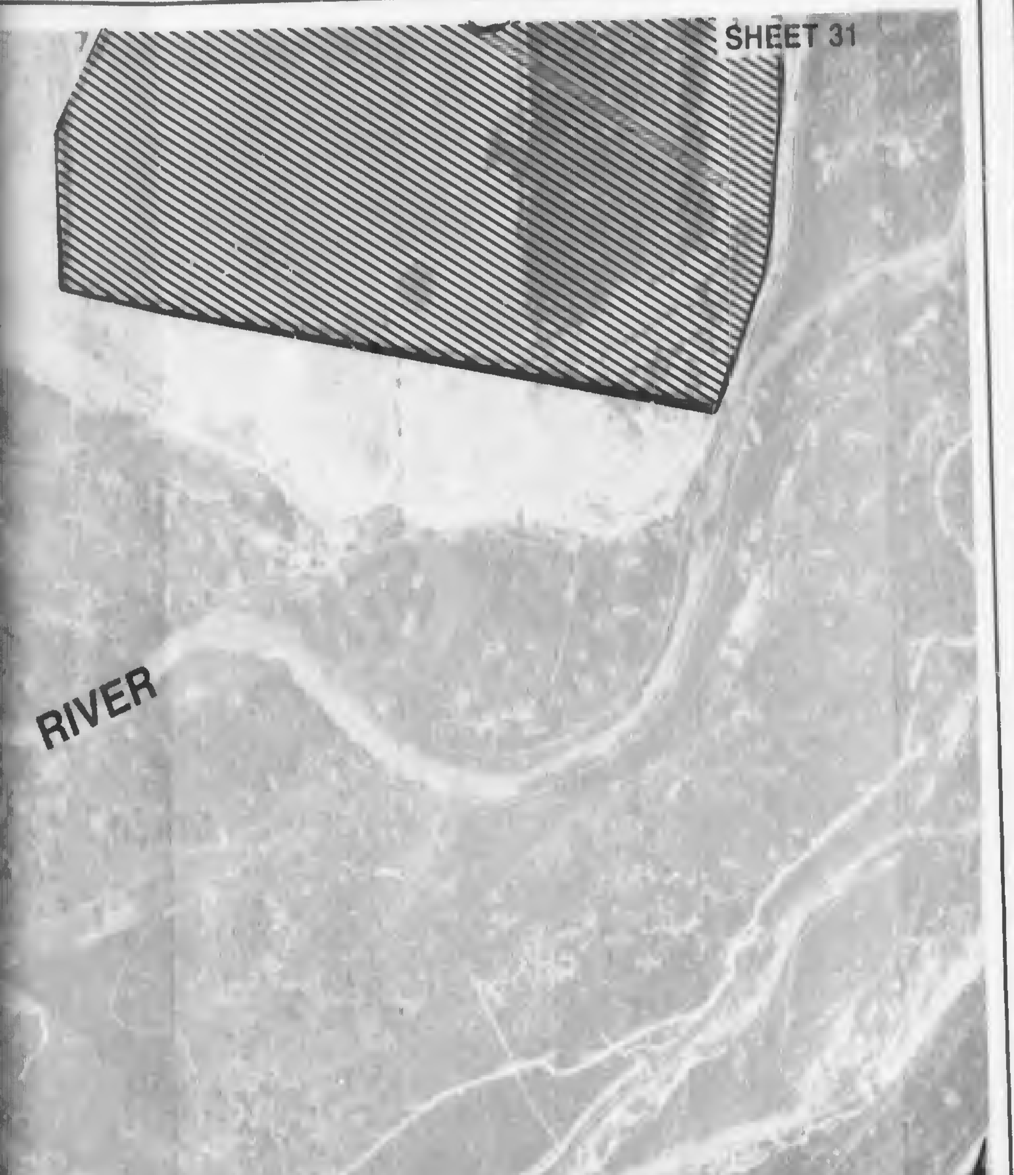
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-106

2



RIVER
ANA
SANTA



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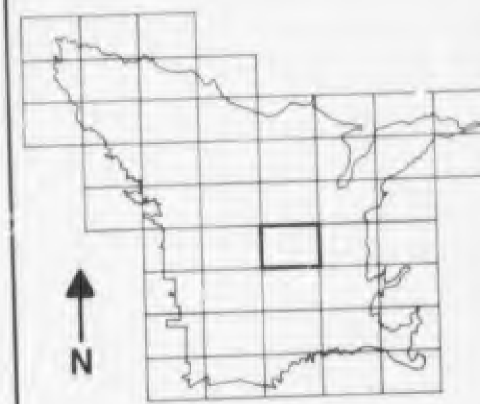

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EXISTING RESERVOIR

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ALTERNATIVE 5 (580′ ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582′ ELEVATION) AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
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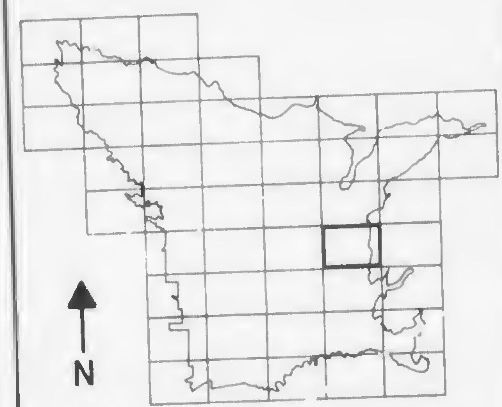
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT


SANTA ANA RIVER
BLUFF
WAGON WHEEL ROAD
STREET
ROUNDUP ROAD
TRAIL
STREET
BLUFF
580

KEY



LEGEND

EXISTING RESERVOIR

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ALTERNATIVE 7 (582′ ELEVATION) AQUISITION LIMITS

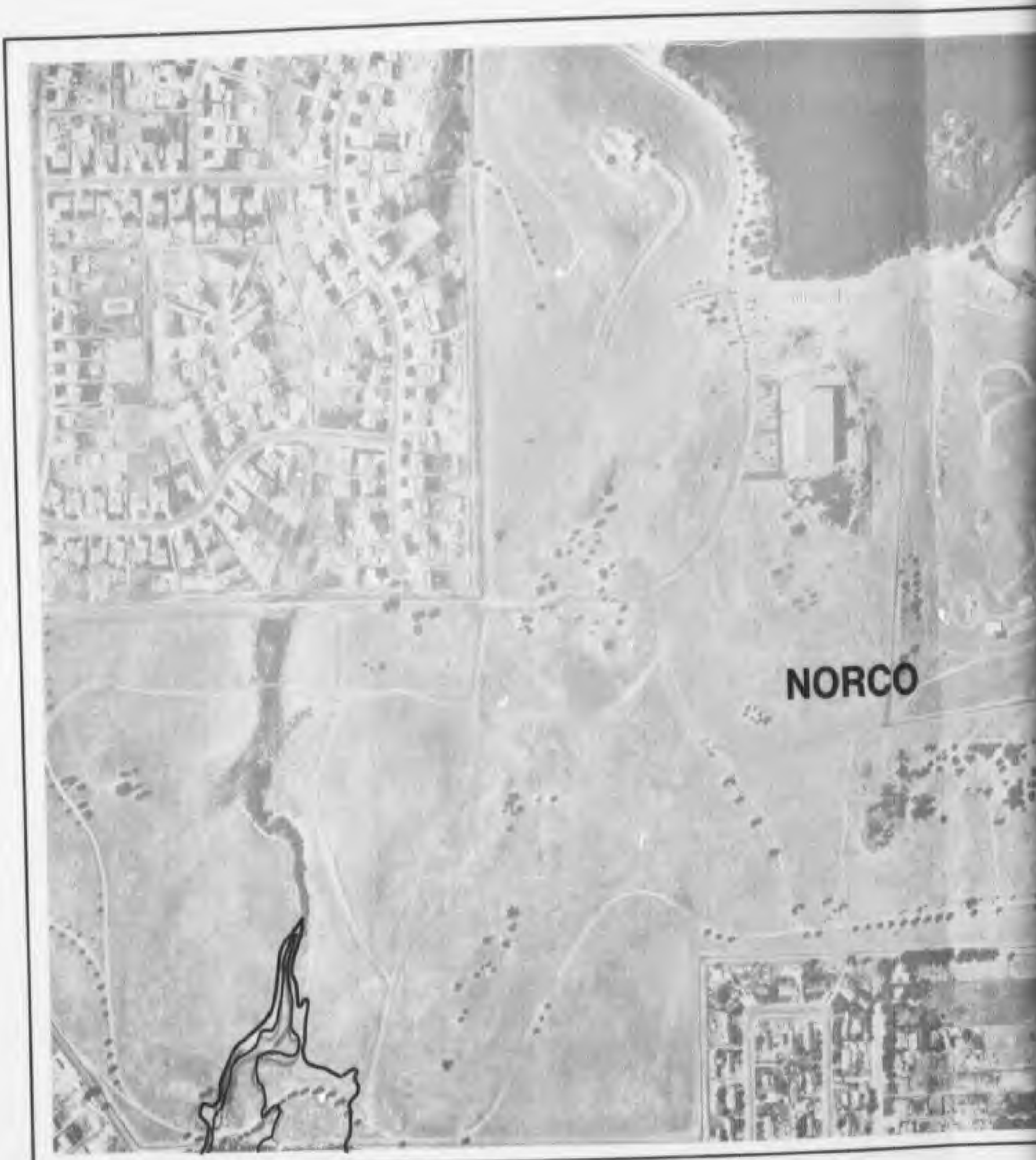
SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT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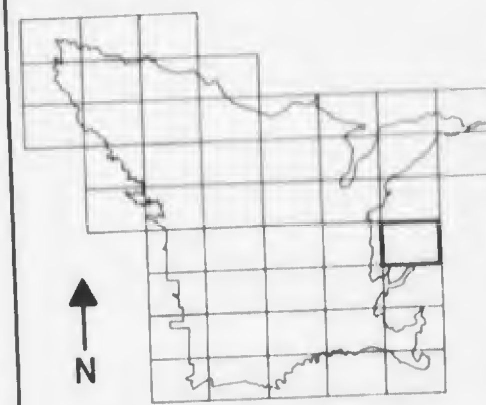

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
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AQUISITION LIMITS

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(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

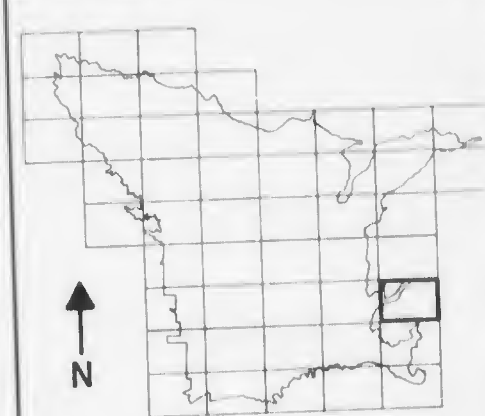
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-108

2


582
580
580
PARKRIDGE AVENUE

KEY



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AQUISITION LIMITS

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AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

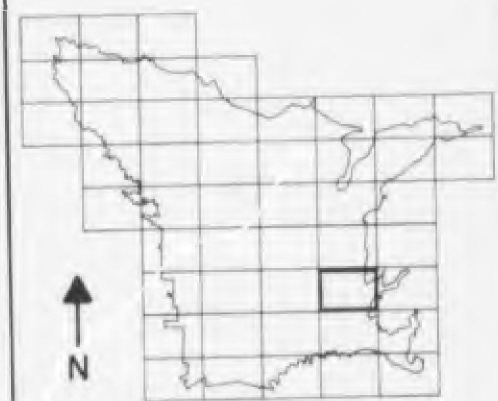
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-111

KEY



LEGEND

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AQUISITION LIMITS

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AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

SCALE

500′ 0 500′

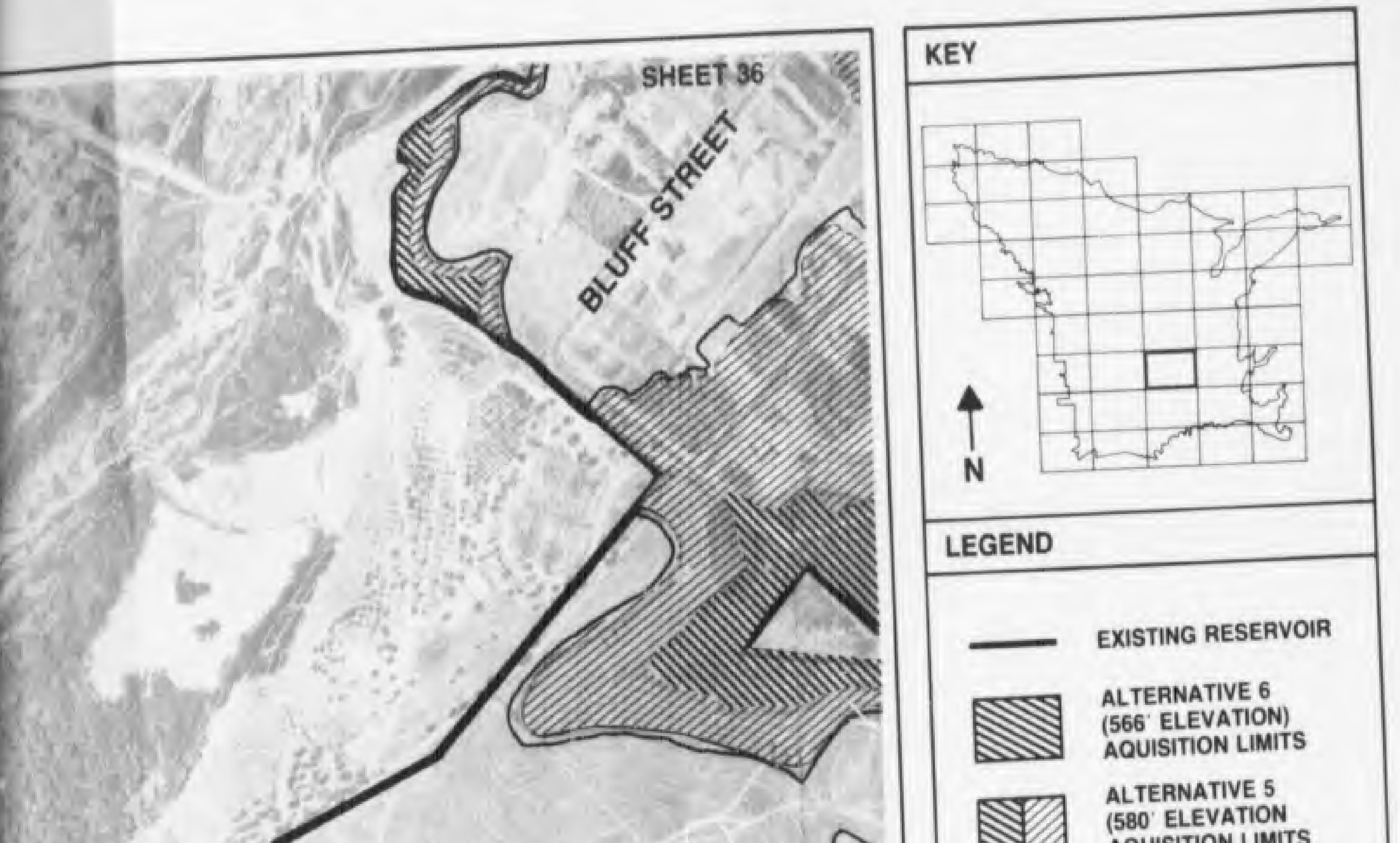
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-135

2


SHEET 36
BLUFF STREET
KEY
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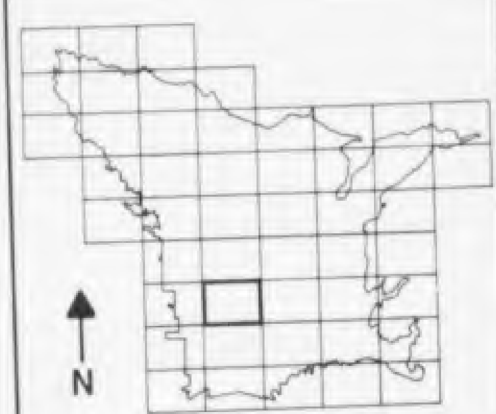
LEGEND
EXISTING RESERVOIR
ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS
ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS
ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS
SCALE
500' 0 500'
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR
ENLARGEMENT PLANS
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

5-136

2


490
RIVER
ANA
SANTA
490

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

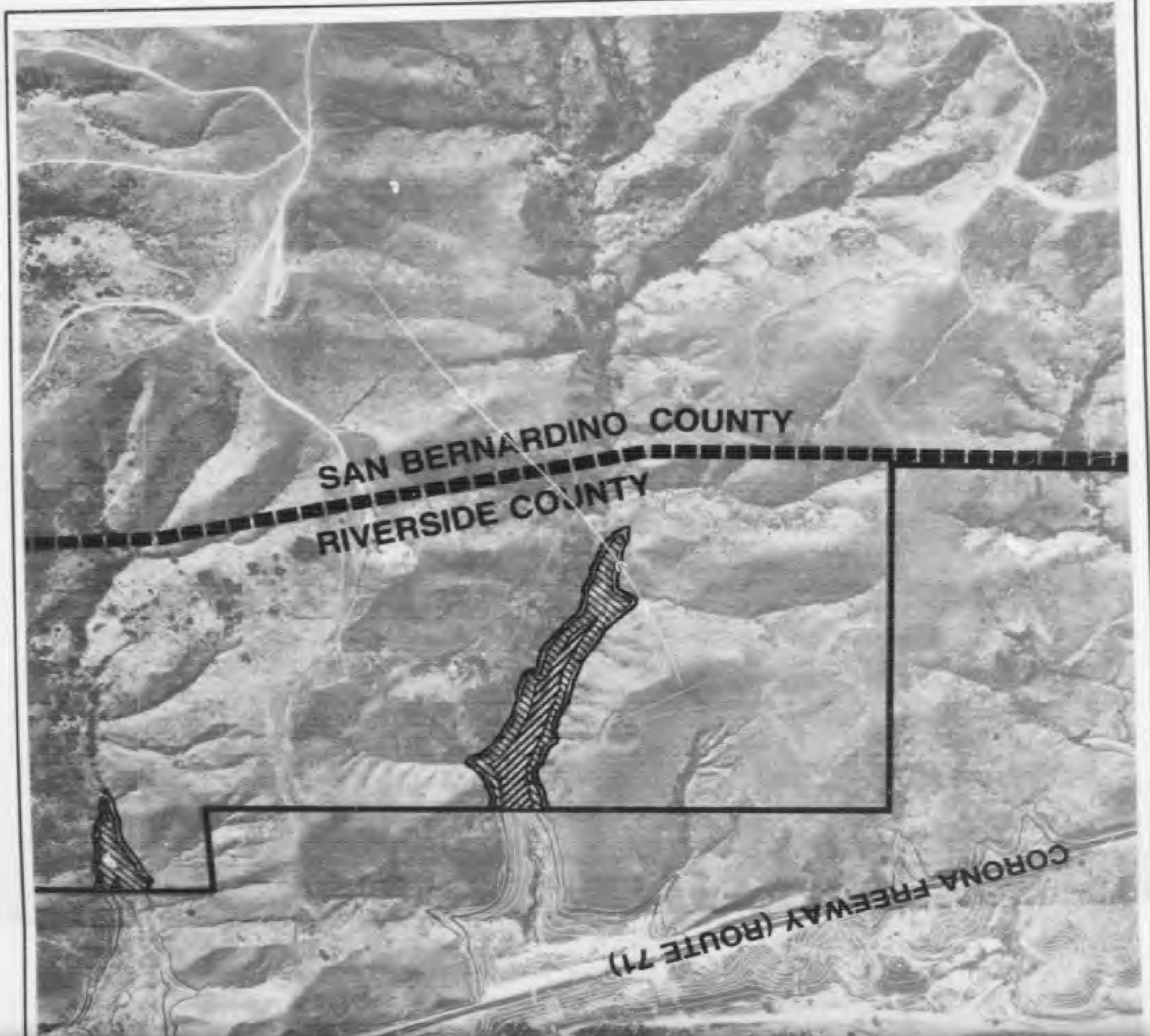
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-105

2



SAN BERNARDINO COUNTY
RIVERSIDE COUNTY
CORONA FREEWAY (ROUTE 71)

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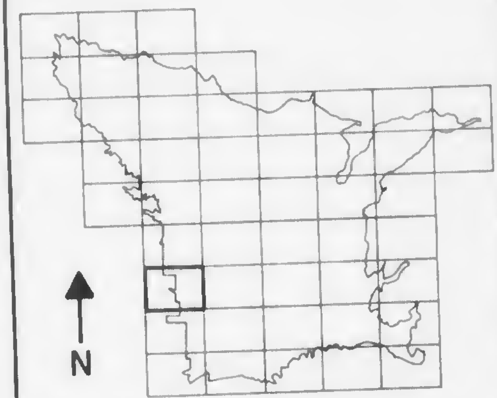


LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566′ ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580′ ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582′ ELEVATION)
AQUISITION LIMITS

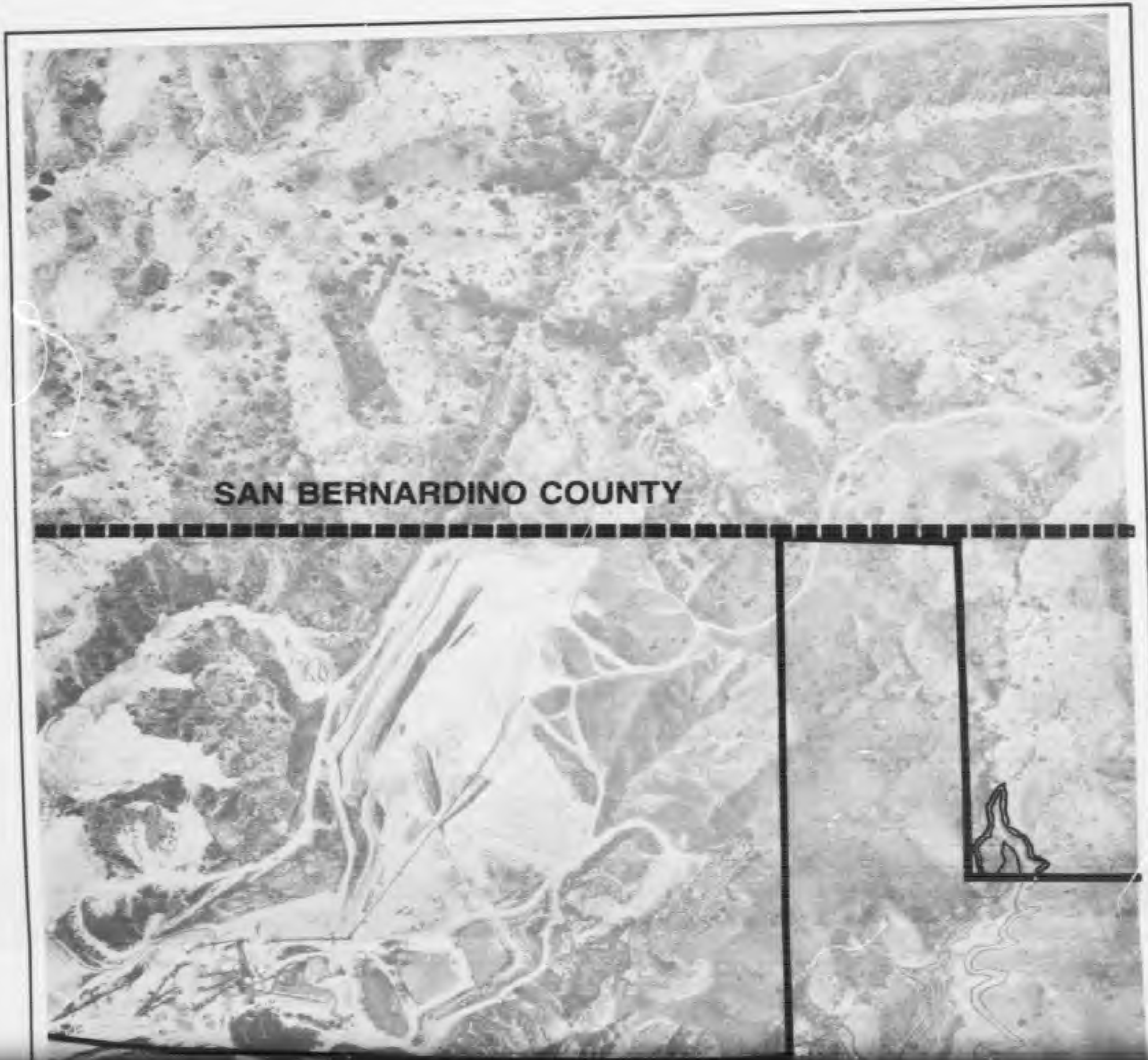
SCALE

500′ 0 500′

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT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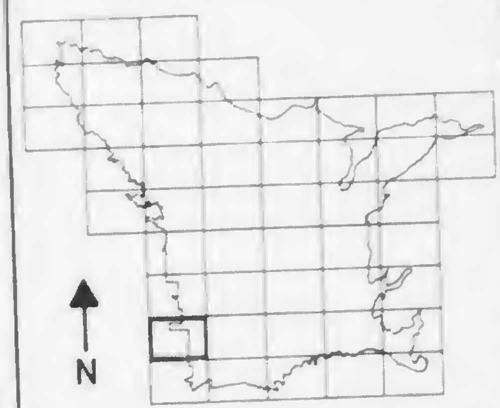

LEGEND

EXISTING RESERVOIR

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AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
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GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

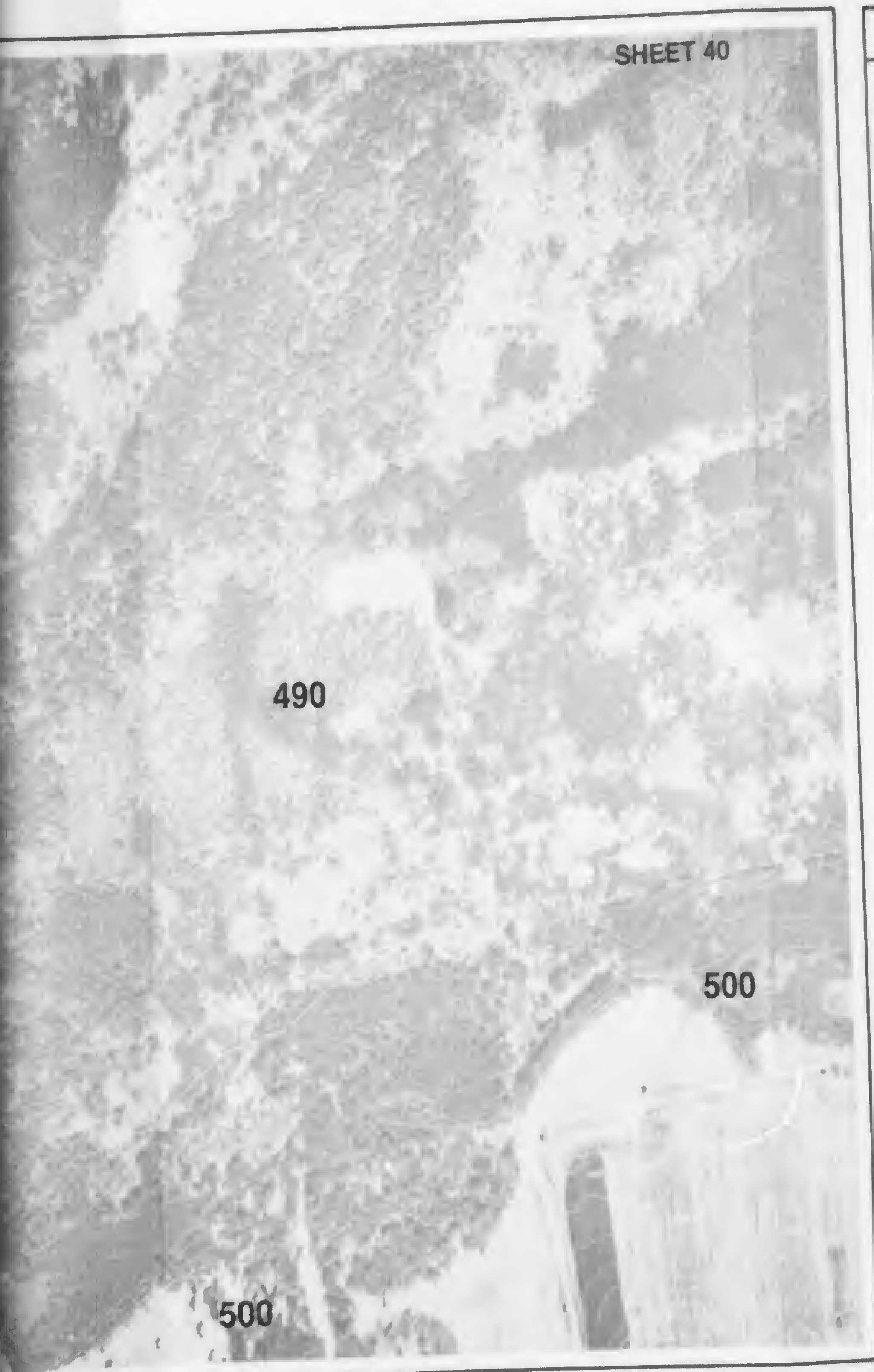
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-104

2


PRADO DAM
500



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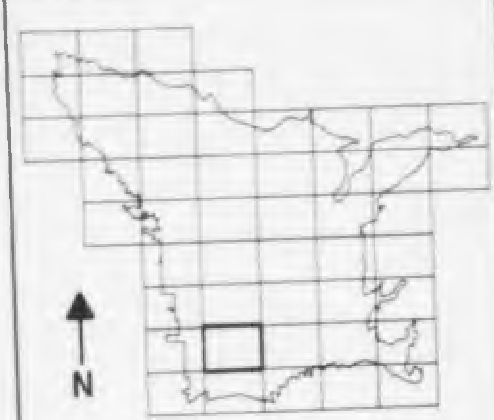

LEGEND

EXISTING RESERVOIR

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AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

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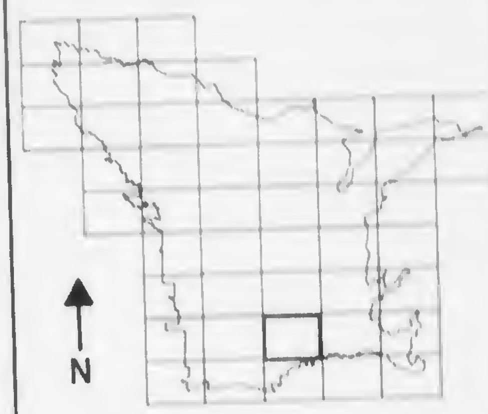
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-127

2


510
510
520
PROPOSED
DIKE
SEWAGE
PLANT

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
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AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582 ELEVATION)
AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
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PRADO RESERVOIR

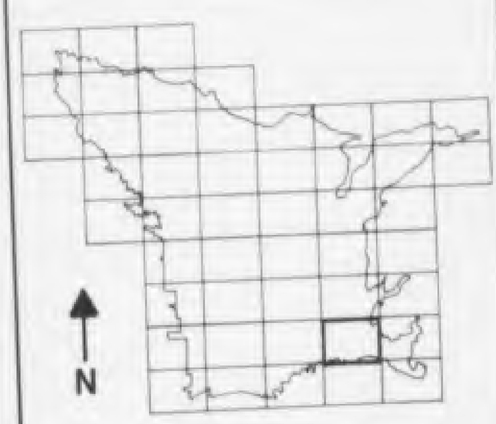
ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

2


GOLF COURS
530
CORONA AIRPORT
DIK
540
PROPOSED
580

KEY



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ALTERNATIVE 5 (580' ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582' ELEVATION) AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
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PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-130

2


582
RIVER ROAD
580
TEMESCAL WASH
580

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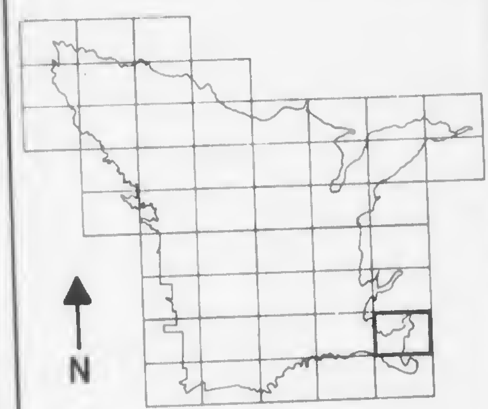

LEGEND

EXISTING RESERVOIR

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ALTERNATIVE 5 (580′ ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582′ ELEVATION) AQUISITION LIMITS

SCALE

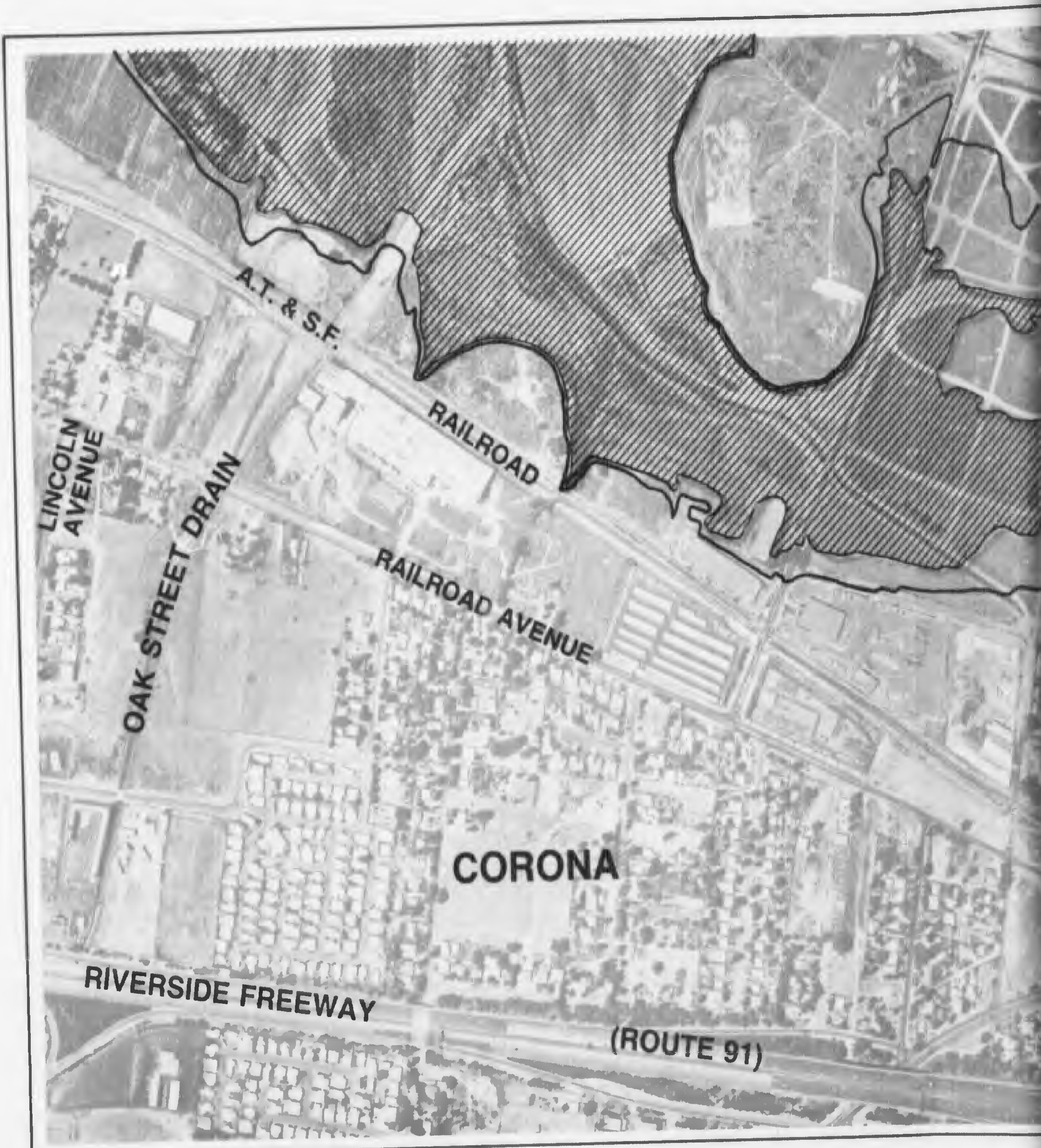
500′ 0 500′

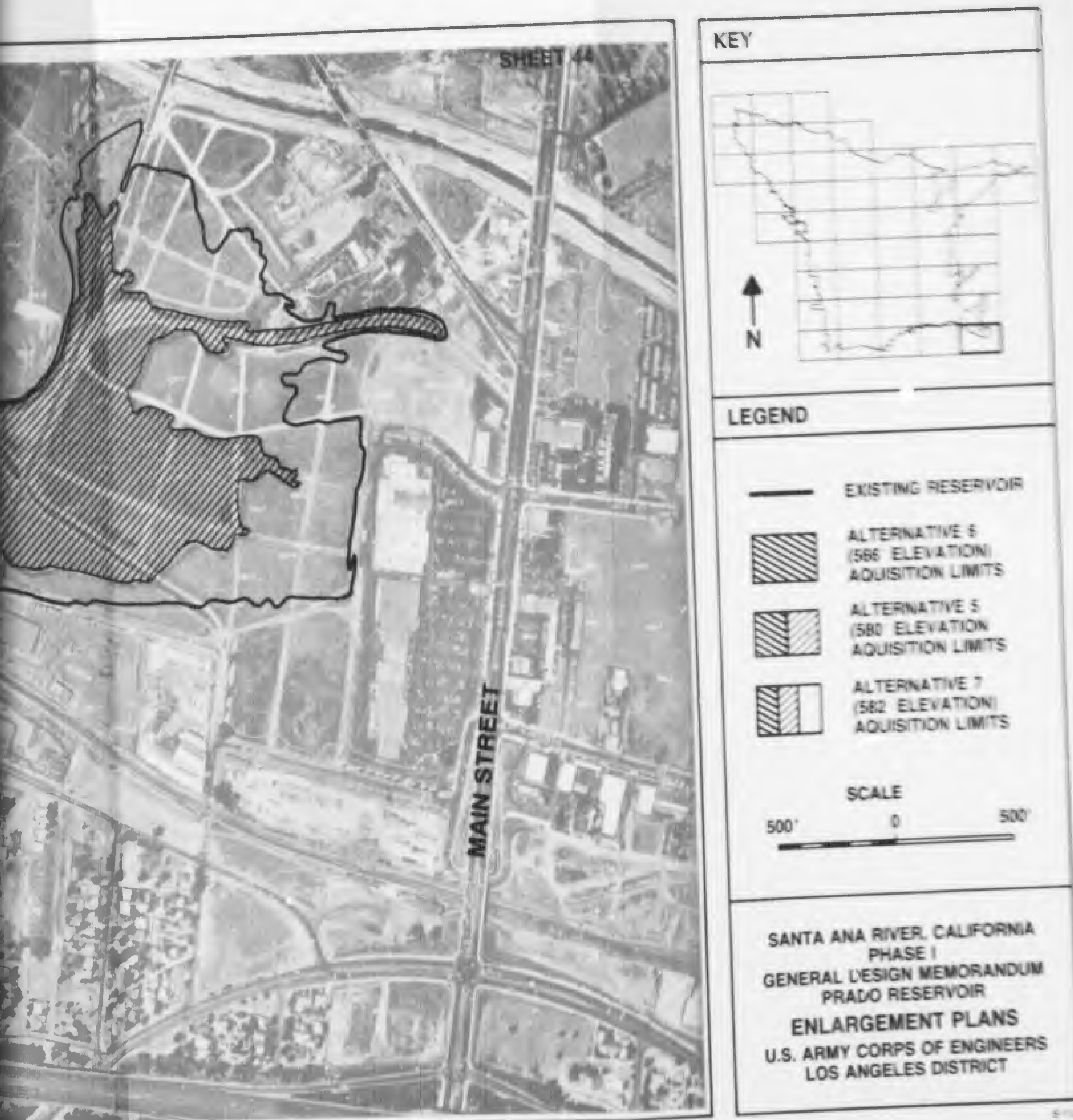
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-138

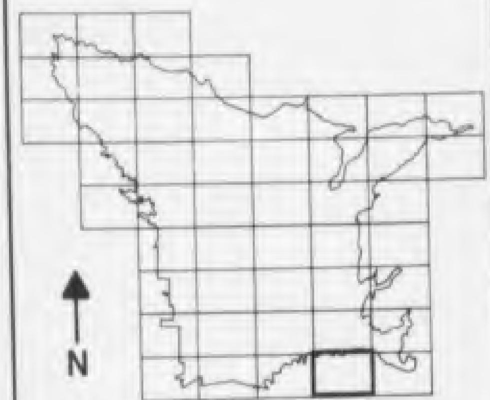

A.T. & S.F.
RAILROAD
LINCOLN AVENUE
OAK STREET DRAIN
RAILROAD AVENUE
CORONA
RIVERSIDE FREEWAY
(ROUTE 91)


SHEET 44
MAIN STREET
KEY
N
LEGEND
EXISTING RESERVOIR
ALTERNATIVE 6 (566 ELEVATION) AQUISITION LIMITS
ALTERNATIVE 5 (580 ELEVATION AQUISITION LIMITS
ALTERNATIVE 7 (582 ELEVATION) AQUISITION LIMITS
SCALE
500' 0 500'
SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR
ENLARGEMENT PLANS
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

2


A.T. & S.F. RAILROAD
SMITH AVENUE
RIVERSIDE FREEWAY
(ROUTE

KEY



LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6 (566' ELEVATION) AQUISITION LIMITS

ALTERNATIVE 5 (580' ELEVATION AQUISITION LIMITS

ALTERNATIVE 7 (582' ELEVATION) AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-139

2


A.T. & S.F. RAILROAD
RIVERSIDE FREEWAY
(ROUTE 91)

KEY

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE

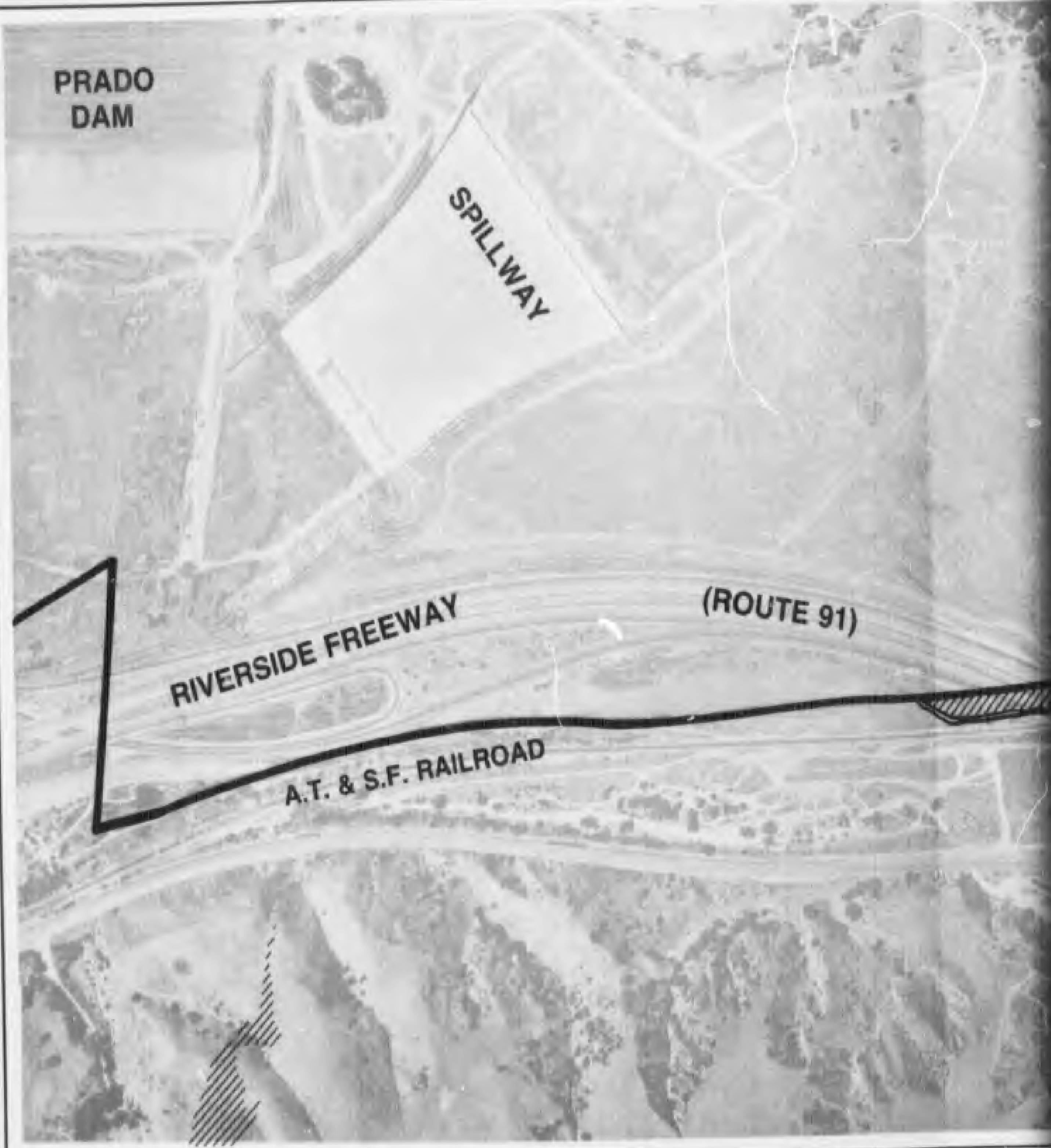
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

2


PRADO DAM
SPILLWAY
RIVERSIDE FREEWAY
(ROUTE 91)
A.T. & S.F. RAILROAD

KEY

LEGEND

- EXISTING RESERVOIR
- ALTERNATIVE 6 (566' ELEVATION) AQUISITION LIMITS
- ALTERNATIVE 5 (580' ELEVATION AQUISITION LIMITS
- ALTERNATIVE 7 (582' ELEVATION) AQUISITION LIMITS

SCALE

500' 0 500'

SANTA ANA RIVER, CALIFORNIA
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PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

S-140

2


SANTA
ANA
RIVER
RIVERSIDE FREEWAY (ROUT

KEY

LEGEND

EXISTING RESERVOIR

ALTERNATIVE 6
(566' ELEVATION)
AQUISITION LIMITS

ALTERNATIVE 5
(580' ELEVATION
AQUISITION LIMITS

ALTERNATIVE 7
(582' ELEVATION)
AQUISITION LIMITS

SCALE
500' 0 500'

SANTA ANA RIVER, CALIFORNIA
PHASE I
GENERAL DESIGN MEMORANDUM
PRADO RESERVOIR

ENLARGEMENT PLANS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

F Design & Cost Estimates

DESIGN AND COST ESTIMATE

APPENDIX F

U. S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

To accompany Phase I General Design Memorandum for

Santa Ana river, California

APPENDIX F
DESIGN AND COST ESTIMATES
CONTENTS

Page

Contents (Continued)

Contents (Continued)

Tables

Plates

Plates (Continued)

Appendix F
DESIGN AND COST ESTIMATES
I. INTRODUCTION

1.01 The recommended project plan would improve flood control along the Santa Ana River from the foot of the San Gabriel Mountains to the Pacific Ocean. The plan would include construction of Mentone Dam, improvement of Oak Street Drain and its tributaries, modification of Prado Dam, and improvement of Santiago Creek and the Santa Ana River below Prado Dam. Improvement of the Santa Ana River between Mentone Dam and Prado Dam would not be economically justifiable; therefore, the application of flood plain zoning, flood proofing and flood insurance is recommended for the prevention and reduction of future flood damages on the unimproved reach. Features of the recommended flood control plan are presented in the following pages of this report.

II. MENTONE DAM

DAM UNDER THE ALL RIVER PLAN

General

2.01 Mentone Dam would be located approximately two miles downstream of the confluence between Mill Creek and the Santa Ana River north of the city of Redlands in the southwestern part of San Bernardino County, California. This damsite was recommended in the Review Report dated December 1975, and the site is still considered the best after evaluation of the size of dam needed and location of the San Andreas Fault. At this site, the dam would control runoffs from a drainage area of 260 square miles. Major features of the dam would include the embankment, spillway, outlet works, and a low flow channel. In addition the existing Mill Creek levee would be raised and extended to divert floodwaters into the basin of Mentone Dam.

Embankment

2.02 With the top of embankment at elevation 1573.5 feet above the mean sea level, the horse shoe-shaped embankment would be approximately 17,200 feet in length, and 220 feet above the existing ground surface. The embankment would have a top width of 66 feet, and side slopes of 1V on 3H. The compacted-multizoned-earthfilled embankment would contain approximately 66-million cubic yards of materials including 10-million cubic yards of core material to be imported from sites at least 30 miles away.

Outlet Works

2.03 The outlet works (see pl. F-7) would consist of the intake structure with a gate operation room and access gallery, a transition structure, conduit under the embankment and a stilling basin.

INTAKE STRUCTURE. Concrete wing walls would be provided at the upstream end of the structure for the retention of embankment materials away from the entrance channel to the outlet conduit. In order to prevent floatable debris from entering into the outlet works, a metal trash rack would be installed on the upstream face of the intake structure. The gate operating room would be directly above the gate chambers where three 5.5 foot by 8.5 foot gates would be housed. An access gallery with stairs and an electric tramcar would provide access from the top of dam to the gate operating room. A slot for stop logs would be provided at the upstream end of the bellmonth.

TRANSITION STRUCTURE. Downstream from the intake structure would be the 145-foot long reinforced concrete transition structure where outflow from the three gate chambers would be conveyed to a single 14-foot diameter conduit.

CONDUIT. The reinforced concrete conduit under the embankment would be 1223.5 feet in length, and would be circular in cross section. The conduit would be designed for four different depths of fill over the conduit.

STILLING BASIN. The concrete stilling basin would be located at the downstream end of the conduit. The basin would have a rectangular cross sections with widths ranging from 14 feet to 38 feet, and wall heights varying from 7 feet to 21.4 feet. The total length of the basin would be 308 feet. Five baffle blocks would be provided at the downstream portion of the basin for dissipation of hydraulic energy.

Gates and Operation Equipments

2.04 The flow of floodwaters through the outlet works would be controlled by a hydraulically operated slidegate located in three rectangular gate passages in the gate chamber of the intake tower. No emergency gates would be provided. Instead stop logs would be utilized for emergency inspection or repair of the service gates. Three 36-inch diameter air vents would be installed near the access gallery to provide the necessary air for the operation of the gates. Chain hoists and jib cranks would be furnished in the operating chambers. An emergency generator would supply the electricity for lighting and operation of the tramcar. The generator would be housed in a control house at the top of the embankment near the outlet works. For details of the gates and operation equipment, see plate F-8.

Spillway

2.05 The spillway site was selected for the following considerations: (1) The spillway would be located within the exisitng streambed of the Santa Ana River, thereby eliminating the need for acquisition of a large area of additional rights-of-way, or flowage easement, between the downstream end of the spillway and the existing river and (2) by delaying the construction of the ogee section, the 6,100-foot-long-rectangular-concrete spillway channel could be utilized for diversion and control of floodflows from the Santa Ana River and Mill Creek during the relocation of the Santa Fe Railroad track and the construction of the outlet works and dam embankment.

The spillway would have a crest length of 1,000 feet, and a mass concrete ogee section with a crest elevation of 1,548.5 feet above the mean sea level. The rectangular concrete lined spillway would have wall heights ranging from one foot to 59 feet, and a length of 6,478 feet.

Spillway walls in the vicinity of the ogee section would have heights ranging from 20 feet to 59 feet. Due to their tremendous heights, the walls would be designed as counterfort walls supported by bearing piles. Walls less than 20 feet in height, would be designed as L-type walls. The spillway, downstream of the ogee section, would be equipped with a subdrainage system.

A downstream energy dissipator would be required at the end of the concrete lined spillway. Details of the entire spillway are shown on plates F-5 and 6.

Diversion and Control of Water During Construction

2.06 The concrete lined spillway between sta. 41+55 and sta. 109+20, excluding the Ogee section, would be the first structure to be constructed during a dry season. A temporary upstream and protection to be constructed with sheet piles and grouted stone would be provided at sta. 109+20 so that the spillway can be utilized during the subsequent rainy seasons to adequately convey floodwaters from both the Santa Ana River and Mill Creek. A temporary dike, approximately 14 feet in height and 0.6 miles in length, would be constructed in a northeastern direction from the upstream limit of the north spillway wall to Cone Camp Road for prevention of floodwaters from intruding the construction areas for the embankment foundation and outlet works. Local runoffs from the borrow area west of the temporary dike would be diverted to Plunge Creek by construction of levees and ditches around the working area to allow construction of the outlet works. Upon completion of the outlet works and the dam embankment south of Plunge Creek to elevation 1380, all flows from Plunge Creek would then be diverted to the outlet works by constructing a 2,600-foot long levee extending from the completed dam embankment to high grounds on the northern bank of Plunge Creek. The levee would have a maximum height of 17 feet with the top of levee at elevation 1380. The outlet work under this condition, without gates, would be capable of discharging a 25-year frequency flood of 4,800 cfs from Plunge Creek. Construction of the remaining dam embankment would continue until its completion. Removal of the temporary spillway protection, and construction of the Ogee Section would be accomplished during a dry season after completion of the dam embankment.

Instrumentation

2.07 For monitoring the rate and magnitude of foundation and embankment settlement, soil pressures in the embankment, seismic activity in the project area, and water levels at the downstream toe area of the embankment; following instruments would be installed in various strategic locations of the flood control facility: 15 settlement plates, 80 settlement monuments, 30 pressure plates, 30 piezometeres with observation wells, and 4 accelerometers.

Groin Field

2.08 Eight grouted-stone groins would be constructed in the reservoir area downstream from the confluence of the Santa Ana River with Mill Creek. Groins would be designed to deflect floodflows away from the intake area of the spillway. Length of groins would vary from 1,600 feet to 2,400 feet, and the distance between groins would be ranging from 700 feet to 1,500 feet. Groins would have a trapezoidal cross section with a top width of 12 feet, side slopes of 1V on 2H, and a 36-inch thick grouted-stone armor which would extend at least 6 feet below the existing ground surface.

Mill Creek Levee

2.09 EXISTING LEVEE. Construction of the existing levee along the left bank of Mill Creek was authorized by an act of Congress on 17 May 1950 under Public Law 516, 81st Congress, 2d Session. The levee was constructed by the U.S. Army Corps of Engineers, Los Angeles District, between May and September of 1960. The San Bernardino County Flood Control District owns all project land, and operates and maintains the flood control facilities. The levee was designed to control the standard project flood peak discharge of 33,000 cubic feet per second.

The existing compacted earthfill levee has a top width of 18 feet, and a total length of 2.6 miles. The channel face of the levee was revetted with grouted stone except for the downstream 0.35 miles where two feet of dumped stone was placed on the slope. The top of the levee was between 4 to 11 feet above the design grade, and the stone revetment was extended at least 7 feet below the invert grade line.

RECOMMENDED LEVEE. In order to have a positive diversion of major floods from the Santa Ana River into the recommended Mentone Dam, an extension of 6,720 feet would be added to the downstream terminus of the existing Mill Creek Levee. The recommended levee would have a top width of 18 feet, and a height of approximately 20 feet above the existing streambed. For adequate protection at the toe of the first 2,600 feet of levee the bottom 17 feet of the levee would be constructed with gabions, and the remaining top portion of the levee would be armored with 18 inches of grouted stone. A typical section of the recomended levee is shown on plate F-9. The remaining 4,120 feet of levee would have the same structural section as the existing levee, i.e., compacted earthfill embankment with 18-inch grouted stone protection.

MODIFICATION OF EXISTING LEVEE. 13,100 linear-feet of the existing Mill Creek levee would be raised to provide the necessay freeboard to safely divert the major floodflows into Mentone Dam. The existing levee would be raised from 2 to 12 feet as shown on plates F-9 and F-10.

Access Roads

2.10 A single access road to the top of dam would be constructed from Water Street near the left abutment of Mentone Dam. From the top of the dam an access road would be provided to the inlet area atop the intake structure of the outlet works; another road would be constructed to the right bank of the spillway. Turnarounds would be provided at various locations when necessary. Access to the top of Mill Creek levee would be from Opal Avenue and Garnet Street. Paved ramps from the public streets would be provided.

Relocations

2.11 SANTA FE RAILWAY. An existing spur line owned by the Santa Fe Railway runs through the reservoir area of Mentone Dam. Construction of the dam would require relocation of the spur line to the downstream toe of the dam structure. The total length of a new track would be approximately 3.8 miles including a 1,000-foot long bridge over the spillway. The track would be designed to have a maximum slope of 2 percent, and the compacted embankment would have a maximum height of 45 feet above the existing ground surface. The new track would be constructed as the first item of work for the flood control facility, while the existing spur line would remain to serve without the need of a shoofly.

EXISTING ROADS. There are several roads and streets in the project area that would be affected by the construction of Mentone Dam. Considering the existing housing development of the area, only the following roads and streets are selected for relocation or modification:

Church Street would be modified as a dip crossing over the invert of the outlet channel from Mentone Dam.

At the upstream side of Mentone Dam, Greenspot Road would be relocated to join Santa Ana Canyon Road, then join Weaver Street, and continue on Baseline Road. From Baseline Road, the relocated road would follow the toe of the dam to connect to the existing Santa Ana Canyon Road; then continue southwest to Greenspot Road.

Opal Avenue would be modified to go over Mill Creek levee at Sta. 28+00 and Sta. 50+00.

Garnet Street would be raised to meet the gradeline at the top of Mill Creek levee.

UTILITIES. Two existing powerlines, one along Abbey Way and another along Greenspot Road, would be relocated to go around the north abutment of Mentone Dam.

An existing 78-inch waterline owned by the San Bernardino Valley Municipal Water District would be under the right (north) abutment, and would be in the reservoir area of the recommended Mentone dam. (See plate F-2.) The 500-foot long portion of the waterline under the embankment would be relocated around the right abutment. An additional 6,800 feet of the waterline within the reservoir area would be protected in place with reinforced concrete encasement to prevent the waterline from rupture under a pool of floodwaters.

Real Estate Requirements

2.12 The guide taking for the reservoir area would be along elevation 1551.5 feet, which would be 3 feet above the maximum water surface of the Standard Project Flood. Approximately 1,800 acres would be required for the reservoir area. In addition, about 2,500 acres of permanent rights-of-way would be required for the structures of Mentone Dam and Mill Creek Levee.

Cost Estimate

2.13 The estimated first cost for Mentone Dam is $386,321,000, including $360,325,000 for construction and $25,996,000 for land and relocations but excluding the cost for recreational facilities. The construction costs are based on similar types of work in Southern California. Detail of estimated costs is present in Table 4.

III. OAK STREET DRAIN

EXISTING CONDITION

Existing Debris Basin

3.01 Since the Review Report was completed in December 1975, the Riverside Flood Control District has constructed a debris basin at the upper reach of Oak Street Drain. The outlet works consist of an ungated intake structure and a 48-inch diameter reinforced concrete pipe under the spillway. The 171-foot long rectangular concrete spillway has a width of 120 feet and wall heights varying between 4 feet to 12 feet from the spillway crest to the downstream energy dissipator.

Existing Channel

3.02 The existing channel improvement consists of 1.75 miles of vertical rail and wire mesh fence, 0.62 miles of concrete walls and invert, 0.94 miles of concrete lined trapezoidal section, and 0.18 miles of unlined trapezoidal section.

RECOMMENDED IMPROVEMENT

Open Channel

3.03 The recommended Oak Street Drain would begin at the downstream end of the existing spillway structure. Connection of the concrete channel walls to the existing spillway wall would occur 105 feet downstream from the existing spillway crest. The transition channel would have a total length of 1,000 feet. Within this length and beginning at the upstream end, 300 feet would have 7-foot high concrete walls with the invert to join the existing invert. The balance of 700 feet of transition would be a rectangular concrete section with wall heights ranging from 6 feet to 8 feet. The channel transition widths would vary from 120 feet upstream to 20 feet downstream. Beginning at the downstream end of the transition, a 2.8 mile long reinforced-concrete open channel and a short portion of covered channel under the Riverside Freeway would be constructed.

The open channel would terminate near the Temescal Creek where a cut-off wall would be provided. Wall heights of the rectangular open channel section would vary from 8.5 feet upstream to 11 feet downstream. Base widths range from 20 feet to 26 feet. At the downstream end of the rectangular section, for a distance of 300 feet, a grouted dumped stone trapezodial transition section would be provided as an energy dissipator to slow discharge flows into the Temescal Creek. The transition widths would vary from 26 feet to 100 feet. The final 500-foot reach of the project would be a dumped stone trapezoidal section.

Covered Channel

3.04 Approximately 1,000 feet upstream from the Riverside Freeway there would be a change from an open channel section to a rectangular reinforced-concrete covered section. The covered section would be 24-feet wide by 11-feet high and would continue for 1,400 feet downstream crossing under Lincoln Avenue and under the westbound ramps of the freeway. At the north side of the freeway a change in dimensions would occur and a box section with a height of 13 feet and width of 23 feet would begin. This 250-foot segment would continue under the freeway, and then transition from a covered section to the open channel section.

Stone Protection for Existing Basin

3.05 At the downstream end of the project and along the west embankment of an existing settling basin from a sewage treatment plant, 500 feet of stone revetment would be provided to preclude toe erosion of the existing embankment.

Lincoln Avenue Diversion Channel Confluence

3.06 A 300-foot long confluence structure would be provided between Sta. 93+00 and Sta. 96+00 of Oak Street Drain for the proposed Lincoln Avenue Diversion Channel by local interests. A 5-foot invert cutoff wall would be provided at the upstream limit of the confluence structure.

Mangular Channel and Confluence

3.07 A 400-foot long reinforced-concrete open channel and 300-foot long confluence segment would be constructed to join the existing concrete lined channel. The channel and the confluence structure at Oak Street Drain would have a wall height of 12 feet and a base width of 18 feet.

Access Roads

3.08 Roads for inspection and maintenance purposes would be provided along the banks of the open channel. Access to the roads from public streets would be provided as necessary.

Relocations

3.09 New bridges over the proposed Oak Street Drain would be constructed at Railroad Street, Pomona Road, 6th Street, 10th Street, and Ontario Avenue. In addition, a 1,400-foot long covered section would be provided under the existing Riverside Freeway. The covered section would be constructed in stages to minimize disruption of traffic on the freeway, and to reduce the cost for detours.

A 30-inch gasline located at the downstream end of the project would need relocation.

Real Estate Requirements

3.10 Only 7 acres of additional rights-of-way would be required for Oak Street Drain. During construction, easements would also be needed for detours, haul roads, stockpiles area, and contractor's work area and storage yard.

Cost Estimate

3.11 The total first cost of Oak Street Drain is presently estimated at $11,049,000; which includes cost for channel construction, rights-of-way, and relocations. Cost for recreational facilities is presented in another appendix. A detail cost estimate is shown in table 5 on page F-36.

IV. PRADO DAM

EXISTING PRADO DAM

History

4.01 The existing Prado Dam was authorized by the Flood Control Act of 22 June 1936, as amended by the Flood Control Act of 28 June 1938, as part of a general plan for the construction of reservoirs and related work in the metropolitan area of Orange County, California. The dam was constructed between October 1938 and April 1941 under the supervision of the U.S. Army Corps of Engineers, Los Angeles District. The United States Government owns all the rights-of-way and easements for the project since its completion in 1941, and the Los Angeles District has operated and maintained the flood control facilities.

Project Features

4.02 The drainage area above Prado Dam comprises the upper 2,244 square miles of the Santa Ana River Basin, including 767 square miles of tributary to Lake Elsinore. The capacity of the reservoir at the spillway crest is approximately 223,000 acre-feet. The dam contains approximately 3,889,000 cubic yards of earthfill and stands 106 feet above the streambed. The embankment is approximately 2,280 feet long with a paved roadway 30 feet wide on top of the embankment at elevation 566.0. The upstream slope is covered with a blanket of 12-inch stone and the downstream slope is covered with approximately 12 inches of cobble.

The approach channel to the outlet works is located in the west abutment of the dam and is of irregular shape and variable width with side slopes and invert of paved rock. The intake structure consists of two gravity type concrete entrance walls, an invert slab, piers and six steel trash racks. The outflows are controlled by six 7 foot by 12 foot tractor gates. In front of each gate are slots to accommodate stop logs.

The control tower was constructed of reinforced concrete columns with horizontal struts. The control house is an integral part of the intake structure, and the finished floor of the control house, is 66 feet above the top of the trash rack.

A 90-foot long transition section connects the six chambers at the gates and the double rectangular conduits under the dam. The conduit under the dam consists of a monolithic double-barreled box which is 591 feet in length. The outlet structure is 366 feet long, and consists of three types of sections: 126 feet of rectangular channel, 80 feet of transition chute and a 120-foot long stilling basin. From the stilling basin a trapezoidal outlet channel extends 1,800 feet and is designed to carry 10,000 cfs at a velocity of 6 fps and a depth of 7 feet.

Deficiency

4.03 A review of design features of the existing Prado Dam based on present hydrologic and hydraulic design criteria was completed in November 1969. It was found that the existing dam is incapable of controlling the standard project flood (SPF) and a maximum probable flood (MPF) under the existing conditions. The deficiency can be attributed to the following factors: (1) the inflows to the reservoir are estimated to be much higher than those used in the original design as a result of urban developments over most of the drainage area, (2) the existing reservoir does not have sufficient capacity to accommodate the estimated volume of floodwaters to be stored behind the dam, (3) the outlet works do not have the capacity of releasing more than 17,000 cfs, and (4) the existing Santa Ana River Channel cannot safely convey more than 16,000 cfs without severe erosion to the streambed and toe of embankments.

Conclusion

4.04 In order to reestablish the existing dam as a key flood control element along the Santa Ana River, the dam must be modified to meet the current conditions and design criteria. Increased reservoir storage at Prado Dam would also provide additional protection below the dam; threfore two alternatives would be considered for enlargement of Prado Dam. One alternative would be to raise the dam crest to elevation 596 feet and to provide maximum outlet releases capacity of 45,000 cfs. This alternative assumes the construction of Mentone Dam and provides the highest degree of protection possible to the area below the dam without causing major social dislocations. The other alternative would be to raise the top of Prado Dam to elevation 609 feet and providing maximum outlet releases capacity of 45,000 cfs. This alternative assumes Mentone Dam would not be constructed and would need a larger storage capacity at Prado Dam.

DAM UNDER ALL RIVER PLAN

Embankment

4.05 Under this alternative the main embankment would be raised to elevation 596, a total of 30 feet above the existing embankment crest. This would create a maximum impoundment of about 19,150 surface acres and a gross storage capacity of about 838,000 acre feet. It would be required to remove the paved road and the upper portion of the dam embankment, as well as the removal of the downstream cobblestone facing and a signifcant portion of the upstream section of riprap over the earth embankment prior to placement of the new fill material. The existing dam tender's house would be relocated to allow room for placement of the new embankment. Under the reservoir spill condition, the tail water surface along the downstream slope of the dam would be at approximately elevation 494. Stone protection would be provided on the downstream face of the embankment below elevation 500 feet.

Outlet Works

4.06 Because of inadequate discharging capacity as well as insufficient structural strength to support the additional load induced by raising the dam, the existing outlet works would have to be abandoned and new outlet works would be constructed at a location between the east dam abutment and the spillway. The existing outlet works would operate as a diversion structure during construction and would be plugged with lean-mix concrete grout throughout its entire length upon completion of the new outlet works. The outlet works would consist of two reinforced concrete towers with three control gates in each structure. Each tower would control a 25-foot diameter, concrete-lined conduit.

Spillway Modification

4.07 The ogee spillway crest elevation would be raised 20 feet above the existing spillway crest to elevation 563 feet. A 10-foot freeboard above the peak spillway flow elevation would be provided along both walls of the new spillway. The existing walls on the each side of spillway would be removed and the spillway crest length would be increased from 1,000 feet to 1,300 feet to the east of the existing spillway. New cantilevered spillway walls would be constructed on each side of the modified spillway.

Erosion protection for the Riverside Freeway embankment would be provided by constructing a 38-foot high by 20-foot long concrete wall on the east side of the spillway, downstream from the existing spillway flip-bucket.

Auxiliary Dike Structure

4.08 Raising the embankment and spillway crest elevations would require construction of a 7,800-foot long auxiliary dike structure extending southeast from the east spillway wall. This dike would prevent floodwaters from escaping the dam and would provide flood protection to the Riverside Freeway, the Santa Fe Railroad, and the metropolitan area downstream. The dike would be a compacted earthfill structure with a service road, on top of dike at elevation 596 feet. A 10-foot freeboard above the maximum water surface of the spillway flood would be provided.

Ring Dikes

4.09 The California Institution for Women, the Chino Sewage Treatment Plant, and the Alcoa Aluminum Plant, (see Appendix E, Real Estate Sheets Nos. 12, 41 and 42 respectively), are all located on the fringe area of the proposed reservoir. In order to avoid expensive relocation costs for these facilities, a ring dike would be provided around each one as protection from reservoir flooding. The compacted earth dikes would have a top width of 12 feet at elevation 566, and would have 12-inch stone revetment on the 1V on 2.25H side slopes. The dike around the Institution for Women would be approximately 5,000 feet in length, and 4.5 feet in average height. The dike around the sewage treatment plant would be about 2,400 foot long, and average 34 feet in height. A 4,400-foot long and 7-foot high dike would be provided around the Alcoa Aluminum Plant. Drain pipes with flap gates would be provided at each dike for draining local runoffs.

Instrumentation

4.10 Fifteen settlement plates and 30 settlement monuments would be installed in the dam and auxiliary dike embankments to monitor the rate and magnitude of foundation and embankment settlements. Four accelerometers would be placed in the embankments to record seismic activities in the area. Thirty pressure plates would be installed to measure soil pressure in the embankments. Twenty piezometers with observation wells would be placed at the downstream toe of embankments to monitor water levels.

Relocations

4.11 The existing Corona Freeway would be modified at two locations. (1) In order to prevent erosion of the dam embankment, the existing bridge would be elevated to allow for a maximum downstream water surface elevation of 494 feet which is under reservoir spill conditions. This modification would include the construction of a 2,350-foot long bridge which would connect on to the north abutment of existing Corona Freeway and extend south following the existing alignment and cross over both the east and west bound lanes of the Riverside Freeway. The Corona and Riverside Freeway interchange would be revised by constructing a series of ramps from the proposed bridge. No increase in width or additional traffic lanes have been considered. (2) Relocation of the Corona Freeway would be required to allow the freeway to go over the crest elevation of 596 feet of the new embankment at the west abutment of the main dam.

Access Roads

4.12 Existing access roads to the various features of Prado Dam would be utilized and modified, as necessary, to meet the need of vehicular access to all parts of the proposed flood control facility. Ramps from Pomona Rincon Road to the top of the auxiliary dike would be provided.

Real Estate Requirements

4.13 The guide taking line for the existing dam and reservoir was established at elevation 556 feet msl when the dam was built between 1938 and 1941. Under the all river plan, the guide taking line would be at elevation 566 feet msl which would be 3 feet above the water surface of the standard project flood at elevation of 563. About 1,460 acres of additional rights-of-way or easement would be required for this plan of improvement. Relocation of many homes would be necessary.

Cost Estimate

4.14 The project first cost of improvement under this plan is estimated at $207,580,000 based on the price level of October 1979. Table 6 on page F-37 shows the cost for various features. Recreation development cost is presented in another appendix.

DAM UNDER PLAN TO PROVIDE SPF PROTECTION BELOW DAM

Embankment

4.15 The main embankment under this alternative would be raised to elevation 609, a total of 43 feet above the existing embankment crest. The added embankment height would create a maximum reservoir of about 23,000 surface acres and a storage capacity of 1,080,000 acre-feet. The downstream slope of the new embankment would intersect the slope of existing dam embankment at approximately elevation 465. This line falls 29 feet below the tail water pool elevation. Riprap protection to an elevation of 500 feet would be provided across the downstream face of the dam. Removal of existing fill material and other facilities prior to the placement of the new fill material would be similar to that required for the 596 elevation (all river plan) alternative.

Outlet Works

4.16 The outlet works for the 609 elevation alternative are similar as those described under the 596 elevation alternative.

Spillway Modification

4.17 The existing spillway modification would be similar to that described under the all river plan alternative, except the ogee crest elevation would be raised to elevation 577 feet. Erosion protection for the Riverside Freeway embankment would also be included as described under the 596 elevation alternative.

Auxiliary Dike Structure

4.18 The dike for the 609 elevation alternative is similar to that described under the 596 elevation alternative except that the dike crest elevation would be 609 feet and the length of dike would be 9,300 feet from the east spillway wall.

Ring Dike

4.19 In order for the Chino Sewage Treatment Plant to continue operation at its present location, a compacted earth dike, approximately 1 mile in length would be provided around the facility. This dike would have an average height of 35 feet and would have 12-inch thick facing stone on top and on its 1V on 2.25H side slopes. The dike crest could be 12 feet in width at elevation 582 which would be 3 feet above the SPF pool behind the dam. Corrugated metal pipes with flap gates would be provided for draining local runoffs.

Instrumentation

4.20 Under this plan, instruments to be installed in the dam and dike embankments would be similar to those recommended for Prado dam under the all river plan.

Relocations

4.21 The Corona Freeway Modifications are the same as for the 596 elevation alternative except for the additional earthfill that would be required to allow the freeway to pass over the crest elevation of 609 feet at the west abutment of the main dam.

Access Roads

4.22 Access roads under this plan would be also utilizing existing roads with modifications and addition of ramps to the top of auxiliary dike from Pomona Rincon Road. However cost for access roads would be higher under this plan than under the all river plan.

Real Estate Requirements

4.23 Under the national economic development plan, the guide taking line for the reservoir at Prado Dam would be at elevation 580 feet msl, which would be 24 feet above the existing guide taking line at elevation 556 feet. The new guide taking line would be 3 feet above the maximum water surface of the standard project flood (spillway crest elevation). Most of the additional rights-of-way would be located in urbanized areas surrounding the existing reservoir. Acquisition of these lands would require relocation of hundreds of families, and the cost for land is presently estimated at $332,800,000.

Cost Estimate

4.24 Table 10 shows the detailed cost estimate under the national economic development plan. The first cost for the project including rights-of-way is estimated at $490,490,000.

V. SANTIAGO CREEK

EXISTING CONDITION

5.01 Santiago Creek is located in Orange County, and is a major tributary of the Santa Ana River south of Prado Dam. At the Santa Ana Freeway near the confluence with the Santa Ana River, the creek has a drainage area of approximately 101 square miles. The existing creek between Prospect Street and the Santa Ana River is about 5.5 miles in length, of which most have been improved except for a 3,000-foot reach at the upstream end and a 6,000-foot reach at the downstream end. Upstream of Prospect Street are a series of sand-and-gravel borrow pits owned by a private company. The existing gravel pits are bounded by Prospect Street, Bond Avenue, Hewes Avenue, Santiago Canyon Road and a boundary line running southwest for approximately 3,500 feet between Santiago Canyon Road and Prospect Street. Mining of the materials in these pits has been in operation for many years, and it will probably continue for several more years to come. At this time, it is difficult to predict the final configuration of the pits prior to commencement of project construction. However, an assumption is made that most of the naturally deposited materials will be removed, and the banks of the pits will be left on a steep slope of 1V on 1H. The existing pits have a total capacity of approximately 7,000 acre-feet, and their ultimate volume could be as much as 8,000 acre-feet. These large pits are suitable for the disposal of 4.75 million cubic yards of surplus excavated materials from the Santa Ana River, or the storage of floodwaters for the creation of recreational lakes.

RECOMMENDED IMPROVEMENT

General

5.02 The plan for improvement of Santiago Creek would consist of two portions of the existing creek: (1) Provision for a storage reservoir at the sand and gravel borrow pits with an inlet structure and outlet channel, and (2) improvement of the downstream 6,000 feet of the creek near its confluence with the Santa Ana River. Plates F-24 thru F-28 show the location, plan and profile of the structures.

MODIFICATION OF BORROW PITS

General

5.03 The pits would be modified so that their banks would be stable under a saturated or submerged condition. Compacted fill would be placed along the bank on a minimum slope of 1V on 2H. As a reservoir the pits would be designed and modified to meet required capacity of 3,500 acre-feet between elevations 280 feet and 298 feet, and 600 acre-feet between elevations 264.5 feet and 268 feet.

INLET STRUCTURE. The inlet structure would be 518 feet long and would cross under Santiago Canyon Road at a point 1,450 feet west from the intersection with Hewes Avenue. The rectangular concrete covered section under Santiago Canyon Road would be located 95 feet from the entrance to the structure and would have wall heights of 35 feet, base width of 60 feet and a length of 40 feet. Reinforced concrete wing walls would be constructed at each end of the bridge structure with variable wall heights and base width of 60 feet between the walls. The total length from the entrance to the end of the wing walls, including the bridge, would be 160 feet. Continuing from the end of the wing walls there would be a rectangular concrete channel section with a length of 160 feet, wall heights of 12 feet and base width of 60 feet. The balance of inlet structure would be a 184-foot long rectangular concrete channel baffled with piers on an invert slope of 1 vertical on 2.5 horizontal. The chute would have wall heights of 14 feet, a base width of 60 feet, and 19 rows of concrete baffle piers. A 2-foot thick rock blanket would be provided around the perimeter of the inlet structure. The designed maximum flow would be 5,600 cfs.

OUTLET STRUCTURE. The entrance to the structure would be a vertical concrete headwall with 3 gate openings into a 46-foot long by 76-foot wide open rectangular concrete gate chamber. The gate chamber would contain 3-constant downstream Neyrpic level gates in series. An open rectangular concrete stilling chamber would follow with a base width of 80 feet and length of 60 feet. Each chamber would have wall heights of approximately 17.5 feet. The maximum outlet flow would be 3,500 cfs.

Channel

5.04 Certain reaches of the existing channel would be improved for adequate conveyance of the design flood in accordance with the need and condition of the channel. Two types of channel are proposed at locations as described in the following paragraphs.

RECTANGULAR CONCRETE LINED CHANNEL. A 328-foot long rectangular concrete lined open channel would begin at the end of the stilling chamber. The first 50 feet would be a transitional reach with a base width upstream of 80 feet and a width of 30 feet downstream. Wall heights would be 16.5 feet. The next 178 feet would continue under Prospect Street and have wall heights of 14.5 feet and a base width of 30 feet. The last 100 feet of concrete lined channel would be a transition section from a rectangular channel to a trapezoidal channel section with a base width of 30 feet, variable wall heights from 14.5 feet to 12 feet and side slopes from vertical to 1.0 vertical on 2.5 horizontal. A 6-foot deep concrete cutoff wall would be provided at the downstream end of the concrete channel.

UPSTREAM TRAPEZOIDAL CHANNEL. Downstream of the rectangular concrete lined channel would begin a 3,400-foot long trapezoidal channel section. This section would be lined with 18 inches minimum of riprap and have a base width of 30 feet, channel height of 11 feet, and side slopes of 1.0 vertical on 2.5 horizontal. The last 100 feet of channel would be a transition trapezoidal section with an upstream base width of 30 feet and downstream width of 150 feet. The downstream end of the proposed improvement would be near the end of Walnut Avenue.

DOWNSTREAM TRAPEZOIDAL CHANNEL. Approximately 600 feet downstream from the Santa Ana Freeway would begin a 6,000-foot long trapezoidal channel section. This section would be lined with 18 inches minimum of riprap and have a base width of 30 feet, channel height of 11 feet and side slopes of 1V on 2H. Downstream end of improvement would be at the confluence with the Santa Ana River.

Relocations

5.05 Prospect Street bridge would be reconstructed, and approximately 500 linear-feet of overhead power lines at or near the reservoir outlet structure would be relocated.

Access Roads

5.06 The proposed reservoir at the existing borrow pits would be bound on three sides by existing streets; therefore, access road would be required only along the west bank of the pits. Access road would also be provided along the banks of trapezoidal channels. Ramps from streets and turnaround would be provided as necessary.

Real Estate Requirements

5.07 Approximately 306 acres of land would be required for the reservoir and the outlet structure at the upstream end of the project. In addition, 306 acres of land would be needed for the downstream portion of the Santiago Creek.

Cost Estimate

5.08 The first cost of the project based on October 1979 price level is estimated at $10,003,000. Detail of the cost estimate is presented on table 7.

VI. LOWER SANTA ANA RIVER

EXISTING CONDITION

General

6.01 The Santa Ana River between Prado Dam and the Pacific Ocean is approximately 30.5 miles in length; of which the upstream 2.5 miles is located in Riverside County, and the remaining 28 miles is within the Orange County limits. The river winds through the narrow and relatively undeveloped Santa Ana Canyon for a distance of about 10 miles before it turns southwest into the alluvial plain of the metropolitan area of Orange County.

PRADO DAM TO IMPERIAL HIGHWAY. The upper reach of the river is unimproved and still in its native condition except for a few streambed stabilizers at selected locations near the mouth of the canyon. As the river enters the alluvial plain, runoffs in the river are diverted into water spreading basins for flood control and water conservation purpose. Formal channelization of the river begins in the vicinity of Imperial Highway (Sta. 1057+50) which is approximately 10 miles downstream from Prado Dam.

IMPERIAL HIGHWAY TO KATELLA AVENUE. From Imperial Highway downstream to a point about 1,100 feet south of Katella Avenue (Sta. 701+10), a distance of 7 miles, the existing channel is trapezoidal in cross section with a soft-bottom invert and stone revetted side slopes of 1V on 2H. It has a base width ranging from 300 feet at the upstream end to 320 feet near Katella Avenue, and levee heights ranging from 12 to 18.5 feet. Within this reach there are seven drop structures which function as hydraulic energy dissipators and streambed stabilizers.

KATELLA AVENUE TO GARDEN GROVE FREEWAY. Downstream from Katella Avenue to the Garden Grove Freeway (Sta. 595+00), a channel reach of 2.1 miles, the earth-bottom trapezoidal channel has a base width varying between 240 to 270 feet, and side slopes changing from IV on 1.5H to 1V on 3H. The upper 500 feet of channel with steeper side slopes has concrete slope protection, and the remaining reach of this channel has stone-revetted slopes. Within this reach of channel two drop structures, approximately one-mile apart, were constructed by the Orange County Flood Control District.

GARDEN GROVE FREEWAY TO 17TH STREET. For a distance of 1.5 miles south of the Garden Grove Freeway to the vicinity north of 17th Street (Sta. 513+40), the existing river has only limited improvement. About half of the banks are protected by pipe and wire fence, and the remaining banks are stabilized by turf which is a part of River View Golf Course. One drop structure has been constructed at the southern end of this reach.

17TH STREET TO ADAMS AVENUE. From approximately 1,200 feet upstream of 17th Street to about 3,000 feet downstream of Adams Avenue (Sta. 163+80), a reach of 7.4 miles, the existing Santa Ana River channel is well entrenched with soft bottom, trapezoidal cross section, and wall heights ranging from 13 to 17 feet. The side slopes varying from 1V on 1.5H to 1V on 2H are protected with reinforced concrete. The base width of the channel varies significantly within this reach ranging from 160 to 250 feet with a, design capacity of 40,000 cfs for the entire distance.

ADAMS AVENUE TO PACIFIC COAST HIGHWAY. Downstream from the above section for a distance of 1.8 miles, the base width of the soft-bottom trapezoidal channel is 160 feet. The channel wall height is approximately 16.5 feet. The side slopes of the channel are 1V on 3H except at both ends where transition of the slopes occur. The sideslopes are protected with grouted stone.

From the above reach to the Pacific Coast Highway (Sta. 9+60) is 0.6 miles. The improved channel has either a concrete or grouted stone invert. The channel width is 160 feet except at the downstream 0.2 miles where the width changes to 180 feet. The channel changes in cross section from trapezoidal to rectangular as it flows downstream. Wall height for both type of channel sections are approximately 16 feet. The side slopes of trapezoidal section are protected with grouted stone, and the 564 feet transition structure and vertical channel walls are constructed with reinforced concrete.

The outlet channel of the Santa Ana River is located south of Pacific Coast Highway in Huntington Beach where the river enters the Pacific Ocean. The outlet channel consists of a transition section, from rectangular to trapezoidal section, with a stone jetty. The 700-feet long channel has a soft-bottom invert with a base width varying from 180 to 316 feet.

Deficiency

6.02 Although the existing Santa Ana River channel was designed to have a capacity ranging from 30,000 to 40,000 cfs, severe erosion of the unlined channel invert would occur if more than 5,000 cfs is released from Prado Dam. Discharge of more than 5,000 cfs from the dam would undermine the toe of channel embankments and would erode the foundation materials underneath the piers of many bridges. The channel invert remains unlined is to allow recharge of the underground water reservoir by floodwaters. The Environment Management Agency of Orange County has been consistently improving the capability of the Santa Ana River Channel during the last 20 years, but the invert of the channel must be stabilized and the channel banks strengthened before the channel can convey the proposed design flood.

RECOMMENDED PLAN

General

6.03 Proposed channel improvement for the Santa Ana River was developed according to the conditions and needs of the existing channel. In general there are five methods of improvement proposed for various reaches of the channel: (1) intermittent levee and bank protection, (2) trapezoidal earth-bottom channel with revetted side slopes, (3) rectangular concrete-lined channel, (4) rectangular concrete wall channel with soft-bottom, and (5) outlet channel structure.

Levee and Bank Protection

6.04 Intermittent levee and bank protection would be provided mainly along the upstream 8.1 miles (Sta. 1610+03 to Sta. 1210+00) of the Santa Ana River. Stone revetment with a thickness of 24 inches would be placed at strategic locations along 3.3 miles of the river banks. Another 4,700 feet of 18-inch grouted stone revetment would be provided at two locations where severe scouring is anticipated. Both types of stone revetment would be extended between 15 feet and 25 feet above the streambed, and 5 feet to 10 feet below the streambed. Details of the intermittent levee and bank protection and various locations are shown on plates F-29 through F-35.

Santa Ana River Channel

6.05 Starting at a point approximately 2.6 miles upstream from Imperial Highway (Sta. 1057+80) to the vicinity of River View Golf Course (between Sta. 560+00 and Sta. 528+00), this 12.0-mile reach of existing channel would be improved by deepening the invert and raising the banks. The channel invert would remain unlined to allow recharging of underground water reservoirs, but the channel slopes would be revetted with 18 inches of stone over 6 inches of filter material except for the downstream 1,500 feet where the stone thickness would be increased to 24 inches.

Twenty stabilizers would be constructed at approximately 2,000-foot intervals in order to stabilize the channel invert during floodflows. The stabilizers would be constructed with 24 inches of grouted stone. (See plate F-36). Five existing drop structures would be modified to increase their capacity; in addition, four new drop structures would be built at critical locations to reduce the velocity of floodflows. Proposed drop structures would be constructed with reinforced concrete walls and invert. The downstream toe of the drop structures would be protected with a stone-revetted apron which would be extended along 1V on 2H slope, 15 feet below the channel invert elevation.

At the River View Golf Course, the proposed channel would be irregular in cross section. The levee at the right bank would have a slope of 1V on 3.5H, and would have a minimum of 2 feet of top soil over 24-inch riprap and 6-inch filter material. A vertical reinforced concrete wall would be constructed along the eastern boundary of the golf course to prevent floodwaters from overtopping the existing bank. The invert of the channel would remain in its existing natural condition as a golf course.

Downstream from the golf course in the vicinity of 17th Street (Sta. 513+40) to a point about 2,000 feet south of Adams Avenue (Sta. 163+70), a reinforced concrete-lined rectangular channel would be constructed. The 7.1-mile reach of channel would have a base width ranging from 230 feet to 450 feet, and wall heights varying from 12.5 feet to 20.0 feet. A reinforced concrete rectangular channel is recommended for this reach of river; because of dense urban developments along the river banks. A rectangular channel would minimize the need of additional rights-of-way. The existing streambed would be deepened by a maximum of 10 feet in order to carry the design capacity floodwaters.

As a result of the proposed channel improvement, eight street bridges and one railroad bridge would require reconstruction. In addition the Slater Avenue bridge would be modified to accommodate the design flows. A subdrainage system would be required under the invert of the rectangular concrete channel.

In order to minimize disturbance to the habitat area in the estuary of the Santa Ana River, the downstream 2.6 miles of the proposed channel to Pacific Coast Highway would be designed to have a soft bottom with vertical concrete channel walls. The channel walls would be designed as T-type walls, but the toe of the walls would be heavily protected by a 15-foot deep cutoff wall and a layer of riprap which would extend from the wall, on a 1V on 2H slope, to a depth of 25 feet below the channel invert. The width of the channel would vary from 450 feet to 480 feet, and the height of channel walls would be 18.5 feet above the channel invert.

The recommended outlet channel structure would be located south of the Pacific Coast Highway where the Santa Ana River enters the Pacific Ocean. The channel within this reach would be 750 feet in length including a 200-foot transition section between the rectangular channel and a trapezoidal section at the downstream end. The side slope of the trapezoidal section would be covered with a 48-inch layer of stone revetment over 12 inches filter material. The stone revetment would be extended to a depth of 10 feet below the invert elevation. The height of channel walls above invert grade would range from 18.5 feet to 14.85 feet, whereas the channel invert would vary from 480 feet to 450 feet in width.

Greenville-Banning Channel

6.06 The recommended improved channel would be located adjacent to the east bank of the existing Santa Ana River channel. The existing channel with limited improvement by local interests has insufficient capacity for conveyance of major floods. The recommended improvement for the existing channel would begin at approximately 1,600 feet south of San

Diego Freeway, and would end at about 2,000 feet south of Victoria Street for a total distance of 3.3 miles. Due to urbanization along the channel, the recommended channel would have a rectangular cross section with reinforced concrete invert and walls. The channel invert would vary from 50 feet to 60 feet in width, and channel wall heights would range from 13.5 feet to 17 feet. An upstream transition section would be provided to join the improved rectangular channel with the existing channel which is trapezoidal in cross section. Plan and profile of the Greenville-Banning Channel are shown on plates F-54 through F-57.

Huntington Beach Channel

6.07 Due to the widening of Santa Ana River channel, the existing Huntington Beach channel would be relocated about 240 feet to the west of its existing alinement. The relocated channel would be approximately 1,500 feet in length, and would be trapezoidal in cross section. The soft-bottom channel would be designed and constructed to match the physical dimensions of the existing channel section.

Disposal of Excavated Materials

6.08 The proposed widening and deepening of the existing Santa Ana River would create a surplus of 7.5 million cubic yards of excavated materials. The borrow pits along Santiago Creek, upstream of Prospect Street, are considered as the primary disposal sites which could take 4.75 million cubic yards of the materials. The borrow pit on the east bank of Santa Ana River upstream from Lincoln Avenue is ideally located, but its limited capacity could hold only 1.75 million cubic yards. The remaining one million cubic yards of sandy material from the downstream portion of Santa Ana River would be suitable for replenishment of beach sand. Up to 650,000 cubic yards of the material could be placed in the existing Newport Beach groin field, and 350,000 cubic yards could be placed between the Newport Beach groin and the proposed Santa Ana River east jetty extension.

Access Roads

6.09 To provide access to the channel banks and inverts, a system of access roads would be included in the recommended Santa Ana River channel improvements. In the upper portion of the Santa Ana River, where intermittent protection would be provided on the slopes of river banks, existing public streets would be utilized for inspection and maintenance purposes.

Along the banks of both trapezoidal and rectangular channel section a 30-foot wide berm would be provided for maintenance vehicles. This width of berm was selected to reduce the amount of excess excavated materials. Access from public street to the berms and access to the channel invert would be provided as necessary. A 20-foot wide bridge for operation and maintenance would be provided over the Greenville-Banning Channel in the vicinity of its confluence with the Santa Ana River.

Relocations

6.10 In order to increase the channel capacity, the existing Santa Ana River Channel must be widened and its invert deepened. As a result, many of the existing railroad and highway bridges would be reconstructed; and a large number of existing utilities would be relocated. Relocation of recreation trails in the lower Santa Ana River is discussed in Appendix G Recreation.

RAILROAD BRIDGES. Three bridges owned by the Southern Pacific Transportation Company and one bridge owned by the Santa Fe Railway would be completely reconstructed to span the recommended channel. Shooflies would be constructed, if necessary, during construction of the new bridge.

STREET AND HIGHWAY BRIDGES. A total of 13 highway and street bridges would require complete reconstruction. The bridge locations are: Lincoln Avenue, Katella Avenue, Orangewood Avenue, 17th Street, Fairview Street, 5th Street, Bolsa Avenue (First Street), Edinger Avenue, Harbor Boulevard, Warner Avenue, Talbert Avenue, Victoria Street (Hamilton Avenue), and Pacific Coast Highway. During reconstruction of bridges, vehicular traffic crossing the river would be rerouted to an existing or newly completed bridge to avoid the cost for providing detours except at Lincoln Avenue and Pacific Coast Highway. A detour immediately adjacent to the last two bridges would be provided during construction due to their isolated locations.

UTILITIES. The horizontal alinement of the recommended Santa Ana River channel has been selected to avoid relocation or modification of major facilities or utilities. However, the needs for relocation or alteration of certain utilities are unavoidable. Following is a list of major utilities which would be affected as a result of construction of the proposed Santa Ana River channel.

LOCATION	LENGTH AND TYPE OF UTILITY TO BE RELOCATED
Sta. 930+40	450 feet of 14-inch waterline
Sta. 813+00	One pole, and 400 feet of overhead telephone cable.
Sta. 812+70	450 feet of 22-inch gasline
Sta. 700+30	300 feet of 24-inch sewerline
Sta. 584+20	500 feet of 35-inch waterline
Sta. 513+90	400 feet of 3-inch gasline
Sta. 465+50	400 feet of 10-inch gasline
Sta. 342+80	450 feet of 10-inch gasline
Sta. 342+73	450 feet of telephone cable
Sta. 249+40	400 feet of 48-inch sewerline
Sta. 245+20	500 feet of 39-inch sewerline
Sta. 164+35	550 feet of 24-inch waterline

Sta. 189+00 to Sta. 169+00	7 pole and 2,000 feet of overhead powerline
Sta. 109+80	400 feet of 30-inch gasline
Sta. 82+60	5 poles and 650 feet of overhead powerline
Sta. 82+50	700 feet of 12-inch conduit
Sta. 82+60 to Sta. 79+40	380 feet of 12-inch irrigation line
Sta. 75+80	Caped oil well
Sta. 60+70	Oil well and appurtenance
Sta. 52+90	700 feet of 12-inch waterline
Sta. 55+10 to Sta. 31+70	15 poles and 2,400 feet overhead powerline
Sta. 45+40	Oil well (Possible protected in place)
Sta. 42+50	550 feet of 6-inch oil line
Sta. 40+62	550 feet of 6-inch oil line
Sta. 37+30	Oil well (Possible protected in place)
Sta. 25+90	800 feet of 33-inch sewerline
Sta. 10+20	850 feet of 10-inch gasline

Real Estate Requirements

6.11 The recommended channel improvement would be constructed mostly within the existing rights-of-way or easements owned by the local flood control districts. However, additional rights-of-way would be required for channel widening, ramps at street crossing, access roads, and recreational facilities. Easements would be needed during construction for detours, shooflies, haul roads and disposal of 7.5 million cubic yards of surplus excavated materials. Additional information is given in Appendix entitled "Real Estate".

Cost Estimate

6.12 The construction cost for the proposed channel based on October 1979 price level for similar type of works in southern California is estimated at $254,052,000 including relocation of railroad bridges and 25 percent contingencies. Additional costs for engineering and design, rights-of-way, relocation of street bridges and utilities, and mitigation would increase the total project cost to $300.1 million. Cost for recreational facilities is included in another appendix. Detailed cost estimate for flood control including mitigation is presented in Table 8.

VII. FIRST COST RECOMMENDED PROJECT

7.01 The total project first cost is estimated at $914,763,000 based on price level of 1 October 1979 for similar type of works in southern California. Detailed cost estimate for each feature of the recommended all river plan is shown on pages F-34 through F-43. A summary of cost estimates for the plan is presented in Table 9. Unit prices for various items of work were based on the following considerations and assumptions.

(a) Mentone Dam.

(1) Impervious material would be imported from the reservoir area of Prado Dam which is located approximately 30 miles southwest of Mentone Dam.

(2) Prices for other embankment materials are based on the Contractor setting up an onsite plant for processing materials from the required excavation in the reservoir area. Average one-way hauling distance would be about 1.2 miles.

(3) Concrete would be produced by an onsite plant.

(b) Prado Dam.

Materials for the embankments of dam and dike would be obtained from a borrow area within the basin area. Average one-way haul distance would be approximately 1.5 miles.

(c) Lower Santa Ana River.

The 4.75 milion cubic yard of surplus excavated materials would be deposited in gravel pits at the upstream end of Santiago Creek. An average one-way hauling distance would be about 12 miles. The remain 1.75 million cubic yard of excess materials would be deposited in a gravel pit near Lincoln Avenue; average one-way hauling distance would be approximately 10 miles.

(d) Entire project.

(1) Unit price for Bentonite slurry was obtained from a firm who specializes in this type of construction. Price includes all related earthwork.

(2) With the exception of Mentone Dam, all concrete would be obtained from a ready-mixed commercial plant located within 20 miles of the project.

(3) Unit price for stone riprap was based on a recent quote from a local supplier whose quarry is located within 30 miles of the project.

VIII. COMPARISON OF REVIEW REPORT AND PRESENT ESTIMATES

8.01 Table 9 shows the summary of the Santa Ana River project first costs, one of which was made by the Board of Engineers for Rivers and Harbors, another was made for PB-3, and the remaining one is the presnet estimate for the Phase I study. A comparison of the first costs between the estimate which was made by BEHR and the present estimate is not made, because former estimate is based on teh price level of September 1975. The Phase I estimate was developed first by redevelopment of entire project design, and then by reevaluation of the construction cost for similar types of work in southern California. The costs for engineering and design, and supervision and administration were estimated at the prevalent rates of the construction cost. The costs for land and relocations were updated to conform with the Phase I design, and to reflect the latest condition of development within the project area.

8.02 The difference in estimated costs between the PB-3 (Effective date: 1 October 1979) and the present estimate is presented in the following paragraphs:

(a) The decrease in cost of $58,560,000 for Mentone Dam is due to elimination of the impervious blanket in the reservoir, change of the downstream embankment slope from a maximum of 1V on 9H to 1V on 3H, and reduction of excavation in the basin.

(b) The decrease in construction cost of $1,463,000 for Oak Street Drain is a direct result of the construction of a sediment trap by local interests.

(c) The increase in construction cost of $7,009,000 for Prado Dam is due to more detailed studies on design, and updated construction costs.

(d) The decrease of $7,792,000 in the construction cost of Santiago Creek is due to a complete design change from a concrete-lined and stone-revetted channel capable of conveying a standard project flood to a green belt type of channel which would have a capacity for only a 100-year flood.

(e) The increase of $34,578,000 in construction cost of the Santa Ana River channel is due to inclusion of a shoofly for each railroad road relocation, the need for removal of newly constructed stone and concrete revetment on existing channel slopes by local interests and a major design change between channel stations 18+00 and 157+00 to allow minimum disturbance to the habitat area of least tern.

(f) The decrease of $39,993,000 in cost of land is due to at least three factors: (1) market value of land in the flood plain at Mentone Damsite and along the proposed Santa Ana River channel has been increasing at a slower rate than the cost of construction which was used as the basis for escalating the costs of rights-of-way, (2) reduction of rights-of-way for Mentone Dam, and (3) acquisition of interests in the marshland along the lower Santa Ana River has been revised.

(g) The decrease of $25,065,000 in relocation cost is a result of detailed study on all existing bridges and utilities.

(h) The increase of $19,538,000 in engineering and design is due to the fact that the prevalent percentage rate of construction cost for engineering and design is higher than that of 1975; although the total construction cost of the project is less than the formerly recommended project.

(i) The decrease of $9,543,000 in cost for supervision and administration is primarily due to (1) reduction of construction cost at Mentone Dam, and (2) the supervision and administration cost as a percentage of construction cost is less than previously used for PB-3.

XI. OPERATION AND MAINTENANCE COST

9.01 The annual operation and maintenance cost for the flood control facilities under the all river plan is estimated at $2,640,000. The estimate includes costs for channel and dam embankment repair, debris removal in the channel as well as in the outlet works of dams, and a full-time dam operator at Mentone Dam. (An average annual cost of $348,000 for removal and disposal of 145,000 cubic yards of debris at the lower Santa Ana River is used.) Cost for recreation and maintenance of recreation facilities is presented in Appendix G, Recreation.

Estimated annual cost for operation and maintenance is tabulated as follows:

Mentone Dam	$ 950,000
Oak Street Drain	50,000
Prado Dam	280,000
Lower Santa Ana River including Greenville-Banning and Huntington Beach channels	1,330,000
Santiago Creek	30,000
Total	$2,640,000

TABLE 1
Mentone Dam
Pertinent Data Under All River Plan

Drainage area	sq mi	260
Dam (rolled earthfill)		
Crest elevation	ft msl	1,573.5
Maximum height above streambed	ft	226
Crest length	ft	17,700
Freeboard	ft	8
Spillway (overflow, concrete)		
Crest elevation	ft msl	1,548.5
Crest length	ft	1,000
Elevation of maximum water surface	ft msl	1,565.5
Outlet works (gated conduit)		
Diameter of conduit	ft	14
Length of conduit	ft	1,373
Intake elevation	ft msl	1,335
Reservoir		
Area at spillway crest	acre	1,167
Capacity (gross) at spillway crest	acre-ft	181,500
Storage allocation below spillway crest		
Flood control	acre-ft	144,500
Sedimentation (100-year storage)	acre-ft	37,000
Standard-project flood		
Total volume (4 days)	acre-ft	160,000
Peak inflow	cfs	126,000
Peak outflow	cfs	6,000
Probable maximum flood		
Total volume	acre-ft	508,200
Peak inflow	cfs	265,000
Peak outflow	cfs	256,000

TABLE 2
Prado Dam
Pertinent Data for Dam of 1940 Design

Drainage area	sq mi	2,255
Dam (rolled earthfill)		
Crest elevation	ft msl	566
Maximum height above streambed	ft	106
Crest length	ft	2,280
Freeboard	ft	10
Spillway (detached, overflow concrete)		
Crest elevation	ft msl	543
Crest length	ft	1,000
Elevation of maximum water surface	ft msl	556
Outlet works (6-gated)		
Conduit Dimension	ft	13.5x13.5
Length of conduit	ft	750
Intake elevation	ft msl	460
Reservoir		
Area at spillway crest	acre	6,695
Capacity (gross) at spillway crest	acre ft	212,500
Storage allocation below spillway crest		
Flood control	acre-ft	200,500
Sedimentation (100-year storage)	acre-ft	12,000
100-year flood*		
Total volume (7 days)	acre-ft	275,200
Peak inflow	cfs	193,000
Peak outflow	cfs	9,350
Probable maximum flood*		
Total volume	acre-ft	233,000
Peak inflow	cfs	289,000
Peak outflow	cfs	181,000

*Design floods of 1940

TABLE 3
Prado Dam
Pertinent data for alternative designs

		All River Plan	National Economic Development Plan
Drainage area	sq mi	2,225	2,255
Dam (rolled earthfill)			
Crest elevation	ft msl	596	611
Maximum height above streambed	ft	136	151
Crest length	ft	3,890	3,910
Freeboard	ft	10	10
Spillway (detached, overflow concrete)			
Crest elevation	ft ms	563	579
Crest length	ft	1,300	1,300
Elevation of maximum water surface	ft msl	586	599
Outlet works (6 gates, 2-conduit)			
Diameter of conduit	ft	25	25
Length of conduit	ft	1,600	1,600
Intake elevation	ft msl	470	470
Saddle dike			
Crest elevation	ft msl	596	609
Crest length	ft	7,800	9,300
Maximum height above existing ground	ft	56	69
Reservoir			
Area at spillway crest	acre	10,400	14,500
Capacity (gross) at spillway crest	acre-ft	363,000	562,500
Storage allocation below spillway crest			
Flood control	acre-ft	300,000	480,000
Sedimentation (100-year storage)	acre-ft	63,000	82,500
Standard-project flood			
Total volume (4 days)	acre-ft	426,000	574,000
Peak inflow	cfs	265,000	317,000
Peak outflow	cfs	30,000	30,000
Probable maximum flood			
Total volume	acre-ft	1,570,000	1,570,000
Peak inflow	cfs	700,000	700,000
Peak outflow	cfs	605,000	605,000

TABLE 4
Mentone Dam
Detailed Cost Estimates Under All River Plan
(October 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Flood Control Cost Construction and Railroad Modification Cost:					
RELOCATIONS					
Relocate Santa Fe Single Track Railroad					
Earthwork	CY	1,610,700	$0.60	$966,000	
Trackage	L.F.	34,900	47.98	1,675,000	
Bridge	S.F.	24,400	280.00	6,832,000	
Dam Cost:					
Diversion and Control of Water	JOB	1	L.S.	1,200,000	
Clear and Remove Obstruction	AC	430	5348.84	2,300,000	
Excavation					
Basin	CY	56,036,000	1.55	86,856,000	
Impervious Material	CY	10,344,000	4.15	42,928,000	
Cutoff	CY	1,555,000	1.25	1,944,000	
Foundation	CY	3,390,000	1.25	4,238,000	
Embankment					
Bedding Material	CY	308,000	7.50	2,310,000	
Impervious Material	CY	10,344,000	0.22	2,276,000	
Pervious and Rock	CY	46,738,000	0.18	8,413,000	
Filter Blanket	CY	3,258,500	5.80	18,899,000	
Riprap	CY	615,000	5.80	3,567,000	
Transition	CY	4,441,700	0.23	1,022,000	
Bentonite Slurry	CY	261,200	27.00	7,052,000	
A.C. Paving	TON	3,920	35.00	137,000	
Outlet Works					
Earthwork	JOB	1	L.S.	630,000	
Backfill	JOB	1	L.S.	205.000	
Concrete					
Intake and Trash Structure	JOB	1	L.S.	283,000	
Gate Structure	JOB	1	L.S.	650,000	
Conduit Transition	JOB	1	L.S.	1,016,000	
Conduit	CY	12,590	169.34	2,132,000	
Outlet Structure	CY	3,040	151.00	459,000	
Access Gallery	JOB	1	L.S.	1,455,000	
Grout With Cement	CY	170	95.18	16,000	
Dumped Stone	CY	1,330	11.10	15,000	
Trash Rack	JOB	1	L.S.	119,000	
Stop Log	JOB	1	L.S.	95,000	
Control and Generator Houses	JOB	1	L.S.	30,000	
Hydrographic Facilites	JOB	1	L.S.	95,000	
Slide Gate	EA	3	350,000	1,050,000	

TABLE 4 (Continued)

DESCRIPTION	UNIT	QUALITY	UNIT PRICE	SUBTOTAL	TOTAL
Electrical System	JOB	1	L.S.	$200,000	
Other Equipment	JOB	1	L.S.	762,000	
Ladder, Platforms, and Appurtenances	JOB	1	L.S.	100,000	
Float Well and Gate Position Indicator	JOB	1	L.S.	46,000	
Spillway					
Excavation	JOB	1	L.S.	727,000	
Backfill	JOB	1	L.S.	918,000	
Concrete					
Ogee	CY	56,130	$67.00	3,761,000	
Cutoff Walls	CY	3,610	40.00	144,000	
Walls	CY	10,700	120.00	1,284,000	
Wall Footing	CY	11,900	49.00	583,000	
Invert Slab	CY	264,500	35.00	9,258,000	
Cement	CWT	1,958,100	6.00	11,749,000	
Rein. Steel	TON	10,750	720.00	7,740,000	
Structural Steel	L.F.	60,320	17.90	1,080,000	
Grout With Cement	CY	1,340	95.22	128,000	
Down Stream End Protection Sta. 48+00 to Sta. 40+25	JOB	1	L.S.	3,216,000	
Subdrain System	JOB	1	L.S.	1,170,000	
Beautification	JOB	1	L.S.	1,077,000	
Mill Creek Levee					
Diversion and Control of Water	JOB	1	L.S.	10,000	
Clear and Remove Obstruction	AC	28	500.00	14,000	
Excavation	JOB	1	L.S.	25,000	
Backfill	JOB	1	L.S.	151,000	
Grout Stone	TON	44,460	15.00	667,000	
Grout With Cement	CY	34,500	95.22	3,285,000	
Groins	JOB	1	L.S.	7,925,000	
Subtotal				257,375,000	
Contingencies (25%)				64,344,000	
Subtotal				321,719,000	
Engineering and Design (7%)				22,520,000	
Supervision And Administration (5%)				16,086,000	
Total Construction					$360,325,000
Lands and Relocations					
Lands	JOB	1	L.S.	21,500,000	
Relocations: 78-inch waterline	JOB	1	L.S.	3,863,000	
Others	JOB	1	L.S.	633,000	
Total, Lands and Relocation					25,996,000
Grand Total, First Cost for Mentone Dam					$386,321,000

TABLE 5
Oak Street Drain
Detailed Cost Estimate Under All River Plan
(October 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Construction and Railroad Modification Costs:					
Railroad Bridge Modification	EA	1	L.S.	$170,000	
Channel Costs					
Diversion and Control Water	JOB	1	L.S.	16,000	
Clear and Remove Obstructions	AC	22	$14,282	314,000	
Excavation	CY	227,000	2.10	477,000	
Compacted Fill	CY	105,000	1.70	179,000	
Concrete					
Walls	CY	10,300	76.00	782,000	
Invert	CY	14,600	36.00	526,000	
Cement	CWT	137,000	6.00	822,000	
Reinf. Steel	TON	980	720.00	706,000	
Box Section (Sta. 32+00 to Sta. 46+00)	JOB	1	L.S.	752,000	
Mangular Channel	JOB	1	L.S.	122,000	
Downstream End	JOB	1	L.S.	148,000	
Side Drain Structures	JOB	1	L.S.	108,000	
Shoring at Church and Trailer Park	JOB	1	L.S.	52,000	
A.C. Paving	TON	3,800	35.00	133,000	
Fencing Including Gates	L.F.	30,200	6.06	183,000	
Beautification	JOB	1	L.S.	118,000	
Subtotal				5,608,000	
Contingencies (25%)				1,402,000	
Subtotal, Construction				7,010,000	
Engineering and Design (10%)				710,000	
Supervision and Administration(7%)				491,000	
Total, Construction					$8,202,000
Lands and Relocation					
Land	JOB	1	L.S.	930,000	
Highway Bridges	JOB	1	L.S.	1,527,000	
Utilities	JOB	1	L.S.	390,000	
Total Lands and Relocations					2,847,000
Total Project, Oak Street Drain					11,049,000

TABLE 6
Prado Dam
Detailed Cost Estimates Under All River Plan
(October 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Flood control costs-					
Dam Construction:					
Diversion and control of water	JOB	1	L.S.	$199,000	
Clear and Remove Obstructions	JOB	1	L.S.	171,000	
Borrow Excavation	CY	2,532,502	$1.40	3,546,000	
Main Embankment and Toe Excavation	CY	1,371,500	1.40	1,920,000	
Auxiliary Dike	CY	365,000	0.80	292,000	
Embankment					
Prado Dam	CY	1,264,539	0.60	759,000	
Bentonite Slurry Cutoff	JOB	1	L.S.	520,000	
Dike	CY	988,093	0.60	592,000	
Replace Cobbles	CY	41,700	7.40	309,000	
Grouted Gutters	CY	2,080	45.00	94,000	
Riprap U.S. and D.S. Face	CY	140,133	22.60	3,167,000	
Sand And Gravel Drains	CY	279,870	7.72	2,161,000	
Dike and Embankment Roads	S.Y.	34,933	8.10	283,000	
Modification of Spillway					
Excavation					
Rock	CY	1,141,100	6.70	7,645,000	
Common	CY	197,100	0.70	138,000	
Cribbing	CY	87,000	67.00	5,829,000	
Backfill					
Structural	CY	37,600	3.35	126,000	
Common	CY	197,100	0.80	158,000	
Cribbing	CY	52,000	6.70	348,000	
Demolition of Spillway Walls	CY	4,200	60.00	252,000	
Demolition of Spillway Invert	CY	4,700	60.00	282,000	
Concrete Chipping	CY	370	134.00	50,000	
Sandblasting	S.F.	57,400	1.34	77,000	
Drill 2"Ø Dowell Holes	L.F.	7,100	10.00	71,000	
Grout and Place Dowels	EA	3,600	10.00	36,000	
Sheet Piling	S.F.	38,800	8.25	320,000	
Subdrain	JOB	1	L.S.	57,000	
Concrete					
Cutoff Walls	CY	1,700	40.00	68,000	
Ogee Section	CY	45,119	67.00	3,023,000	
Invert	CY	14,100	80.00	1,128,000	
Flip Bucket Invert	CY	6,900	67.00	462,000	
Flip Bucket Walls	CY	100	140.00	14,000	
Freeway Retaining Walls	CY	12,940	143.00	1,850,000	
Freeway Retaining Wall Toe Stone	CY	15,000	13.60	204,000	
Crib Cutoff Wall	CY	30,100	47.00	1,415,000	
Rebuild Existing Spillway Invert	JOB	1	L.S.	648,000	

TABLE 6 (Continued)
Prado Dam
Detailed Cost Estimate Under All River Plan

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Right and Left Spillway Walls Sta. 10+00 to Sta. 13+00	CY	5,800	$110.00	$ 638,000	
Right and Left Spillway Walls Sta. 13+00 to Sta. 22+10	CY	5,055	143.00	723,000	
Cement	CWT	23,560	5.50	130,000	
Rein Steel	TON	5,956	720.00	4,288,000	
Tunnel and Outlet Works					
Excavation					
Outlet Works	CY	28,900	2.01	58,000	
Tunnel Transition	CY	86,700	2.01	174,000	
Tunnel	CY	65,500	67.00	4,389,000	
Stilling Basin	CY	93,500	0.94	88,000	
Toe	CY	10,000	1.00	10,000	
Disposal of Surplus Material Waste	CY	150,100	0.67	101,000	
Backfill					
Outlet Structure	CY	29,500	1.34	40,000	
Stilling Basin	CY	34,800	1.34	47,000	
Toe	CY	4,900	0.67	3,000	
Dumped Stone	CY	7,400	16.75	124,000	
Tunnel					
Steel Supports	LB	4,200,000	0.74	3,108,000	
Wood Lagging	MBM	125	804.00	101,000	
Chainlink Fabric	SY	2,850	8.00	23,000	
Concrete Lining	CY	19,800	100.00	1,980,000	
Rein-Steel	TON	891	720.00	642,000	
Cement	CWT	111,700	5.50	614,000	
Intake Tower					
Gate and Regulating Equipment	JOB	1	L.S.	1,943,000	
Trash Rack	JOB	1	L.S.	268,000	
Stop Log	JOB	1	L.S.	134,000	
Concrete	CY	20,400	143.00	2,734,000	
Rein-Steel	TON	1,020	720.00	734,000	
Cement	CWT	115,056	5.50	633,000	
Inlet Transition					
Concrete	CY	15,281	168.00	2,567,000	
Rein-Steel	TON	764	720.00	550,000	
Cement	CWT	86,185	5.50	474,000	
Dumped Stone	CY	1,000	16.75	17,000	
Outlet Transition					
Concrete	CY	20,200	87.00	1,757,000	
Rein-Steel	TON	1,505	720.00	1,084,000	
Cement	CWT	113,310	5.50	632,000	
Tunnel Grouting	JOB	1	L.S.	189,000	
Plug Existing Outlet	JOB	1	L.S.	283,000	

TABLE 6 (Continued)
Prado Dam
Detailed Cost Estimate Under All River Plan

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Outlet Works					
Service Bridge and Appurtenances	JOB	1	L.S.	$606,000	
Elevator	JOB	1	L.S.	134,000	
Gate Position Indicator	JOB	1	L.S.	49,000	
Electrical System	JOB	1	L.S.	35,000	
Control House and Equipment	JOB	1	L.S.	150,000	
Generator House and Equipment	JOB	1	L.S.	95,000	
Floatwell System	JOB	1	L.S.	14,000	
Hydrographic Facilities	JOB	1	L.S.	97,000	
Overhead Hoist (10 Ton)	JOB	1	L.S.	11,000	
Beautification	JOB	1	L.S.	1,776,000	
Ring Dikes	JOB	1	L.S.	1,023,000	
Subtotal				73,829,000	
Contingencies (25%)				18,457,000	
Subtotal Dam				92,286,000	
Engineering and Design (10%)				9,229,000	
Supervision and Construction (7%)				6,460,000	
Total Construction					$107,975,000
Lands and Relocations					
Upgrade Title to Land Below El. 556	JOB	1	L.S.	31,300,000	
Land Above El. 556*	JOB	1	L.S.	60,000,000	
Corona Freeway	JOB	1	L.S.	7,233,000	
Utilities	JOB	1	L.S.	1,072,000	
Total, Lands and Relocations					99,605,000
Grand Total, First Costs Prado Dam and Reservoir					207,580,000

TABLE 7
Santiago Creek Channel
Detailed Cost Estimate Under All River Plan
(October 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Channel Costs					
Diversion and Control of Water	JOB	1	L.S.	20,000	
Inlet					
Excavation	CU.YD	57,000	$0.90	$51,000	
Fill, Compacted	CU.YD	19,000	1.50	29,000	
Concrete					
Cutoff	CU.YD	30	100.00	3,000	
"L" Wall	CU.YD	730	100.00	73,000	
Box Wall and Pier	CU.YD	270	150.00	41,000	
"L" Footing	CU.YD	880	65.00	57,000	
Box Footing	CU.YD	215	65.00	14,000	
Box Top Slab	CU.YD	200	200.00	40,000	
Center Slab Includes 18" Slab and 24" Sides	CU.YD	1,480	90.00	133,000	
Baffles	CU.YD	170	400.00	68,000	
Cement	CWT	22,420	6.00	129,000	
Reinforcing Steel	TON	188	720.00	135,000	
2' Rock Blanket	CU.YD	1,160	22.50	26,000	
2.5' Rock Blanket	CU.YD	70	22.50	2,000	
7' Rock Blanket	CU.YD	450	22.50	10,000	
6' Chain Link Fence	LIN.FT	1,010	5.50	6,000	
Compacted Fill, Basin	CU.YD	1,332,000	0.60	799,000	
Outlet					
Excavation	CU.YD	429,000	0.90	386,000	
Fill	CU.YD	7,100	0.60	4,000	
Misc. Fill	CU.YD	2,800	0.25	1,000	
Concrete					
Wall	CU.YD	1,240	200.00	248,000	
Footing	CU.YD	1,270	65.00	91,000	
Side Slope	CU.YD	140	80.00	11,000	
Invert	CU.YD	210	65.00	21,000	
Cement	CWT	16,130	6.00	105,000	
Reinforcing Steel	TON	152	720.00	117,000	
18" Rip Rap	CU.YD	17,300	22.50	389,000	
Neyrpic Gate Avio 280L	EA	3	50,000.00	150,000	
6' Chain Link Fence	LIN.FT	320	5.50	2,000	
A.C. Paving	TON	480	35.00	17,000	
Channel					
Excavation	CU.YD	84,000	0.90	76,000	
Fill	CU.YD	15,000	0.60	9,000	
18" Rip Rap Blanket	CU.YD	29,400	22.50	662,000	

TABLE 7 (Continued)
Santiago Creek Channel
Detailed Cost Estimate Under All River Plan

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Beautification	JOB	1	L.S.	$ 312,000	
Subtotal				4,237,000	
Contingencies (25%)				1,060,000	
Subtotal, Construction				5,297,000	
Engineering and Design (10%)				530,000	
Supervision and Administration (7%)				371,000	
Total, Construction					$6,198,000
Lands And Relocations					
Lands	JOB	1	L.S.	3,500,000	
Bridge	SQ.FT.	4,240	70.00	297,000	
Utilities	JOB	1	L.S.	8,000	
Total, Lands and Relocations					3,805,000
Total Costs, Santiago Creek					$10,003,000

TABLE 8
Lower Santa Ana River Including Greenville-Banning Channel
Detailed Cost Estimate Under All River Plan
(October 1979 Price Level

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Flood Control Costs					
Construction And Railroad Modification Costs					
Railroad Shoofly	S.F.	26,860	$210.00	$5,641,000	
Railroad Bridges	S.F.	26,860	280.00	7,521,000	
Channel					
Diversion and Control of Water	JOB	1	L.S.	1,283,000	
Clear And Grub	JOB	1	L.S.	979,000	
Stone Removal	TON	871,000	8.00	6,968,000	
Concrete Removal	CY	86,000	35.00	3,010,000	
Earth Work					
Channel Excavation	CY	11,718,000	2.40	28,123,000	
Toe Excavation	CY	1,467,000	2.35	3,477,000	
Subgrade Preparation	S.Y.	1,145,000	0.10	115,000	
Levee Fill	CY	45,000	0.35	16,000	
Channel Wall Fill	CY	3,259,000	1.35	4,400,000	
Toe Backfill	CY	1,115,000	0.95	1,059,000	
Misc. Fill	CY	389,000	1.00	389,000	
Grout	CY	5,000	50.00	250,000	
Stone Levee	TON	966,000	15.00	14,490,000	
Filter Levee	CY	198,000	22.50	4,455,000	
Concrete					
Wall	CY	130,000	76.00	9,880,000	
Footing and Invert	CY	421,000	49.00	20,629,000	
Cutoff Wall	CY	15,500	505.00	7,828,000	
Cement	CWT	3,233,000	6.00	19,398,000	
Rein Steel	TON	26,790	720.00	19,418,000	
Subdrain System	JOB	1	L.S.	6,965,000	
Side Drains	JOB	1	L.S.	780,000	
A.C. Paving	TON	28,400	35.00	994,000	
Fencing	L.F.	138,600	5.50	762,000	
Drop Structures	S.F.	59,500	38.40	2,285,000	
Beautification	JOB	1	L.S.	2,309,000	
Bridge Over Greenville-Banning channel	JOB	1	L.S.	286,000	
Subtotal				173,710,000	
Contingencies (25%)				43,428,000	
Subtotal Channel				217,138,000	
Engineering And Design (10%)				21,714,000	
Supervision And Administration (7%)				15,200,000	
Total Construction					$254,052,000

TABLE 8 (Continued)
Lower Santa Ana River Including Greenville-Banning Channel
Detailed Cost Estimate Under All River Plan
(October 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Lands And Relocations					
R/W Santa Ana Canyon	JOB	1	L.S.	$13,000,000	
R/W Urban Reach	JOB	1	L.S.	6,040,000	
R/W Mitigation And Wildlife	JOB	1	L.S.	4,220,000	
Subtotal R/W Costs				23,260,000	
Relocations					
Roads and Bridges	JOB	1	L.S.	21,376,000	
Utilities	JOB	1	L.S.	1,447,000	
Total Lands And Relocations					$46,083,000
Total Flood Control Costs					300,135,000

TABLE 9
Comparison of First Costs Under All River Plan

Feature	BEHR Revised Estimate (Sep. 1975)	PB-3 (Oct. 1979)	Present Est. (Oct. 1979)
Mentone Dam			
Rights-of-Way and Relations	$31,700,000	$30,028,000	$25,996,000
Rights-of-Way	(27,500,000)	(30,028,000)	(21,500,000)
Relocation Excluding Railroad	(4,200,000)	(Included in R/W)	(4,496,000)
Dam Including Railroad	317,900,000	371,500,000	321,719,000
Levee	Included in Dam	8,779,000	(Included with Dam)
Engineering and Design	" " "	19,063,000	22,520,000
Supervision and Administration	" " "	26,745,000	16,086,000
Subtotal	349,600,000	456,115,000	386,321,000
Oak Street Drain			
Rights-of-Way and Relocations	1,670,000	2,351,000	2,847,000
Rights-of-Way	(472,000)	(798,000)	(930,000)
Relocations Excluding Railroad	(1,198,000)	(1,553,000)	(1,917,000)
Channels Including Railroad	6,080,000	8,473,000	7,010,000
Engineering and Design	425,000	575,000	701,000
Supervision and Administration	305,000	406,000	491,000
Subtotal	8,480,000	11,805,000	11,049,000
Prado Dam			
Rights-of-Way and Relocations	79,000,000	109,414,000	99,605,000
Lands and Damages	-------	(88,584,000)	(91,300,000)
Relocations	-------	(20,830,000)	(8,305,000)
Dam	61,770,000	85,277,000	92,286,000
Engineering and Design	3,090,000	5,358,000	9,229,000
Supervision and Administration	4,340,000	7,246,000	6,460,000
Subtotal	148,200,000	207,295,000	207,580,000
Santiago Creek			
Rights-of-Way and Relocations	4,000,000	5,533,000	3,805,000
Rights-of-Way	(1,500,000)	(2,076,000)	(3,500,000)
Relocations Excluding Railroad	(2,500,000)	(3,457,000)	(305,000)
Channel Including Railroad	9,450,000	13,089,000	5,297,000
Engineering and Design	800,000	1,092,000	530,000
Supervision and Administration	750,000	1,031,000	371,000
Subtotal	15,000,000	20,745,000	10,003,000

Table 9 (Cont'd)

Feature	BEHR Revised Estimate (Sep. 1975)	PB-3 (Oct. 1979)	Present Est. (Oct. 1979)
Lower SantaAna River			
Rights-of-Way and Relocations	67,250,000	93,368,000	46,083,000
Rights-of-Way	(41,000,000)	(56,930,000)	(23,260,000)
Relocations Excluding Railroad	(26,250,000)	(36,438,000)	(22,823,000)
Mitigation and Preservation	240,000	2,700,000	(Included in R/W)
Channels Including Railroad	131,818,000	182,560,000	217,138,000
Engineering and Design	6,596,000	9,068,000	21,714,000
Supervision and Administration	9,236,000	12,723,000	15,200,00
Subtotal	215,140,000	300,419,000	300,135,000
Total--Flood Control including Mitigation	$736,420,000	$996,379,000	$915,088,000
SUMMARY			
Rights-of-way including Mitigation and Preservation		$181,116,000	$141,123,000
Relocations		62,278,000	37,213,000
Construction		669,678,000	643,450,000
Engineering and Design		35,156,000	54,694,000
Supervision and Administration		48,151,000	38,608,000
Total--Flood Control including Mitigation and Preservation		$996,379,000	$915,088,000

TABLE 10
Prado Dam
Detailed Cost Estimates Under
Plan to Provide SPF Protection Below Dam
(October 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Flood Control Costs					
Dam Construction					
Diversion and Control of Water	JOB	1	L.S.	$199,000	
Clear And Remove Obstructions	JOB	1	L.S.	171,000	
Earthwork					
Borrow Excavation	CY	4,385,730	$1.40	6,140,000	
Main Embankment And Toe Excavation	CY	1,371,500	1.40	1,920,000	
Auxuliary Dike (Native Removal)	CY	698,520	0.80	559,000	
Embankment					
Prado Dam	CY	2,075,665	0.60	1,245,000	
Bentonite Slurry Cutoff	CY	35,000	14.85	520,000	
Santa Fe Railway levee	CY	1,810,565	0.60	1,086,000	
Replace Cobbles	CY	52,600	7.40	389,000	
Grouted Gutters	CY	2,630	45.00	118,000	
Riprap U.S. and D.S Face	CY	142,200	22.60	3,214,000	
Sand and Gravel Drains	CY	499,500	7.72	3,856,000	
Dike and Embankment Roads	S.Y.	44,466	8.10	360,000	
Instrumentation	JOB	1	L.S.	345,000	
Modification of Spillway					
Excavation					
Rock	CY	1,141,100	6.70	7,645,000	
Common	CY	197,100	0.70	138,000	
Cribbing	CY	87,000	67.00	5,829,000	
Backfill					
Structural	CY	37,600	3.35	126,000	
Common	CY	197,100	0.80	158,000	
Cribbing	CY	52,000	6.70	348,000	
Demolition of Spillway walls	CY	4,200	60.00	252,000	
Demolition of Existing Spillway Invert	CY	4,700	60.00	282,000	
Concrete Chipping	CY	370	134.00	50,000	
Sandblasting	S.F.	57,400	1.34	77,000	
Drill 2"Ø Dowell Holes	L.F.	7,100	10.00	71,000	
Grout and Place 1-1/4x4 Dowels	EA	3,600	10.00	36,000	
Sheet Piling	S.F.	38,800	8.25	320,000	
Subdrain	JOB		L.S.	57,000	
Concrete					
Cutoff Walls	CY	1,700	40.00	68,000	
Ogee Section	CY	65,345	316.00	20,649,000	
Invert	CY	14,100	80.00	1,128,000	
Flip Bucket Invert	CY	6,900	67.00	462,000	
Flip Bucket Walls	CY	100	143.00	14,000	
Freeway Retaining Walls	CY	12,940	143.00	1,850,000	
Crib Cutoff Wall	CY	30,100	47.00	1,415,000	
Rebuild Existing Spillway Invert	JOB	1	L.S.	648,000	

TABLE 10 (Continued)
Prado Dam
Detailed Cost Estimates Under
Plan to Provide SPF Protection Below Dam
(October 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Right and Left Spillway Walls Sta. 10+00 to Sta. 13+00	CY	9,180	$143.00	$1,313,000	
Right and Left Spillway Walls Sta. 13+00 to Sta. 22+10	CY	9,100	143.00	1,301,000	
Cement	CWT	24,029	5.50	132,000	
Rein Steel	TON	7,483	720.00	5,388,000	
Tunnel and Outlet Works					
Excavation					
Outlet Works	CY	28,900	2.01	58,000	
Tunnel Transition	CY	86,700	2.01	174,000	
Tunnel	CY	65,500	67.00	4,389,000	
Stilling Basin	CY	93,500	0.94	88,000	
Toe	CY	10,000	1.00	10,000	
Waste	CY	150,000	0.67	101,000	
BackFill					
Outlet Structure	CY	29,500	1.34	40,000	
Stilling Basin	CY	34,800	1.34	47,000	
Toe	CY	4,900	0.67	3,000	
Dumped Stone	CY	7,400	16.75	124,000	
Tunnel					
Steel Supports	LB	4,200,000	0.74	3,108,000	
Wood Lagging	MBM	125	804.00	101,000	
Chainlink Fabric	SY	2,850	8.00	23,000	
Concrete	CY	19,800	100.00	1,980,000	
Rein-Steel	TON	891	720.00	642,000	
Cement	CWT	111,700	5.50	614,000	
Intake Tower					
Gate and Regulating Equipment	JOB	1	L.S.	1,943,000	
Trash Rack	JOB	1	L.S.	268,000	
Stop Log	JOB	1	L.S.	134,000	
Concrete	CY	20,400	134.00	2,734,000	
Rein Steel	TON	1,020	720.00	734,000	
Cement	CWT	115,056	5.50	633,000	
Inlet Transition					
Concrete	CY	15,281	168.00	2,567,000	
Rein-Steel	TON	764	720.00	550,000	
Cement	CWT	86,185	5.50	474,000	
Dumped Stone	CY	1,000	16.75	17,000	
Outlet Transition					
Concrete	CY	20,200	87.00	1,757,000	
Rein-Steel	TON	1,505	720.00	1,084,000	
Cement	CWT	113,310	5.50	632,000	
Tunnel Grouting	JOB	1	L.S.	189,000	
Plug Existing Outlet	JOB	1	L.S.	283,000	

TABLE 10 (Continued)
Prado Dam
Detailed Cost Estimates Under
Plan to Provide SPF Protection Below Dam
(Octoer 1979 Price Level)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	SUBTOTAL	TOTAL
Outlet Works					
Service Bridge and Appurtenances	JOB	1	L.S.	$606,000	
Elevator	JOB	1	L.S.	134,000	
Gate Position Indicator	JOB	1	L.S.	49,000	
Electrical System	JOB	1	L.S.	35,000	
Control House And Equipment	JOB	1	L.S.	150,000	
Generator House And Equipment	JOB	1	L.S.	95,000	
Floatwell System	JOB	1	L.S.	14,000	
Hydrographic Facilities	JOB	1	L.S.	97,000	
Overhead Hoist (10 Ton)	JOB	1	L.S.	11,000	
Beautification	JOB	1	L.S.	1,776,000	
Ring Dike	JOB	1	L.S.	2,910,000	
Subtotal				101,323,000	
Contingencies (25%)				25,331,000	
Subtotal Dam				126,654,000	
Engineering and Design (10%)				12,665,000	
Supervision and Construction (7%)				8,866,000	
Total Construction					$148,185,000
Lands and Relocations					
Upgrade Title To Land Below El. 556	JOB	1	L.S.	31,300,000	
Land Above El. 556	JOB	1	L.S.	302,700,000	
Corona Freeway Relocation	JOB	1	L.S.	7,233,000	
Utilities	JOB	1	L.S.	1,072,000	
Total, Lands and Relocations					$342,305,000
Total Flood Control Costs					490,490,000

KEY

LEGEND

RECOMMENDED PLAN

CHANNEL IMPROVEMENT

DAM AND RESERVOIR

EXISTING

FLOOD CONTROL DAM

CHANNEL

LEVEE

BOUNDARY OF SANTA ANA RIVER DRAINAGE AREA

5 0 5

SCALE IN MILES

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

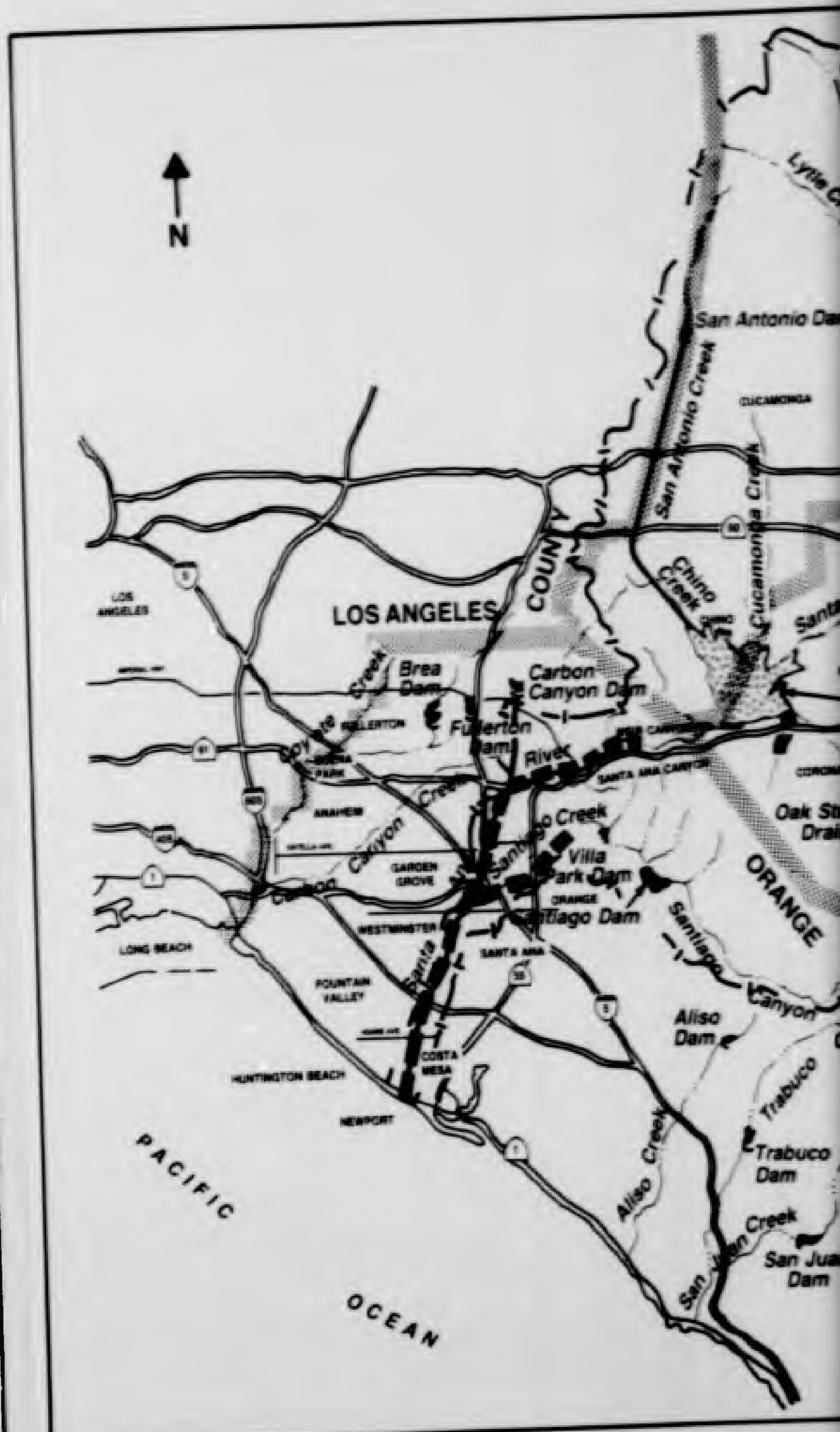
SANTA ANA RIVER, CALIFORNIA

PHASE I

GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER BASIN

LOCATION MAP



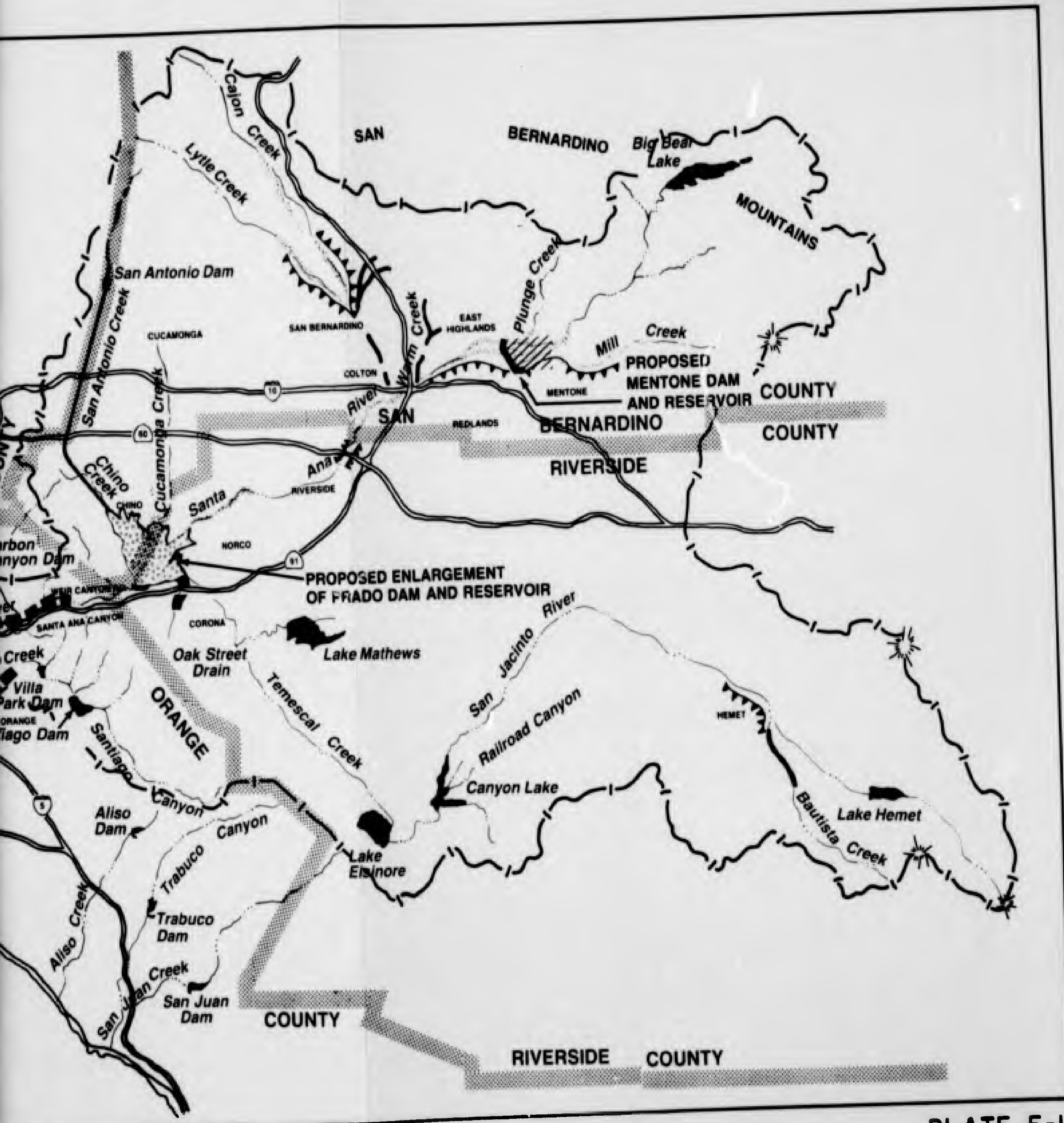
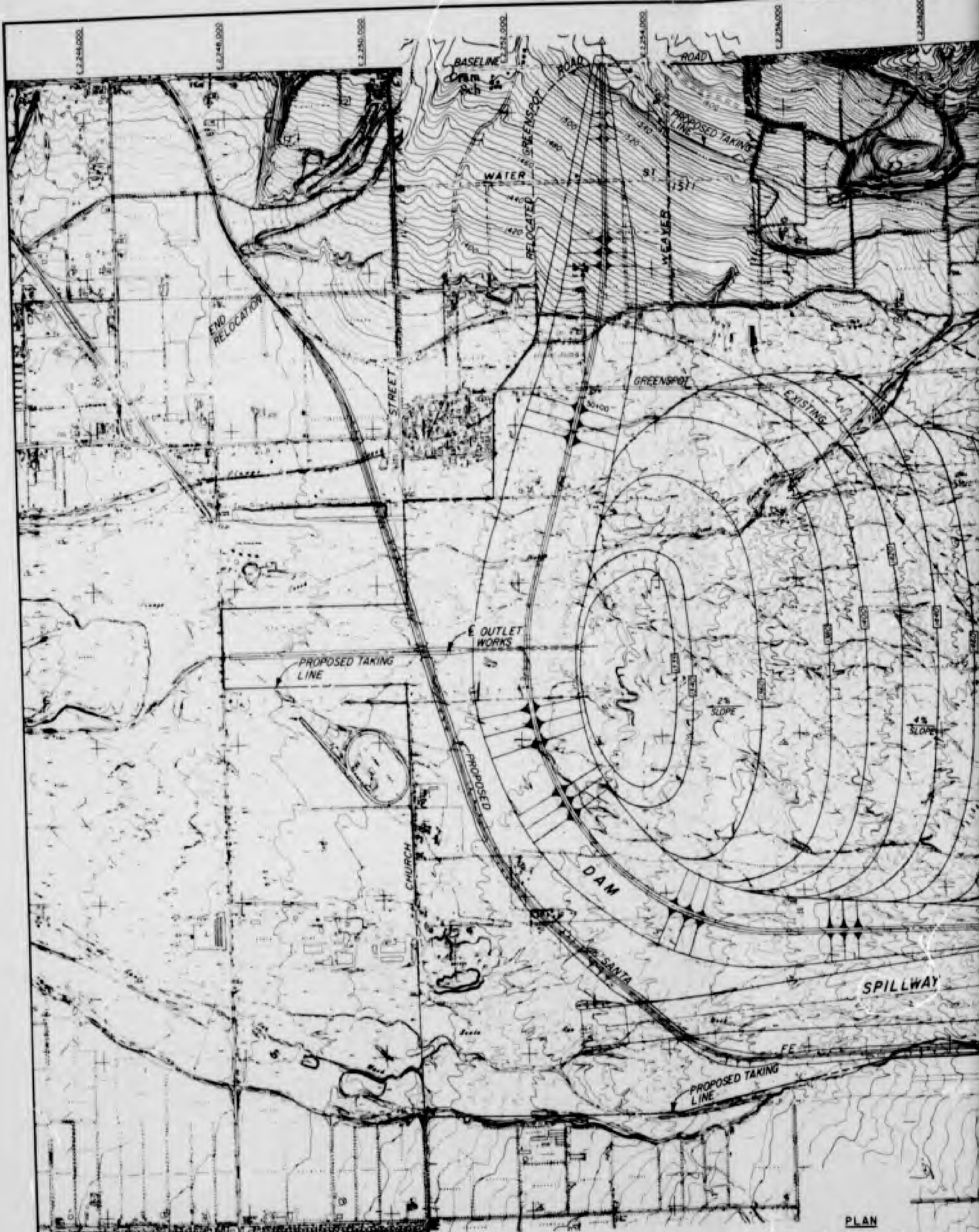

SAN
BERNARDINO
MOUNTAINS
Big Bear Lake
Cajon Creek
Lytle Creek
San Antonio Dam
San Antonio Creek
CUCAMONGA
Cucamonga Creek
SAN BERNARDINO
EAST HIGHLANDS
Plunge Creek
Warm Creek
Mill Creek
PROPOSED MENTONE DAM AND RESERVOIR
COLTON
MENTONE
REDLANDS
SAN BERNARDINO COUNTY
RIVERSIDE COUNTY
Santa Ana River
Chino Creek
CHINO
RIVERSIDE
NORCO
PROPOSED ENLARGEMENT OF PRADO DAM AND RESERVOIR
CORONA
SANTA ANA CANYON
Lake Mathews
Oak Street Drain
Temescal Creek
San Jacinto River
Railroad Canyon
Canyon Lake
HEMET
Lake Hemet
Bautista Creek
Lake Elsinore
Villa Park Dam
ORANGE
Santiago Canyon
Aliso Dam
Trabuco Canyon
Trabuco Dam
Aliso Creek
San Juan Creek
San Juan Dam
ORANGE COUNTY
RIVERSIDE COUNTY

PLATE F-1

2


BASELINE
ROAD
WATER
RELOCATED
GREENSPOT
PROPOSED TAKING LINE
WEAVER
ST
END RELOCATION
STREET
GREENSPOT
EXISTING
OUTLET WORKS
PROPOSED TAKING LINE
2% SLOPE
4% SLOPE
PROPOSED
CHURCH
DAM
SANTA
FE
SPILLWAY
PROPOSED TAKING LINE
PLAN

VALUE ENGINEERING PAYS

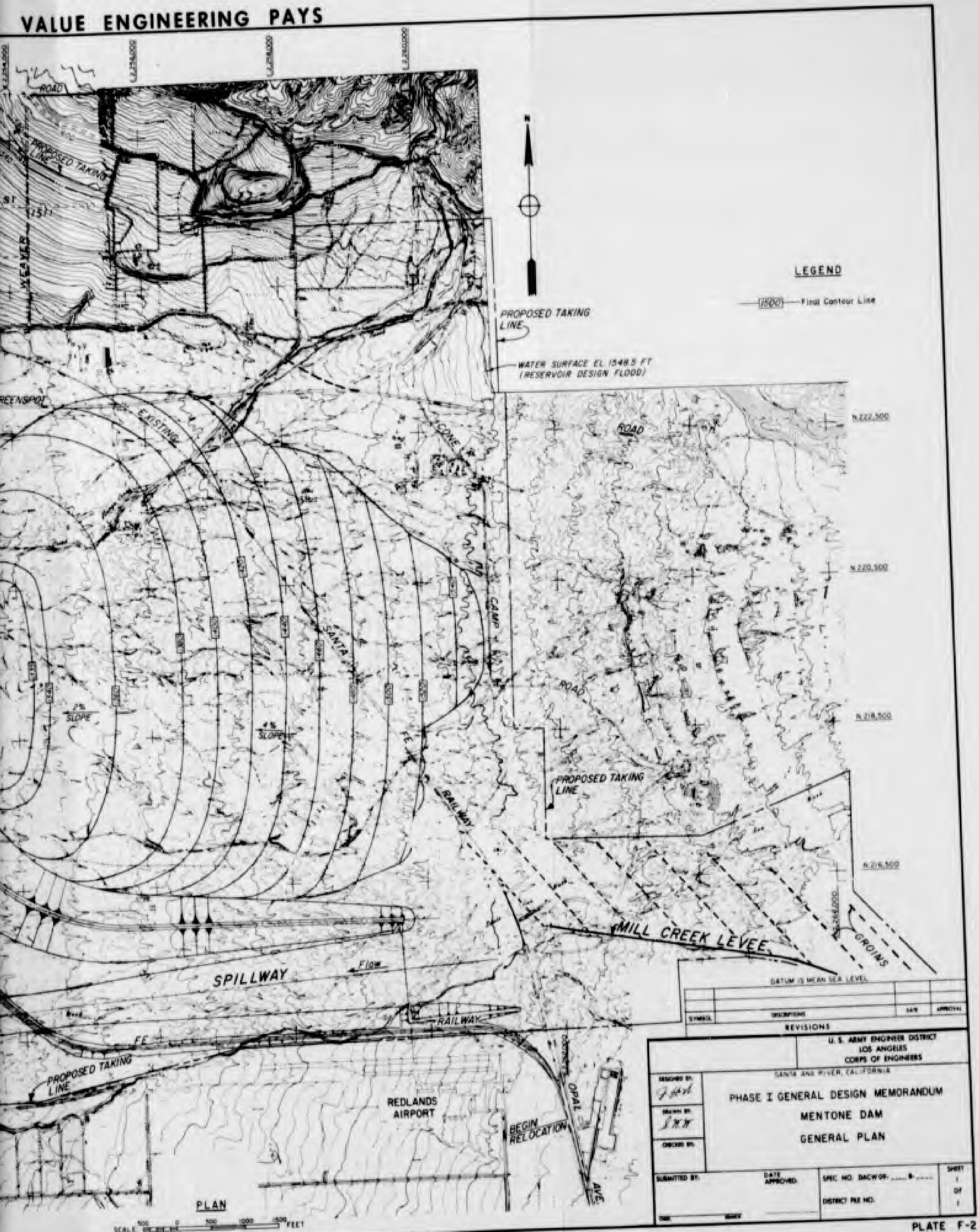
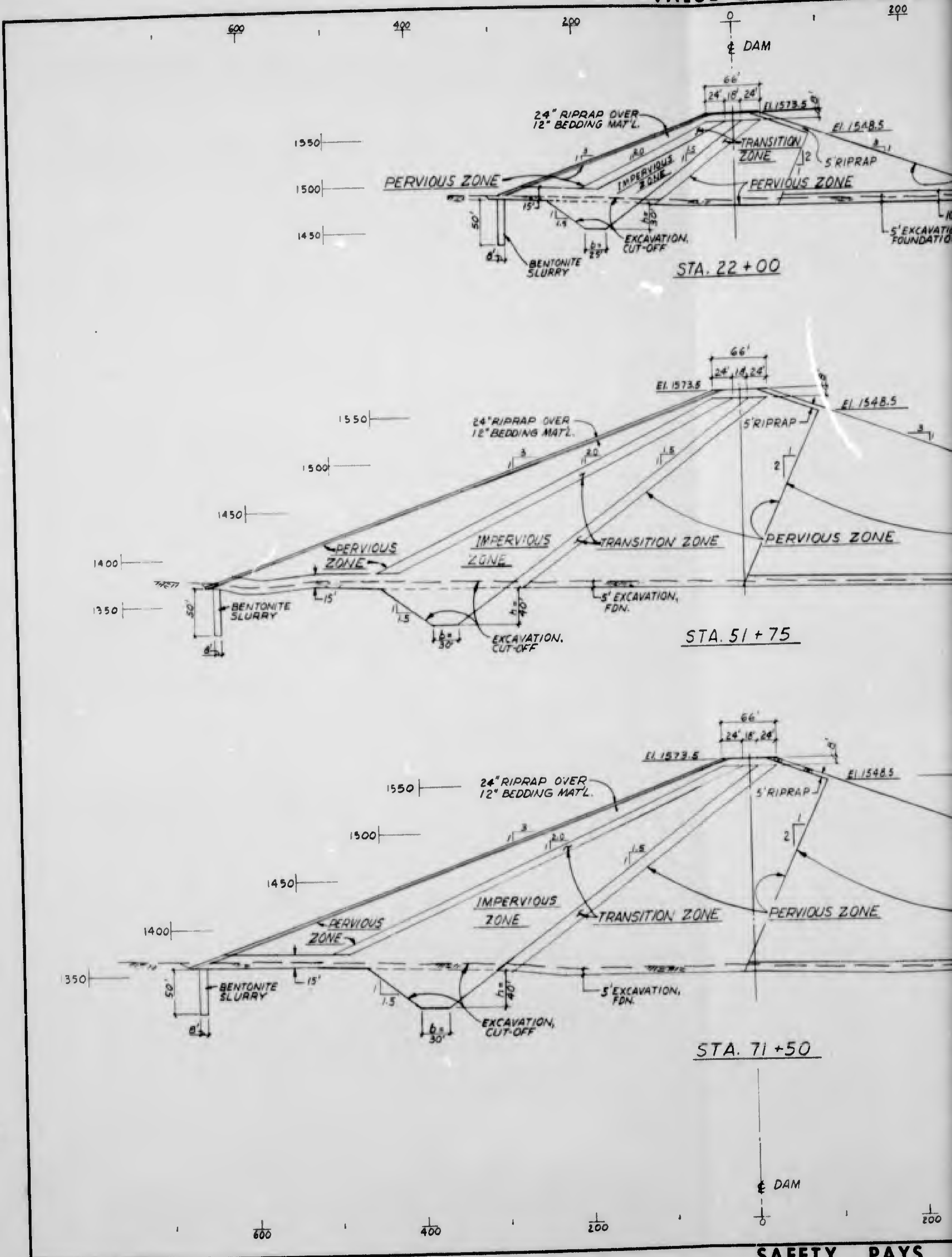
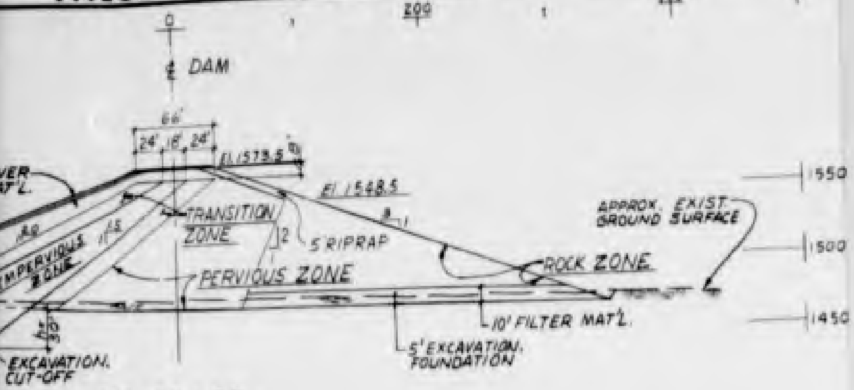


PLATE F-2

SAFETY PAYS



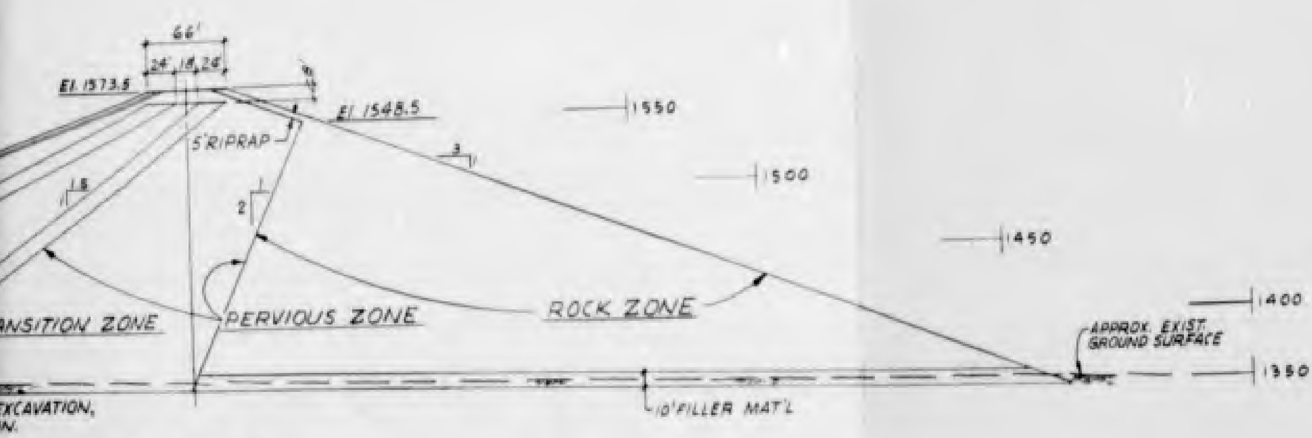
℄ DAM
66'
24' 18' 24'
El. 1573.5
El. 1548.5
24" RIPRAP OVER 12" BEDDING MAT'L.
PERVIOUS ZONE
IMPERVIOUS ZONE
TRANSITION ZONE
5' RIPRAP
5' EXCAVATION, FOUNDATION
EXCAVATION, CUT-OFF
BENTONITE SLURRY
STA. 22 + 00
5' EXCAVATION, FDN.
STA. 51 + 75
STA. 71 + 50
600
400
200
0
1350
1400
1450
1500
1550



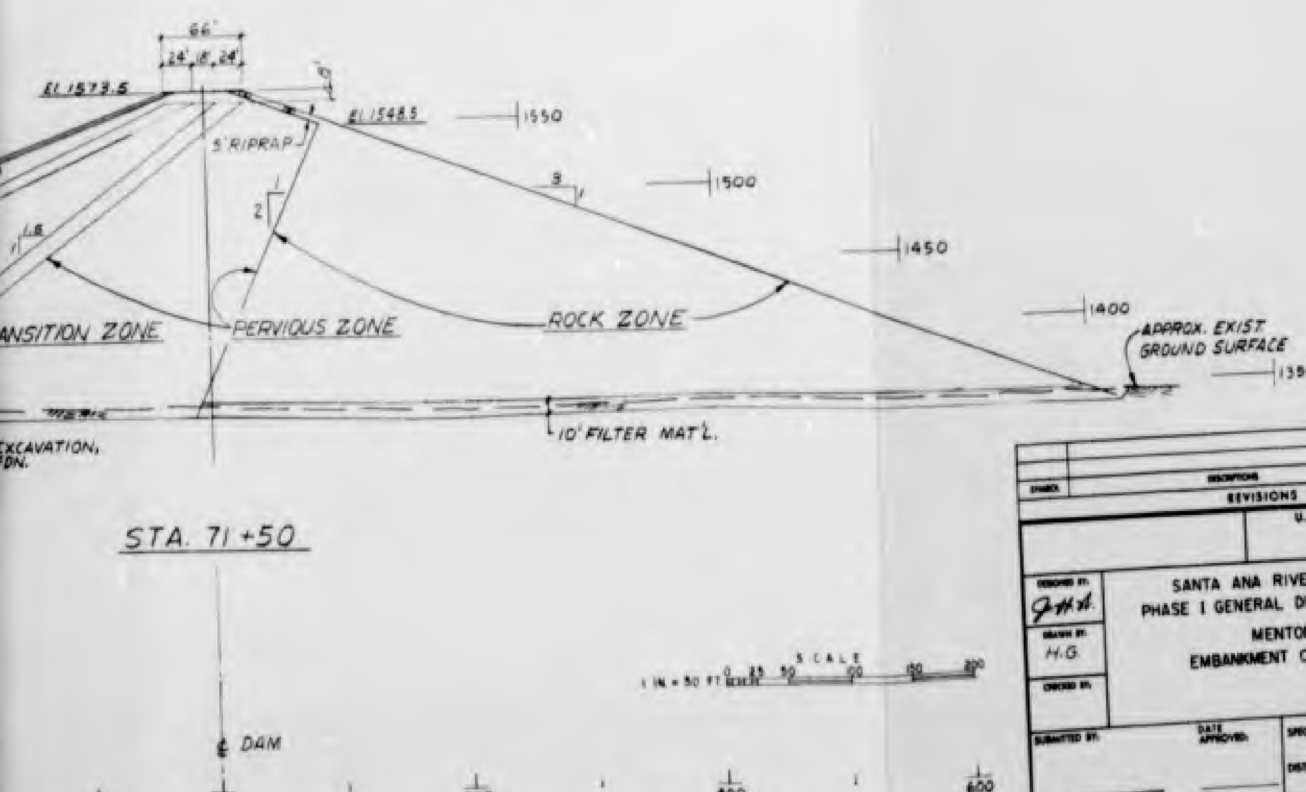
STA. 22 + 00

NOTES:

1. Cross sections are drawn looking south.
2. Dimensions for cut-off excavation are as follows:
   Between sta 40+00 & 100+00, b=30', h=40'
   Between sta. 20+00 & 33+50, b=25', h=30'



STA. 51 + 75



STA. 71 + 50

SYMBOL	DESCRIPTION	DATE	APPROVAL
REVISIONS			

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

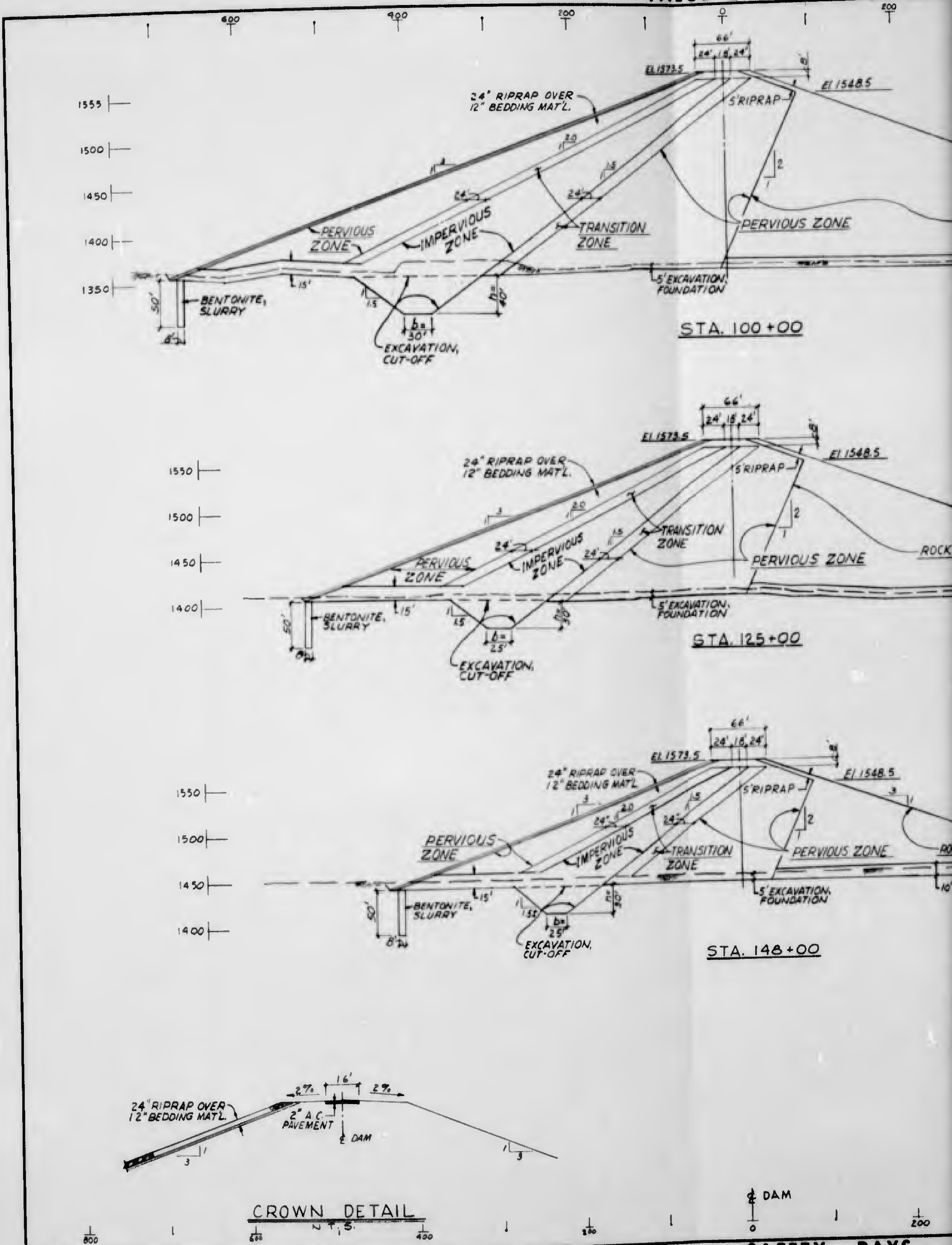
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
MENTONE DAM
EMBANKMENT CROSS SECTIONS

DESIGNED BY: J.H.A.
DRAWN BY: H.G.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 1 OF 2

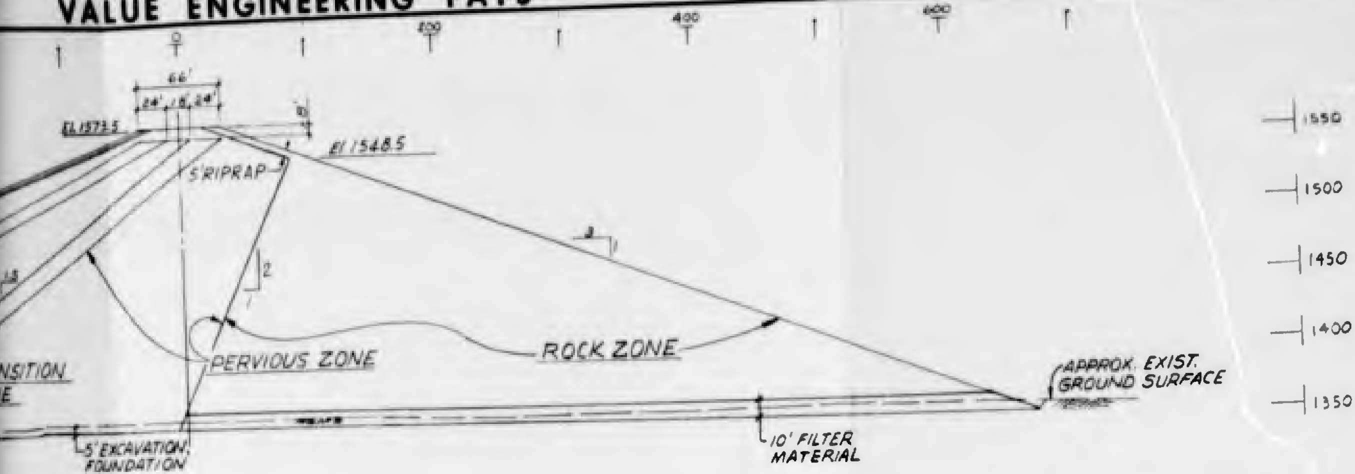
PLATE F-3

2

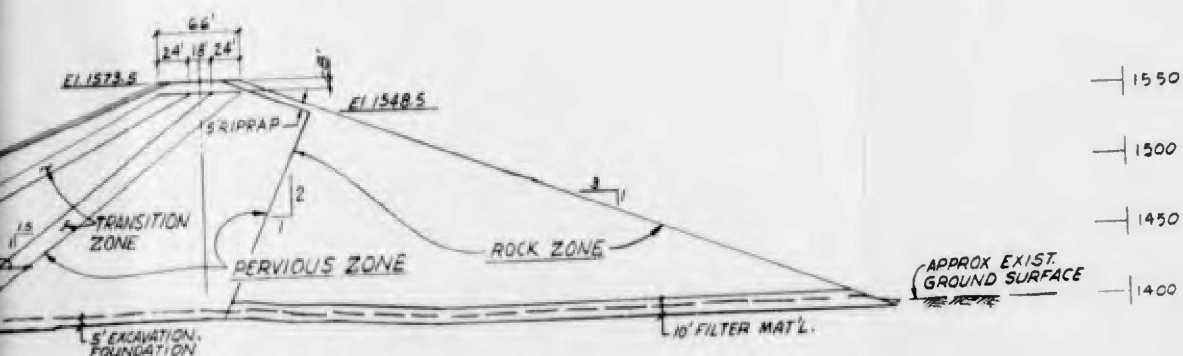
VALUE ENGINEERING PAYS



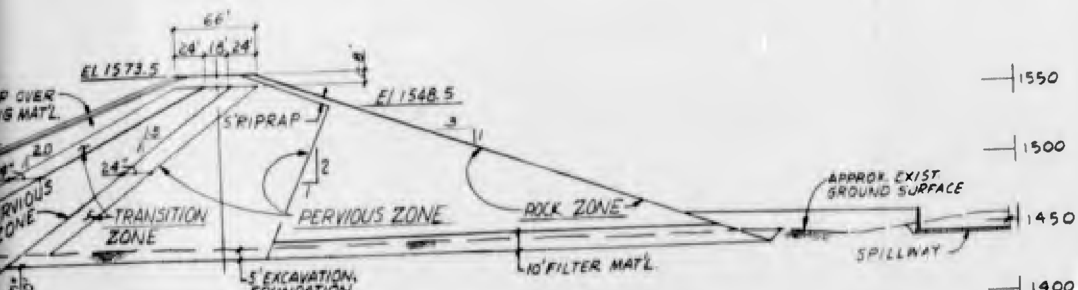
SAFETY PAYS



STA. 100+00



STA. 125+00



STA. 148+00

NOTES:

1. Cross sections are drawn looking either south or east.
2. Dimensions for cut-off excavation are as follows:
   Between Sta. 100+00 & Sta. 110+00, b=30', h=40'
   Between Sta. 109+50 & Sta. 170+00, b=25', h=30'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: J.H.A.
DRAWN BY: H.G.
CHECKED BY:

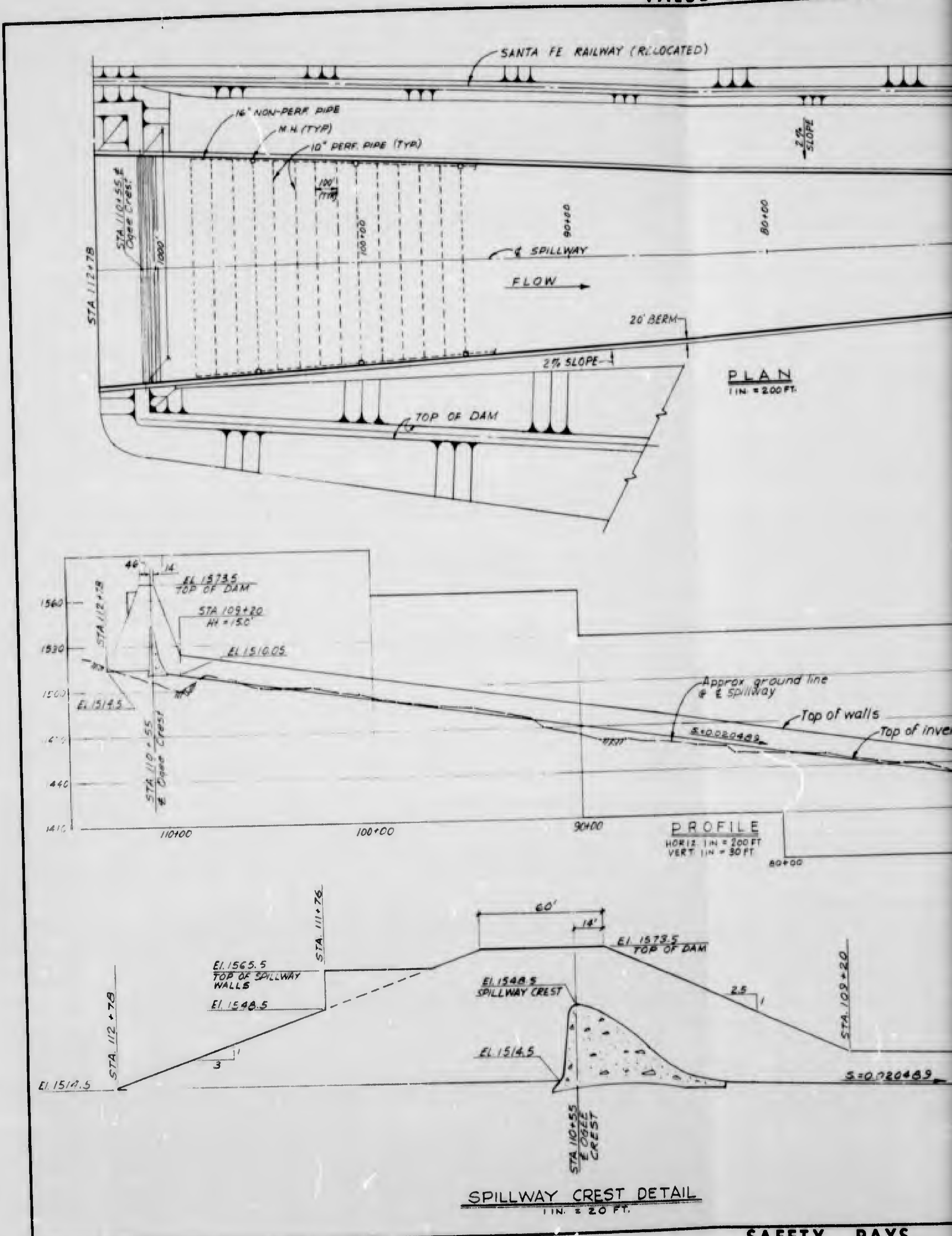
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
MENTONE DAM
EMBANKMENT CROSS SECTIONS

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

℄ DAM 0 200 400 600

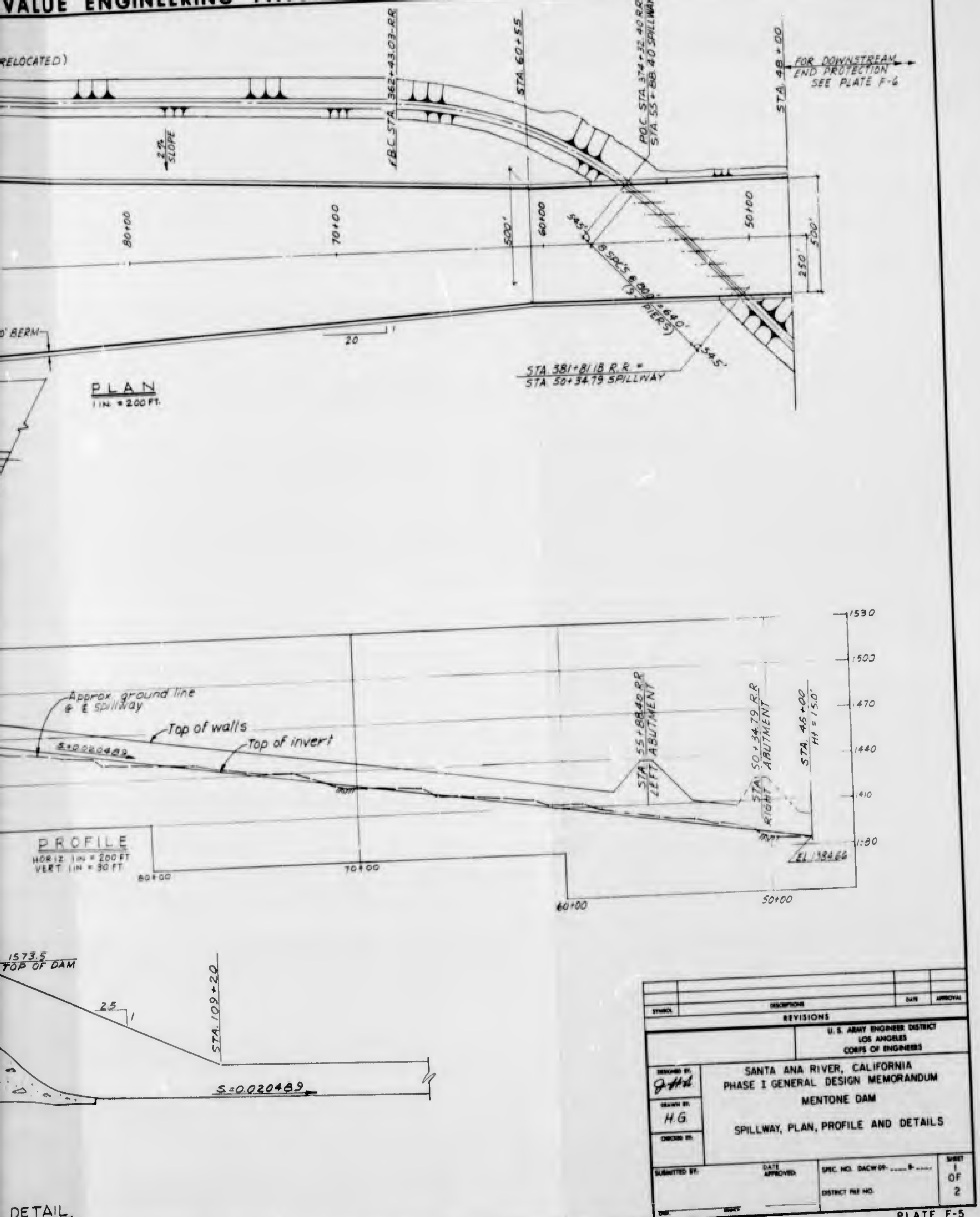
PLATE F-4

VALUE ENGINEERING PAYS

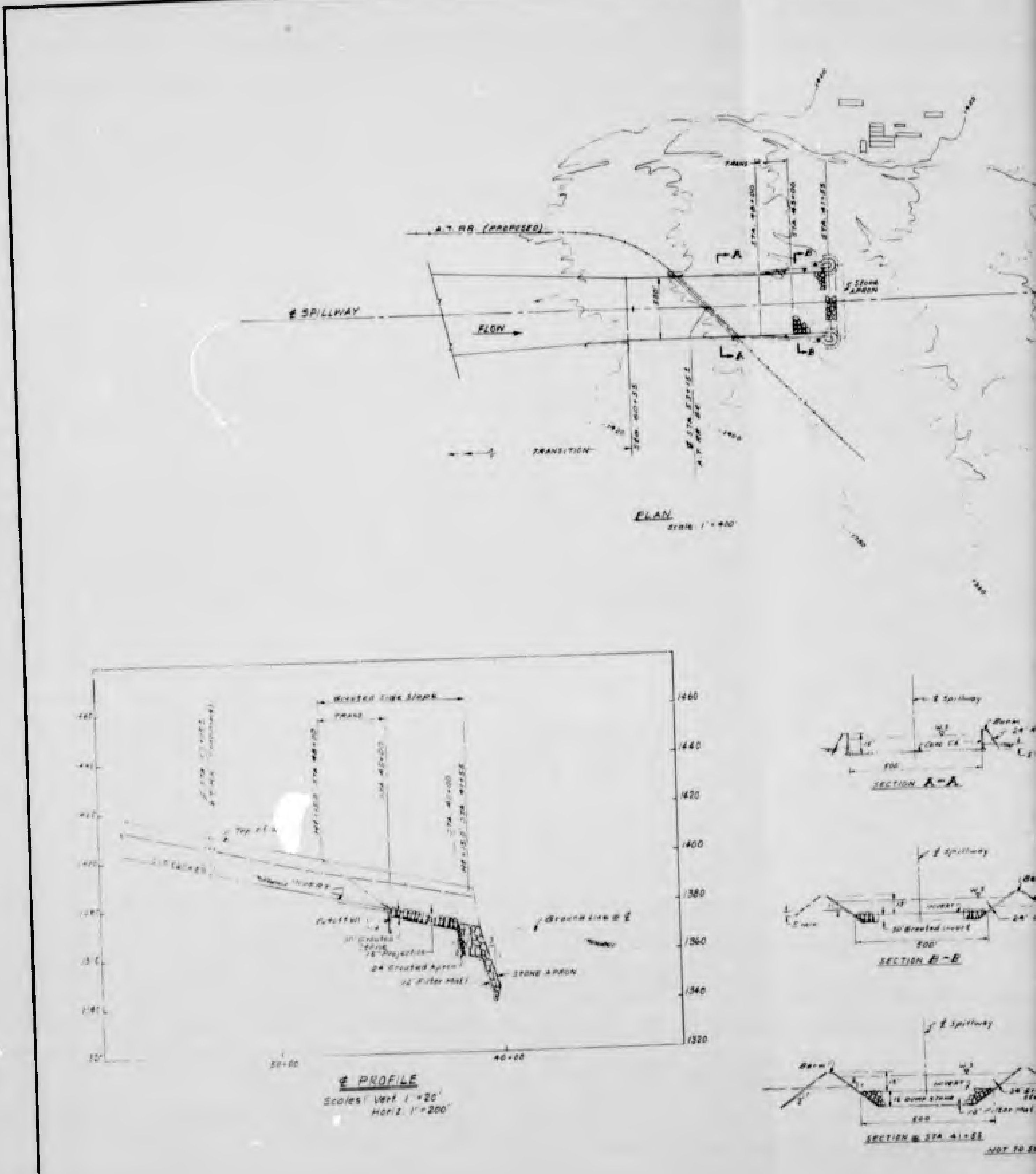


SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS



A.T. RR (PROPOSED)
¢ SPILLWAY
FLOW
TRANSITION
PLAN
Scale: 1" = 400'
Grouted Side Slope
Ground Line @ ¢
STONE APRON
24" Grouted Apron
12" Filter Mat
15" Projection
¢ PROFILE
Scales: Vert. 1" = 20'
Horiz. 1" = 200'
¢ Spillway
SECTION A-A
¢ Spillway
30' Grouted Invert
300'
SECTION B-B
¢ Spillway
Berm
SECTION @ STA. 41+55

REVISIONS

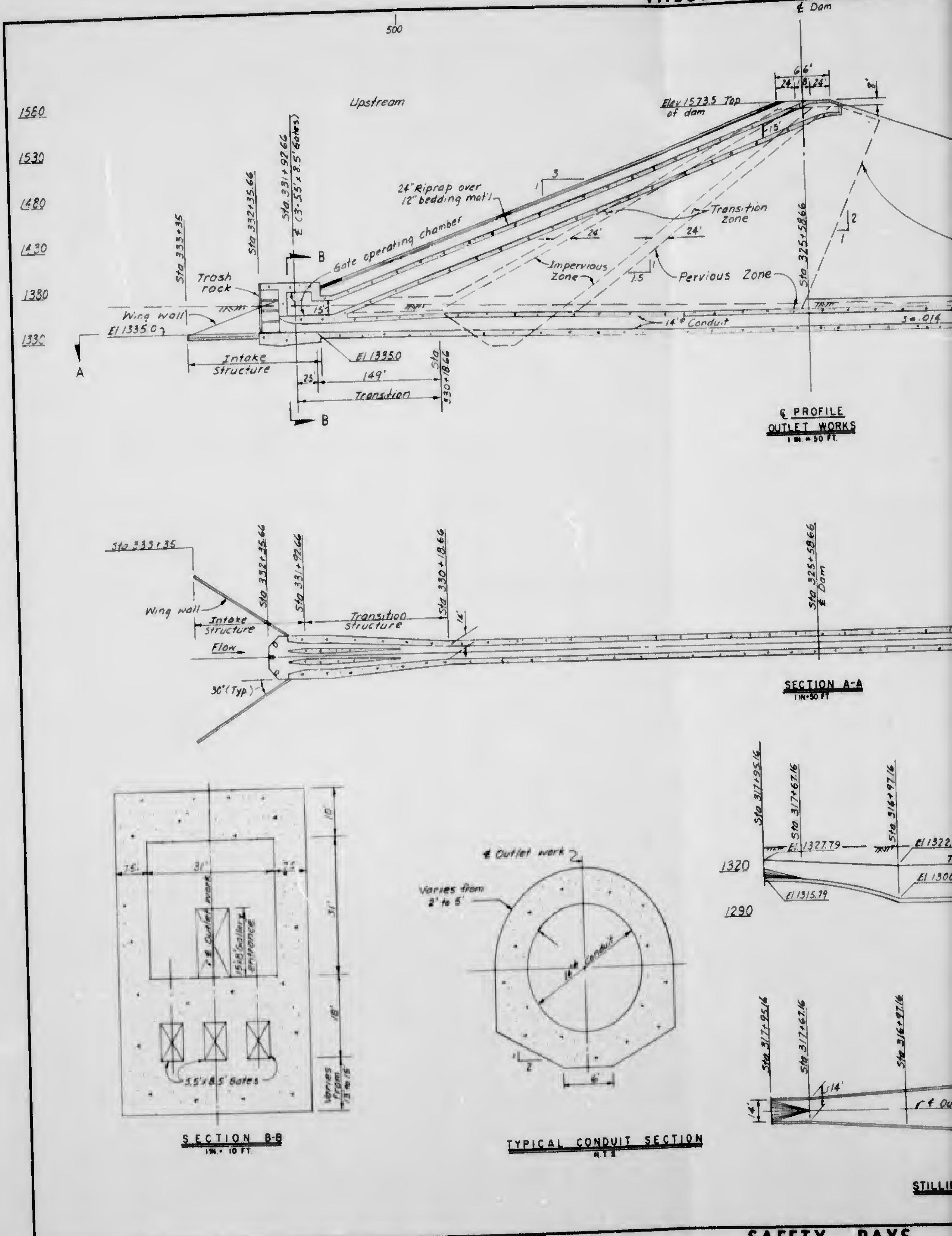
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
MENTONE DAM

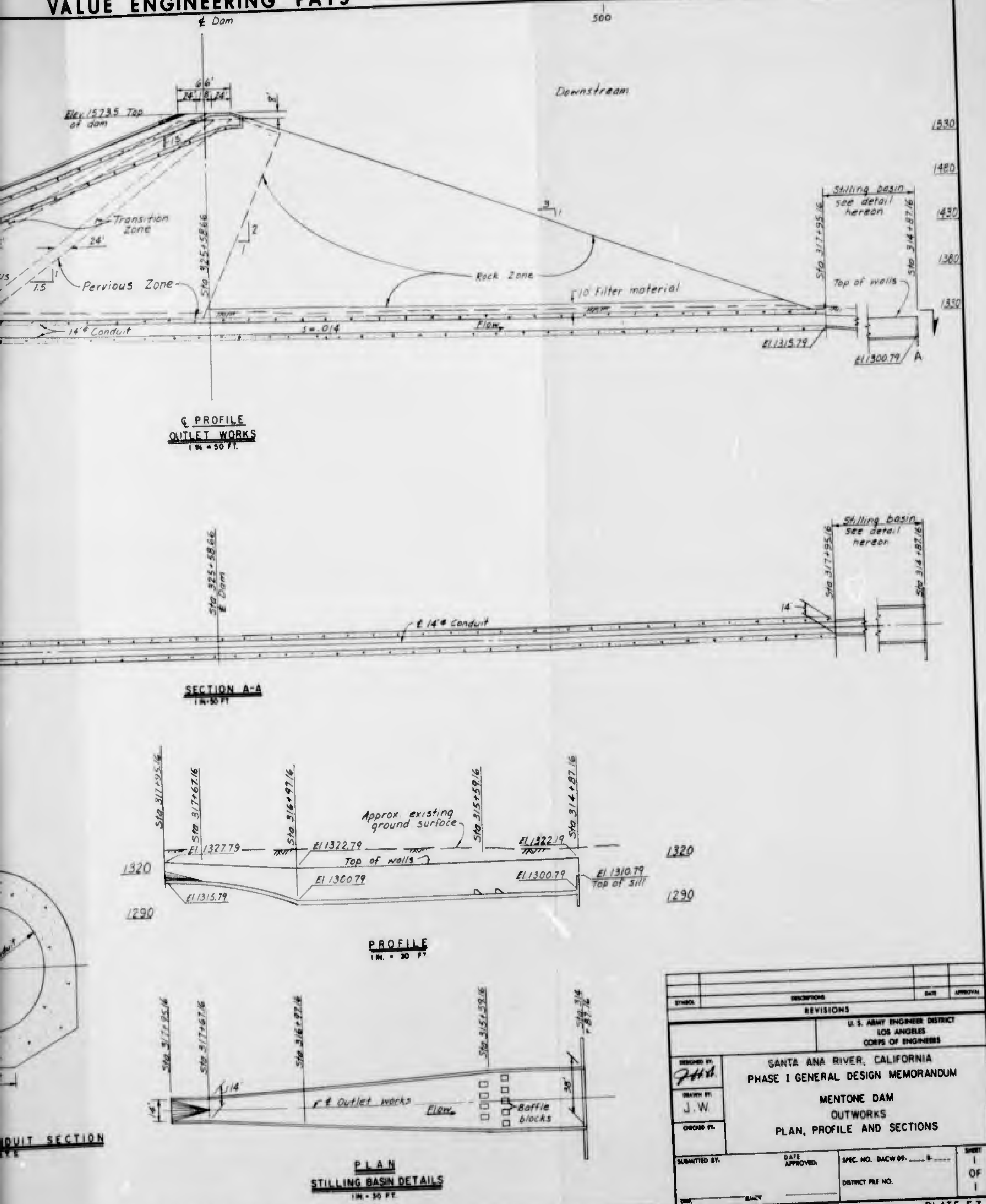
SPILLWAY, PLAN, PROFILE AND SECTIONS

SHEET 2 OF 2

PLATE F-6

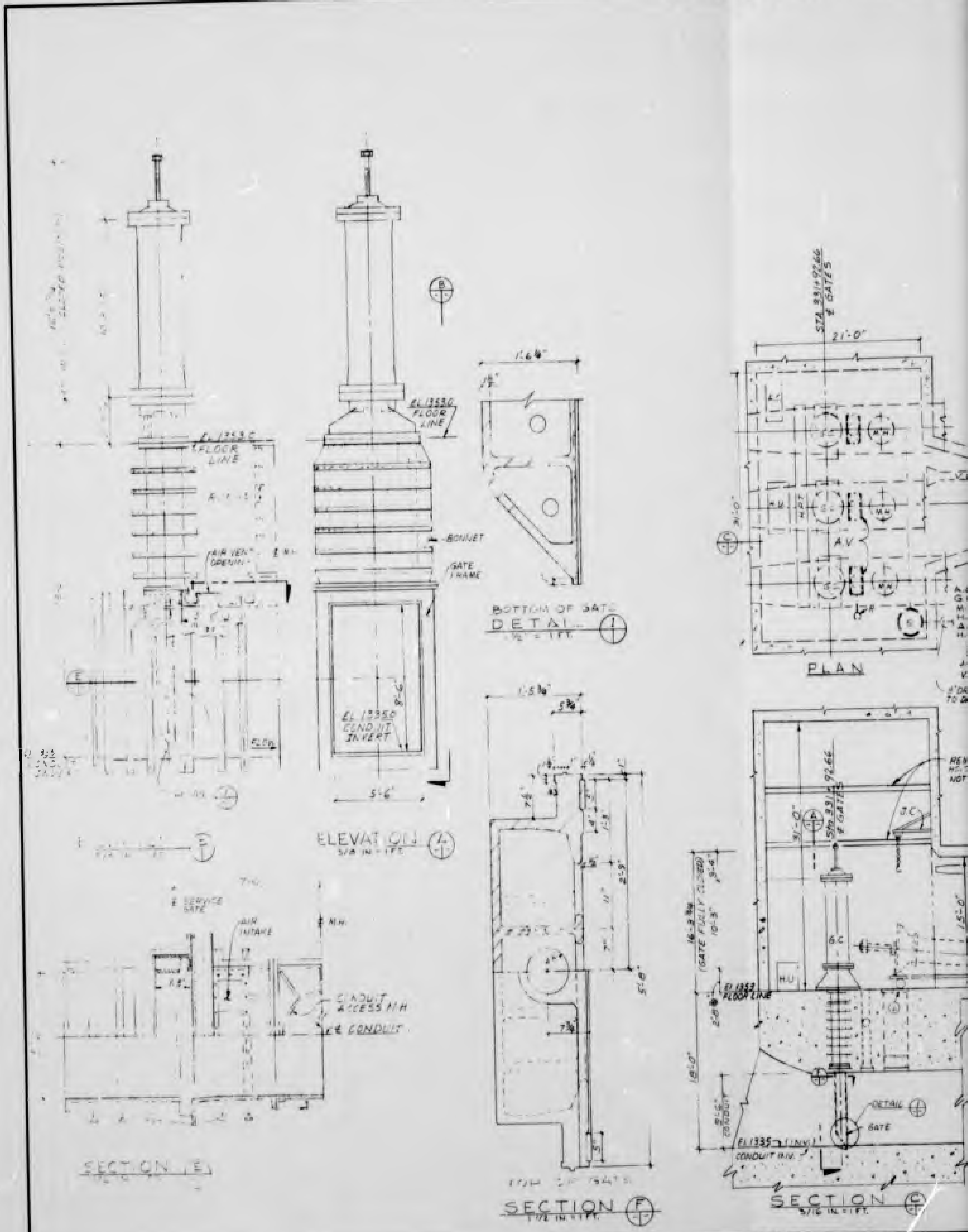

℄ Dam
500
Upstream
1560
1530
1480
1430
1380
1330
Elev 1573.5 Top of dam
24" Riprap over 12" bedding mat'l
Gate operating chamber
Transition Zone
Impervious Zone
Pervious Zone
Trash rack
Wing wall
El 1335.0
Intake Structure
El 1335.0
149'
Transition
14' ⌀ Conduit
S=.014
Sta 333+35
Sta 332+35.66
Sta 331+92.66
℄ (3-5.5' x 8.5' Gates)
Sta 330+18.66
Sta 325+58.66
℄ PROFILE
OUTLET WORKS
1 IN. = 50 FT.
Sta 333+35
Wing wall
Intake Structure
Flow
Transition structure
30° (Typ)
Sta 332+35.66
Sta 331+92.66
Sta 330+18.66
Sta 325+58.66
℄ Dam
SECTION A-A
1 IN. = 50 FT.
℄ Outlet work
(5'x8') Gallery entrance
5.5' x 8.5' Gates
Varies from 13' to 15'
SECTION B-B
1 IN. = 10 FT.
℄ Outlet work
Varies from 2' to 5'
14' ⌀ Conduit
TYPICAL CONDUIT SECTION
N.T.S.
Sta 317+95.16
Sta 317+67.16
Sta 316+97.16
El 1327.79
El 1315.79
1320
1290
STILLI

VALUE ENGINEERING PAYS



SAFETY PAYS

2


ELEVATION
5/8 IN = 1 FT.
BONNET
GATE FRAME
EL 1335.0 FLOOR LINE
EL 1335.0 CONDUIT INVERT
5'-6"
BOTTOM OF GATE
DETAIL
1'-6¼"
1'-5¾"
SECTION F
SECTION E
AIR INTAKE
CONDUIT ACCESS M.H
℄ CONDUIT
STA 331+92.66 ℄ GATES
21'-0"
31'-0"
A.V.
PLAN
J.C.
G.C.
H.U.
EL 1353 FLOOR LINE
EL 1335 (INV.) CONDUIT INV.
GATE
SECTION C
5/16 IN = 1 FT.

VALUE ENGINEERING PAYS

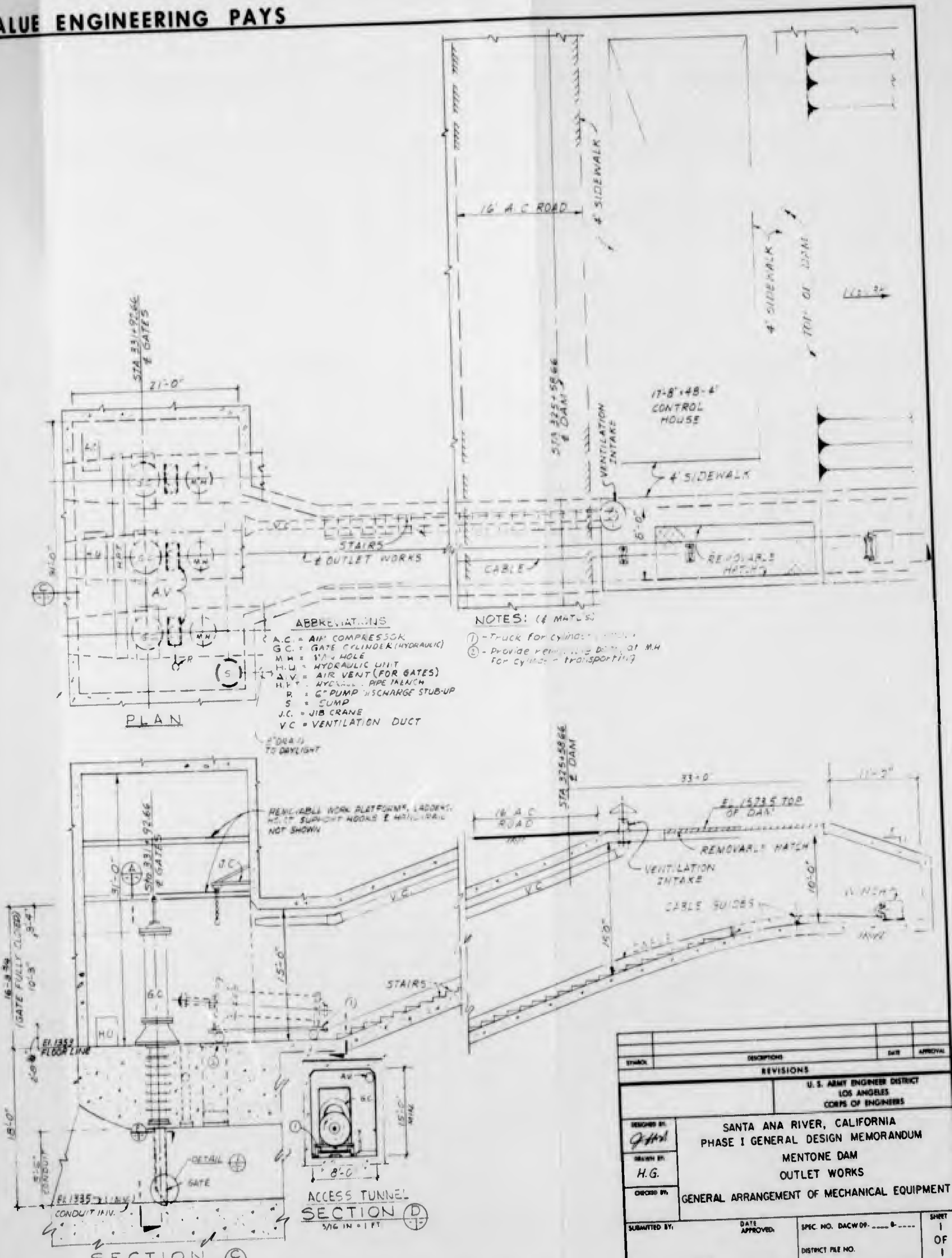
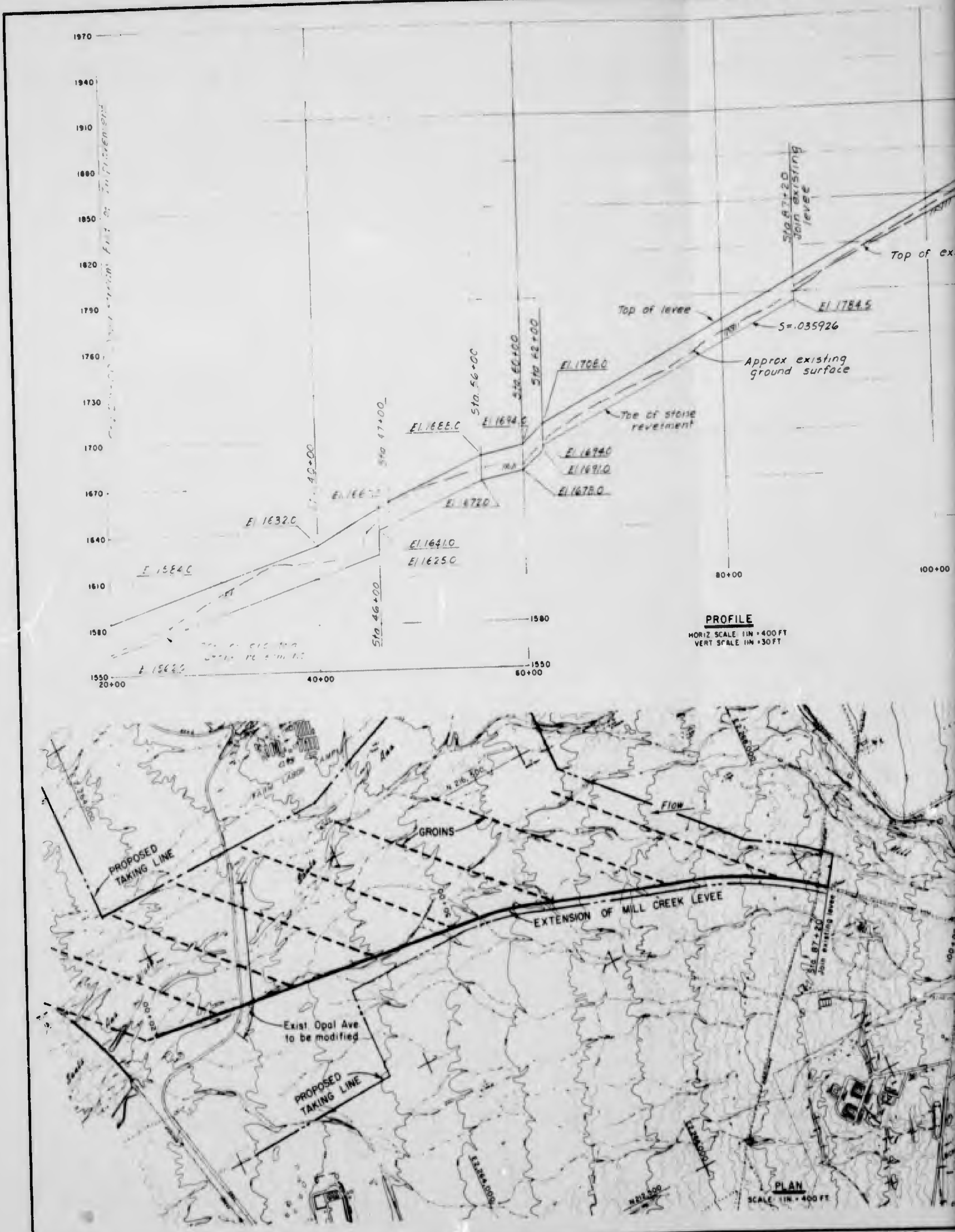


PLATE F-8

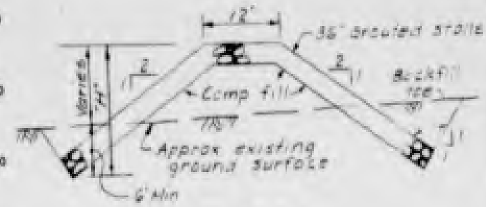
SAFETY PAYS

2



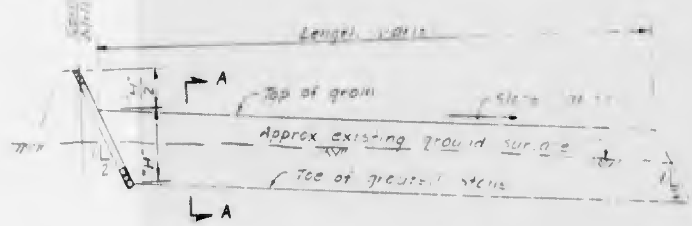
Sta 87+20 Join existing levee
Top of ex
Top of levee
El 1784.5
S=.035926
Approx existing ground surface
Toe of stone revetment
El 1706.0
El 1694.0
El 1691.0
El 1678.0
El 1666.0
El 1672.0
El 1632.0
El 1641.0
El 1625.0
Sta 56+00
Sta 47+00
Sta 46+00
80+00
100+00
40+00
60+00
20+00
PROFILE
HORIZ SCALE 1IN = 400 FT
VERT SCALE 1IN = 30 FT
GROINS
Flow
PROPOSED TAKING LINE
EXTENSION OF MILL CREEK LEVEE
Exist Opal Ave to be modified
PLAN
SCALE 1IN = 400 FT

PROFILE
HORIZ SCALE 1IN = 400 FT
VERT SCALE 1IN = 30 FT

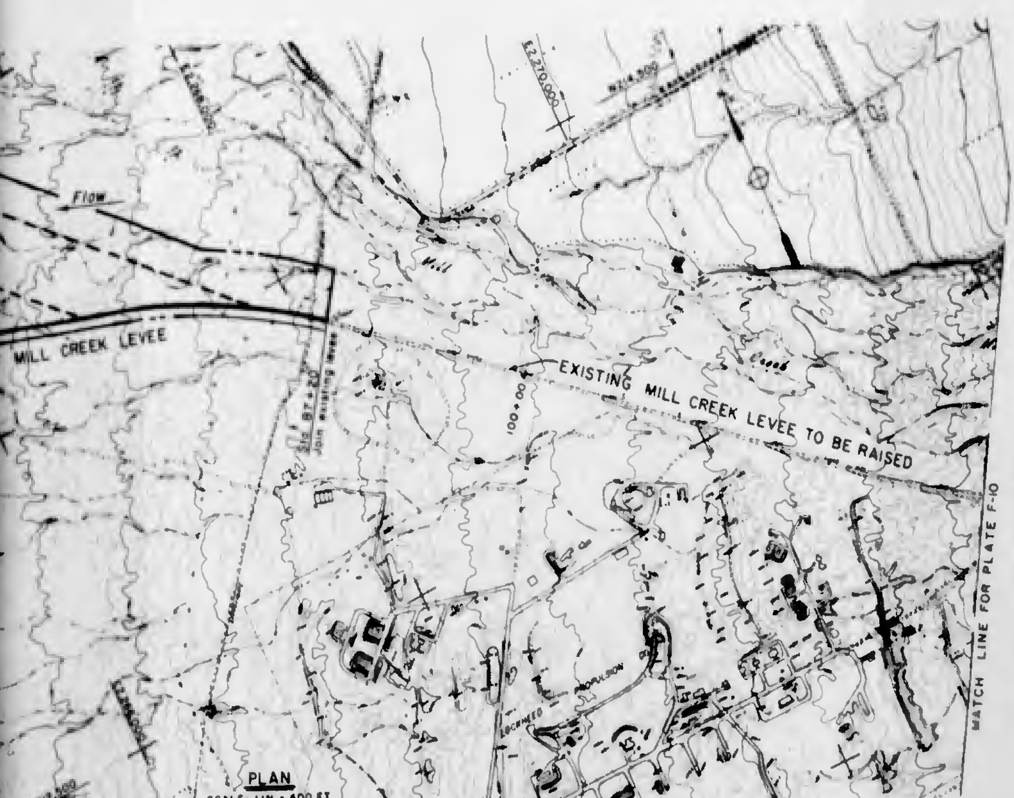


SECTION A-A
NOT TO SCALE

GROIN LOCATION	LENGTH	"H"	SLOPE
Sta 19+00	1600'	19'	.0176
Sta 26+00	1,700'	15'	.0180
Sta 34+00	1860'	19'	.0197
Sta 42+00	2200'	17'	.0182
Sta 50+00	2200'	21'	[illegible]
Sta 60+00	2200'	15'	[illegible]
Sta 71+30	2430'	15'	.0359
Sta 96+25	2350'	8'	[illegible]



TYPICAL PROFILE OF GROINS
NOT TO SCALE



PLAN
SCALE 1IN = 400 FT.

Note: For typical levee section see Plate F-10

HORIZ. SCALE 400 0 400 800 1200 FEET
VERT SCALE 30 0 30 60 FEET

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

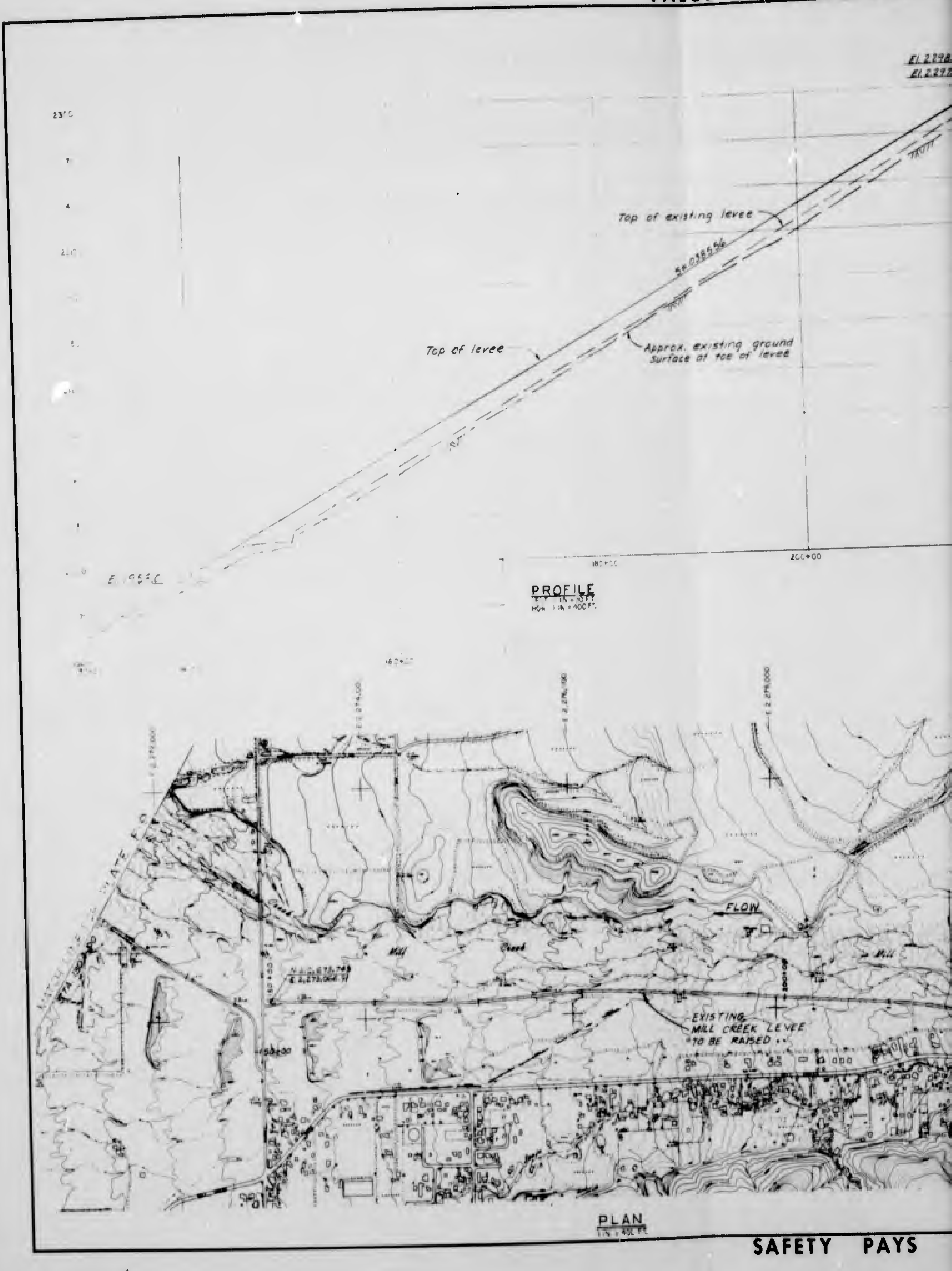
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

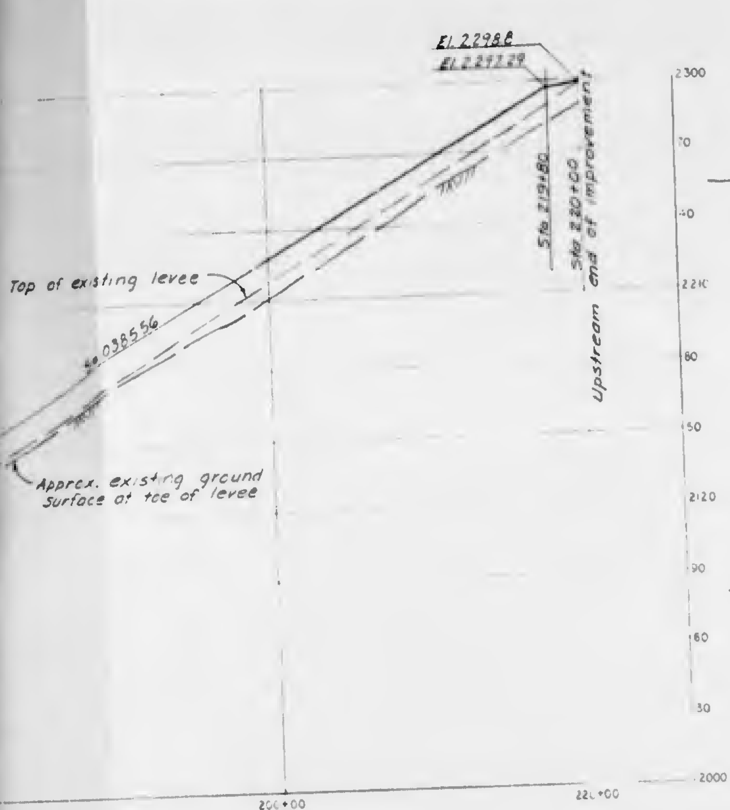
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
MENTONE DAM
MODIFICATION OF MILL CREEK LEVEE
PLAN, PROFILE, AND SECTION

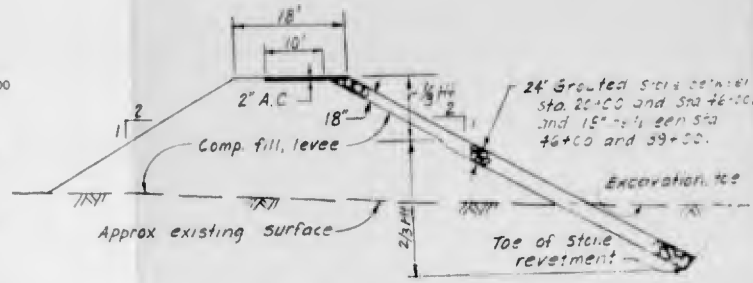
SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 1 OF 2 SHEETS

2

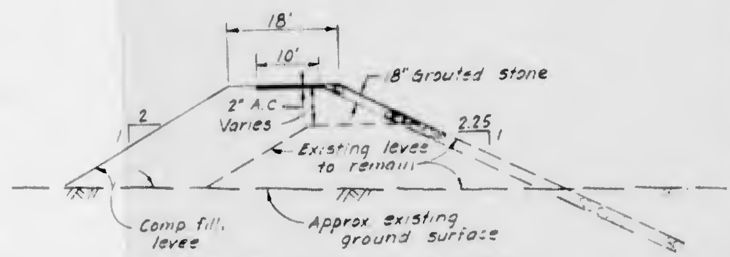

Top of existing levee
Top of levee
Approx. existing ground
Surface at toe of levee
200+00
PROFILE
FLOW
EXISTING
MILL CREEK LEVEE
TO BE RAISED
PLAN

VALUE ENGINEERING PAYS





STA. 20+00 TO STA. 89+00



STA. 89+00 TO STA. 220+00

TYPICAL SECTIONS

REVISIONS

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

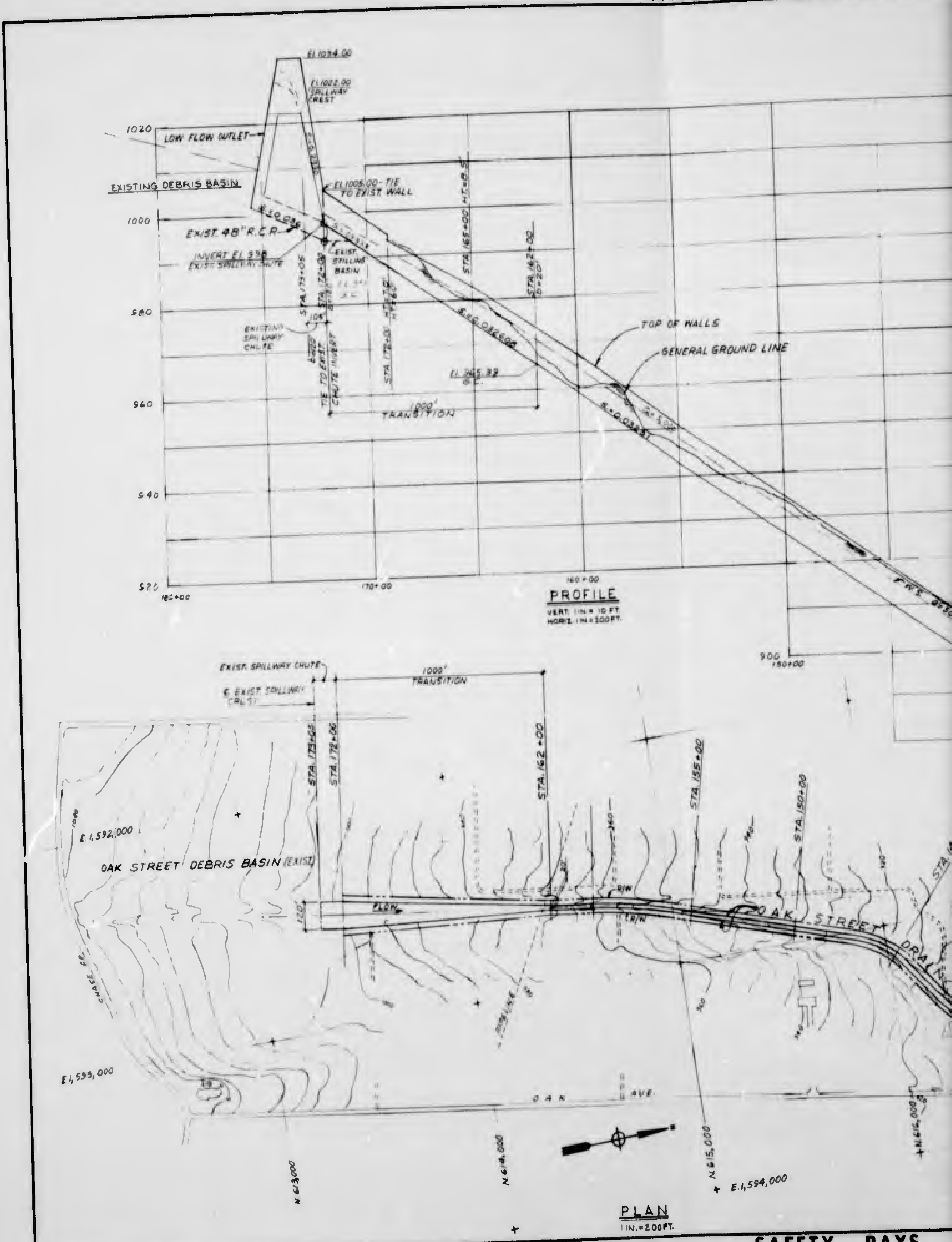
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
MENTONE DAM
MODIFICATION OF MILL CREEK LEVEE
PLAN, PROFILE AND SECTIONS

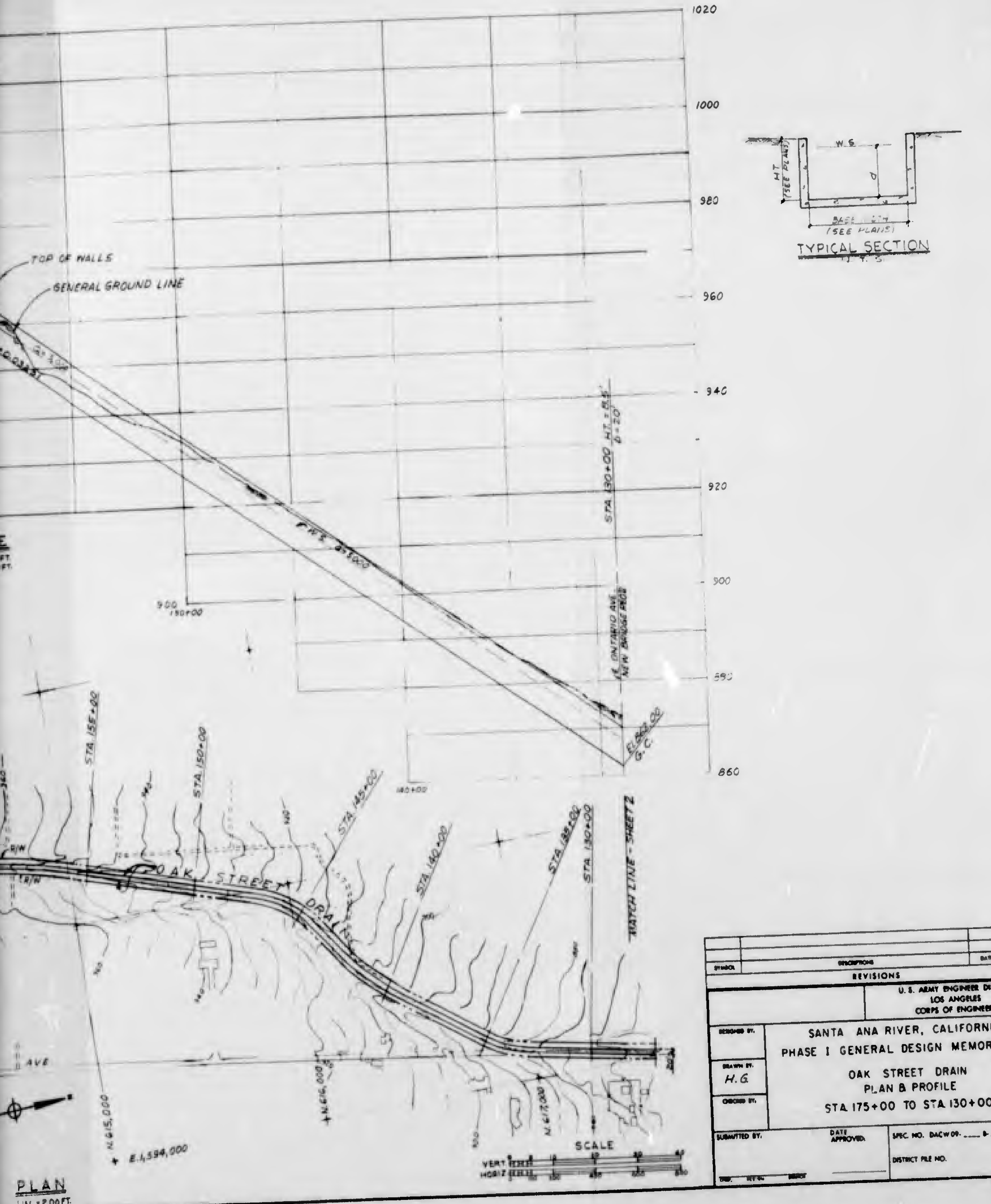
DESIGNED BY: JHH
DRAWN BY: H.G
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 2 OF 2

PLATE F-10

PLAN

SAFETY PAYS


El 1034.00
El 1022.00 SPILLWAY CREST
LOW FLOW OUTLET
EXISTING DEBRIS BASIN
El 1005.00 - TIE TO EXIST. WALL
EXIST. 48" R.C.P.
INVERT El 998
EXIST. SPILLWAY CHUTE
EXIST. STILLING BASIN
EXISTING SPILLWAY CHUTE
TIE TO EXIST. CHUTE INVERT
STA. 165+00
STA. 162+00
TOP OF WALLS
GENERAL GROUND LINE
1000' TRANSITION
PROFILE
VERT. 1 IN. = 10 FT.
HORIZ. 1 IN. = 100 FT.
EXIST. SPILLWAY CHUTE
℄ EXIST. SPILLWAY CREST
1000' TRANSITION
STA. 173+05
STA. 172+00
STA. 162+00
STA. 155+00
STA. 150+00
E 1,592,000
OAK STREET DEBRIS BASIN (EXIST)
FLOW
R/W
OAK STREET DRAIN
CHASE DR.
E 1,593,000
OAK AVE.
N 613,000
N 614,000
N 615,000
N 616,000
E 1,594,000
PLAN
1 IN. = 200 FT.



REVISIONS

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

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

DESIGNED BY:
DRAWN BY: H.G.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
OAK STREET DRAIN
PLAN & PROFILE
STA 175+00 TO STA 130+00

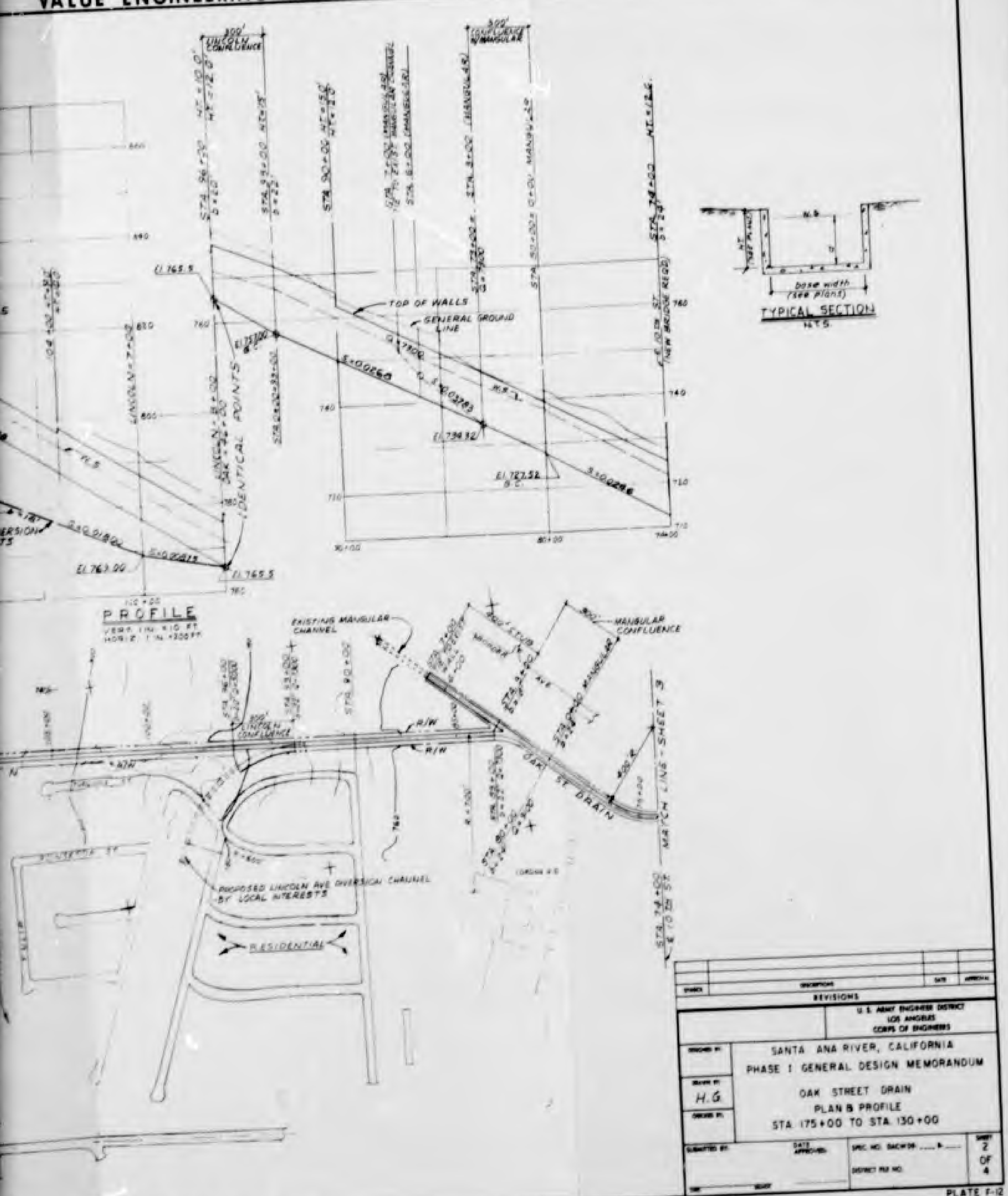
SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 1 OF 4

PLATE F-11

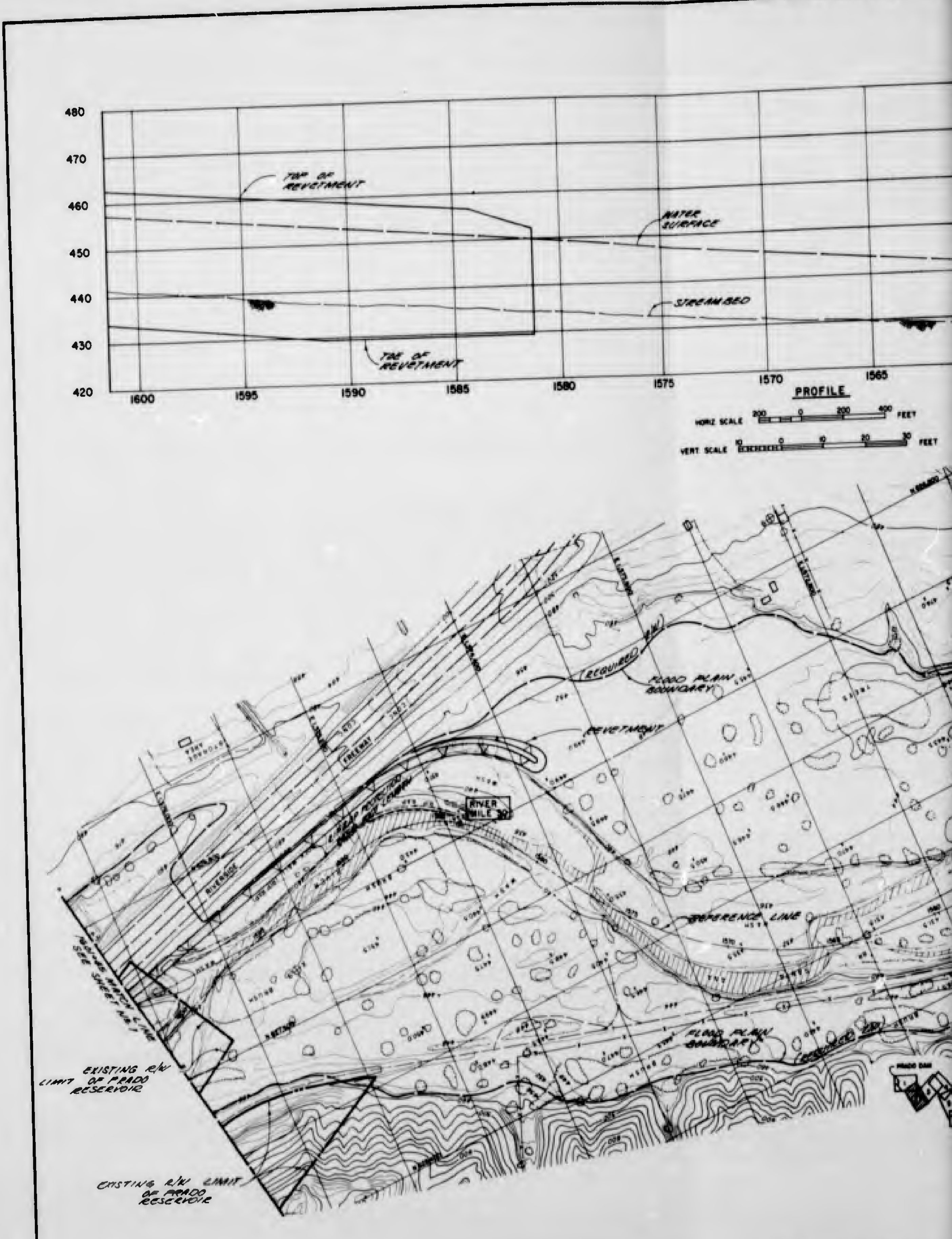
2


TOP OF WALLS
GENERAL GROUND LINE
TOP OF WALLS
LINCOLN CONFLUENCE
IDENTICAL POINTS
PROPOSED LINCOLN AVE. DIVERSION CHANNEL - BY LOCAL INTERESTS
El 788.00
El 765.00
El 765.5
PROFILE
VERT. 1 IN. = 10 FT.
HORIZ. 1 IN. = 200 FT.
EXISTING MANSU
CHANNEL
STA. 90+00
R/W
DRAIN
POINSETTIA ST.
RESIDENTIAL
PROPOSED LINCOLN AVE DIVERSI
-BY LOCAL INTERESTS
RESIDENTIAL
N.
℄ LINCOLN AVE
PLAN
1 IN. = 200 FT.

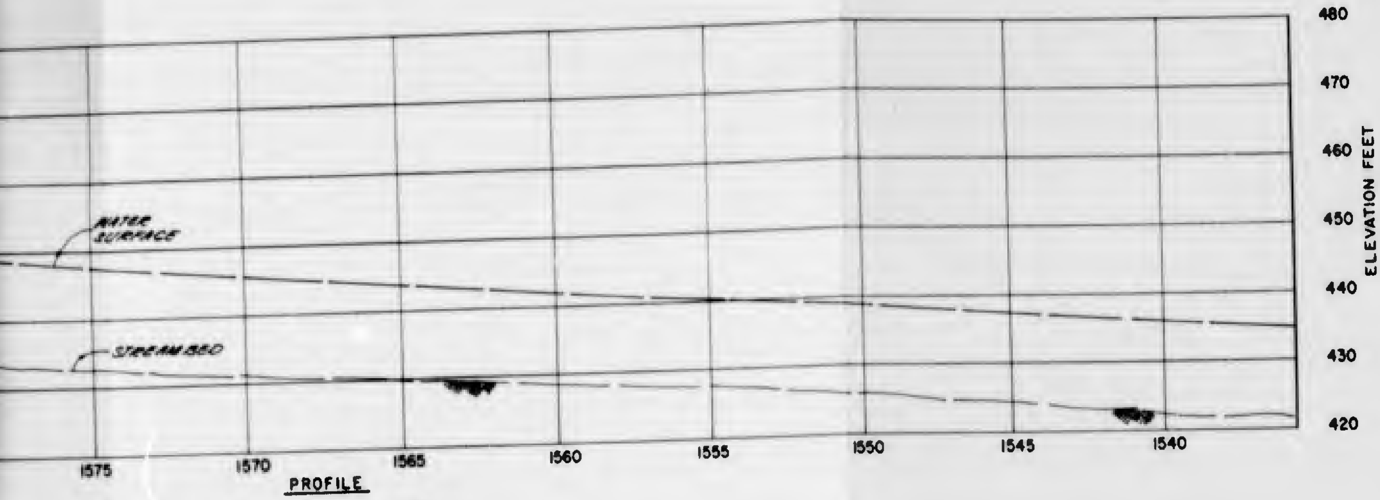
VALUE ENGINEERING PAYS

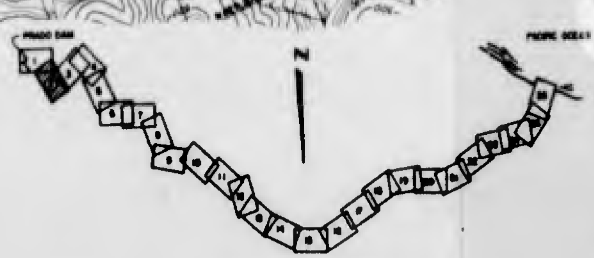


SAFETY PAYS


480
470
460
450
440
430
420
TOP OF REVETMENT
WATER SURFACE
STREAMBED
TOE OF REVETMENT
1600
1595
1590
1585
1580
1575
1570
1565
PROFILE
HORIZ SCALE
200 0 200 400 FEET
VERT SCALE
10 0 10 20 30 FEET
REQUIRED R/W
FLOOD PLAIN BOUNDARY
REVETMENT
RIVER MILE 30
REFERENCE LINE
FLOOD PLAIN BOUNDARY
REQUIRED R/W
FREEWAY
RIVERSIDE
EXISTING R/W LIMIT OF PRADO RESERVOIR
EXISTING R/W LIMIT OF PRADO RESERVOIR

VALUE ENGINEERING PAYS





INDEX TO SHEETS

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD
ANAHEIM, CALIF. 92805
(714) 774-8740

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

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:

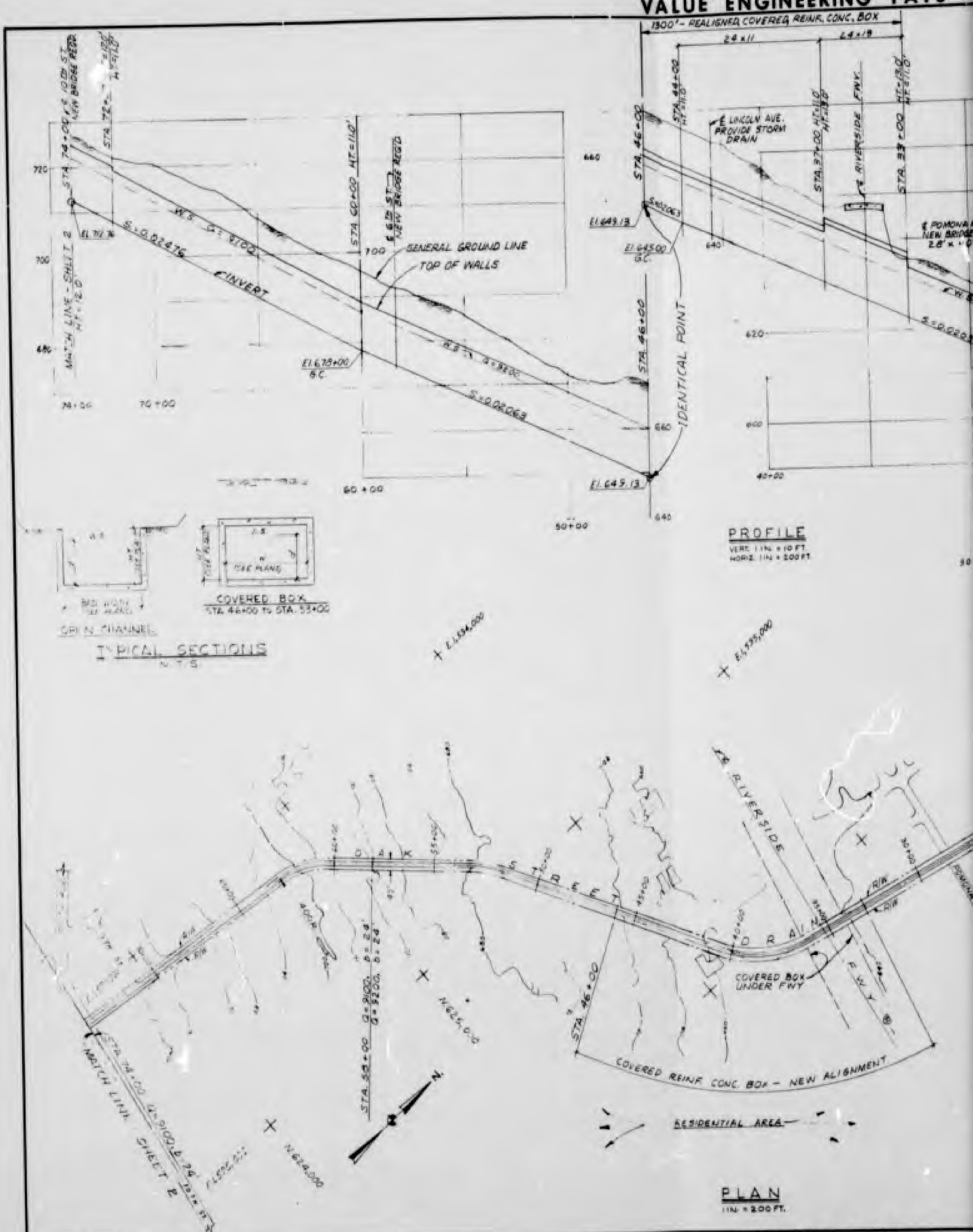
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTA ANA RIVER CHANNEL
PLAN AND PROFILE
STA. 1601+45 TO STA. 1535+75

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW 09-___ B-___
DISTRICT FILE NO.
SHEET 2 OF 29 SHEETS

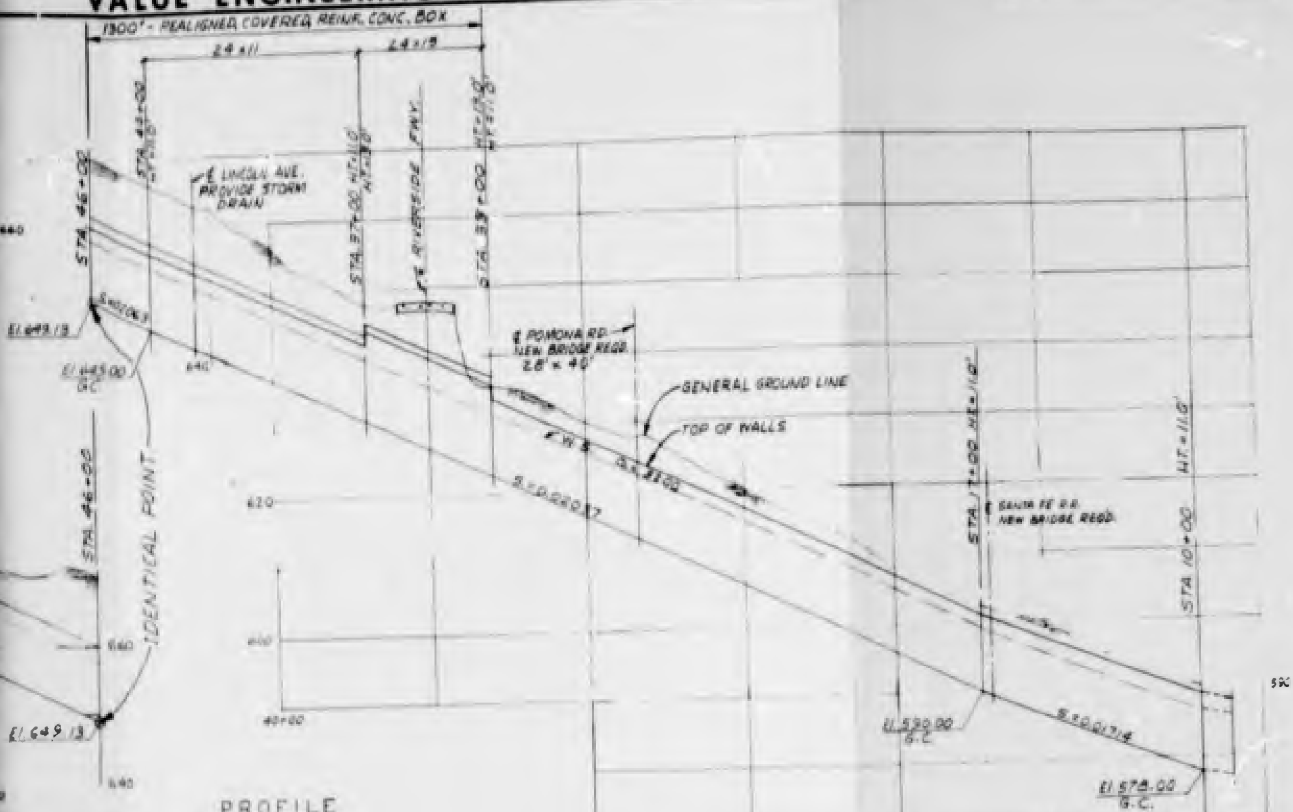
PLATE F-30

SAFETY PAYS

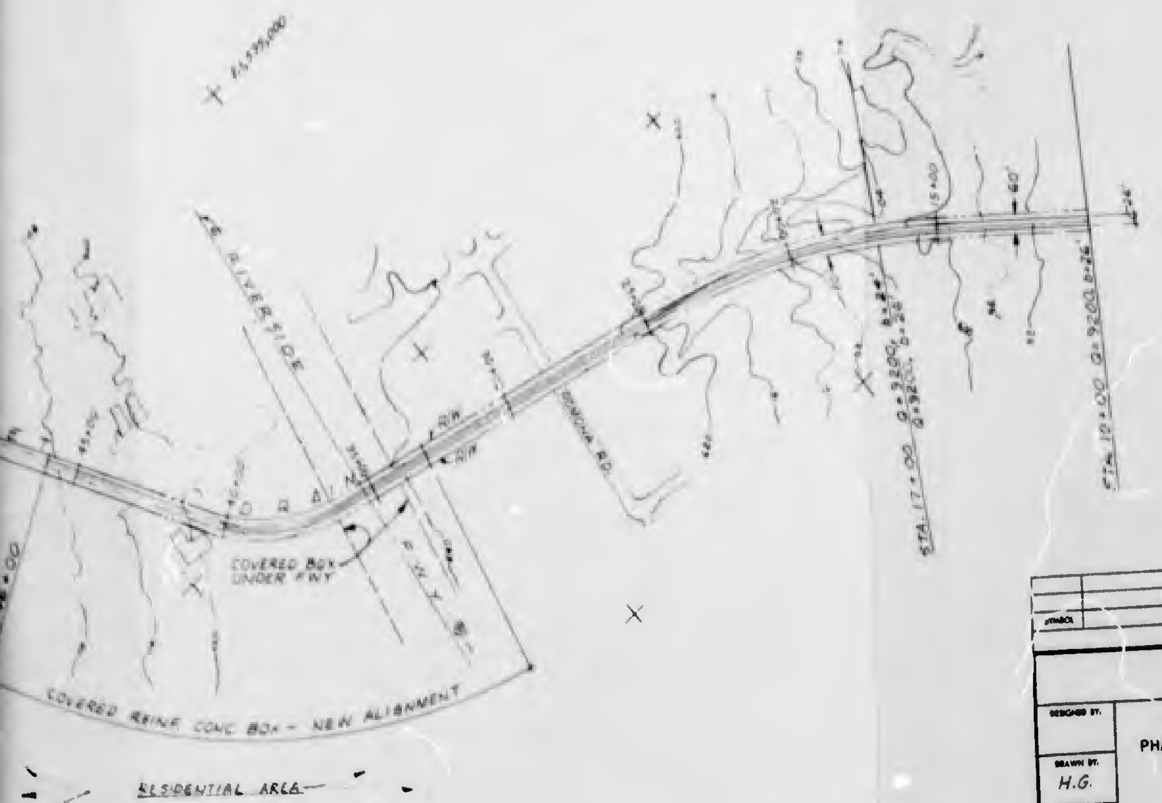
VALUE ENGINEERING PAYS



SAFETY PAYS



PROFILE
VERT. 1 IN. = 10 FT.
HORIZ. 1 IN. = 200 FT.



PLAN
1 IN. = 200 FT.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY:
DRAWN BY: H.G.
CHECKED BY:

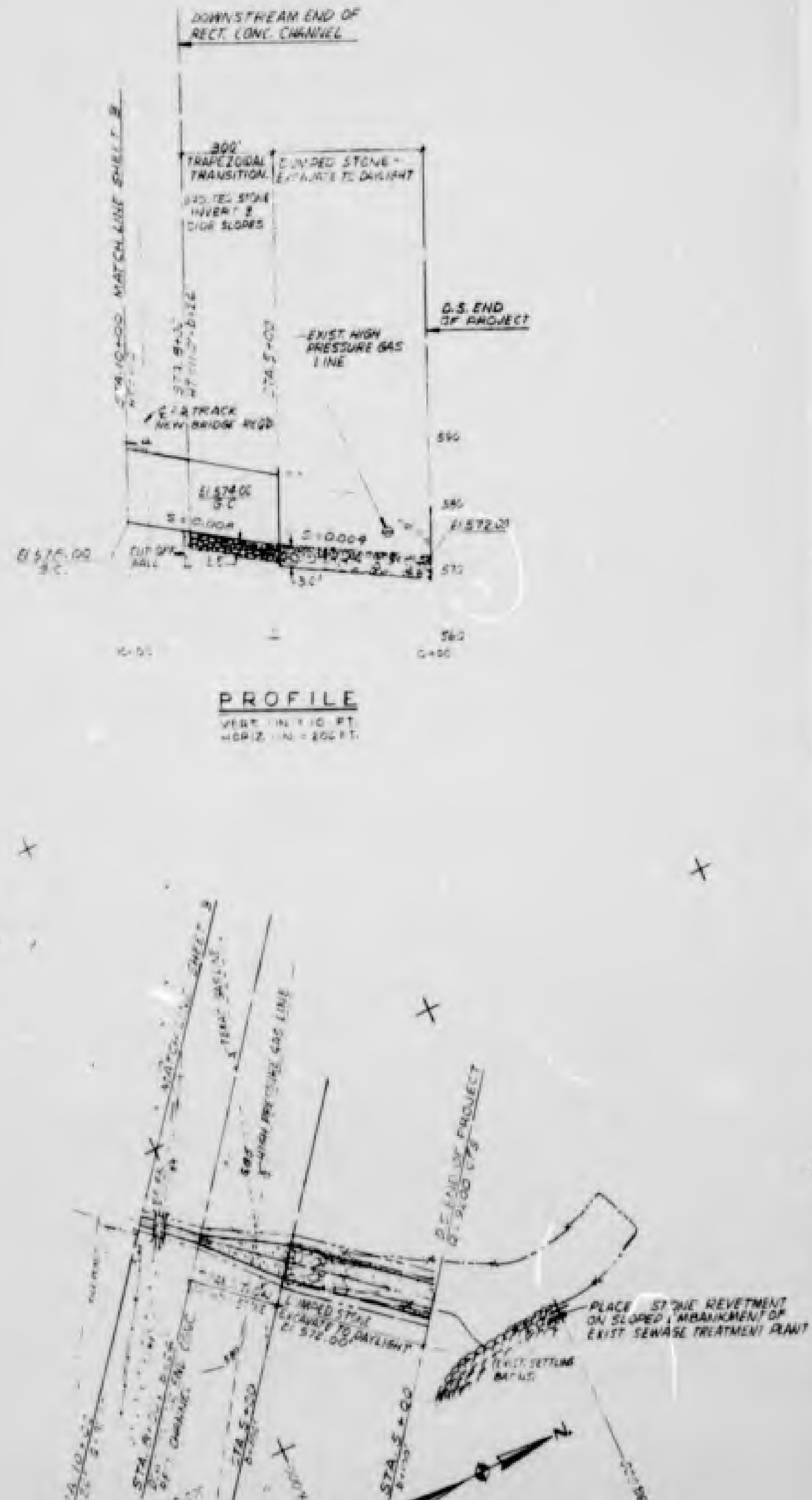
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

OAK STREET DRAIN
PLAN & PROFILE
STA. 74+00 TO STA. 10+00

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
SHEET 3 OF 4

PLATE F-13

2



PLAN

1 IN. = 200 FT.

VALUE ENGINEERING PAYS

REVISIONS

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

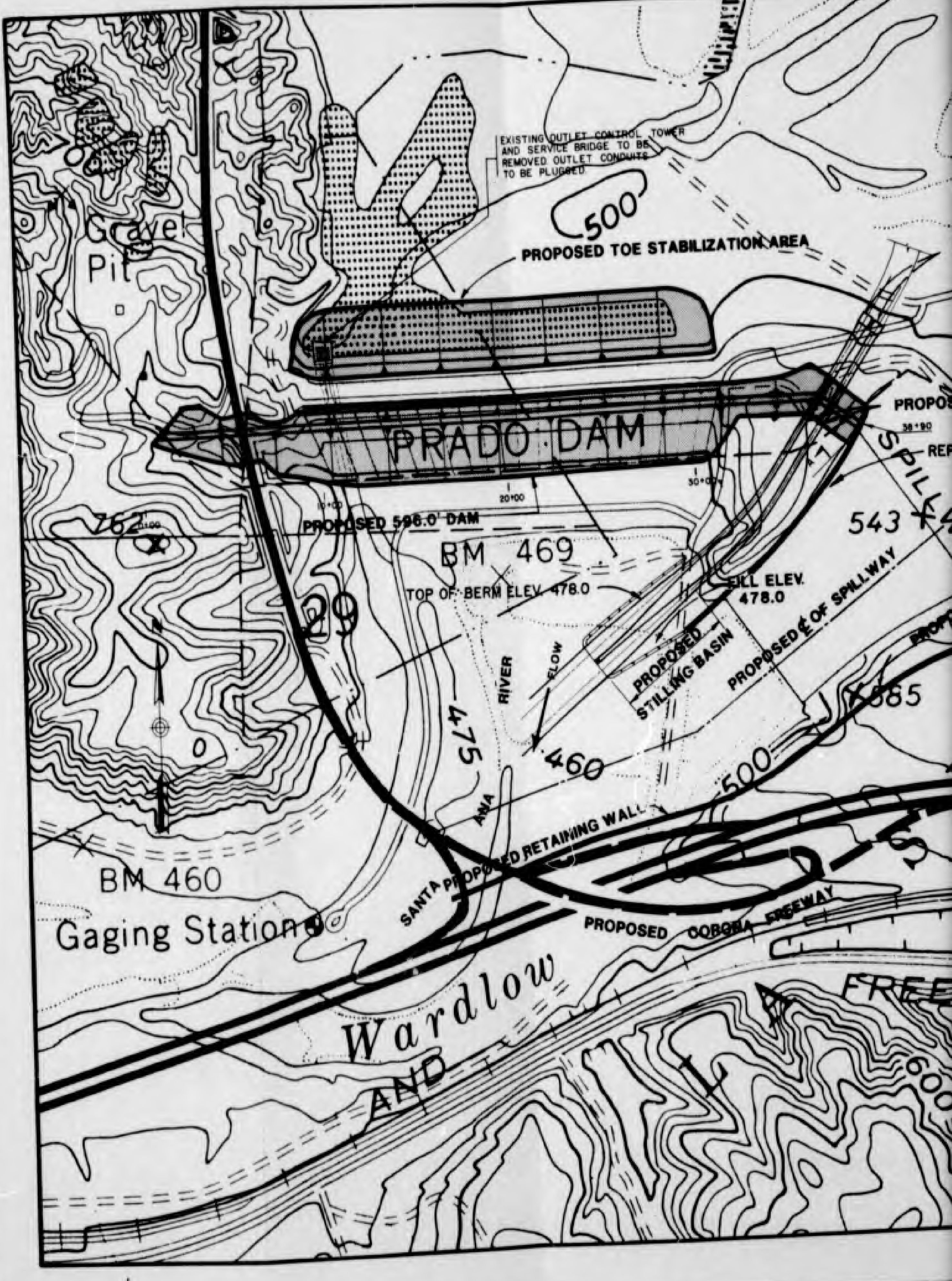
OAK STREET DRAIN
PLAN & PROFILE
STA. 10+00 TO STA. 0+00

DRAWN BY: H.G.

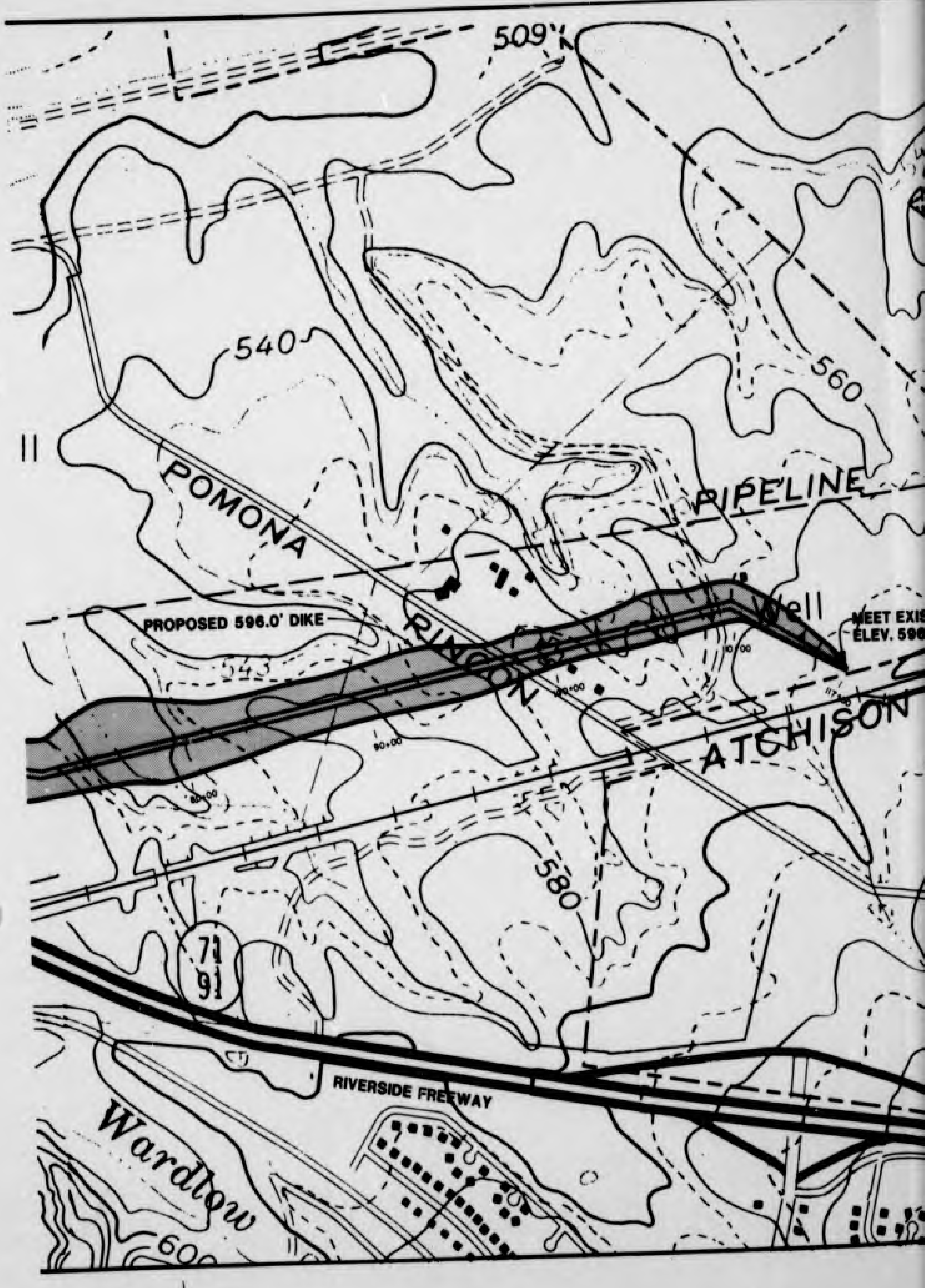
SHEET 4 OF 4

PLATE F-14

SAFETY PAYS


EXISTING OUTLET CONTROL TOWER AND SERVICE BRIDGE TO BE REMOVED OUTLET CONDUITS TO BE PLUGGED
500
PROPOSED TOE STABILIZATION AREA
Gravel Pit
PRADO DAM
PROPOS
SPILL
REF
10+00
20+00
30+00
38+90
762
PROPOSED 596.0' DAM
BM 469
543
TOP OF BERM ELEV. 478.0
FILL ELEV. 478.0
29
RIVER
FLOW
PROPOSED STILLING BASIN
PROPOSED ℄ OF SPILLWAY
585
475
460
500
SANTA ANA
PROPOSED RETAINING WALL
BM 460
Gaging Station
PROPOSED CORONA FREEWAY
Wardlow
AND
FREE
600


EXISTING OUTLET CONTROL TOWER AND SERVICE BRIDGE TO BE REMOVED OUTLET CONDUITS TO BE PLUGGED
500
PROPOSED TOE STABILIZATION AREA
PROPOSED INTAKE TOWERS
500
PROPOSED 25' DIA. CONDUITS
We
ADO DAM
38+90
REPLACE RIGHT SPILLWAY WALL
SPILLWAY
20+00
30+00
DAM
543
PROPOSED RELOCATED SPILLWAY WALL
BM 469
OF BERM ELEV. 478.0
SILL ELEV. 478.0
PROPOSED ℄ OF SPILLWAY
51+60
PROPOSED STILLING BASIN
FLOW
RIVER
475
585
460
500
60+00
91
71
70+00
PROPOSED RETAINING WALL
ANA
SANTA FE
Wash
PROPOSED CORONA FREEWAY
FREEWAY LANE
dlow
600



509'
540
560
POMONA
PIPELINE
PROPOSED 596.0' DIKE
RINCON
Well
ATCHISON
580
71
91
RIVERSIDE FREEWAY
Wardlow

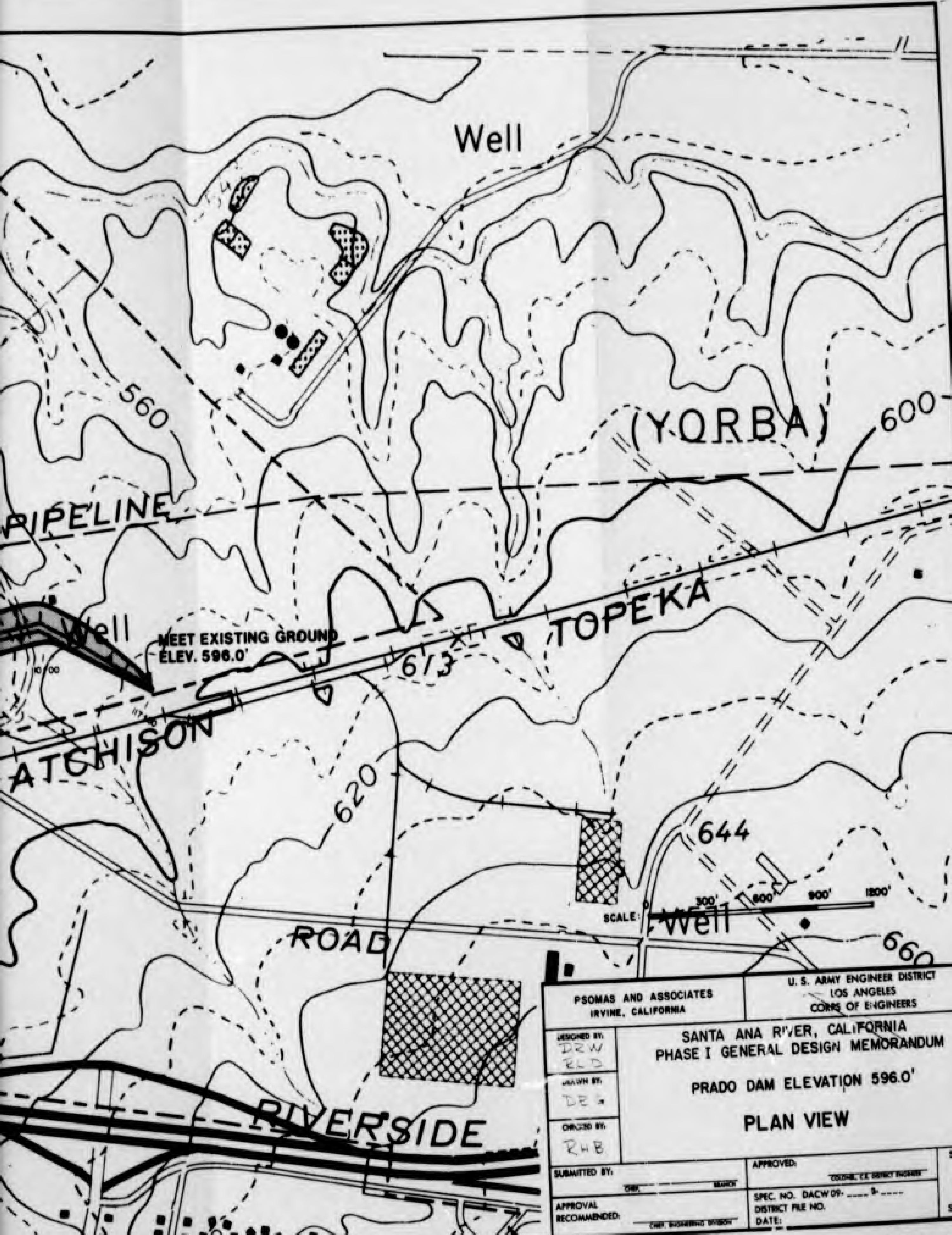
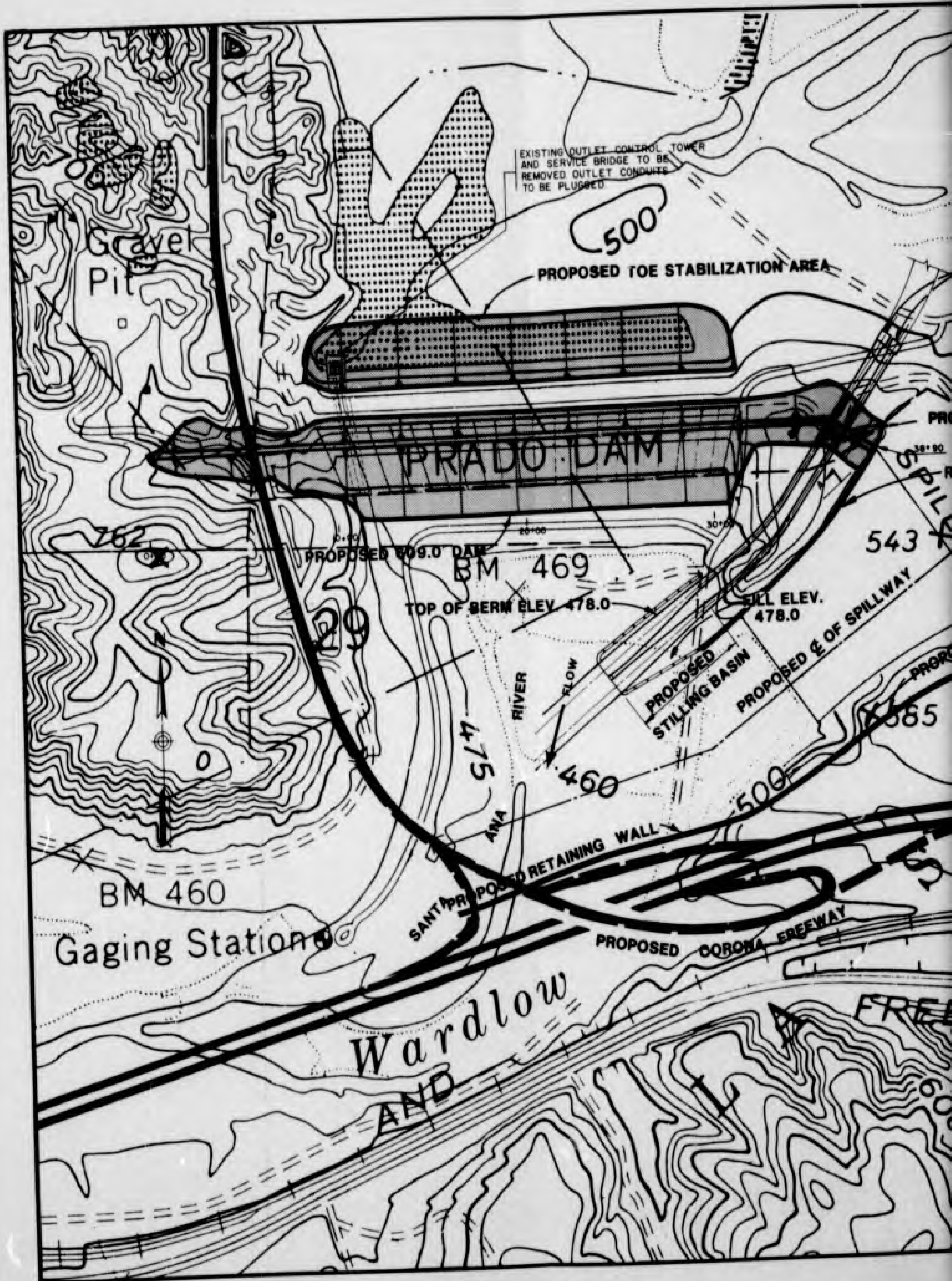
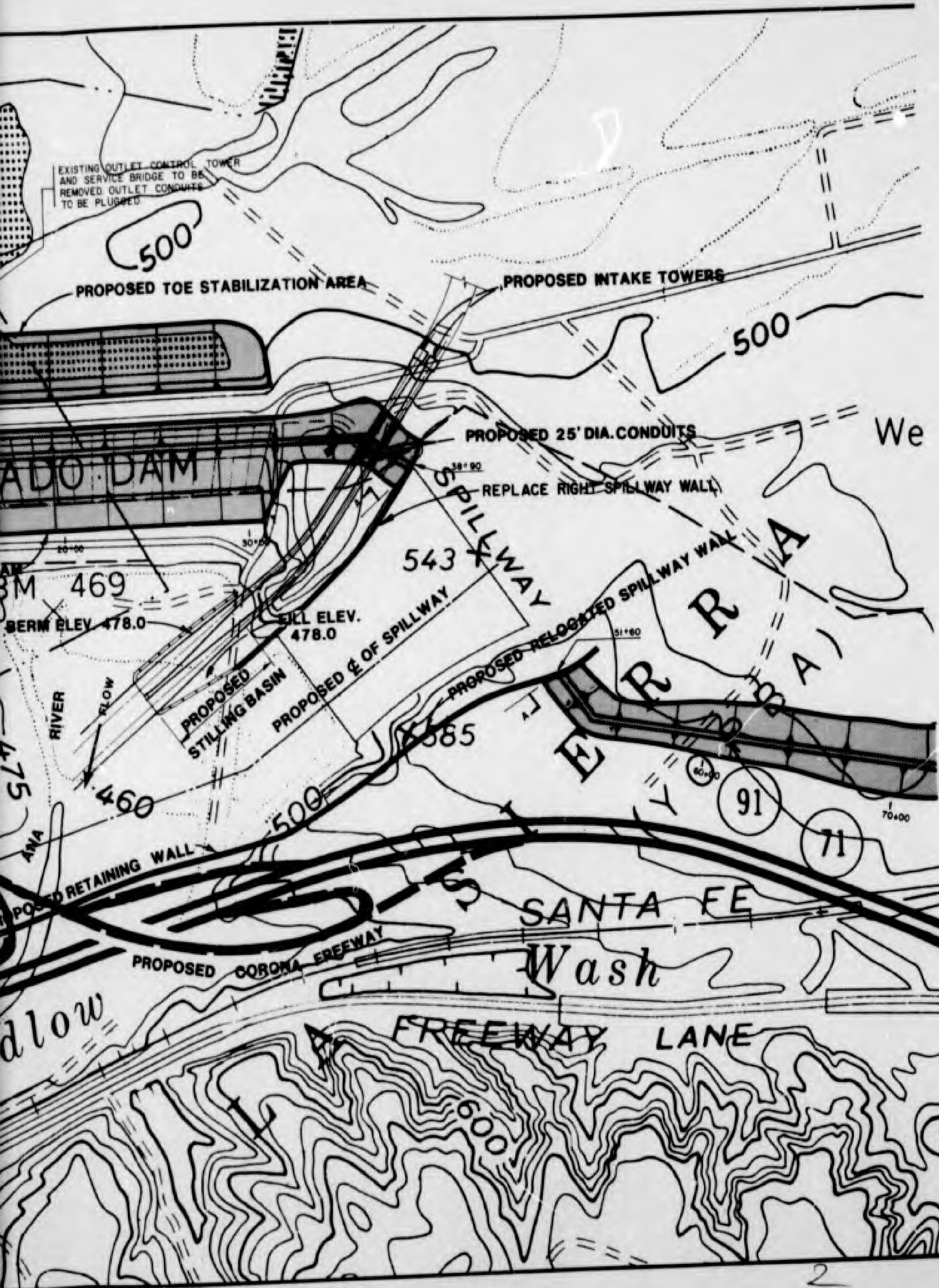

Well
(YORBA)
600
560
PIPELINE
Well
MEET EXISTING GROUND
ELEV. 596.0'
TOPEKA
613
ATCHISON
620
644
SCALE:
300'
600'
900'
1200'
Well
660
ROAD
RIVERSIDE
PSOMAS AND ASSOCIATES
IRVINE, CALIFORNIA
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
DESIGNED BY:
DRW
RLD
DRAWN BY:
DEG
CHECKED BY:
RHB
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
PRADO DAM ELEVATION 596.0'
PLAN VIEW
SUBMITTED BY:
APPROVED:
COLONEL, C.E. DISTRICT ENGINEER
SHEET
OF
SHEETS
APPROVAL
RECOMMENDED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
DATE:

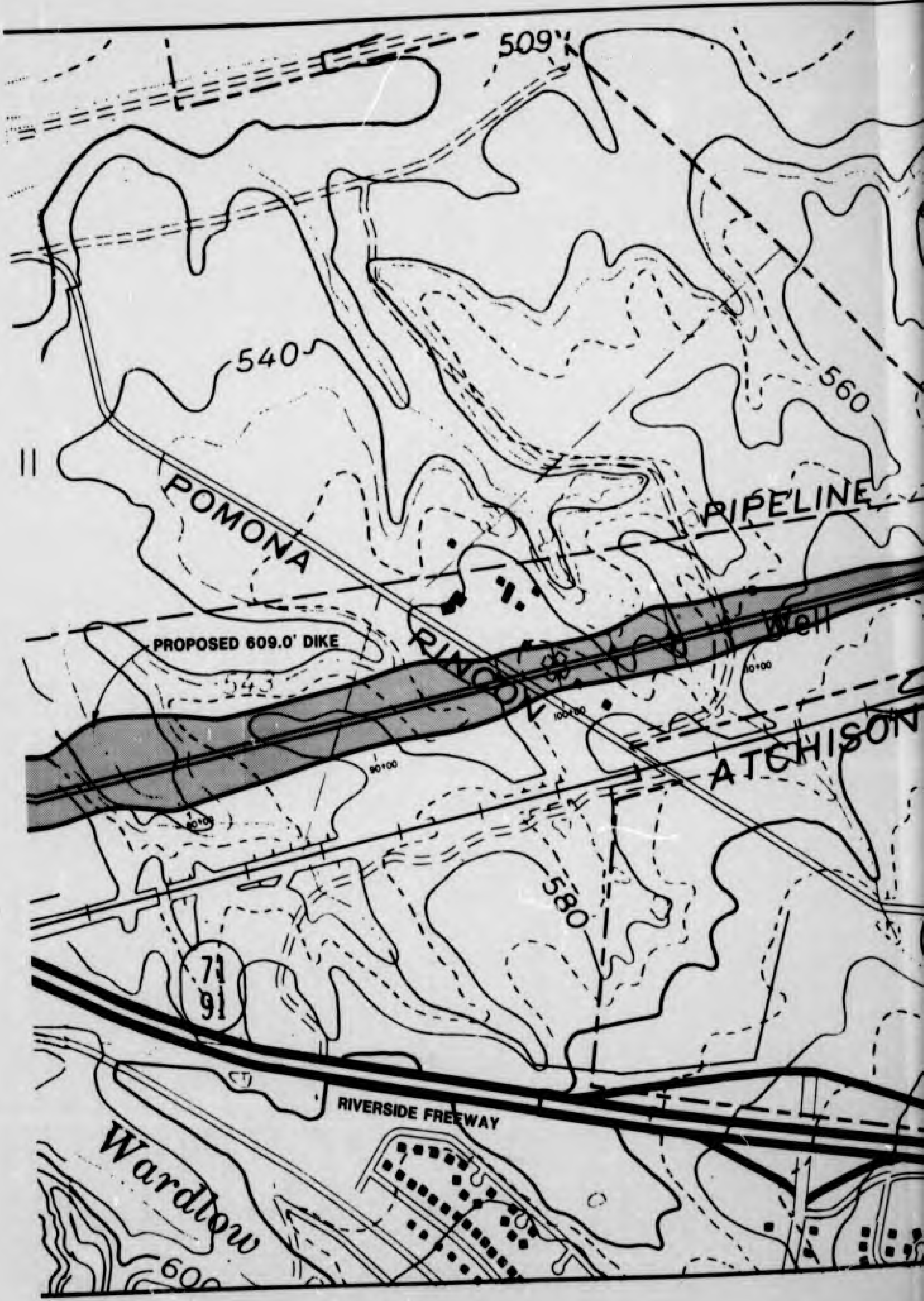
PLATE F-15

2

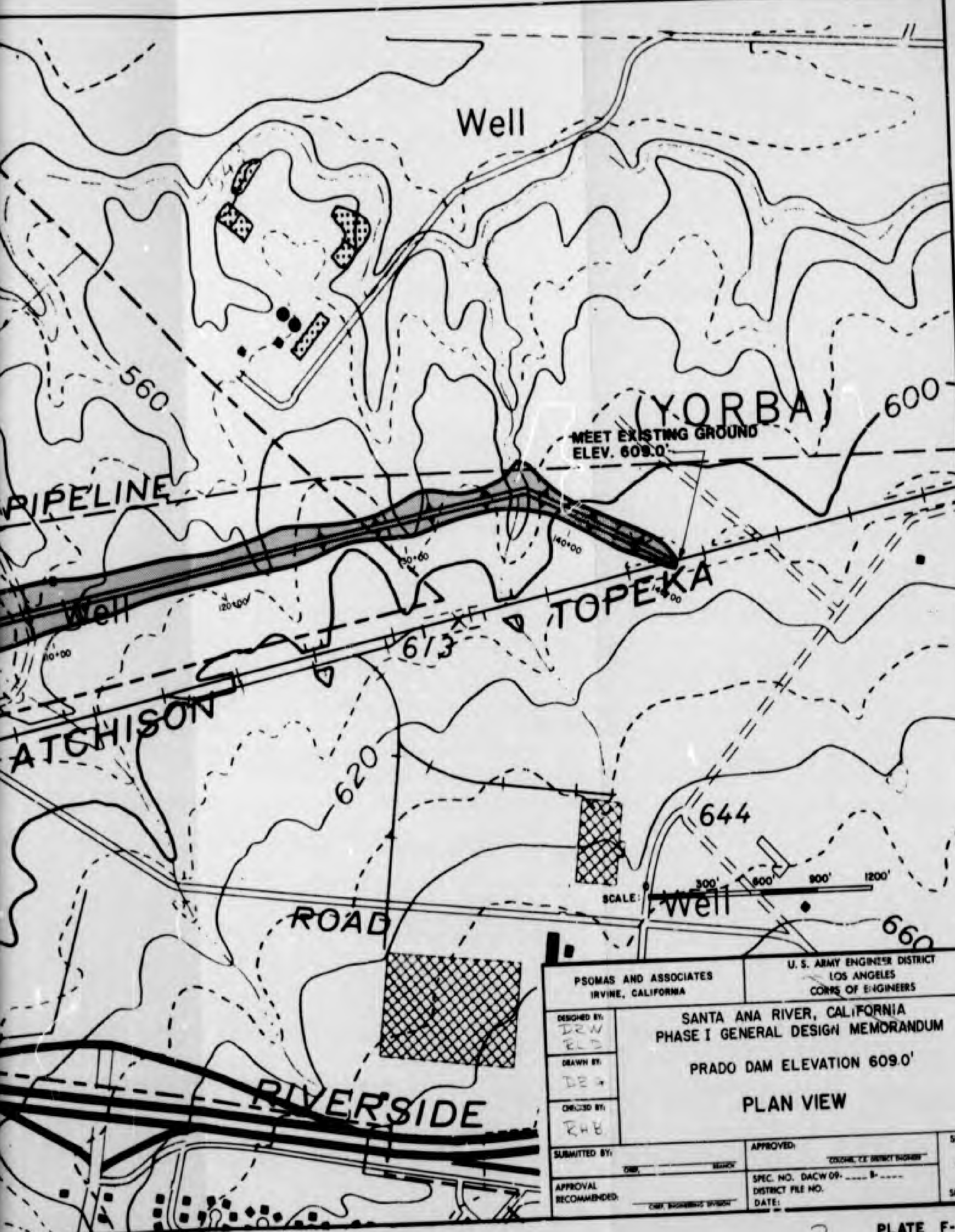


EXISTING OUTLET CONTROL TOWER AND SERVICE BRIDGE TO BE REMOVED OUTLET CONDUITS TO BE PLUGGED
500
PROPOSED TOE STABILIZATION AREA
Gravel Pit
PRADO DAM
762
PROPOSED 609.0 DAM
BM 469
TOP OF BERM ELEV 478.0
FILL ELEV. 478.0
29
PROPOSED STILLING BASIN
PROPOSED ℄ OF SPILLWAY
543
585
RIVER
FLOW
475
460
500
SANTA ANA
PROPOSED RETAINING WALL
BM 460
Gaging Station
PROPOSED CORONA FREEWAY
Wardlow
AND


EXISTING OUTLET CONTROL TOWER AND SERVICE BRIDGE TO BE REMOVED OUTLET CONDUITS TO BE PLUGGED
500
PROPOSED TOE STABILIZATION AREA
PROPOSED INTAKE TOWERS
500
PROPOSED 25' DIA.CONDUITS
We
ADO DAM
REPLACE RIGHT SPILLWAY WALL
SPILLWAY
543
BM 469
BERM ELEV. 478.0
SILL ELEV. 478.0
PROPOSED STILLING BASIN
PROPOSED ℄ OF SPILLWAY
PROPOSED RELOCATED SPILLWAY WALL
FLOW
RIVER
585
460
500
91
71
PROPOSED RETAINING WALL
SANTA FE
Wash
PROPOSED CORONA FREEWAY
FREEWAY
LANE
600



509'
540
560
11
POMONA
PIPELINE
PROPOSED 609.0' DIKE
545
Well
100+00
90+00
ATCHISON
580
71
91
RIVERSIDE FREEWAY
Wardlow
600



PSOMAS AND ASSOCIATES
IRVINE, CALIFORNIA

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

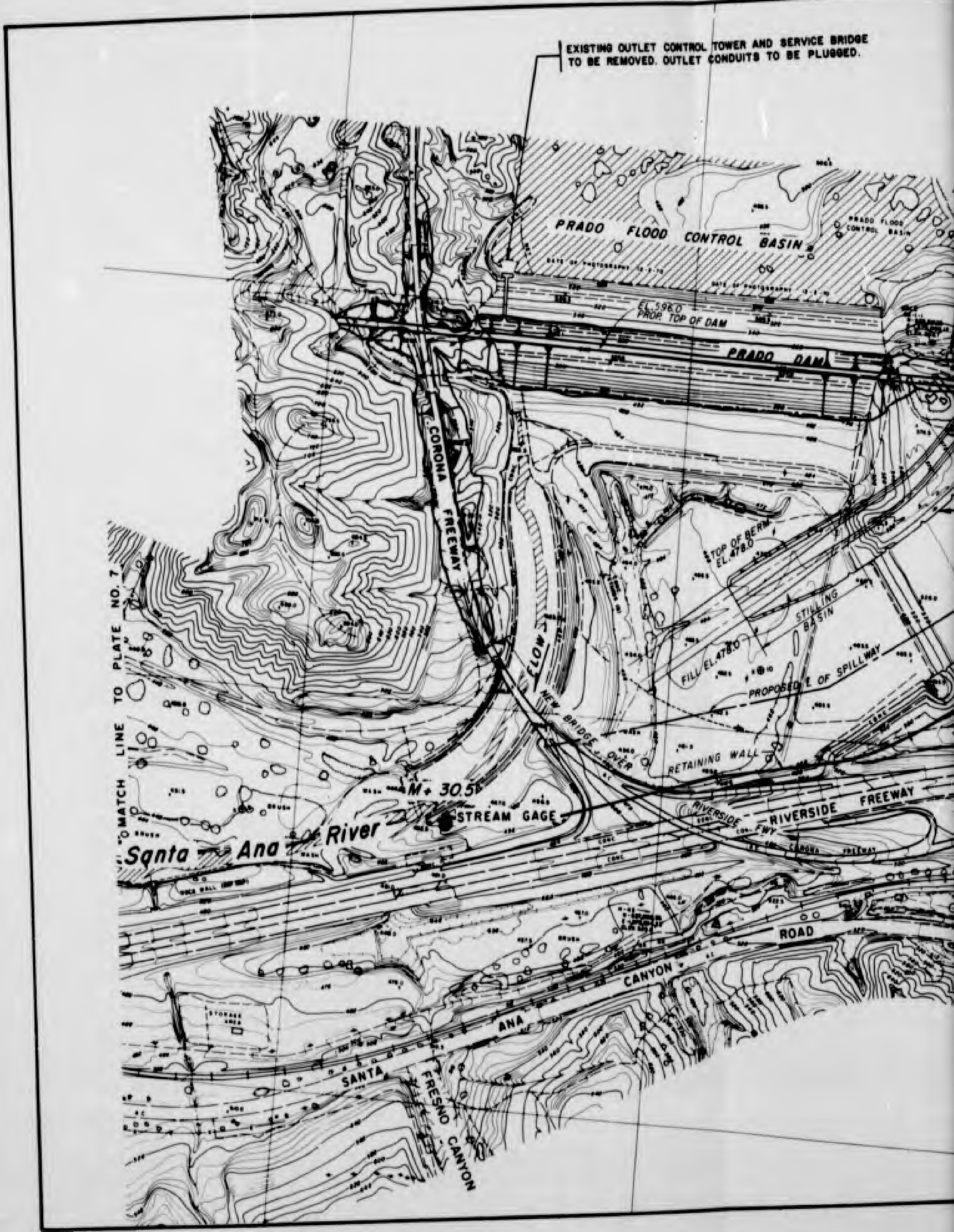
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM ELEVATION 609.0'

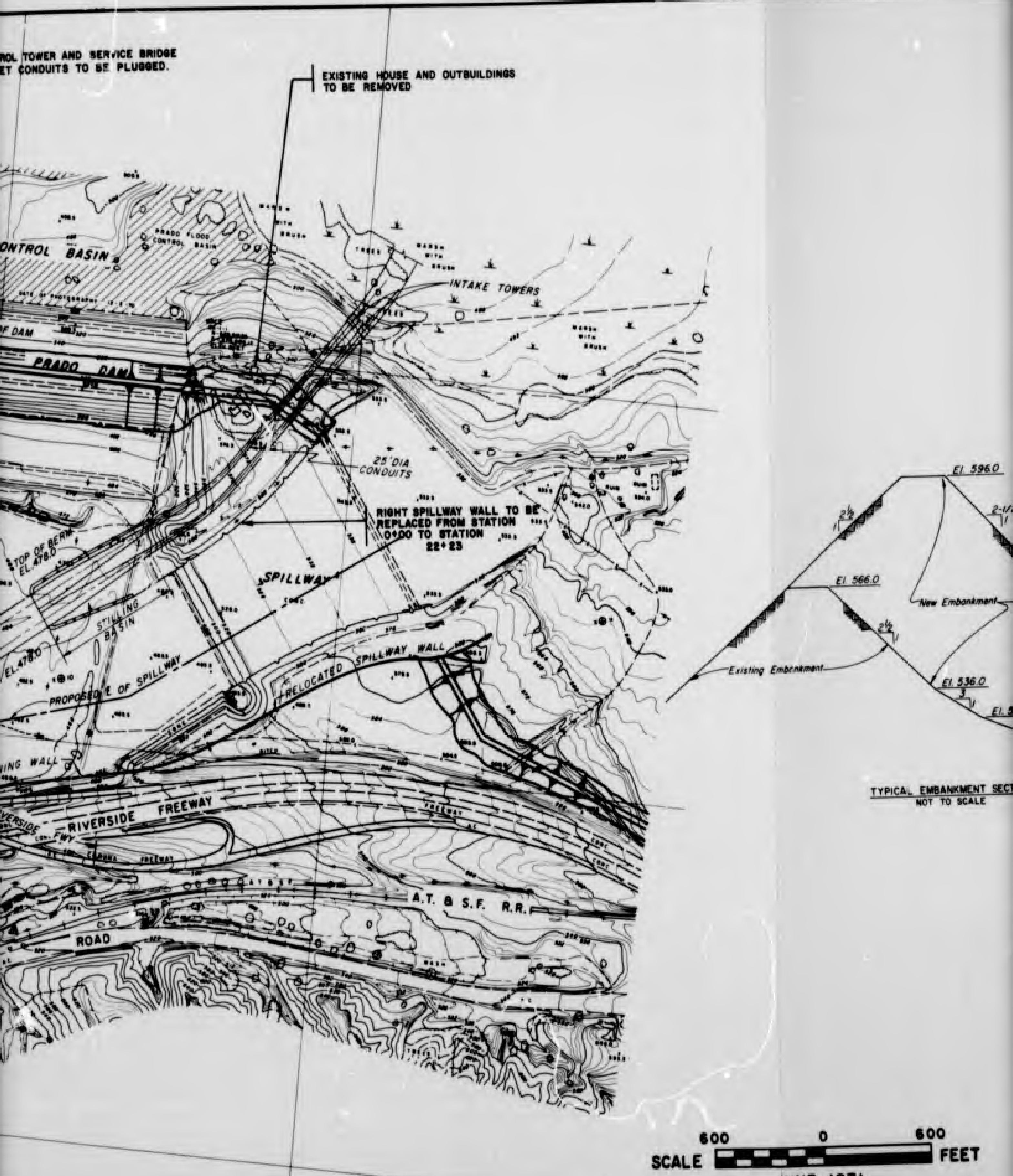
PLAN VIEW

DESIGNED BY: DRW ELD
DRAWN BY: DEJ
CHECKED BY: RHB
SUBMITTED BY: CHIEF, BRANCH
APPROVAL RECOMMENDED: CHIEF, ENGINEERING DIVISION
APPROVED: COLONEL, C.E. DISTRICT ENGINEER
SPEC. NO. DACW09-____B-____
DISTRICT FILE NO.
DATE:
SHEET OF SHEETS

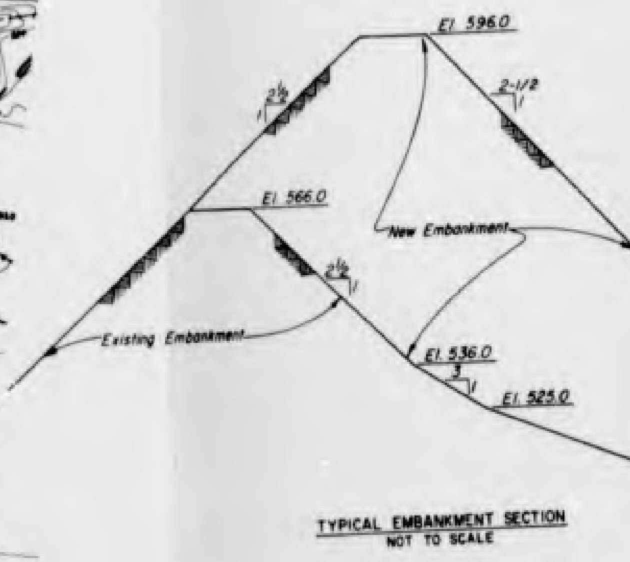
PLATE F-16



EXISTING OUTLET CONTROL TOWER AND SERVICE BRIDGE TO BE REMOVED. OUTLET CONDUITS TO BE PLUGGED.
PRADO FLOOD CONTROL BASIN
EL.596.0 PROP. TOP OF DAM
PRADO DAM
CORONA FREEWAY
TOP OF BERM EL.478.0
STILLING BASIN
FILL EL.478.0
PROPOSED ℄ OF SPILLWAY
FLOW
NEW BRIDGE OVER
RETAINING WALL
M+ 30.5
STREAM GAGE
RIVERSIDE FREEWAY
Santa Ana River
MATCH LINE TO PLATE NO. 7
ROAD
CANYON
SANTA ANA
FRESNO CANYON


ROL TOWER AND SERVICE BRIDGE
ET CONDUITS TO BE PLUGGED.
EXISTING HOUSE AND OUTBUILDINGS
TO BE REMOVED
ONTROL BASIN
INTAKE TOWERS
F DAM
PRADO DAM
25' DIA
CONDUITS
RIGHT SPILLWAY WALL TO BE
REPLACED FROM STATION
0+00 TO STATION
22+23
TOP OF BERM
EL.478.0
SPILLWAY
STILLING
BASIN
EL.476.0
PROPOSED ℄ OF SPILLWAY
RELOCATED SPILLWAY WALL
ING WALL
RIVERSIDE FREEWAY
A.T. & S.F. R.R.
ROAD
El. 596.0
2-1/
El. 566.0
New Embankment
Existing Embankment
El. 536.0
TYPICAL EMBANKMENT SECT
NOT TO SCALE
600
0
600
SCALE
FEET
JUNE 1971

DINGS



TYPICAL EMBANKMENT SECTION
NOT TO SCALE

DATUM IS MEAN SEA LEVEL

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

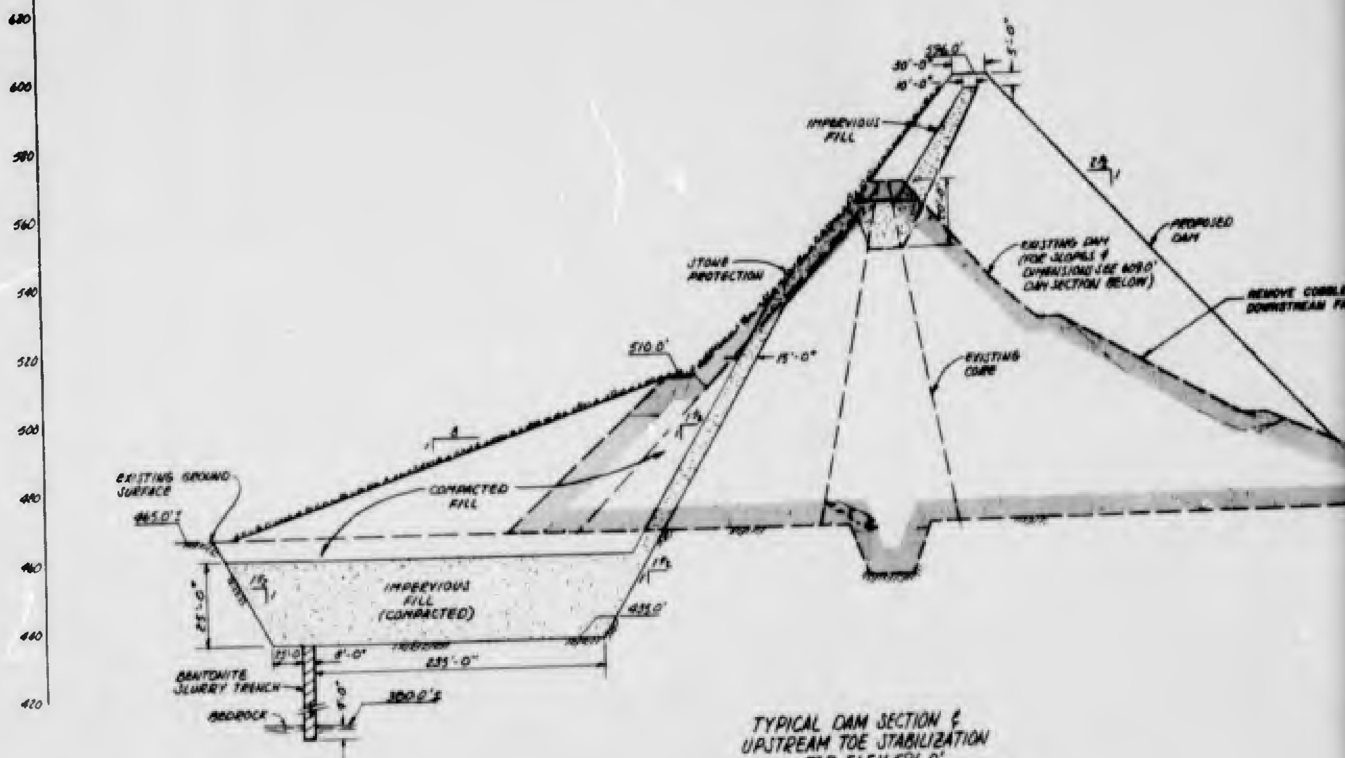
PRADO DAM
EMBANKMENT & SPILLWAY PLAN
ELEVATION 596.0'

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

SCALE 600 0 600 FEET

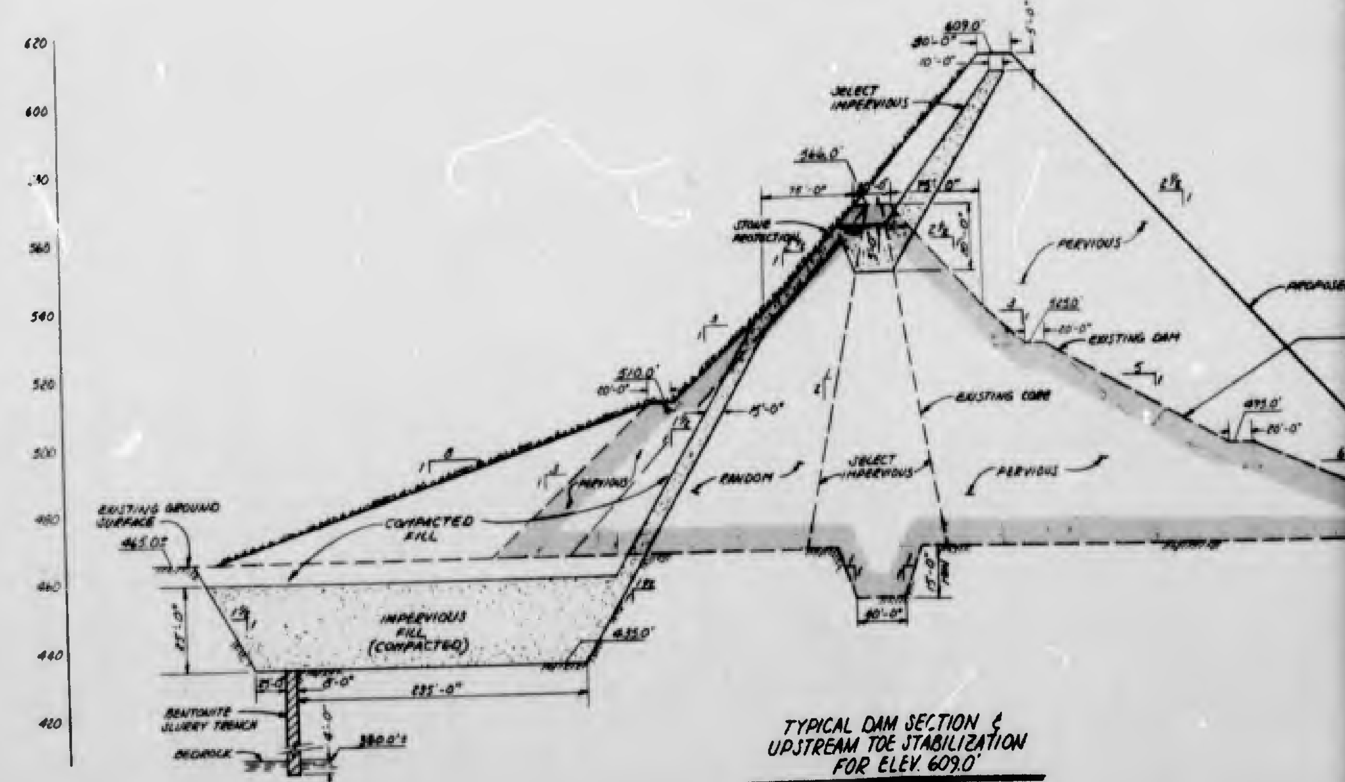
JUNE 1971

PLATE F-17



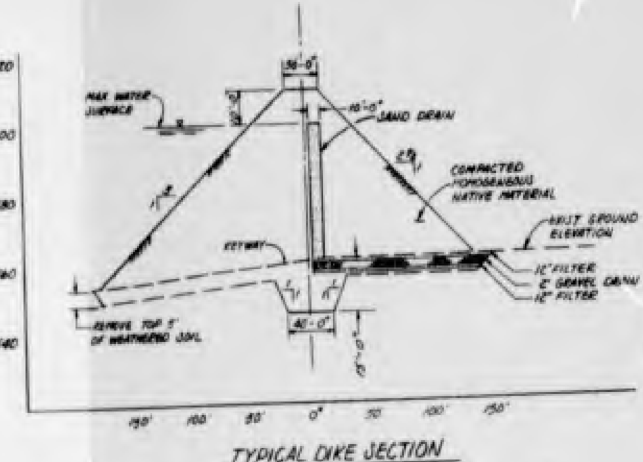
TYPICAL DAM SECTION & UPSTREAM TOE STABILIZATION FOR ELEV. 596.0'

SCALE: HORIZ. 1"=50'-0"
VERT. 1"=20'-0"

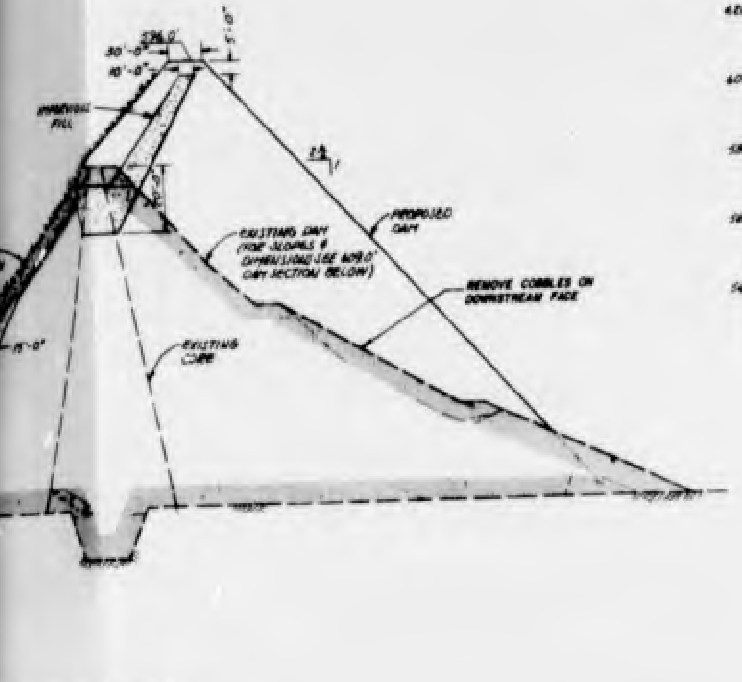


TYPICAL DAM SECTION & UPSTREAM TOE STABILIZATION FOR ELEV. 609.0'

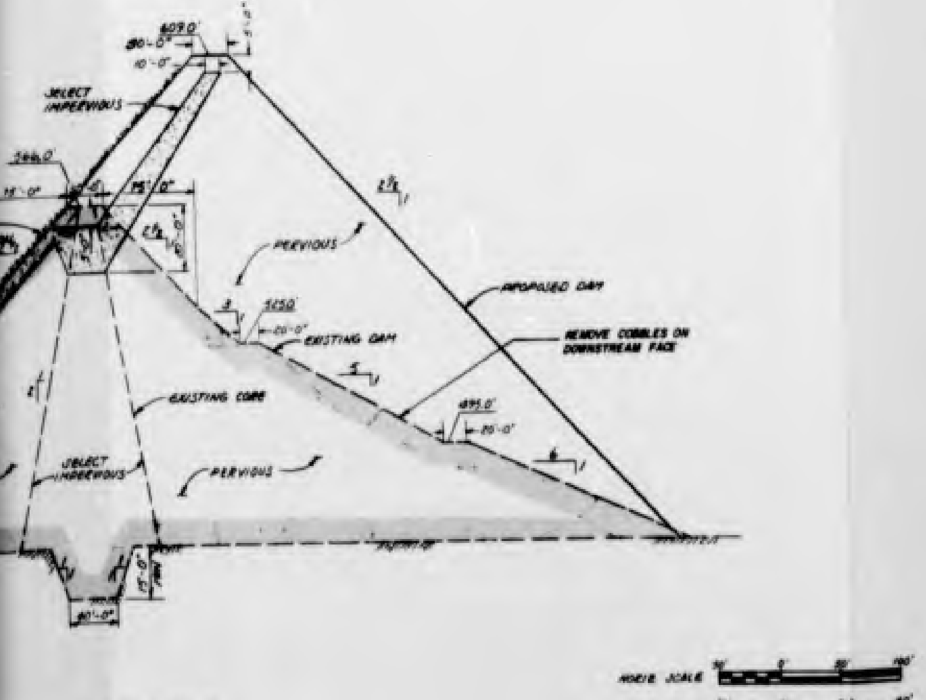
SCALE: HORIZ. 1"=50'-0"
VERT. 1"=20'-0"



TYPICAL DIKE SECTION
SCALE HORIZ. 1"=50'-0" VERT. 1"=20'-0"



...ICAL DAM SECTION & ...REAM TOE STABILIZATION FOR ELEV. 596.0'
SCALE: HORIZ. 1"=50'-0" VERT. 1"=20'-0"



TYPICAL DAM SECTION & ...STREAM TOE STABILIZATION FOR ELEV 609.0'
SCALE: HORIZ. 1"=50'-0" VERT. 1"=20'-0"

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

PSOMAS AND ASSOCIATES
IRVINE, CALIFORNIA

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

DESIGNED BY: D.R.W. R.L.D.
DRAWN BY: B.H. D.G.
CHECKED BY: R.H.B.

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
PRADO DAM ELEVATIONS 596.0' AND 609.0'

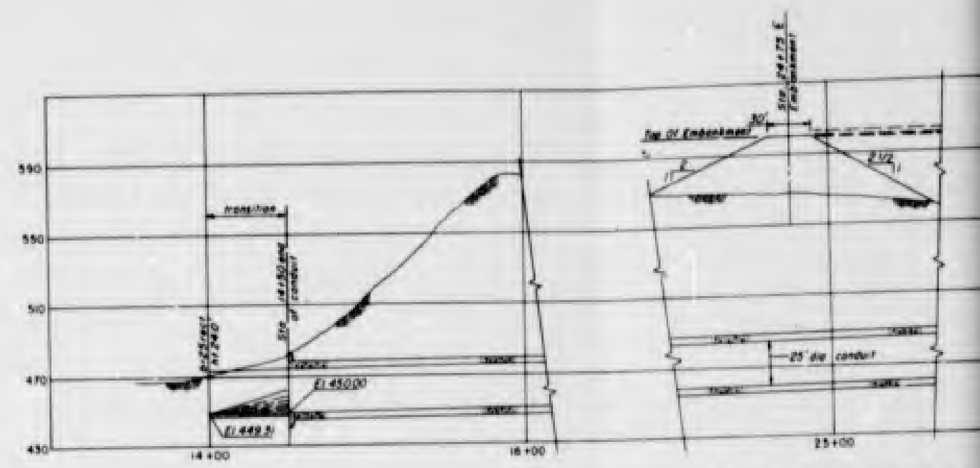
TYPICAL DAM AND DIKE SECTIONS

SUBMITTED BY: | DATE APPROVED: | SPEC. NO. DACW09- | SHEET
DISTRICT FILE NO.

PLATE F-18

2

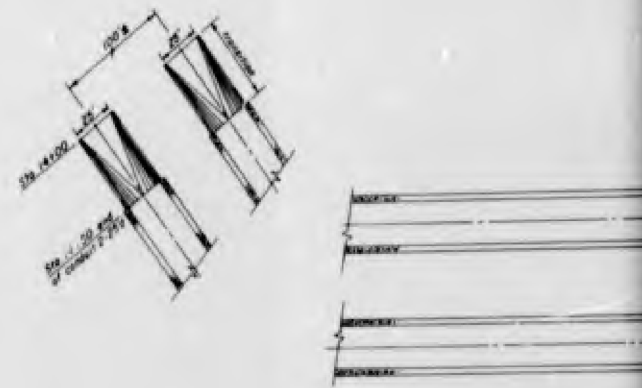
U.S. ARMY ENGINEER DISTRICT



℄ SECTIONAL PROFILE OF

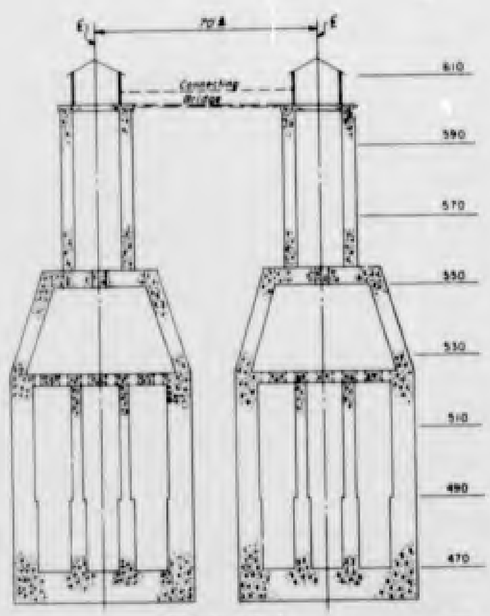
HORIZ SCALE

VERT SCALE



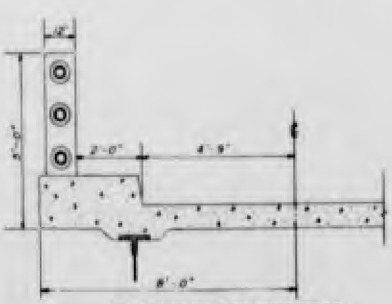
℄ SECTIONAL PLAN OF

HORIZ SCALE 0 40 80



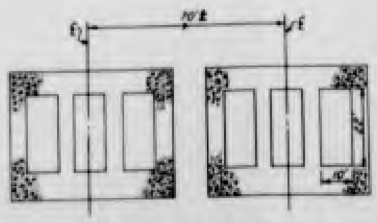
SECTION A-A

SCALE 10 0 10 20 30 40 50 FEET



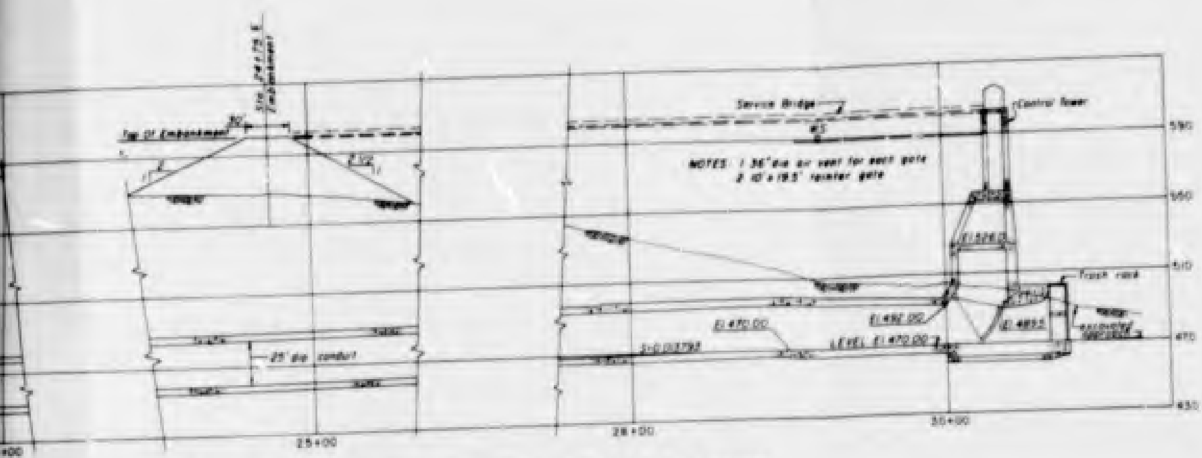
HALF SECTION SERVICE BRIDGE

SCALE FEET



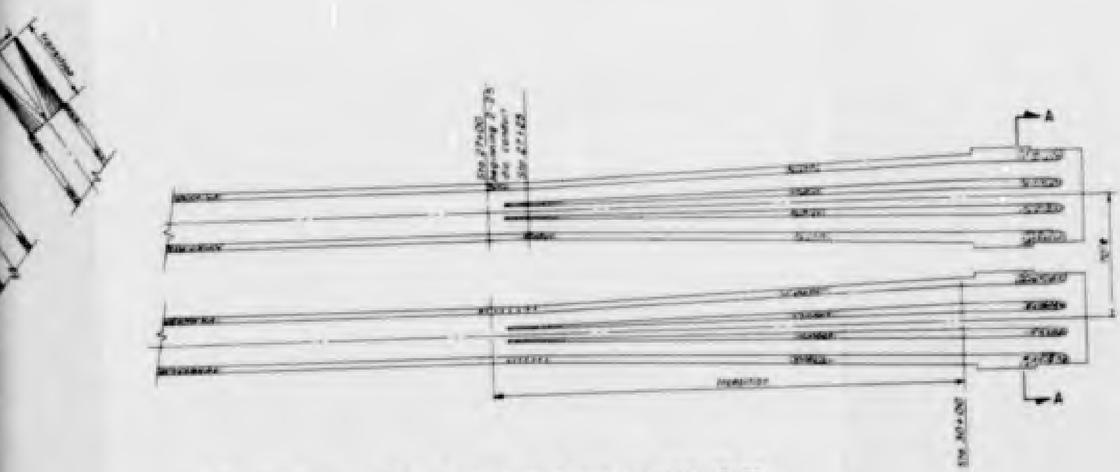
SECTION STA 30+00

SCALE FEET



℄ SECTIONAL PROFILE OF CONDUIT & OUTLET STRUCTURE

HORIZ SCALE FEET
VERT SCALE FEET



℄ SECTIONAL PLAN OF CONDUIT & OUTLET STRUCTURE

HORIZ SCALE 0 40 80 120

...N STA. 30+00

SECTION STA. 27+25

SCALE FEET

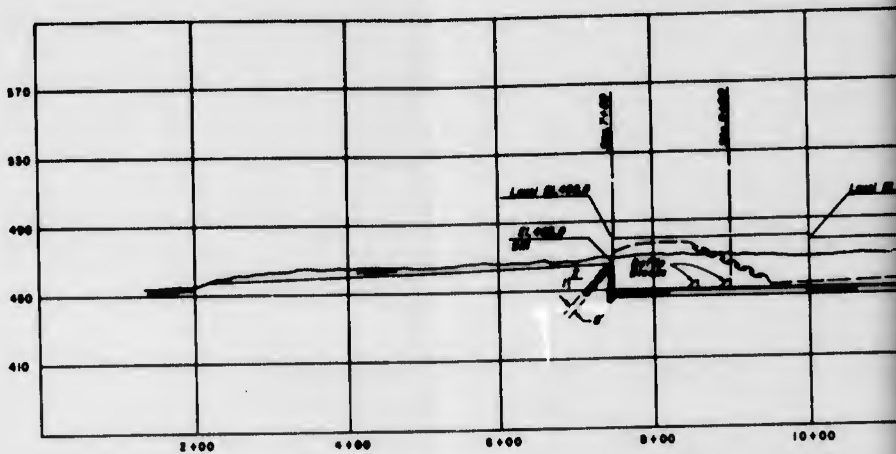
DATUM IS MEAN SEA LEVEL

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

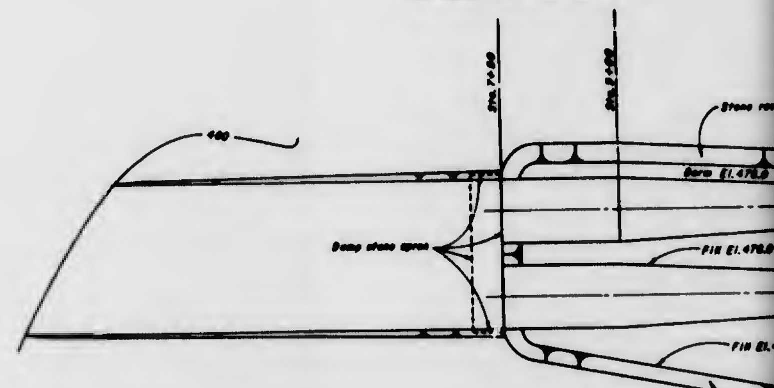
PRADO DAM ELEVATIONS 596.0' AND 609.0'

OUTLET WORKS

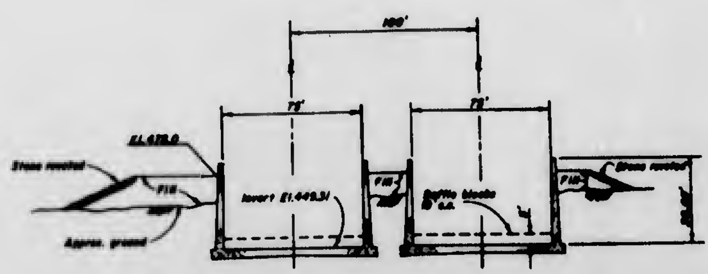
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



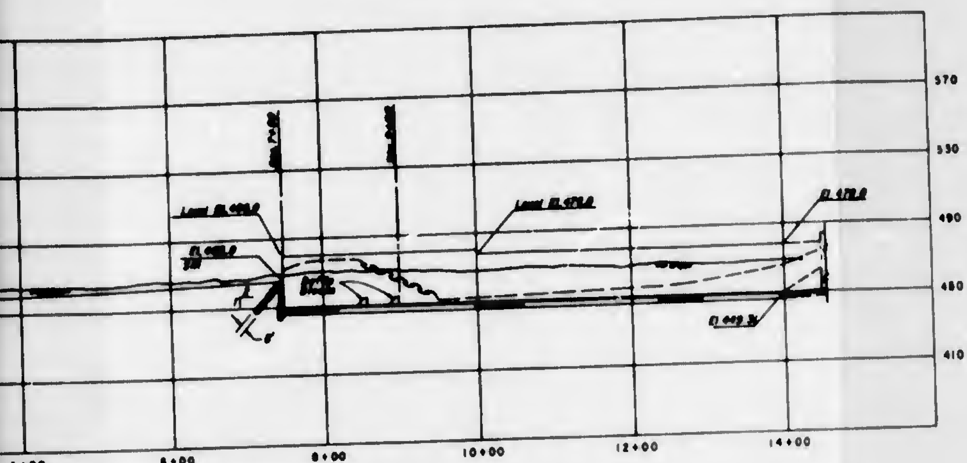
STILLING BASIN PROFILE
HORIZ. SCALE FEET
VERT. SCALE FEET



PLAN
SCALE: 1 IN. = 80 FT.



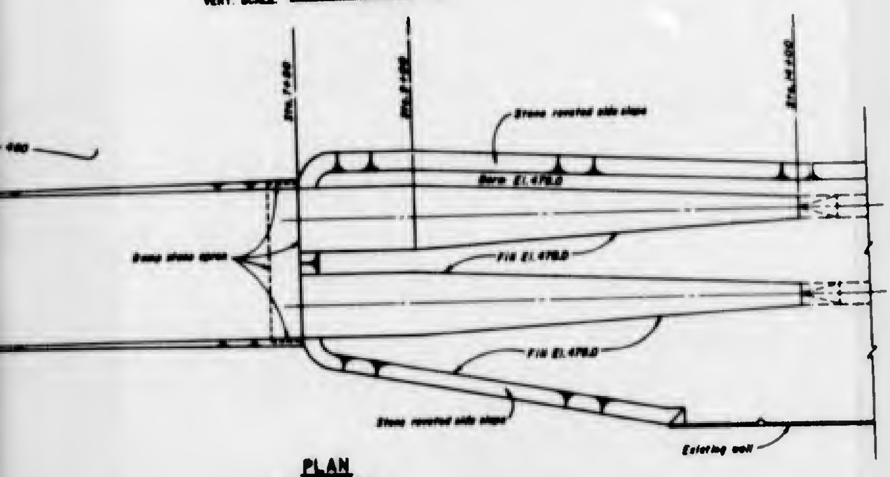
CROSS SECTION STA. 9+00
SCALE: NOT TO SCALE



STILLING BASIN PROFILE

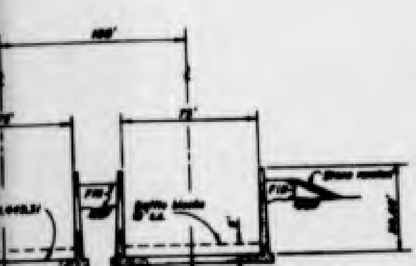
HORIZ SCALE — FEET

VERT. SCALE — FEET



PLAN

SCALE: 1 IN. = 80 FT.



CROSS SECTION STA. 9+00

DATUM IS MEAN SEA LEVEL

SANTA ANA RIVER, CALIFORNIA

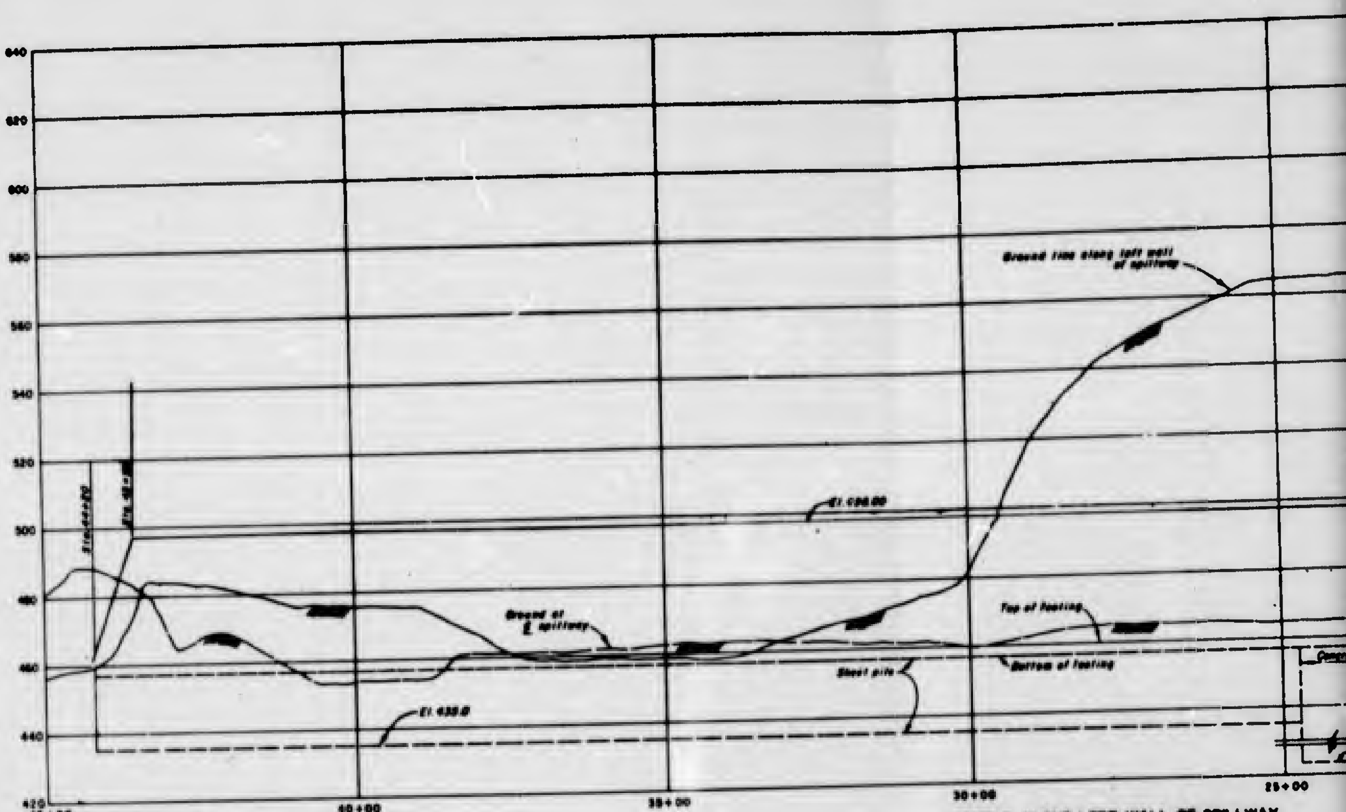
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM ELEVATIONS 596.0' AND 609.0'

OUTLET WORKS

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

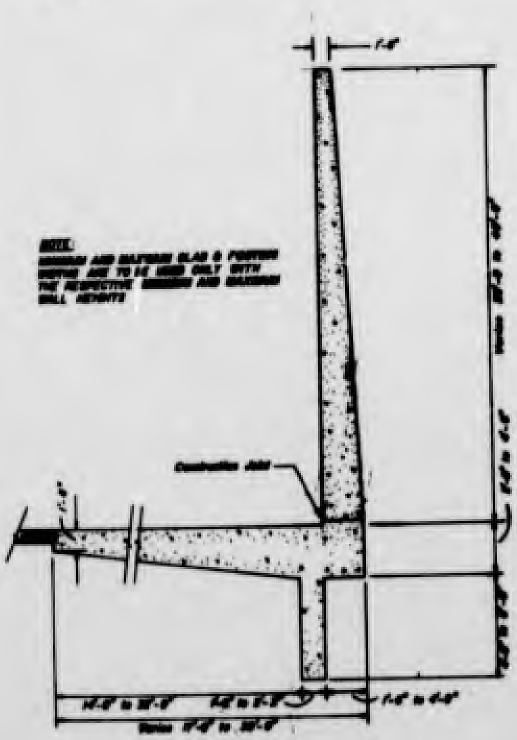
2



PROFILE ALONG LEFT WALL OF SPILLWAY

HORIZ. SCALE: FEET

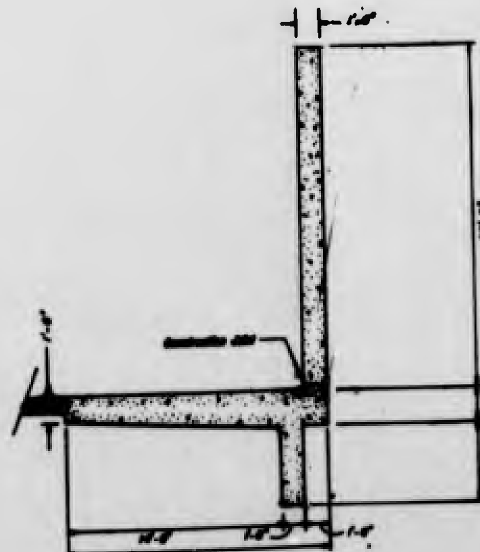
VERT. SCALE: FEET



TYPICAL SPILLWAY WALL SECTION

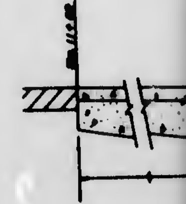
STA. 10+00 TO 13+00

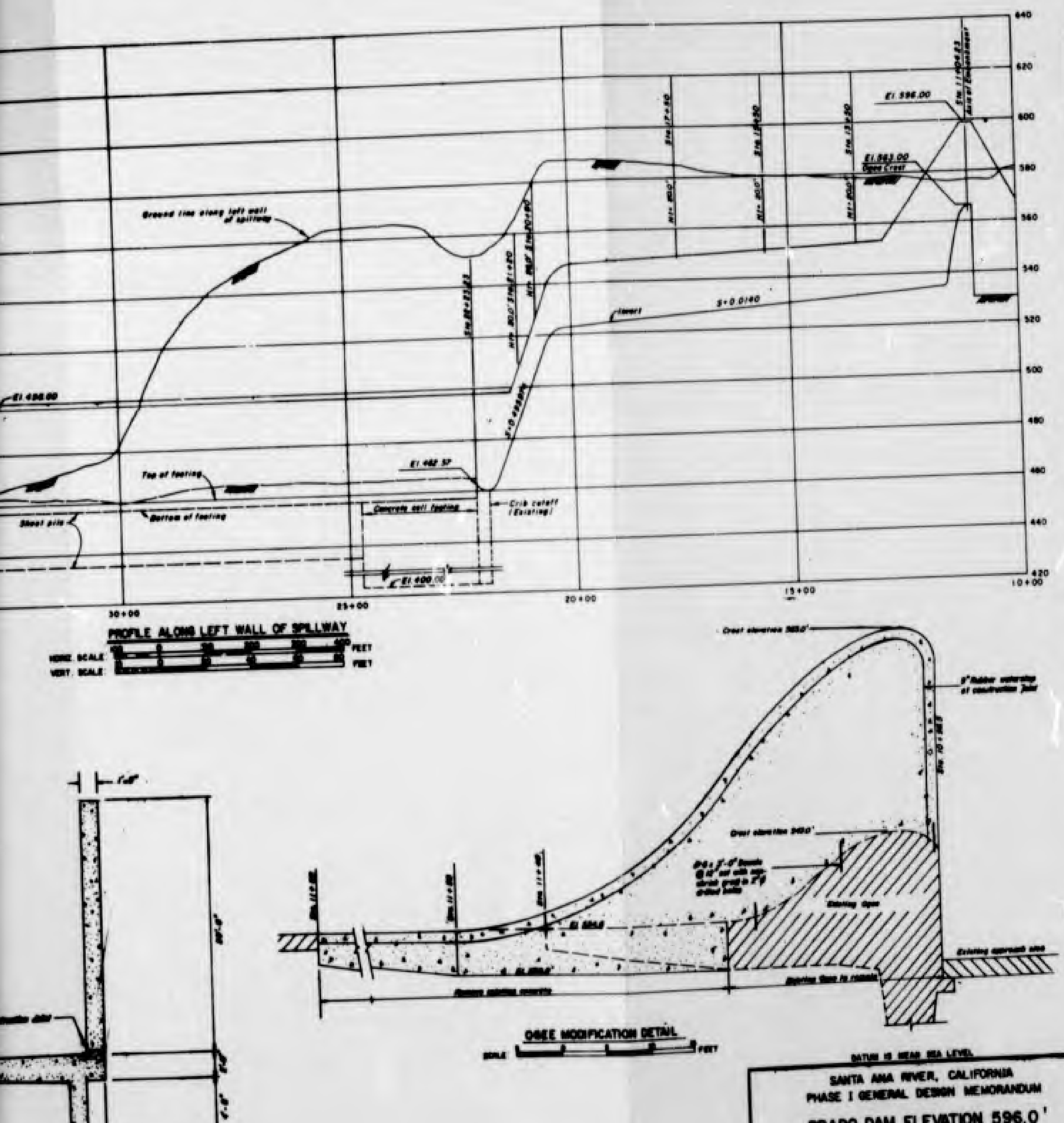
SCALE: FEET



TYPICAL SPILLWAY WALL SECTION

SCALE: FEET





DATUM IS MEAN SEA LEVEL

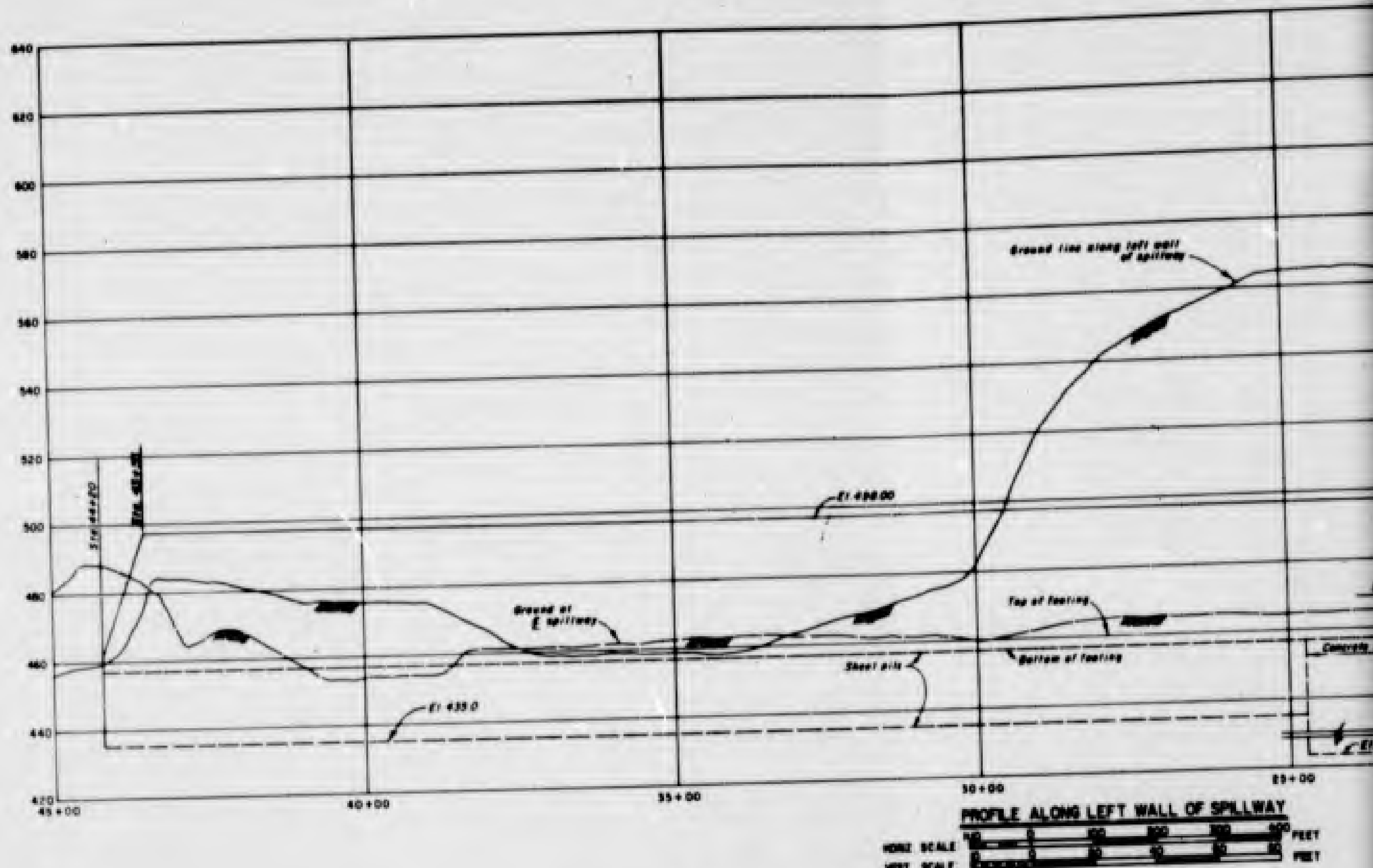
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM ELEVATION 596.0'

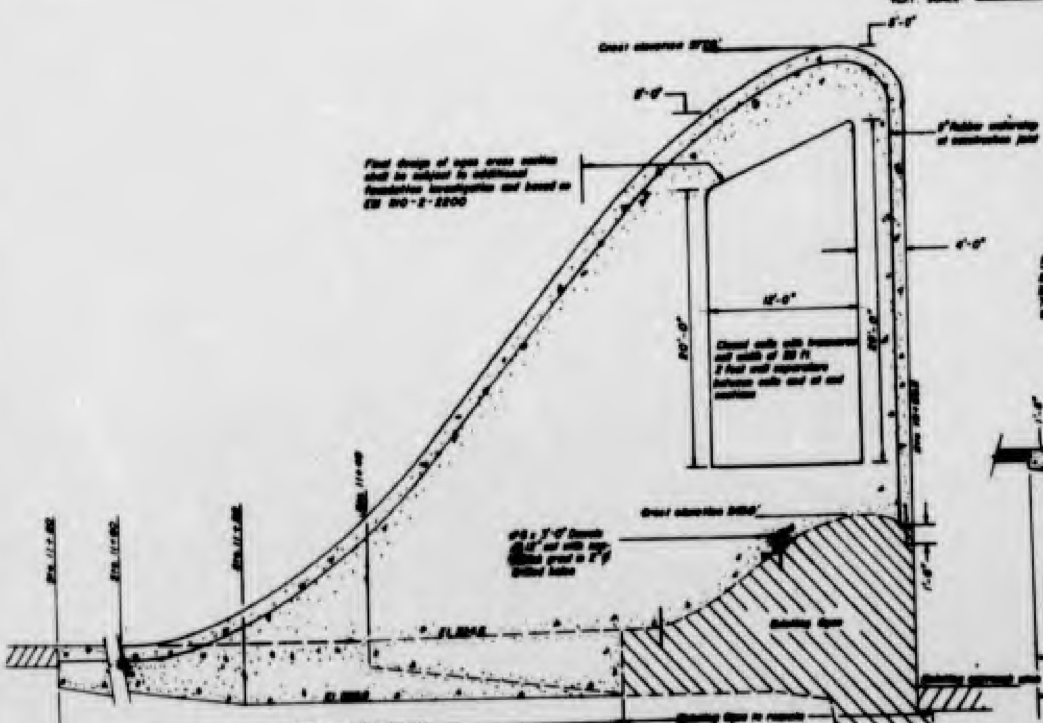
SPILLWAY DETAILS AND LEFT WALL PROFILE

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

U.S. ARMY ENGINEER DISTRICT

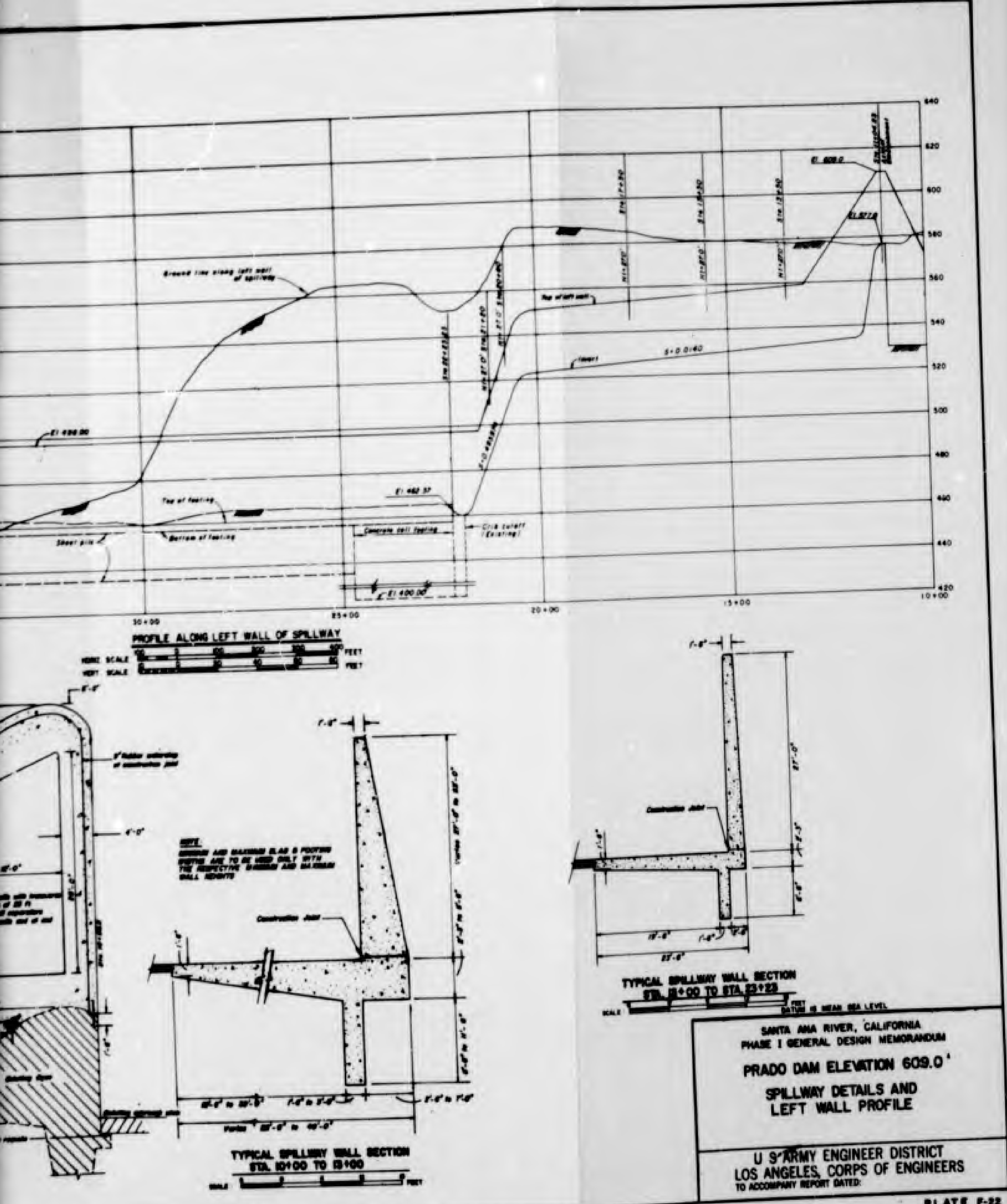


PROFILE ALONG LEFT WALL OF SPILLWAY



OGEE MODIFICATION DETAIL

TYPICAL SPILLWAY WALL SECTI
STA. 10+00 TO 13+00

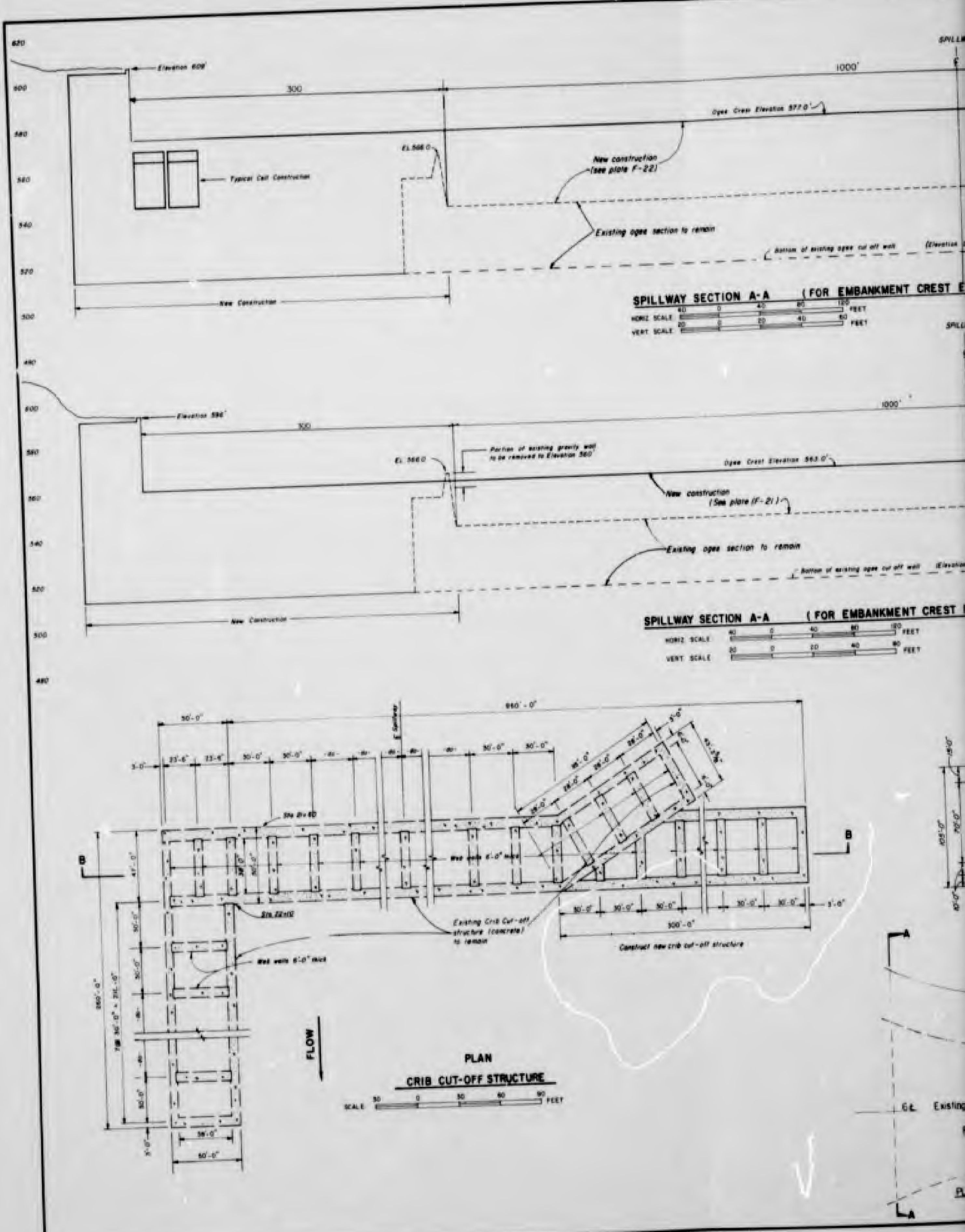


SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

PRADO DAM ELEVATION 609.0'

SPILLWAY DETAILS AND LEFT WALL PROFILE

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



Elevation 609'
300
1000'
Ogee Crest Elevation 577.0'
EL 566.0
New construction (see plate F-22)
Typical Cell Construction
Existing ogee section to remain
Bottom of existing ogee cut off wall
New Construction
SPILLWAY SECTION A-A (FOR EMBANKMENT CREST E
HORIZ SCALE
VERT SCALE
FEET
Elevation 596'
Portion of existing gravity wall to be removed to Elevation 560'
Ogee Crest Elevation 563.0'
EL 566.0
New construction (See plate (F-21))
Existing ogee section to remain
SPILLWAY SECTION A-A (FOR EMBANKMENT CREST
860' - 0"
Web walls 6'-0" thick
Existing Crib Cut-off structure (concrete) to remain
300'-0"
Construct new crib cut-off structure
FLOW
PLAN
CRIB CUT-OFF STRUCTURE
SCALE
FEET
Existing

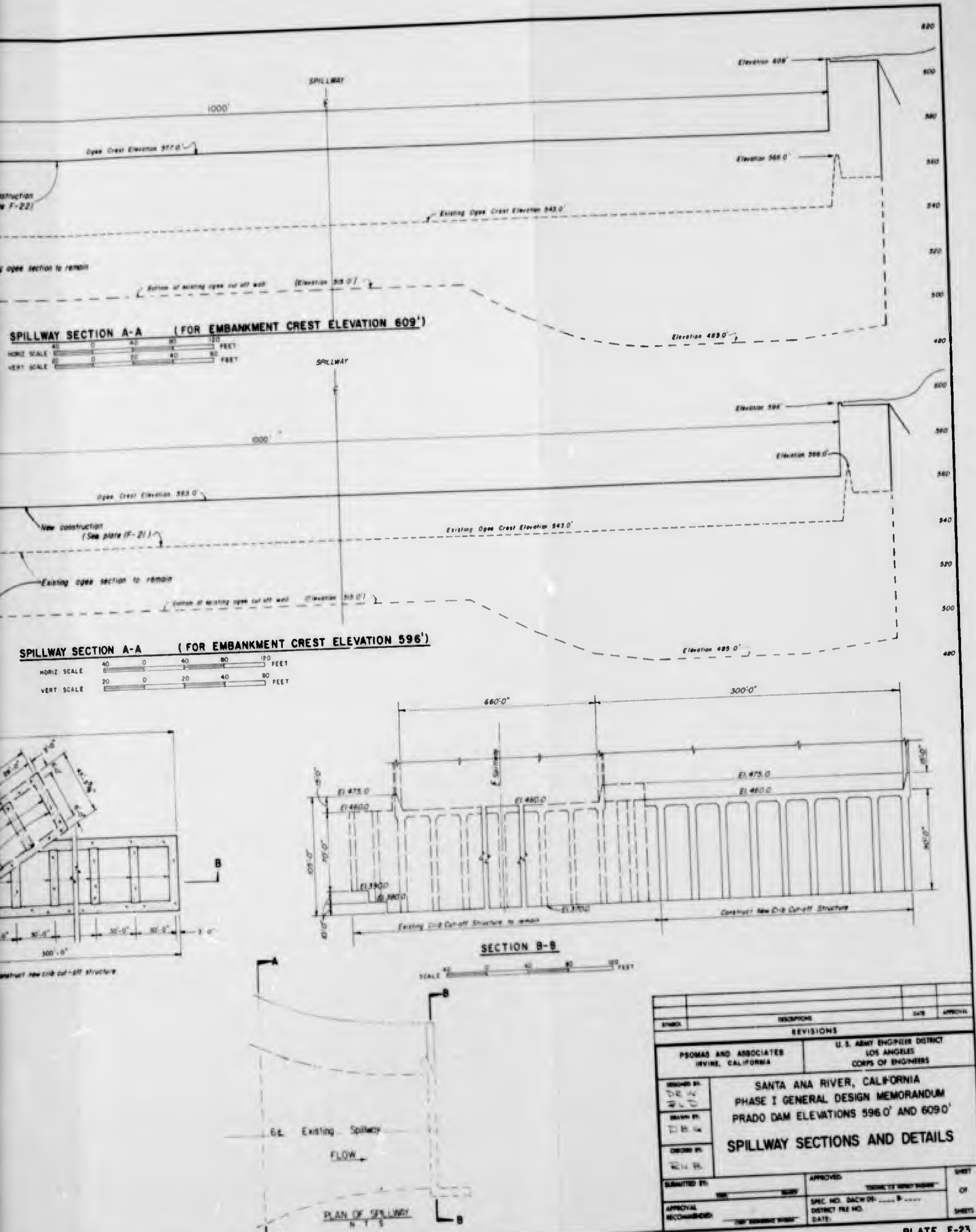

SPILLWAY
1000'
Elevation 609'
Elevation 566.0'
Existing Ogee Crest Elevation 543.0'
Elevation 485.0'
SPILLWAY SECTION A-A (FOR EMBANKMENT CREST ELEVATION 609')
HORIZ SCALE
VERT SCALE
FEET
SPILLWAY
1000'
Elevation 596'
Elevation 566.0'
Ogee Crest Elevation 583.0'
New construction
(See plate (F-21))
Existing Ogee Crest Elevation 543.0'
Existing ogee section to remain
Bottom of existing ogee cut off wall
(Elevation 515.0')
Elevation 485.0'
SPILLWAY SECTION A-A (FOR EMBANKMENT CREST ELEVATION 596')
HORIZ SCALE
VERT SCALE
FEET
660'-0"
300'-0"
El. 475.0
El. 460.0
El. 460.0
El. 475.0
El. 460.0
El. 390.0
El. 380.0
El. 370.0
Existing Crib Cut-off Structure to remain
Construct New Crib Cut-off Structure
SECTION B-B
SCALE
FEET
300'-0"
construct new crib cut-off structure
Existing Spillway
FLOW
PLAN OF SPILLWAY
N. T. S.
REVISIONS
DESCRIPTIONS
DATE
APPROVAL
SYMBOL
PSOMAS AND ASSOCIATES
IRVINE, CALIFORNIA
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
PRADO DAM ELEVATIONS 596.0' AND 609.0'
SPILLWAY SECTIONS AND DETAILS
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
APPROVAL RECOMMENDED:
APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
DATE:
SHEET
OF
SHEETS

PLATE F-23


Top of reservoir
min. elev. 302.0
Inlet (See sheet F-25)
Q Max = 5,600 c.f.s.
PLAN
SCALE 200 100 0 100 200 FEET

DATUM IS MEAN SEA LEVEL

REVISIONS

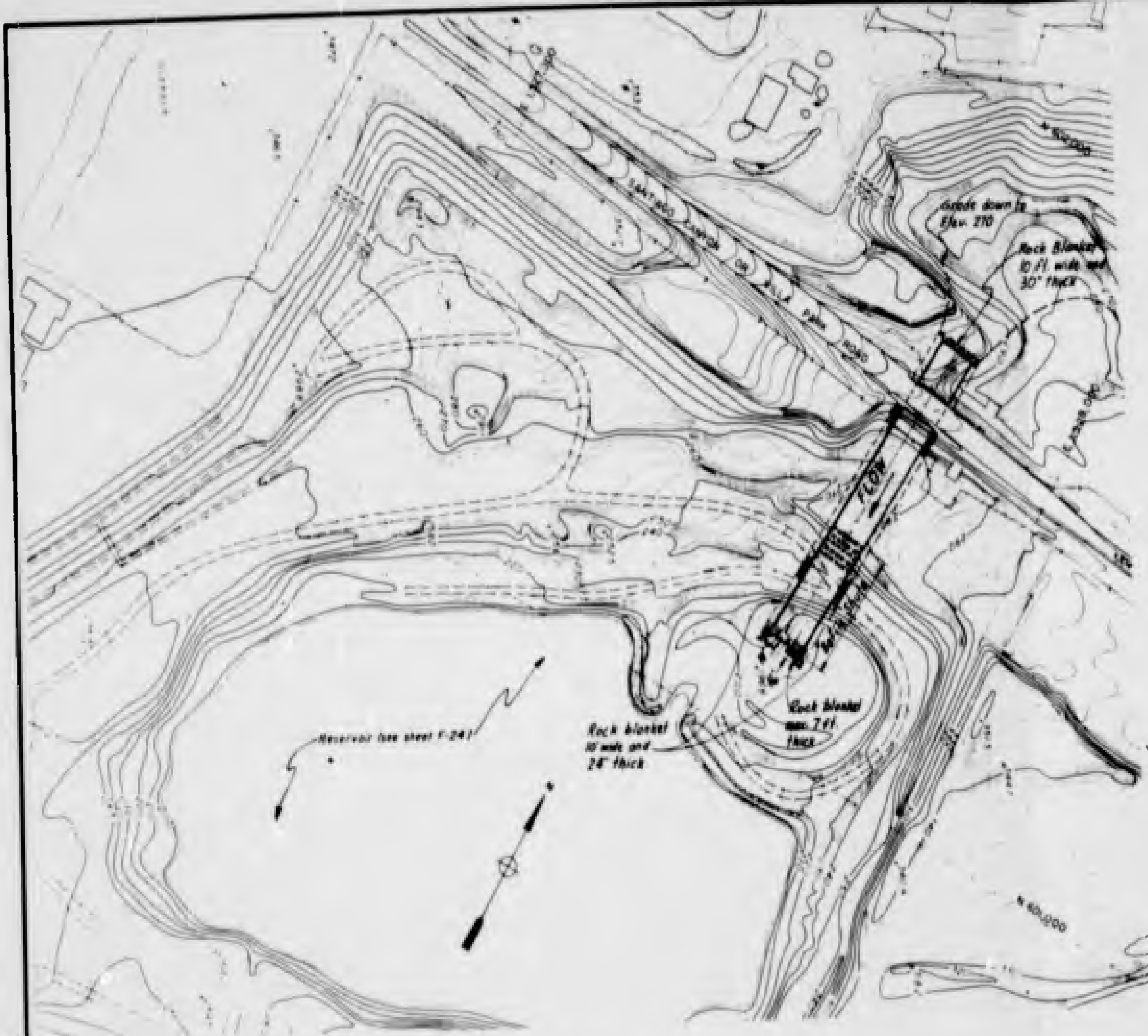
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTIAGO CREEK RESERVOIR

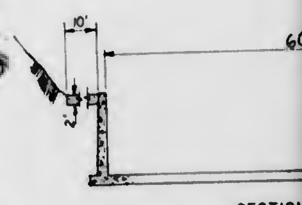
PLAN

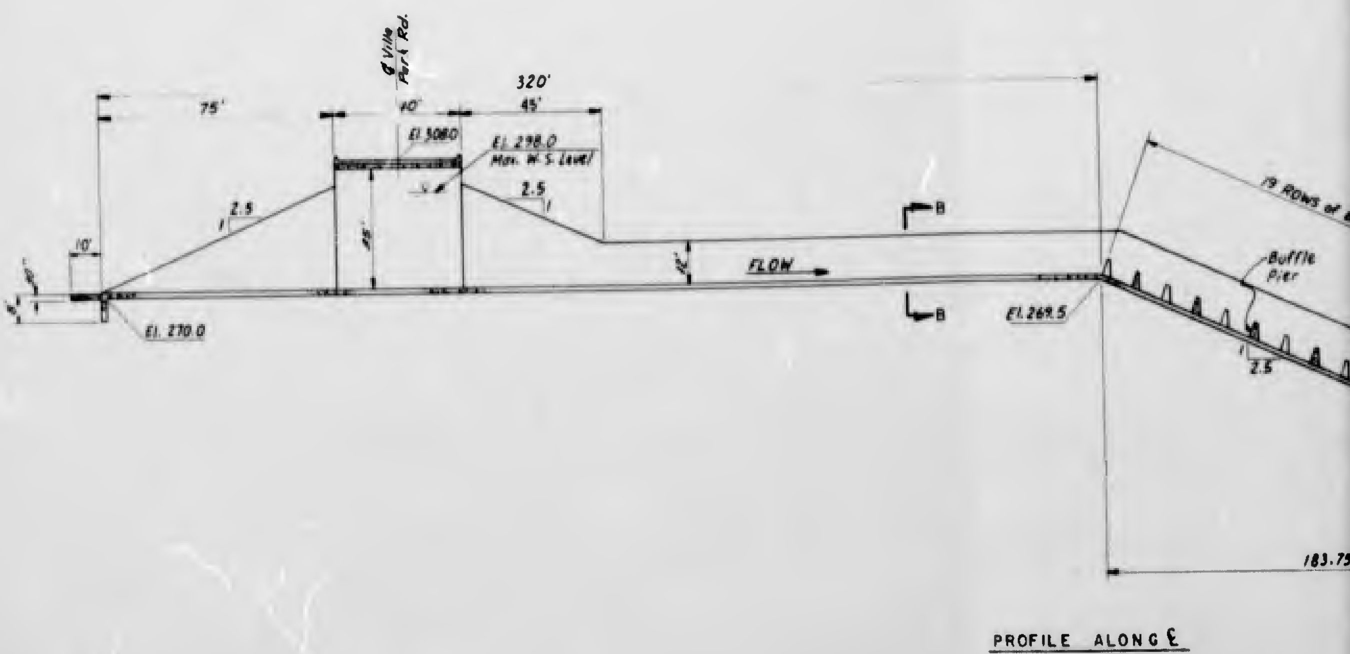
DESIGNED BY:
DRAWN BY: C.W.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
SHEET

PLATE F-24



PLAN
SCALE 1 IN = 100 FT.





PROFILE ALONG ℄
SCALE 1 IN = 20 FT

VALUE ENGINEERING PAYS

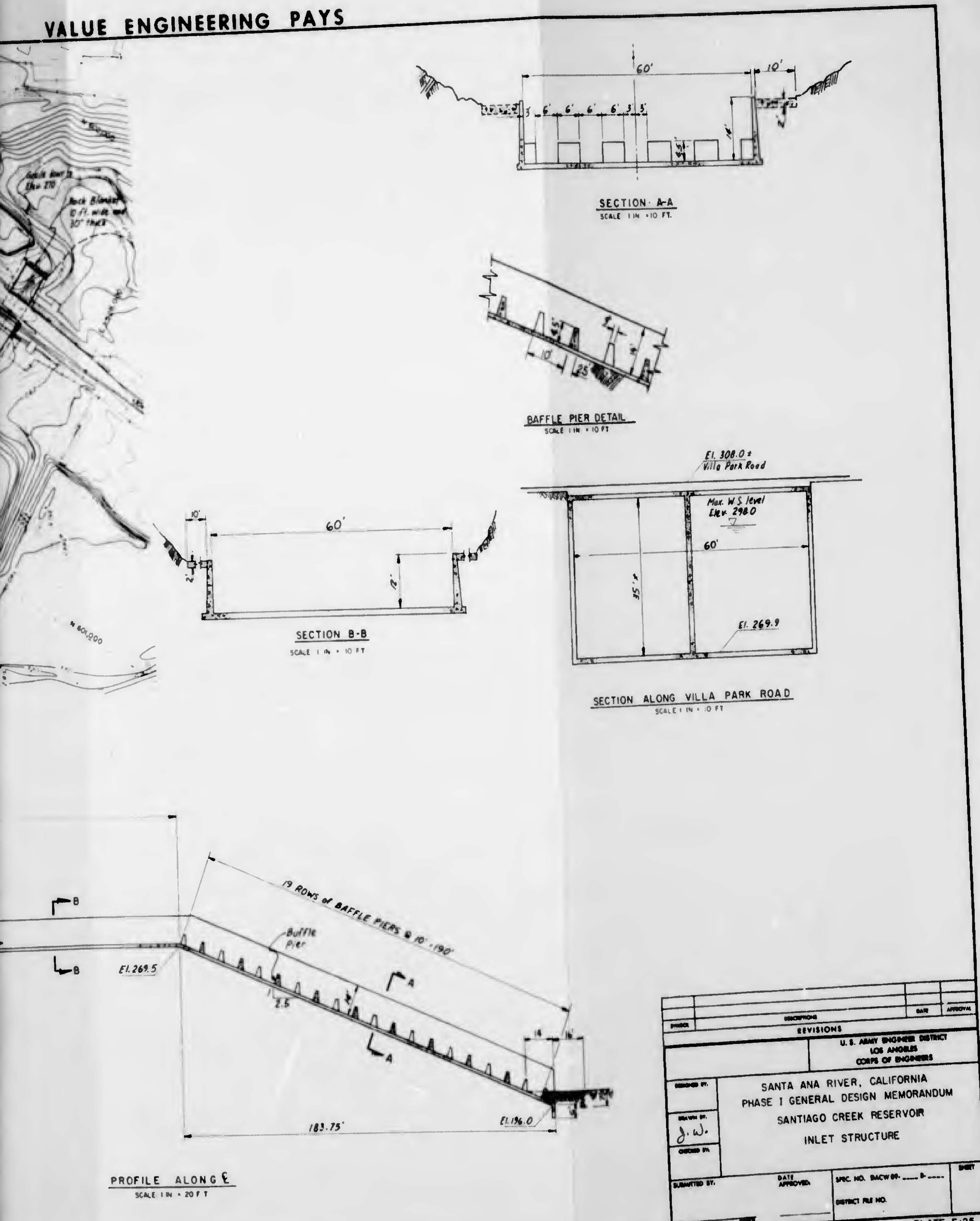
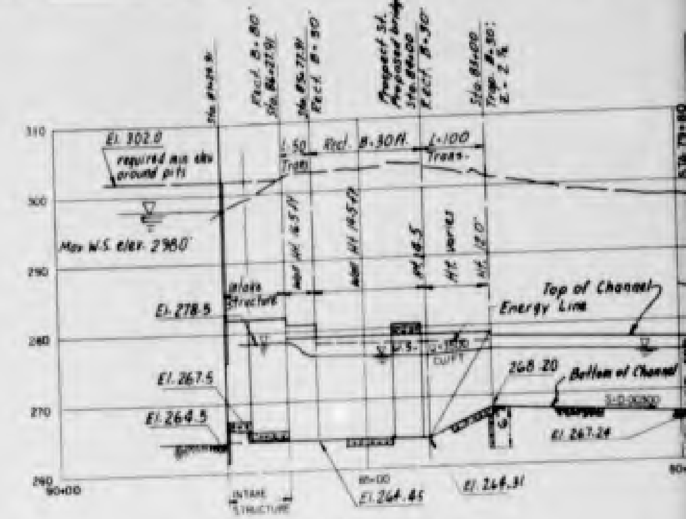


PLATE F-25

SAFETY PAYS

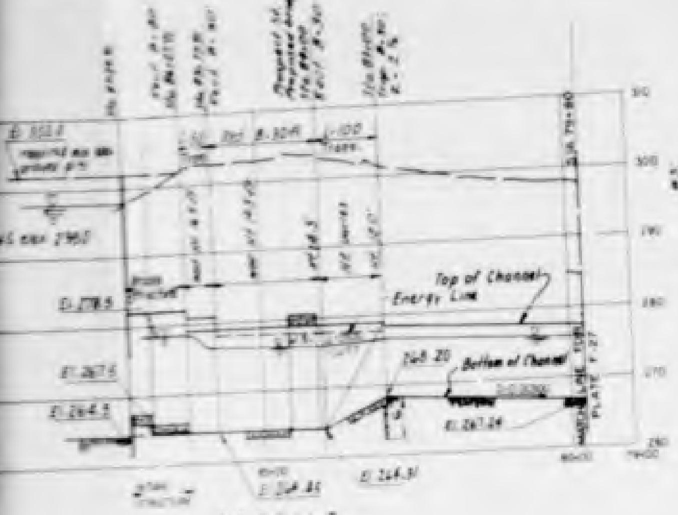
2



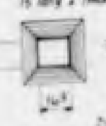
PROFILE
HORIZONTAL SCALE 1 IN = 100 FT
VERTICAL SCALE 1 IN = 10 FT

PLAN
SCALE 1 IN = 100 FT

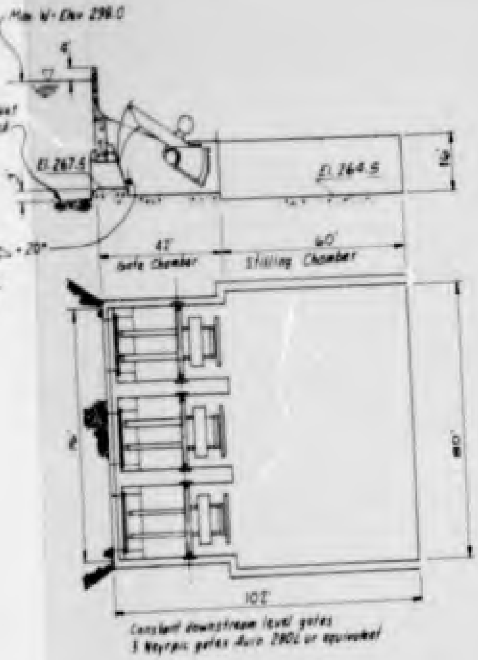
VALUE ENGINEERING PAYS



PROFILE

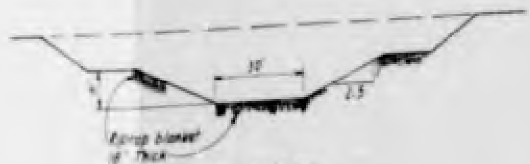


GATE OPENING
FRONT VIEW



PLAN AND PROFILE
INTAKE STRUCTURE

PLAN



TYPICAL SECTION
STA 83+00 TO STA 50+00

REVISIONS

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

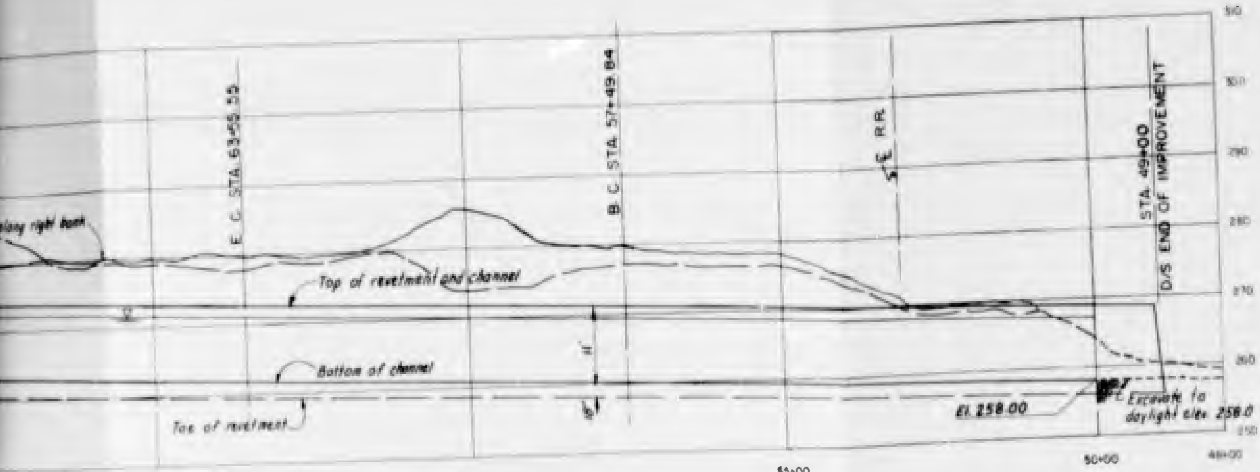
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTIAGO CREEK RESERVOIR
OUTLET STRUCTURE

PLATE F-26

SAFETY PAYS

VALUE ENGINEERING PAYS

SAFETY PAYS



HYDRAULIC ELEMENTS

Q = 3800 cfs, S = .00300, Z = 2.5, B = 30

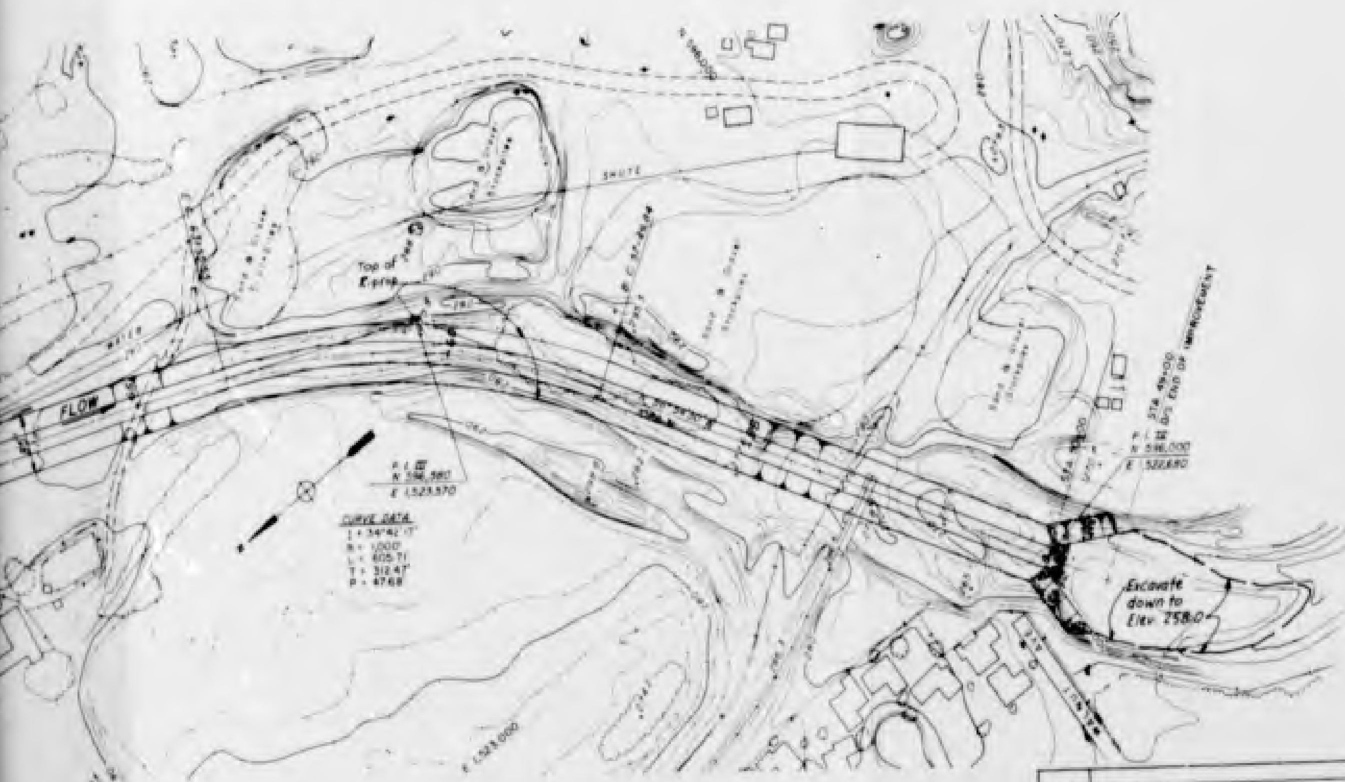
n = .035, d = 9.2 ft, vn = 7.8 ft/sec.

n = .025, dn = 7.7 ft, vn = 10.0 ft/sec

PROFILE

HORIZONTAL SCALE 1 IN = 100 FT

VERTICAL SCALE 1 IN = 10 FT



PLAN

TYPICAL SECTION STA. 50+00 TO STA. 63+00

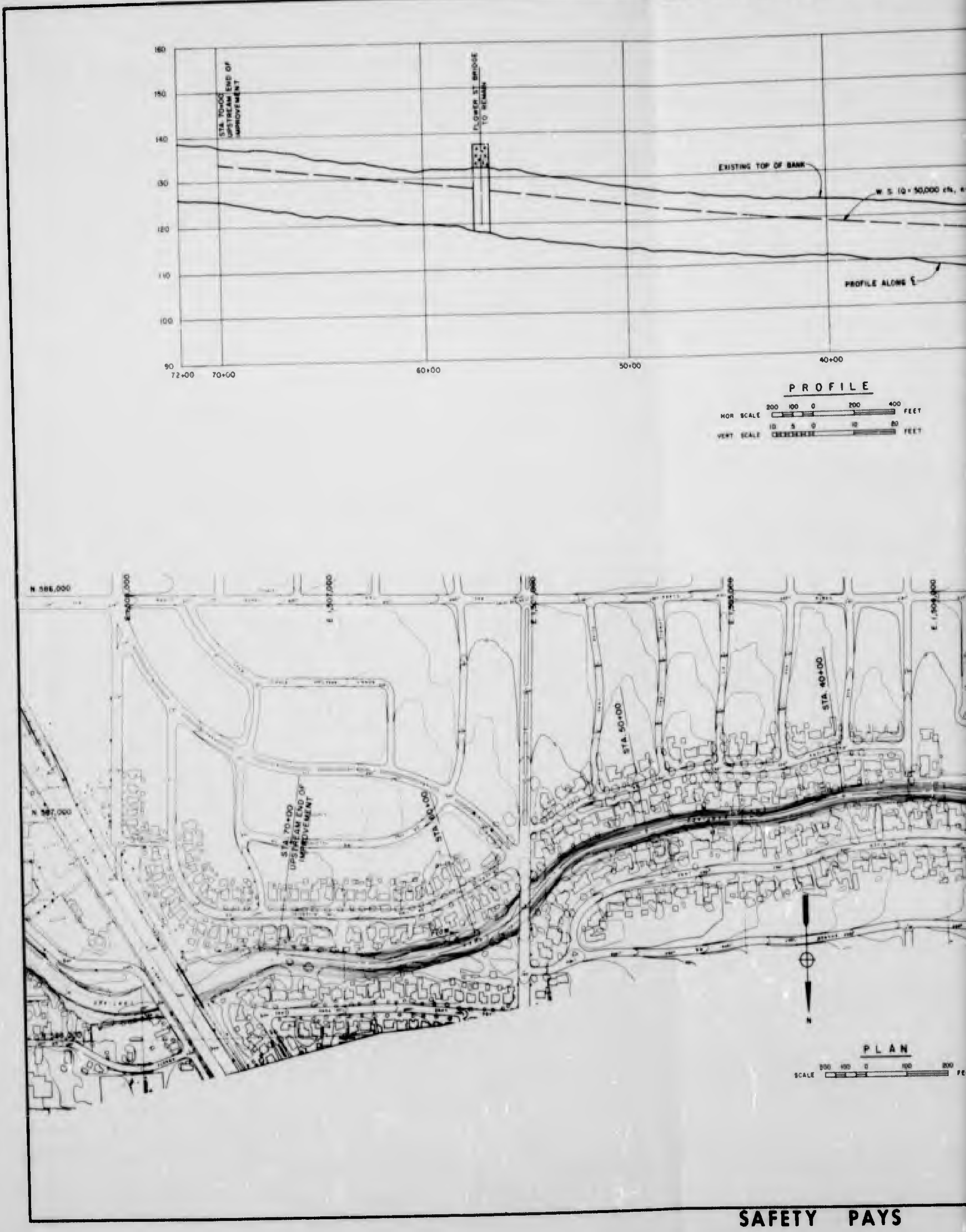
REVISIONS

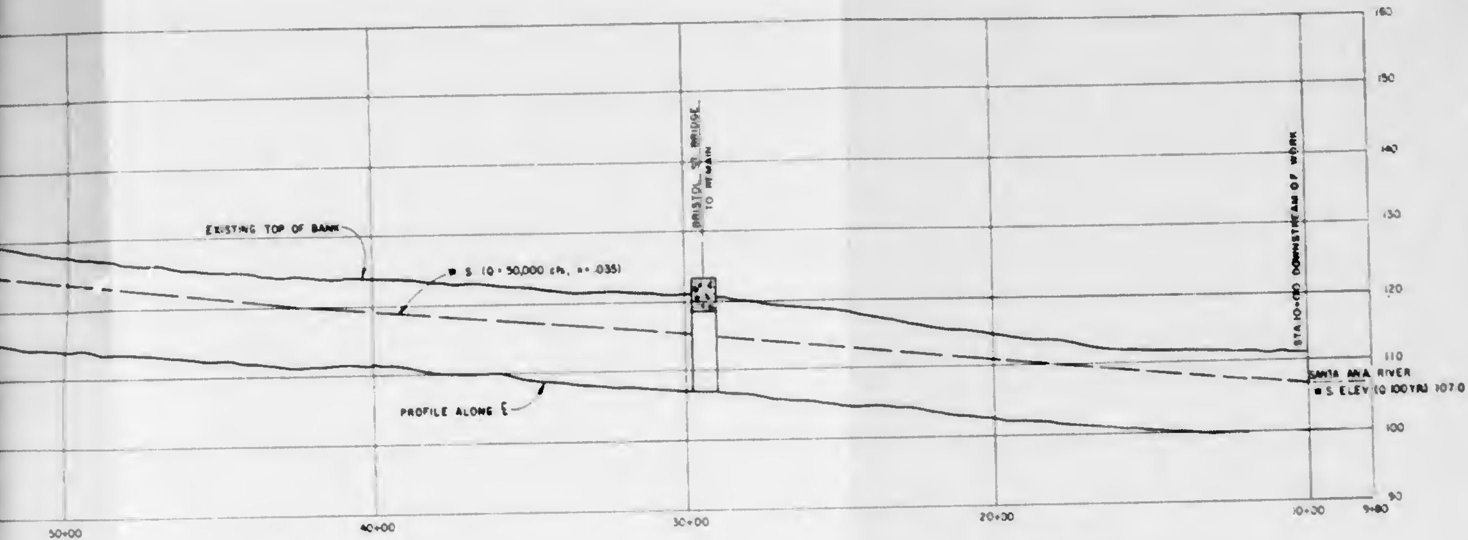
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTIAGO CREEK RESERVOIR
OUTLET CHANNEL

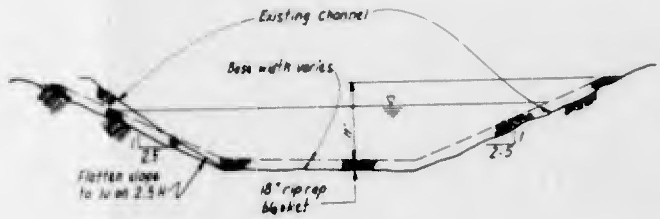
DESIGNED BY:
DRAWN BY: g. w.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW 09-____ B-____
DISTRICT FILE NO.

PLATE F-27

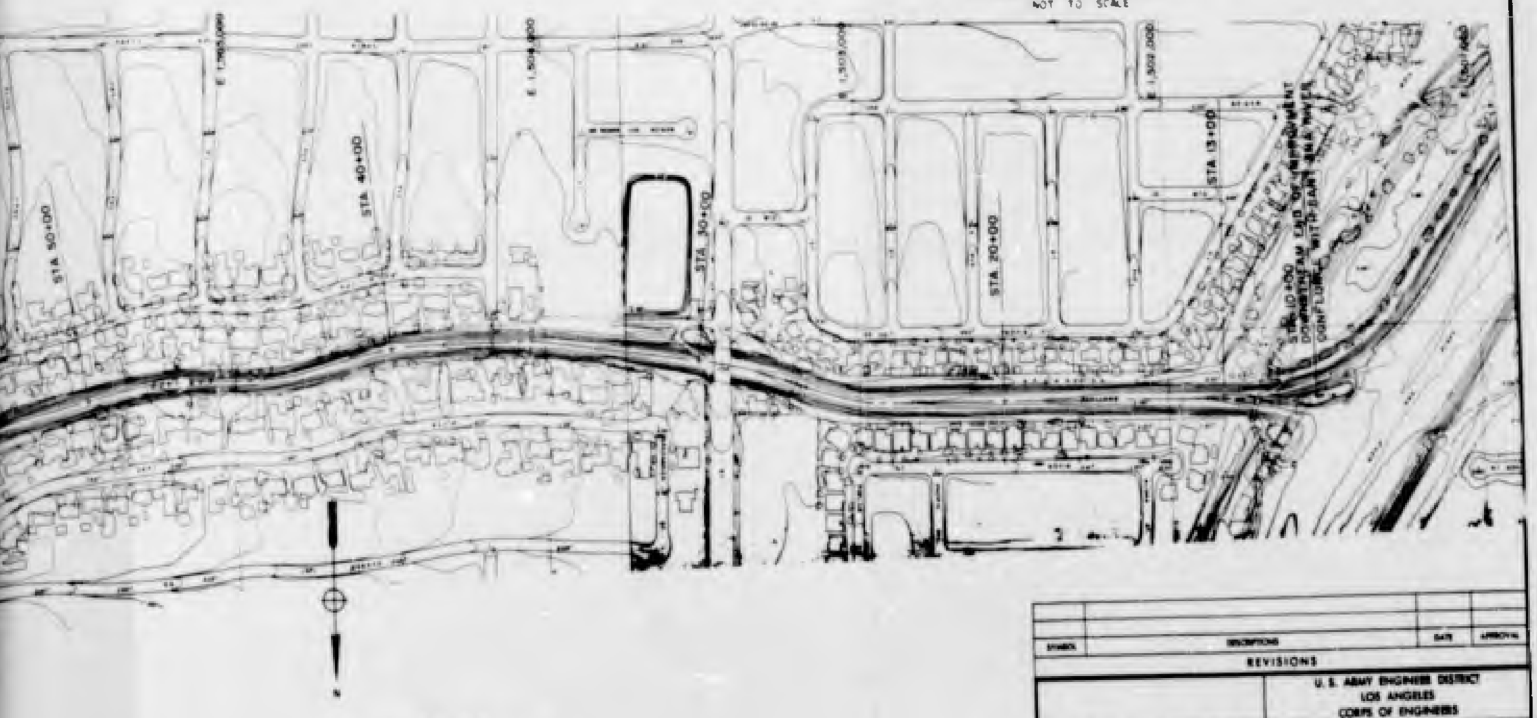

STA 70+00 UPSTREAM END OF IMPROVEMENT
FLOWER ST BRIDGE TO REMAIN
EXISTING TOP OF BANK
PROFILE ALONG ℄
PROFILE
HOR SCALE
VERT SCALE
FEET
STA 70+00 UPSTREAM END OF IMPROVEMENT
STA 60+00
STA 50+00
STA 40+00
N 586,000
N 587,000
PLAN
SCALE



PROFILE



CHANNEL SECTION
STA 10+00 to STA 70+00
NOT TO SCALE



PLAN

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

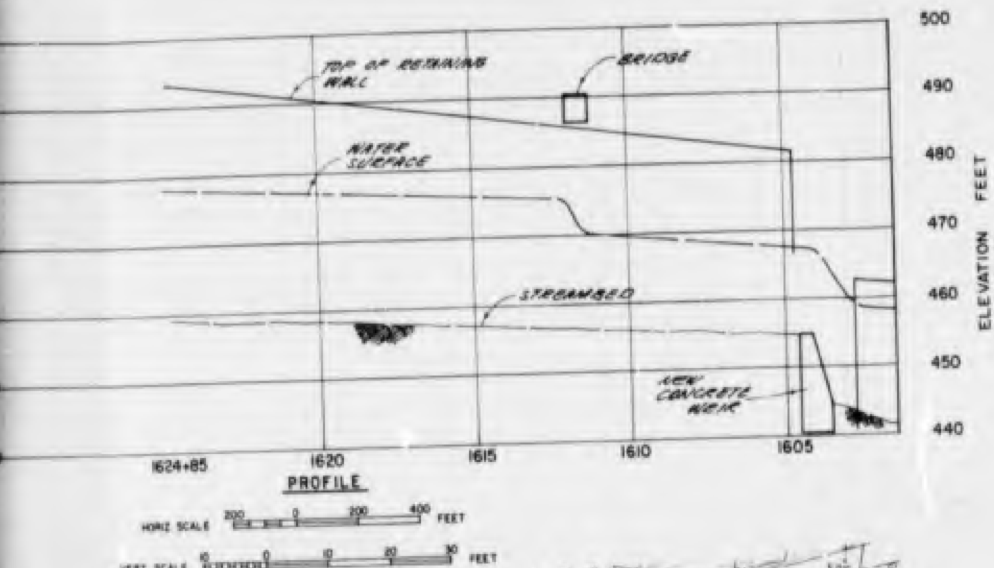
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTIAGO CREEK CHANNEL
PLAN AND PROFILE

DESIGNED BY:
DRAWN BY: J.W.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET

PLATE F-28


500
490
480
470
460
450
440
TOP OF RETAINING WALL
WATER SURFACE
1624+85
1620
1615
PROFILE
HORIZ SCALE
VERT SCALE
FEET
EXCAVATE TO ELEVATION 460
PROPOSED OUTLET STRUCTURE
PRADO DAM
PRADO DAM

VALUE ENGINEERING PAYS



INDEX TO SHEETS

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

WILLDAN ASSOCIATES

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

SANTA ANA RIVER, CALIFORNIA

PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

PLAN AND PROFILE

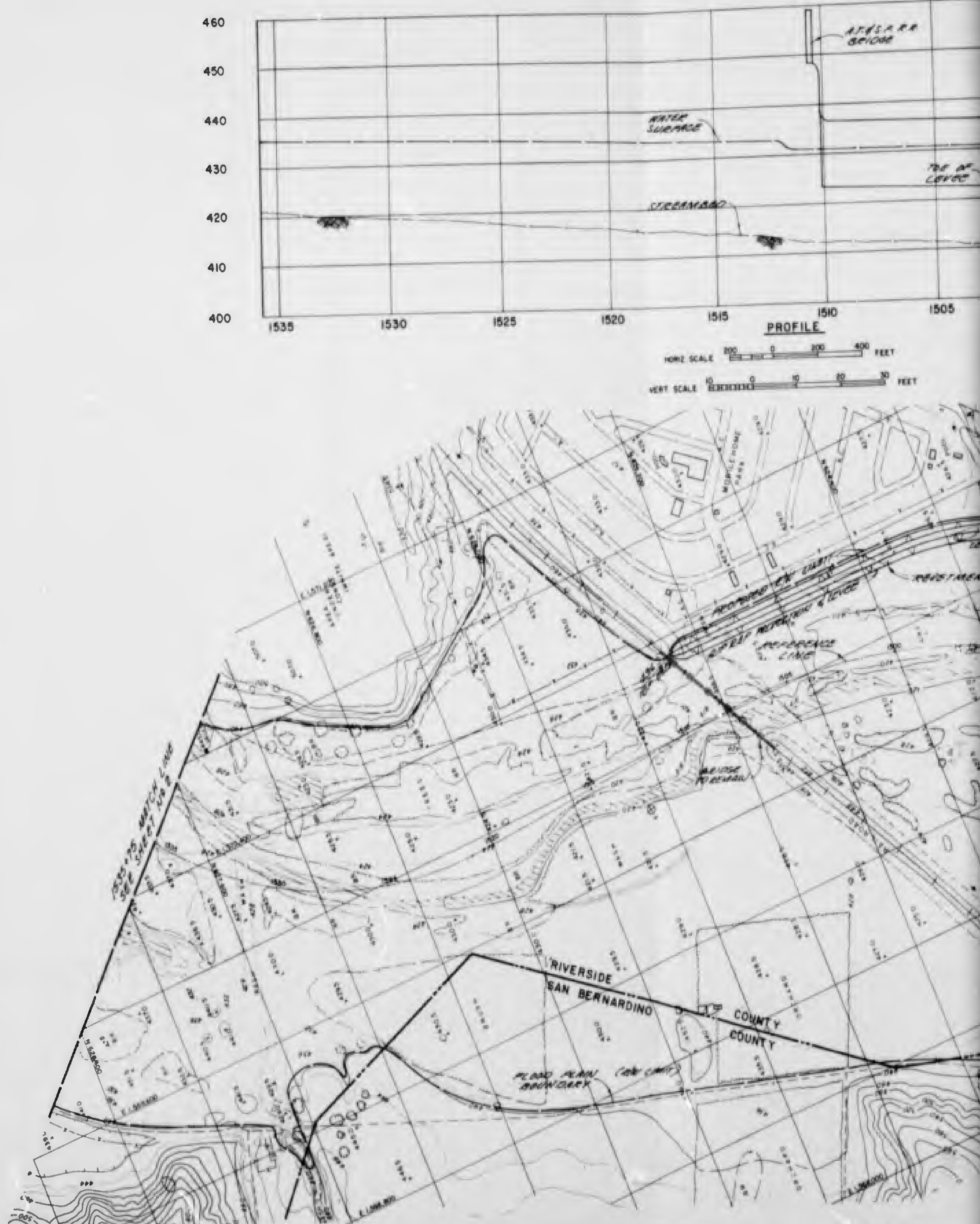
STA. 1624+85 TO STA. 1601+45

DESIGNED BY: | DRAWN BY: H.C.W. S.D.S. | CHECKED BY: | SUBMITTED BY: | DATE APPROVED: | SPEC. NO. DACW 09-___-B-___ | DISTRICT FILE NO. | SHEET 1 OF 29 SHEETS

PLATE F-29

SAFETY PAYS

2


460
450
440
430
420
410
400
AT&S.F. R.R. BRIDGE
WATER SURFACE
TOE OF LEVEE
STREAMBED
1535
1530
1525
1520
1515
1510
1505
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
MOBILE HOME PARK
REFERENCE LINE
BRIDGE TO REMAIN
RIVERSIDE
SAN BERNARDINO
COUNTY
COUNTY
FLOOD PLAIN BOUNDARY

VALUE ENGINEERING PAYS

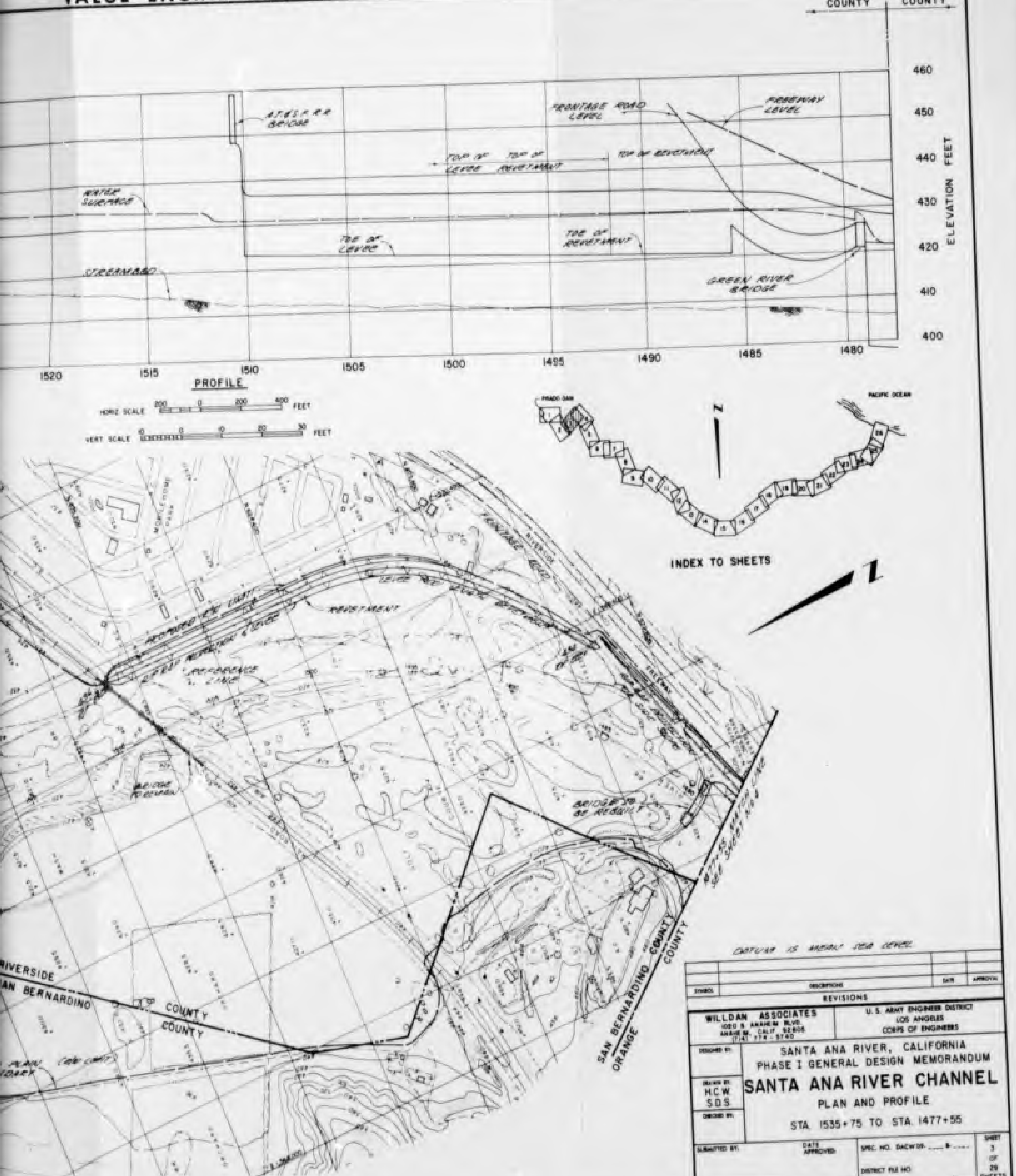
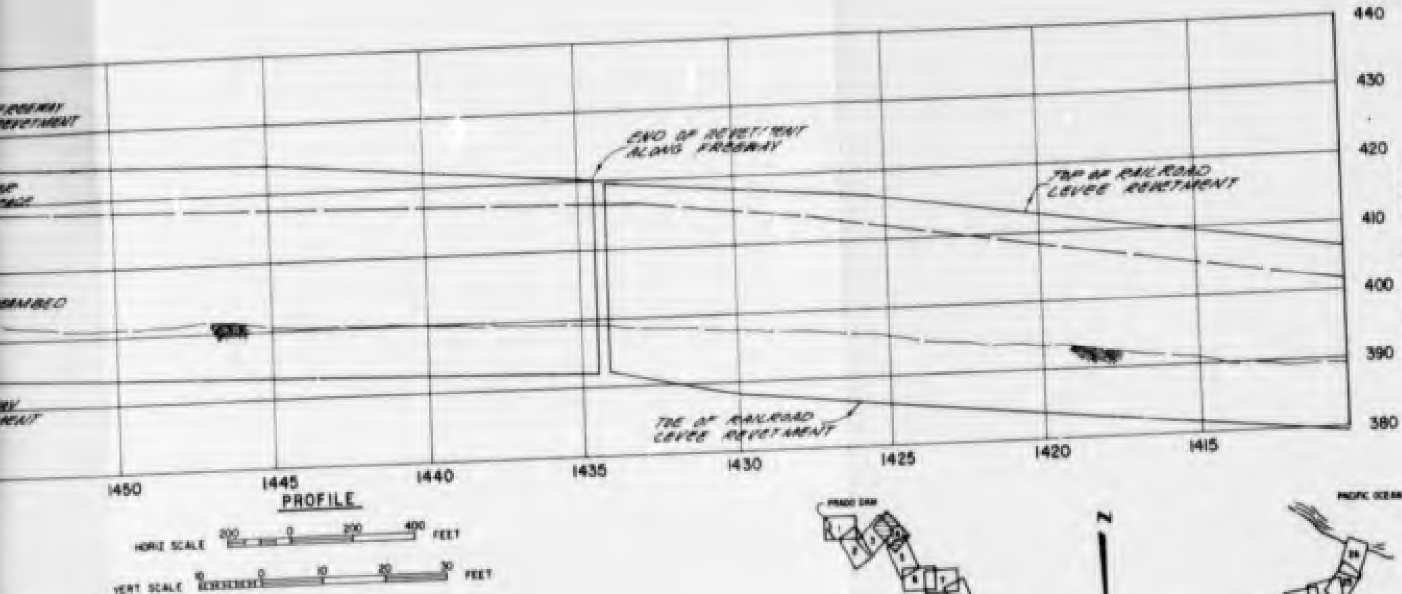


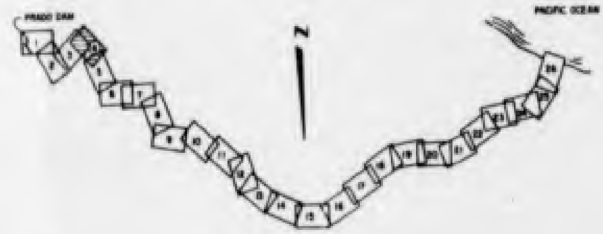
PLATE F - 31

SAFETY PAYS

2


RIVERSIDE COUNTY
ORANGE COUNTY
440
430
420
410
400
390
380
FREEWAY LEVEL
TOP OF FREEWAY LEVEE REVETMENT
WATER SURFACE
STREAMBED
TOE OF FREEWAY LEVEE REVETMENT
1475
1470
1465
1460
1455
1450
1445
1440
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
FLOOD PLAIN BOUNDARY
REFERENCE LINE
REVETMENT
FLOOD PLAIN BOUNDARY (REQD. R/W)
RIVERSIDE COUNTY
ORANGE COUNTY
SAN BERNARDINO COUNTY





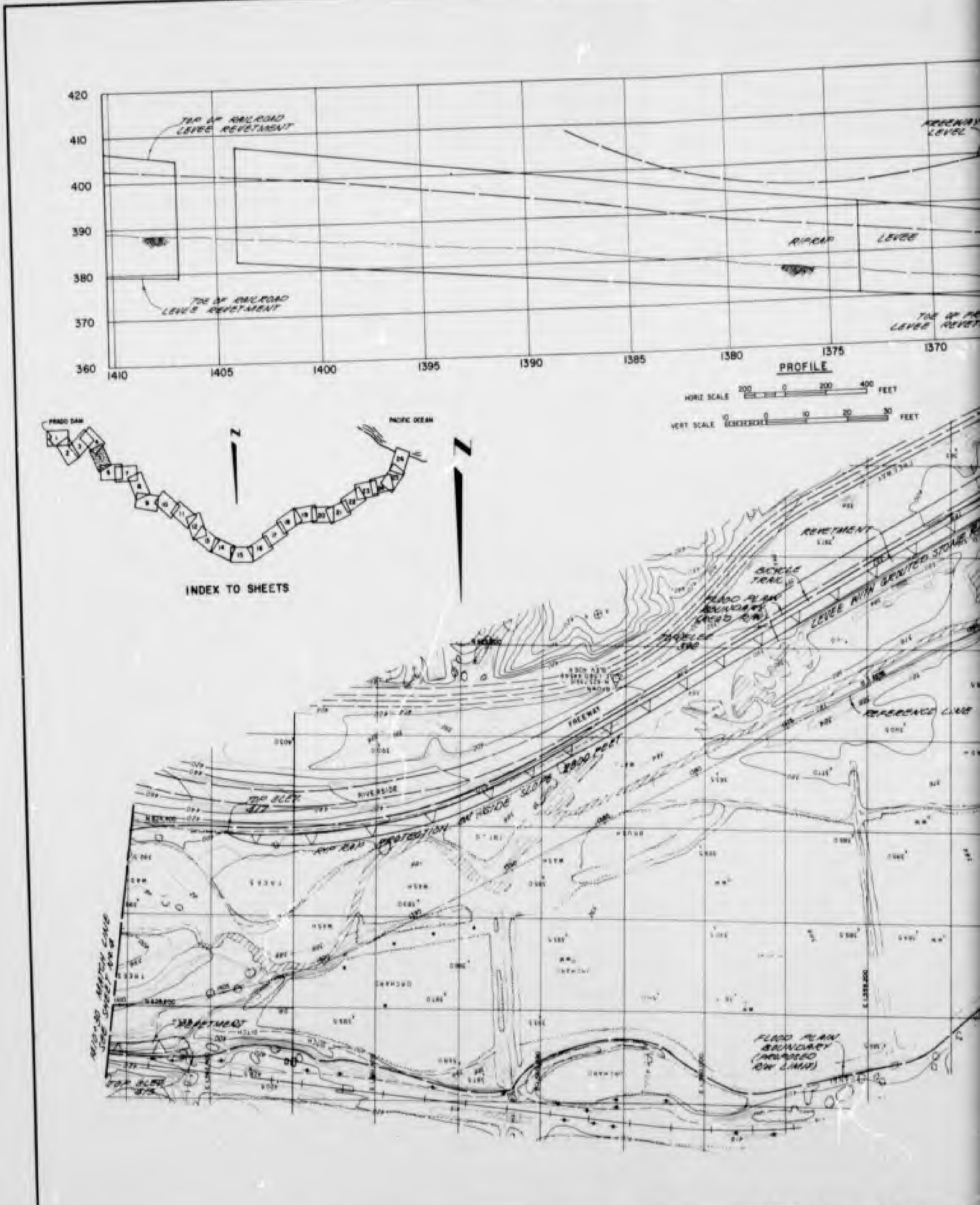
INDEX TO SHEETS

DATUM IS MEAN SEA LEVEL.

SYMBOL	DESCRIPTION	DATE	APPROVAL
REVISIONS			

WILLDAN ASSOCIATES 1060 S. ANAHEIM BLVD. ANAHEIM, CALIF. 92805 (714) 774-5740	U. S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY:	SANTA ANA RIVER, CALIFORNIA PHASE I GENERAL DESIGN MEMORANDUM
DRAWN BY: H.C.W. S.D.S.	SANTA ANA RIVER CHANNEL PLAN AND PROFILE
CHECKED BY:	STA. 1477+55 TO STA. 1410+30
SUBMITTED BY: DATE APPROVED:	SPEC. NO. DACW09-___-B-___ DISTRICT FILE NO. — SHEET 4 OF 29 SHEETS

2


420
410
400
390
380
370
360
TOP OF RAILROAD LEVEE REVETMENT
TOE OF RAILROAD LEVEE REVETMENT
RIPRAP
LEVEE
1410
1405
1400
1395
1390
1385
1380
1375
1370
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
PRADO DAM
PACIFIC OCEAN
INDEX TO SHEETS
REVETMENT
BICYCLE TRAIL
FREEWAY
RIVERSIDE
REFERENCE LINE
FLOOD PLAIN BOUNDARY (PROPOSED R/W LIMIT)

VALUE ENGINEERING PAYS

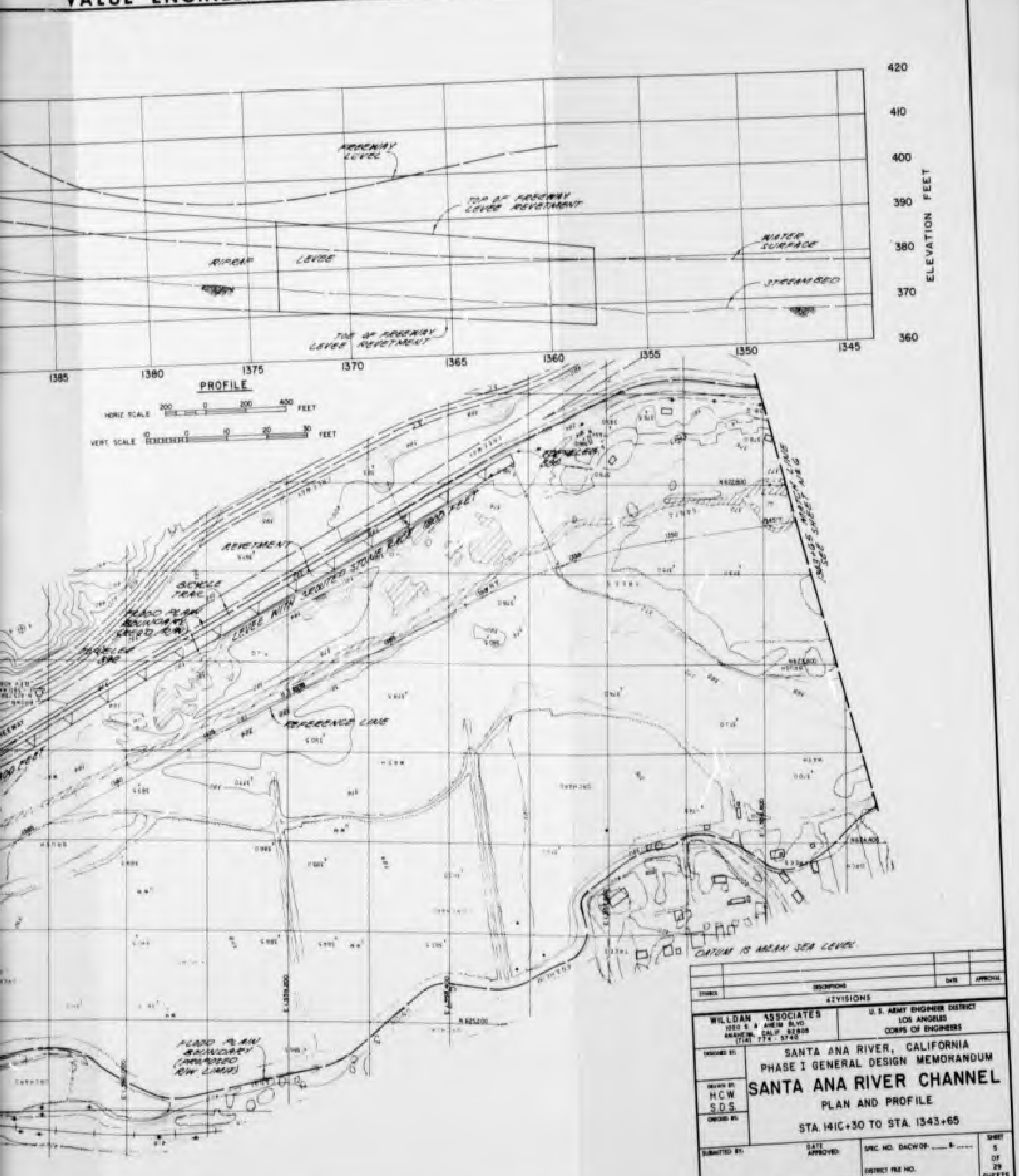
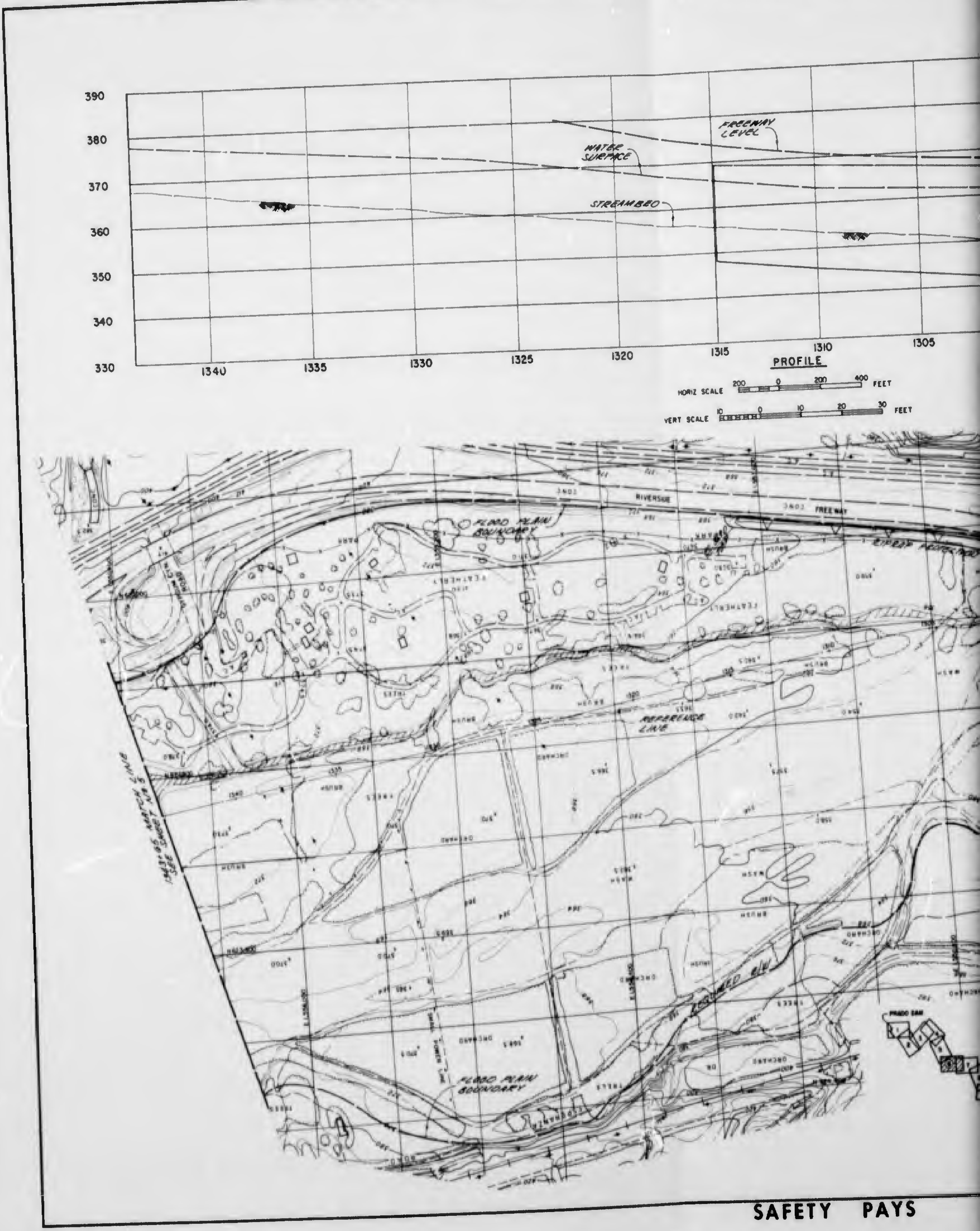


PLATE F-33

SAFETY PAYS

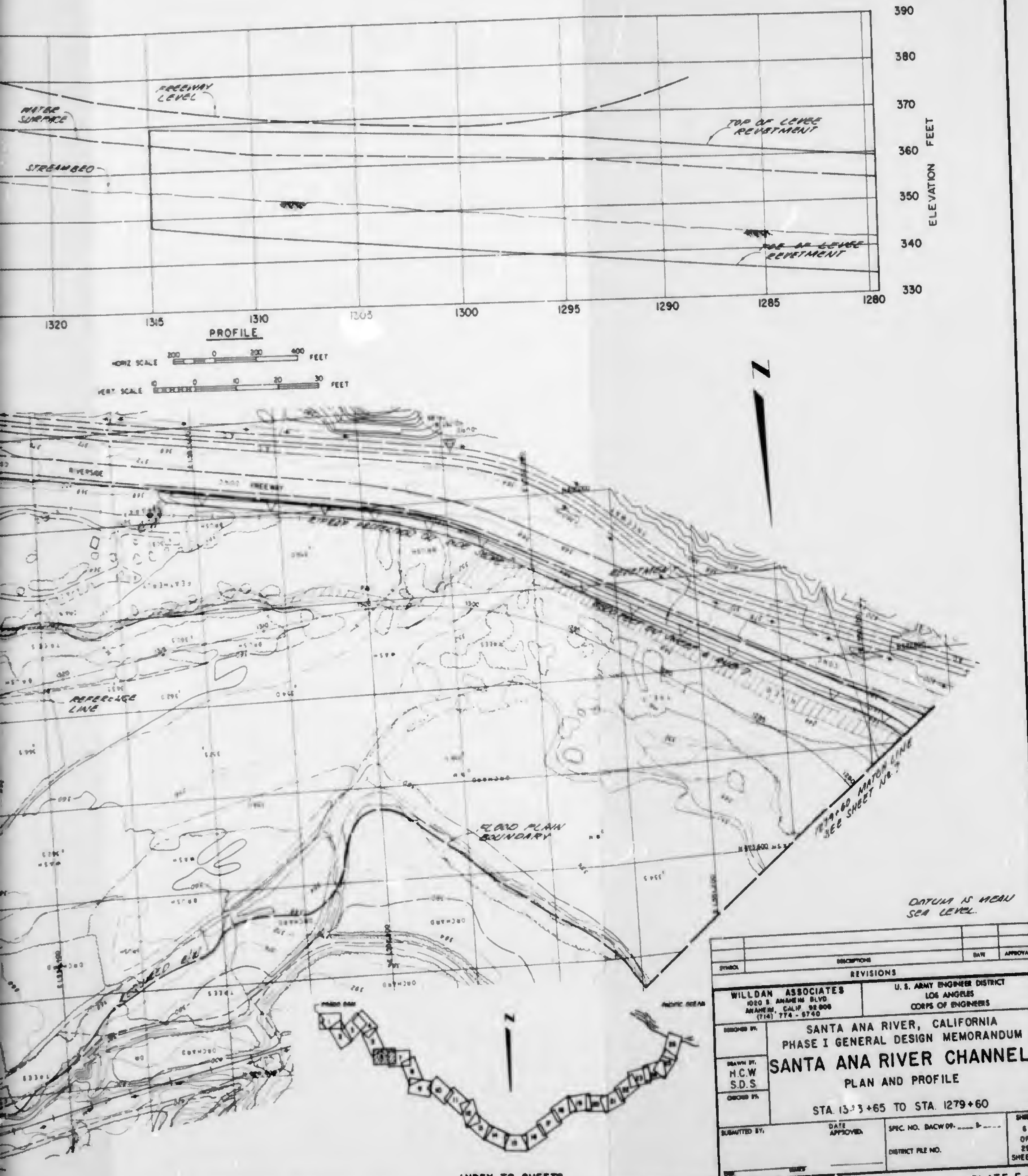
2

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD
ANAHEIM, CALIF. 92805
(714) 774-5740

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL
PLAN AND PROFILE

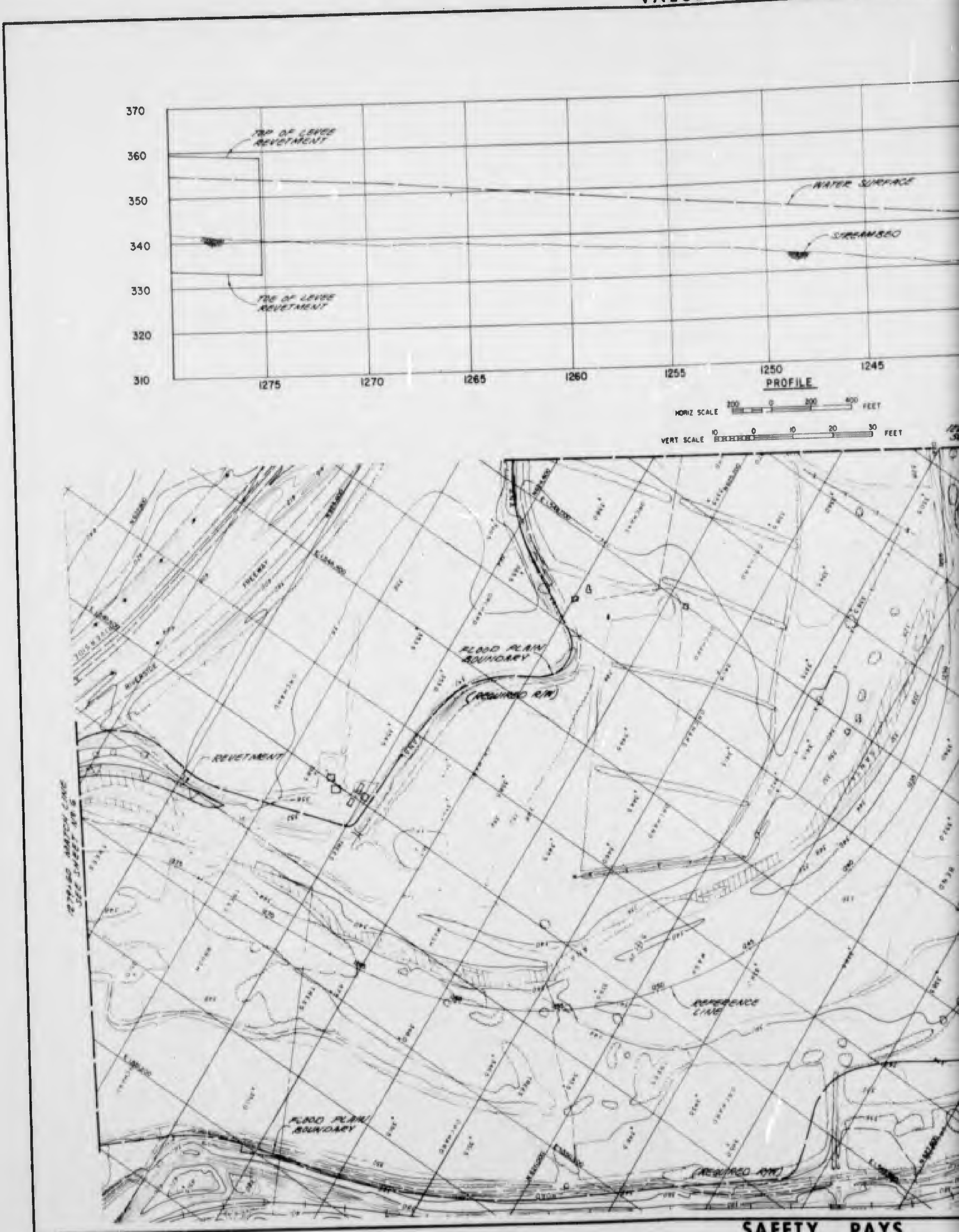
STA. 13?3+65 TO STA. 1279+60

DESIGNED BY:
DRAWN BY: H.C.W S.D.S
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 6 OF 29 SHEETS

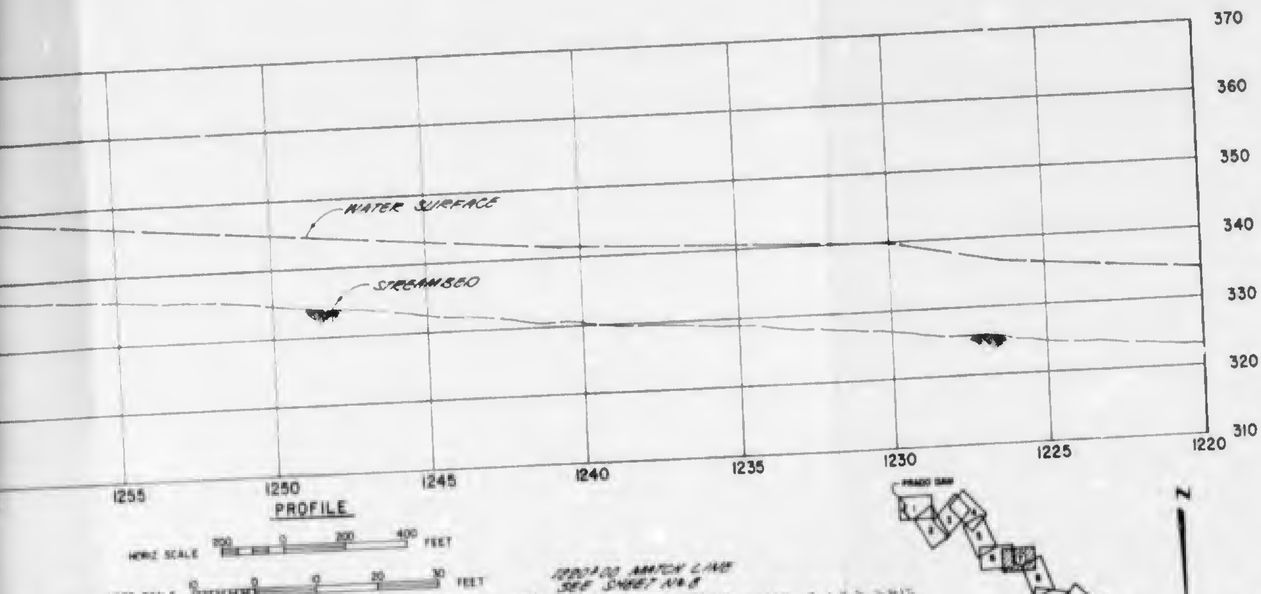
PLATE F-34

SAFETY PAYS

2


370
360
350
340
330
320
310
1275
1270
1265
1260
1255
1250
1245
TOP OF LEVEE REVETMENT
TOE OF LEVEE REVETMENT
WATER SURFACE
STREAMBED
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
FLOOD PLAIN BOUNDARY
(REQUIRED R/W)
REVETMENT
REFERENCE LINE
FREEWAY

VALUE ENGINEERING PAYS



INDEX TO SHEETS

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD.
ANAHEIM, CALIF. 92805
(714) 774-5740

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

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTA ANA RIVER CHANNEL
PLAN AND PROFILE
STA 1279+60 TO STA 1220+00

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09- ___ - B - ___ DISTRICT FILE NO.
SHEET 7 OF 29 SHEETS

PLATE F-35

SAFETY PAYS

2

VALUE ENGINEERING PAYS

SAFETY PAYS

VALUE ENGINEERING PAYS

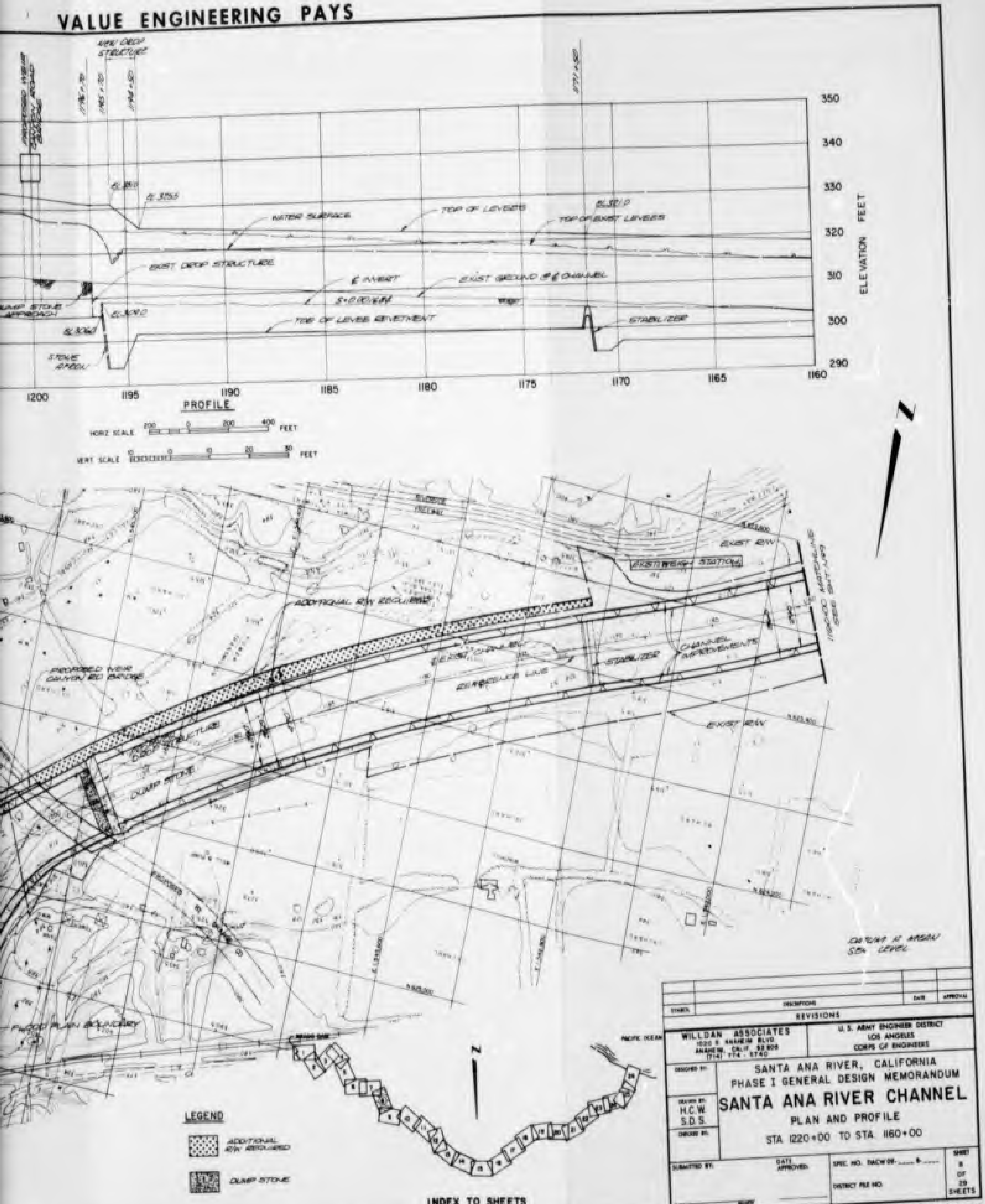
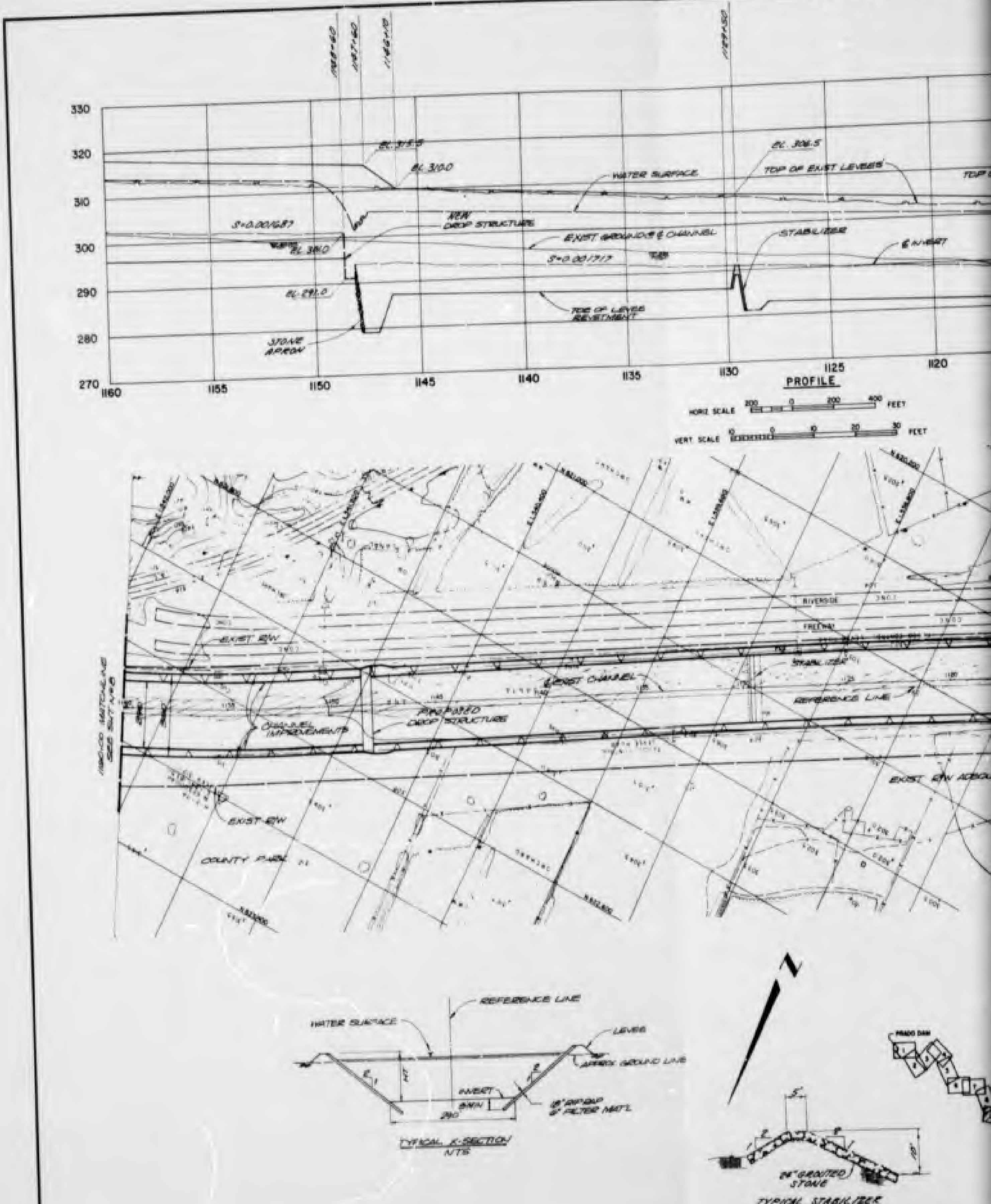


PLATE F-36

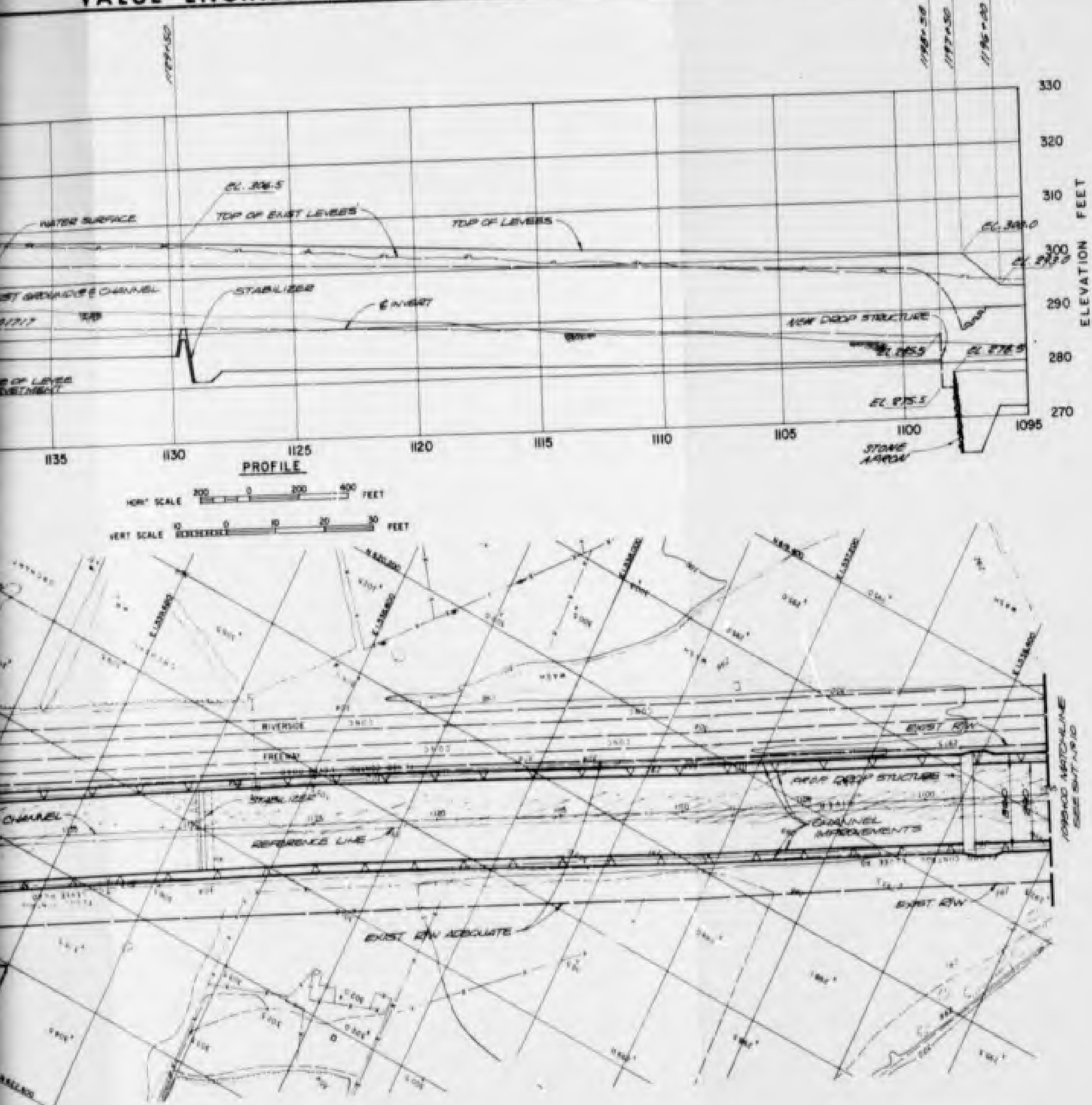
SAFETY PAYS

VALUE ENGINEERING PAYS



SAFETY PAYS

VALUE ENGINEERING PAYS



DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

WILLDAN ASSOCIATES
1000 S. ANAHEIM BLVD.
ANAHEIM, CALIF. 92805
(714) 774-5740

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

DESIGNED BY:

DRAWN BY: H.C.W. S.D.S.

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

PLAN AND PROFILE

STA. 1160+00 TO STA. 1095+00

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____ B ____

DISTRICT FILE NO.

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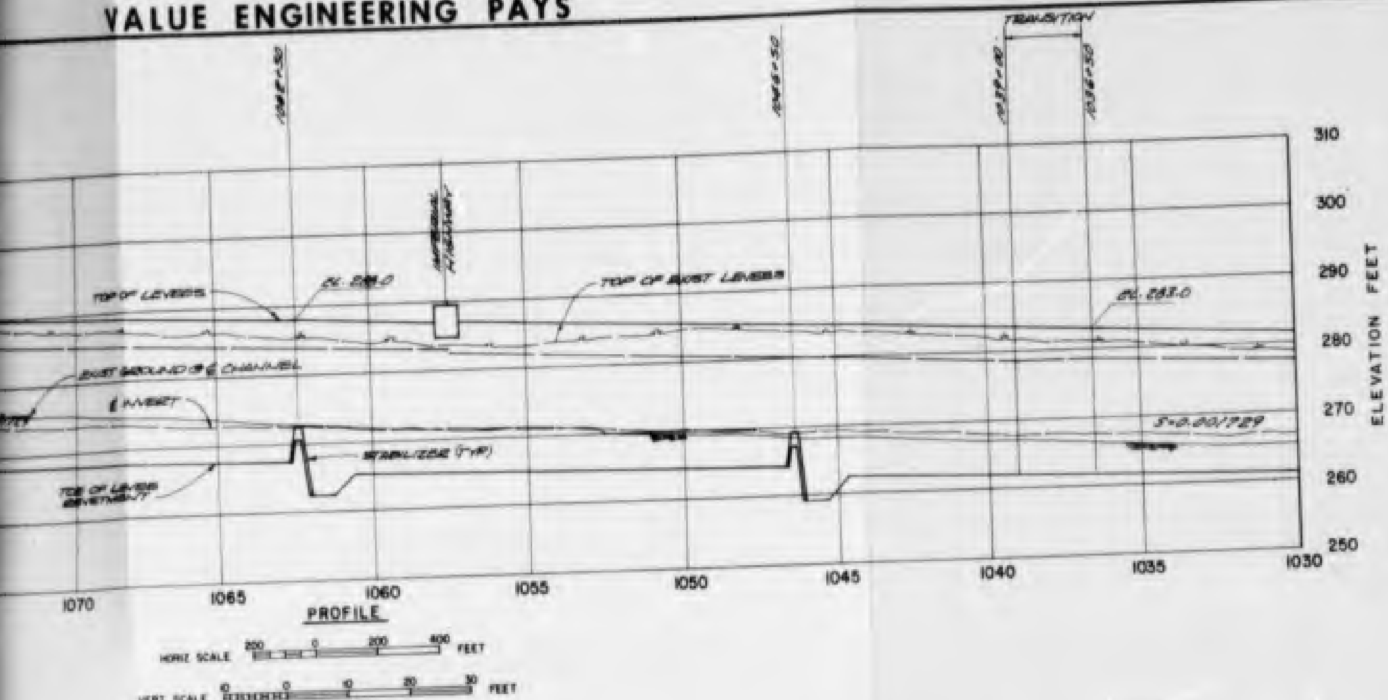
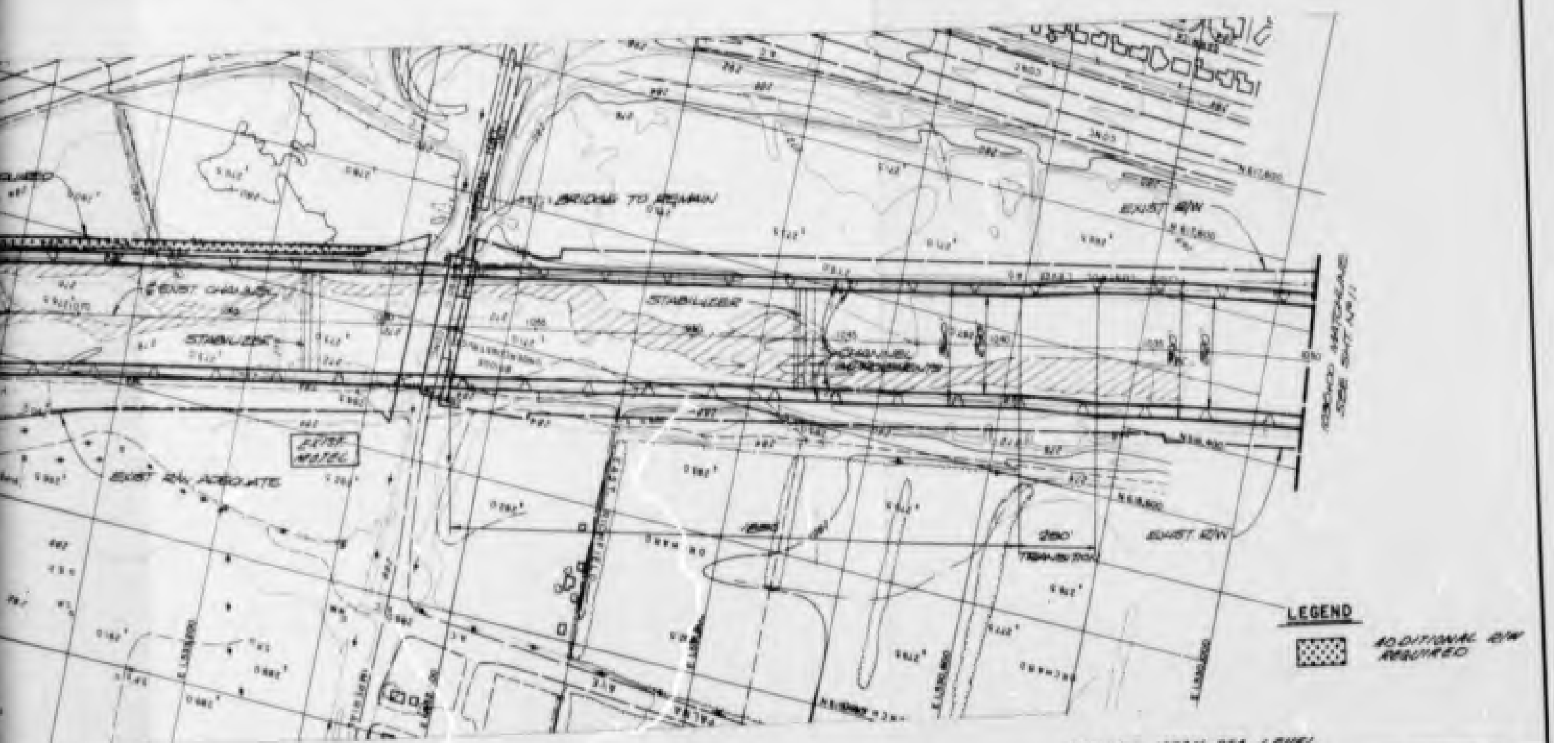
PLATE F-37

SAFETY PAYS

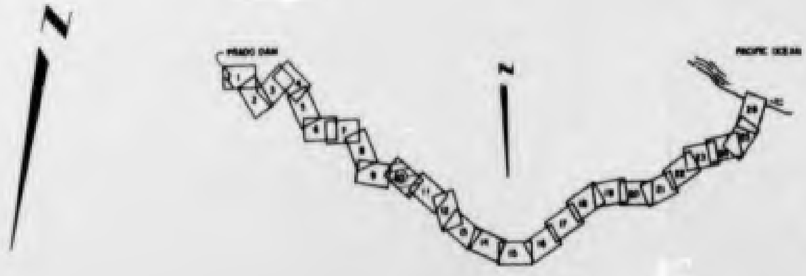
2


WATER SURFACE
TOP OF LEVEES
EXIST GROUND @ ℄ CHANNEL
℄ INVERT
STABILIZE (TYP)
TOP OF LEVEE REVETMENT
310
300
290
280
270
260
250
1095
1090
1085
1080
1075
1070
1065
1060
1055
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
ADDITIONAL R/W REQUIRED
EXIST R/W
REVETMENT LINE
STABILIZE
CHANNEL IMPROVEMENTS
℄ EXIST CHANNEL
EXIST R/W ADEQUATE
EXIST MOTEL
℄ CHANNEL
WATER SURFACE
LEVEE
APPROX. GROUND LINE
INVERT
5' MIN
290' TO 320'
18" RIP RAP
6" FILTER MAT'L
TYPICAL X-SECTION
N.T.S.





DATUM IS MEAN SEA LEVEL



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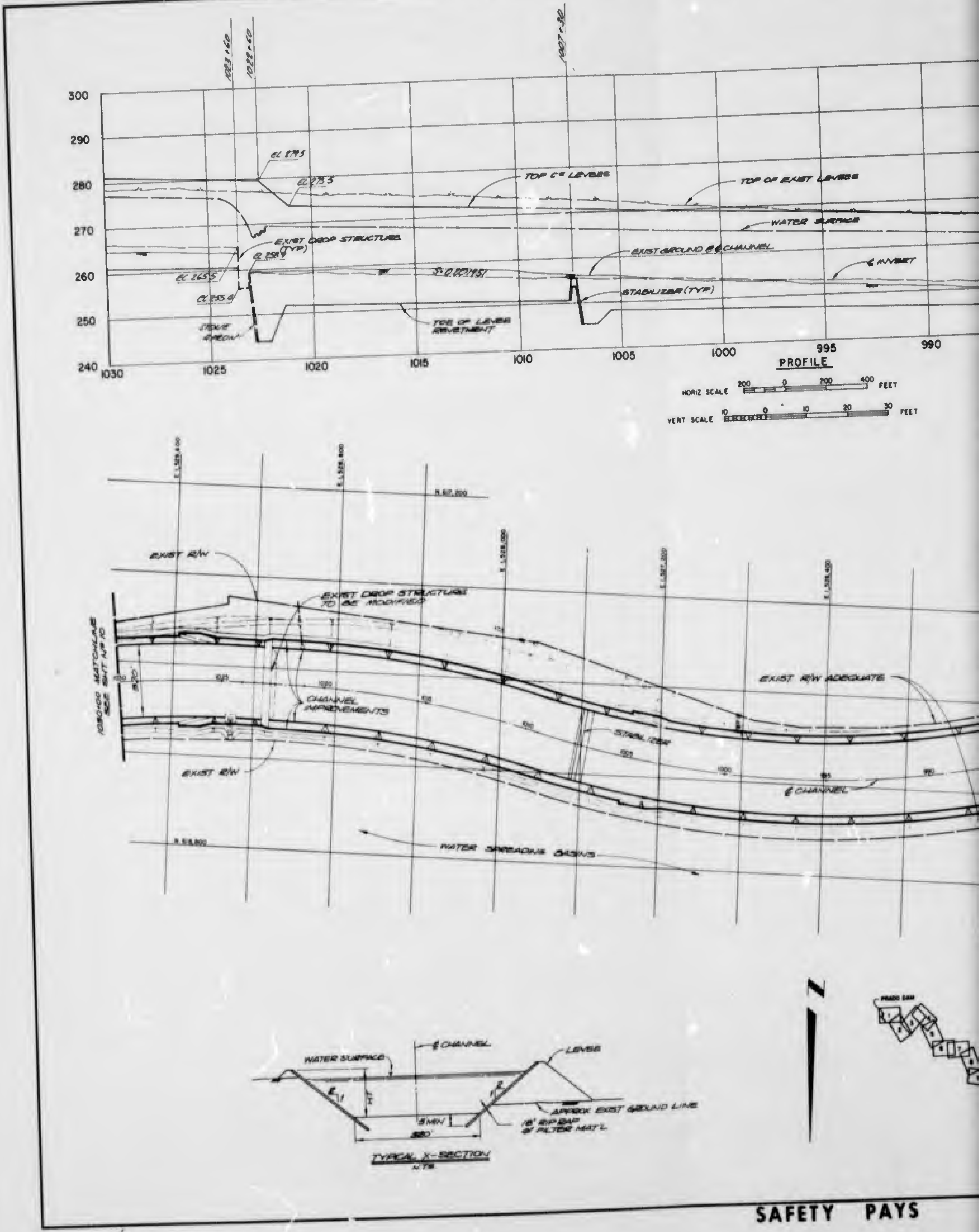
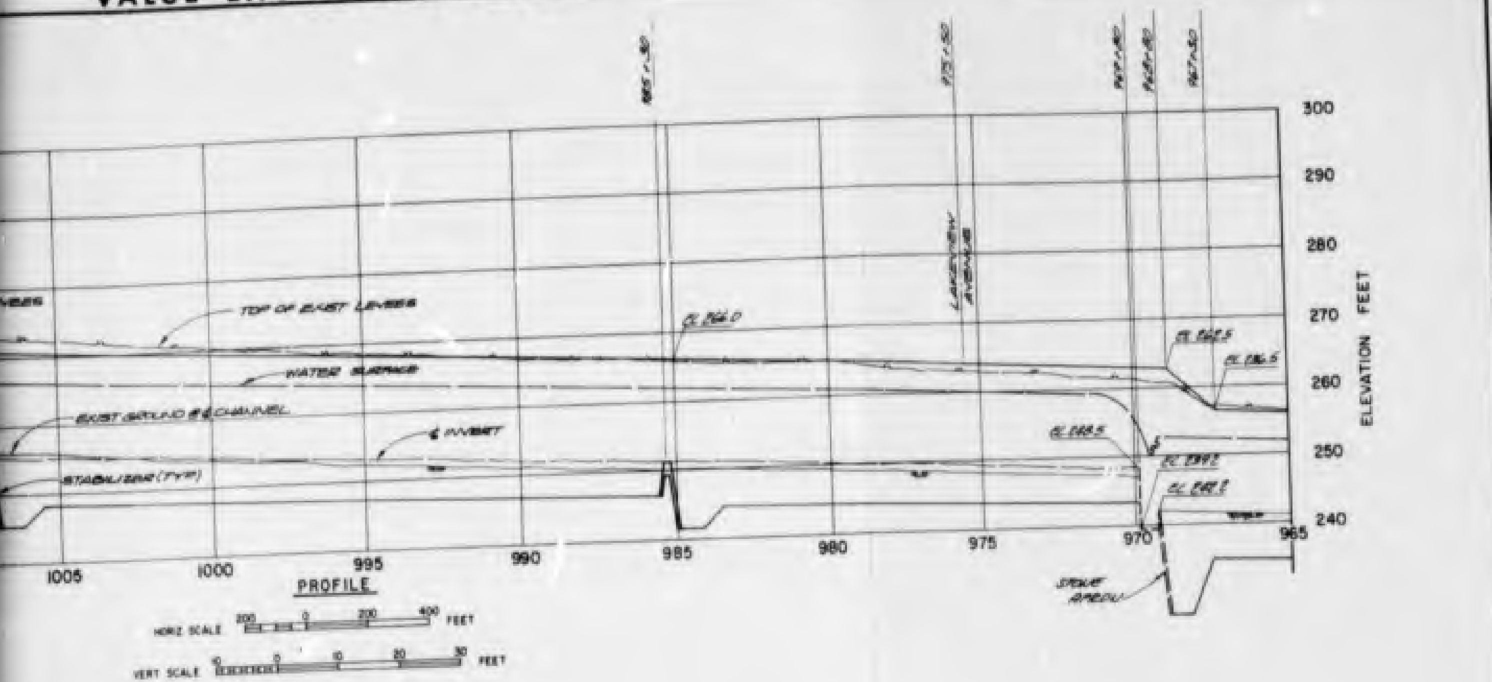
	REVISIONS		

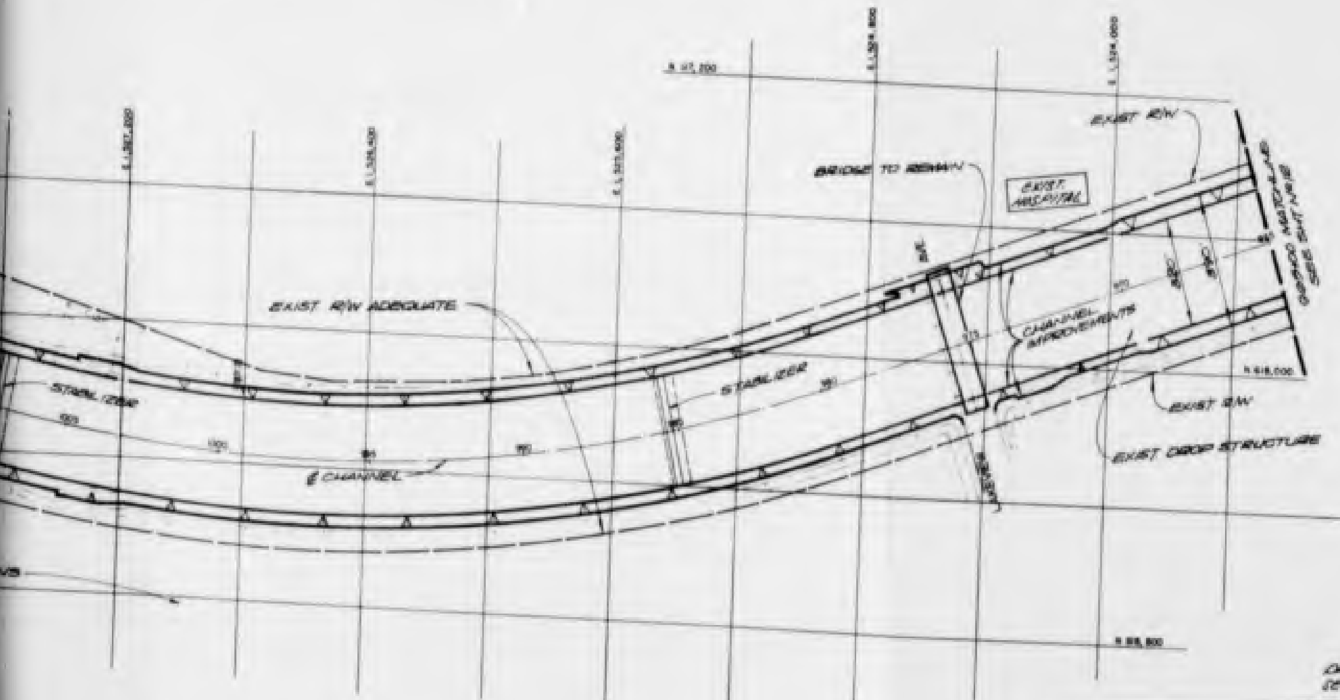
WILLDAN ASSOCIATES	U. S. ARMY ENGINEER DISTRICT, LOS ANGELES, CORPS OF ENGINEERS
DESIGNED BY:	SANTA ANA RIVER, CALIFORNIA PHASE I GENERAL DESIGN MEMORANDUM
DRAWN BY: H.C.W. S.D.S.	SANTA ANA RIVER CHANNEL PLAN AND PROFILE
CHECK'D BY:	STA. 1095+00 TO STA. 1030+00
SUBMITTED BY:	DATE APPROVED: SPEC. NO. DACW09- ... DISTRICT FILE NO. SHEET 10 OF 29 SHEETS

PLATE F-38

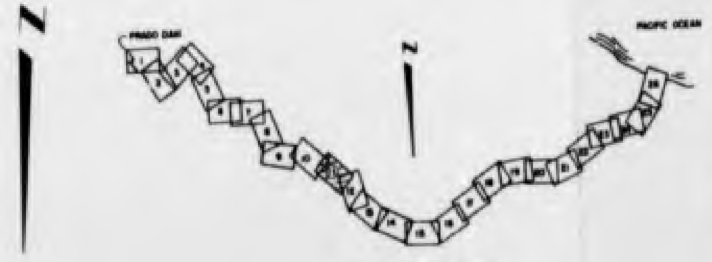

1023+60
1022+60
1007+30
EL 279.5
EL 273.5
TOP C= LEVEES
TOP OF EXIST LEVEES
WATER SURFACE
EXIST DROP STRUCTURE (TYP)
EXIST GROUND @ ¢ CHANNEL
¢ INVERT
STABILIZER (TYP)
TOE OF LEVEE REVETMENT
STONE APRON
EL 265.5
PROFILE
HORIZ SCALE
VERT SCALE
FEET
EXIST R/W
EXIST DROP STRUCTURE TO BE MODIFIED
CHANNEL IMPROVEMENTS
EXIST R/W ADEQUATE
STABILIZER
¢ CHANNEL
WATER SPREADING BASINS
WATER SURFACE
¢ CHANNEL
LEVEE
APPROX EXIST GROUND LINE
5 MIN
TYPICAL X-SECTION
NTS
PRADO DAM

VALUE ENGINEERING PAYS





DATA IS MEAN SEA LEVEL



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REVISIONS

WILLDAN ASSOCIATES
U. S. ARMY ENGINEER DISTRICT, LOS ANGELES, CORPS OF ENGINEERS

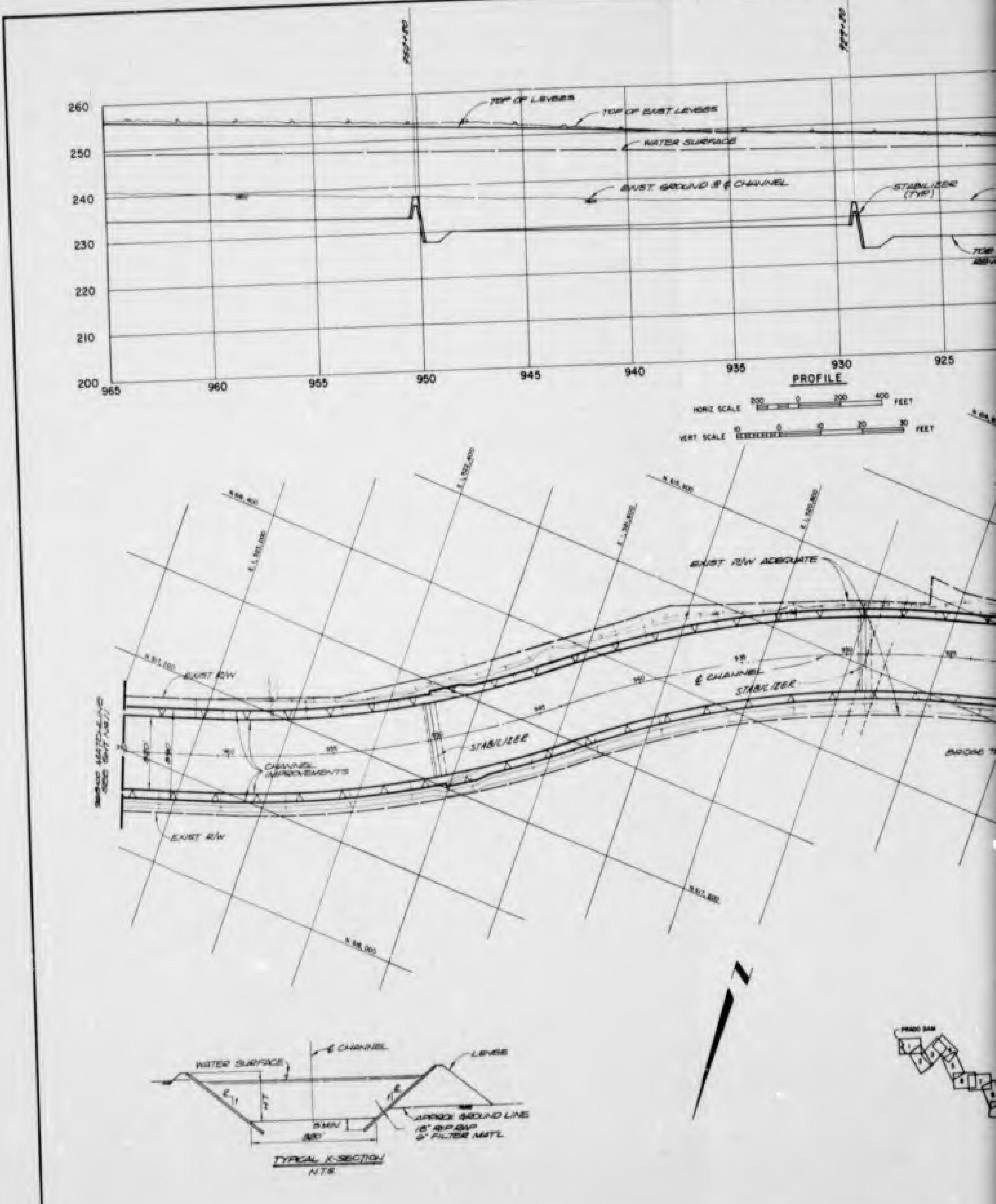
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTA ANA RIVER CHANNEL
PLAN AND PROFILE
STA. 1030+00 TO STA. 965+00

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 11 OF 29 SHEETS

PLATE F-39

SAFETY PAYS

2


TOP OF LEVEES
TOP OF EXIST LEVEES
WATER SURFACE
EXIST. GROUND @ ℄ CHANNEL
STABILIZER (TYP)
260
250
240
230
220
210
200
965
960
955
950
945
940
935
930
925
PROFILE
HORIZ SCALE
FEET
VERT. SCALE
FEET
EXIST. R/W ADEQUATE
℄ CHANNEL
STABILIZER
EXIST R/W
STABILIZER
CHANNEL IMPROVEMENTS
EXIST R/W
WATER SURFACE
℄ CHANNEL
LEVEE
APPROX GROUND LINE
18" RIP RAP
6" FILTER MAT'L
TYPICAL X-SECTION
N.T.S.
PRADO DAM

VALUE ENGINEERING PAYS

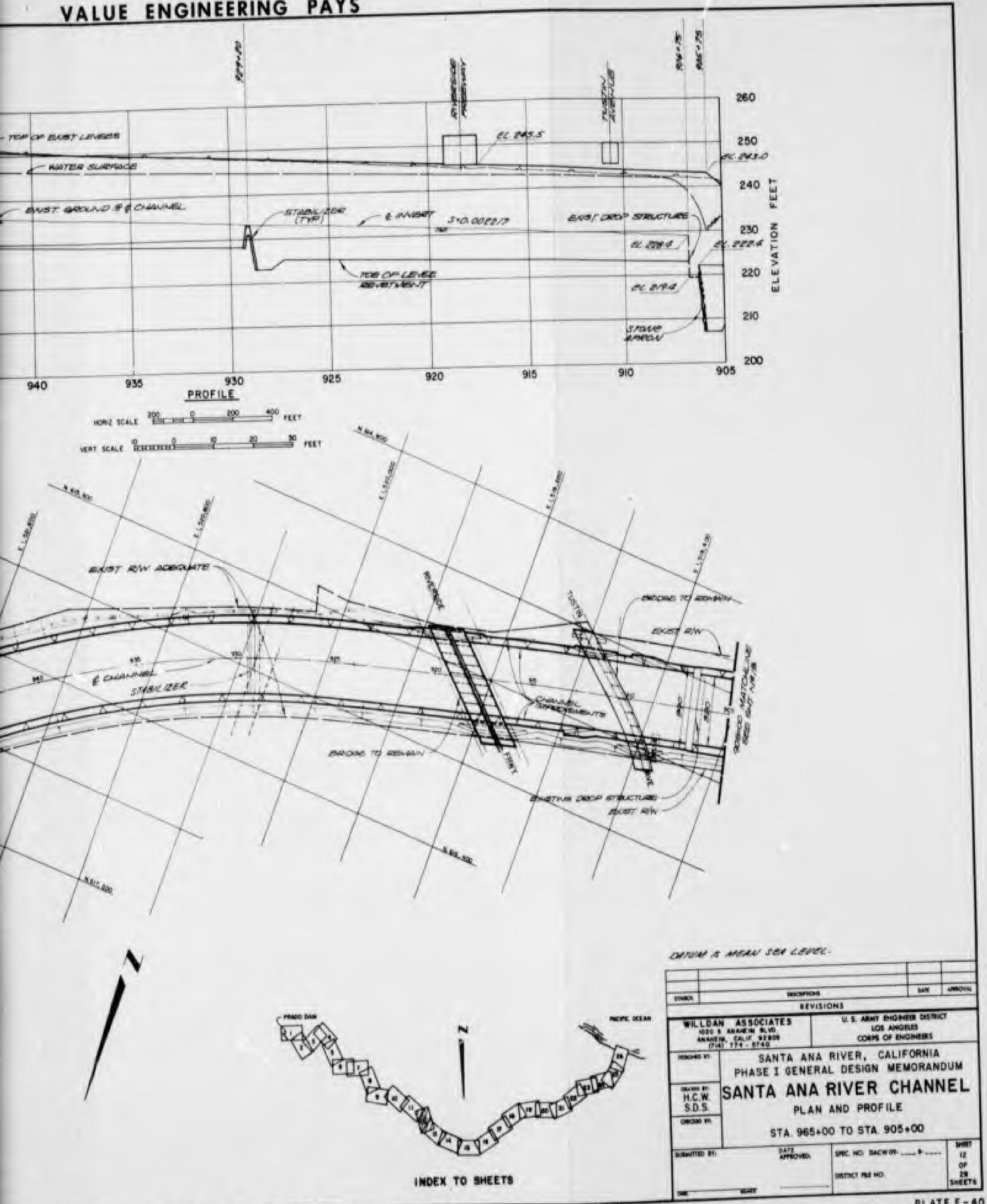
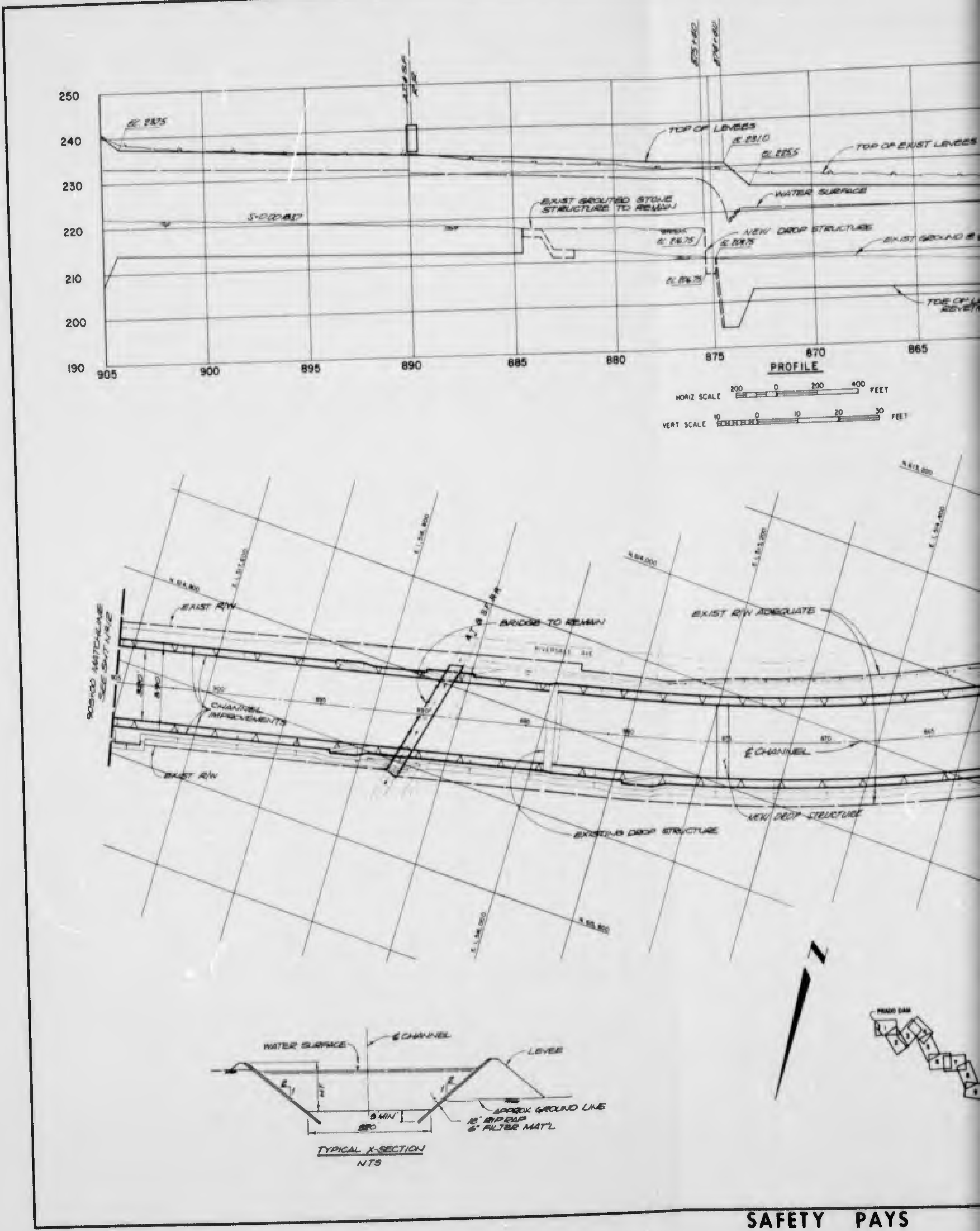


PLATE F-40

SAFETY PAYS

2


250
240
230
220
210
200
190
905
900
895
890
885
880
875
870
865
TOP OF LEVEES
TOP OF EXIST LEVEES
WATER SURFACE
EXIST GROUTED STONE STRUCTURE TO REMAIN
NEW DROP STRUCTURE
PROFILE
HORIZ SCALE
200
0
200
400
FEET
VERT SCALE
10
0
10
20
30
FEET
EXIST R/W
BRIDGE TO REMAIN
EXIST R/W ADEQUATE
CHANNEL IMPROVEMENTS
€ CHANNEL
EXIST R/W
NEW DROP STRUCTURE
EXISTING DROP STRUCTURE
PRADO DAM
WATER SURFACE
€ CHANNEL
LEVEE
APPROX GROUND LINE
18" RIP RAP
6" FILTER MAT'L
TYPICAL X-SECTION
NTS

VALUE ENGINEERING PAYS

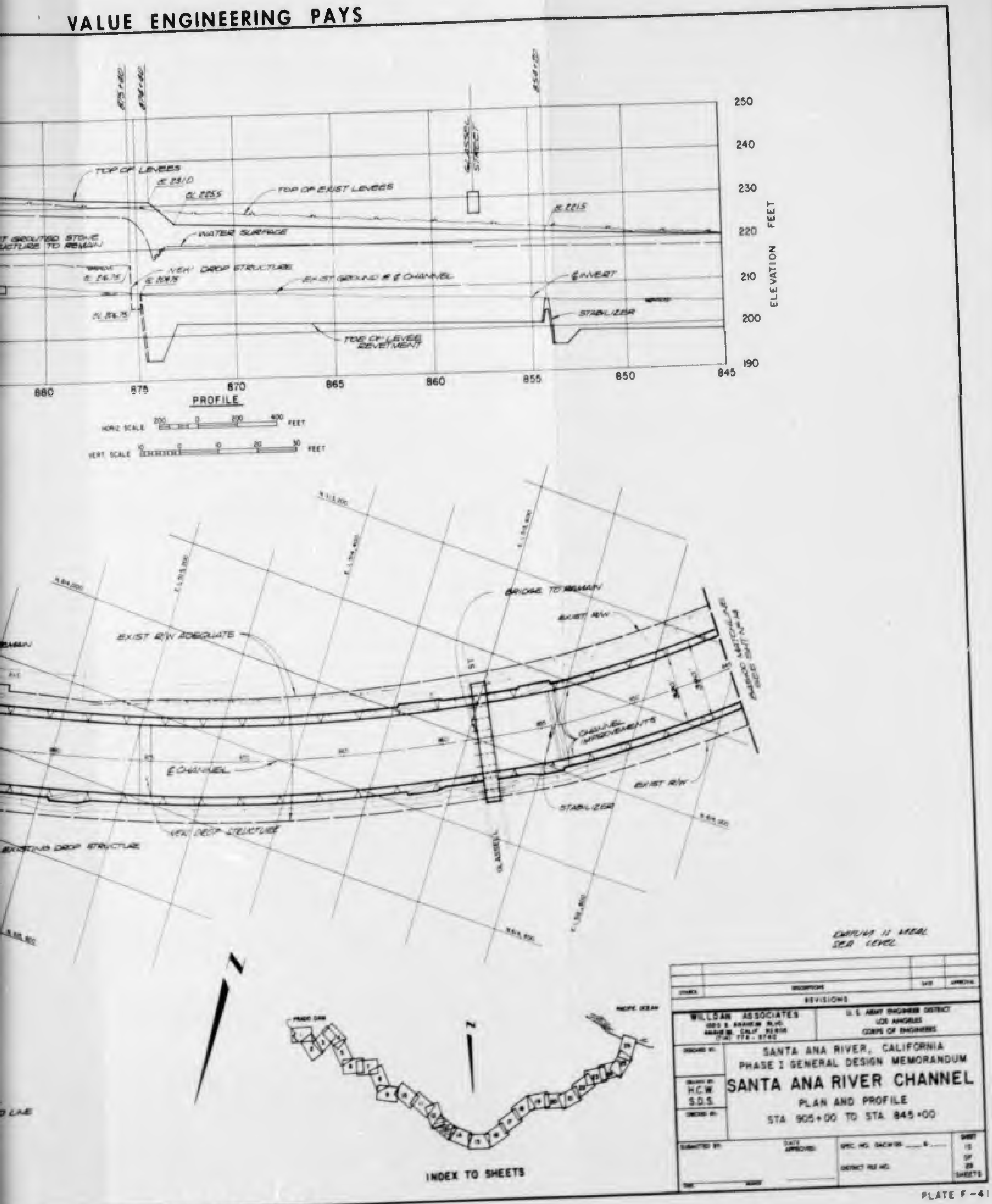
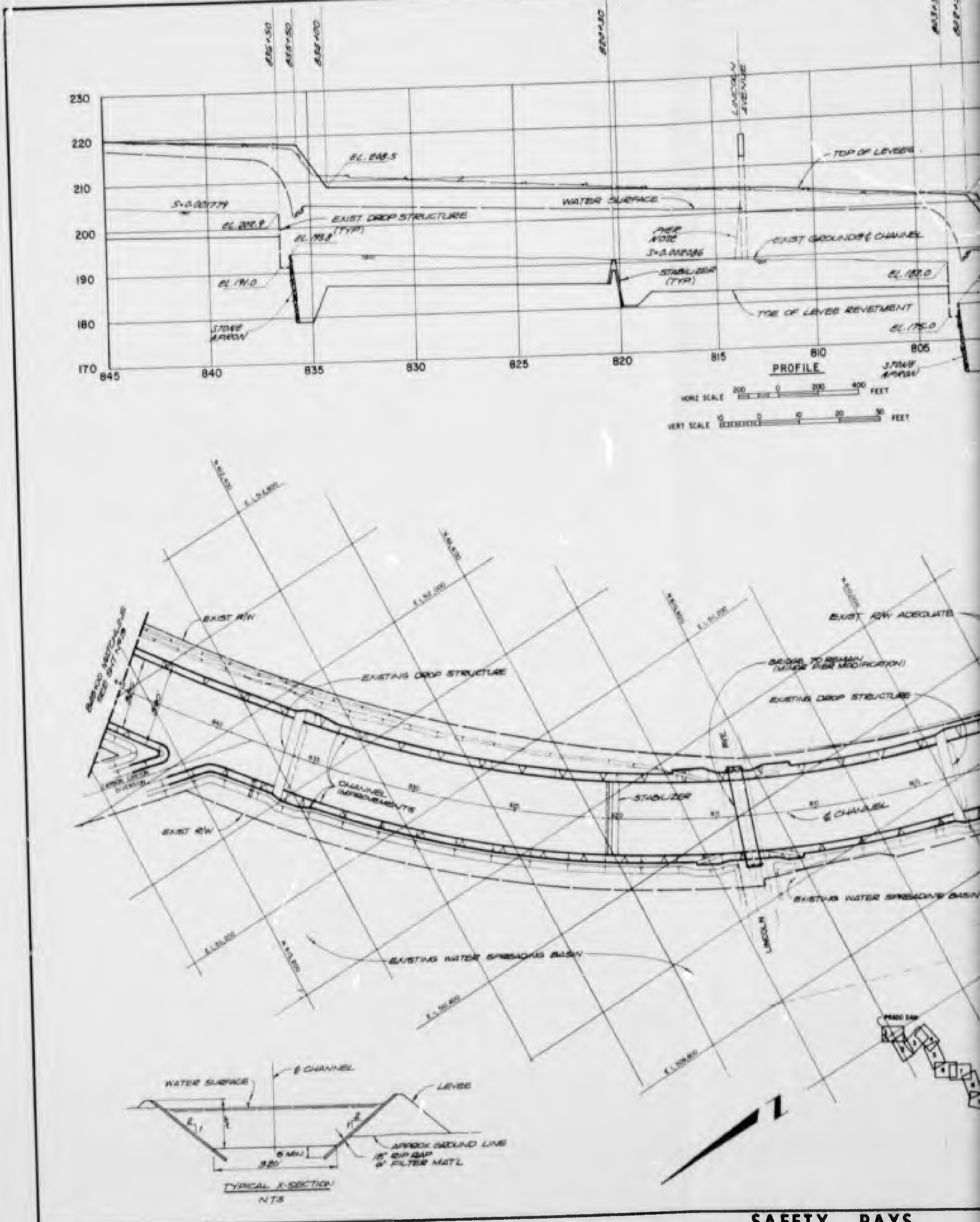
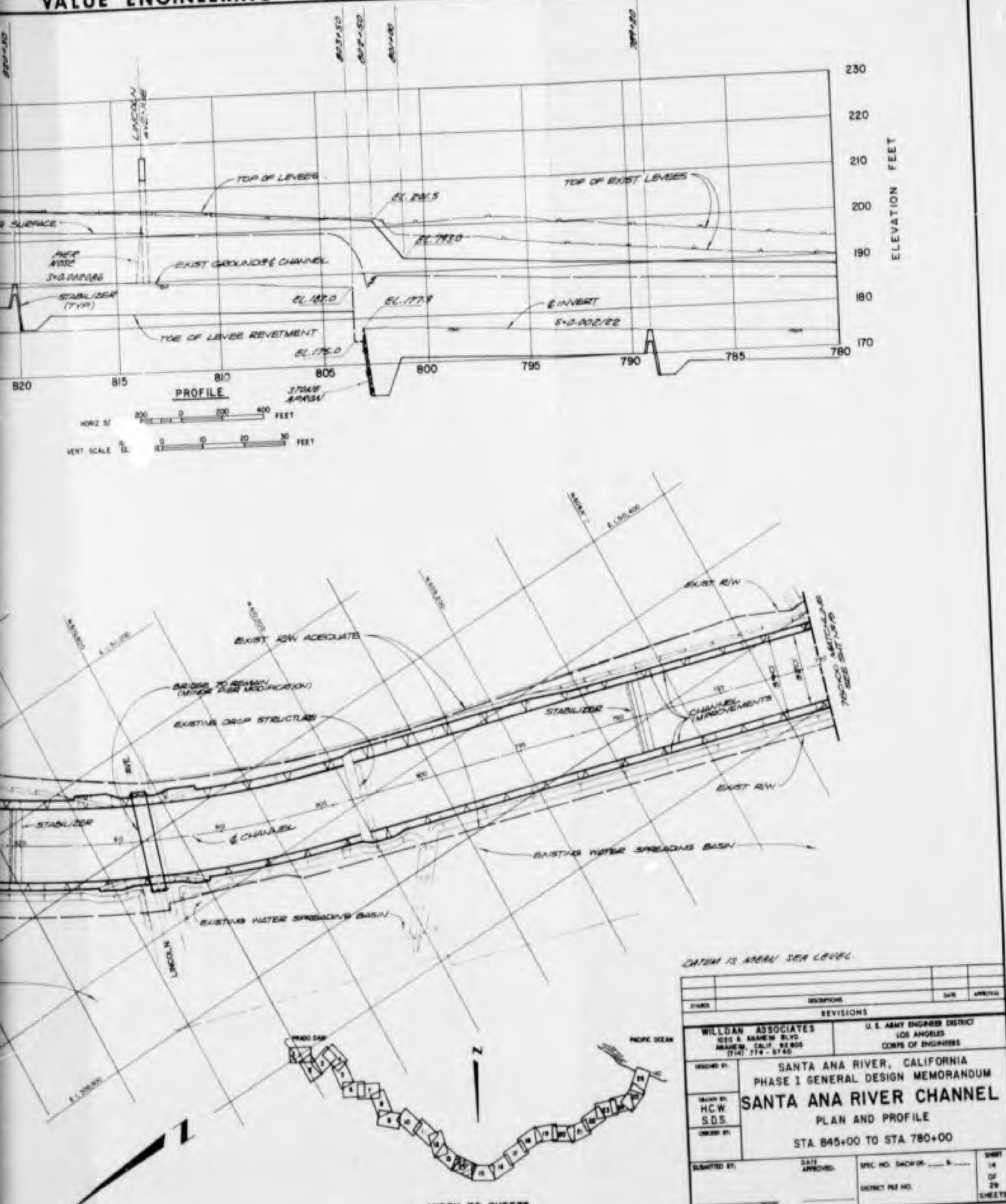


PLATE F-41

SAFETY PAYS


PROFILE
TOP OF LEVEES
WATER SURFACE
EXIST DROP STRUCTURE (TYP)
EXIST GROUND @ ℄ CHANNEL
STABILIZER (TYP)
TOE OF LEVEE REVETMENT
STONE APRON
EL. 208.5
EL. 202.9
EL. 191.0
EL. 187.0
EL. 175.0
HORIZ SCALE
VERT SCALE
FEET
EXIST R/W
EXISTING DROP STRUCTURE
CHANNEL IMPROVEMENTS
STABILIZER
℄ CHANNEL
EXISTING WATER SPREADING BASIN
EXIST R/W ADEQUATE
LINCOLN
TYPICAL X-SECTION
NTS
WATER SURFACE
℄ CHANNEL
LEVEE
APPROX GROUND LINE
18" RIP RAP W/ FILTER MAT'L

VALUE ENGINEERING PAYS



SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD.
ANAHEIM, CALIF. 92805
(714) 774-5740

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

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

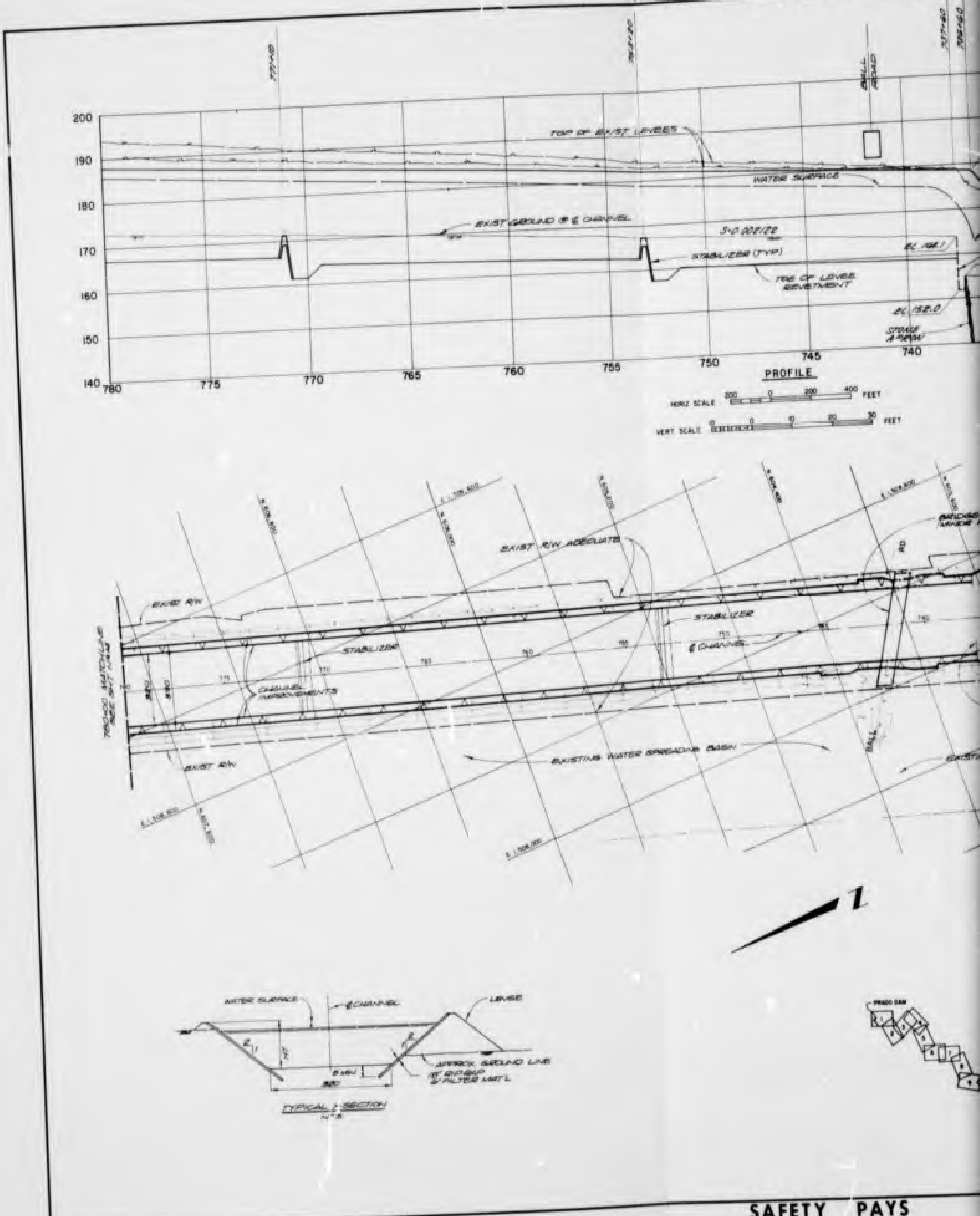
PLAN AND PROFILE

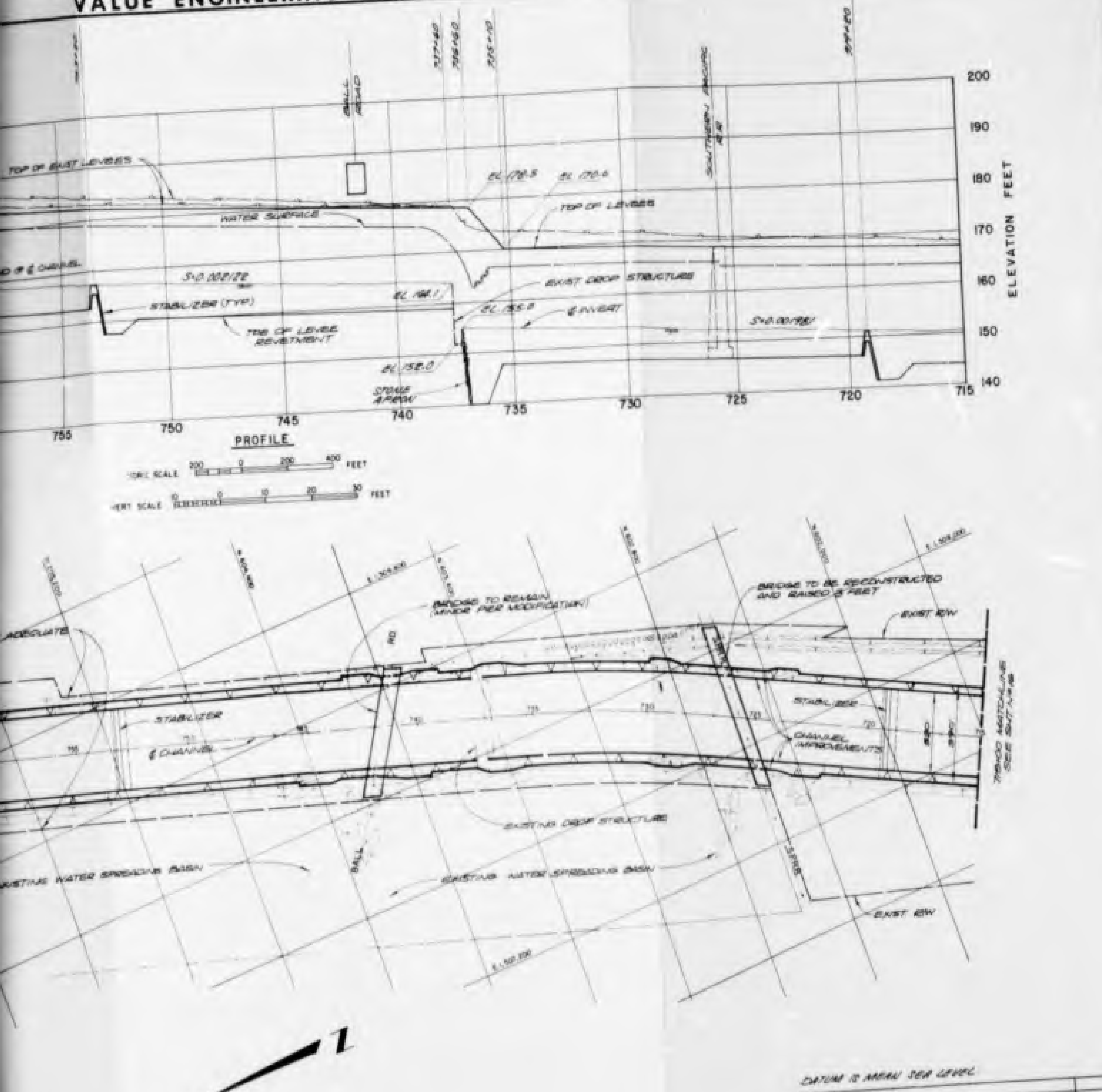
STA. 845+00 TO STA. 780+00

SUBMITTED BY: | DATE APPROVED: | SPEC. NO. DACW09-___-___ | DISTRICT FILE NO. | SHEET 14 OF 29 SHEETS

PLATE F-42

SAFETY PAYS


PROFILE
TOP OF EXIST LEVEES
WATER SURFACE
EXIST GROUND @ ℄ CHANNEL
STABILIZER (TYP)
TOE OF LEVEE REVETMENT
EL 152.0
STONE APRON
BALL ROAD
HORIZ SCALE
VERT SCALE
FEET
EXIST R/W ADEQUATE
EXIST R/W
STABILIZER
℄ CHANNEL
CHANNEL IMPROVEMENTS
EXISTING WATER SPREADING BASIN
BALL RD
PRADO MATCHLINE SEE SHT
WATER SURFACE
℄ CHANNEL
LEVEE
APPROX GROUND LINE
18" RIPRAP
8" FILTER MAT'L
TYPICAL X-SECTION
PRADO DAM



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SYMBOL	DESCRIPTION	DATE	APPROVAL
REVISIONS			

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD
ANAHEIM, CALIF. 92805
(714) 774-5740

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

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

PLAN AND PROFILE

STA. 780+00 TO STA. 715+00

SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
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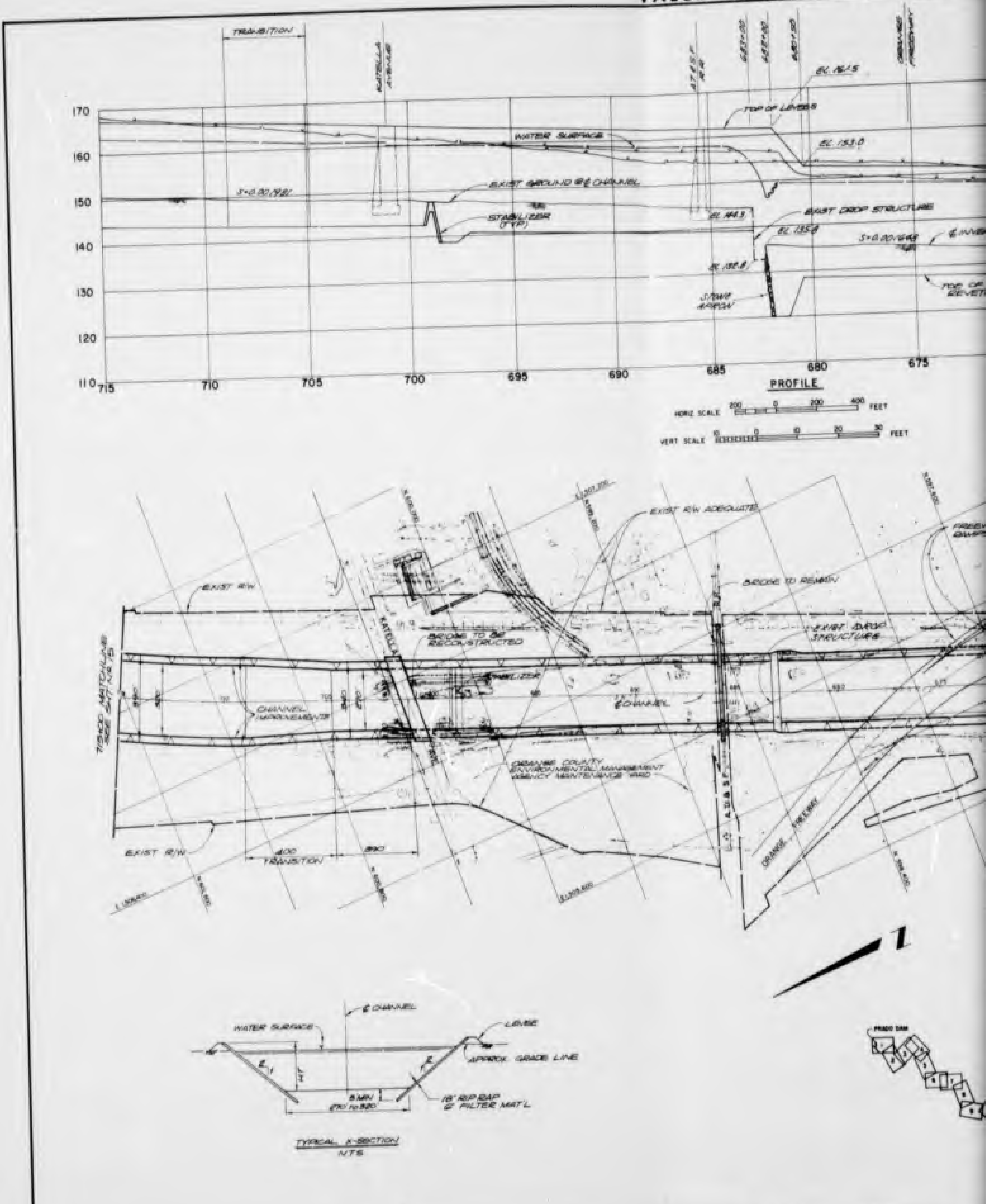
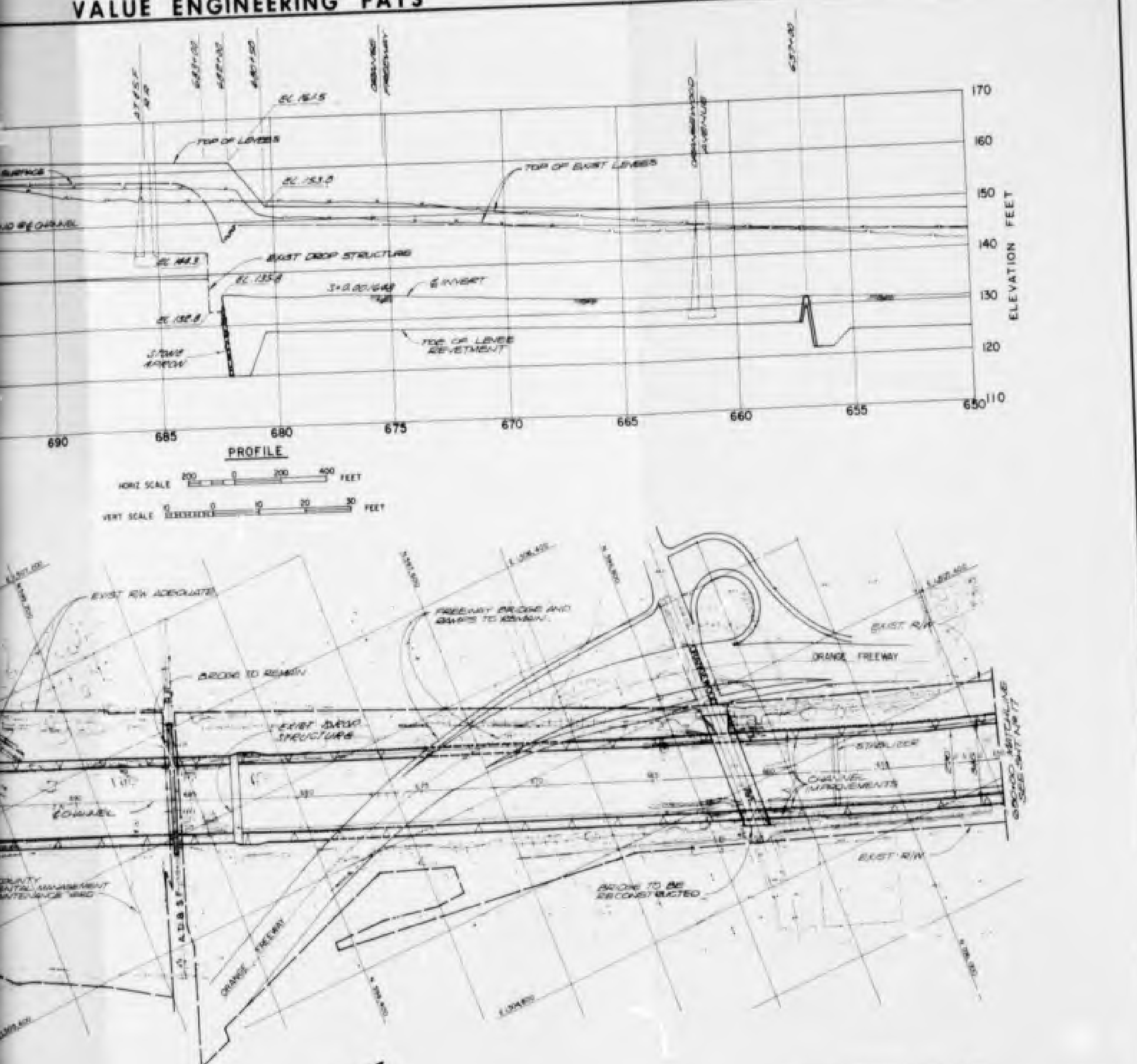
PLATE F-43

2


TRANSITION
KATELLA AVENUE
EL. 161.5
TOP OF LEVEES
WATER SURFACE
EL. 153.0
EXIST GROUND @ ℄ CHANNEL
S=0.00/1981
STABILIZER (TYP)
EL. 144.3
EXIST DROP STRUCTURE
EL. 135.8
EL. 132.8
STONE APRON
170
160
150
140
130
120
110
715
710
705
700
695
690
685
680
675
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
EXIST R/W ADEQUATE
EXIST R/W
BRIDGE TO REMAIN
BRIDGE TO BE RECONSTRUCTED
EXIST DROP STRUCTURE
CHANNEL IMPROVEMENTS
℄ CHANNEL
ORANGE COUNTY ENVIRONMENTAL MANAGEMENT AGENCY MAINTENANCE YARD
ORANGE FREEWAY
400 TRANSITION
890
℄ CHANNEL
WATER SURFACE
LEVEE
APPROX. GRADE LINE
5 MIN
270' to 320'
18" RIPRAP 6" FILTER MAT'L
TYPICAL X-SECTION
NTS
PRADO DAM



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REVISIONS

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD.
ANAHEIM, CALIF. 92805
(714) 774-5740

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

DESIGNED BY:

DRAWN BY: H.C.W. S.D.S.

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

PLAN AND PROFILE

STA 715+00 TO STA. 650+00

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

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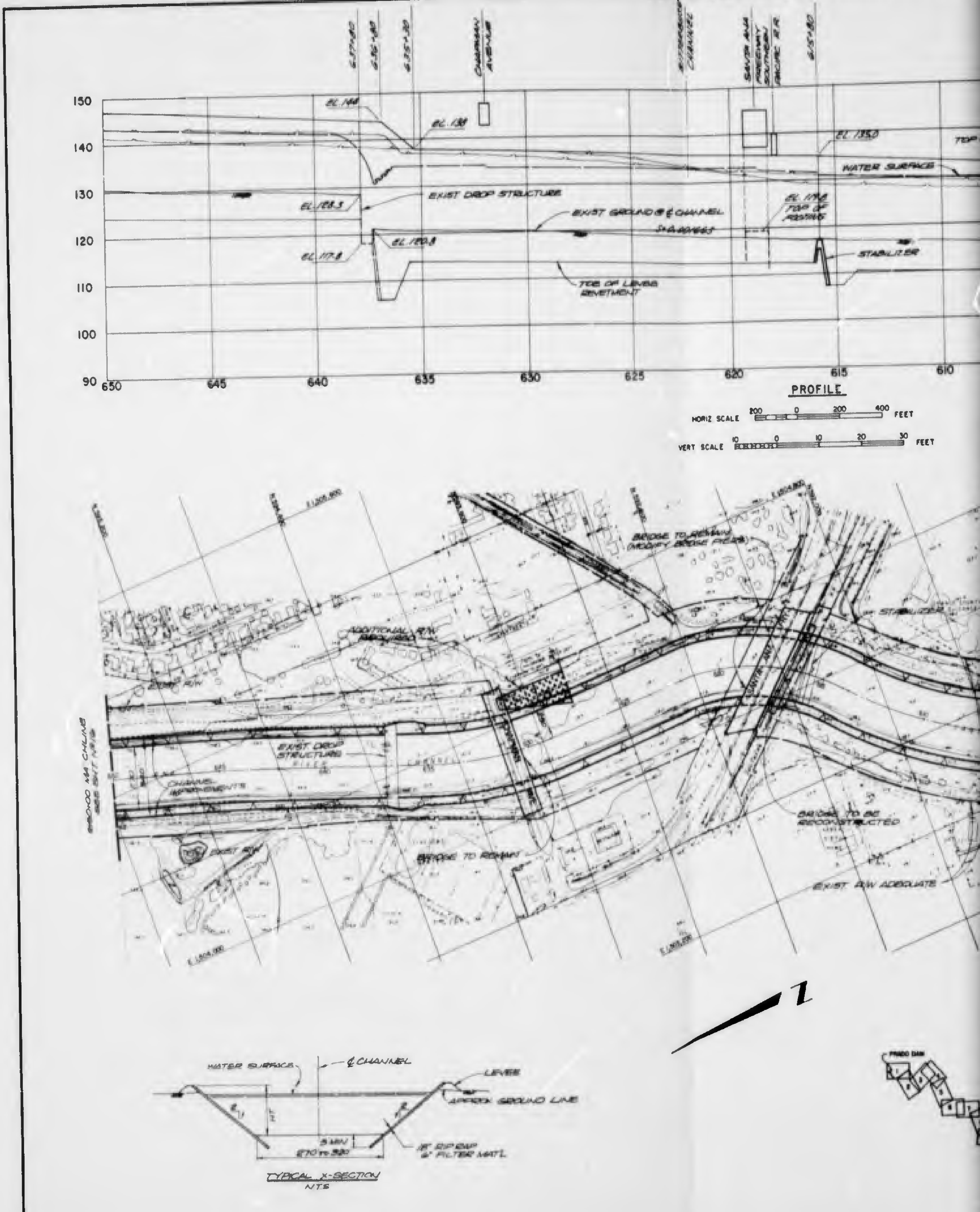
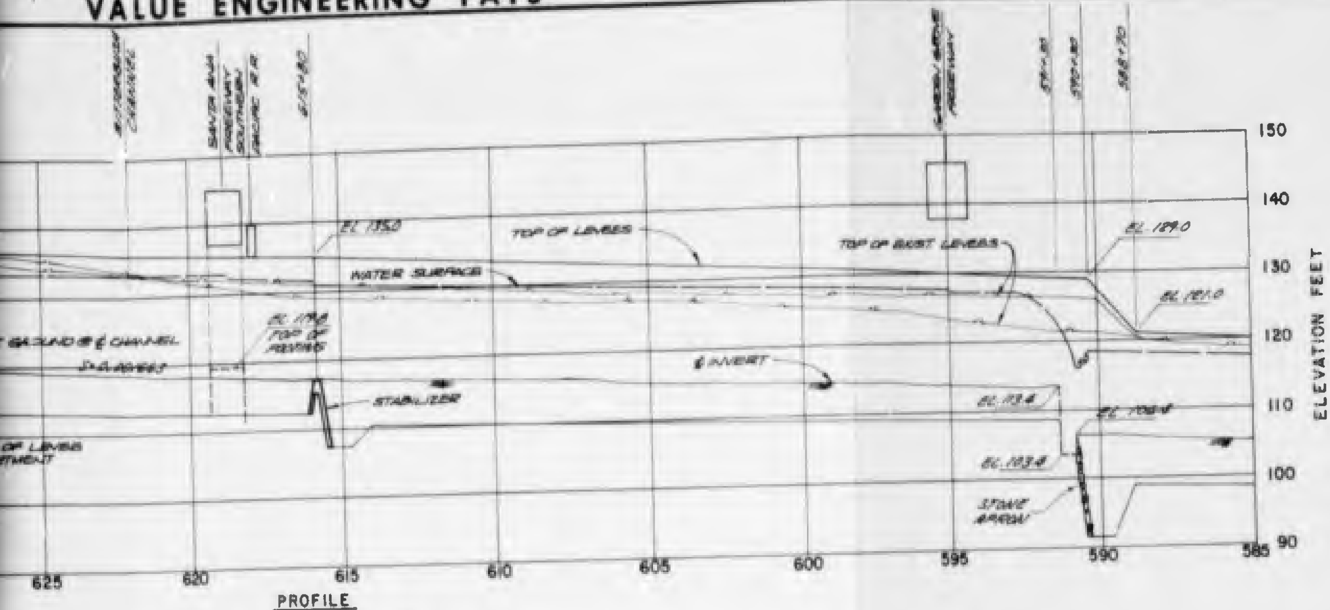
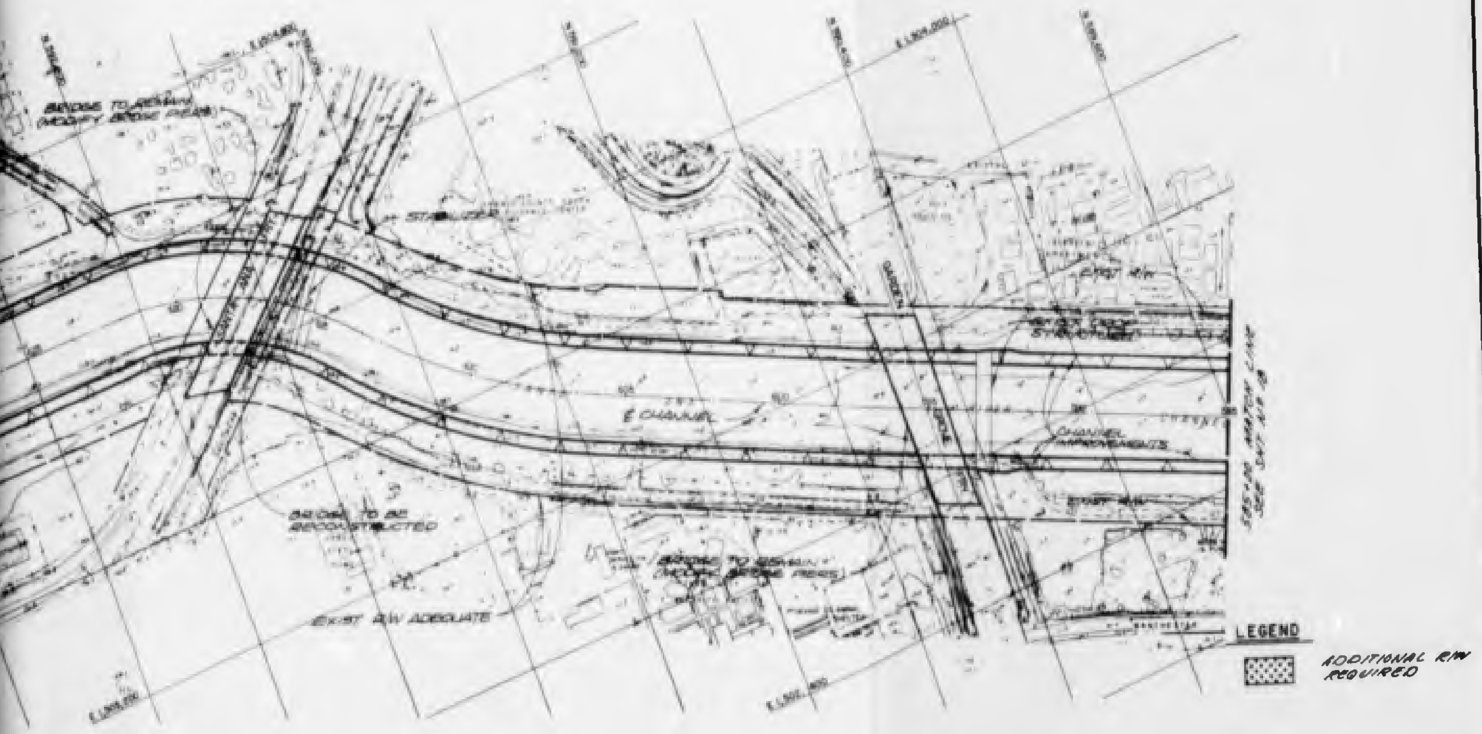
PLATE F-44

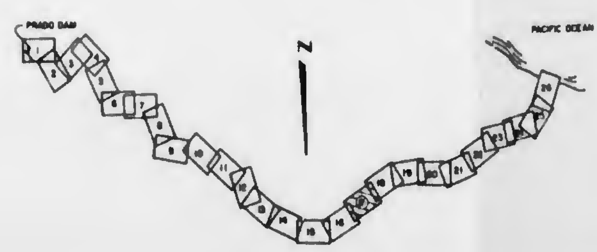
2


EL 144
EL 138
EL 135.0
WATER SURFACE
EXIST DROP STRUCTURE
EL 128.3
EL 117.8
EL 120.8
EXIST GROUND @ ℄ CHANNEL
TOE OF LEVEE
REVETMENT
STABILIZER
PROFILE
HORIZ SCALE
FEET
VERT SCALE
FEET
BRIDGE TO REMAIN
ADDITIONAL R/W
REQUIRED
EXIST DROP
STRUCTURE
BRIDGE TO BE
RECONSTRUCTED
EXIST R/W ADEQUATE
WATER SURFACE
℄ CHANNEL
LEVEE
APPROX GROUND LINE
TYPICAL X-SECTION
NTS





DATUM IS MEAN SEA LEVEL



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REVISIONS

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD
ANAHEIM, CALIF. 92805
(714) 774-8740

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

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

PLAN AND PROFILE

STA. 650+00 TO STA. 585+00

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
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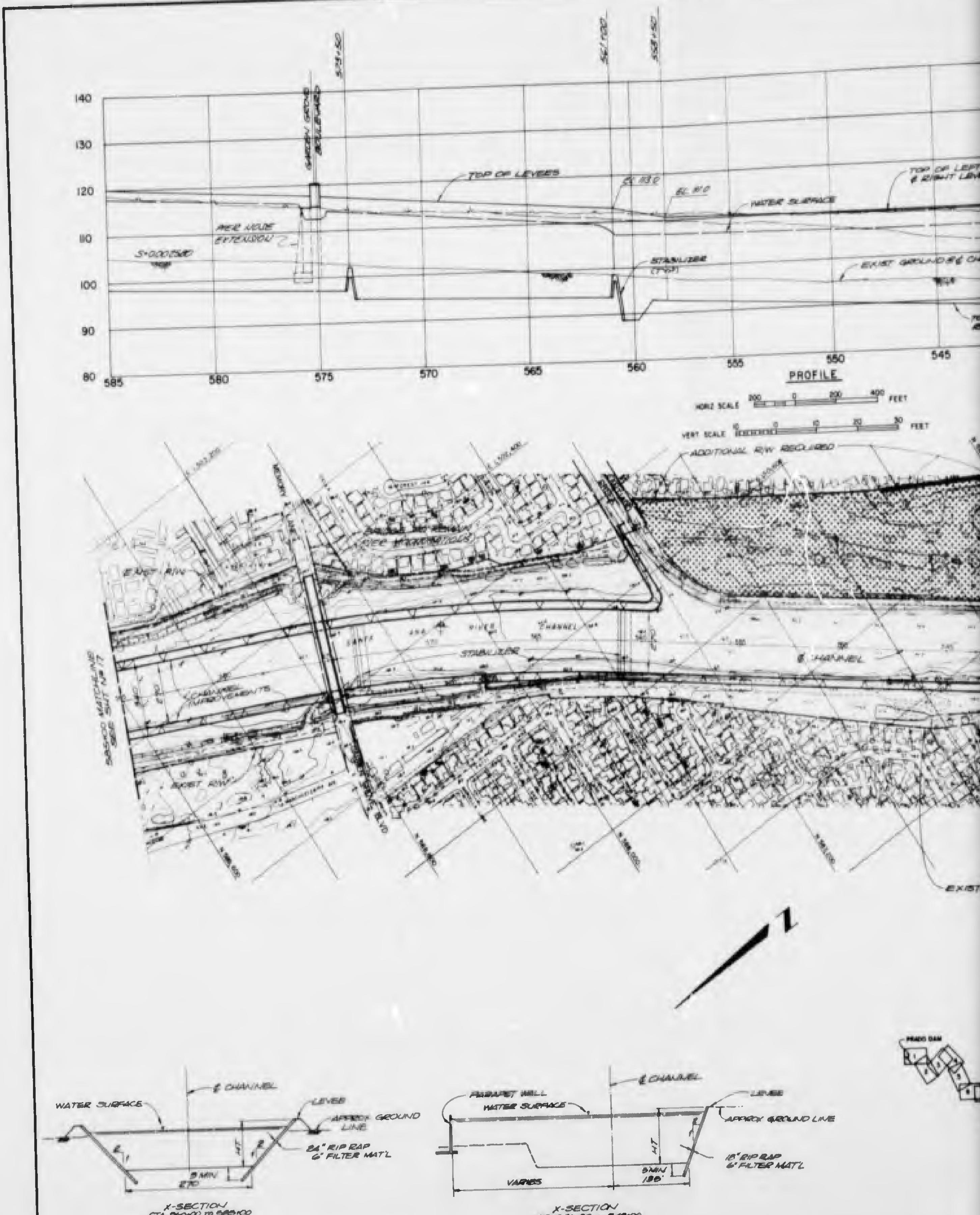
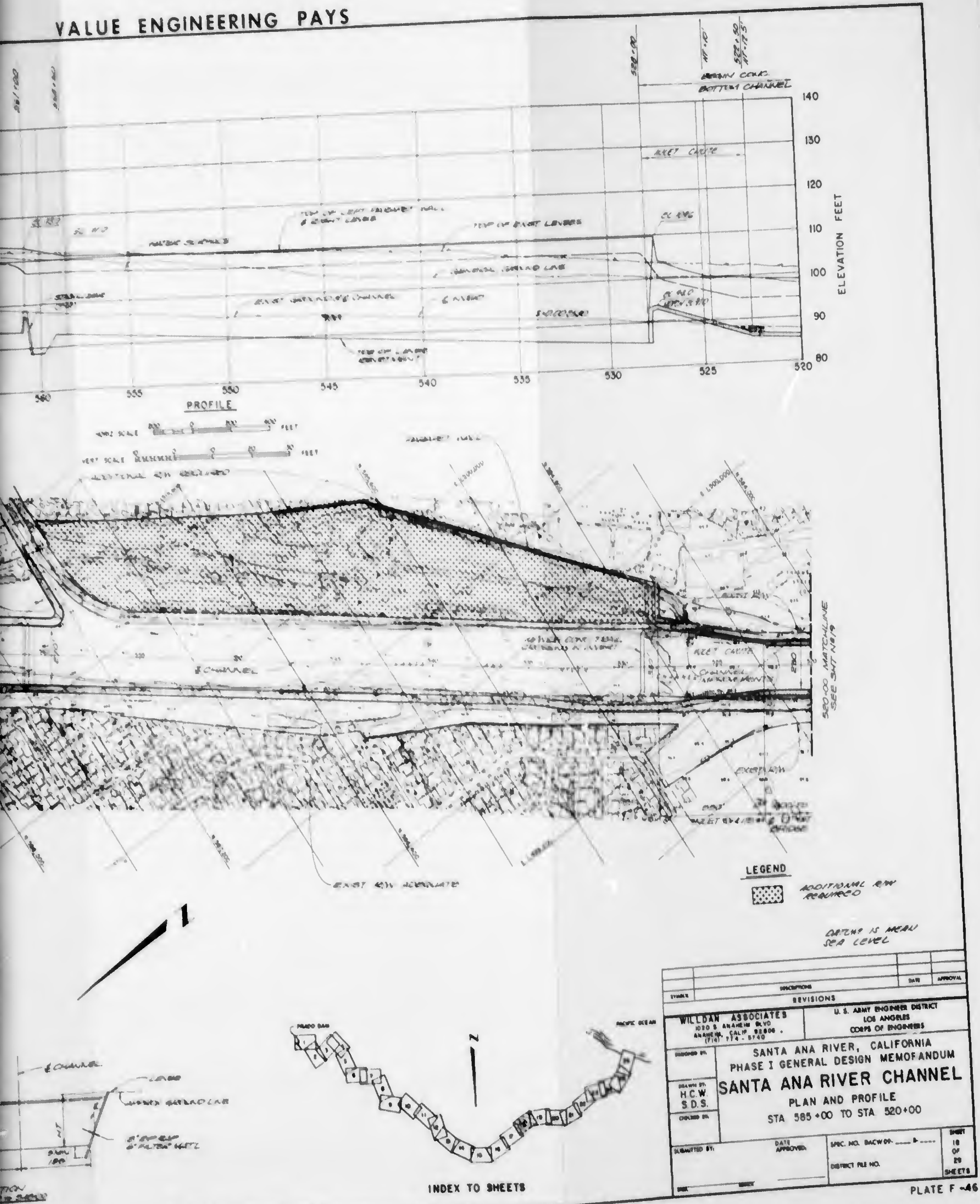
PLATE F-45

2

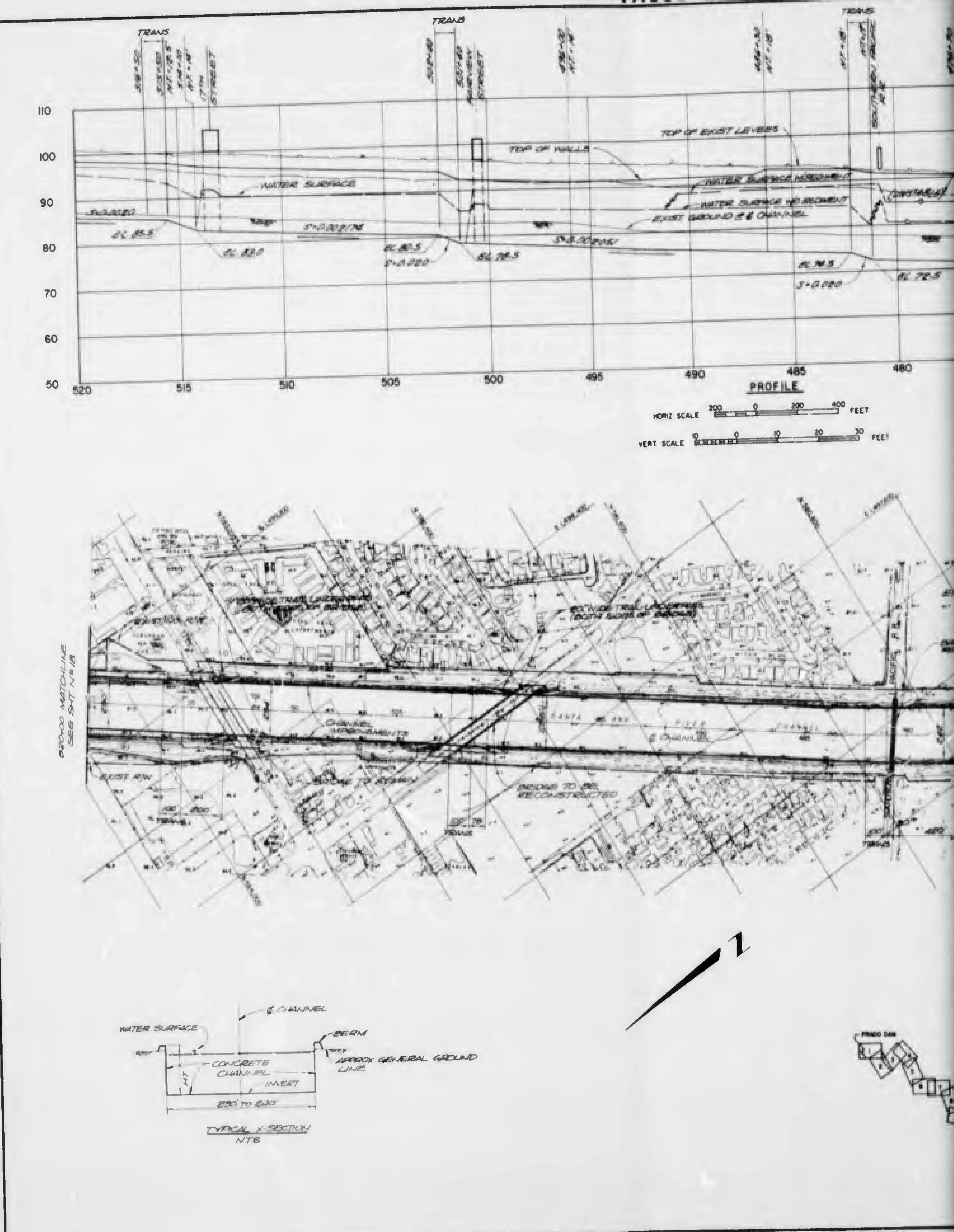

TOP OF LEVEES
WATER SURFACE
PIER NOSE EXTENSION
STABILIZER
EXIST GROUND
PROFILE
HORIZ SCALE
VERT SCALE
FEET
ADDITIONAL R/W REQUIRED
EXIST R/W
STABILIZER
℄ CHANNEL
WATER SURFACE
℄ CHANNEL
LEVEE
APPROX GROUND LINE
24" RIP RAP
6" FILTER MAT'L
X-SECTION
NTS
PARAPET WALL
WATER SURFACE
℄ CHANNEL
LEVEE
APPROX GROUND LINE
18" RIP RAP
6" FILTER MAT'L
VARIES
X-SECTION
N.T.S.

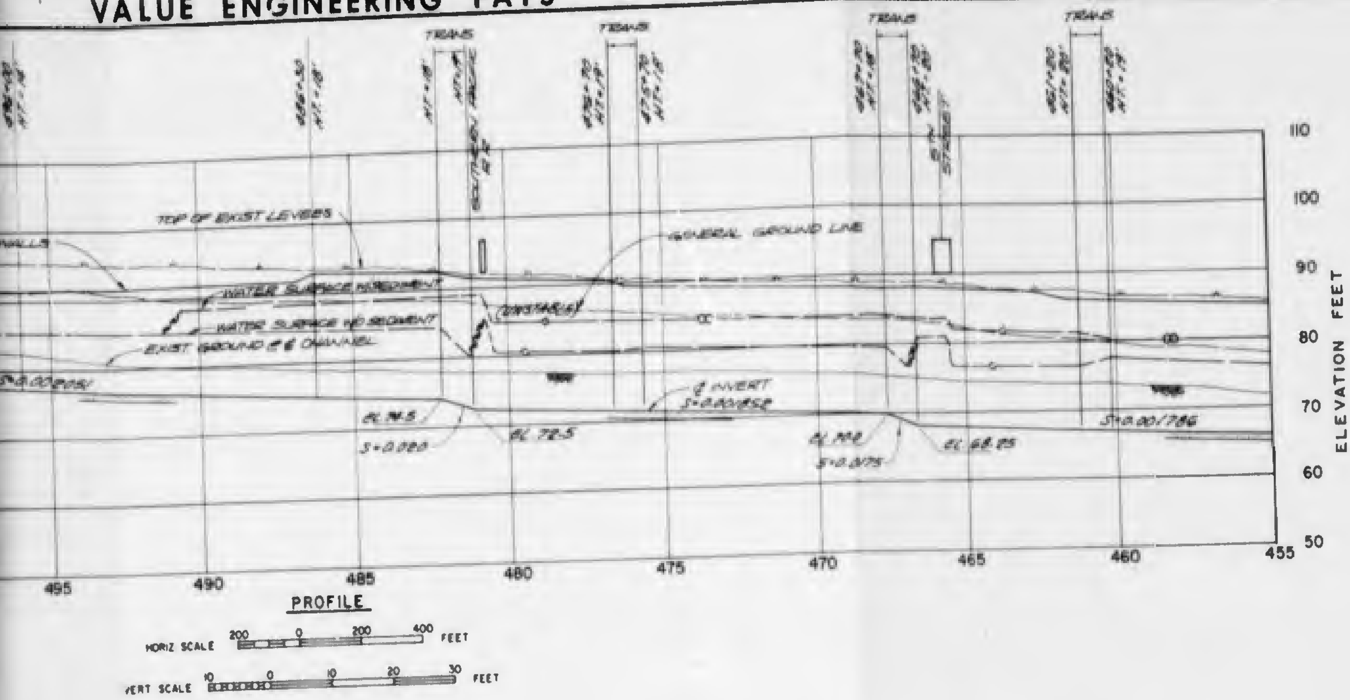
VALUE ENGINEERING PAYS

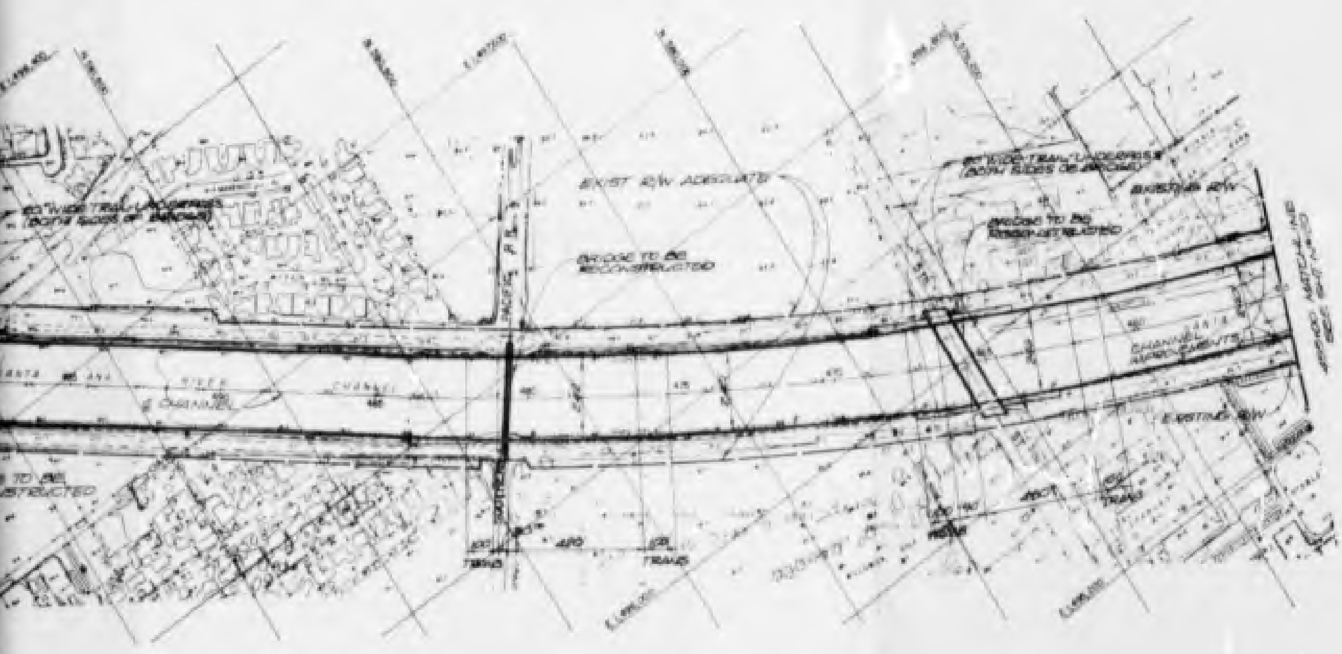


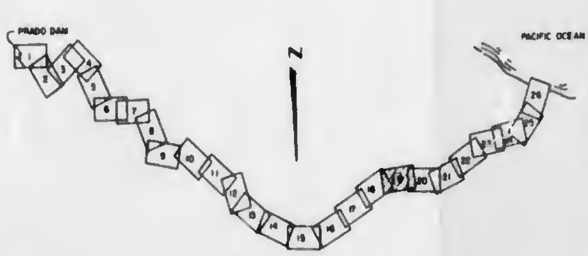
SAFETY PAYS

2


TRANS
17TH STREET
FAIRVIEW STREET
TOP OF EXIST LEVEES
TOP OF WALLS
WATER SURFACE
WATER SURFACE W/SEDIMENT
WATER SURFACE W/O SEDIMENT
EXIST GROUND @ ℄ CHANNEL
S=0.0020
EL 85.5
EL 83.0
S=0.002176
EL 80.5
S=0.020
EL 78.5
S=0.002051
EL 74.5
S=0.020
EL 72.5
110
100
90
80
70
60
50
520
515
510
505
500
495
490
485
480
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
520+00 MATCHLINE SEE SHT No 18
CHANNEL IMPROVEMENTS
SANTA ANA RIVER CHANNEL
℄ CHANNEL
EXIST R/W
BRIDGE TO REMAIN
BRIDGE TO BE RECONSTRUCTED
WATER SURFACE
℄ CHANNEL
BERM
CONCRETE CHANNEL
INVERT
APPROX GENERAL GROUND LINE
230' TO 240'
TYPICAL X-SECTION
NTS
PRADO DAM







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— ~ — UNSTABLE WATER SURFACE

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

WILLDAN ASSOCIATES
1020 S ANAHEIM BLVD
ANAHEIM, CALIF 92805
(714) 774-5740

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

DESIGNED BY:

DRAWN BY: H.C.W S.D.S.

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTA ANA RIVER CHANNEL
PLAN AND PROFILE
STA. 520+00 TO STA. 455+00

SUBMITTED BY:

DATE APPROVED:

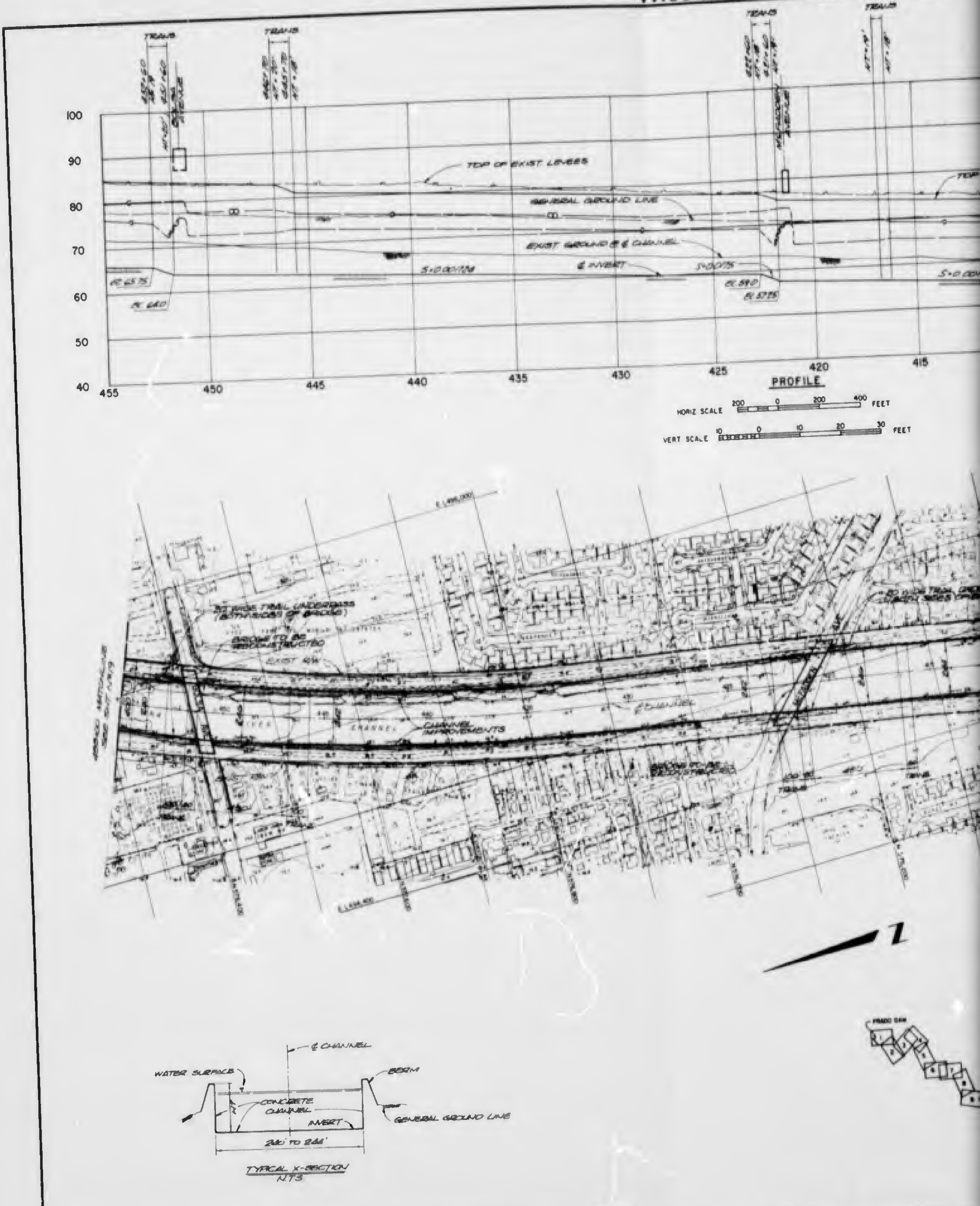
SPEC. NO. DACW 09-

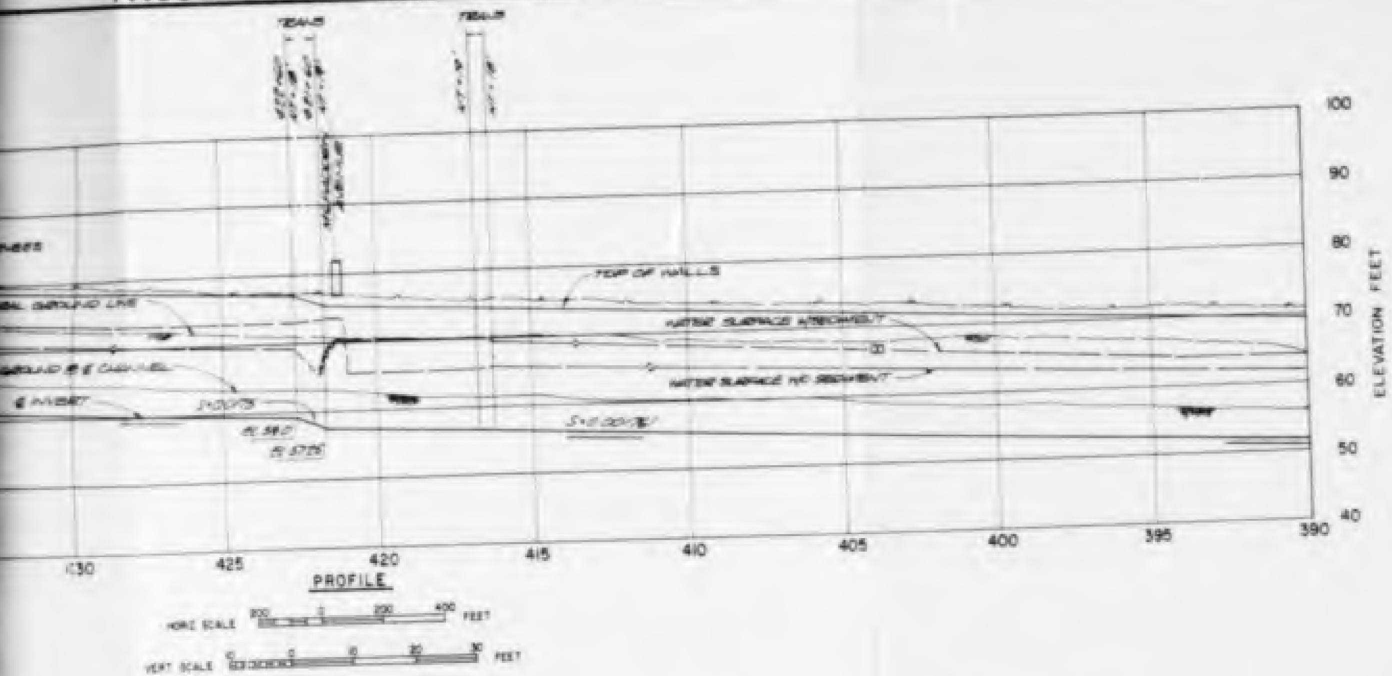
DISTRICT FILE NO.

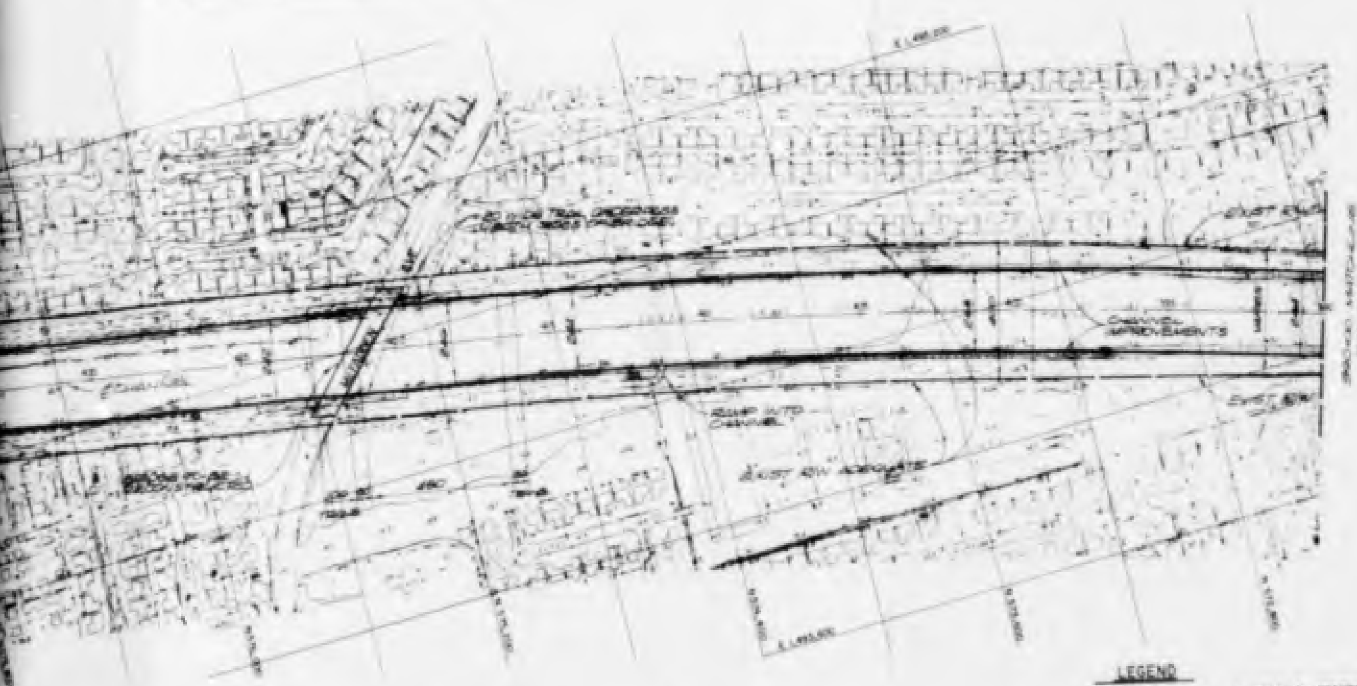
SHEET 19 OF 29 SHEETS

PLATE F -47

2


TRANS
TOP OF EXIST LEVEES
GENERAL GROUND LINE
EXIST GROUND @ ℄ CHANNEL
℄ INVERT
100
90
80
70
60
50
40
455
450
445
440
435
430
425
420
415
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
EXIST R/W
℄ CHANNEL
CHANNEL
CHANNEL IMPROVEMENTS
℄ CHANNEL
WATER SURFACE
BERM
CONCRETE CHANNEL
INVERT
GENERAL GROUND LINE
TYPICAL X-SECTION
N.T.S

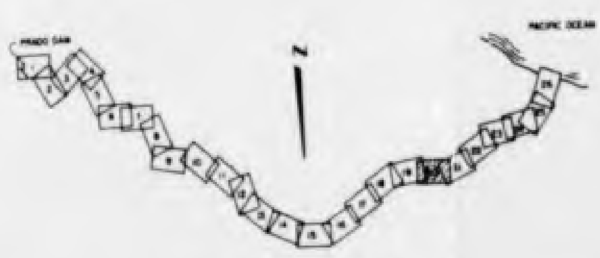




LEGEND

UNSTABLE WATER SURFACE

DATUM IS MEAN SEA LEVEL



SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD.
ANAHEIM, CALIF. 92805
(714) 774-5740

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

DESIGNED BY:
DRAWN BY: H.C.W.
S.D.S.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

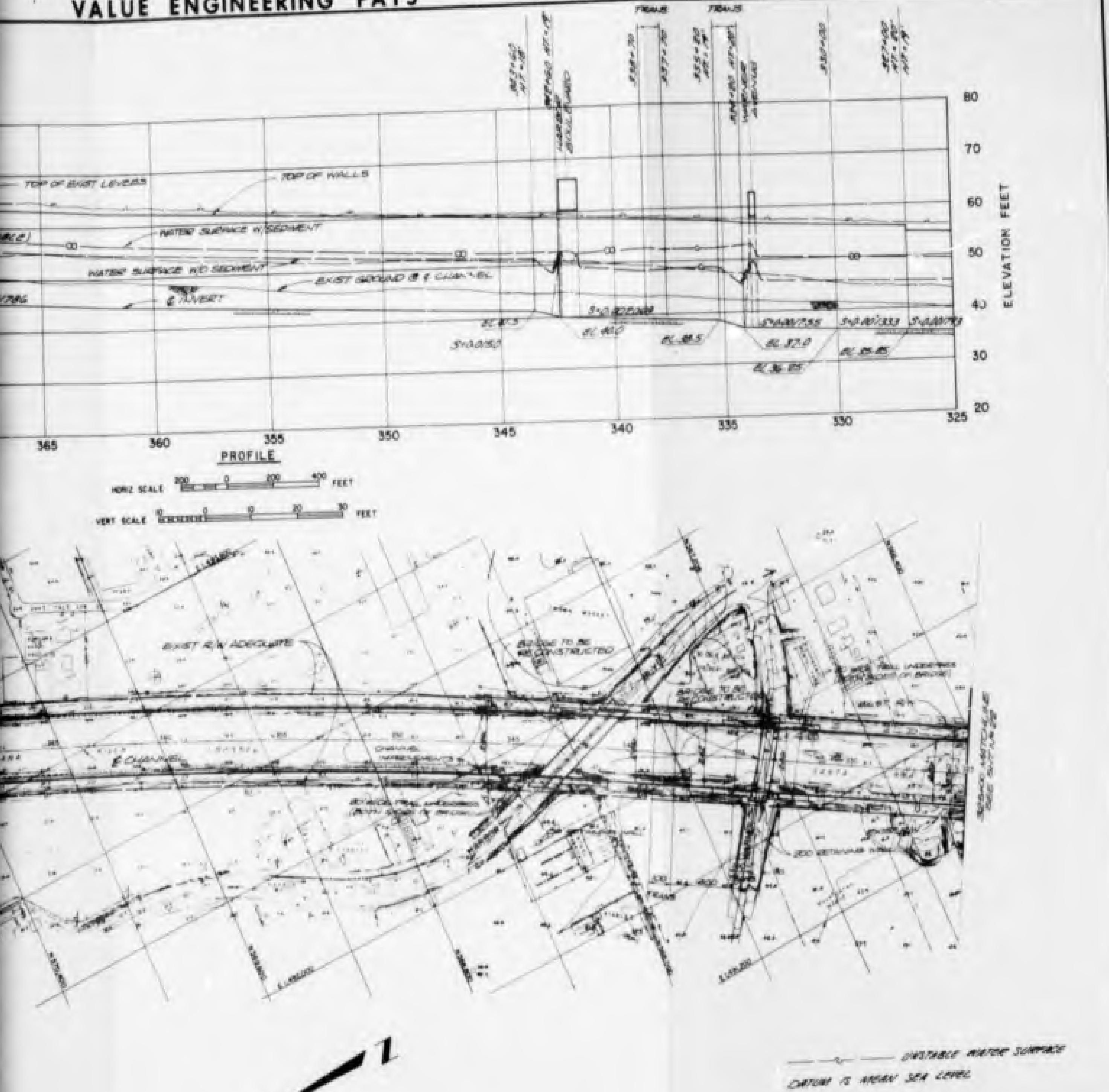
PLAN AND PROFILE
STA. 455+00 TO STA. 390+00

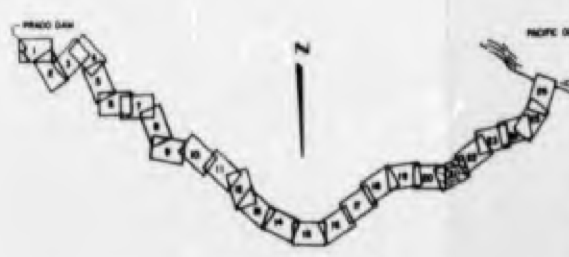
SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 20 OF 29 SHEETS

PLATE F-48

2


TRANS
TOP OF EXIST LEVEES
TOP OF WALLS
GENERAL GROUND LINE
WATER SURFACE W/SEDIMENT
WATER SURFACE W/O SEDIMENT
℄ INVERT
EL. 51'
EL. 49
80
70
60
50
40
30
20
390
385
380
375
370
365
360
355
350
PROFILE
HORIZ SCALE
200
0
200
400
FEET
VERT SCALE
10
0
10
20
30
FEET
EXIST R/W ADEQUATE
℄ CHANNEL
CHANNEL IMPROVEMENTS
WATER SURFACE
℄ CHANNEL
BERM
CONCRETE CHANNEL
INVERT
APPROX GROUND LINE
TYPICAL X-SECTION
NTS





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	REVISIONS		

WILLDAN ASSOCIATES

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

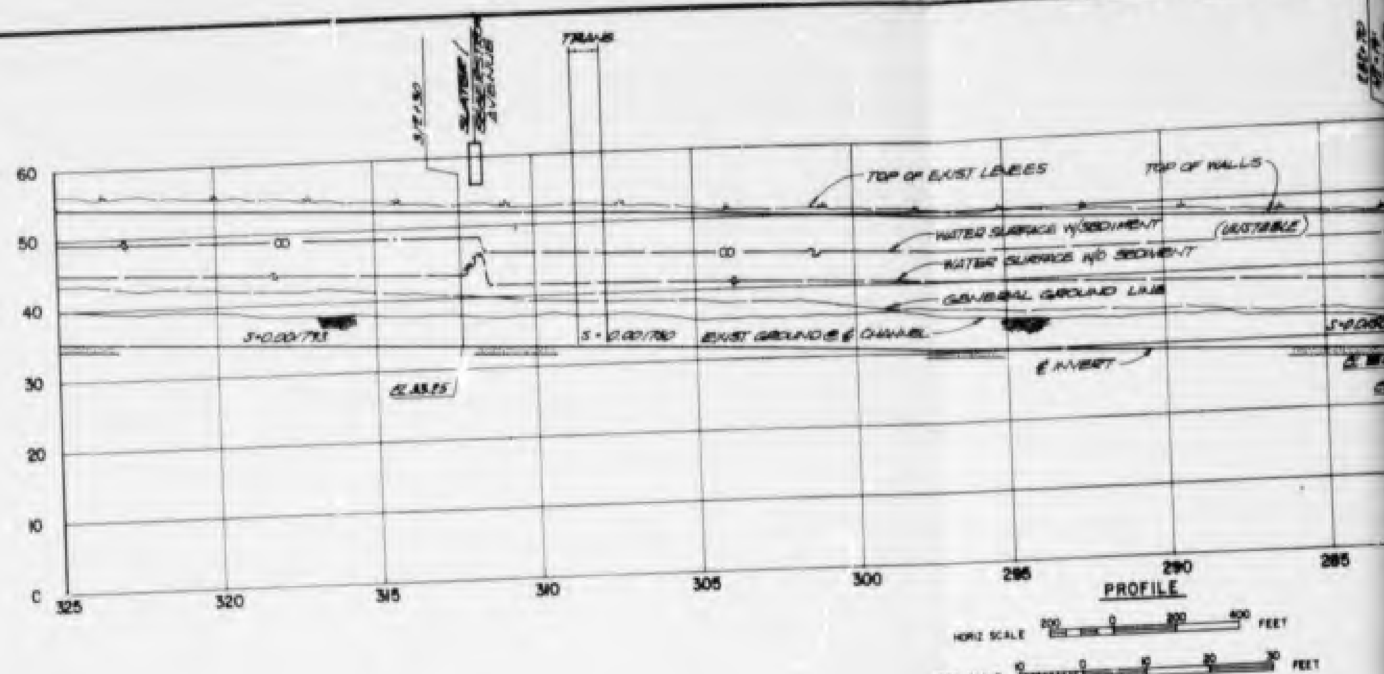
PLAN AND PROFILE

STA. 390+00 TO STA. 325+00

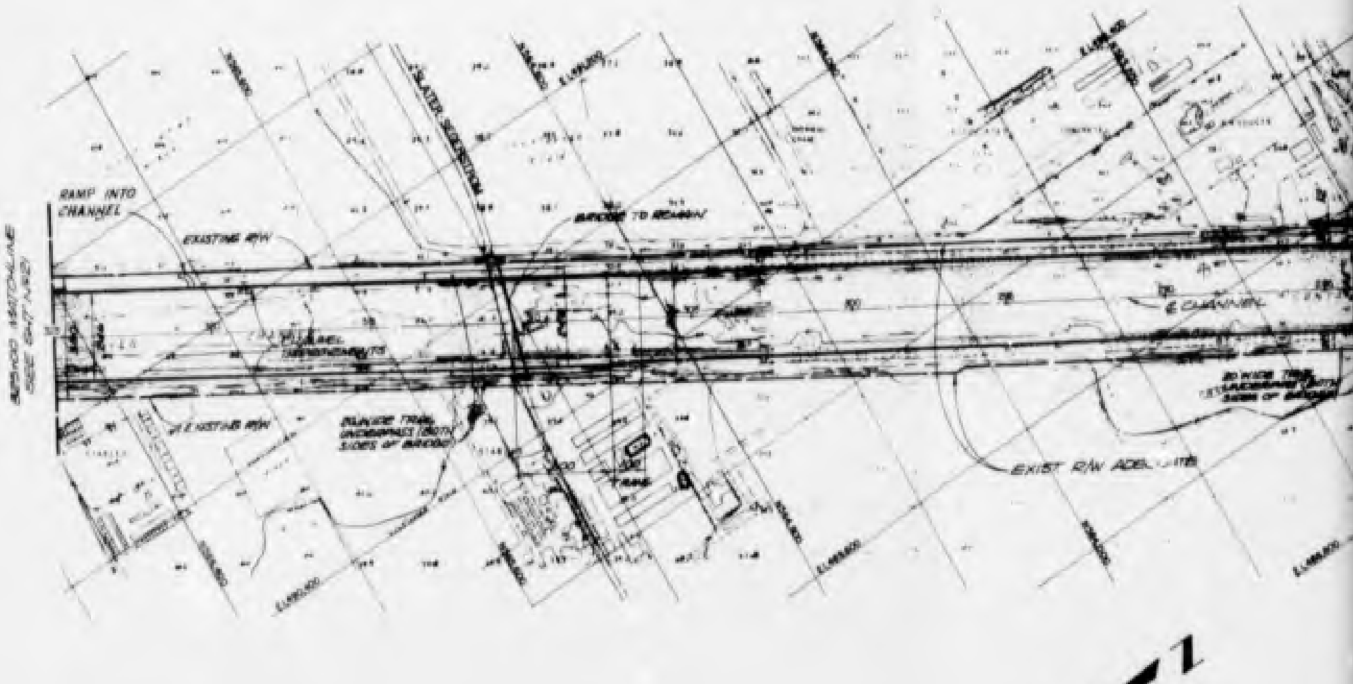
DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW 09-
DISTRICT FILE NO.
SHEET 21 OF 29 SHEETS

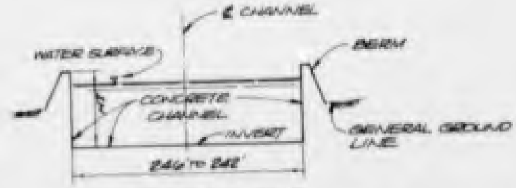
PLATE F-49

2

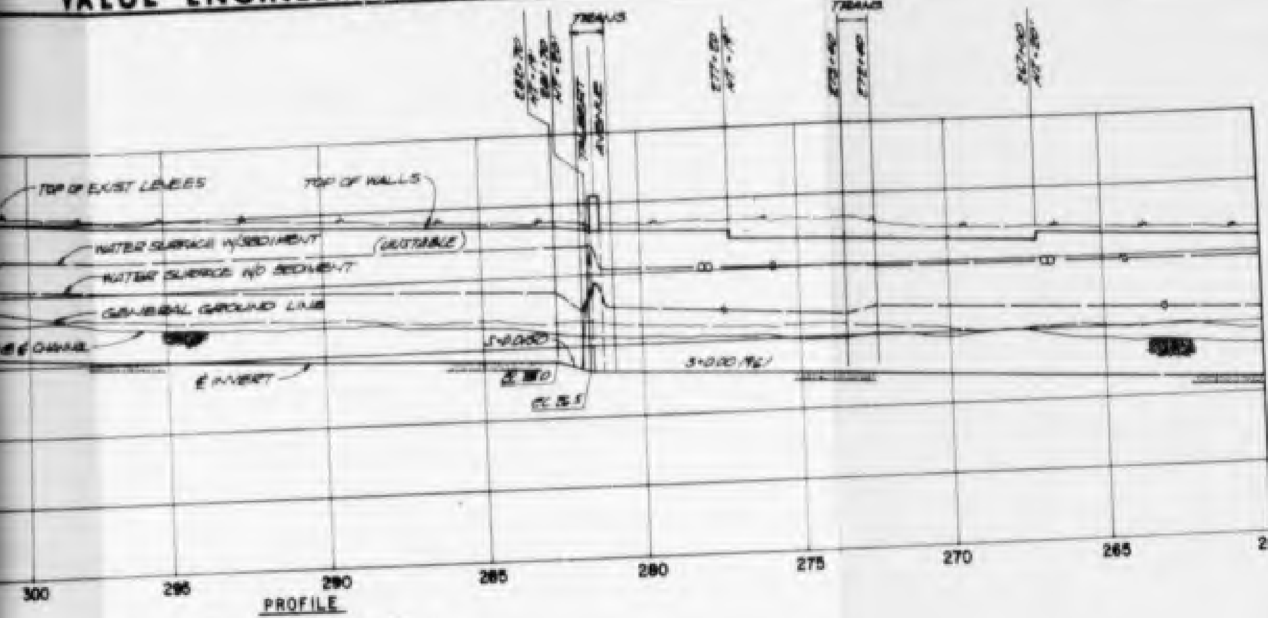


PROFILE





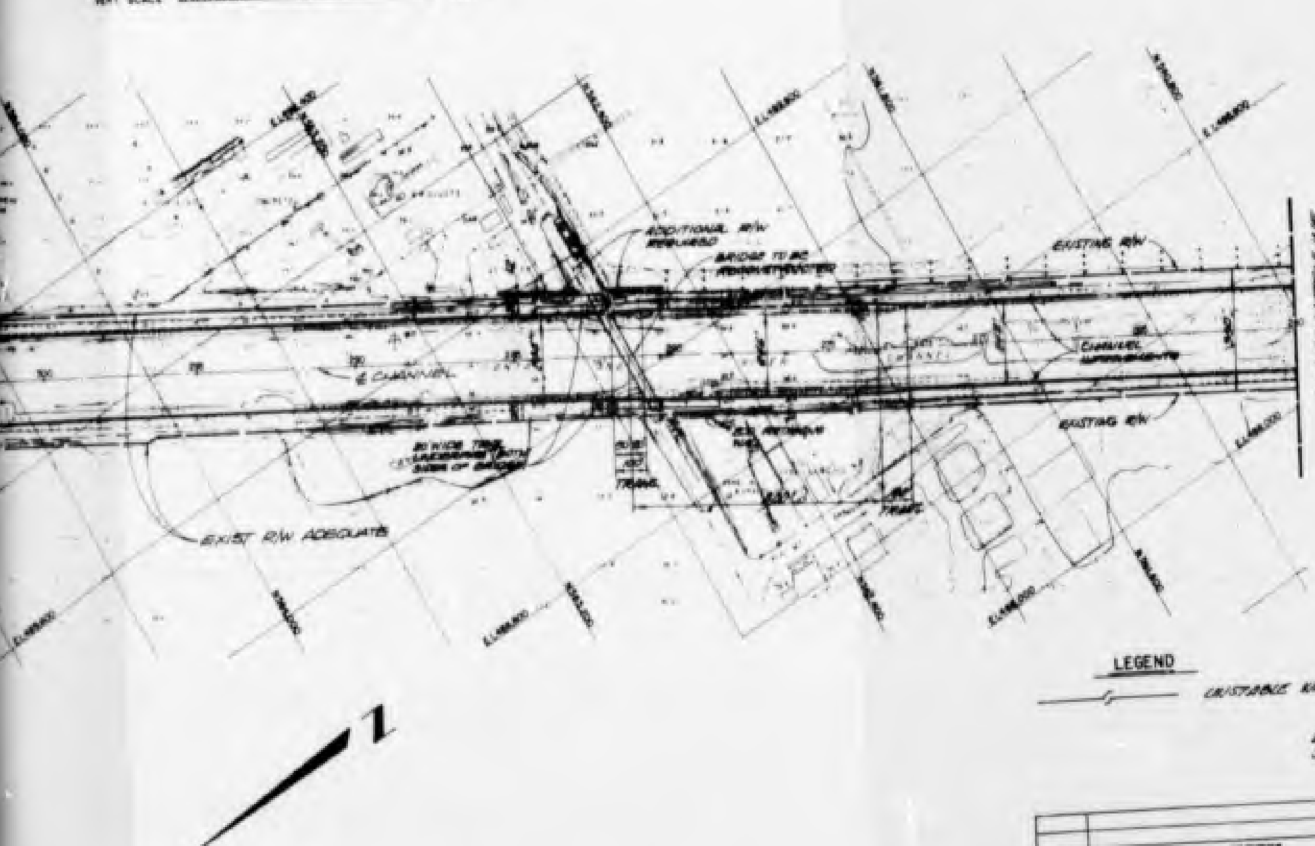
TYPICAL X-SECTIONS
NTS



PROFILE

HORIZ SCALE 200 0 200 400 FEET

VERT SCALE 10 0 10 20 30 FEET



LEGEND

UNSTABLE WATER

DATUM IS MEAN SEA LEVEL

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SYMBOL	DESCRIPTION	DATE	APPROVAL
REVISIONS			

WILLDAN ASSOCIATES
1000 S. ANAHEIM BLVD.
ANAHEIM, CALIF. 92805
(714) 774-6740

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

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

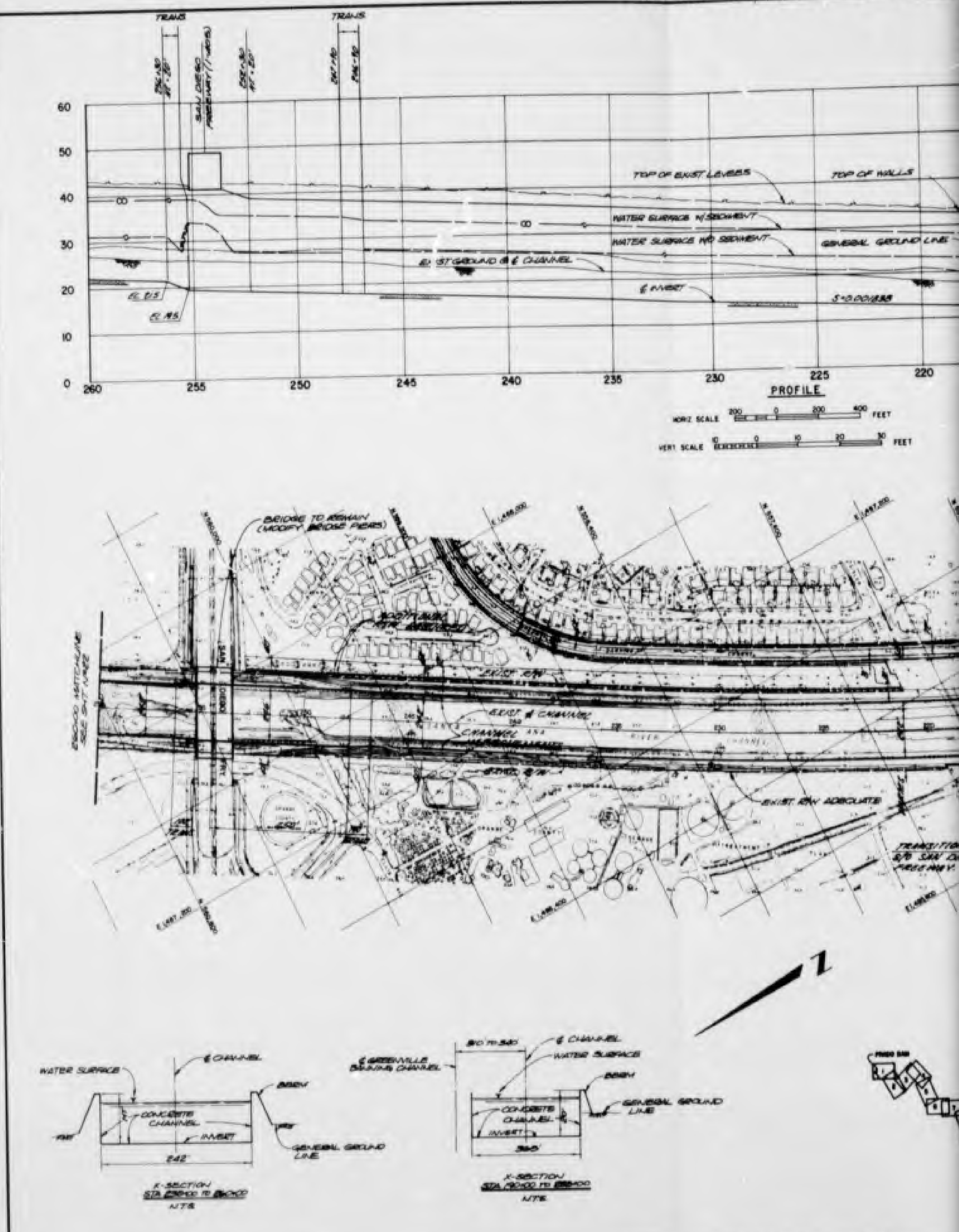
PLAN AND PROFILE

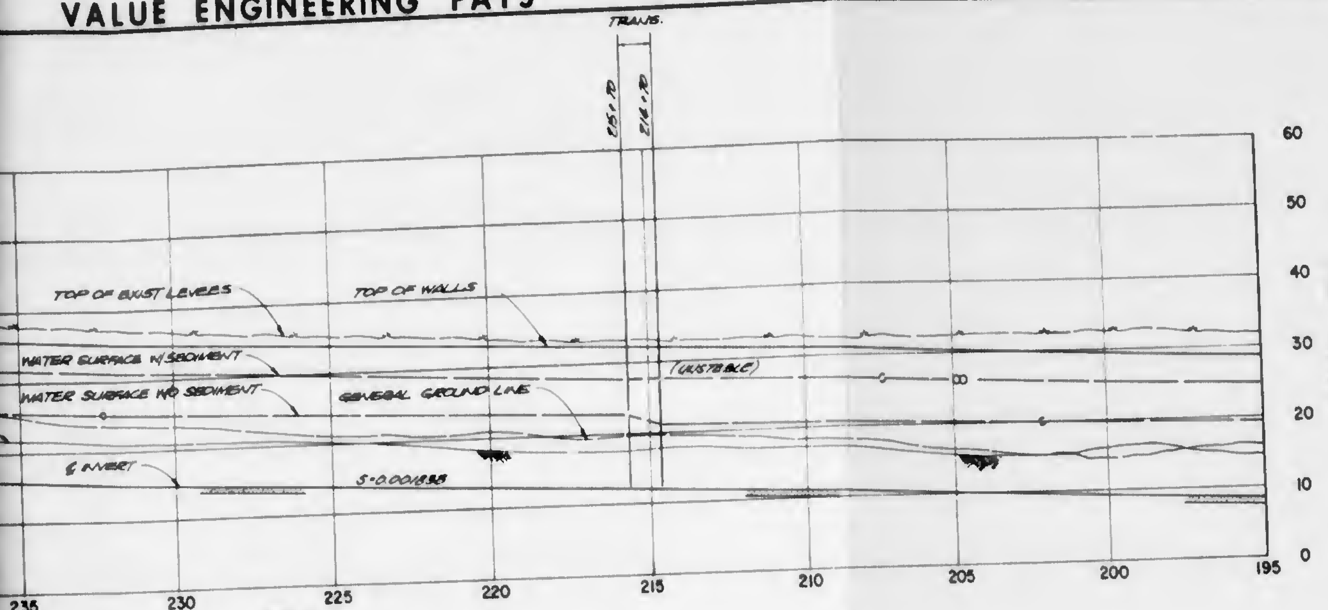
STA. 325+00 TO STA. 260+00

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 22 OF 29 SHEETS

PLATE F-60

2

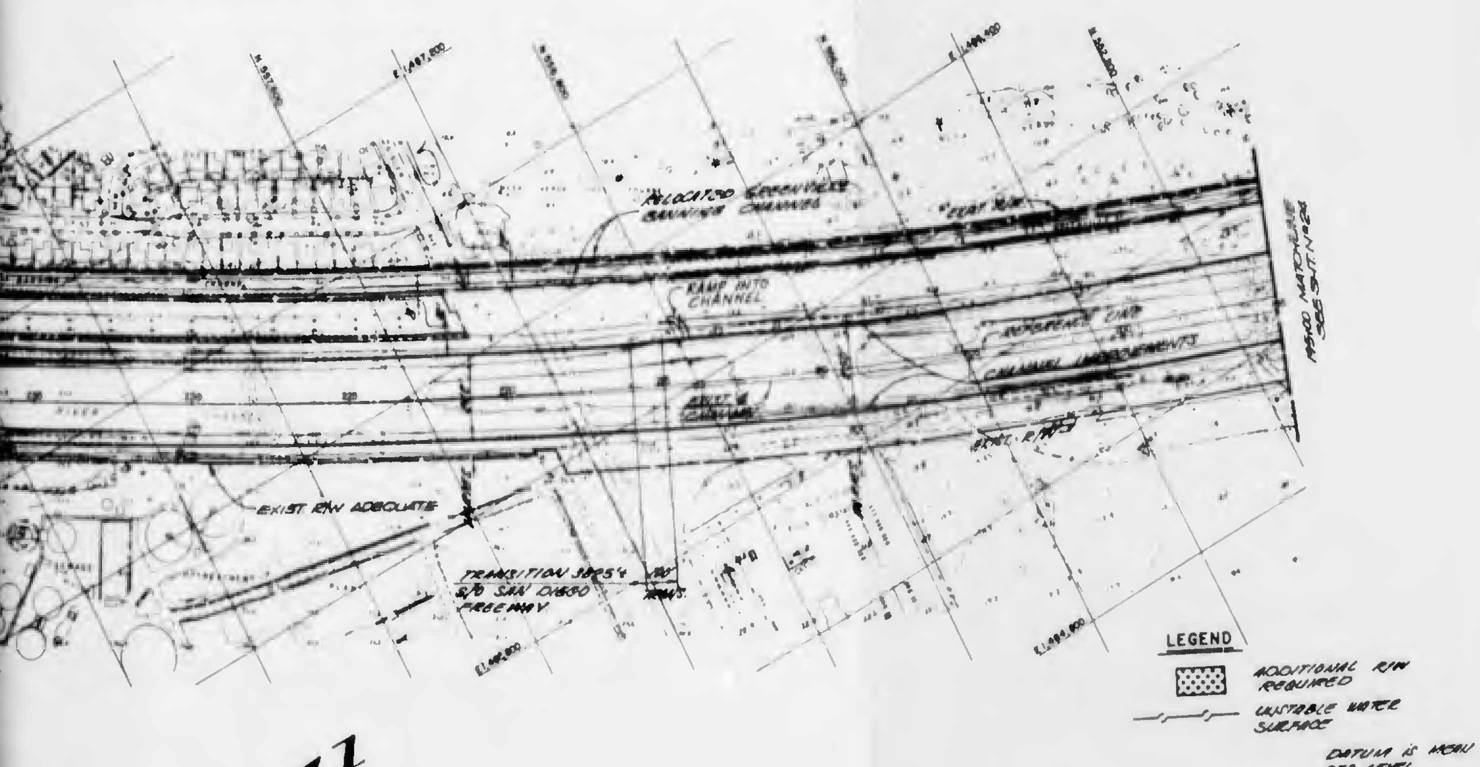

TRANS
TRANS
60
50
40
30
20
10
0
260
255
250
245
240
235
230
225
220
TOP OF EXIST LEVEES
TOP OF WALLS
WATER SURFACE W/SEDIMENT
WATER SURFACE W/O SEDIMENT
GENERAL GROUND LINE
EXIST GROUND @ ¢ CHANNEL
¢ INVERT
PROFILE
HORIZ SCALE
VERT SCALE
FEET
BRIDGE TO REMAIN (MODIFY BRIDGE PIERS)
EXIST. R/W
EXIST. ¢ CHANNEL
EXIST. R/W ADEQUATE
WATER SURFACE
¢ CHANNEL
BERM
CONCRETE CHANNEL
INVERT
GENERAL GROUND LINE
242'
X-SECTION
NTS
WATER SURFACE
BERM
GENERAL GROUND LINE
X-SECTION
NTS



PROFILE

HORIZ SCALE 200 0 200 400 FEET

VERT SCALE 10 0 10 20 30 FEET



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	REVISIONS		

WILLDAN ASSOCIATES
1900 S. ANAHEIM BLVD.
ANAHEIM, CALIF. 92805
(714) 774-8740

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

DESIGNED BY:

DRAWN BY: H.C.W. S.D.S.

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL

PLAN AND PROFILE

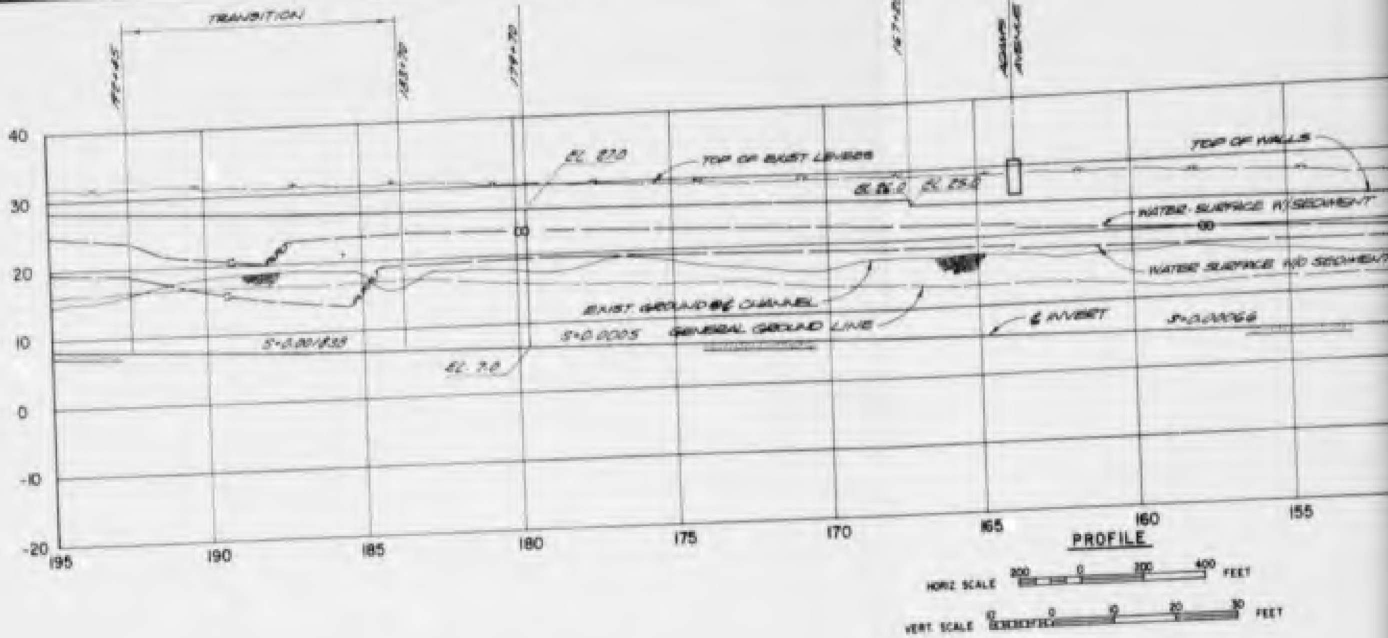
STA 260+00 TO STA 195+00

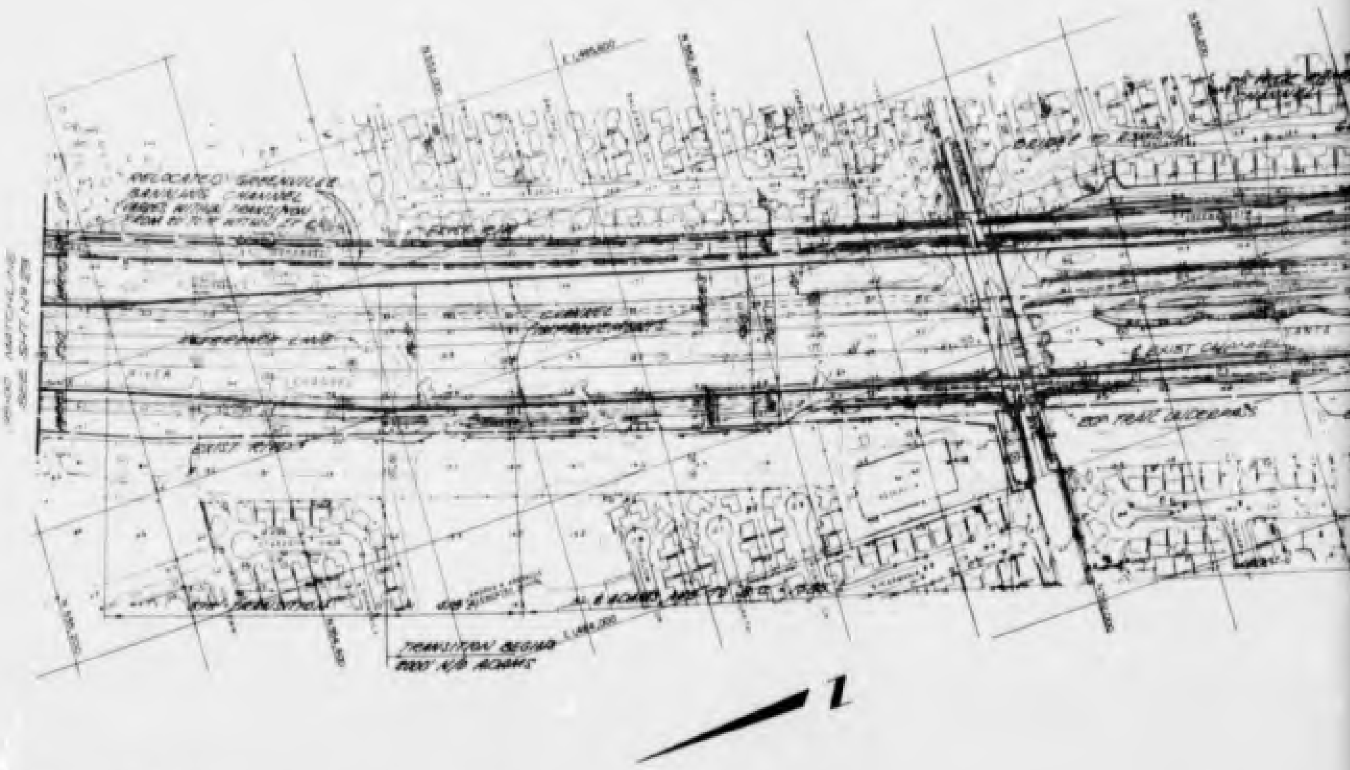
SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09- ___ B- ___ SHEET 23 OF 29 SHEETS

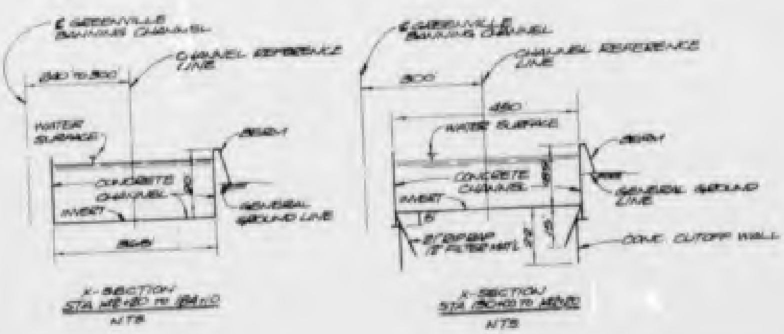
DISTRICT FILE NO.

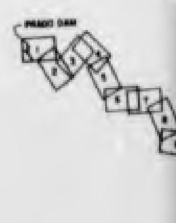
PLATE F-51

2

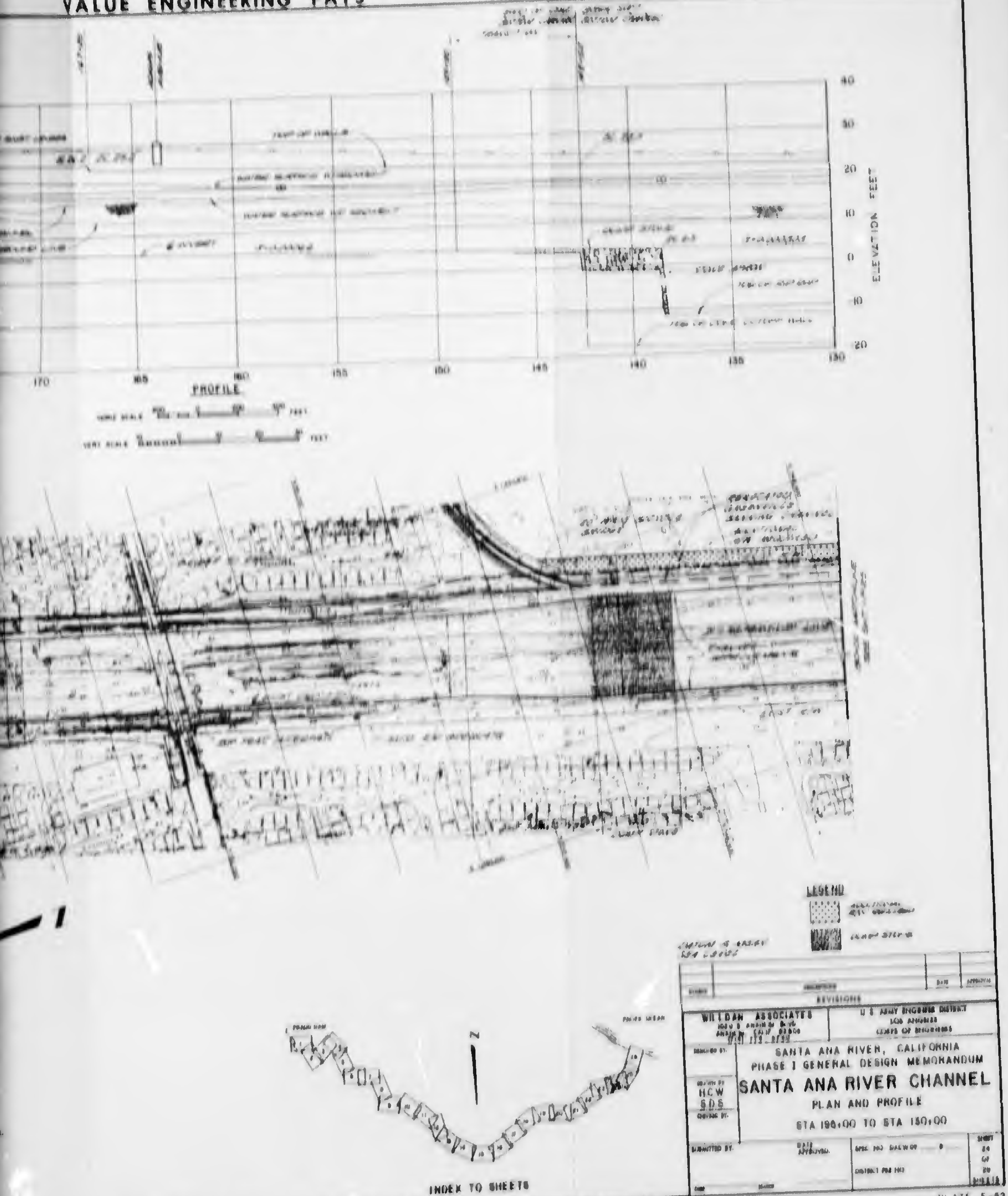

TRANSITION
TOP OF EXIST LEVEES
EL 27.0
TOP OF WALLS
WATER SURFACE W/ SEDIMENT
WATER SURFACE W/O SEDIMENT
EXIST. GROUND ℄ CHANNEL
GENERAL GROUND LINE
℄ INVERT
S=0.0005
S=0.00066
EL 7.0
40
30
20
10
0
-10
-20
195
190
185
180
175
170
165
160
155
PROFILE
HORIZ SCALE
VERT. SCALE
FEET




CHANNEL REFERENCE LINE
WATER SURFACE
BERM
CONCRETE CHANNEL
INVERT
GENERAL GROUND LINE
CONC. CUTOFF WALL
X-SECTION
NTS

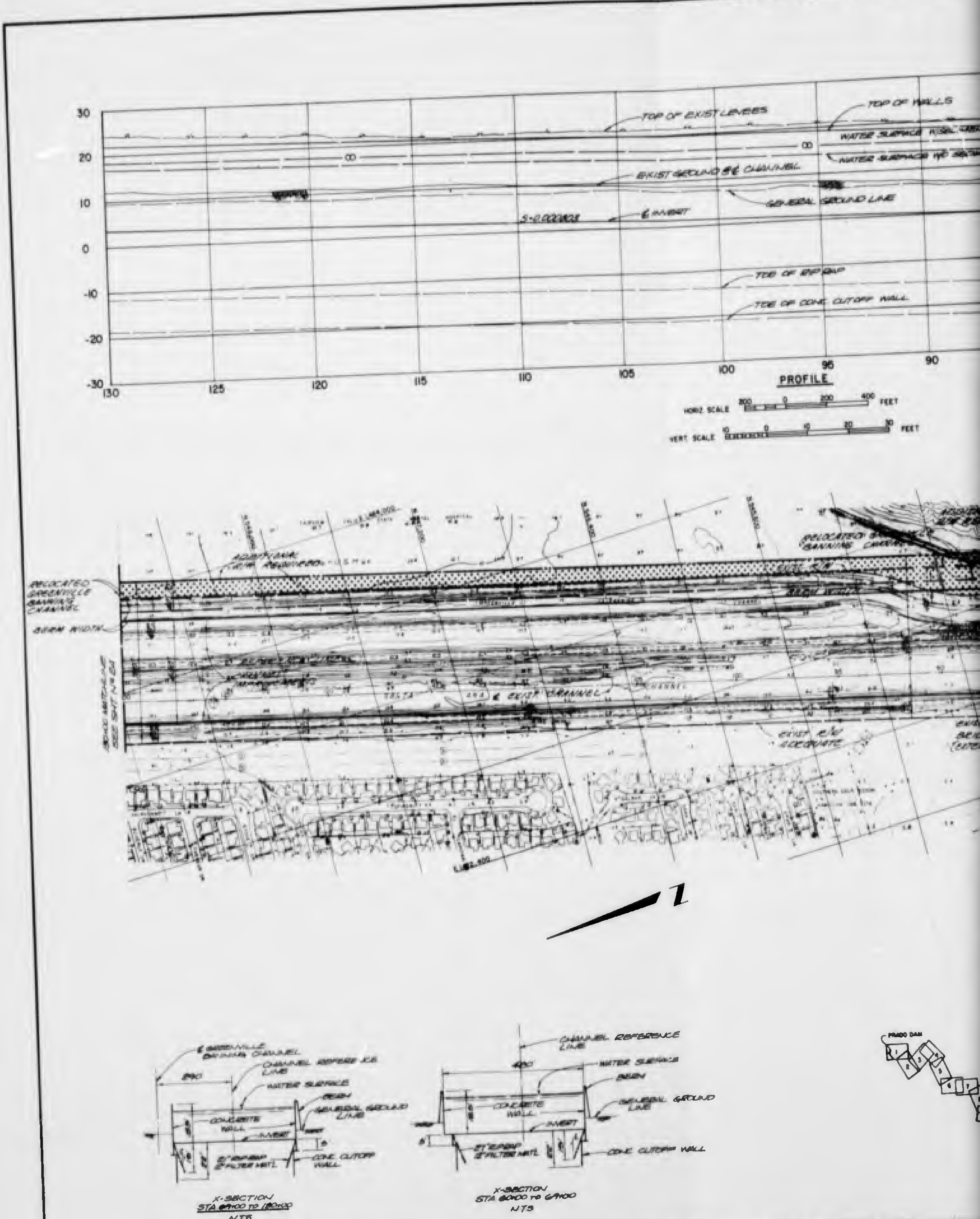


VALUE ENGINEERING PAYS

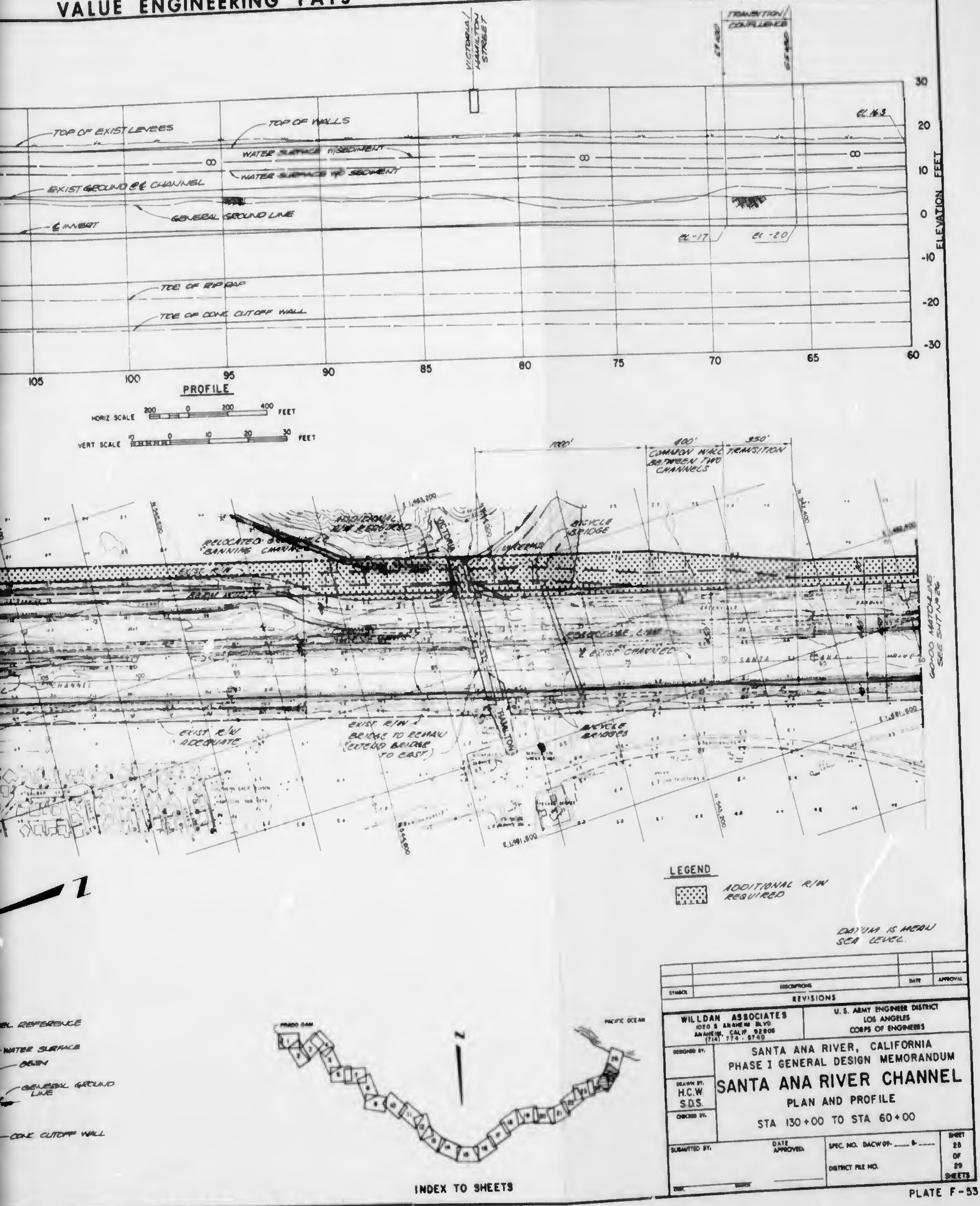


SAFETY PAYS

2


TOP OF EXIST LEVEES
TOP OF WALLS
WATER SURFACE W/SCL
WATER SURFACE W/O
EXIST GROUND @ ℄ CHANNEL
GENERAL GROUND LINE
S=0.000203
℄ INVERT
TOE OF RIP RAP
TOE OF CONC CUTOFF WALL
30
20
10
0
-10
-20
-30
130
125
120
115
110
105
100
95
90
PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
RELOCATED GREENVILLE BANNING CHANNEL
BERM WIDTH
RELOCATED BANNING CHANNEL
EXIST CHANNEL
EXIST R/W ADEQUATE
℄ GREENVILLE BANNING CHANNEL
CHANNEL REFERENCE LINE
WATER SURFACE
BERM
GENERAL GROUND LINE
CONCRETE WALL
INVERT
CONC CUTOFF WALL
X-SECTION
STA 69+00 TO 130+00
NTS
CHANNEL REFERENCE LINE
WATER SURFACE
BERM
GENERAL GROUND LINE
CONCRETE WALL
INVERT
CONC CUTOFF WALL
X-SECTION
STA 60+00 TO 69+00
NTS
PRADO DAM

VALUE ENGINEERING PAYS



SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

WILLDAN ASSOCIATES
1020 S ANAHEIM BLVD
ANAHEIM, CALIF 92806
(714) 774-8740

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

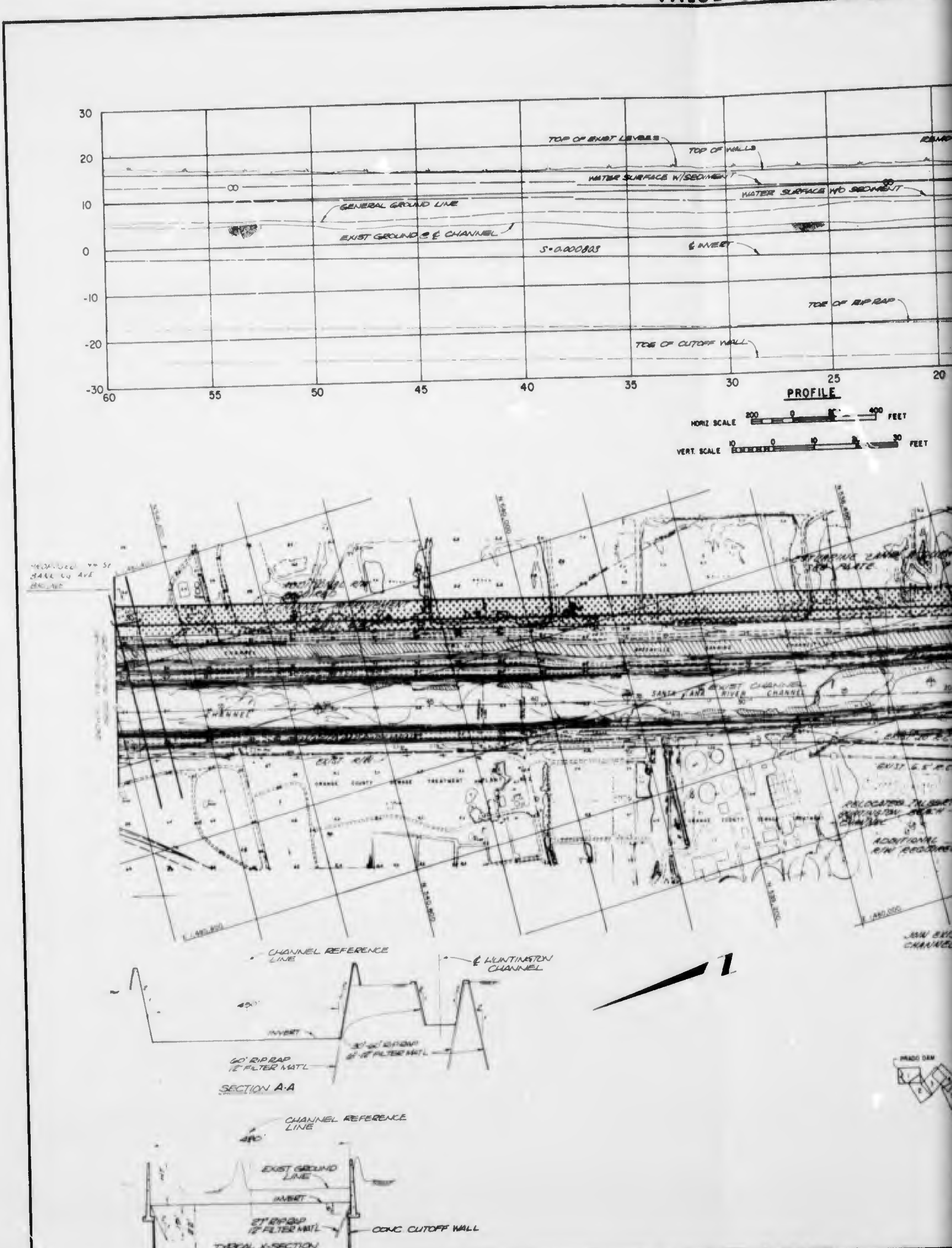
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SANTA ANA RIVER CHANNEL
PLAN AND PROFILE
STA 130+00 TO STA 60+00

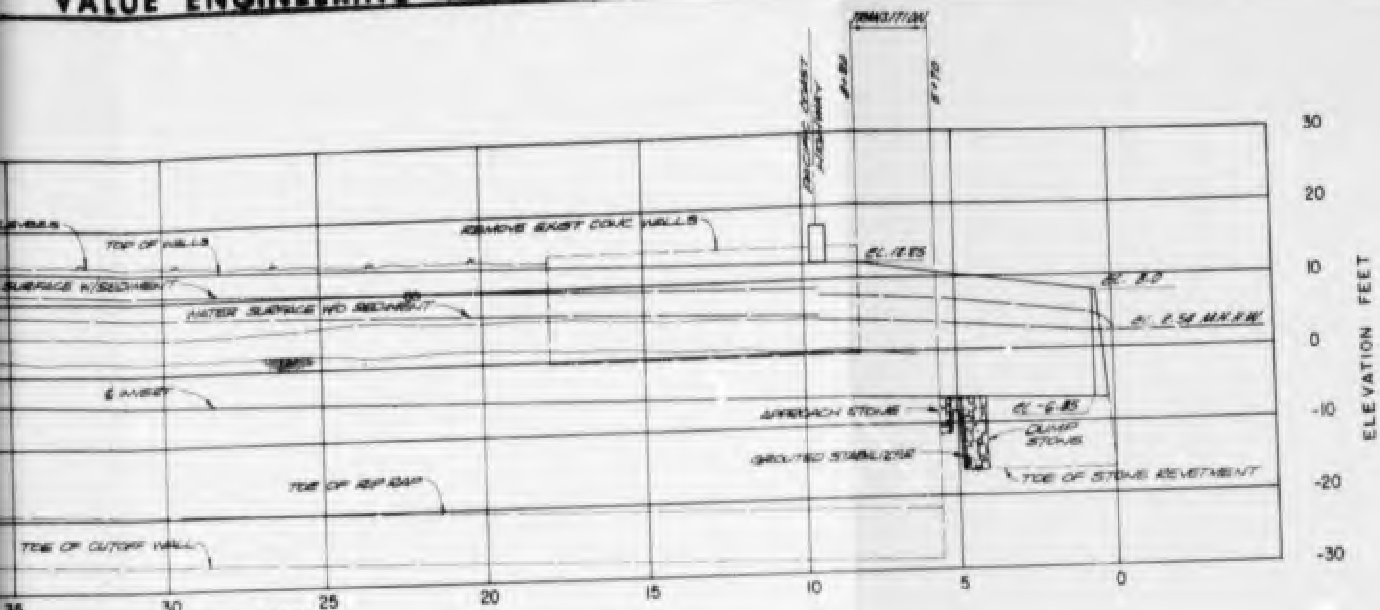
DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET 25 OF 29 SHEETS

PLATE F-53

VALUE ENGINEERING PAY



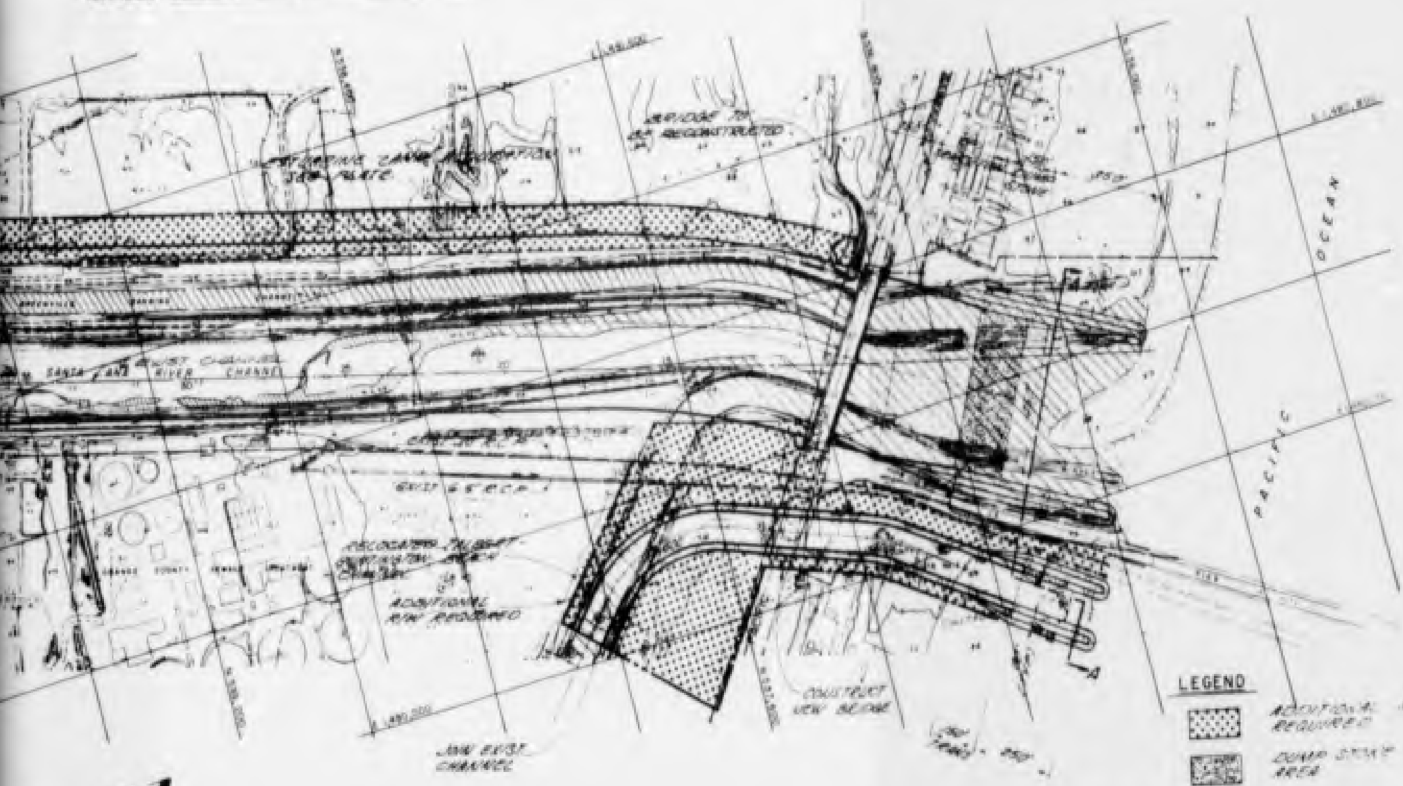
SAFETY PAYS



PROFILE

HORIZ SCALE 200 0 200 400 FEET

VERT SCALE 10 0 10 20 30 FEET



DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

WILLDAN ASSOCIATES
1020 S. ANAHEIM BLVD
ANAHEIM, CALIF. 92805
(714) 774-5740

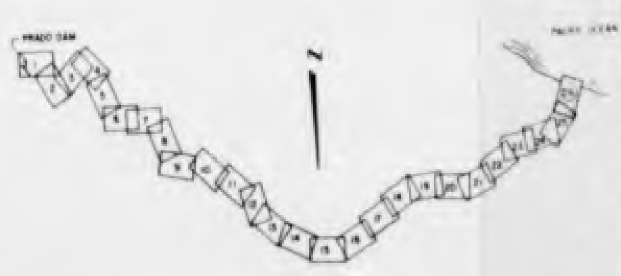
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
SANTA ANA RIVER CHANNEL
PLAN AND PROFILE
STA. 60+00 TO STA. 0+00

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

SHEET 26 OF 29 SHEETS



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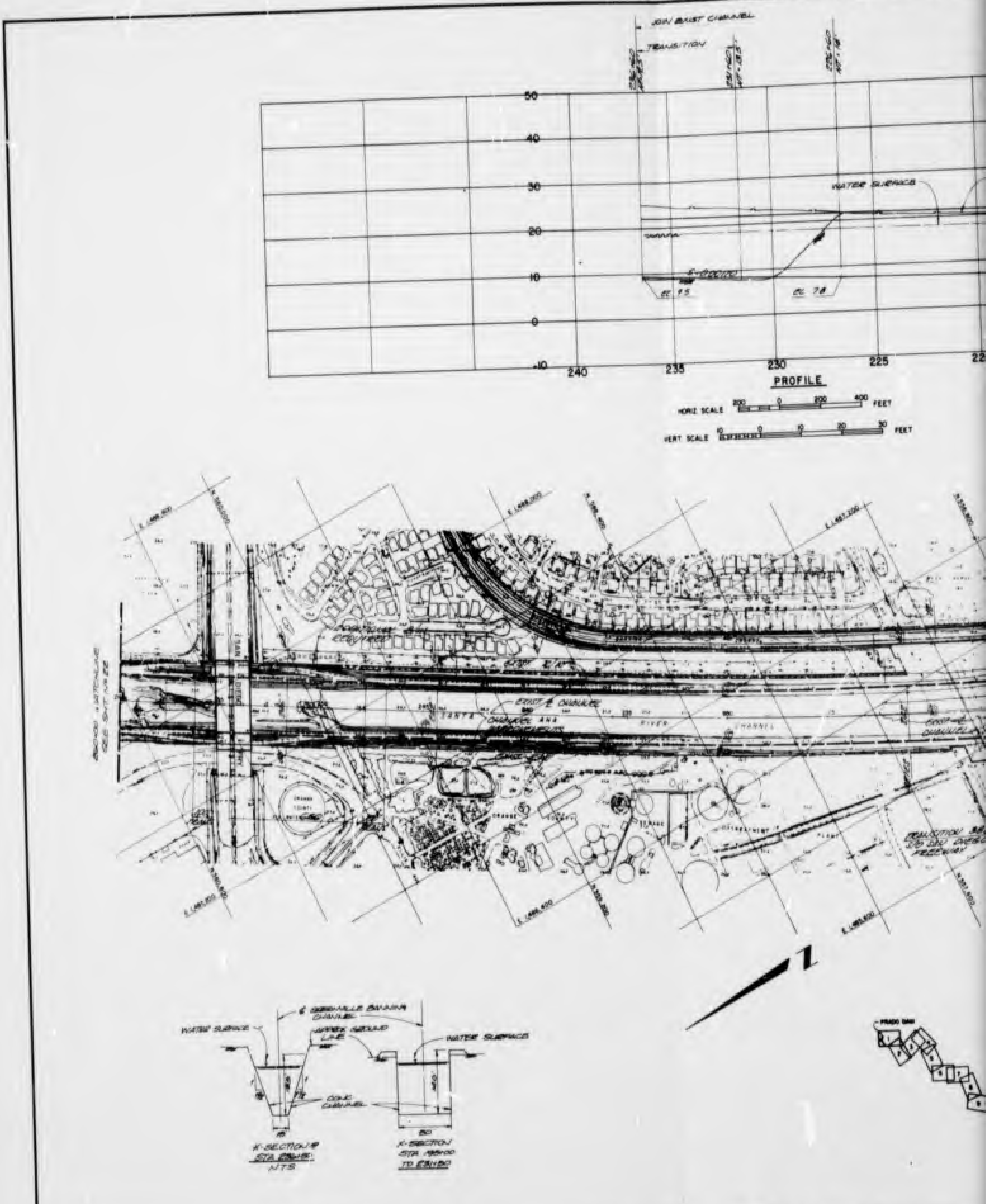
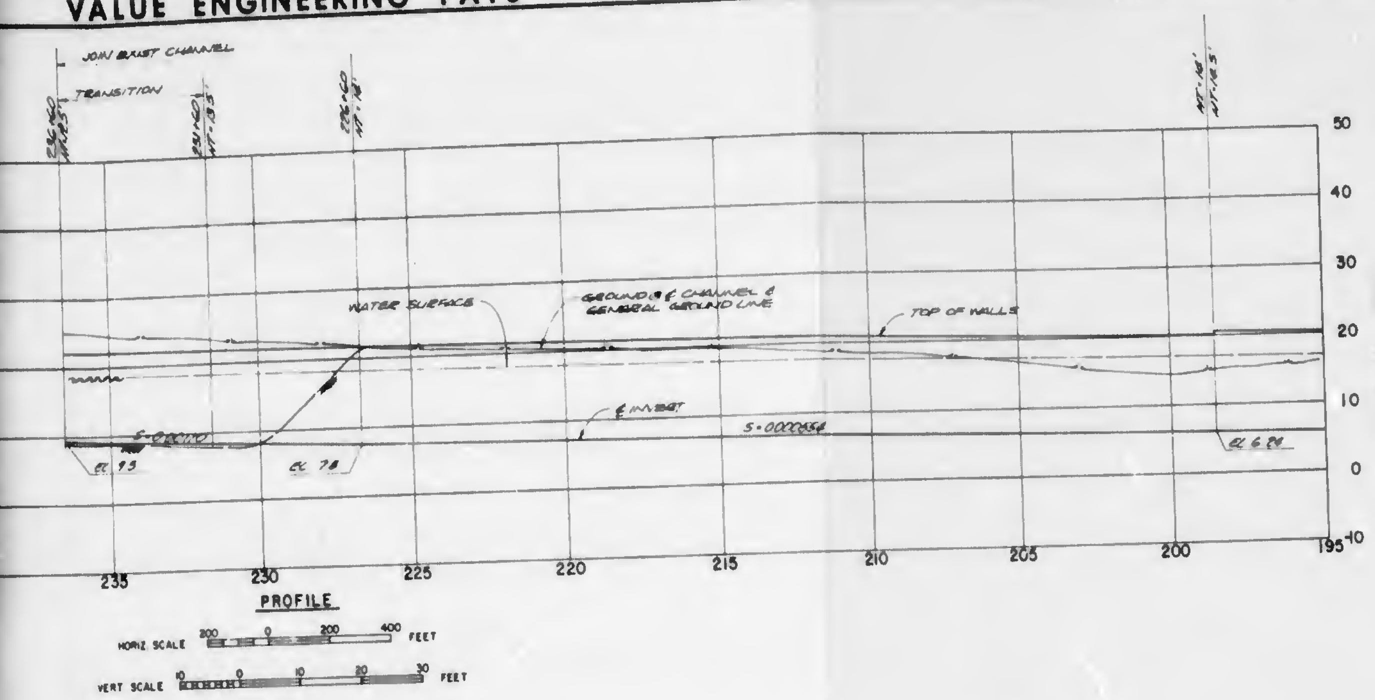
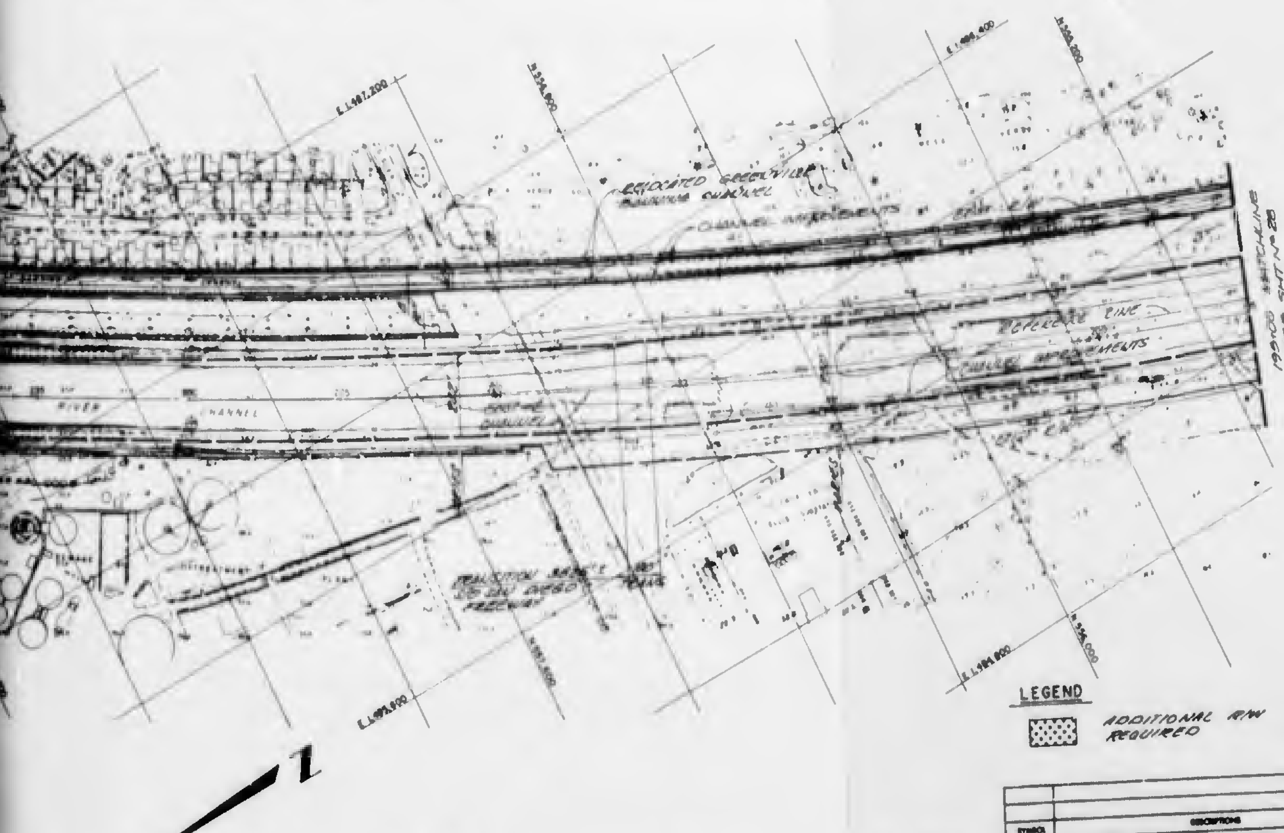

TRANSITION
WATER SURFACE
50
40
30
20
10
0
-10
240
235
230
225
PROFILE
HORIZ SCALE
FEET
VERT SCALE
FEET
SANTA
RIVER
CHANNEL
WATER SURFACE
WATER SURFACE
X-SECTION
X-SECTION

VALUE ENGINEERING PAYS



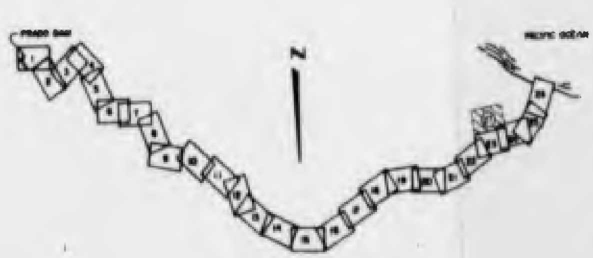
PROFILE



LEGEND

ADDITIONAL R/W REQUIRED

DATUM IS MEAN SEA LEVEL



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REVISIONS

WILLDAN ASSOCIATES
1050 S. ANAHEIM BLVD
ANAHEIM, CALIF 92805
(714) 774-9740

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

DESIGNED BY:
DRAWN BY: H.C.W. S.D.S.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
GREENVILLE BANNING CHANNEL
PLAN AND PROFILE
STA 240 + 00 TO STA. 195 + 00

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW 09-
DISTRICT FILE NO.
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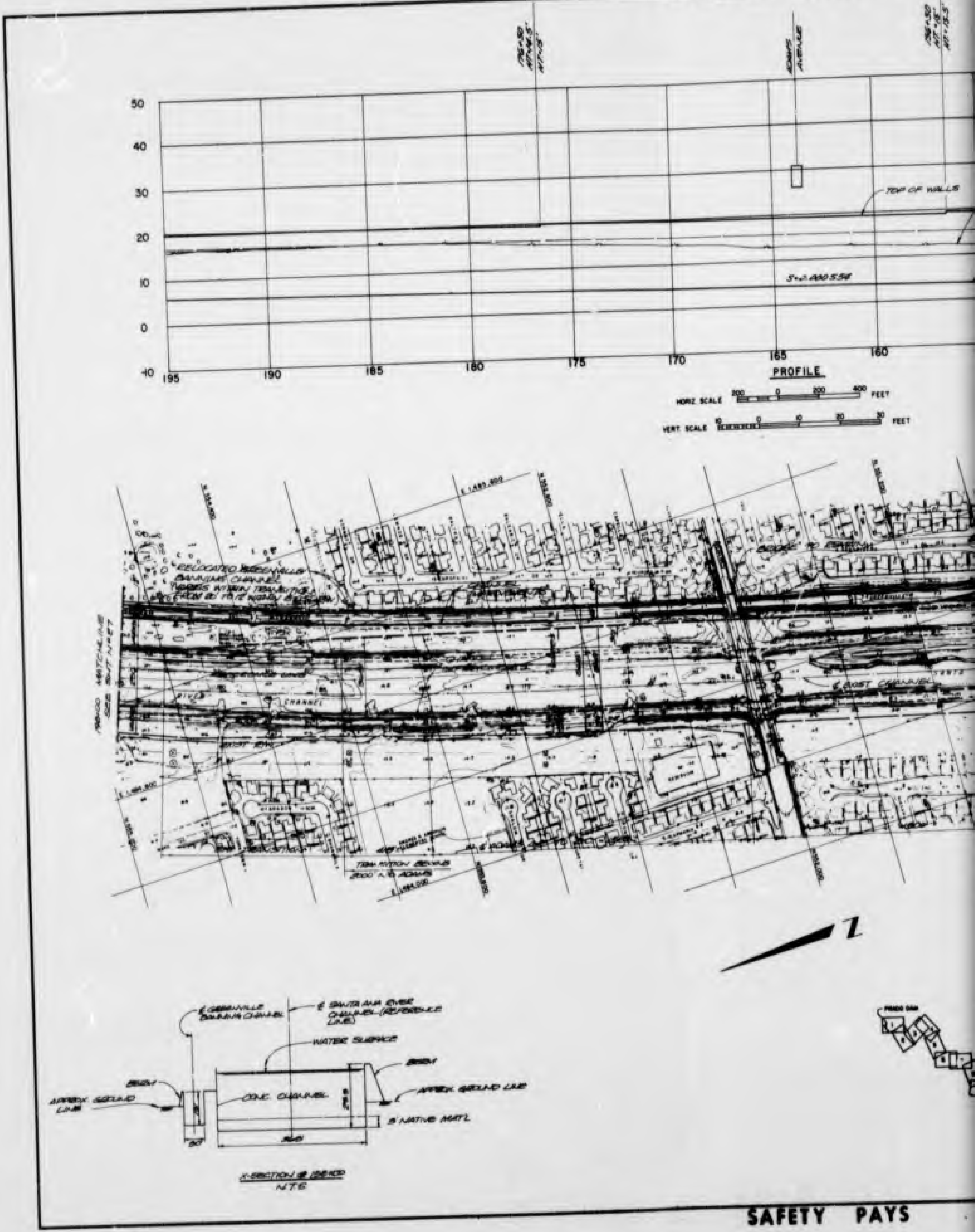
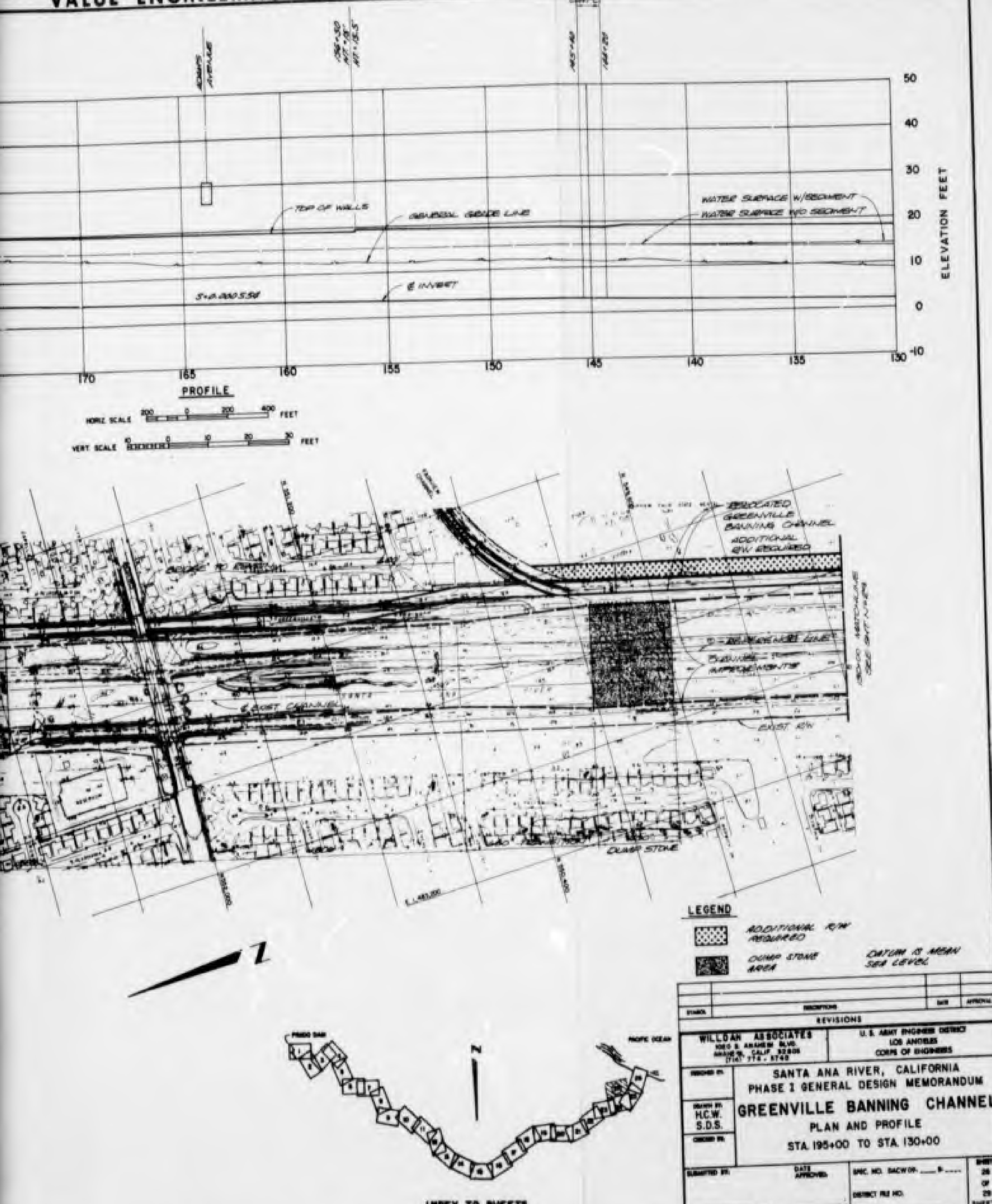
PLATE F-55

SAFETY PAYS



50
40
30
20
10
0
-10
195
190
185
180
175
170
165
160
TOP OF WALLS
PROFILE
HORIZ SCALE
FEET
VERT SCALE
FEET
CHANNEL
WATER SURFACE
BERM
APPROX. GROUND LINE
CONC. CHANNEL
NTS


PROFILE
HORIZ SCALE 200 0 200 400 FEET
VERT SCALE 10 0 10 20 30 FEET
ELEVATION FEET
50
40
30
20
10
0
-10
170
165
160
155
150
145
140
135
130
TOP OF WALLS
GENERAL GRADE LINE
WATER SURFACE W/SEDIMENT
WATER SURFACE W/O SEDIMENT
℄ INVERT
RELOCATED GREENVILLE BANNING CHANNEL
ADDITIONAL R/W REQUIRED
EXIST CHANNEL
EXIST R/W
DUMP STONE
LEGEND
ADDITIONAL R/W REQUIRED
DUMP STONE AREA
DATUM IS MEAN SEA LEVEL
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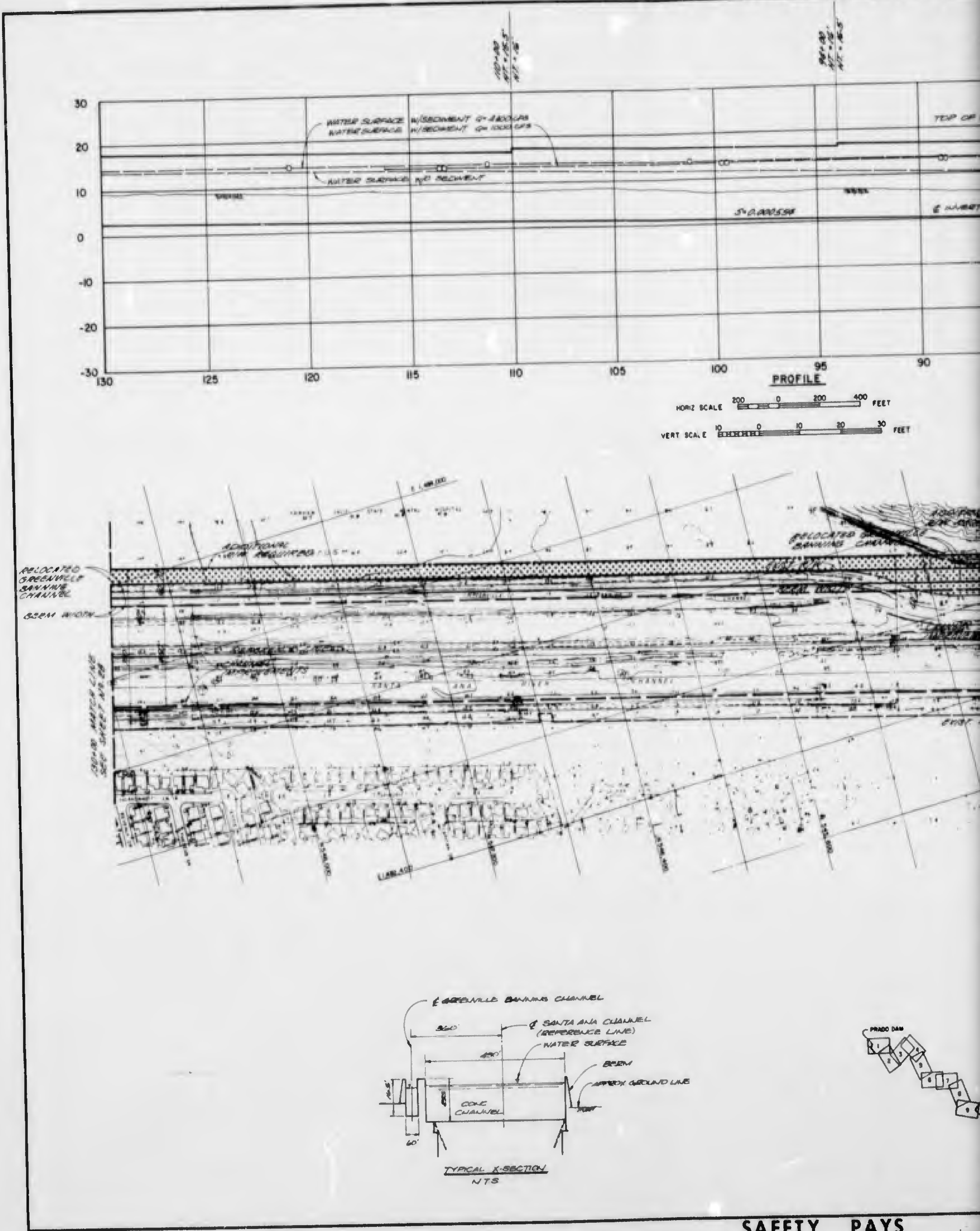
PACIFIC OCEAN
N
REVISIONS
WILLDAN ASSOCIATES
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM
H.C.W.
S.D.S.
GREENVILLE BANNING CHANNEL
PLAN AND PROFILE
STA. 195+00 TO STA. 130+00
SHEET 28 OF 29 SHEETS

PLATE F-56

2



WATER SURFACE W/SEDIMENT
WATER SURFACE W/O SEDIMENT
PROFILE
HORIZ SCALE
FEET
VERT SCALE
FEET
RELOCATED GREENVILLE BANNING CHANNEL
BERM WIDTH
ADDITIONAL LEVEE REQUIRED
RELOCATED BANNING CHANNEL
GREENVILLE BANNING CHANNEL
SANTA ANA CHANNEL (REFERENCE LINE)
WATER SURFACE
BERM
APPROX GROUND LINE
CONC CHANNEL
TYPICAL X-SECTION
NTS
PRADO DAM

VALUE ENGINEERING PAYS

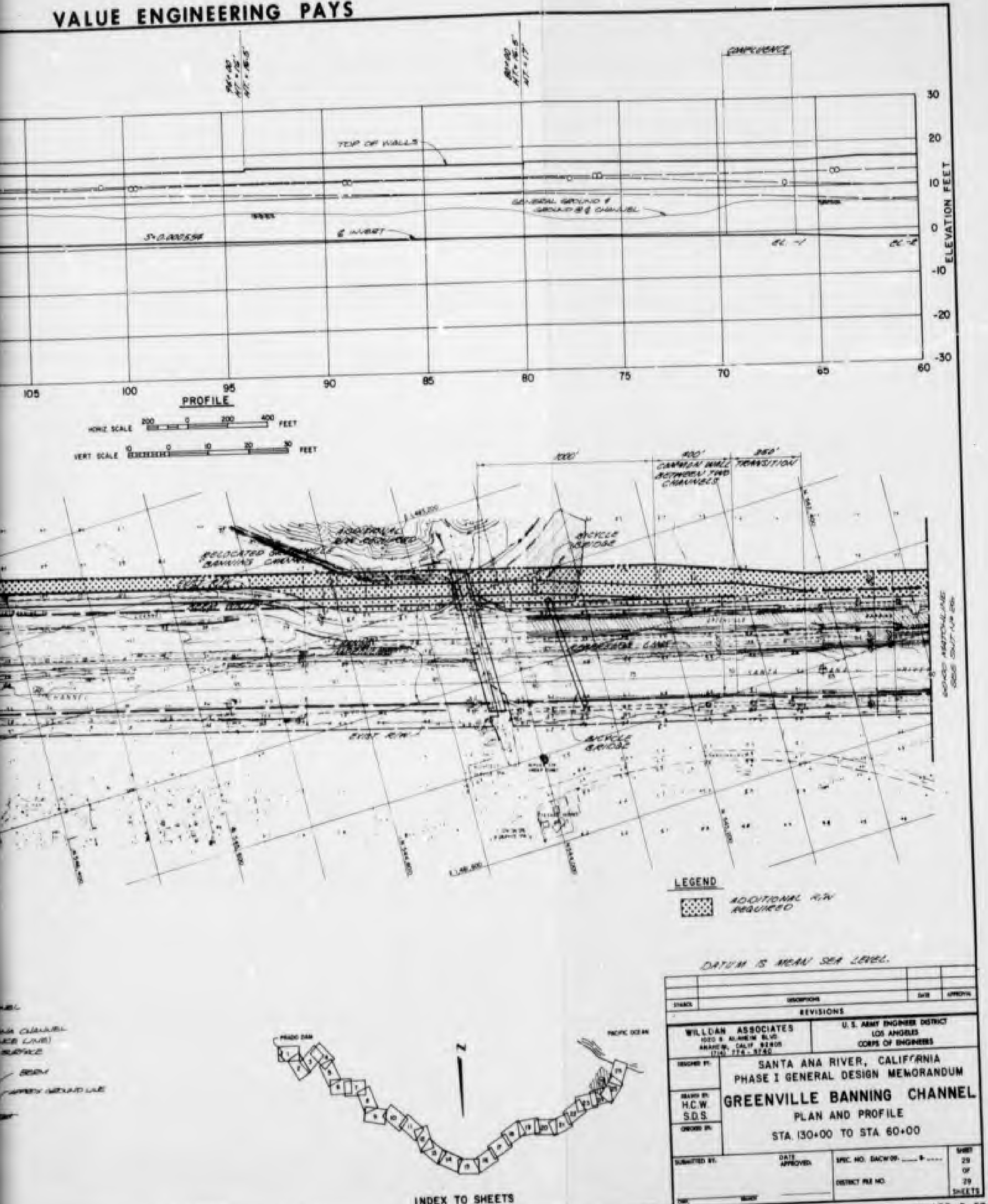


PLATE F-57

SAFETY PAYS