Attachment 3
EXISTING CONDITION HYDROLOGY AND HYDRAULICS MEMORANDUM

Flood Control District of Maricopa County

El Rio Watercourse Master Plan and Area Drainage Master Plan

Contract FCD 2001C024
Stantec Project No. 82000240

April 2003
Revised December 2005

Stantec

FINAL REPORT

HYDROLOGY AND HYDRAULICS MEMORANDUMS

This notebook contains hydrology and hydraulics memorandums that have been developed to date for the El Rio Watercourse Master Plan. The Field survey Memorandum describes the results of comparing field survey to the projects topographic mapping was written after the completion of the Flow Breakout Memo

Memorandums presented are:

1. **FEMA Hydraulic Model Memorandum**

   Describes the hydraulic analysis conducted for the FEMA FIS study of the Salt and Gila Rivers.

2. **Hydrology Memorandum**

   Describes the hydrology used for hydraulic evaluations conducted of the El Rio Project.

3. **Flow Breakout Memorandum**

   Memo describes analysis conducted to quantify/define the hydraulic conditions in the flow breakout area.

4. **Field Survey Memorandum**

   The Field Survey Memorandum describes the results of comparing field survey to the projects topographic mapping. Results of the comparison suggest that there is a topographic bubble in the mapping in the vicinity of the flow breakout area.

El Rio Watercourse Master Plan

FEMA Hydraulic Model Memorandum

PREPARED FOR:

FLOOD CONTROL DISTRICT
of
Maricopa County

PREPARED BY:

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April 2002
Revised November 2005
STANTEC CONSULTING Inc.
Project No. 8200240

Contract FCD 2001C024

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GILA AND SALT RIVERS FIS HYDRAULIC MODEL SUMMARY

GENERAL

Baseline hydraulic data for the El Rio Watercourse Master Plan is taken from *the Gila and Salt Rivers Flood Insurance Study* (Baker, 1999). That study, herein referred to as the FEMA Study, was authorized in 1992 by the Flood Control District of Maricopa County and extends from 1.4 miles downstream of State Route 85 on the Gila River to the Granite Reef Dam on the Salt River. Originally the downstream limits of the study were at Gillespie Dam on the Gila River. The original study was broken into 5 reaches. A change in scope resulted in the majority of the lower reach, Reach 1, being removed from the study. The remainder of Reach 1 was added to Reach 2. The El Rio Watercourse Master Plan (ERWMP) study reach extends from State Route 85 to the confluence with the Agua Fria and Gila Rivers and lies entirely within Reach 2 of the FEMA study as shown in Figure 1. The data, procedures and results for Reach 2 of the FEMA Study were reviewed in regard to the purposes of the ERWMP. The following is a summary of the key elements of the FEMA Study hydraulic model.

TOPOGRAPHIC MAPPING

Topographic mapping was prepared as part of the FEMA Study contract by Michael Baker Jr. Inc. and McLain Harbors Company, Inc. Initial mapping was prepared from circa 1991 and 1992 aerial photography at a scale of 1 inch = 400 feet with a contour interval of 4 feet. Portions of the main channel were remapped in April 1993 by Michael Baker Jr. Inc. and McLain Harbors Company, Inc after the January 1993 flooding event. The remapped portions are shown in Figure 2. The horizontal datum for both the 1992 and 1993 mapping sets is the North American Datum (NAD) of 1983 on the state plane coordinate system. A statement in the FEMA Study report suggests that the mapping may have been prepared on some other coordinate system and "adjusted" to the state plane coordinate system. The vertical datum for both the 1992 and 1993 mapping sets is the National Geodetic Vertical Datum (NGVD) of 1929 and a conversion factor to NGVD 1988 was provided.

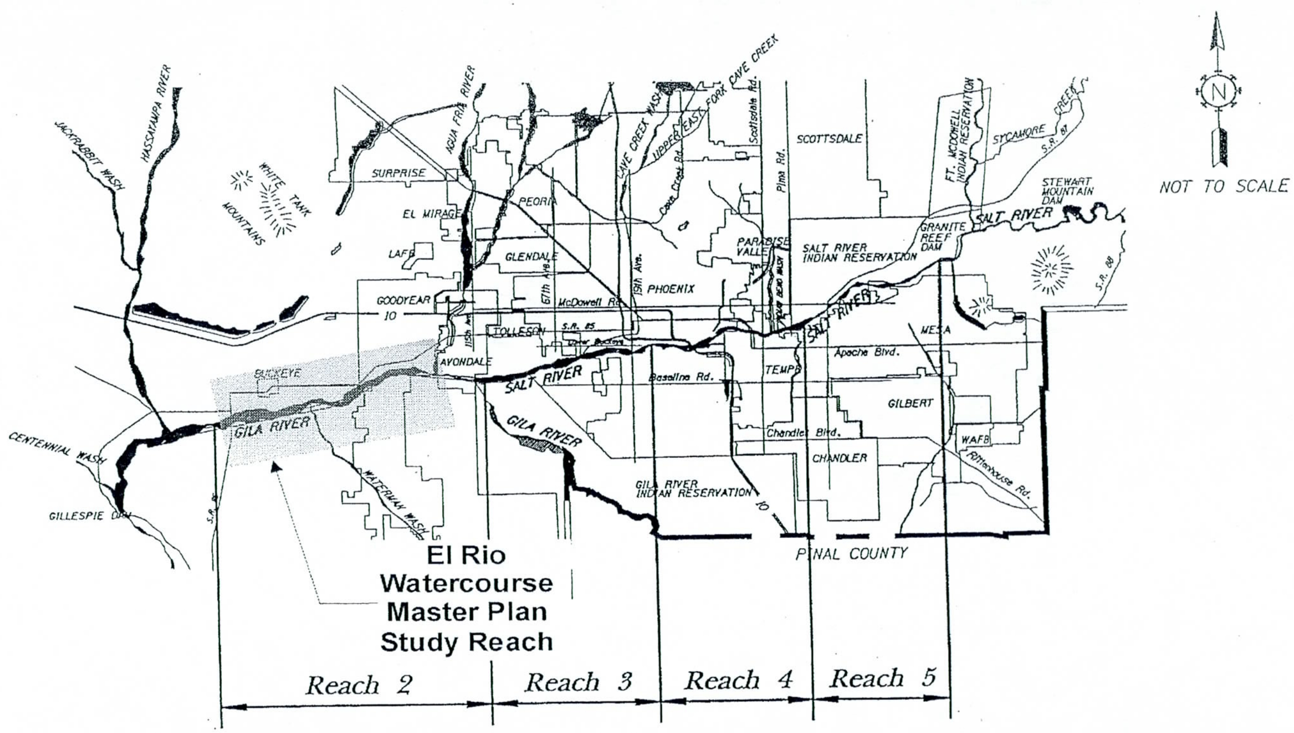


Figure 1-Location Map

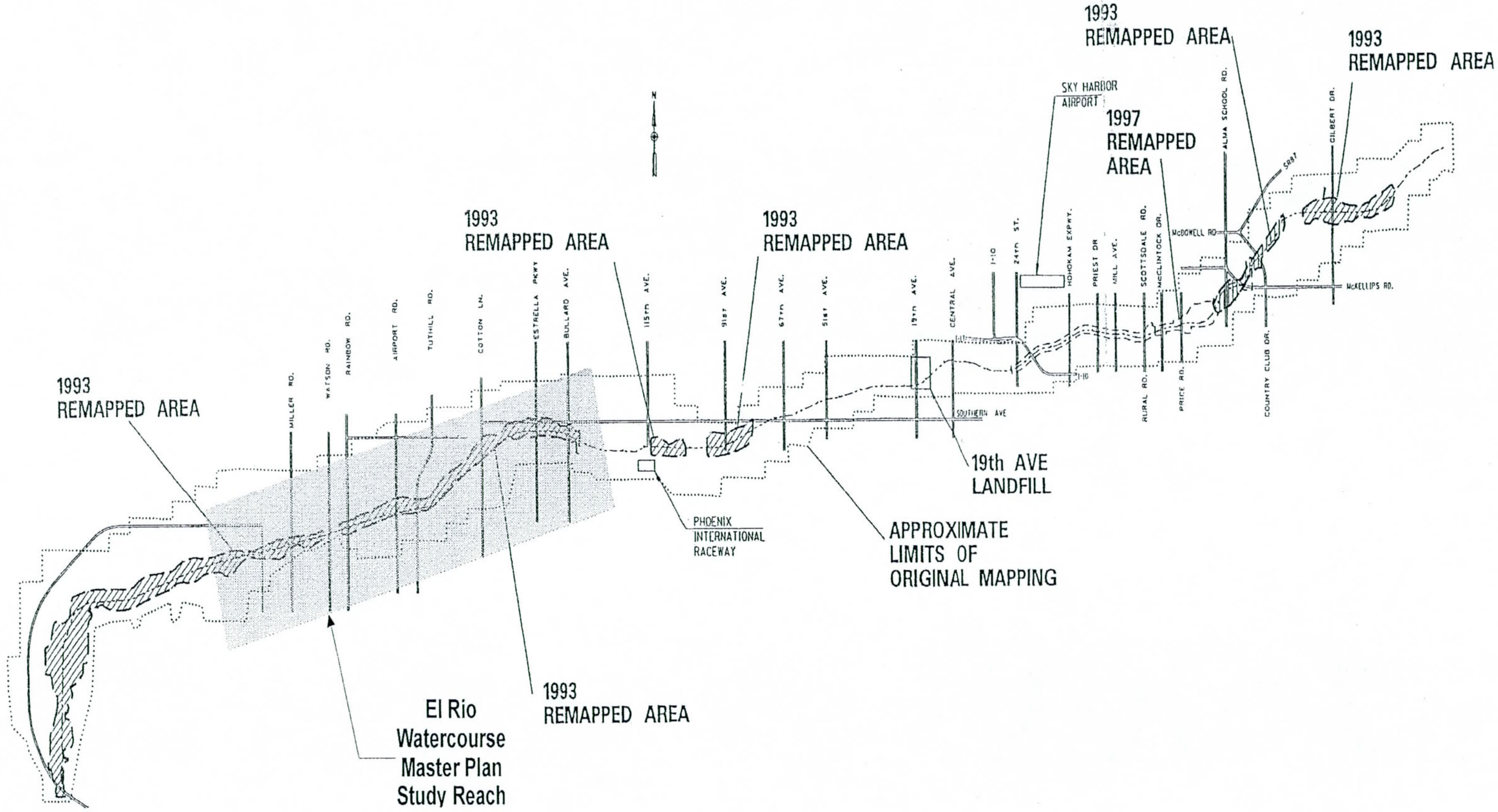


Figure 2-Remapped Areas

ROUGHNESS COEFFICIENTS

Roughness coefficients developed for the FEMA Study are documented in a separate report entitled *"n" Value Report* (Baker 1994). Manning's n-values were estimated using circa 1991 and 1993 aerial photography, ground photographs and video and the 1 inch = 400 foot, 4 foot contour interval mapping. Ground photograhs and videos were taken during field reconnaissance trips on 9 April and 18 November 1992 and 6 May and 16 June 1993.

The base Manning's n-value for the channel assumes a movable bed condition in the upper flow regime. The channel bed material was categorized as medium to course grain sand with an n-value of 0.025. The base Manning's n-value for the overbank area assumes stable flow conditions. The overbank bed material was categorized as firm soil with an n-value of 0.025. The cultivated portions of the floodplain were assumed to be out of season. That assumption was based on a 100-year flood event that would occur in mid-winter to late spring.

The base n-values were adjusted to account for channel irregularity and vegetation. Vegetation was categorized based on density and each cross section was subdivided according to the categories. Six vegetation density categories were used and an adjustment value was assigned to each category as follows:

- Dense = 0.15
- Medium-dense = 0.036
- Medium = 0.014
- Light-medium = 0.009
- Light = 0.004
- Clear = 0.000

For areas of dense vegetation, the vegetation adjustment factor was assigned as the Manning's n-value for that portion of the cross section regardless of other roughness factors. For Reach 2, the channel irregularity adjustment was considered to be minor given that the channel flow width to depth ratio is large. Channel irregularity adjustment values used in the analysis ranged between 0.001 and 0.008. Channel irregularity was not considered to vary significantly between cross sections, therefore values were selected for every fifth cross section for each vegetation density category present. Those values were then averaged for each vegetation density category except dense and clear. The total Manning's n-value for each subsection of a cross section is the sum of the base Manning's n-value, the vegetation adjustment value and the average channel irregularity value for the particular vegetation density category. For the clear vegetation density category, the channel irregularity values selected were added directly to the base Manning's n-value without averaging. The total Manning's n-value for each vegetation density category are as follows.

- Dense = 0.15
- Medium-dense = 0.065 (0.025 + 0.036 + 0.004)
- Medium = 0.043 (0.025 + 0.014 + 0.004)
- Light-medium = 0.037 (0.025 + 0.009 + 0.003)
- Light = 0.032 (0.025 + 0.004 + 0.003)
- Clear = varies between 0.026 and 0.033

A calibration model was developed using data from the February 1980 flood event to validate the Manning's n-values developed for the Gila and Salt Rivers FIS. Thirty-three typical sections for the entire reach (71 miles) were used in the calibration. The conclusion of the calibration study was that the Manning's n-values were, in general, reasonable given, among other things, the changes in topography since the February 1980 flood.

HYDROLOGY

Peak discharges used in the hydraulic analysis were taken from the March 1996 U.S. Army Corps of Engineers report for the modifications to Roosevelt Dam entitled *Gila River Basin, Arizona, Section 7 Study for Modified Roosevelt Dam, Arizona, Hydrologic Evaluation of Water Control Plans, Salt River Project to Gila River at Gillespie Dam.* The peak discharges, along with the previous effective model discharges are listed in Table 1.

Table 1 – Summary of Discharges

Location	River Station miles	100-Year Peak Discharge Effective cfs	100-Year Peak Discharge Revised cfs
Granite Reef Dam	237.59	245,000	175,000
Gilbert Road	213.55	230,000	172,000
Tempe Bridge	221.24	215,000	169,000
Central Avenue	213.24	200,000	166,000
67th Avenue	205.40	190,000	164,000
Below Confluence with Gila River	199.82	250,000	227,000
Below Confluence with Waterman Wash	186.10	245,000	210,000
Below Confluence with Hassayampa River	174.81	240,000	203,000

HYDRAULICS

General

Hydraulic modeling was accomplished using the U.S. Army Corps of Engineers HEC-RAS version 2.1 computer program. Each reach is a separate file. For Reach 2, two models were developed for with and without levee conditions. Floodway limits were initially established using the Method 4 encroachment option in HEC-RAS with a target surcharge of one foot. The Method 4 option results were then converted to Method 1 encroachment stations for refinement.

Bridges

There are five bridges crossing the Gila River in Reach 2 of the FEMA Study model. Those locations are State Route 85, Tuthill Road, Estrella Parkway, Bullard Avenue and

115th Avenue. Bridge data used in the model was obtained from field surveys. Flow transitions through the bridges, including the selection of cross section locations were modeled based on the guidelines and procedures in Appendix B of the *HEC-RAS Hydraulic Reference Manual*. Detailed analyses of the flow contraction and expansion conditions were performed for each bridge, however, the results of those analyses were not provided. The determination of contraction and expansion ratios is also related to the selection of contraction and expansion loss coefficients. Inspection of the model data shows that only at two locations, State Route 85 and Estrella Parkway Bridges, were contraction and expansion loss coefficients other than for gradual transitions (0.1 and 0.3) used. At these locations a contraction coefficient of 0.3 was applied to several cross sections both upstream and downstream of the two bridges.

Special Problems

State Route 85 Bridge

The current bridge (completed sometime after 1993) replaced an existing, low-water crossing at this location. The old bridge had a span of 744 feet compared to a span of approximately 3,600 feet for the new. Bank stations at the bridge were set at the bridge abutments, however, the main channel upstream and downstream of the bridge is considerably more narrow. Therefore, channel bank stations immediately upstream and downstream of the bridge were selected by "tappering-in" from the bridge opening to the main channel. Downstream of the bridge, floodway calculations using the Method 4 option of HEC-RAS allowed encroachment almost to the channel bank stations. This condition was determined to be unacceptable therefore encroachment limits were widened and set based on expected changes in the hydraulic character of the river over time.

Dirt Road Upstream of Watson Road

The dirt road on the north side of the river is slightly elevated and parallels the main channel. Ground elevations on the north side of the river are approximately 1 foot lower than the base flood elevation in the main channel. Therefore, the road was modeled as a levee and also assuming that the roadway embankment would fail.

Effective flow limits for river stations 184.62 through 184.14 were set using a contraction ratio of 1:1. Effective flow limits for river stations 185.10 through 184.62 were set using an expansion ratio of 1:1. Comparison of the with and without levee models showed a maximum water surface differential of 0.02 feet. Base flood elevations were shown as continuous from the channel to the right overbank and were based on the without levee model.

Tuthill Road/Jackrabbit Trail

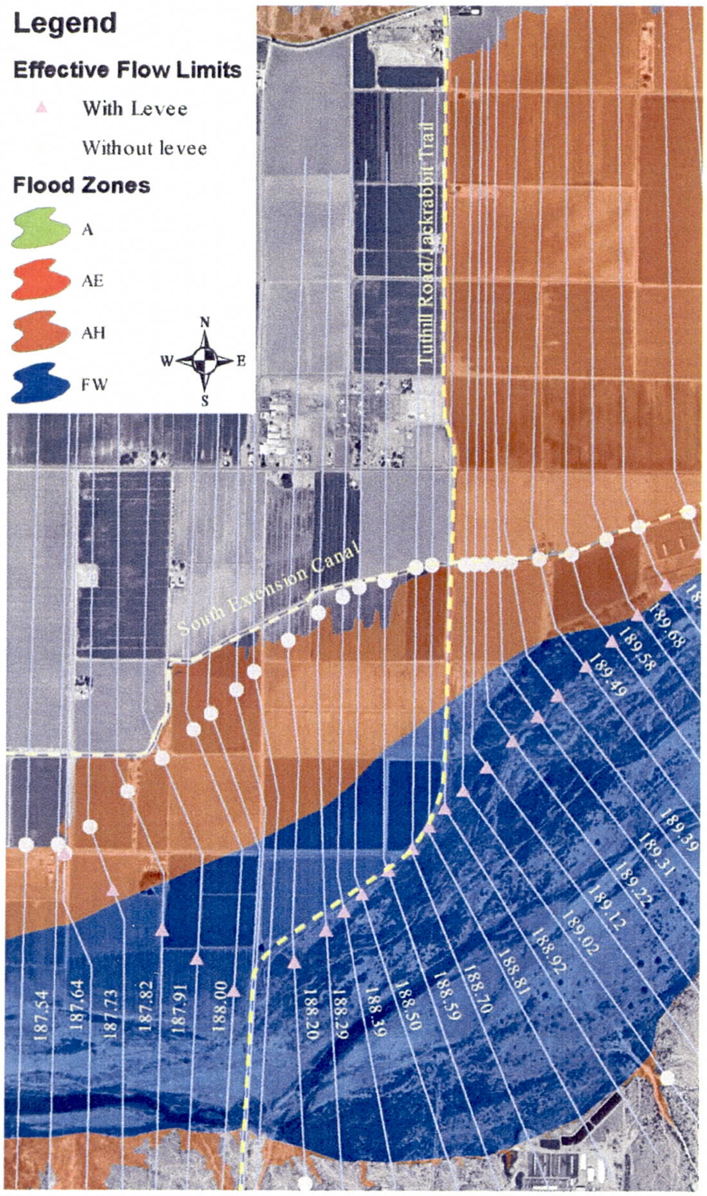
The Tuthill Road bridge is located at river station 188.055. Immediately north of the bridge, the Tuthill Road alignment angles to the northeast. After a short distance it turns due north again as Jackrabbit Trail, perpendicular to the direction of flow as shown in Figure 3. From the bridge to approximately the South Extension Canal crossing, the road is elevated above natural ground and therefore it is model as a levee and also assuming that the embankment will fail.

For the without levee condition, the road is assumed to fail, but the canal is assumed to remain in place. Modeling of the roadway failure was accomplished by removing the cross section that defines the roadway, river station 188.92. Roadway geometry between river stations 188.10 and 188.81 was left in the model. Effective flow limit stationing from river stations 187.54 to 188.39 (upstream and downstream of the bridge) was determined assuming a 1:1 contraction and expansion rate. From river stations 188.39 to 190.05, effective flow limits were set at the South Extension Canal.

For the with levee model, the roadway is assumed to remain in place. Effective flow limits for this condition were set based on the guidelines and procedures in Appendix B of the *HEC-RAS Hydraulic Reference Manual*. Results of the with levee condition model show that the South Extension Canal will be overtopped at a depth of approximately 0.1 feet. Therefore, the canal is treated as a levee that has failed. An overtopping discharge of 900 cfs was estimated using the weir equation with a total length of 970 feet (720 feet at river station 190.05 and 250 feet at river station 189.77), a depth of 0.5 feet (a statement is made in the report that the mapping is not of the accuracy that allows for measurement of 0.1 feet) and a coefficient of discharge of 2.6. Flow overtopping the canal will travel west, overtop Jackrabbit Trail at average depths less than 1 foot and then continue west flowing at average depths less than 1 foot. Flow overtopping Jackrabbit Trail is not mapped given the shallow depth and constantly varying terrain of the agricultural area.

Comparison of the with and without levee models shows lower water surface elevations for the without levee condition due to wider effective flow limits. The without levee model was used to set base flood elevations in the right overbank downstream of Jackrabbit Trail between river stations 188.92 and 188.04. The with levee model was used to set base flood elevations for the channel and left overbank from river station 188.04 to 190.05. The Tuthill Road/Jackrabbit Trail alignment north of the bridge to the South Extension Canal crossing serves as the gutter line between the with levee and without levee conditions. Base flood elevations to the north of the canal are based on the with levee model since effective flow limits would be the same at this location as for the with levee condition for the Tuthill Road/Jackrabbit Trail. Floodplain limits north of the canal were mapped based on the assumption that the canal has failed and the projection of the water surface elevation, not on a hydraulic analysis of the limits of flooding resulting from the 900 cfs overtopping discharge.

Figure 3 - Tuthill Road Location Map



King Ranch Levee

The "King Ranch Levee" extends from the Estrella Parkway bridge (river station 194.195) abutment to river station 193.71 on the south side of the river. Levee certification has never been obtained for this structure, therefore it was modeled as a levee and also assuming that the embankment will fail. Effective flow limits at this location are the same for both the with levee and without levee conditions. This results in identical water surface elevations for each condition.

Vineyard Road

Vineyard Road is an elevated road that runs along the south bank of the river from Estrella Parkway east past Bullard Avenue. The roadway was modeled as a levee and also assuming that the roadway embankment will fail.

For the without levee condition, effective flow limits were set using a 4:1 expansion ratio from river stations 195.66 to 194.53 and a contraction ratio of 1:1 into the Estrella Parkway bridge. This model was used to set base flood elevations south of the road between river stations 195.00 and 195.56. For the with levee condition, effective flow limits were set using guidelines and procedures in Appendix B of the *HEC-RAS Hydraulic Reference Manual*. For this condition, the roadway is overtopped at river stations 194.29, 194.40, 194.81 and 194.91. This model was used to set base flood elevations between river stations 194.21 and 195.00.

El Rio Watercourse Master Plan

Hydrology Memorandum

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El Rio Watercourse Master Plan

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EL RIO HYDROLOGY

GENERAL

The Gila River basin at its confluence with the Colorado River near Yuma is approximately 58,200 square miles in size and encompasses the majority of southern Arizona as well as southern New Mexico west of the continental divide and a small portion of northern Mexico. The major tributaries to the Gila River are listed in Table 1. The El Rio Watercourse Master Plan (WMP) study limits extend approximately 17 miles upstream from State Route 85 (approximately 5 miles upstream of the Hassayampa River) to the Agua Fria River and is shown in relation to the Gila River basin in Figure 1.

Table 1

Major Tributaries of the Gila River

Tributary	Contributing Area sq. miles
Salt and Verde Rivers	13,700
Santa Cruz River	8,600
San Pedro River	4,500
San Francisco River	2,800
San Simon River	2,200
Agua Fria River	2,000
Centennial Wash	1,800
Hassayampa River	1,462
San Carlos River	1,027
Waterman Wash	420

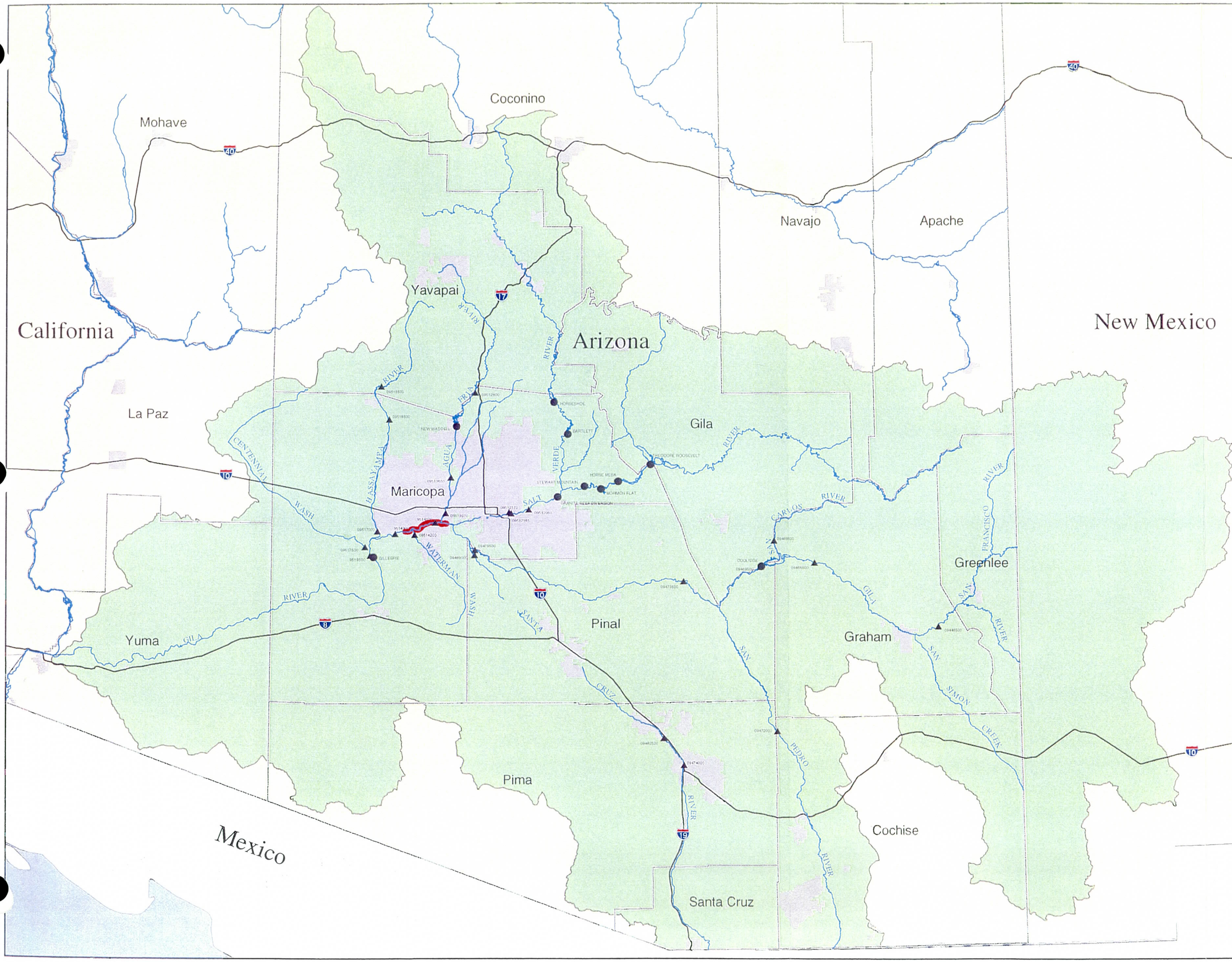
The Gila River basin area at the El Rio WMP study limits is approximately 46,000 square miles in size. Hydrometeorologic and physiographic characteristics of the watershed are highly varied. The upper reaches of the watershed (northern and northeastern portions) consist of rugged, mountainous terrain with incised or canyonized watercourses, many of which have perennial flow. Elevations in these areas are generally greater than 5,000 feet and the climate ranges from semi-arid to cool and humid with annual precipitation reaching as much as 30 inches. The lower reaches of the watershed can be characterized as a basin and range physiographic region with braided, alluvial watercourses of intermittent flow. Elevations in these areas are generally less than 1,000 feet in the basins with the mountain ranges typically not exceeding 4,000 feet. The climate is mostly arid with annual precipitation as low as 4 inches.

Flooding along the El Rio WMP study limits is controlled, to a certain degree, by the numerous dams within the Gila River watershed. Many of the dams have flood control as a primary or secondary purpose, however only a few have a significant impact on flooding within the study reach. The most significant dams, in regard to flooding within the study limits, are the Salt River Project dams, Coolidge Dam and New Waddell Dam. The locations of these dams are shown in Figure 1. The drainage area controlled by each of these dams as well as the total available storage capacity is listed in Table 2. The total area controlled by these dams is approximately 24,000 square miles, nearly half of the total watershed area at the study limits. Significant streamflow in the upper region (upstream of the dams) is generally produced in the winter months (December through March) as a result of frontal or convergence storm activity of large areal extent lasting several days and is usually associated with some degree of snowmelt. Significant streamflow in the lower region is generally produced during the summer months (June through October) as a result of convective storm activity of lesser areal extent and duration than frontal or convergence storm activity. Streamflow produced in these two regions is, therefore, not coincident. Based on the nature of the hydrometeorological conditions of the watershed, flooding events of significance within the study limits occur during the winter months from streamflow produced in the upper region.

Table 2
Key Dams within the Gila River Watershed Upstream of the Study Area

Dam	Total Storage Capacity acre-feet	Dedicated Flood Pool Capacity acre-feet	Contributing Area sq. miles	Purpose
Roosevelt	2,100,000	557,000	5,830	Irrigation/Power
Horse Mesa	245,000	---	5,935	Irrigation/Power
Mormon Flat	58,000	---	6,095	Irrigation/Power
Stewart Mountain	70,000	---	6,221	Irrigation/Power
Horseshoe	131,000	---	5,657	Irrigation
Bartlett	249,700	71,700	5,872	Irrigation
Coolidge	1,100,000	---	12,886	Irrigation/Power
New Waddell	1,108,000	---	1,459	Irrigation

Source: *Section 7 Study for Modified Roosevelt Dam, Arizona, Hydrologic Evaluation of Water Control Plans, Salt River Project to Gila River at Gillespie Dam* (COE 1996).



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EL RIO WATERCOURSE MASTER PLAN
FCD 2001C024

Consultants:

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Notes:

Approximately 1,000 square miles of the watershed limits outside of the U.S.

Client/Project:

El Rio

Title:

FIGURE 1
WATERSHED MAP

Produced by: BGS

Date: 10/21/2002

FLOOD HISTORY

Streamflow measurements along the Gila River and its tributaries are recorded at numerous locations by the United States Geological Survey (USGS). Locations of gaging stations of interest that document the history and nature of flooding events along the study reach are shown in Figure 1. The period of record and contributing drainage area for each of these gaging stations are listed in Table 3.

Table 3

Select USGS Gaging Stations within the Gila River Watershed

Gage No.	Gage Name	Drainage Area (sq. miles)	Period of Record
09448500	Gila R. at head of Safford valley, AZ	7,896	1914 - Present
09466500	Gila River at Calva, AZ	11,740	1916 - Present
09468500	San Carlos River near Peridot, AZ	1,026	1916 - Present
09469500	Gila River below Coolidge Dam, AZ	12,886	1914 - Present
09472000	San Pedro River near Redington, AZ	2,927	1926 - 1997
09473500	San Pedro River n. Winkleman, AZ	4,453	1919 - 1984
09474000	Gila River at Kelvin, AZ	18,011	1912 - Present
09479500	Gila River near Laveen, AZ	20,615	1916 - 1995
09482500	Santa Cruz River at Tucson, AZ	2,222	1915 - Present
09486500	Santa Cruz River at Cortaro, AZ	3,503	1940 - Present
09489000	Santa Cruz River at Leveen, AZ	8,581	1940 - Present
09512060	Salt R. at Alma School Rd	13,000	1991 - 1993
09512165	Salt R. at Priest Drive	13,200	1995 - Present
09512170	Salt River at Jointhead Dam	13,200	1890 - 1980*
09512800	Agua Fria River n. Rock Springs, AZ	1,111	1919 - Present
09513650	Agua Fria River at El Mirage, AZ	1,628	1963 - 1998
09513970	Agua Fria River at Avondale, AZ	2,013	1959 - 1982
(FCDMC)	Agua Fria River at Buckeye	2,241	1988 - Present
09514100	Gila River at Estrella Pkwy	45,585	1993 - Present
09514200	Waterman Wash near Buckeye, AZ	420	1964 - Present
09515500	Hassayampa R. at Box Damsite	417	1925 - 1982
09516500	Hassayampa R. at Morristown, AZ	796	1939 - Present
09517000	Hassayampa River at Arlington, AZ	1,471	1961 - Present
09517500	Centennial Wash near Arlington, AZ	1,810	1961 - 1979
09519500	Gila River below Gillespie Dam	49,650	1921 - Present

A time line representing the recorded flood history for the study reach is shown in Figure 2. The data shown in that figure consists of mean daily and instantaneous discharge records for the gaging station at the Gila River below Gillespie Dam (09519500). This station is used to represent the flood history for the study reach because it is the closest station to the study reach with the longest period of record. The basin area at Gillespie Dam includes the Hassayampa River and Centennial Wash watersheds, which have a combined area of approximately 3,300 square miles. Both of these watersheds can be characterized as basin and range physiographic regions with braided, alluvial watercourses of intermittent flow. Significant streamflow from these watersheds is typically produced during the summer months resulting from rainfall in the upper portions of the watersheds. Because of the nature of the watercourses, there is significant attenuation of the peak discharge such that it becomes insignificant in regard to flood flows in the Gila River, which typically occur in the winter months. Therefore, inclusion of streamflow produced in the Hassayampa River and Centennial Wash watersheds does not impact interpretations of the recorded flood history for the study area. This can be demonstrated by inspection of the gage data for some of the more significant events. The maximum recorded peak discharges along with the date of occurrence for water years 1966, 1970, 1978, 1979, 1980, 1984, 1993 and 1995 are listed in Table 4 for each gaging station shown in Figure 1. Inspection of that table shows that, in general, the major contribution of streamflow at Gillespie Dam is produced in the Salt River watershed. For example, the maximum recorded discharge for the 1979 water year in the Salt River just upstream of the confluence with the Gila River (09512170) was 126,000 cfs occurring on 19 December 1978. The maximum recorded discharge for that water year at Gillespie Dam was 125,000 cfs occurring on 20 December 1978. The maximum recorded discharges in the Hassayampa River (09517000) and Centennial Wash (09517500) for that water year were 3,300 cfs on 11 November 1978 and 818 cfs on 17 January 1979, respectively.

In general, streamflow in the study reach occurs only after long duration rainfall events of large areal extent. The three most significant recorded events occurred in December 1978, February 1980 and January 1993. The flooding of 1978 resulted from a tropical storm that moved across the state dumping large quantities of rainfall over a period of several days. The major reservoirs in the watershed were already near capacity from the unusually wet 1977 – 78 season and large releases were necessary. The maximum discharge recorded at Gillespie Dam (09517500) for this storm was 125,000 cfs. The flooding of 1980 was a result of a series of tropical storms that moved across the state dumping as much as 13 inches of rainfall in the upper portion of the watershed over a ten-day period. The maximum recorded discharge at Gillespie Dam for this storm was 178,000 cfs, which is the largest recorded discharge for the period of record. The flooding of 1993 was a result of a series of winter storms beginning in December 1992 that resulted in record breaking snowpack throughout the state. In January 1993, 15 days of rainfall combined with the rapidly melting snow to fill the major reservoirs that were already near capacity. The maximum estimated discharge at Gillespie Dam for this storm was 130,000 cfs.

Figure 2
Hydrologic Time Line

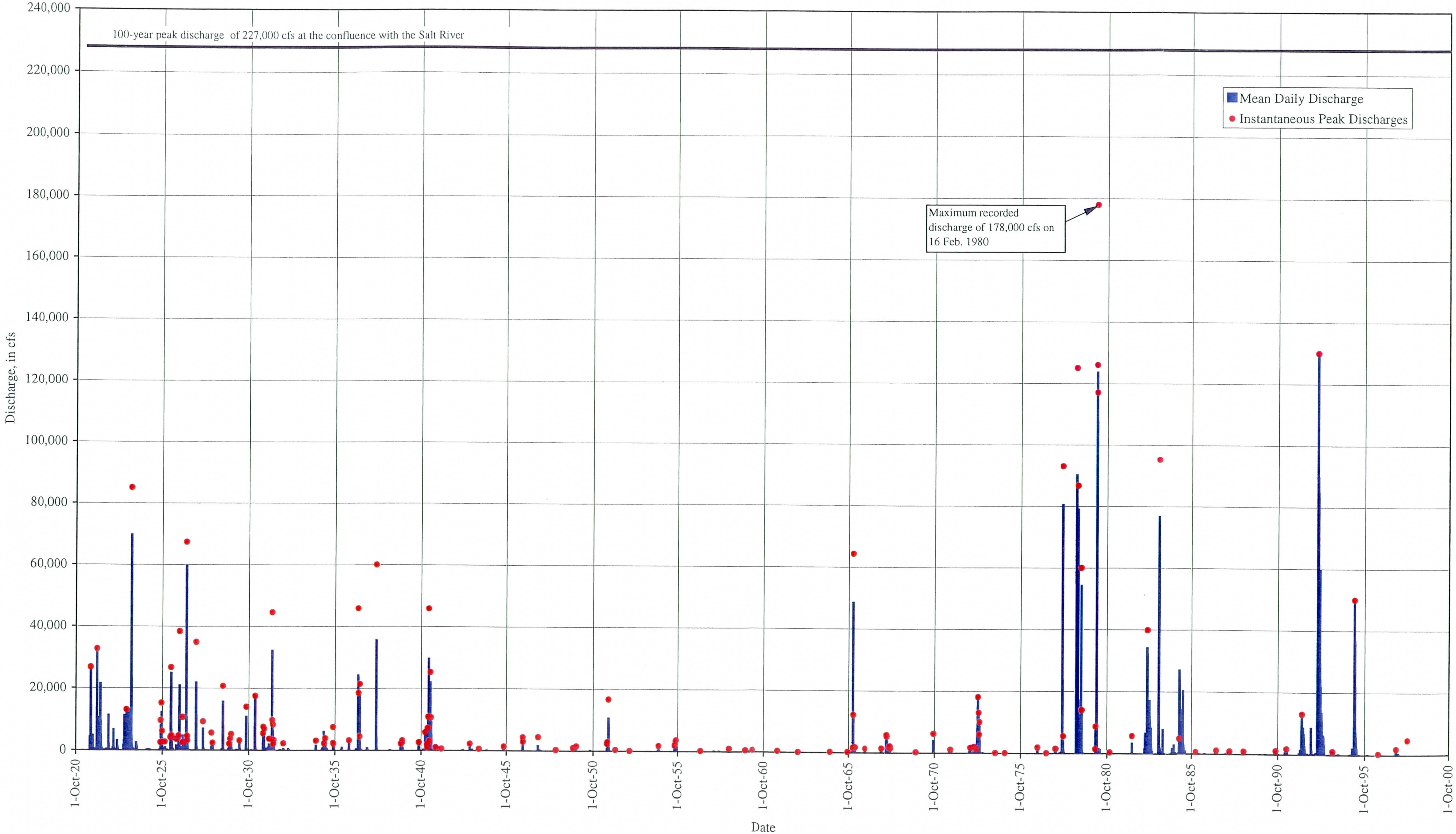


Table 4

Maximum Peak Discharge for Select Stations within the Gila River Watershed

Gaging Station		Drainage Area	Period of Record	Maximum Peak Discharge															
				1966		1970		1978		1979		1980		1984		1993		1995	
Number	Name	sq. mi.		Date	Q	Date	Q	Date	Q	Date	Q	Date	Q	Date	Q	Date	Q	Date	Q
09448500	Gila R. at head of Safford valley, AZ	7,896	1914 - Present	22-Dec-65	43,000	6-Aug-70	2,250	2-Mar-78	21,600	19-Dec-78	100,000	16-Feb-80	25,300	2-Oct-83	132,000	19-Jan-93	86,200	5-Jan-95	62,400
09466500	Gila River at Calva, AZ	11,740	1916 - Present	24-Dec-65	39,000	3-Mar-70	982	4-Mar-78	19,000	19-Feb-78	100,000	16-Feb-80	20,600	3-Oct-83	150,000	20-Jan-93	109,000	6-Jan-95	64,500
09468500	San Carlos River near Peridot, AZ	1,026	1916 - Present	22-Dec-65	36,300	6-Sep-70	5,080	2-Mar-78	18,600	18-Dec-78	22,500	15-Feb-80	12,300	1-Oct-83	10,300	8-Jan-93	54,800	5-Jan-95	27,900
09469500	Gila River below Coolidge Dam, AZ	12,886	1914 - Present	16-Jul-66	954	18-Jul-70	697	14-Jul-78	811	29-Mar-79	1,460	11-Mar-80	2,540	7-Oct-83	4,960	21-Jan-93	29,300	9-Mar-95	4,000
09472000	San Pedro River near Redington, AZ	2,927	1926 - 1997	29-Jul-66	5,890	21-Jul-70	8,490	10-Oct-77	23,000	18-Jan-79	10,800	14-Aug-80	392	2-Oct-83	25,400	19-Jan-93	19,100	5-Jan-95	5,970
09473500	San Pedro River n. Winkleman, AZ	4,453	1919 - 1984	22-Dec-65	16,800	3-Mar-70	6,340	10-Oct-77	16,000	18-Dec-78	18,000	15-Feb-80	2,900	---	135,000	---	---	---	---
09474000	Gila River at Kelvin, AZ	18,011	1912 - Present	23-Dec-65	26,300	3-Mar-70	6,600	11-Oct-77	16,100	19-Dec-78	27,000	15-Feb-80	6,950	2-Oct-83	100,000	19-Jan-93	74,900	6-Jan-95	19,700
09479500	Gila River near Laveen, AZ	20,615	1916 - 1995	26-Dec-65	10,900	5-Mar-70	178	13-Oct-77	6,360	21-Dec-78	9,720	23-Feb-80	545	4-Oct-83	35,000	20-Jan-93	41,600	7-Jan-95	13,090
09482500	Santa Cruz River at Tucson, AZ	2,222	1915 - Present	19-Aug-66	5,500	20-Jul-70	8,530	10-Oct-77	23,700	19-Dec-78	13,500	13-Aug-80	2,760	2-Oct-83	52,700	19-Jan-93	37,400	16-Feb-95	576
09486500	Santa Cruz River at Cortaro, AZ	3,503	1940 - Present	22-Dec-65	16,800	20-Jul-70	11,200	10-Oct-77	23,000	18-Dec-78	18,800	19-Jul-80	2,650	2-Oct-83	65,000	---	---	15-Feb-95	6,170
09489000	Santa Cruz River at Leveen, AZ	8,581	1940 - 2003	26-Dec-65	2,940	9-Sep-70	1,010	13-Oct-77	2,010	22-Dec-78	4,120	20-Feb-80	115	4-Oct-83	33,000	21-Jan-93	11,000	18-Feb-95	726
	Inflow to Roosevelt Dam	5,830	---	22-Dec-65	88,000	5-Sep-70	57,690	2-Mar-78	155,000	18-Dec-78	152,300	15-Feb-80	142,130	2-Oct-83	67,100	8-Jan-93	194,000	---	---
	Inflow to Horseshoe	5,657	---	22-Dec-65	39,300	6-Sep-70	61,900	1-Mar-78	91,400	19-Dec-78	123,000	15-Feb-80	94,800	1-Oct-83	27,200	8-Jan-93	145,000	15-Feb-95	108,000
09512060	Salt R. at Alma School Rd	---	1991 - 1993	---	---	---	---	---	---	---	---	---	---	---	---	8-Jan-93	96,600	---	---
09512165	Salt R. at Priest Drive	13,200	1995 - Present	---	---	---	---	---	---	---	---	---	---	---	---	---	---	16-Feb-95	81,400
09512170	Salt River at Jointhead Dam	13,200	1979 - 1980	31-Dec-65	66,000	5-Sep-70	14,000	2-Mar-78	122,000	19-Dec-78	126,000	16-Feb-80	170,000	---	---	---	---	---	---
09512800	Agua Fria River n. Rock Springs, AZ	1,111	1919 - Present	---	---	5-Sep-70	40,100	2-Mar-78	39,500	18-Dec-78	52,800	19-Feb-80	59,500	17-Aug-84	6,860	8-Jan-93	52,500	15-Feb-95	35,700
09513650	Agua Fria River at El Mirage, AZ	1,628	1963 - 1998	10-Aug-66	2,906	5-Sep-70	5,000	2-Mar-78	9,870	19-Dec-78	58,400	20-Feb-80	41,800	1-Sep-94	1,050	9-Feb-93	7,100	5-Jan-95	77
09513970	Agua Fria River at Avondale, AZ	2,013	1959 - 1982	23-Dec-65	800	6-Aug-70	20,600	2-Mar-78	13,100	19-Dec-78	23,900	20-Feb-80	44,200	---	---	---	---	---	---
(FCDMC)	Agua Fria River at Buckeye	2,241	1988 - Present	---	---	---	---	---	---	---	---	---	---	---	---	11-Jan-93	5,329	17-Feb-95	679
09514100	Gila River at Estrella Pkwy	45,585	1992 - 2004	---	---	---	---	---	---	---	---	---	---	---	---	9-Jan-93	162,000	16-Feb-95	74,900
09514200	Waterman Wash near Buckeye, AZ	420	1964 - Present	13-Sep-66	5,560	9-Aug-70	1,600	4-Aug-78	1,150	---	---	15-Feb-80	2,220	9-Feb-84	3,520	8-Jan-93	670	---	---
09515500	Hassayampa R. at Box Damsite	417	1925 - 1982	10-Dec-65	5,560	5-Sep-70	58,000	2-Mar-78	16,000	28-Mar-79	9,640	19-Feb-80	24,900	---	---	---	---	---	---
09516500	Hassayampa R. at Morristown, AZ	796	1939 - Present	13-Sep-66	3,210	5-Sep-70	47,500	2-Mar-78	18,000	18-Dec-78	9,600	20-Feb-80	17,000	10-Sep-84	26,700	8-Jan-93	26,300	15-Feb-95	20,000
09517000	Hassayampa River at Arlington, AZ	1,471	1961 - Present	10-Dec-65	1,600	5-Sep-70	39,000	2-Mar-78	20,000	11-Nov-78	3,300	20-Feb-80	11,200	2-Sep-84	2,850	8-Jan-93	11,400	15-Feb-95	3,900
09517500	Centennial Wash near Arlington, AZ	1,810	1961 - 1979	13-Sep-66	5,500	5-Sep-70	11,900	2-Mar-78	10,900	17-Jan-79	818	---	---	---	---	---	---	---	---
09519500	Gila River below Gillespie Dam	49,650	1921 - 2001	2-Jan-66	64,200	6-Sep-70	6,180	3-Mar-78	92,900	20-Dec-78	125,000	16-Feb-80	178,000	5-Oct-83	95,200	9-Jan-93	130,000	16-Feb-95	50,000

DATA COLLECTION AND ANALYSIS

The data collected for this study consists of discharge-frequency statistics and streamflow records. Discharge-frequency statistics for the Salt and Gila Rivers were prepared by the U.S. Army Corps of Engineers (COE) as part of the water control plan for the modifications to Roosevelt Dam. Discharge-frequency statistics for the Gila River from the confluence with the Salt River to Gillespie Dam are listed in Table 5. Development of these statistics is documented in the COE report *Section 7 Study for the Modified Roosevelt Dam, Arizona; Hydrologic Evaluation of Water Control Plans, Salt River Project to Gila River at Gillespie Dam* (Mar 1996). In general, discharge-frequency statistics were developed by first estimating inflow hydrographs to the Salt River Project reservoirs and the San Carlos Reservoir. The inflow hydrographs were estimated from measured data and were routed through the reservoirs based on the respective water control plans. Reservoir routing was accomplished using both the HEC-5 computer model and graphical techniques. The resulting peak discharge from the San Carlos Reservoir and at the Granite Reef diversion dam were then routed downstream using the Modified Puls storage routing method. Downstream of the Salt-Gila River confluence, tributary inflow (Agua Fria River, Waterman Wash, Hassayampa River and Centennial Wash) was considered to be non-coincident with peak flow in the Gila River, therefore the peak discharge decreases as the flood wave moves downstream due to channel losses and overbank storage effects.

Table 5

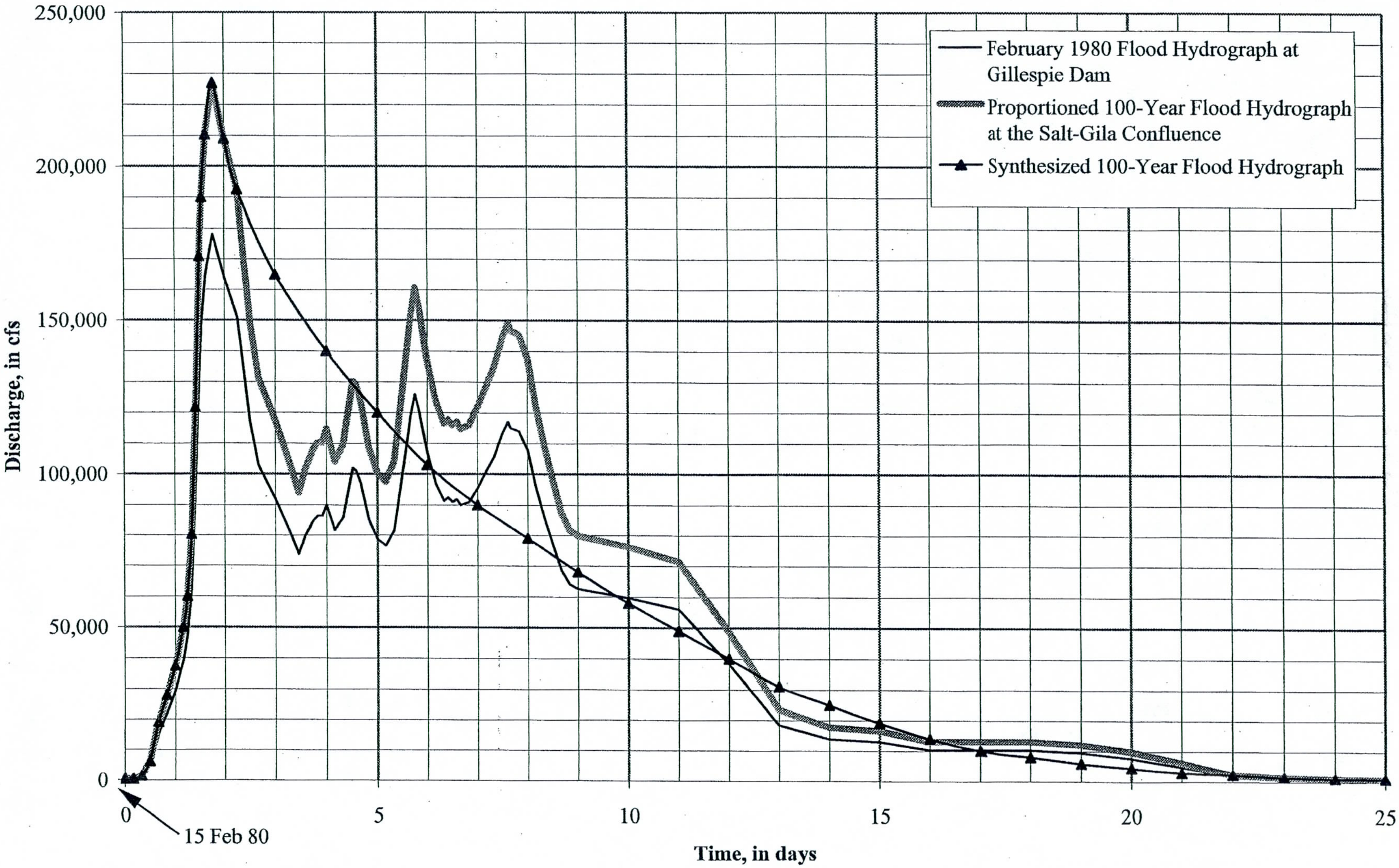
Discharge-Frequency Statistics

	Return Period				
Location	**100-Year**	**50-Year**	**20-Year**	**10-Year**	**5-Year**
Salt River Confluence	227,000	185,000	92,000	57,000	23,500
Waterman Wash Confluence	210,000	160,000	68,000	46,000	17,000
Hassayampa River Confluence	203,000	153,000	67,000	42,000	15,000
Gillespie Dam	195,000	145,000	65,000	38,000	12,000

The streamflow records collected were mean daily and instantaneous peak discharge data recorded by the USGS at the Gila River below Gillespie Dam gaging station, gage number 09519500. This gage has been operated for more than 80 years (1921 to present) and represents the “long term” flood history for the study reach. That data set contains 28,548 mean daily discharge records and 153 instantaneous peak discharge records. A plot of the discharge records is shown in Figure 2. The maximum estimated mean daily discharge is 130,000 cfs occurring on 9 January 1993. The maximum recorded instantaneous discharge is 178,000 cfs occurring on 16 February 1980, which is only slightly less than the estimated 100-year peak discharge of 195,000 cfs. The record set contains 15,393 days of no flow, 220 days of flow greater than the estimated 5-year peak discharge of 12,000 and 56 days of flow greater than the estimated 10-year peak discharge of 38,000 cfs.

In addition to a “long term” hydrologic simulation, a 100-year event is also considered. The discharge-frequency relations developed by the COE were based on graphical analyses of streamflow data, storage-discharge relations and elevation-discharge relations therefore, event specific hydrographs were not computed. Estimation of the 100-year hydrograph was accomplished using streamflow records for the February 1980 flood and the estimated 100-year peak discharge at the confluence of the Salt and Gila Rivers. Streamflow records for the February 1980 flood were obtained from *Floods of February 1980 in Southern California and Central Arizona* (USGS, 1991 Professional Paper 1494). The discharge measurements reported in that document were for the gaging station at the Gila River below Gillespie Dam from 15 February to 23 February. The last discharge reported on 23 February was 62,000 cfs. The receding limb of the hydrograph was completed using mean daily discharges for the same gage for a total duration of 25 days. That hydrograph was then proportioned by the ratio of the peak discharge for the February 1980 event (178,000 cfs) to the 100-year peak discharge at the confluence of the Salt and Gila Rivers (227,000 cfs). The resulting 100-year flood hydrograph was smoothed to facilitate discretization and input to the sediment model. The synthetic 100-year hydrograph at the confluence of the Salt and Gila Rivers is shown in Figure 3 along with the February 1980 flood hydrograph and the proportioned 100-year flood hydrograph.

Figure 3
Flood Hydrographs for the Gila River



El Rio Watercourse Master Plan

Flow Breakout

PREPARED FOR:

FLOOD CONTROL DISTRICT
of
Maricopa County

PREPARED BY:

STANTEC CONSULTING INC.
8211 South 48th Street
Phoenix, Arizona 85044
(602) 438-2200

September 2005
Revised September 12, 2003
STANTEC CONSULTING Inc.
Project No. 8200240

Contract FCD 2001C024

Stephanie R. Gerlach

Registered Professional Engineer (Civil)
CERTIFICATE NO.
33989
STEPHANIE R.
GERLACH
Date Signed 11/14/2005
ARIZONA U.S.A.

Flood Control District of Maricopa County

Stantec

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Flood Control District of Maricopa County

Stantec

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Memo

Stantec

To:	Files Stantec Phoenix	From:	Stephanie R. Gerlach Stantec Consulting Inc.
File:	82000240	Date:	September 23, 2005

The following is a response to the comments provided by Bing Zhao of the Maricopa County Flood Control District (MCFCD) in an email dated November 12, 2004.

Comment (2.A). The complete breach of a 4 foot high, 1400 foot long dike within 2 minutes when the water reaches its base does not seem realistic. The breached flow is conservative and the delineated floodplain may be over-estimated. There was no sensitive analysis of breach parameters in the report. Such an analysis may be helpful since the breach parameters do not seem realistic, though conservative. A field visit was performed by the District staff at the site.

The approach for the breakout analysis was to be conservative in order to delineate the probable maximum the limits of the flooding. There is little or no literature available for estimating the parameters for a levee breach. The most conservative values were selected from dam break breach parameters because of the following reasons

- The peak elevation of the 100-year event is higher than the canal/roadway.
- The embankments do not have engineered erosion protection.
- The canal and roadway fill are not designed for stability and seepage conditions associated with the 100-year event.

The scope of work does not include completing a sensitivity analysis.

Comment (2.B). Storage routing was performed in HEC-RAS for the area between east of Jackrabbit Road and the breached levee. Storage routing was also performed in HEC-RAS for the area west of Jackrabbit Road to determine the overflow at Jackrabbit Road. Supporting documentation is needed to support the accuracy of the stage storage curve for the ponding area east of Jackrabbit Road, especially between 888 and 892.

Additional documentation was added to the report for the stage storage curve.

Comment (2.C). The water surface elevations in the overbank area downstream of Jackrabbit Road were determined using Flowmaster. HEC-RAS is a more appropriate tool for floodplain delineation.

The scope of work included completing a normal depth non-detailed type of flood routing.

STANTEC CONSULTING INC.

Stephanie R. Gerlach, PE
Engineer, Water Resources
sgerlach@stantec.com

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FLOW BREAKOUT, EL RIO WCMP

INTRODUCTION

Just upstream of the Jackrabbit/Tuthill Road bridge, the Buckeye Water Conservation and Drainage District (BWCDD) South Extension Canal and Maricopa County Highway 85 (MC 85) are located close to the Gila River and function as levees between the Gila River and the farm fields to the north. These structures are not considered engineered levees per Federal Emergency Management Agency (FEMA) guidelines for the following reasons (US Government, 2002).

- The canal and MC 85 do not have three feet of freeboard above the Base Flood Elevation (BFE). At cross section 190.05, the peak water surface elevation is higher than the canal/roadway.
- The embankments do not have engineered erosion protection.
- The canal and roadway fill are not designed for stability and seepage conditions associated with the 100-year flood event.

The current floodplain extends a significant distance beyond these structures (to the north) but stops abruptly at Jackrabbit (Tuthill) Road (Michael Baker, Jr., 1999). The Flood Control District of Maricopa County (District) has requested that Stantec Consulting Inc., estimate the discharge that could be expected to breakout of the Gila River and drain downstream overtopping Jackrabbit Road during a 100-year event and delineate the resulting approximate flooding limits, assuming that canal/roadway were to fail.

The approximate floodplain delineation will be preliminary in nature to define any flood hazard issues and will not be presented to FEMA. Further refinement to the delineation will be necessary under a separate task and fee if submittal to FEMA is warranted.

Analysis Technique

The floodplain limits for a levee breach of South Extension Canal and MC 85 were delineated by first modeling the hydrograph of the 100-year discharge event in the Gila

River. Then, the embankment was modeled to breach by piping and the resulting hydrograph through the breach was used to estimate the ponding limits of the area east of Jackrabbit Road. The resultant peak discharge was then used to map inundation limits downstream of Jackrabbit Road. Normal depth analysis was used to estimate the limits of the floodplain west of Jackrabbit Road.

HEC-RAS Version 3.1.1 (US Army Corps of Engineers, 2003) is used to model the following items:

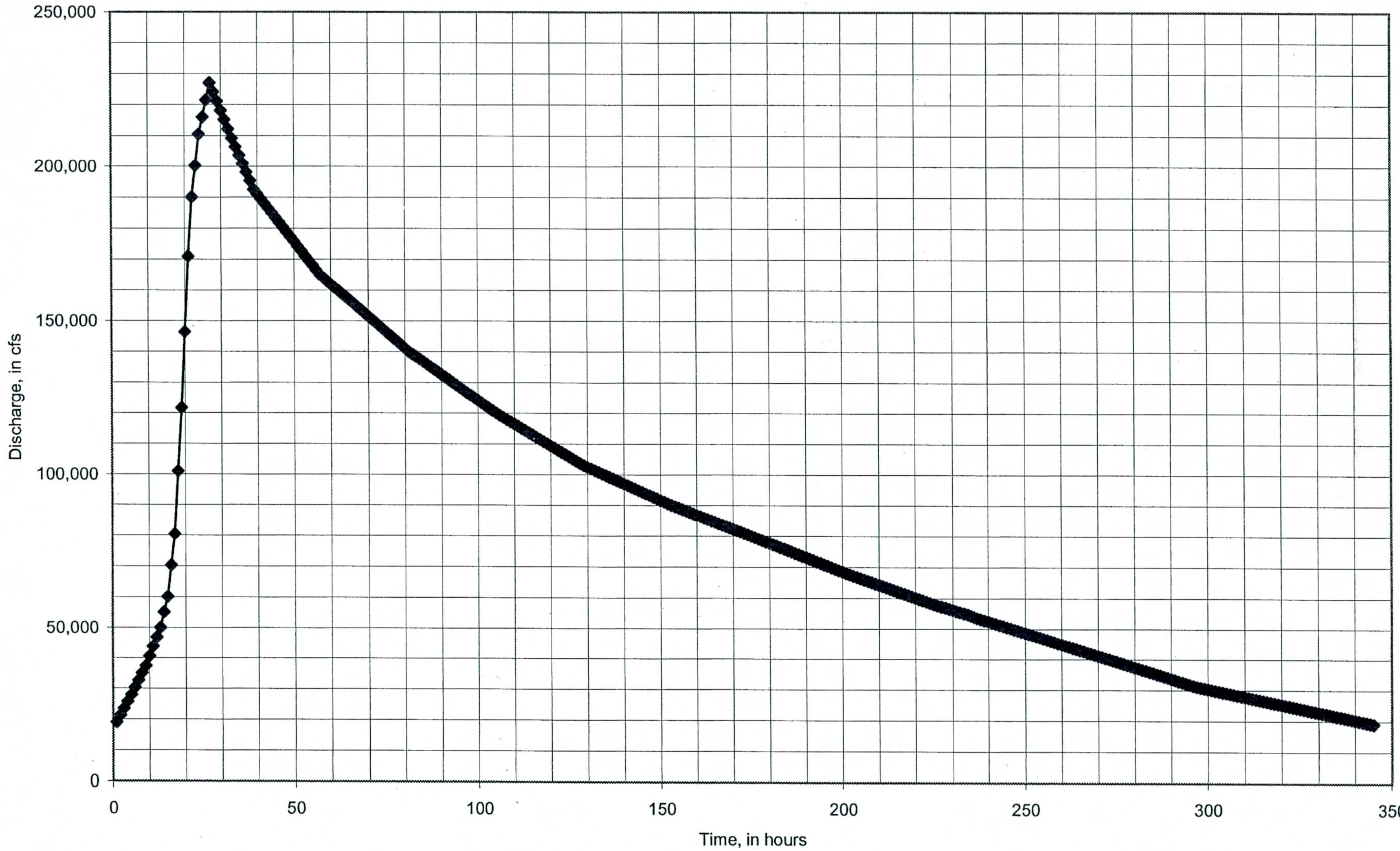
- Flow in the Gila River during the 100-year event,
- The canal/roadway failure during this event,
- Ponding limits for the area north of the canal/roadway and east of Jackrabbit Road,
- Spill over Jackrabbit Road.

The flow breakout model requires five pieces of information; hydraulics of the Gila River, hydrograph of the 100-year event, parameters for canal/roadway failure, stage/storage relationship behind the canal/roadway, and the Jackrabbit Road profile. The base hydraulic model of the Gila River is the current FIS HEC-RAS model (Michael Baker, Jr., 1999). The flow breakout model utilizes the base hydraulic model for the levee breach analysis. The base model was simplified by deleting cross sections that are not in the area of interest (greater than 194.09 and less than 180.09), thus keeping cross sections in the vicinity of the breakout section. Normal depth was used as the downstream boundary condition.

HEC-RAS models a levee breach using the lateral structure option. The data representing the canal/roadway was removed from the cross sectional geometry between cross sections 189.87 and 190.71. Along these same cross sections, a levee was coded into the lateral structure option. Field survey of the canal and MC 85 was used to code in the levee. The tallest structure, canal or roadway, was used as the top of the levee. The toe of the embankment was determined by comparing the survey for the four-foot contour interval mapping prepared for the current FDS and the field survey of the farm fields north of the canal/roadway. The modifications to the cross sections are shown in Appendix A.

The hydrologic input into the model is in the form of a synthetic 100-year hydrograph developed for sediment modeling purposes. That hydrograph is based on the February 1980 flood proportioned to the estimated 100-year peak discharge at the confluence of the Salt and Gila Rivers. A detailed description of the development of the data is included in the El Rio Watercourse Master Plan Hydrology Memorandum (Stantec, 2002). The hydrograph is shown as Figure 1. The hydrograph was coded into the unsteady flow option of HEC-RAS.

Figure 1
100-year Flood Event Hydrograph



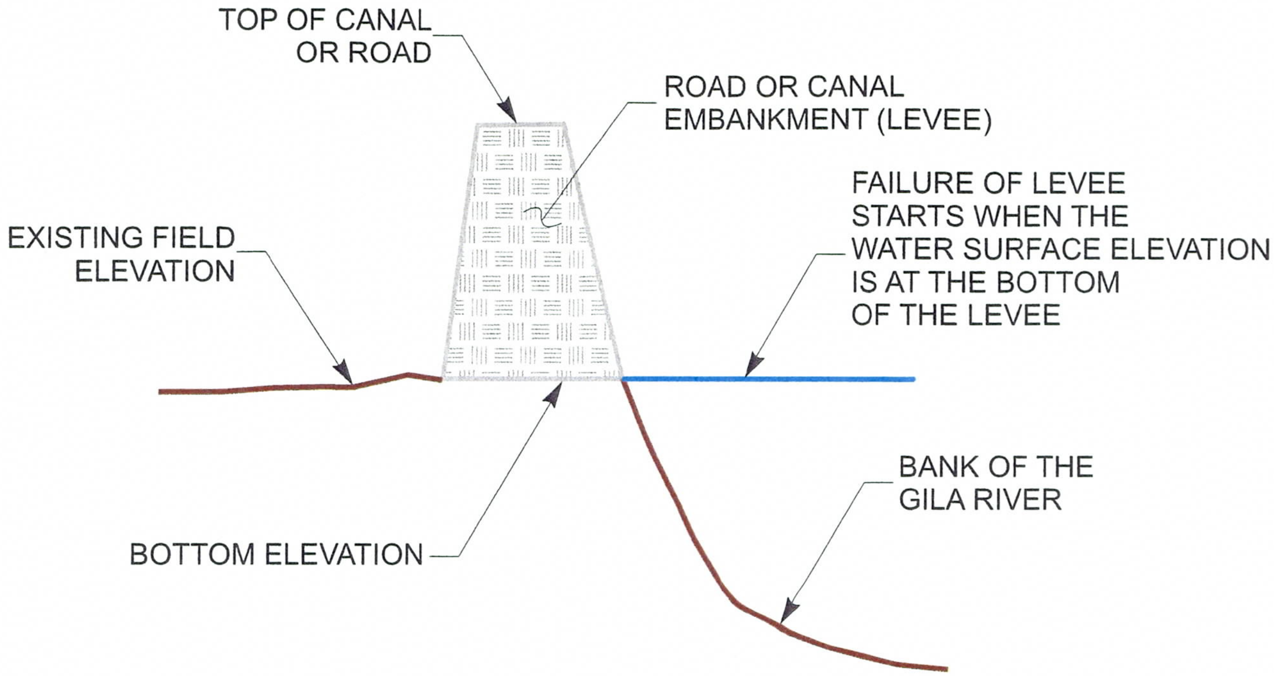
Levee breach parameters for the canal and MC 85 were coded into the breach option of the lateral structure editor in HEC-RAS. Breach parameters include left and right side slope, full formation time (time to breach), bottom width, failure mode and bottom elevation. See Figure 2 for a schematic of the breach. A number of different relationships have been proposed for dam failure parameters and are summarized in *Prediction of Embankment Dam Breach Parameters, A Literature Review and Needs Assessment, DSO-98-004 Dam Safety Research Report by Bureau of Reclamation Dam Safety Office*, dated July 1998. The majority of the relationships are based on dam failure case studies. There is little or no literature available for estimating the parameters for a levee breach. The dam failure relationships were solved and reviewed for reasonableness for levee failure. If the parameter did not seem reasonable based on engineering judgment, a value was determined based on field observation and history of the site. The results of each of the relationships are shown on Table 1. Appendix B includes a detailed solution to each of the relationships.

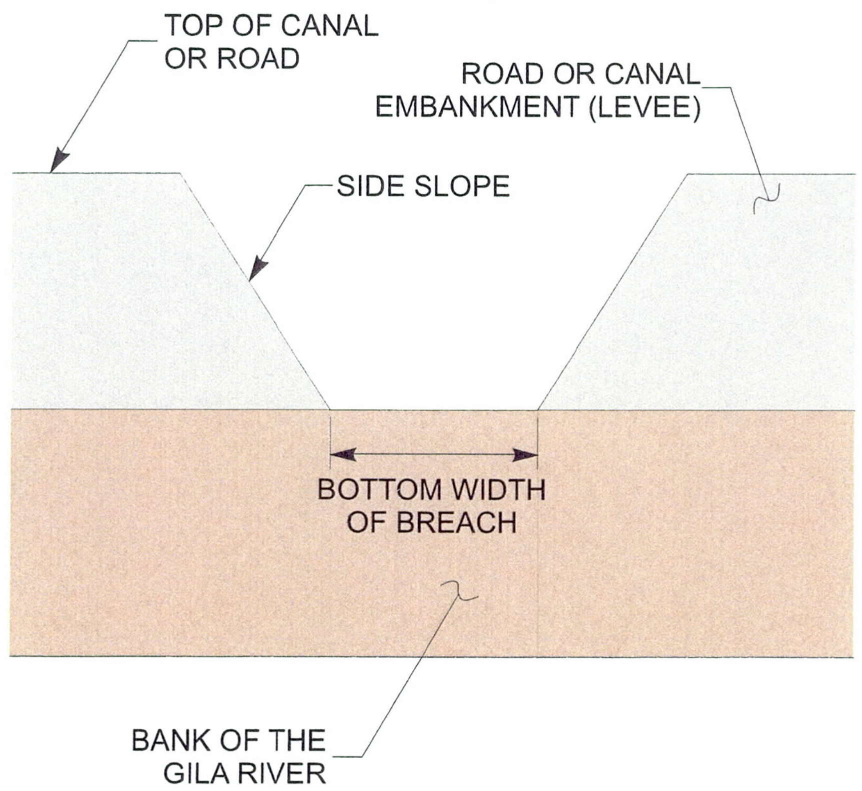
Table 1

Levee Failure Parameters

Relationship	**Bottom Width (feet)**	**Formation Time (hours)**	**Side Slope**
(1)	(2)	(3)	(4)
Johnson & Illes	$3 \leq B \leq 18$	--	Trapezoidal
Singh & Snorrason	$12 \leq B \leq 30$	$0.25 \leq t_f \leq 1.0$	--
Mac Donald & Langridge-Monopolis	--	--	1 hort : 2 vert
FERC	$12 \leq B \leq 24$	$0.1 \leq t_f \leq 0.5$	$1 \leq z \leq 2$
Froehlich, 1987	20*	3.8	
Reclamation	9	0.03	rectangular
Singh & Scarlatos	$1.6 \leq B_{top}/B_{bottom} \leq 1.74$	$t_f < 1.5$	$1.6 \leq z \leq 5.8$
Von Thun & Gillette	28	0.01	--
Froehlich, 1995b	15*	0.26	0.9
Selection	**1,400**	**0.03**	**1**

* Average breach width $(B_{top}/B_{bottom})/2$





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June 2004
82000240

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Client/Project
FCMCD
El Rio WCMP
Breakout Area

Figure No.
2

Title
SCHEMATIC OF
LEVEE BREACH

The following assumptions were employed in the estimation of levee failure parameters.

- Side slope (column 4, Table 1) was estimated based on assuming a trapezoidal section and the most conservative value of the relationships listed in the dam safety research report, one-to-one side slope.
- The full formation time (column 3, Table 1) is the time for the breach to form. It was estimated using the most conservative relationship listed in the report, 1.8 minutes.
- The bottom width relationships in the report (column 2, Table 1) were not used. The canal/roadway between cross sections 190.24 and 190.53 (approximately 1,400 feet long) had loose fill material with no bank protection and was the thinnest in cross section. This area is also located at an outside bend in the Gila River that is subject to attack. This area is used as the breach location with a bottom width of 1,400 feet.
- There are two options for the failure mode, a piping failure or an overtopping failure. Piping was selected as the failure mode starting when the water surface elevation is at the bottom of the levee. The bottom elevation, 886.7 feet, was selected based on the field elevations to the north of the canal and MC 85.

A flow storage area was modeled between Jackrabbit Road and the assumed levee. This represents the ponding area between the canal/roadway and Jackrabbit Road. The HEC-RAS storage area option requires the input of stage/storage relationship. The stage/storage relationships were obtained from the topographic mapping prepared for the current FIS (see Figure 3). The flow over Jackrabbit Road was modeled by adding a second storage area with a storage area connection. The second storage area is only used for storing the flow that comes over the road and does not represent actual conditions. The road is modeled as a storage area connection. The profile data for the connection was obtained from the field survey of the road.

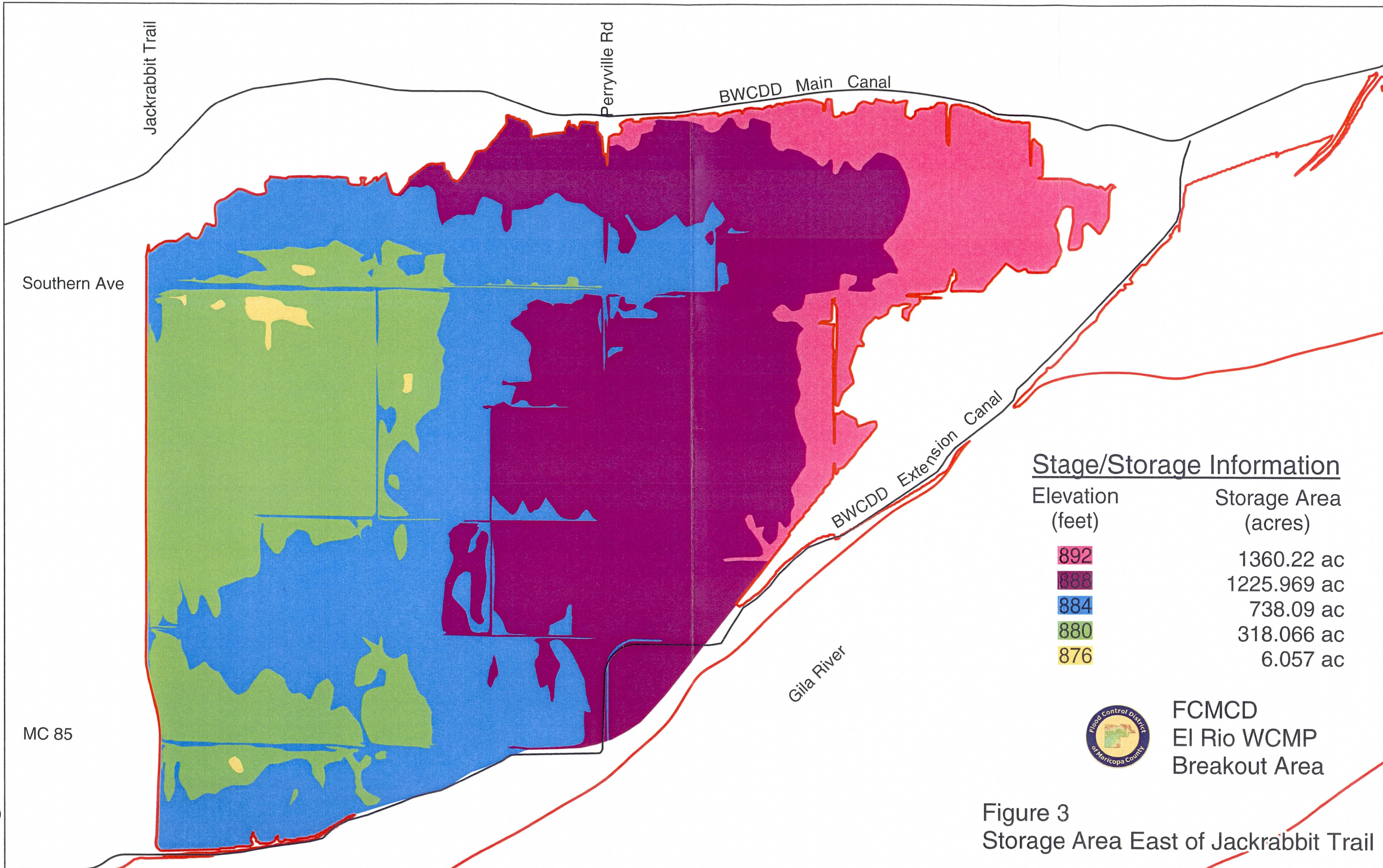
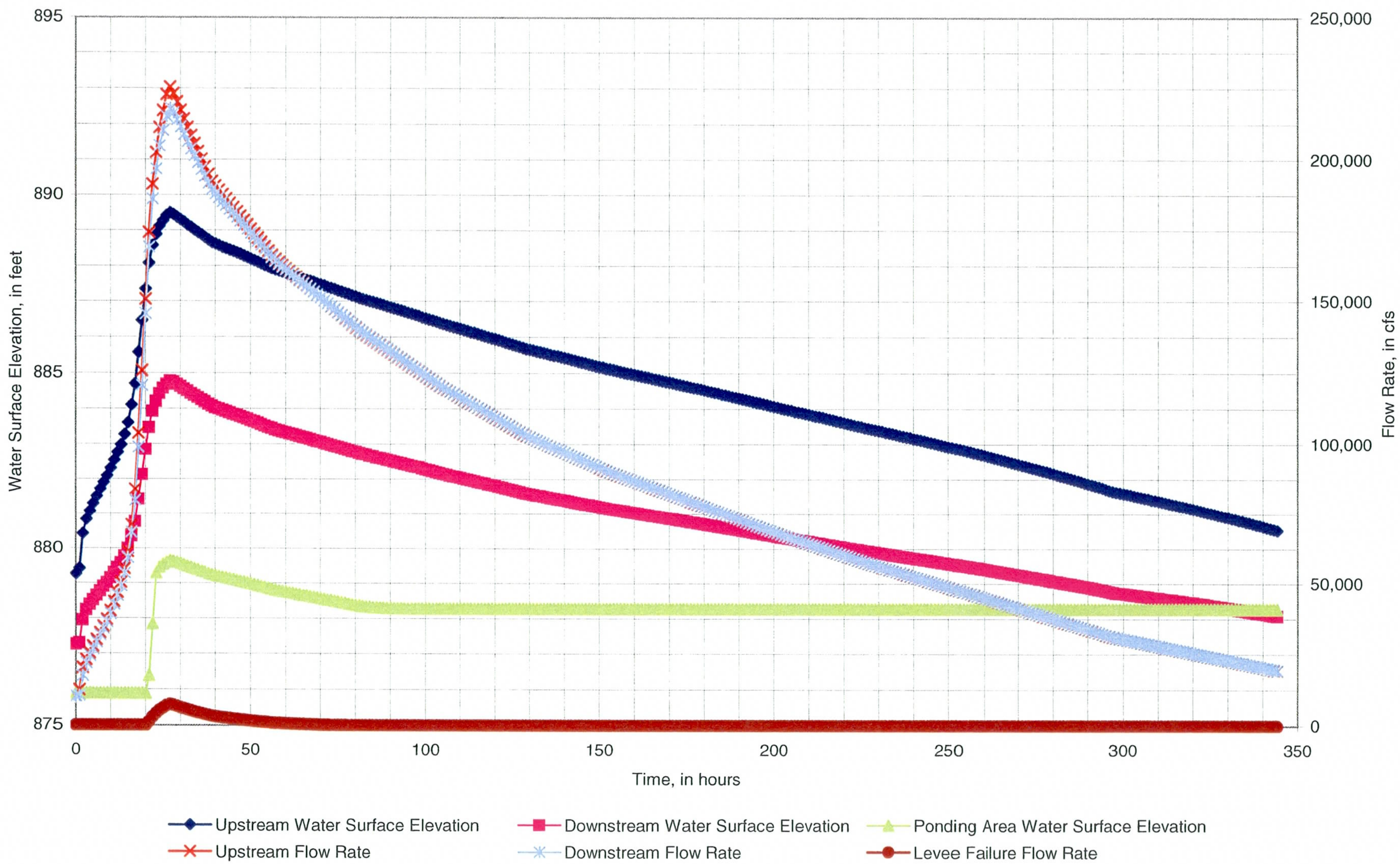


Figure 3
Storage Area East of Jackrabbit Trail

The unsteady flow simulation option in HEC-RAS was used to run the flow breakout model. Figure 4 shows the stage and flow for the cross section upstream and downstream of the breach and in the ponding area. The stage curve for the ponding area behind canal/roadway is shown as a green colored line. At the beginning of the simulation, the ponding stage is at 875.9 feet. This elevation is assumed to be the lowest elevation north of the canal/road (levee). The levee breaches 21 hours into the simulation when the water surface elevation in the Gila River reaches the bottom of the failure, 886.7 feet. At cross section 190.05, the water surface elevation for the 100-year event is higher than the canal/roadway and overtopping occurs. The flow overtopping is added to the flow from the levee failure. The maximum height of water in the ponding area rises to 879.65 feet. As water fills the ponding area, water eventually starts flowing over Jackrabbit Road. As the flood recedes in the Gila River and drops below the bottom of the breach at 91 hours into the simulation, the ponding stage drops to the ground elevation (878.28 feet) over Jackrabbit Road. The flow rate of the cross sections upstream, 190.71, and downstream, 189.87, of the levee are shown as red and light blue colored lines on Figure 4. The maximum flow rate for the Gila River upstream and downstream of the levee breakout area is 225,409 cfs and 218,138 cfs, respectively. The maximum flow rate through the breach, at the peak of the hydrograph, is 7,291 cfs. The maximum height of water over Jackrabbit Road is 1.4 feet. The HEC-RAS model is included on a disk in Appendix C.

Figure 4
Results of Levee Failure Analysis



The flood hazard limits of the area west of Jackrabbit Road are based on normal depth analysis using the Manning's equation. An average Manning's n-value of 0.04 was selected for the flood prone area. This area generally consists of farm fields bisected by dirt roads, paved roads and irrigation canals. There is little development until the Town of Buckeye. Pictures of the field conditions are shown in Appendix D. The photographs show areas that have n-values that are greater and less than 0.04. The 100-year event is most likely to occur during the winter months when the majority of the fields are fallow. The flood flows over the fields are relatively shallow, one to three feet. The n-value of 0.04 was selected as a conservative average value. Cross sectional geometry used to estimate flood hazard limits was determined from topographic mapping used for the current FIS. The digital terrain model (DTM) developed as part of the FIS topographic mapping was used to develop two-foot contour maps of the area. Cross section ground elevation points were extracted from the two-foot contour maps by hand. Cross sections were located approximately 1,000 to 2,000 feet apart parallel to the average flow direction. Flow direction varied because of the many obstructions such as dirt and paved roads, canals and the occasional building. Cross sections are identified numerically starting downstream of Jackrabbit Road (Figure 5). Cross section stationing is from left to right looking downstream. This discharge used for the normal depth analysis is 7,300 cfs (flow breakout discharge). This data was coded into FlowMaster® (Haestad Methods, Inc., 1998) to facilitate the use of Manning's equation. FlowMaster® print outs for the cross sections are included in Appendix E.

Results

The flood hazard limits due to the levee breach and effective floodplain limits for the Gila River are shown on Figure 5. The levee breach ponding limits for the area north of canal/roadway extend almost to the BWCDD main canal to the north and MC 85 to the east. The levee breach floodplain limits between Rainbow Road and Jackrabbit are parallel to the Extension Canal. The canal is raised above the farm fields preventing the flood flows from entering back into the Gila River. The levee breach analysis floodplain joins with the effective Gila River floodplain between Rainbow Road and Apache Road. The depths for the normal depth analysis range from one to four feet.

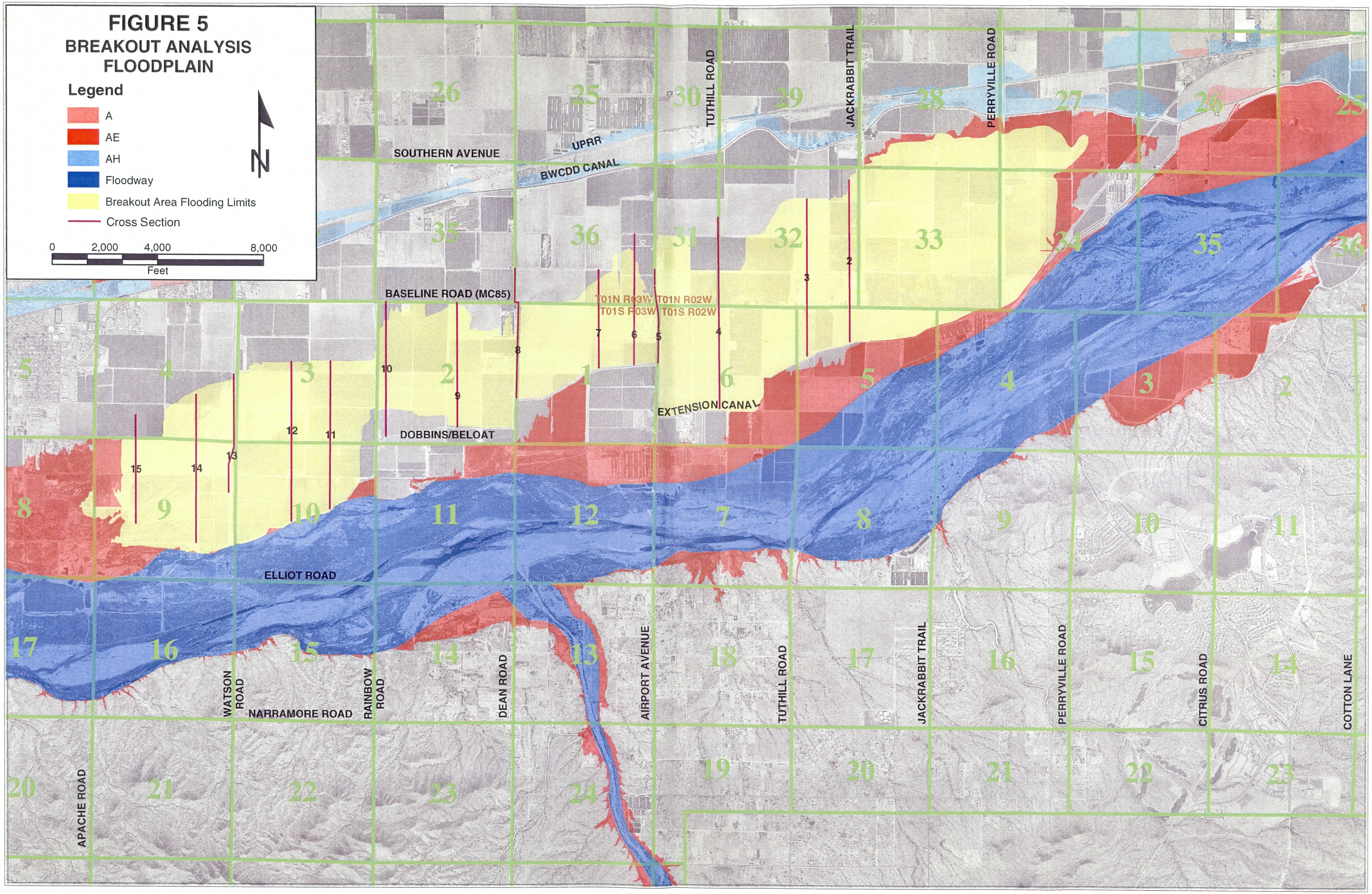


FIGURE 5
BREAKOUT ANALYSIS
FLOODPLAIN
Legend
A
AE
AH
Floodway
Breakout Area Flooding Limits
Cross Section
0
2,000
4,000
8,000
Feet
N
SOUTHERN AVENUE
UPRR
BWCDD CANAL
BASELINE ROAD (MC85)
DOBBINS/BELOAT
EXTENSION CANAL
ELLIOT ROAD
NARRAMORE ROAD
TUTHILL ROAD
JACKRABBIT TRAIL
PERRYVILLE ROAD
WATSON ROAD
RAINBOW ROAD
DEAN ROAD
AIRPORT AVENUE
CITRUS ROAD
COTTON LANE
APACHE ROAD
T01N R03W
T01S R03W
T01N R02W
T01S R02W

References

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Michael Baker, Jr., 1999. Salt-Gila River Floodplain Delineation Restudy Submitted to Flood Control District of Maricopa County, dated May 1999.

Stantec, 2003. El Rio Watercourse Master Plan Hydrology Memorandum, dated April 2002.

US Army Corps of Engineers, 2003. HEC-RAS River Analysis System, Version 3.1.1, Developed by US Army Corps of Engineers, Hydrologic Engineering Center, dated May 2003.

US Government, 2002. Code of Federal Regulations, Title 44 – Emergency Management and Assistance, Volume 1, Chapter I – Federal Emergency Management Agency, Section 65.10, Revised as of October 1, 2002.

Appendix A

HEC-RAS Modified Cross Sections

Figure A-1
Cross Section 189.96
Comparison of the original versus modified cross section

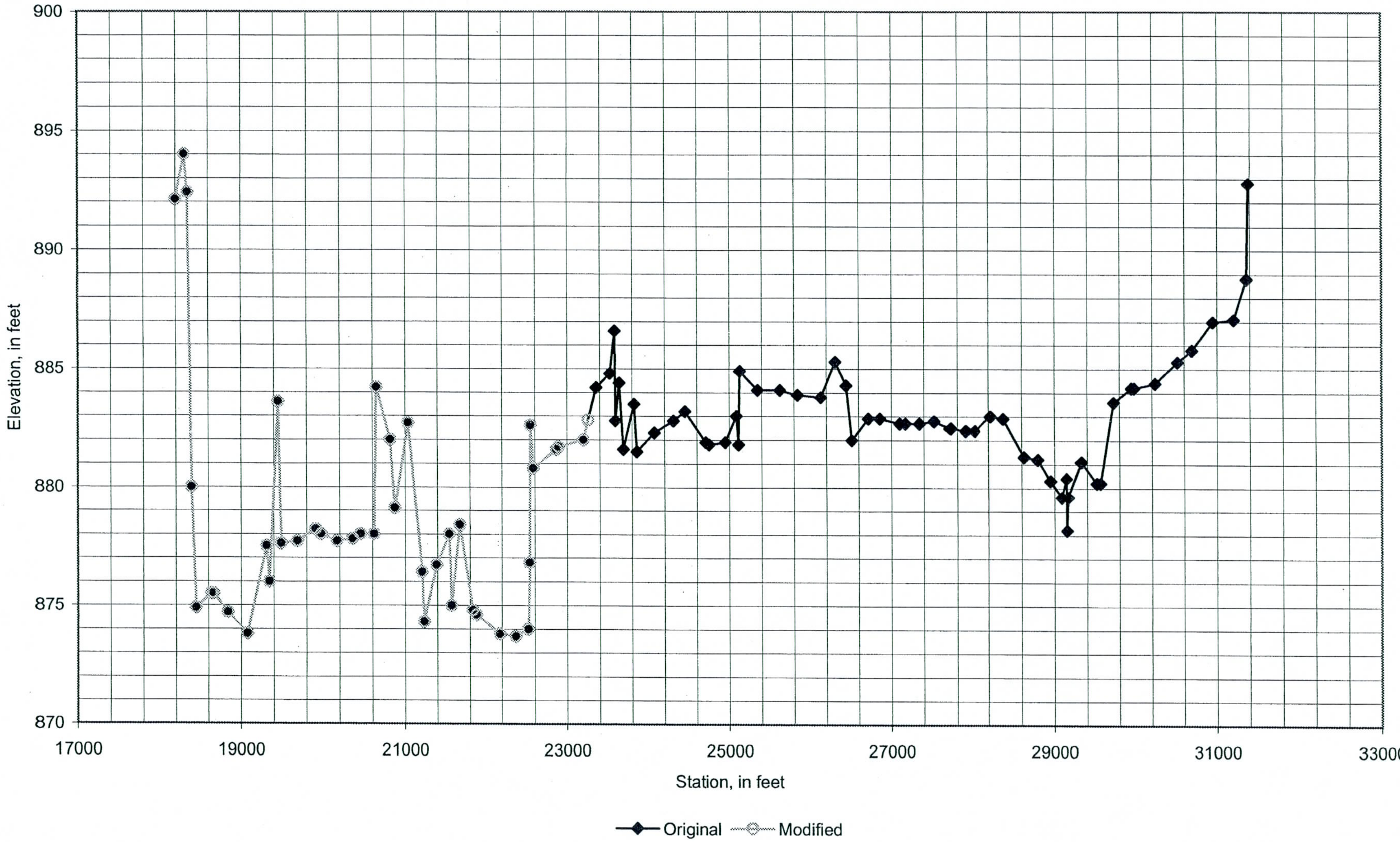


Figure A-2
Cross Section 190.05
Comparison of the original versus modified cross section

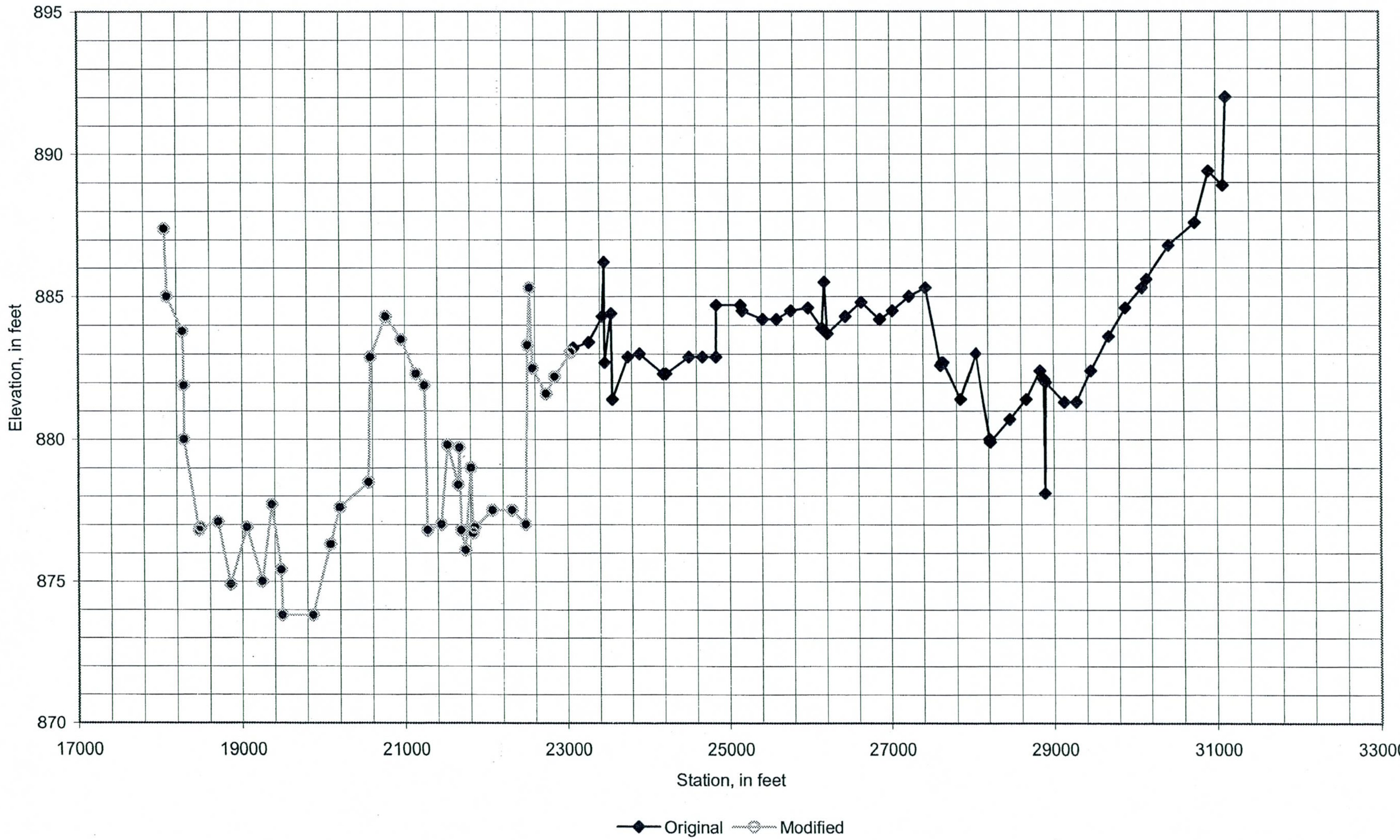


Figure A-3
Cross Section 190.15
Comparison of the original versus modified cross section

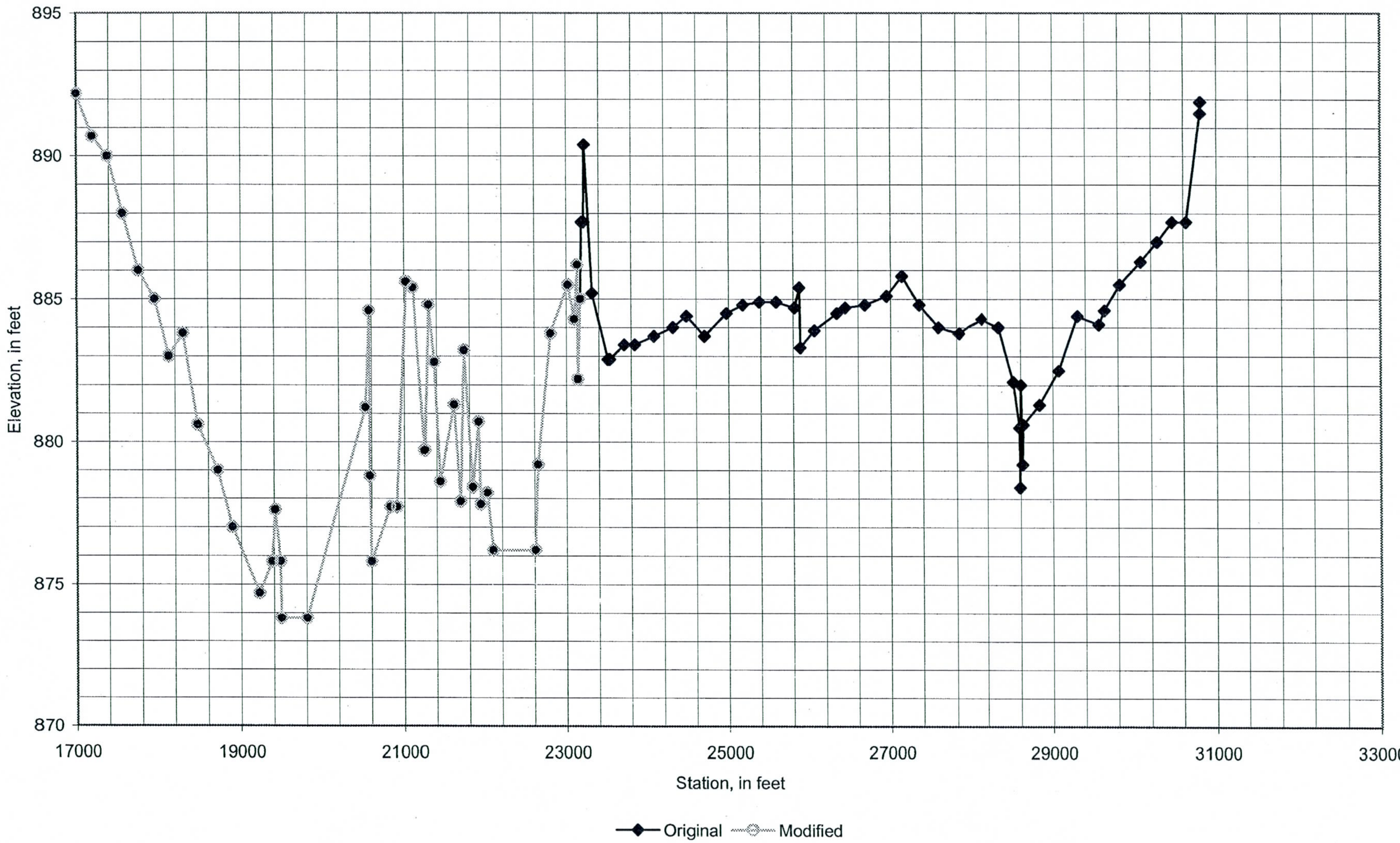


Figure A-4
Cross Section 190.24
Comparison of the original versus modified cross section

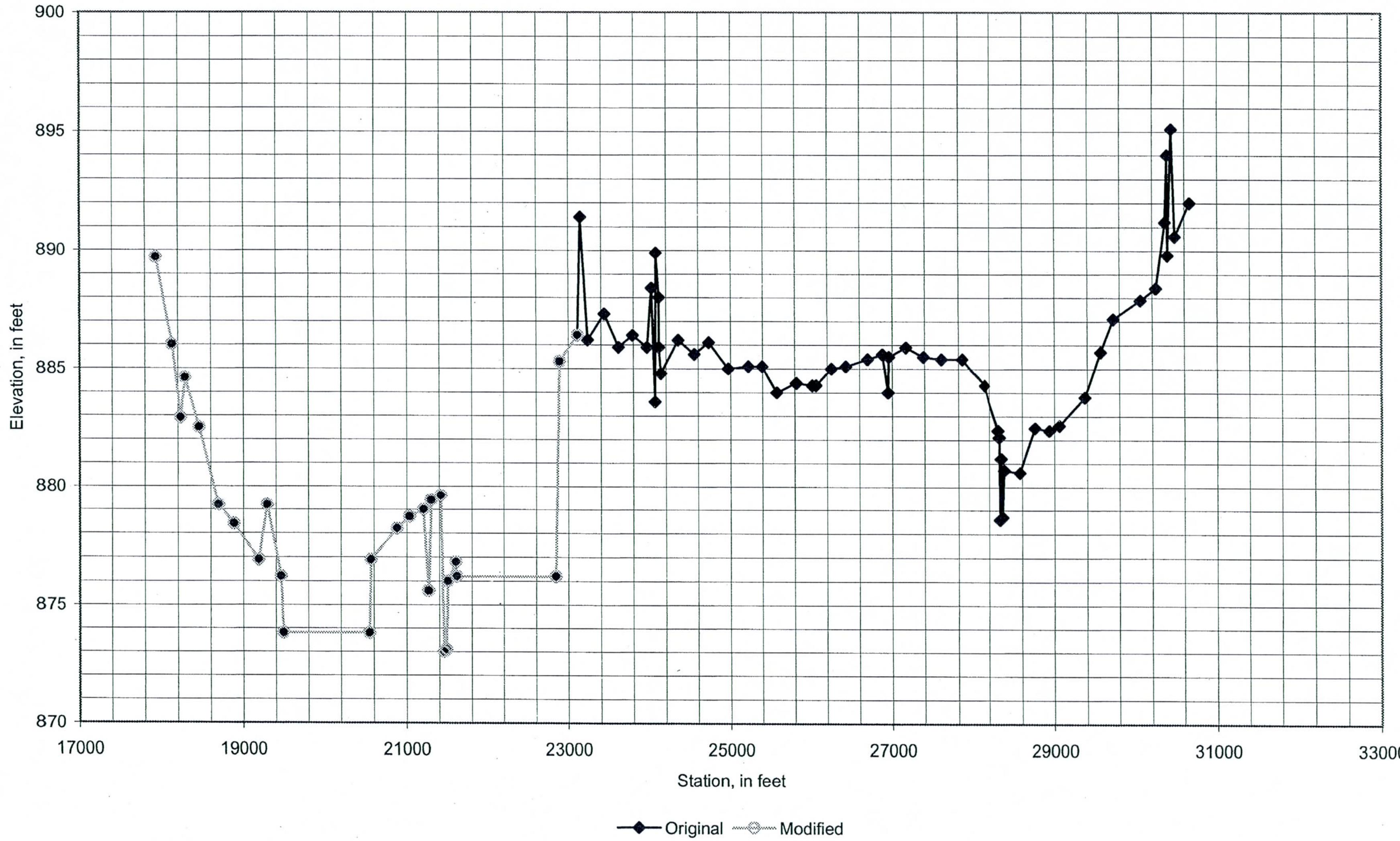


Figure A-5
Cross Section 190.34
Comparison of the original versus modified cross section

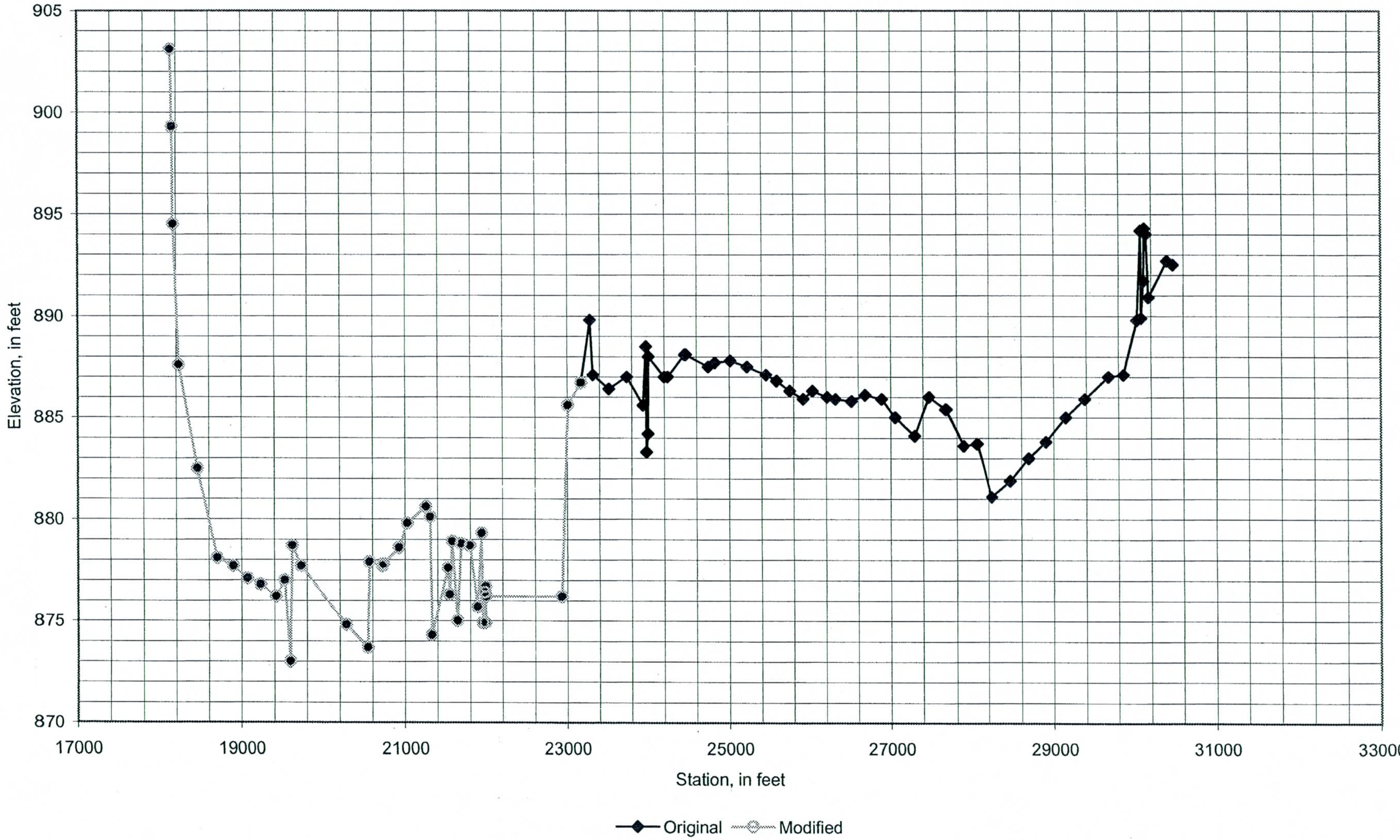


Figure A-6
Cross Section 190.43
Comparison of the original versus modified cross section

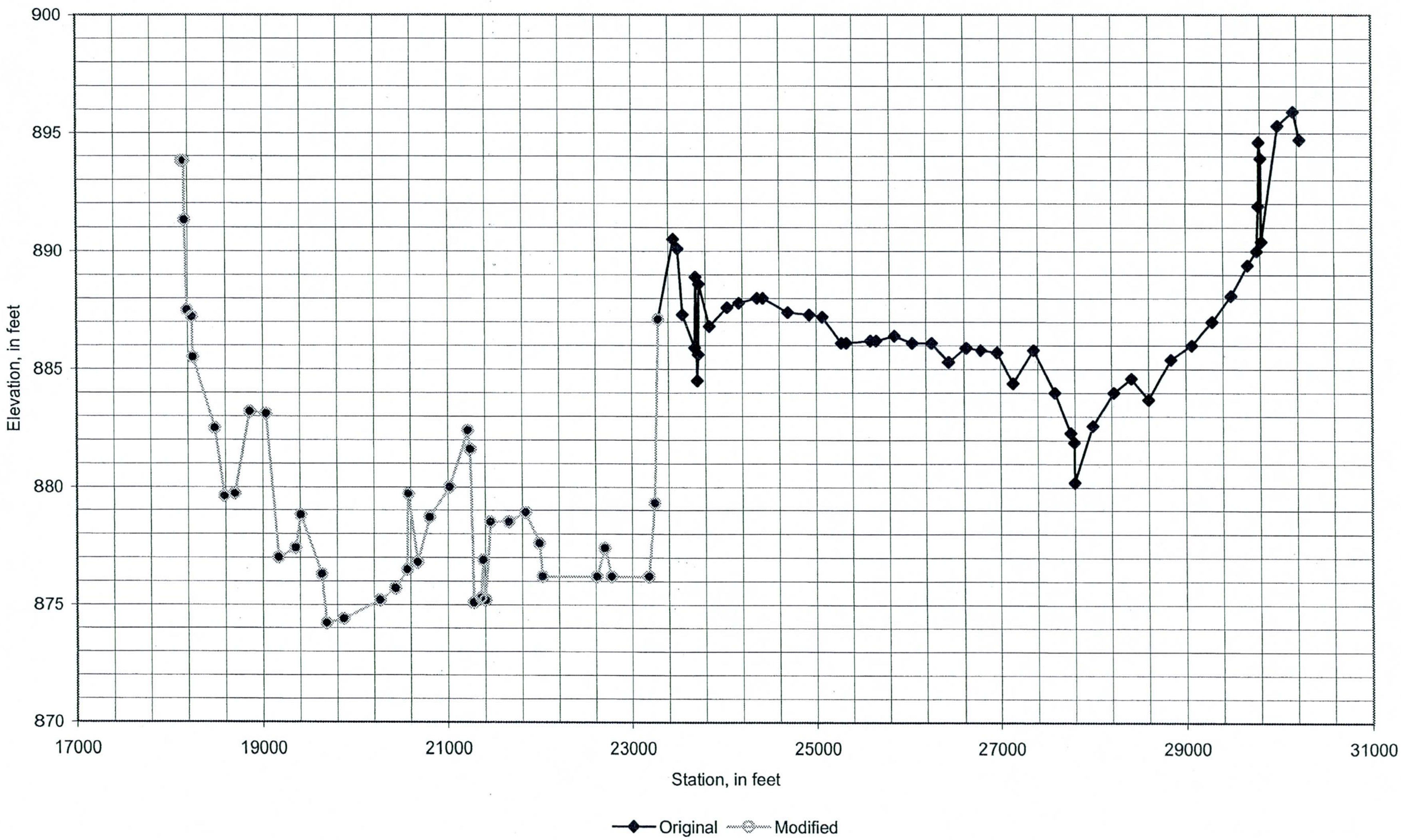


Figure A-7
Cross Section 190.53
Comparison of the original versus modified cross section

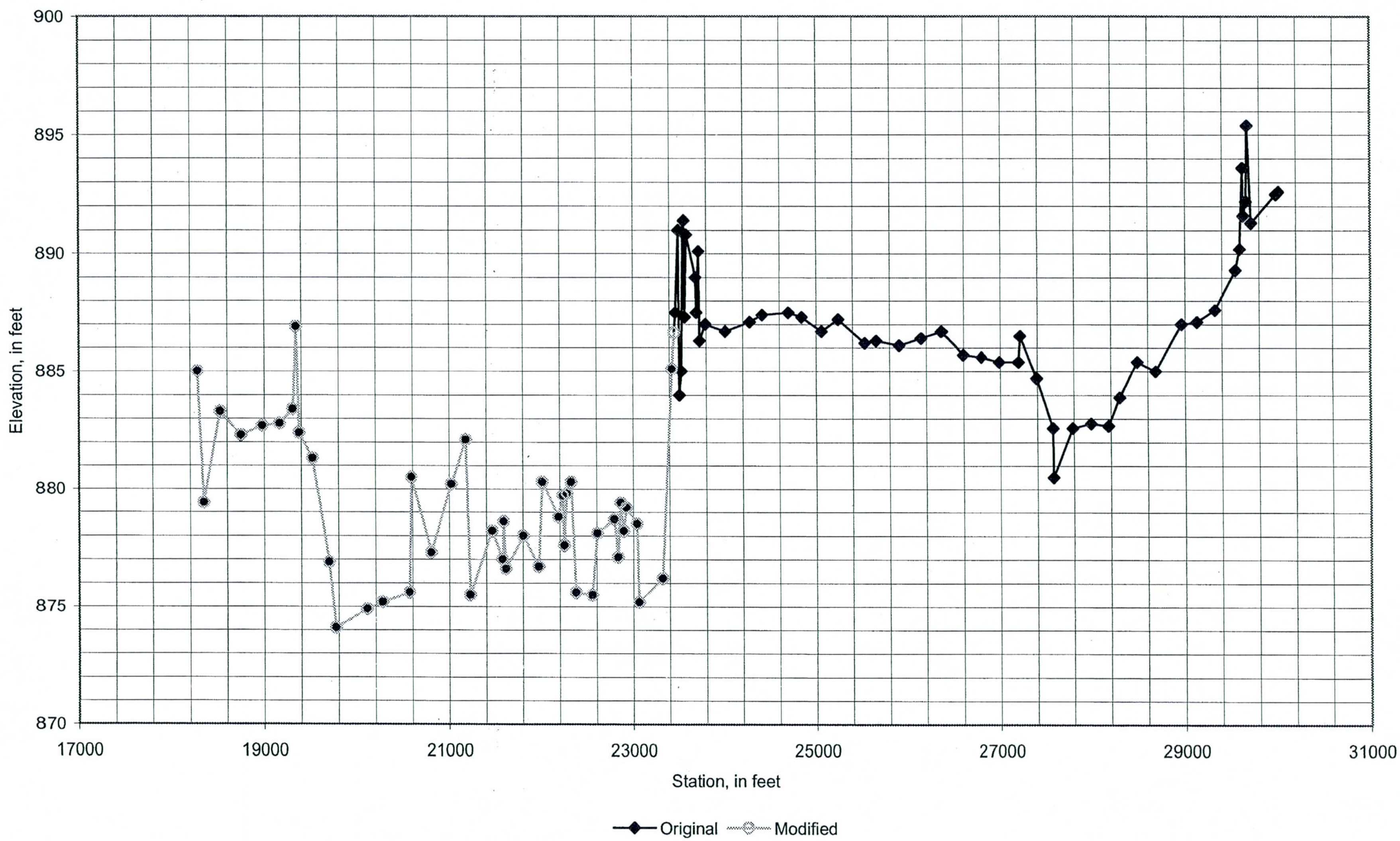
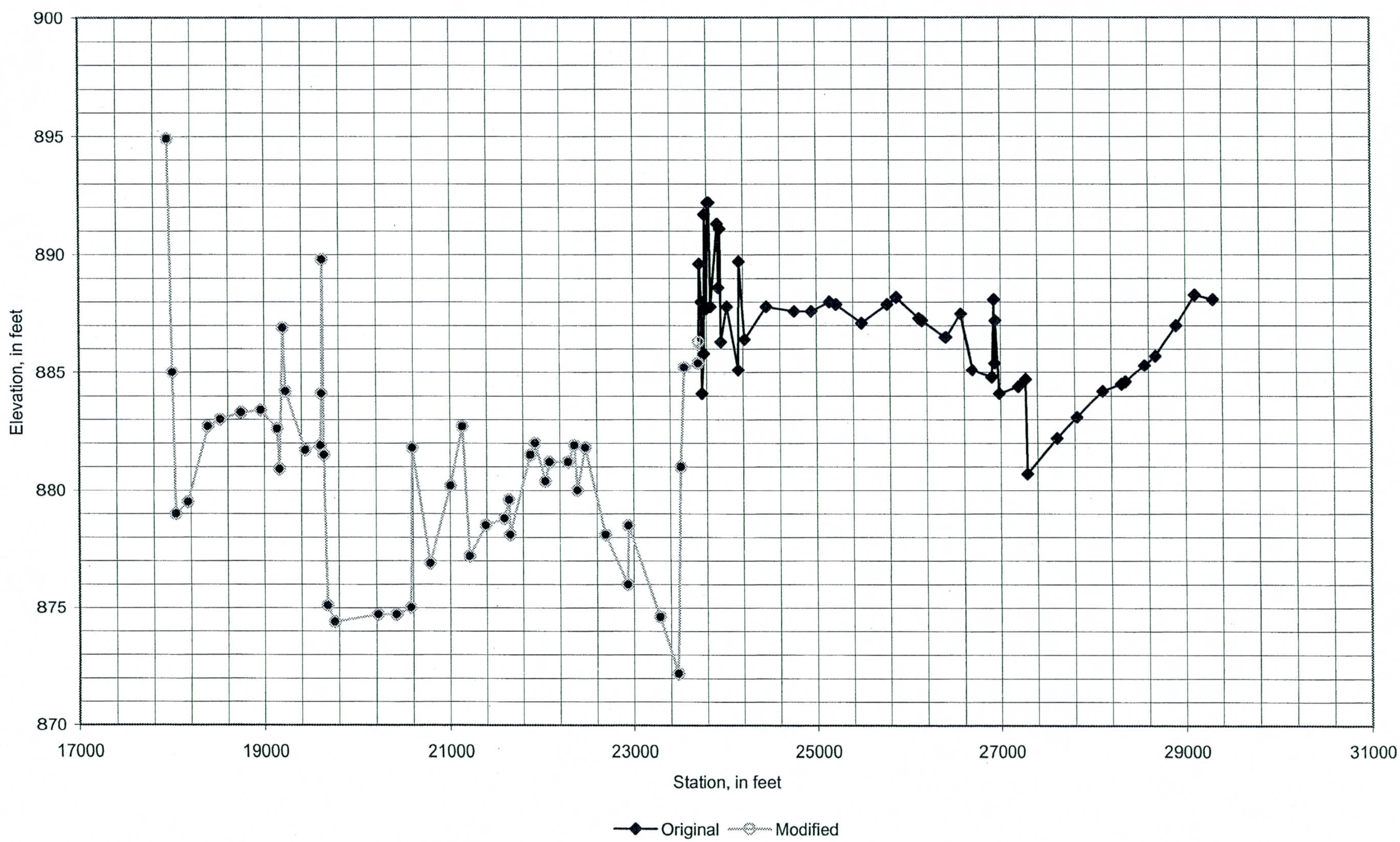


Figure A-8
Cross Section 190.62
Comparison of the original versus modified cross section



Appendix B

Levee Break Parameters

El Rio WCMP
Levee Failure Parameters
ARJ

Reference	B (feet)	t_f (hours)	Z (Zhort : 1ver)
Johnson & Illes	$3' \leq B \leq 18'$	–	trapezoidal
Singh & Snorrason	$12' \leq B \leq 30'$	$0.25hr \leq t_f \leq 1.0hr$	–
MacDonald & Langridge-Monopolis	–		1h : 2V
FERC	$12' \leq B \leq 24'$	$0.1hr \leq t_f \leq 0.5hr$	$1 \leq Z \leq 2.0$
Froehlich (1987)	20*	3.8	
Reclamation	9	0.03	rectangular
Singh & Scarlatos	$1.06 \leq \frac{B_{top}}{B_{bottom}} \leq 1.74$	$t_f < 1.5$	$1.3 \leq Z \leq 5.8$
Von Thun & Gillette	28	0.01	–
Froehlich (1995b)	15*	0.26	0.9

* Average breach width $(B_{top} + B_{bottom})/2$

Selection	30	0.03	1

Stantec

Checked by:

Designed by:

Johnson and Illes

$0.5\,h_d \leq B \leq 3h_d$

h_d - height of dam [m]
B - Breach Width [m]

$h_d = 6' = 1.83m$

$3' \leq B \leq 18'$

Singh and Snorrason

$2h_d \leq B \leq 5h_d$

$12' \leq B \leq 30'$

$0.25\,hr \leq t_f \leq 1.0\,hr$

t_f - breach formation time [hr]

MacDonald and Langridge-Monopolis

$V_{er} = 0.0261\,(V_{out} * h_w)^{0.769}$

$t_f = 0.0179\,(V_{er})^{0.364}$

V_{out} - Volume of water discharged through breach (initial storage + inflow during failure)

h_w - Hydraulic depth of water at dam at failure, above breach bottom

Designed by: Checked by:

V_{out} =

$hw = 3' = 0.91m$

Ver =

t_f =

FERC

$2h_d \le B \le 4h_d$

$12' \le B \le 24'$

$1 \le z \le 2$

z - Breach opening side slope factor (z hort : 1 vert)

$0.1hr \le t_f \le 0.5hr$

Froehlich (1987)

$\bar{B}^* = 0.47 K_o (S^*)^{0.25}$

$\bar{B}^*$ - Dimensionless average breach width ($\bar{B}/h_b$)
$\bar{B}$ - Average breach width $(B_{top} + B_{bottom})/2$
B_{top} - breach width at top of breach
B_{bottom} - breach width at bottom of breach
h_b - height of breach

Stantec

S^* - Dimensionless storage (S/h_b^3) $= \frac{16849}{(1.83)^3} = 2749$

S - storage

$S = 595092.7\ ft^3 = 16849 m^3$ ~ 82000240/calcs/split flow/storage at levee.xls

$K_o = 1.0$ - from paper

$h_b = 6' = 1.83m$

$\bar{B}^* = 0.47(1)(2749)^{0.25} = 3.4$

$\bar{B} = 3.40\ (1.83) = 6.2\ m = 20.4\ ft$

$Z = 0.75\ K_c\ (h_w^*)^{1.57}\ (\bar{W}^*)^{0.73}$

$K_c = 1.0$ - from paper

h_w^* - Dimensionless height of water above breach bottom (h_w/h_b)

$\bar{W}^*$ - Dimensionless average embankment width $(W_{crest} + W_{bottom})/(2h_b)$

$W_{crest} =$

$W_{bottom} =$

$\bar{W}^* =$

$h_w = 3' = 0.91$

$h_b = 6' = 1.83$

$h_w^* = 0.5$

$Z =$

Designed by: Checked by:

$B_{top} =$
$B_{bottom} =$

$t_f^* = 79(S^*)^{0.47} = 79(2749)^{0.47} = 3266$

t_f^* - Dimensionless breach formation time
$= t_f / \sqrt{g h_b}$

$t_f = t^* \sqrt{g h_b} = 3266 \sqrt{9.81(1.83)} = 3.8\,hr$

$\dfrac{hr}{\sqrt{\frac{m}{s^2}\, m}}$ $\quad \dfrac{60s}{m} \; \dfrac{60m}{hr}$

Reclamation

$B = 3 h_w = 3(0.91) = 2.73\,m = 9'$
$h_w\,[m] = 0.91$
$t_f = (0.011) B = 0.011(2.7) = 0.03\,hr$

Singh and Scarlatos

$1.06 \le B_{top}/B_{bottom} \le 1.74$

$10° \le$ side slope $\le 50°$ $\quad h_b = 1.83\,m \quad 5.8 \le z \le 1.3$

$t_f < 3\,hr$, 50% were less than 1.5hr

$\sin\theta = \frac{x}{1.83}$
$\theta = 10°$ $x = 0.3$
$\theta = 50°$ $x = 1.4$

Von Thun and Gillette

$\bar{B} = 2.5 h_w + C_b = 2.5(3) + 20 = 27.5\,ft$

$h_w = 3\,ft = 0.91\,m$
$C_b = 20\,ft$ ~from table on pg 15

$t_f = 0.015 h_w = 0.015(0.91) = 0.01\,hrs$

Designed by: Checked by:

Stantec

Froehlich (1995b)

$$\overline{B} = 0.1803 K_o V_w^{0.32} h_b^{0.19} = 0.1803(1)(16849)^{0.32}(1.83)^{0.19}$$
$$= 4.6m = 15'$$

V_w - volume of water above beach invert elevation at time of breach

$K_o = 1.0$ ~ from paper page 13

$h_b = 6' = 1.83$ m

$V_w = 595092.7 ft^3 = 16849 m^3$

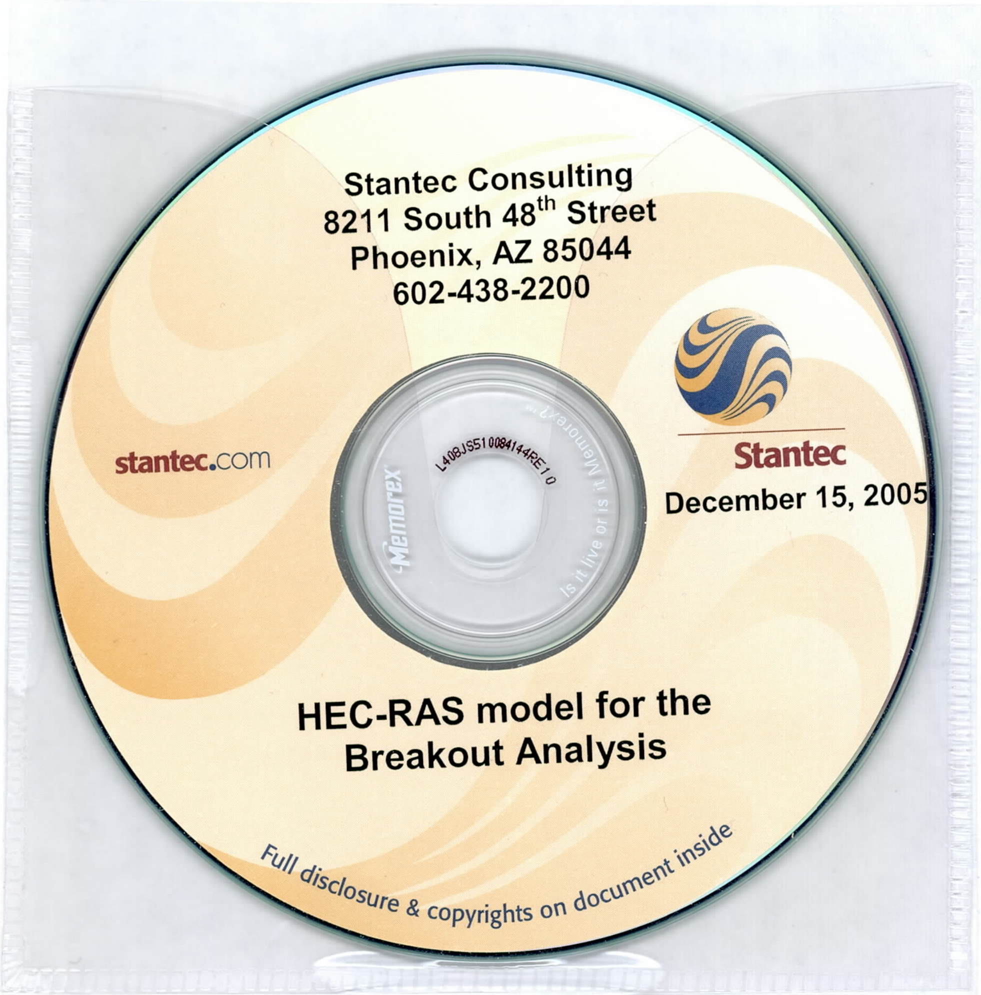
$$t_f = 0.00254\, V_w^{0.53}\, h_b^{(-0.90)}$$

$$= 0.00254(16849 m^3)^{0.53}(1.83m)^{-0.90} = 0.26hr$$

Designed by: Checked by:

Appendix C

HEC-RAS Model



Appendix D

Manning's n-value Photographs


Looking east from Jackrabbit Road


Looking west from Jackrabbit Road

Looking north between Jackrabbit Road and Airport Road

Looking south between Jackrabbit Road and Airport Road

Looking north between Jackrabbit Road and Airport Road

Looking south between Jackrabbit Road and Airport Road

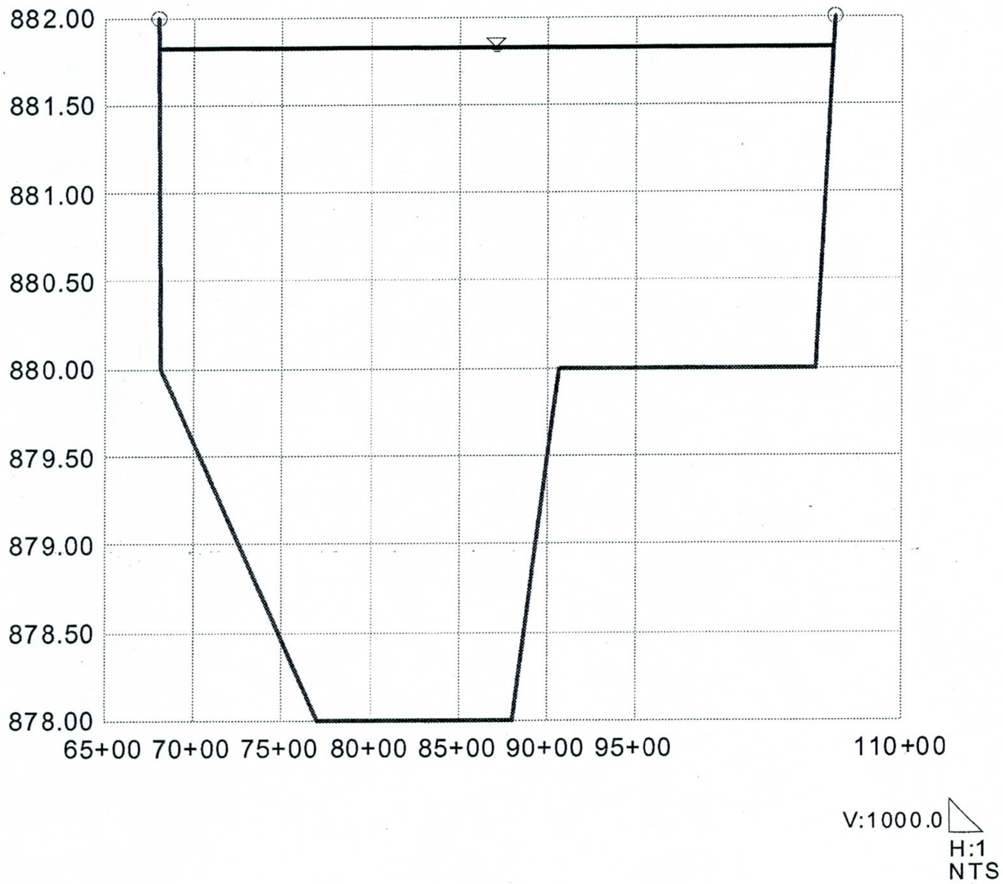
Appendix E

Normal Depth Cross Sections

Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 02
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

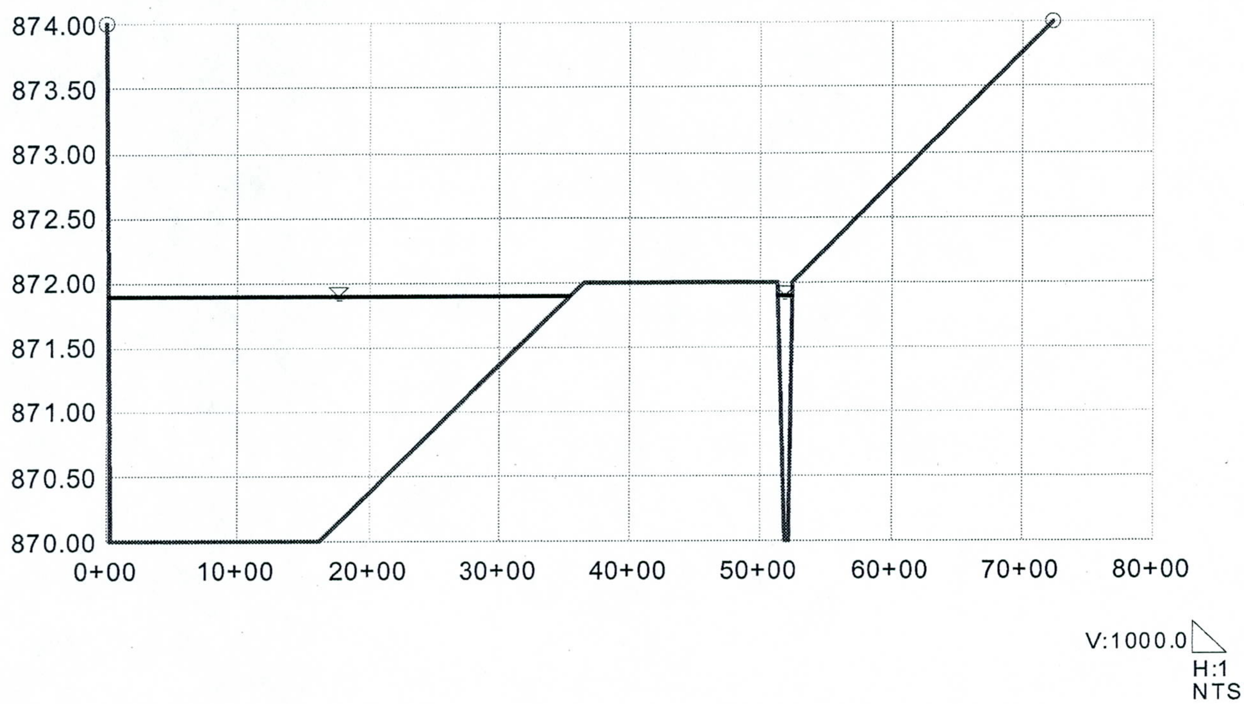
Section Data		
Mannings Coefficient	0.040	
Slope	0.000100	ft/ft
Water Surface Elevation	881.83	ft
Elevation Range	878.00 to 882.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 03
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

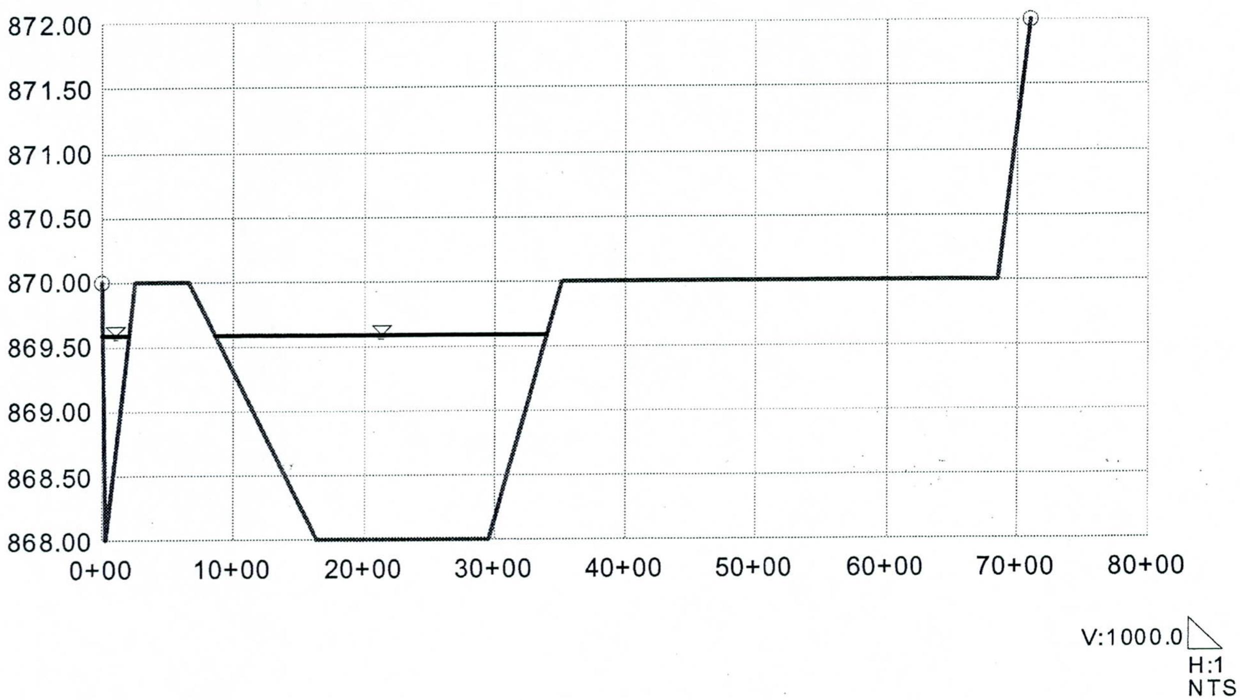
Section Data		
Mannings Coefficient	0.040	
Slope	0.001000	ft/ft
Water Surface Elevation	871.90	ft
Elevation Range	870.00 to 874.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 04
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coefficient	0.040	
Slope	0.003000	ft/ft
Water Surface Elevation	869.59	ft
Elevation Range	868.00 to 872.00	
Discharge	7,300.00	cfs

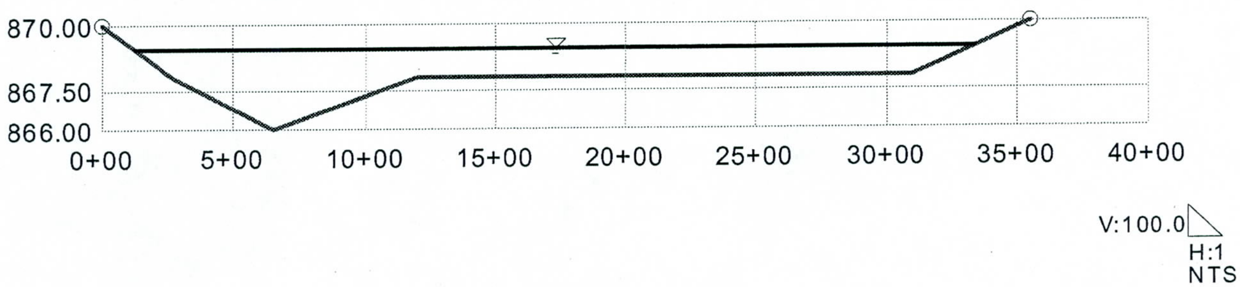


Cella Barr Associates
37 Brookside Road Waterbury, CT 06708 USA (203) 755-1666
Project Engineer: Stantec Consulting Inc

Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross Section 05 - Airport Rd
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

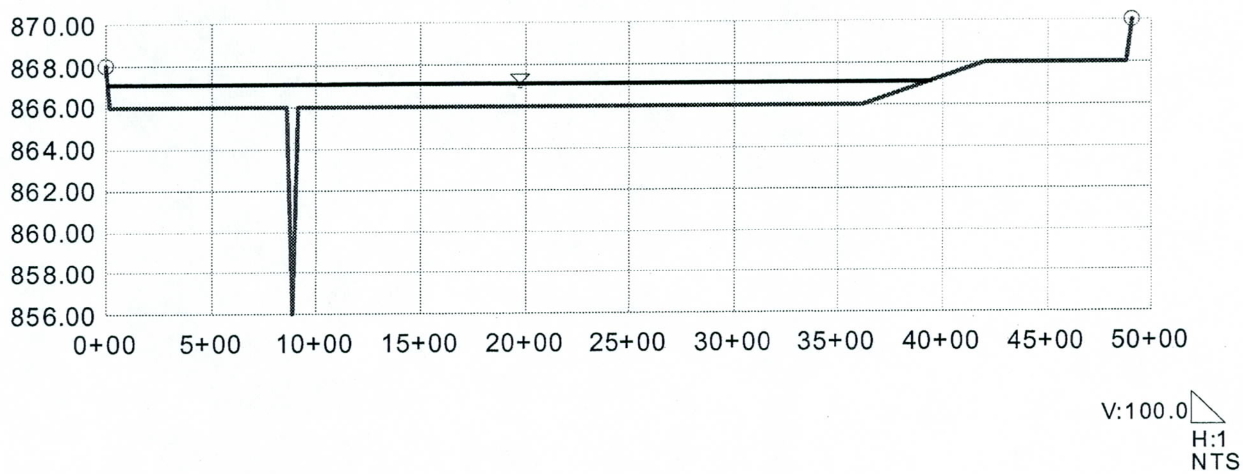
Section Data		
Mannings Coefficient	0.040	
Slope	0.001600	ft/ft
Water Surface Elevation	869.06	ft
Elevation Range	866.00 to 870.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 06
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

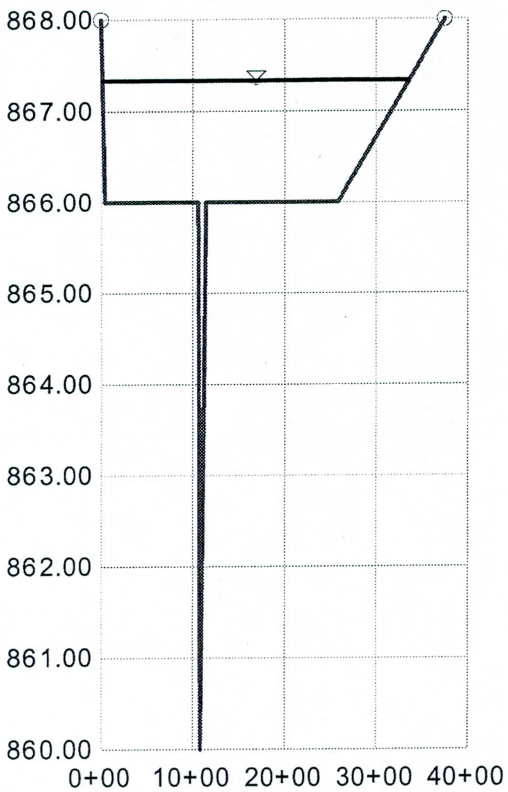
Section Data		
Mannings Coefficient	0.040	
Slope	0.001600	ft/ft
Water Surface Elevation	867.13	ft
Elevation Range	856.00 to 870.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 07
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coefficient	0.040	
Slope	0.001600	ft/ft
Water Surface Elevation	867.34	ft
Elevation Range	860.00 to 868.00	
Discharge	7,300.00	cfs

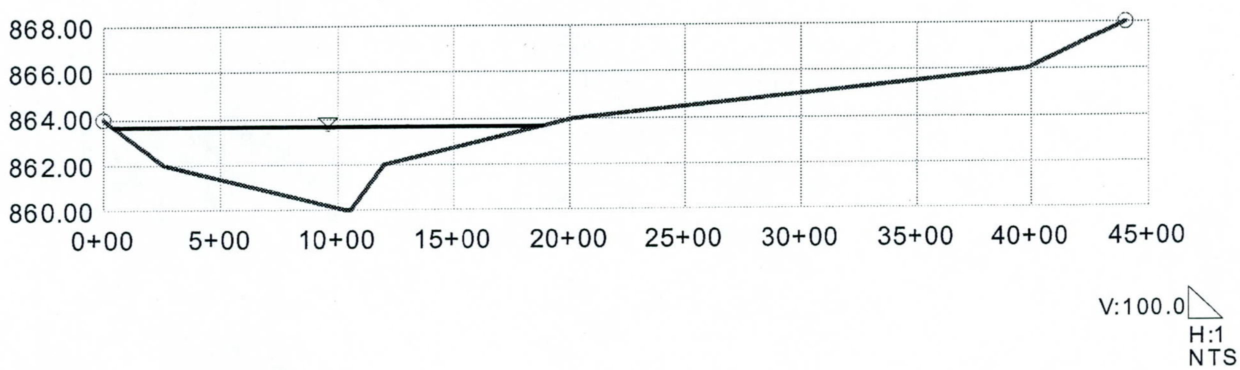


V:1000.0
H:1
NTS

Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 08 - Dean Rd
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coefficient	0.040	
Slope	0.001600	ft/ft
Water Surface Elevation	863.68	ft
Elevation Range	860.00 to 868.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

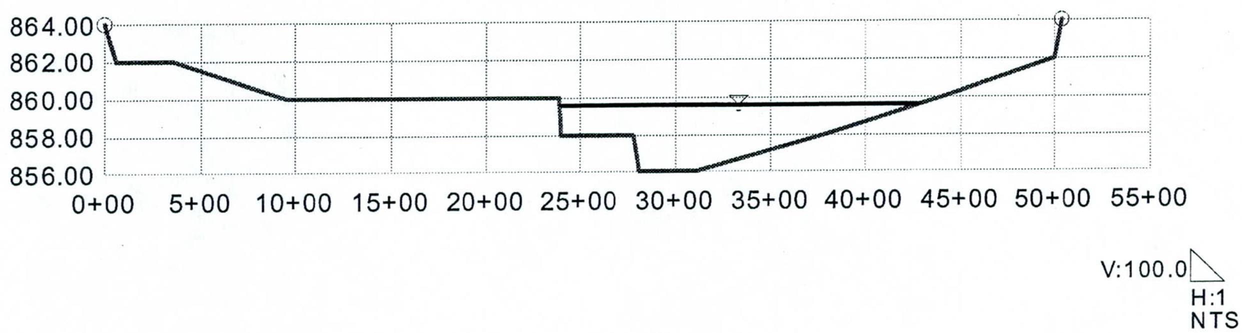
Project Description	
Worksheet	Cross section 09
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coefficient	0.040	
Slope	0.001000	ft/ft
Water Surface Elevation	862.97	ft
Elevation Range	860.00 to 864.00	
Discharge	7,300.00	cfs

Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 10 - u/s of Rainbow
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

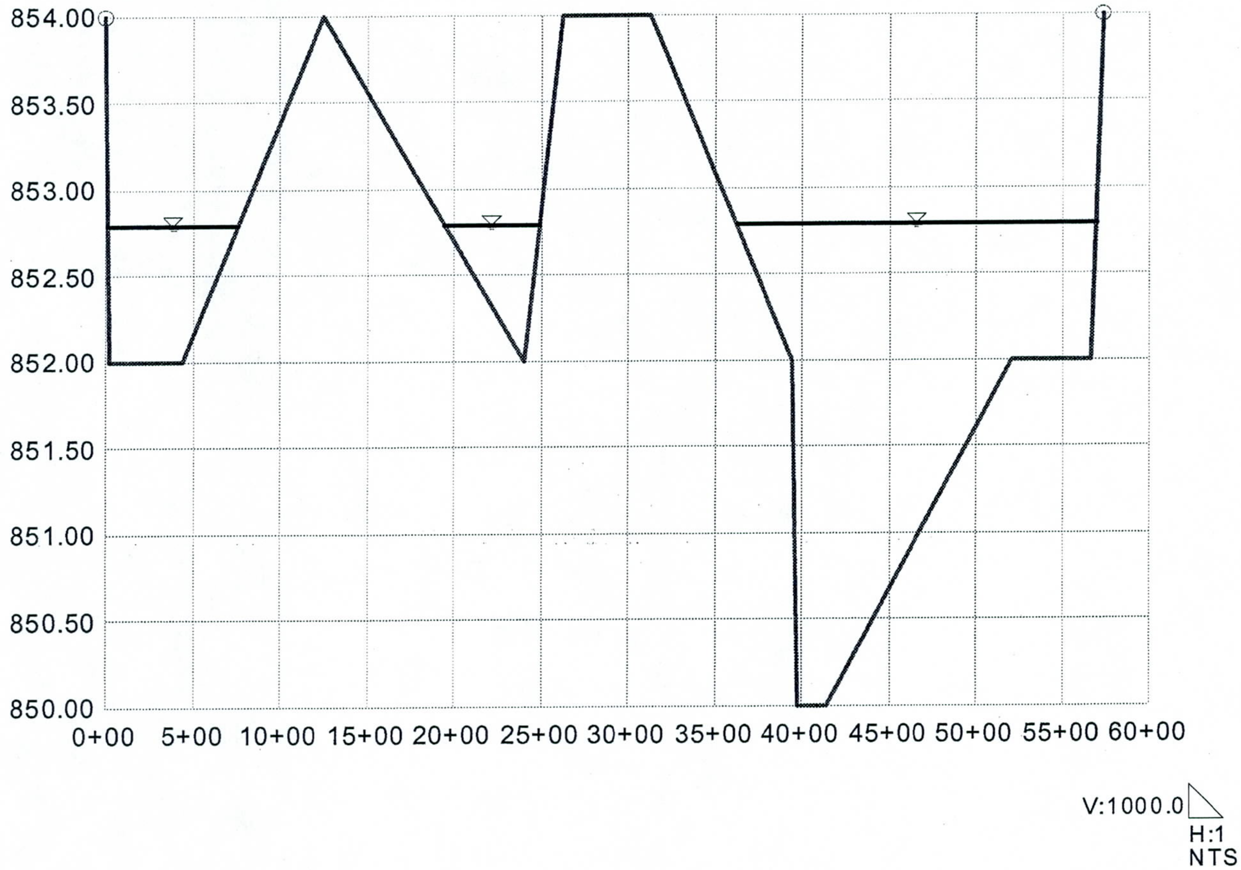
Section Data		
Mannings Coefficient	0.040	
Slope	0.001000	ft/ft
Water Surface Elevation	859.55	ft
Elevation Range	856.00 to 864.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 11
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

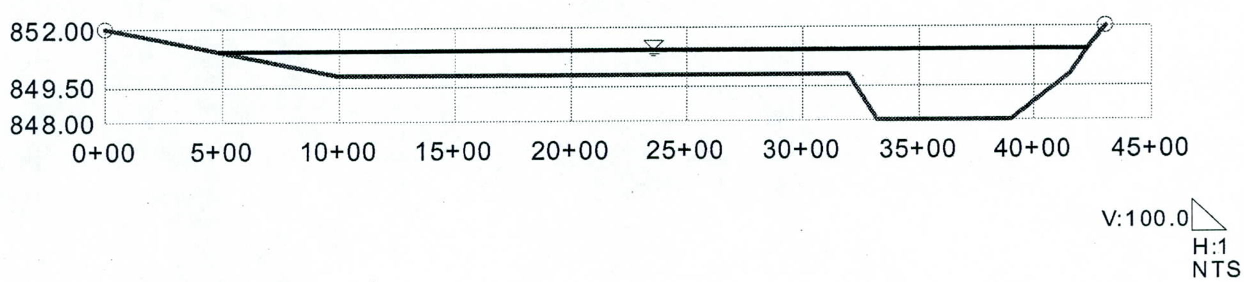
Section Data		
Mannings Coefficient	0.040	
Slope	0.002700	ft/ft
Water Surface Elevation	852.79	ft
Elevation Range	850.00 to 854.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 13 - Watson Rd
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

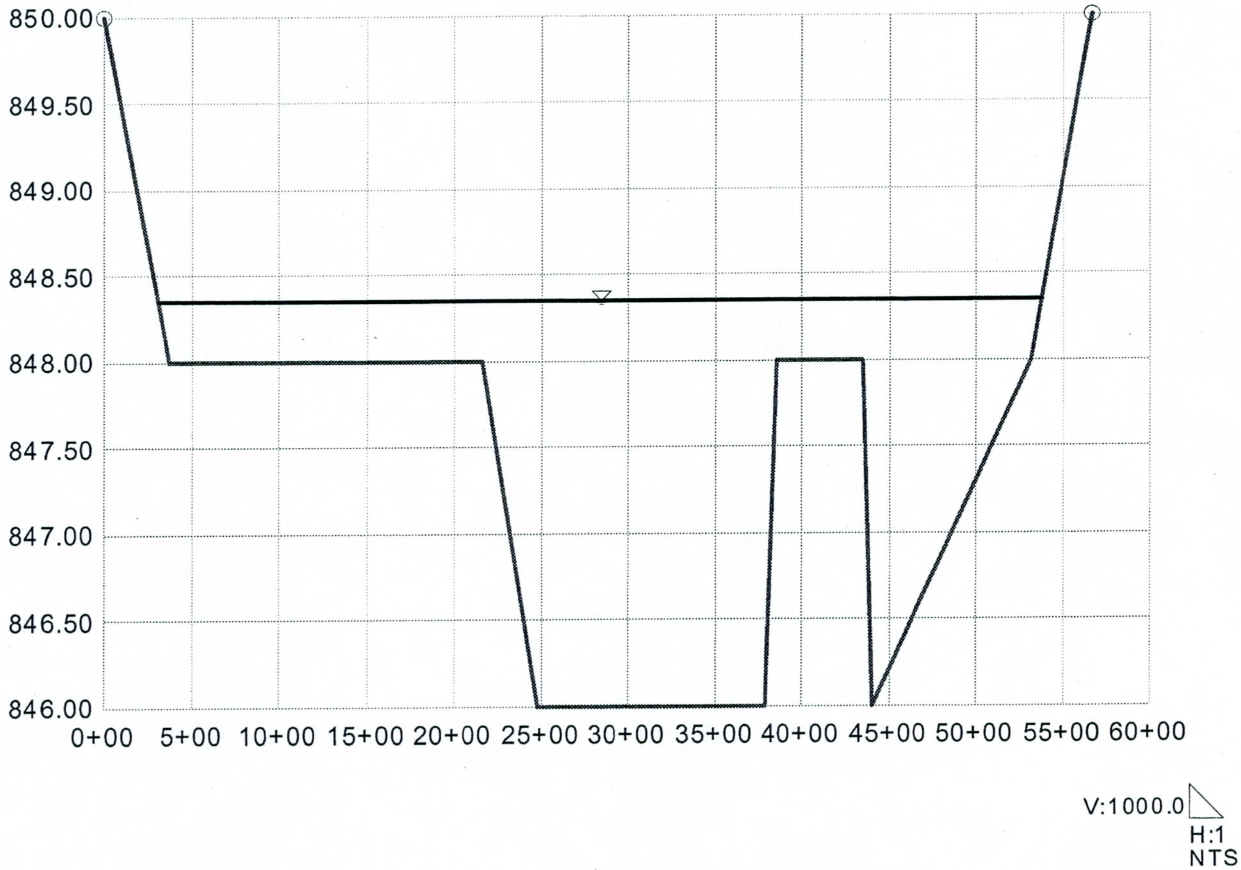
Section Data		
Mannings Coefficient	0.040	
Slope	0.001000	ft/ft
Water Surface Elevation	851.03	ft
Elevation Range	848.00 to 852.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 14
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Discharge

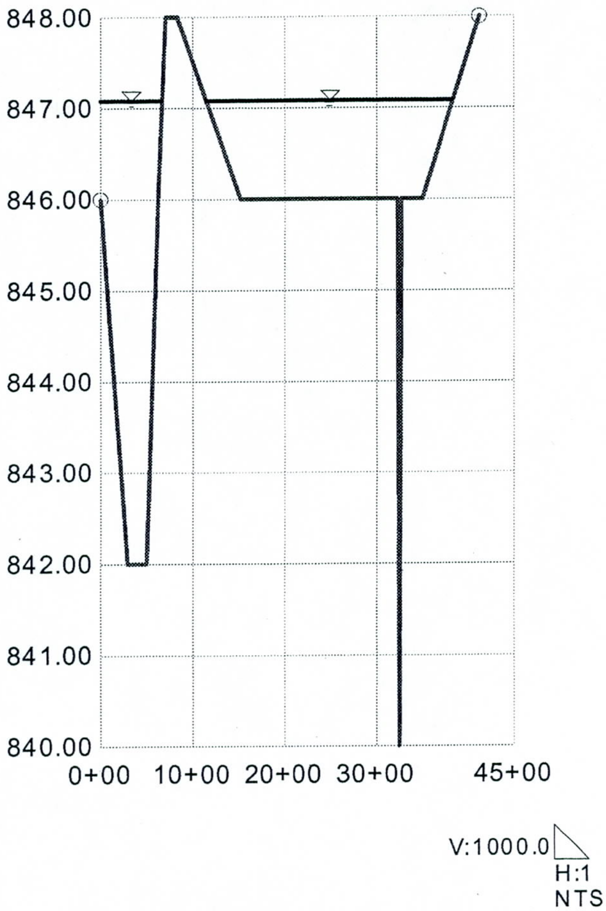
Section Data		
Mannings Coefficient	0.040	
Slope	0.001000	ft/ft
Water Surface Elevation	848.35	ft
Elevation Range	846.00 to 850.00	
Discharge	7,300.00	cfs



Cross Section
Cross Section for Irregular Channel

Project Description	
Worksheet	Cross section 15
Flow Element	Irregular Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coefficient	0.040	
Slope	0.000900	ft/ft
Water Surface Elevation	847.08	ft
Elevation Range	840.00 to 848.00	
Discharge	7,300.00	cfs



El Rio Watercourse Master Plan

Field Survey Memorandum

PREPARED FOR:

FLOOD CONTROL DISTRICT
of
Maricopa County

PREPARED BY:

STANTEC CONSULTING INC.
8211 South 48th Street
Phoenix, Arizona 85044
(602) 438-2200

November 2005

STANTEC CONSULTING Inc.
Project No. 8200240

Contract FCD 2001C024

Flood Control District of Maricopa County

Registered Professional Engineer (Civil)
CERTIFICATE NO
35150
R. MICHAEL C.
GERLACH
Date Signed 15 Nov 05
ARIZONA U.S.A.

R. Michael C. Gerlach

Stantec

El Rio Watercourse Master Plan

Field Survey Memorandum

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R. Michael C. Gerlach

Stantec

TABLE OF CONTENTS

Registered Professional Engineer (Civil)
CERTIFICATE NO
35150
R. MICHAEL C.
GERLACH
Date signed 15 Nov. 05
ARIZONA U.S.A.

R. Michael C. Gerlach

EL RIO WATERCOURSE MASTER PLAN - SURVEY AND MAPPING

BACKGROUND

The base hydraulic model for the El Rio Watercourse Master Plan (El Rio WMP) was prepared as part of the Salt-Gila River Floodplain Delineation Study. Topographic mapping developed for that study was originally prepared over a period of 1991 - 1992 at a contour interval of 4 feet. In February of 1993, portions of the river were remapped due to flooding during the month of January. The conditions in the river when it was remapped are shown in Figure 1. The limits of the revised mapping generally followed the maximum flood inundation area, shown in blue in Figure 1. The discharge in the river at the time of the remapping was approximately 15,000 cfs.

In 2004, new, detailed mapping for approximately 5.5 miles of the Gila River were prepared as part of the King Ranch development and proposed Cotton Lane Bridge. The upstream limit of the new mapping is the Estrella Parkway Bridge (river station 194.2). A hydraulic model for this reach of the river was prepared by the engineers for the King Ranch development and provided to Stantec. New cross sectional geometry and roughness coefficients prepared for this reach of the river were incorporated into the El Rio WMP base model.

Incorporation of the new data into the El Rio WMP base hydraulic model results in reduction of the 100-year water surface elevation through the King Ranch reach, which in turn reduces the water surface elevation upstream for a distance of approximately 3 miles as shown in Figure 2. The maximum reduction in water surface elevation is approximately 4 feet at river station 190.43 as shown in Figure 3. The region of the greatest reduction in water surface elevation coincides with the approximate location of the potential flow breakout identified in the floodplain delineation study and quantified as part of the El Rio WMP prior to the incorporation of the new mapping.

Comparison of the revised roughness coefficients to the original values prepared for the floodplain delineation study suggests that the major influence in water surface elevation differences is due to the circa 2004 mapping. The magnitude of the differences prompted an investigation into the map accuracy of both the circa 1992-93 and 2004 topographic mapping.

Figure 1
February 1993 aerial photography

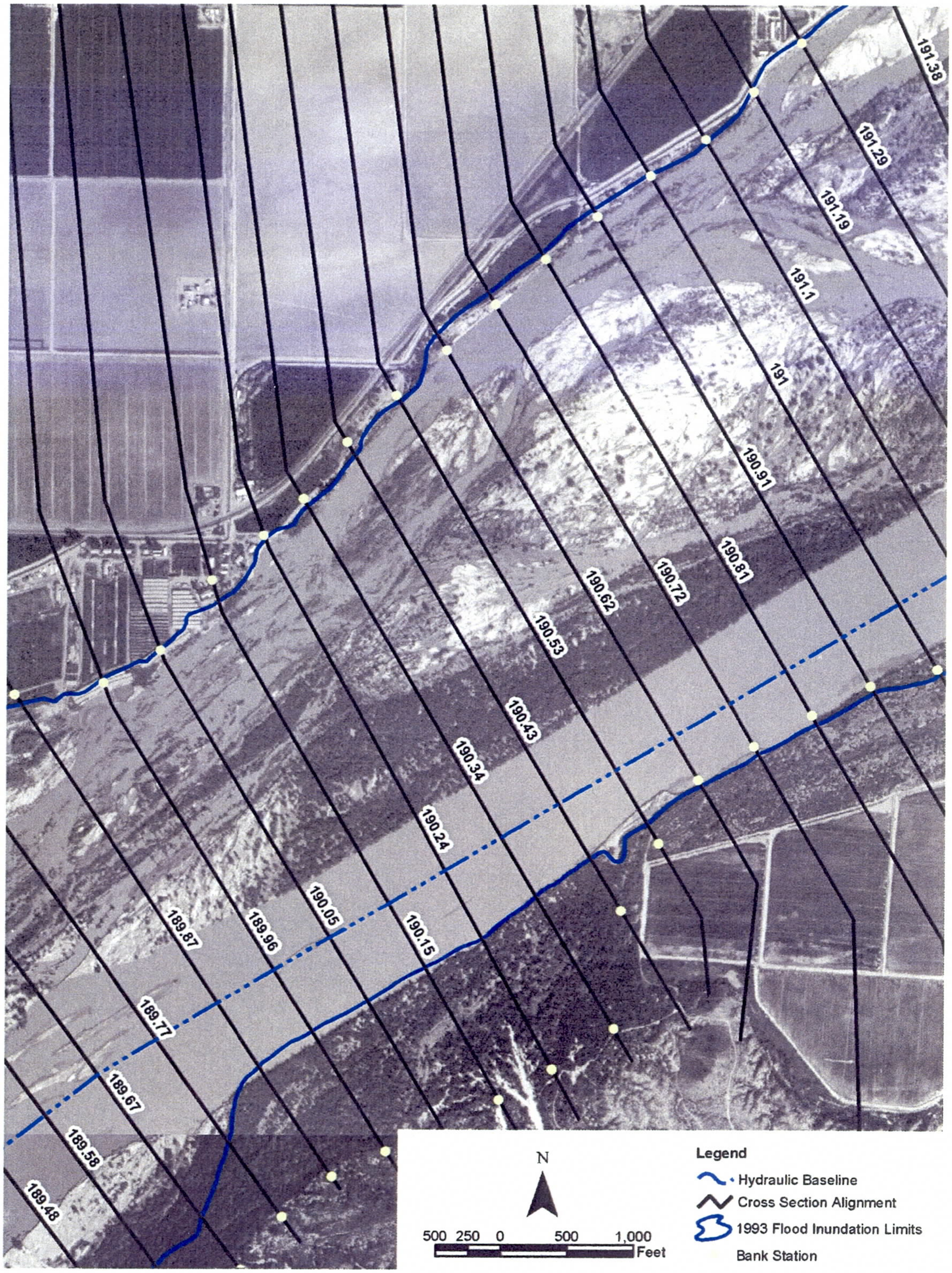


Figure 2
Water surface profile comparison

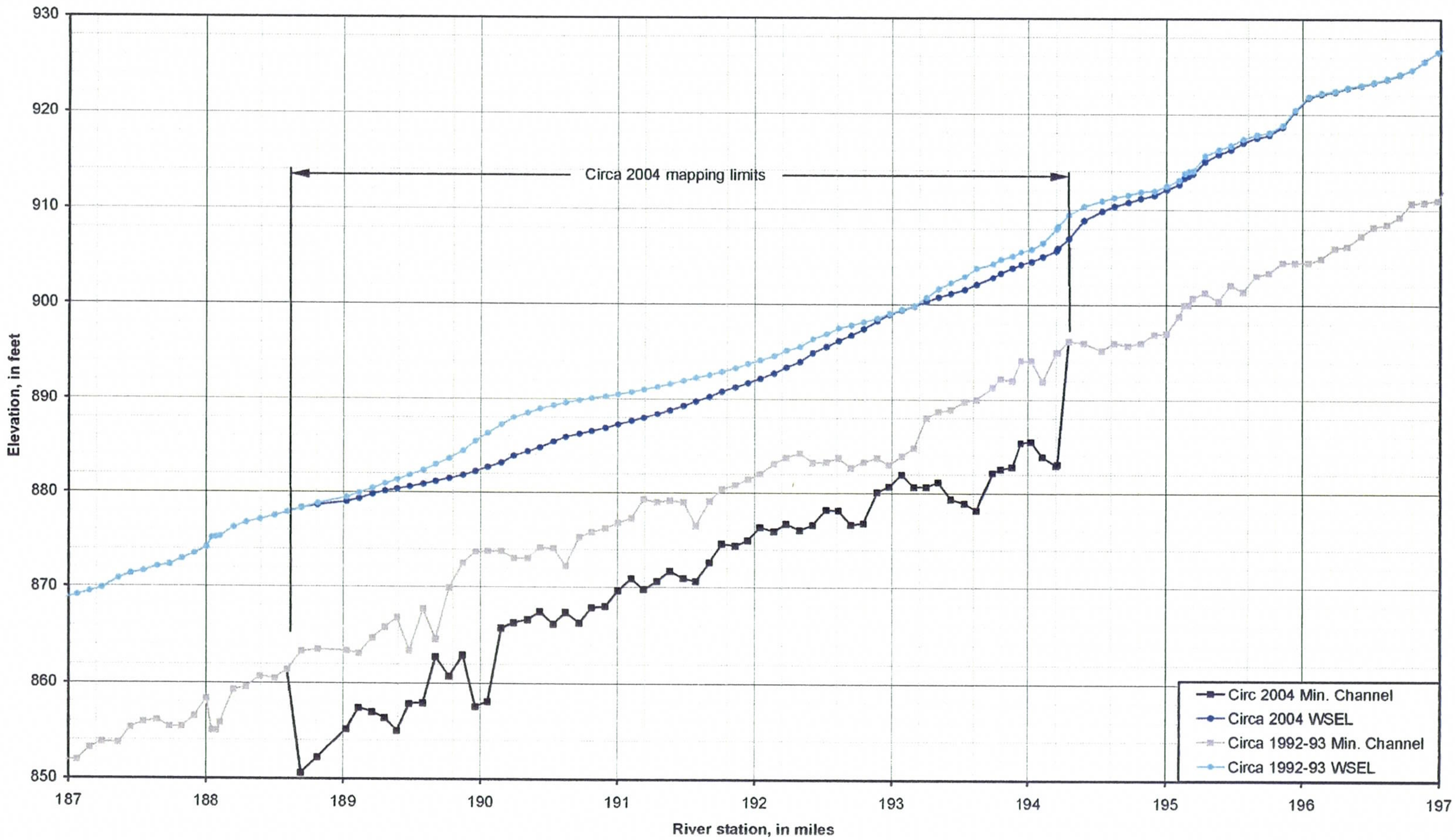
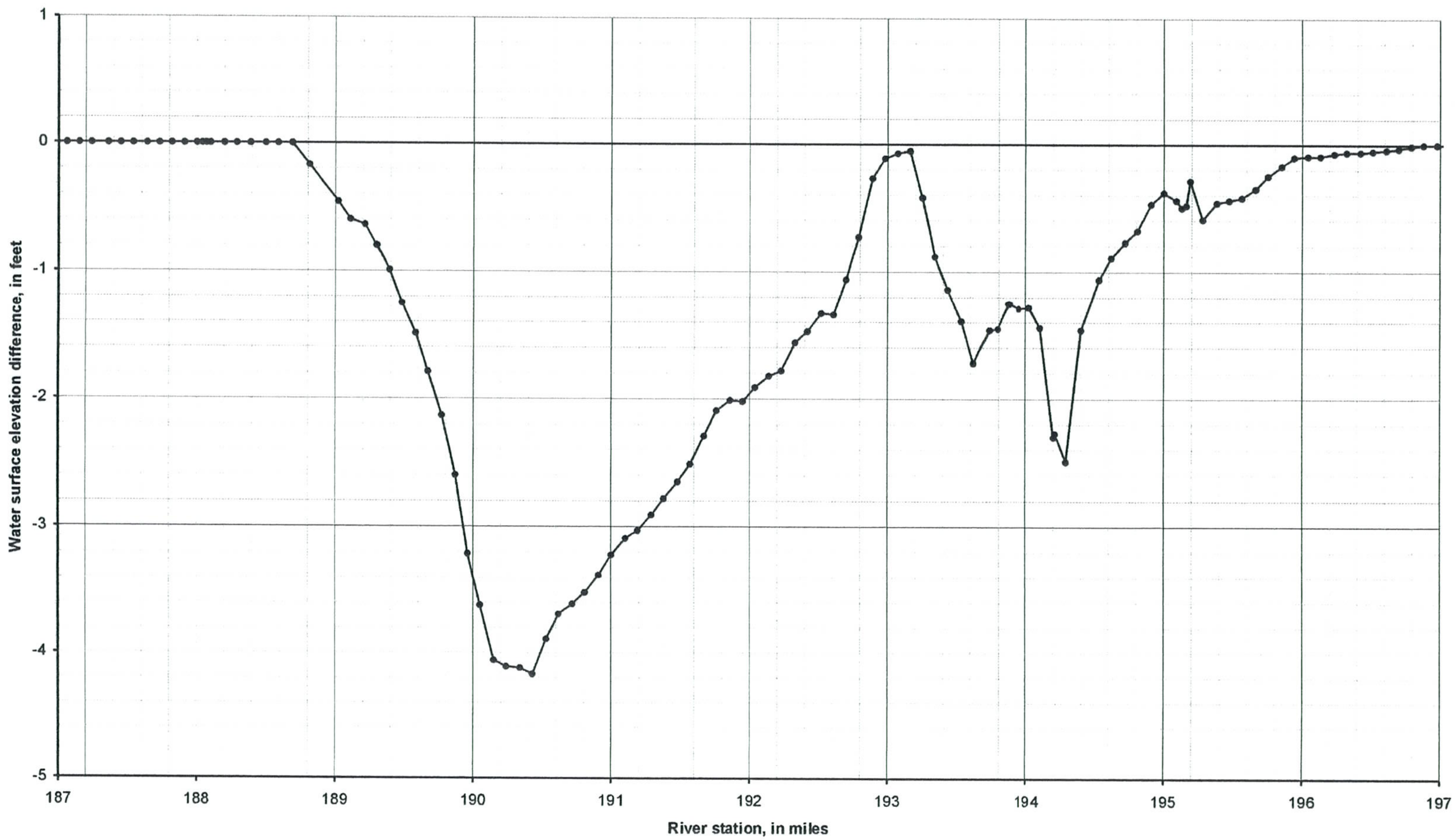


Figure 3
Water surface elevation comparison



MAP VERIFICATION

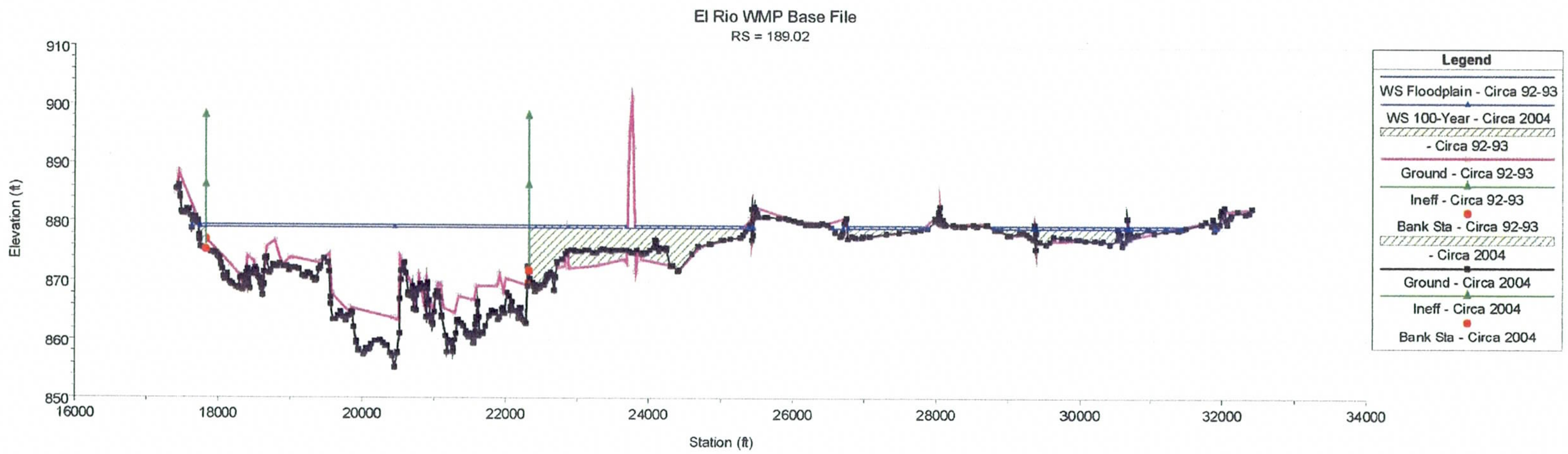
Map verification of the circa 2004 mapping was performed independently by the engineers for the King Ranch development, the Flood Control District of Maricopa County and Stantec. Only the results of the verification performed by Stantec are presented herein.

The first step in the verification process is a comparison of cross sectional geometry through the King Ranch reach from the floodplain delineation study and the El Rio WMP base model that includes the circa 2004 mapping. Comparisons of the cross sectional geometry at the upstream and downstream limits of the circa 2004 mapping are shown in Figure 4. Inspection of that figure shows the circa 2004 cross sectional geometry to be essentially the same as the circa 1992-93 geometry given the differences in contour intervals (4-foot for the circa 1992-93 mapping and 1-foot for the circa 2004 mapping), sampling limitation differences (floodplain delineation model was limited to 100 station elevation data points) and the influence of flooding since the completion of the circa 1992-93 mapping. However, for the reach from river station 189.68 to 190.15 there is a deviation in the two data sets as illustrated in Figure 5. This deviation triggered the second step in the verification process, which consisted of a ground survey.

Two cross sectional alignments in the King Ranch reach were ground surveyed in June and August of 2005 and plotted against the circa 1992-93 and 2004 cross sectional geometry. The two cross sectional alignments surveyed are at river station 189.22 and 189.87. In addition, the original circa 1991-92 mapping was sampled along those alignments and plotted against the other data sets. The comparison plots are shown in Figures 6 and 7. At both locations, the ground survey and the circa 2004 mapping are essentially the same. Furthermore, outside of the main channel (flow line at station 200+00), the circa 1991-92 (pre-flood) geometry reasonably close to the ground survey and circa 2004 mapping. This suggests a "anomaly" in the February 1993 remapped data within this area.

To try and determine the downstream extent of the "anomaly" in the circa 1992-93 mapping and investigate potentially similar conditions near the SR 85 Bridge additional ground surveys were conducted at river stations 188.10 and 180.94. The results of the ground survey are plotted in Figures 8 and 9 along with the cross sectional geometry from the circa 1992-93 mapping and 1991-92 mapping. At river station 188.10 (Figure 8), all three data sets appear within reason of each other considering the presence of a flow of approximately 15,000 cfs at the time of the circa 1992-93 mapping. At river station 180.94 (Figure 9), interpretation of the data is complicated by the general lack of agreement between the three data sets except in the left overbank area. One point of interest is the potential difference in conveyance in the main channel between the circa 1992-93 mapping and the ground survey.

Figure 4
Cross sectional geometry comparison at river station 189.02 and 194.01



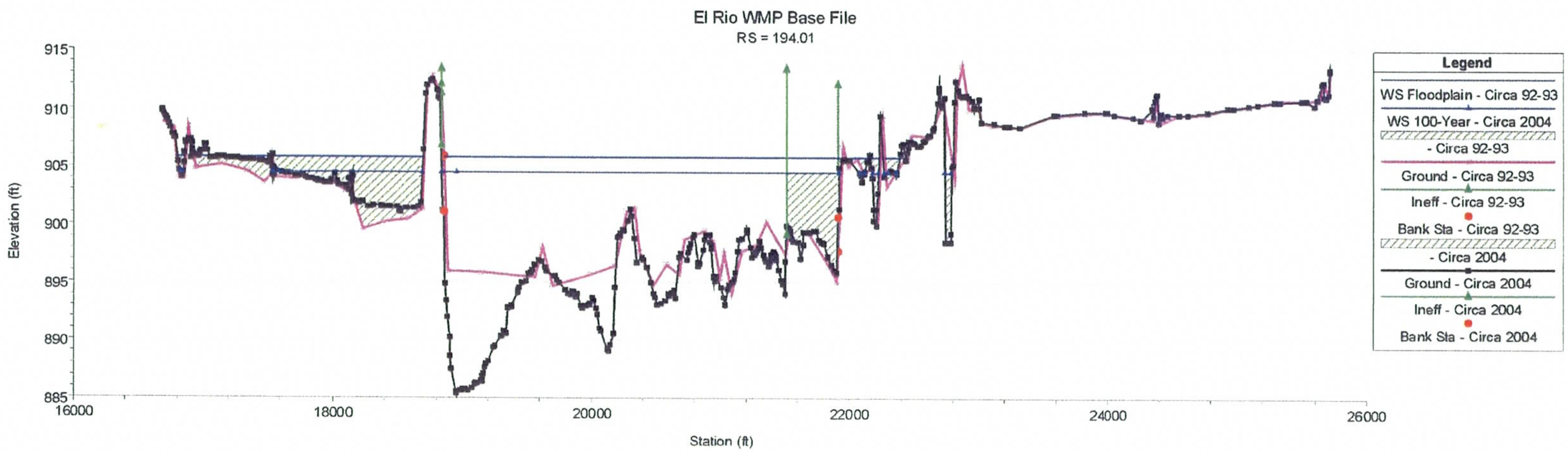
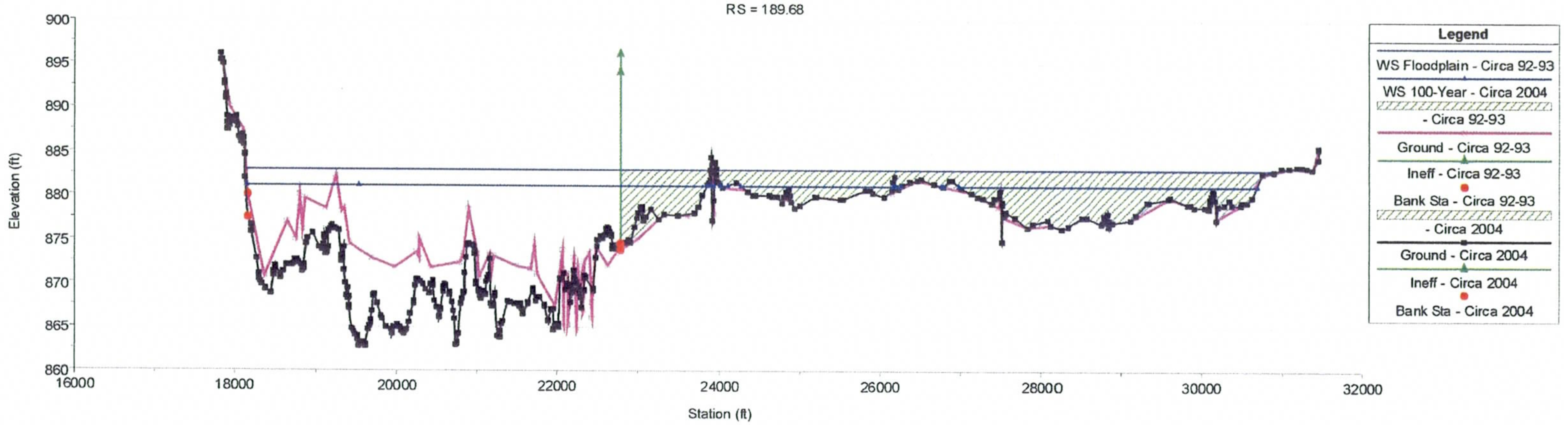


Figure 5
Cross sectional geometry comparison at river station 189.68 and 190.15



7

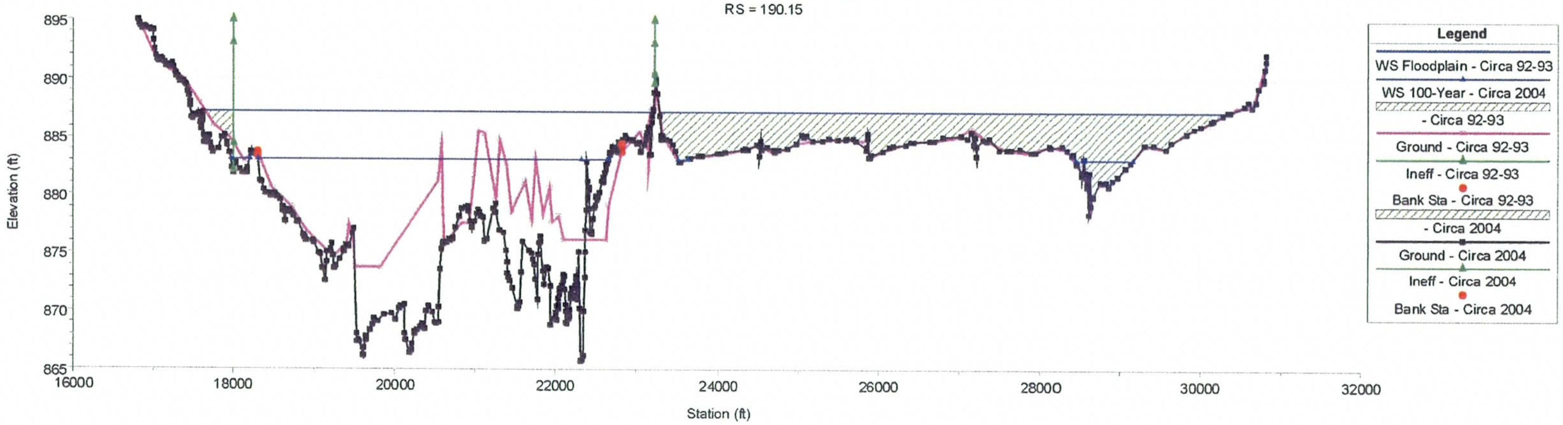


Figure 6
Cross section comparison at river station 189.22

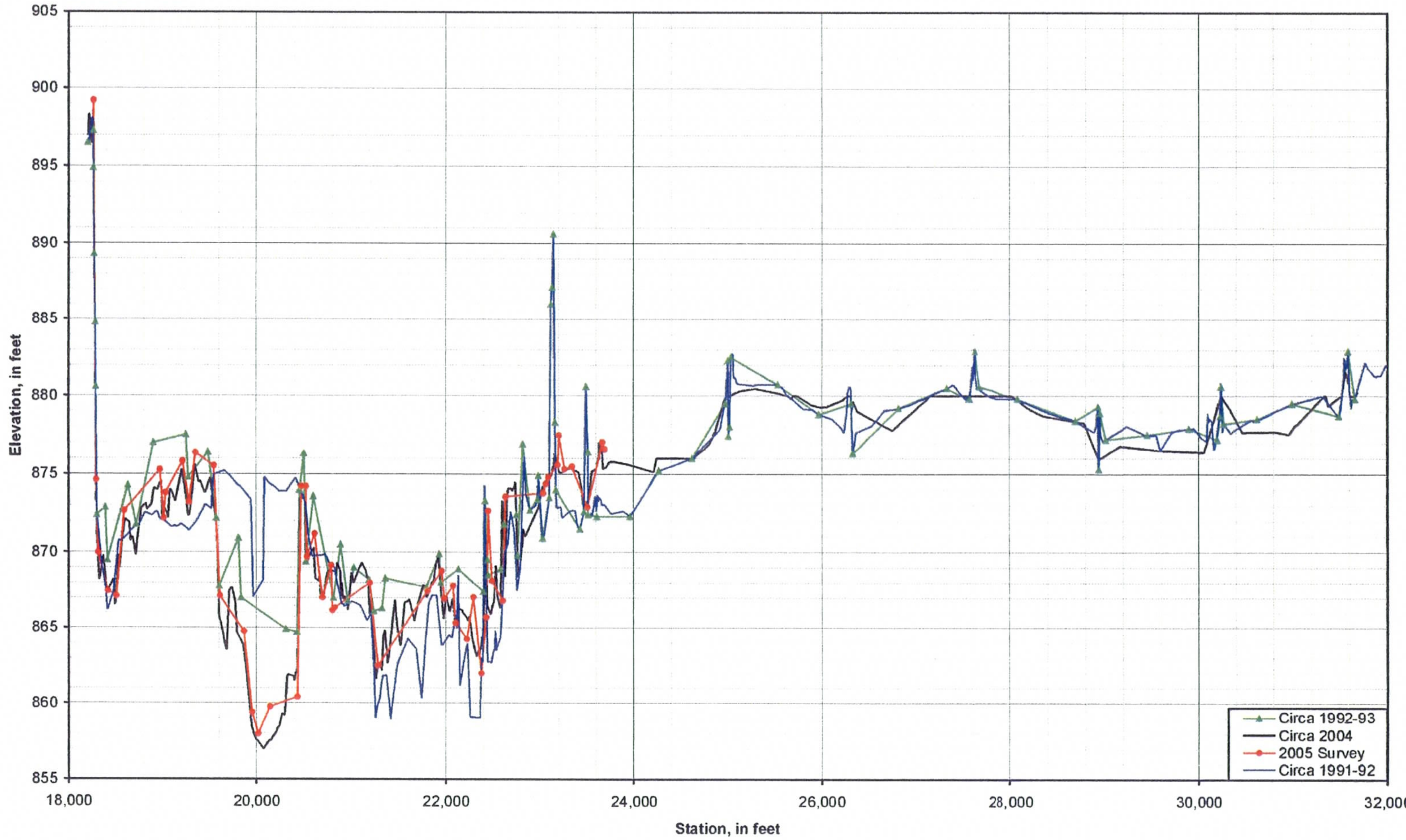


Figure 7
Cross section comparison at river station 189.87

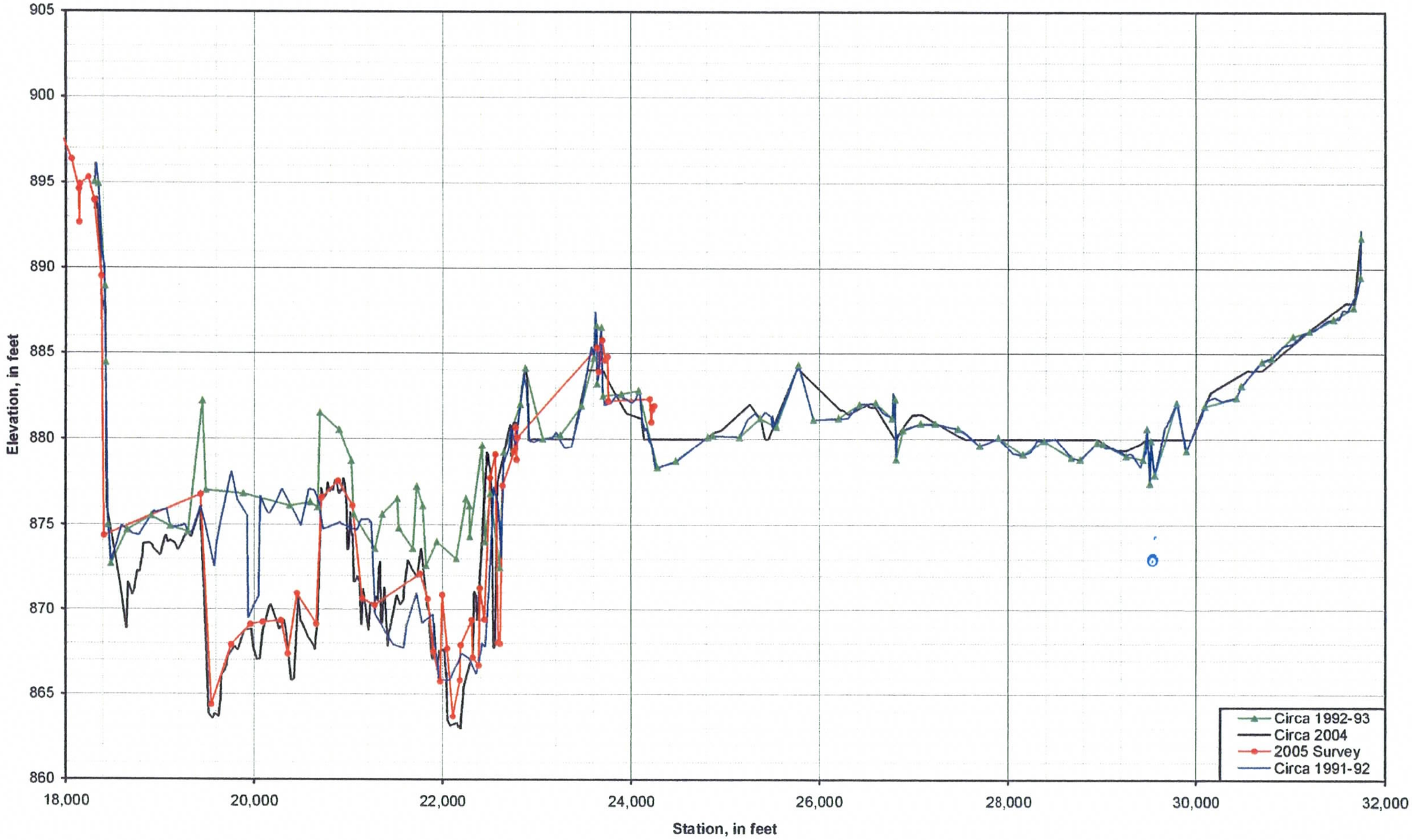


Figure 8
Cross section comparison at river station 188.10

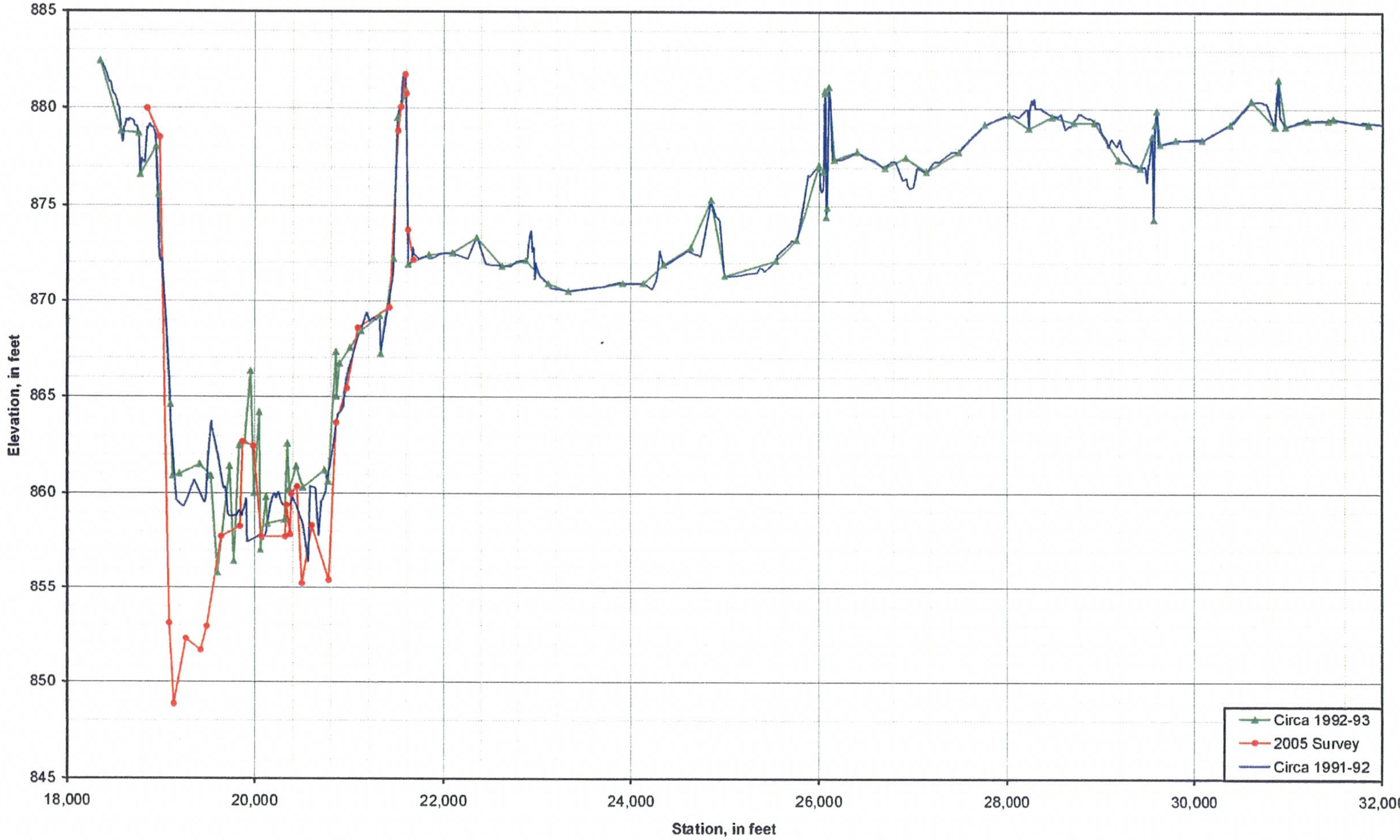
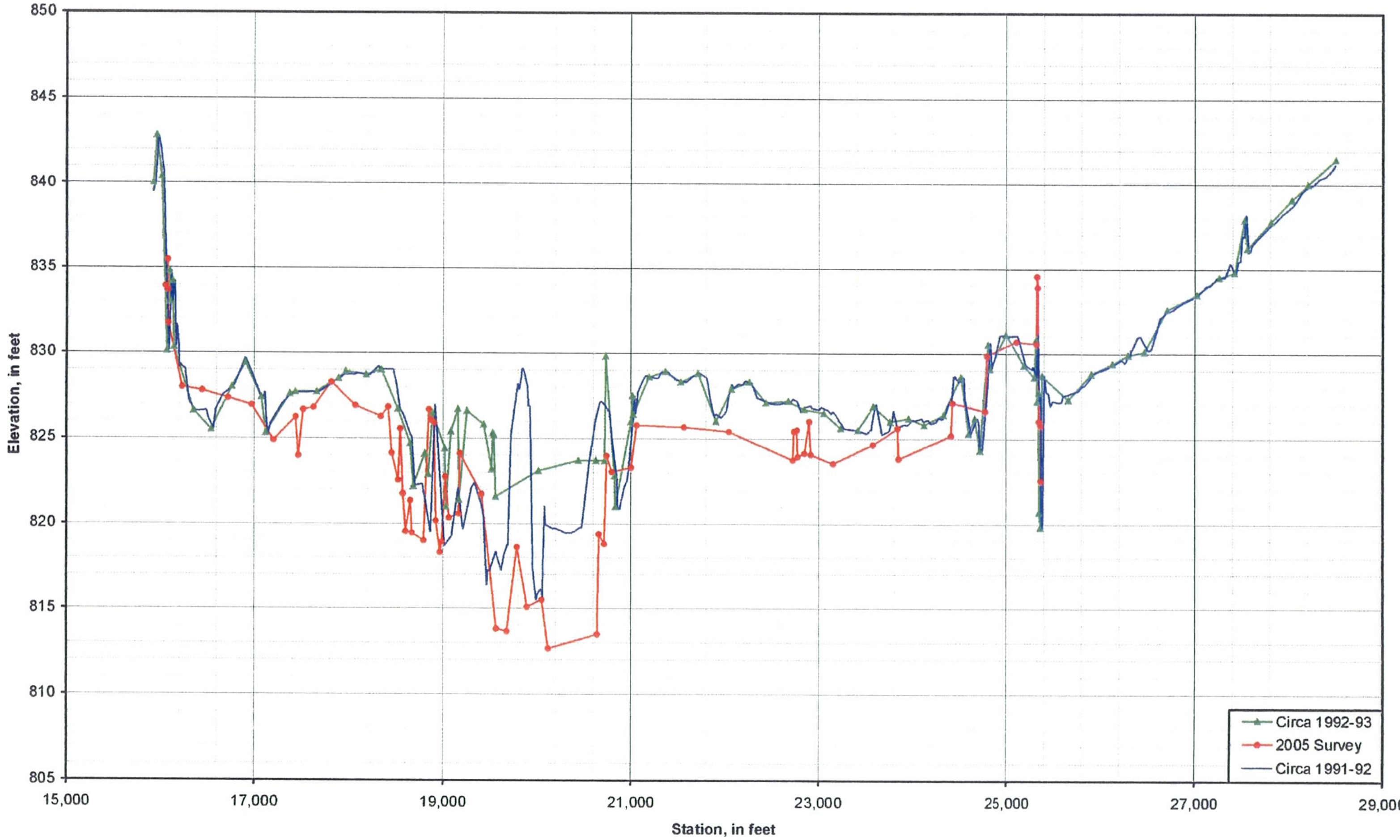


Figure 9
Cross section comparison at river station 180.94



11

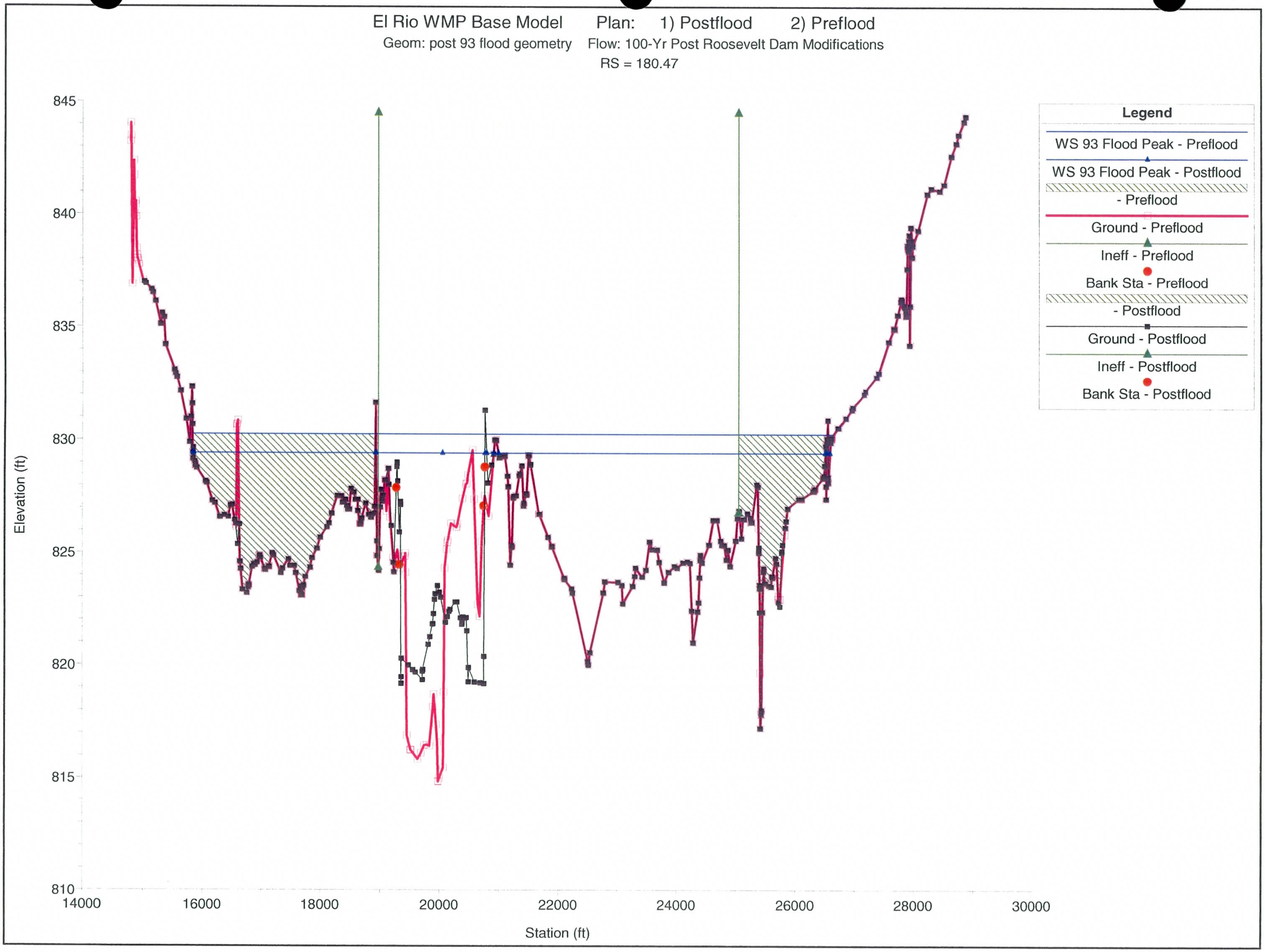
PRE- AND POST 1993 FLOOD MAPPING COMPARISON

Review of the mapping data sets presented in Figures 6 through 9 led to a comparison of the circa 1991-92 (pre-flood) and the circa 1992-93 (post-flood) mapping. Of particular interest was the difference in the right overbank area that was inundated during the 1993 flood. Both the pre-flood and post-flood mapping data sets were sampled along each cross sectional alignment and the resulting geometries compared. At most cross sections, the pre- and post-flood geometry in the right overbank is exactly identical. This is illustrated in the attached set of figures.

CONCLUSION

The results of the map verification show that the circa 2004 topographic mapping prepared for the King Ranch development and proposed Cotton Lane Bridge compare very closely with ground survey data. The results also show that there is an apparent "anomaly" in the circa 1992-93 (post-flood) mapping. The extent of the "anomaly" as it can be determined with the available data, appears to be limited to a short reach near the downstream limits of the circa 2004 mapping. Additional ground survey data shows that there is some uncertainty in the accuracy of the circa 1992-93 mapping in the vicinity of the SR 85 Bridge. Furthermore, a comparison of the pre- and post-flood mapping shows that much of the right overbank area was not remapped after the 1993 flood.

In regard to the proposed features of the El Rio WMP, it is felt that the mapping issues do not change the recommendations of the plan, only the magnitudes of the proposed levee/bank protection dimensions. At a minimum, bank protection is still required to prevent lateral migration. The proposed alignment of the levee/bank protection follows the natural topographic features, which are typically at the edge of the 1993 flood limits and are likely not significantly influenced by potential mapping errors. However, design of future bank protection/levees within the El Rio WMP study limits must be based on new topography that not only extends sufficiently upstream and downstream of the project limits to reduce the potential influence associated with any existing mapping issues, but across the entire width of the channel.

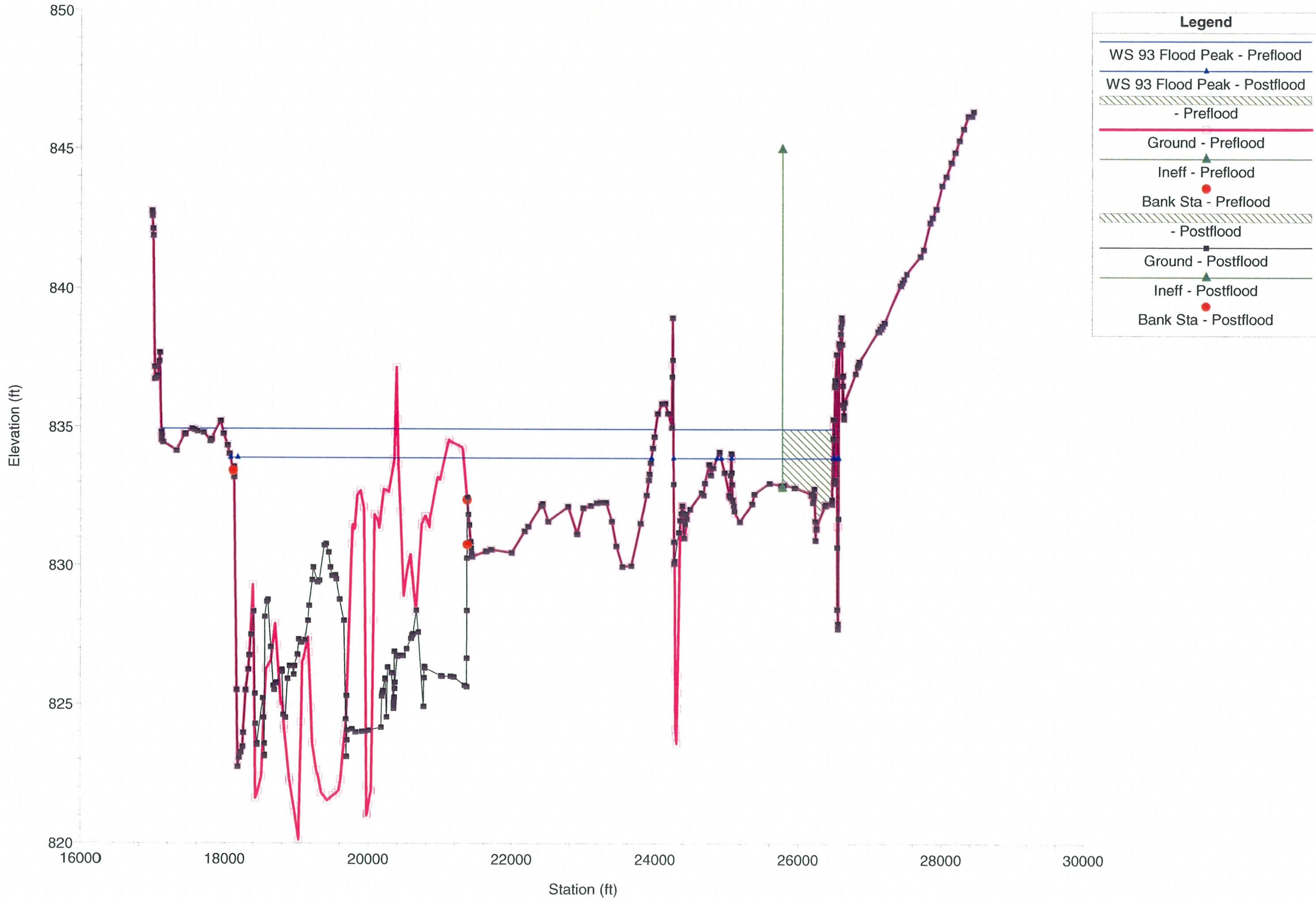


El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 180.47
845
840
835
830
825
820
815
810
Elevation (ft)
14000
16000
18000
20000
22000
24000
26000
28000
30000
Station (ft)
Legend
WS 93 Flood Peak - Preflood
WS 93 Flood Peak - Postflood
- Preflood
Ground - Preflood
Ineff - Preflood
Bank Sta - Preflood
- Postflood
Ground - Postflood
Ineff - Postflood
Bank Sta - Postflood

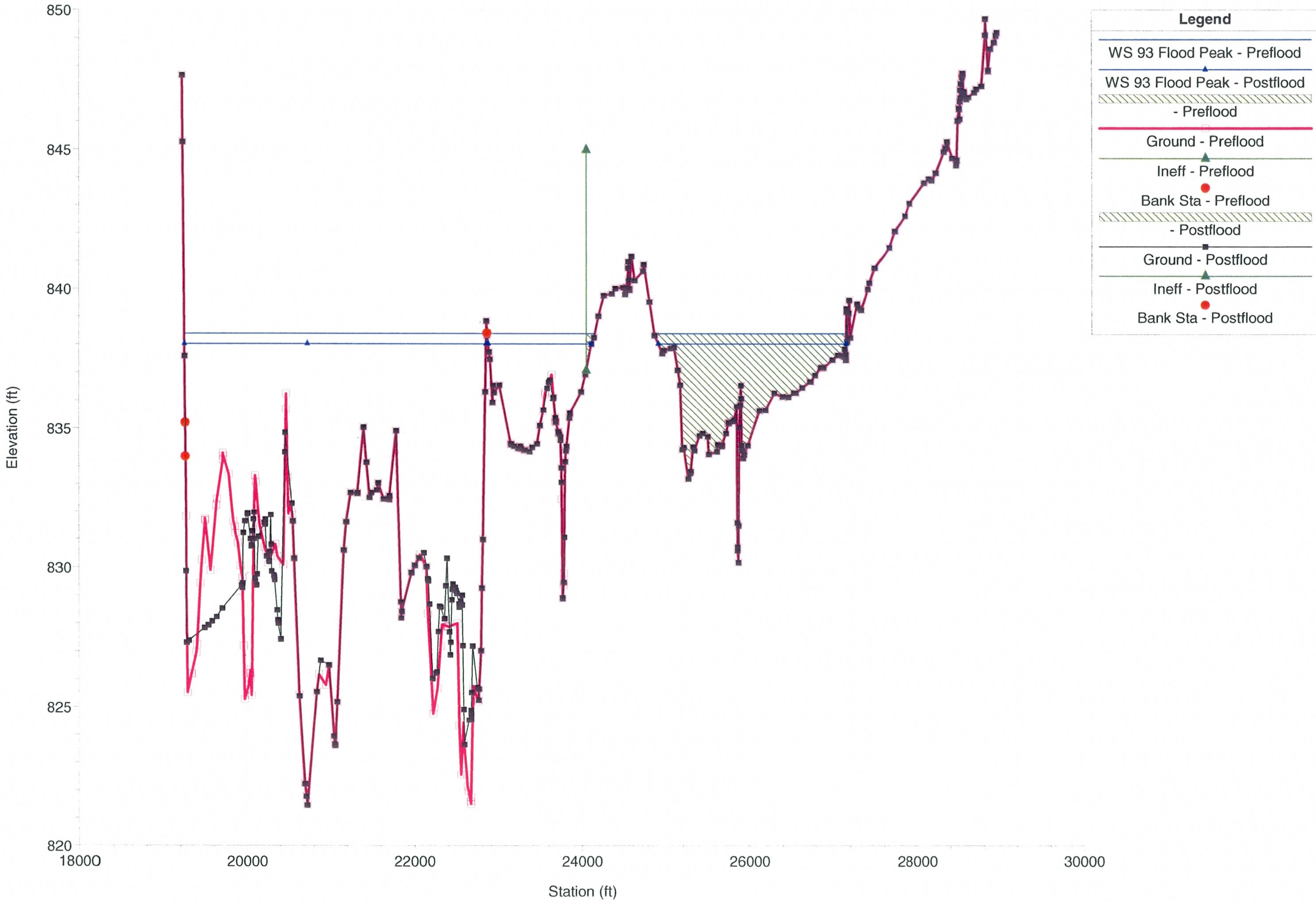
El Rio WMP Base Model Plan: 1) Postflood 2) Preflood

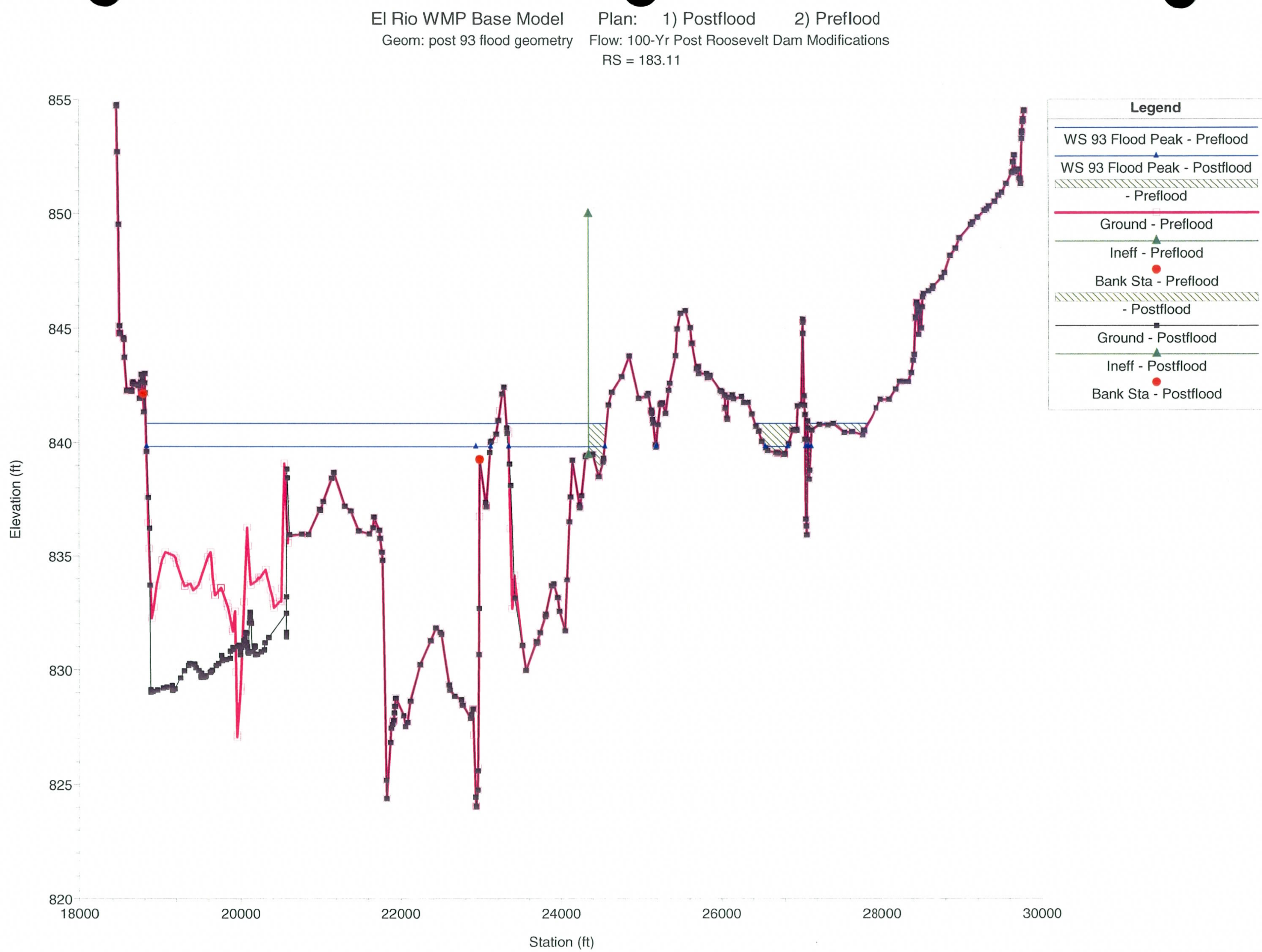
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications

RS = 181.62



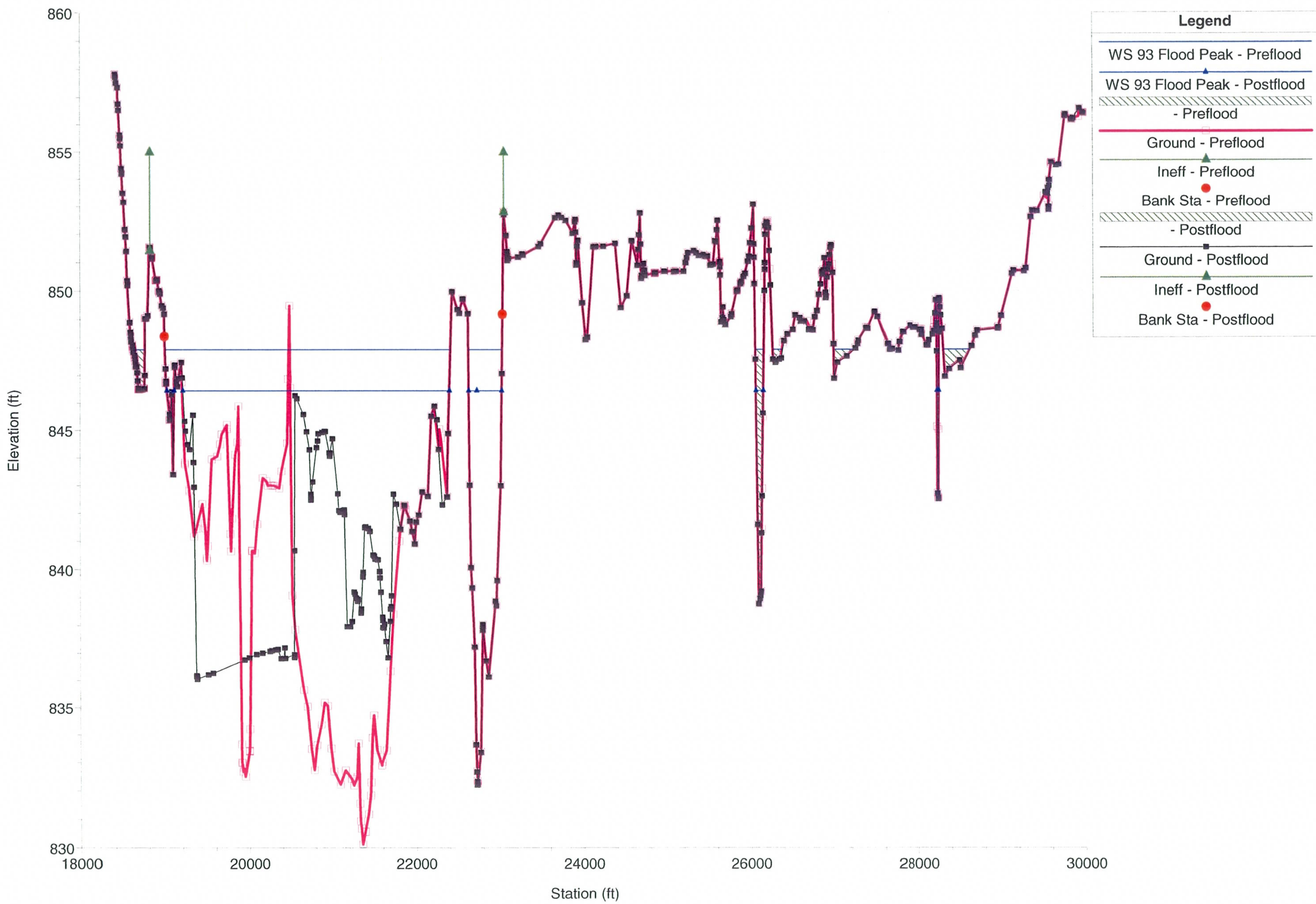
El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 182.45





El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 183.11
Legend
WS 93 Flood Peak - Preflood
WS 93 Flood Peak - Postflood
- Preflood
Ground - Preflood
Ineff - Preflood
Bank Sta - Preflood
- Postflood
Ground - Postflood
Ineff - Postflood
Bank Sta - Postflood
Elevation (ft)
Station (ft)
855
850
845
840
835
830
825
820
18000
20000
22000
24000
26000
28000
30000

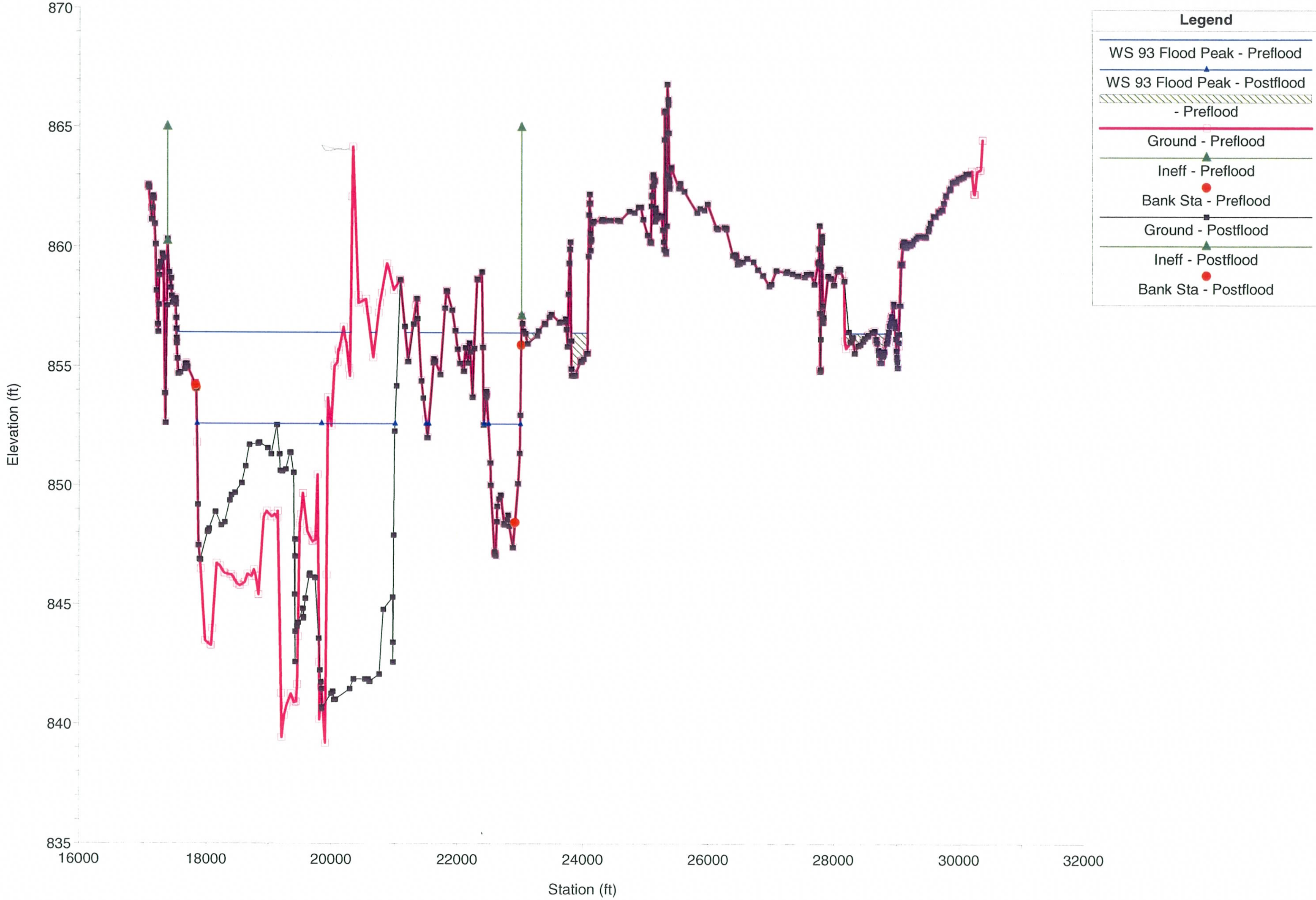
El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 184.05



El Rio WMP Base Model Plan: 1) Postflood 2) Preflood

Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications

RS = 185.28

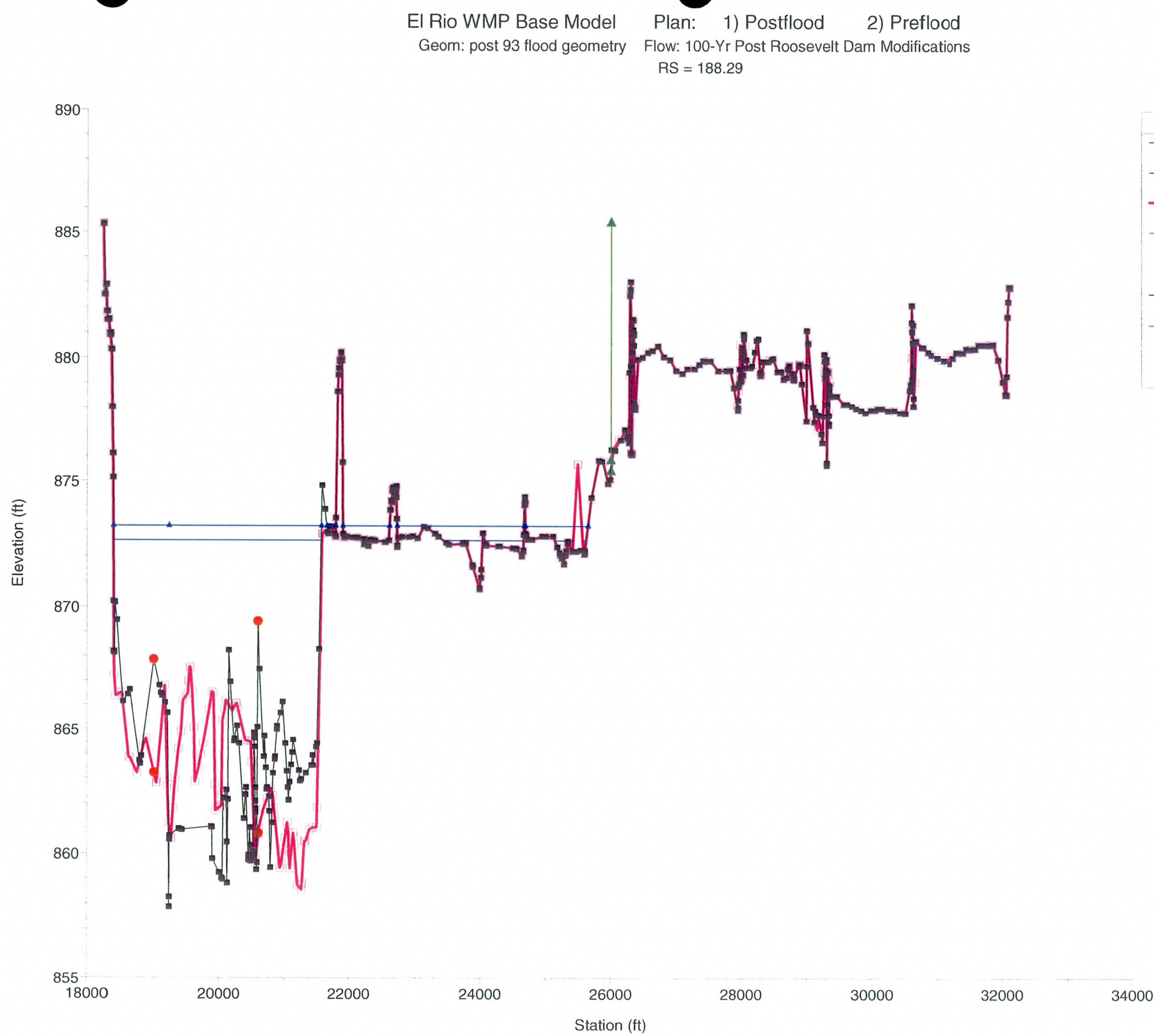



El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 186
Legend
WS 93 Flood Peak - Preflood
WS 93 Flood Peak - Postflood
- Preflood
Ground - Preflood
Ineff - Preflood
Bank Sta - Preflood
- Postflood
Ground - Postflood
Ineff - Postflood
Bank Sta - Postflood
Elevation (ft)
Station (ft)
875
870
865
860
855
850
845
840
16000
18000
20000
22000
24000
26000
28000
30000
32000

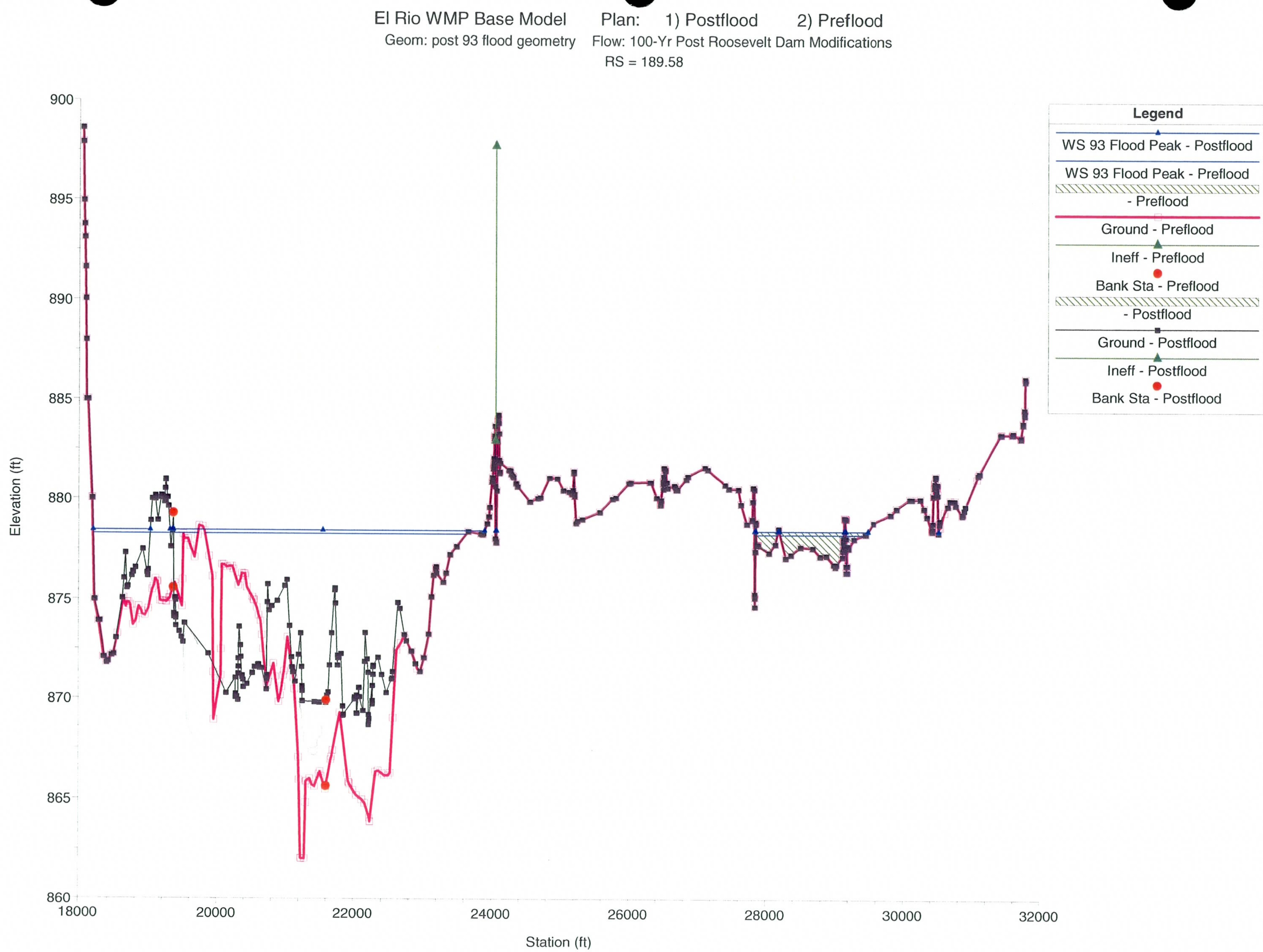
El Rio WMP Base Model Plan: 1) Postflood 2) Preflood

Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications

RS = 187.24

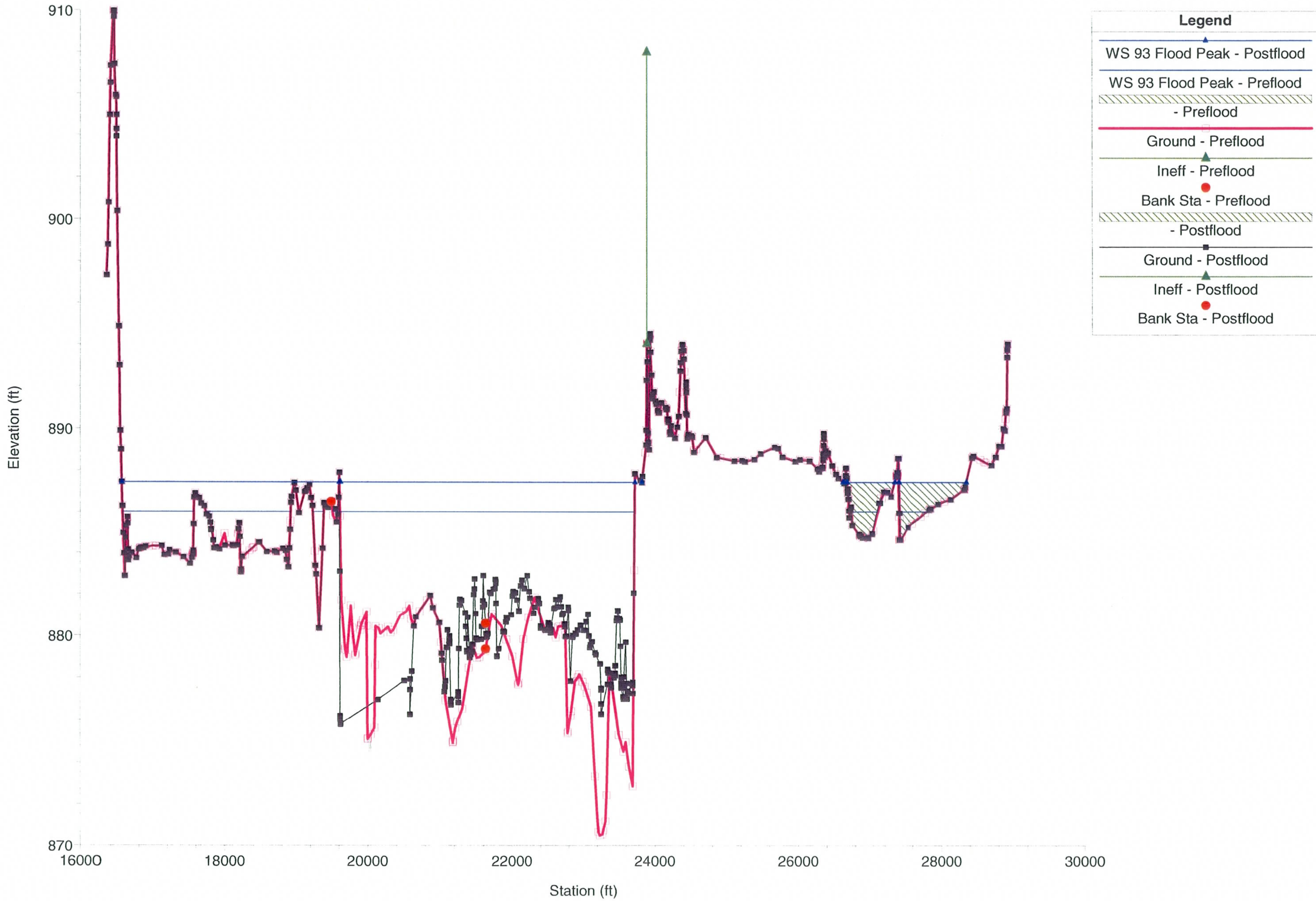


El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 188.29
Legend
WS 93 Flood Peak - Postflood
WS 93 Flood Peak - Preflood
Ground - Preflood
Ineff - Preflood
Bank Sta - Preflood
Ground - Postflood
Ineff - Postflood
Bank Sta - Postflood
Elevation (ft)
890
885
880
875
870
865
860
855
Station (ft)
18000
20000
22000
24000
26000
28000
30000
32000
34000



El Rio WMP Base Model
Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry
Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 189.58
Legend
WS 93 Flood Peak - Postflood
WS 93 Flood Peak - Preflood
- Preflood
Ground - Preflood
Ineff - Preflood
Bank Sta - Preflood
- Postflood
Ground - Postflood
Ineff - Postflood
Bank Sta - Postflood
Elevation (ft)
900
895
890
885
880
875
870
865
860
Station (ft)
18000
20000
22000
24000
26000
28000
30000
32000

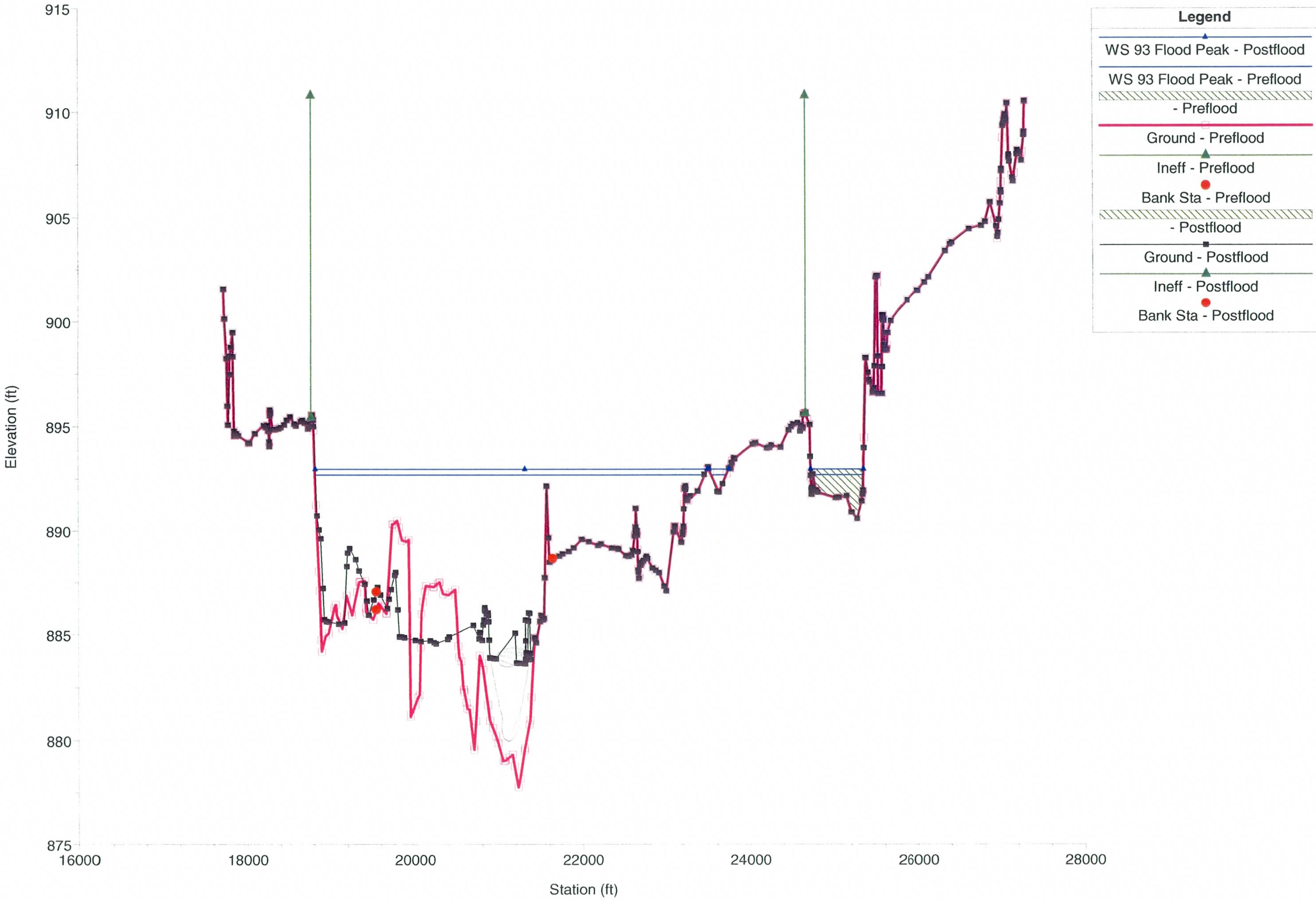
El Rio WMP Base Model   Plan: 1) Postflood   2) Preflood
Geom: post 93 flood geometry   Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 190.8



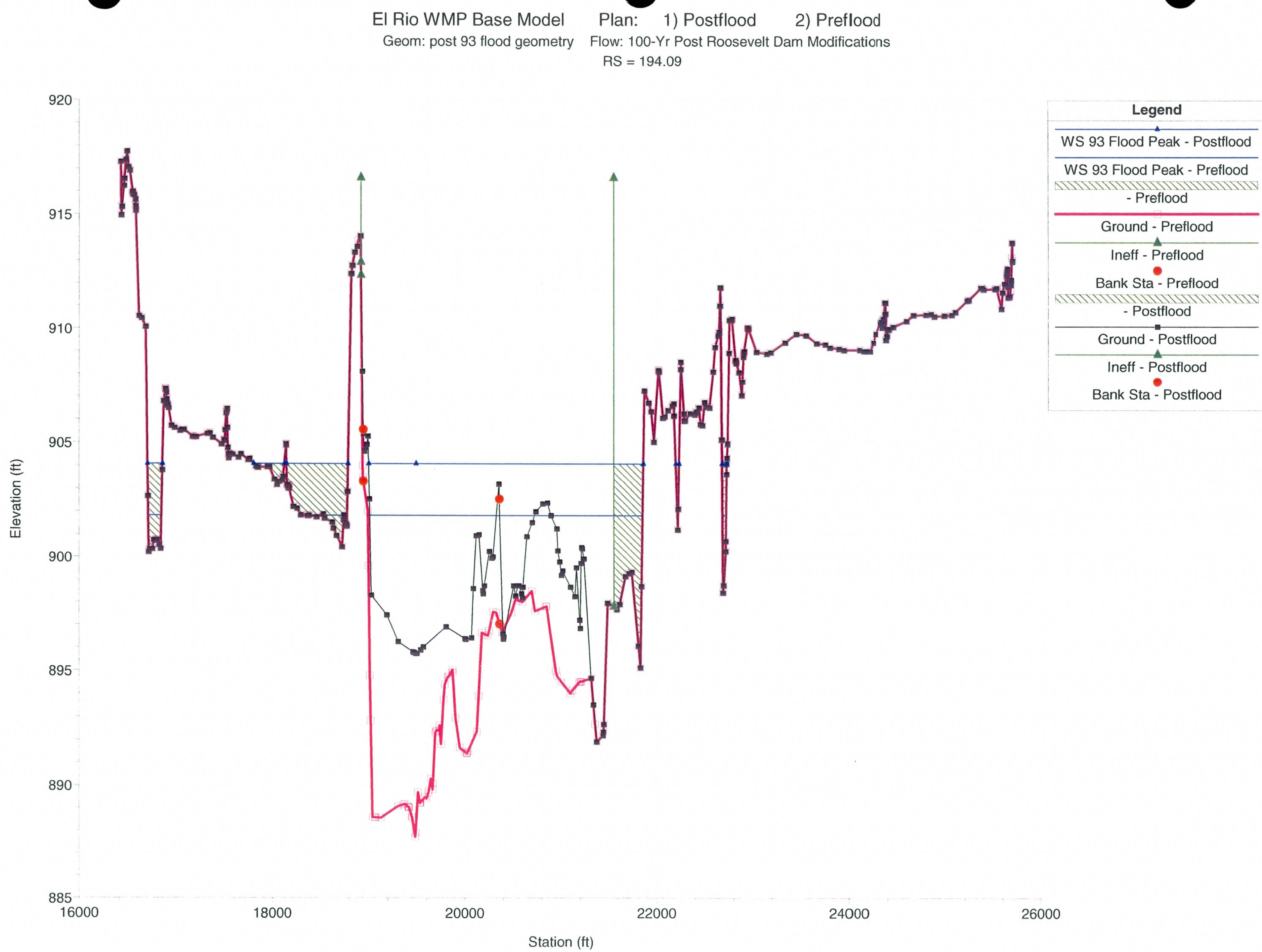
El Rio WMP Base Model Plan: 1) Postflood 2) Preflood

Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications

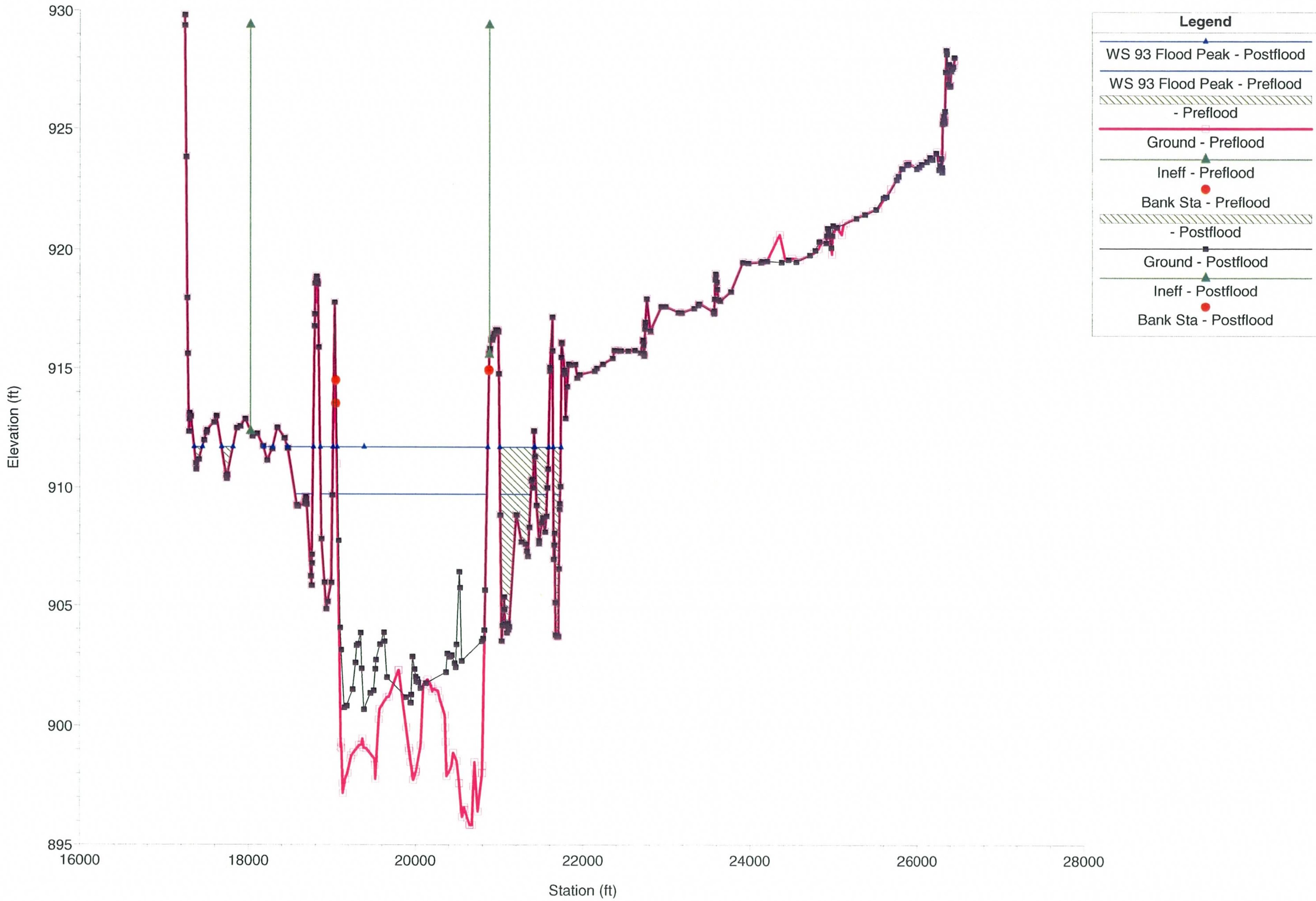
RS = 192.23




El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 193.45
Legend
WS 93 Flood Peak - Postflood
WS 93 Flood Peak - Preflood
- Preflood
Ground - Preflood
Ineff - Preflood
Bank Sta - Preflood
- Postflood
Ground - Postflood
Ineff - Postflood
Bank Sta - Postflood
Elevation (ft)
Station (ft)
920
915
910
905
900
895
890
885
18000
20000
22000
24000
26000


El Rio WMP Base Model Plan: 1) Postflood 2) Preflood
Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 194.09
Legend
WS 93 Flood Peak - Postflood
WS 93 Flood Peak - Preflood
- Preflood
Ground - Preflood
Ineff - Preflood
Bank Sta - Preflood
- Postflood
Ground - Postflood
Ineff - Postflood
Bank Sta - Postflood
Elevation (ft)
Station (ft)
920
915
910
905
900
895
890
885
16000
18000
20000
22000
24000
26000

El Rio WMP Base Model   Plan: 1) Postflood   2) Preflood
Geom: post 93 flood geometry   Flow: 100-Yr Post Roosevelt Dam Modifications
RS = 195.19



El Rio WMP Base Model Plan: 1) Postflood 2) Preflood

Geom: post 93 flood geometry Flow: 100-Yr Post Roosevelt Dam Modifications

RS = 195.75