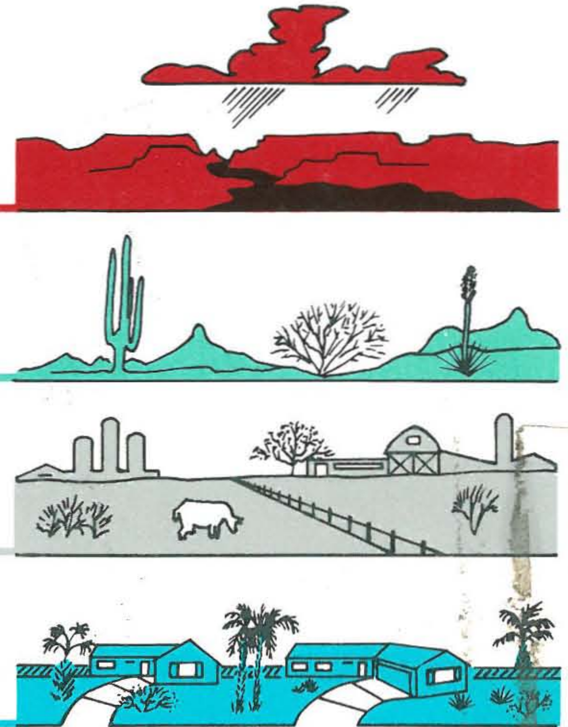

FLOOD CONTROL
DISTRICT OF
MARICOPA COUNTY

A HYDROLOGIC ANALYSIS OF THE POWERLINE F.R.S.

Volume I

OD CONTROL DISTRICT OF MARICOPA COUNTY

Report on

A HYDROLOGIC ANALYSIS OF THE POWERLINE F.R.S.

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Volume I

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A HYDROLOGIC ANALYSIS
OF THE
POWERLINE F.R.S.

Volume I

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I. INTRODUCTION

1.2 Purpose of Study

The purpose of this study was to utilize the 1985 Soil Conservation Service (SCS) Weekes Wash Study (reference 14) to evaluate the Powerline F.R.S. for State Dam Safety stage, storage, and discharge requirements; to update physical changes that have occurred on the watershed; and to suggest solutions if the Dam Safety criteria are not met. The results of this analysis would serve as a basis to reply to the SCS on hydrologic condition of Powerline F.R.S..

1.2 History

The Powerline Floodwater Retarding Structure (F.R.S.) is an earthen flood control "dam" designed and built by the U.S.D.A. Soil Conservation Service (SCS) in 1967. The Powerline F.R.S. located in Pinal County is operated and maintained by the Flood Control District (FCD) of Maricopa County (See Figure 1.1). The Powerline F.R.S. is approximately 2.5 miles in length, has a maximum height of 24 feet, and has a storage volume to the emergency spillway crest of 4200 acre-feet. Although this structure is entirely in Pinal County the protection provided is mainly to Maricopa County.

The Powerline F.R.S. was constructed in 1967 as part of the SCS Apache Junction - Gilbert Watershed workplan. Two sub-watersheds, Weekes Wash and Siphon Draw contribute inflow to the structure. The headwaters of Weekes Wash is in the Goldfield Mountains of the Tonto National Forest and flows south toward Powerline F.R.S.. The Weekes Wash watershed is 14.6 square miles in size at Powerline F.R.S. and is comprised primarily of undisturbed desert lands with some commercial and residential land use within the Apache Junction area. The headwaters of Siphon Draw is located in the Superstition Mountains of the Tonto National Forest. Siphon Draw flows in a southwesterly direction toward Powerline F.R.S.. The Siphon Draw watershed is 32.5 square miles in size at Powerline F.R.S., comprised mostly of undeveloped land (See Plate 1 in folder).

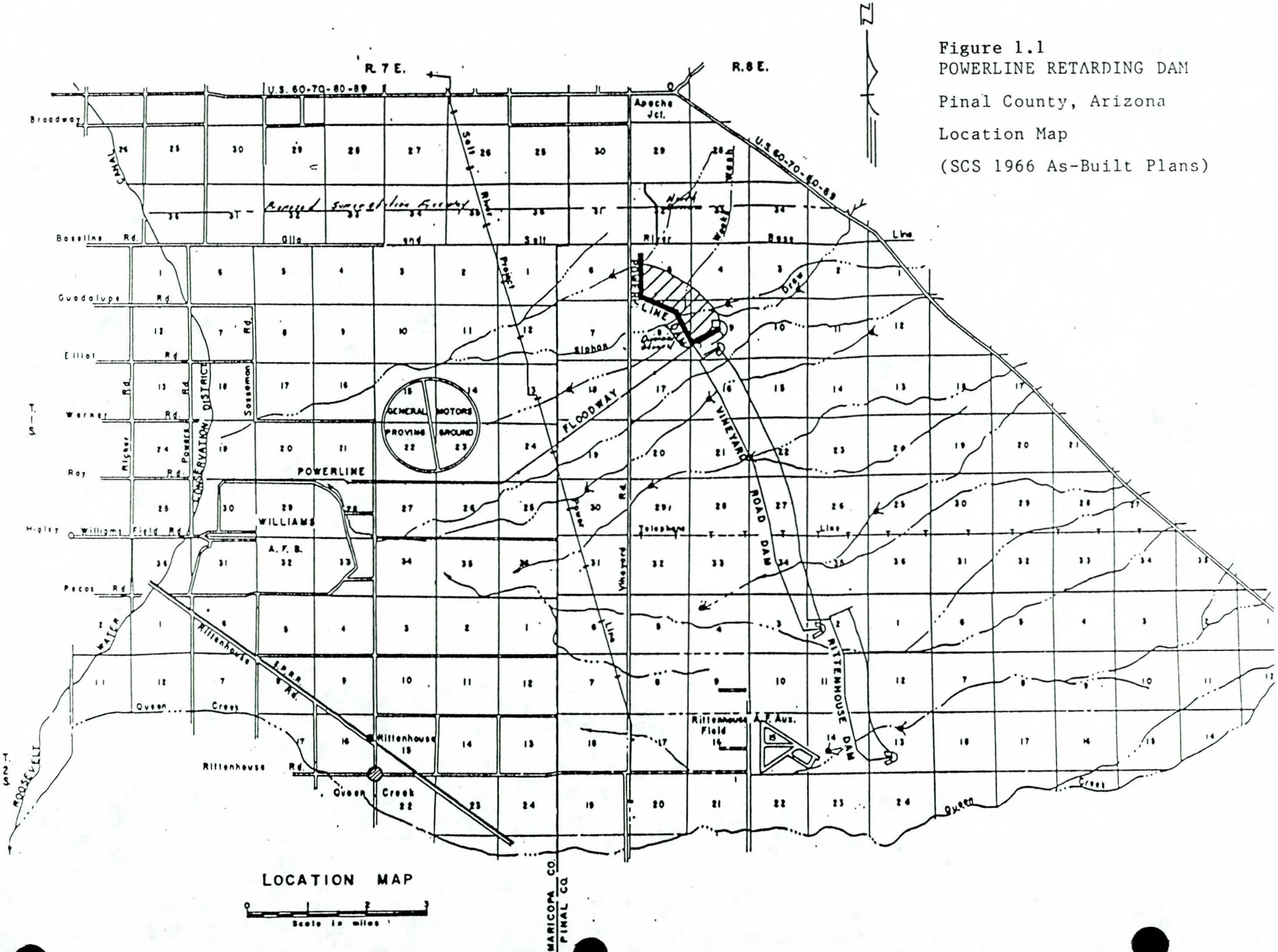


Figure 1.1
POWERLINE RETARDING DAM

Pinal County, Arizona

Location Map

(SCS 1966 As-Built Plans)

In 1985 the Soil Conservation Service (SCS) conducted a hydrologic analysis of the existing Powerline F.R.S. (reference 14). The SCS analysis included eight different storms and three separate alternatives. The SCS concluded that the Powerline Dam would not safely pass the design "Freeboard Storm" either "with" or "without" a proposed Dam on Weekes Wash.

The Flood Control District of Maricopa County (FCD) conducted an analysis to model physical characteristics on the watershed that were not included in the SCS study. Some of these characteristics include the Weekes Wash breakout (Section 4.3) and the Superstition Freeway (Section 4.4). The 1985 SCS study was used as a base for this report and parameters were encompassed into the FCD model. The Powerline F.R.S. falls under the jurisdiction of the Arizona Department of Water Resources (ADWR), Division of Safety of Dams. The structure must meet ADWR's criteria for dam safety, therefore, the SCS design "Freeboard Storm" was not simulated. Siphon Draw subbasin controls the characteristics of the inflow hydrograph to Powerline F.R.S. because it is much larger than the Weekes Wash subbasin, (32.5 square miles versus 14.6 respectively). The larger size and steeper average slope of the Siphon Draw subbasin combine to produce higher peaks than the Weekes Wash subbasin.

1.3 Model Conversion

The base model for this study was the SCS 1985 Weekes Wash Dam Study. It was converted from the SCS TR-20 computer program to the U.S. Army Corps of Engineers HEC-1 computer program. The HEC-1 is a more "user-friendly" program enabling future users to easily follow modeling logic and assumptions. Also, with the exception of how HEC-1 accounts for unit hydrograph computation step and variations in flood routing, the TR-20 and HEC-1 are quite compatible. The HEC-1 results were calibrated to the TR-20 results to ensure consistency. The HEC-1 output for the calibrated model is located in Volume II of this report. Table 1.1 shows a few of the common concentration points and the comparison between the two models.

TABLE 1.1

Calibration Comparison

Concentration Point	Discharge (cfs)		Volume (ac-ft)	
	HEC 1	TR-20	HEC-1	TR-20
Proposed Weekes Wash Dam	6412	6365	966	944
Weekes Wash at Powerline FRS	6408	7061	1453	1451
Siphon Draw at Powerline FRS	11,669	11,751	3246	3240
Inflow to Powerline FRS	17,913	18,279	4699	4690
Reservoir Outflow	602	588	568	467
Stage	1583.99	1583.97		

The SCS Weekes Wash Study assumed that Weekes Wash was contained, and all the flow reached Powerline F.R.S. Other studies (such as FEMA, A-N West) have identified a breakout of Weekes Wash. Following a field verification it was assumed for the purpose of this study that a breakout occurs at the intersection of Weekes Wash and Junction Drive. However, the HEC-1 base model for the conversion assumed the breakout to be consistent with the 1985 SCS Study. In further model modifications the breakout was assumed. This topic will be discussed in Section 3.0.

The following input were used in the converted HEC-1 base model, so as to match the SCS modeling input:

1. 100 year, 24 hour storm with an SCS Type II rainfall distribution.

2. Existing land use conditions.

3. SCS Curve Number Loss Rate to determine rainfall excess.

4. SCS Dimensionless Unit Hydrograph development.

5. The TR-20 uses the Modified Att-Kin procedure to route the flood wave downstream. This routing method is not an option in the HEC-1 model. The Kinematic Wave routing does not account for attenuation therefore Muskingum method of routing was used to route the flows within a channel.

6. The SCS boundary and delineations map for Powerline F.R.S. is used.

7. The dimensions of the Powerline F.R.S. were taken from as-built plans and were assumed to be accurate. The top of the structure is at an elevation of 1589.1. The principle spillway is at 1561.1, and the emergency spillway crest is at 1583.3. (See schematic plans for Emergency Spillway in Appendix F).

II. HYDROLOGIC METHODS

2.1 Introduction

The converted HEC-1 model served as a base model. However, as the study progressed and in-house meetings were held, it was determined that certain physical changes in the watershed condition needed to be incorporated into the base model. Minor changes to input parameters were not made, so as to maintain as close as possible a model based on the 1985 SCS study.

2.2 Physiography

The Powerline F.R.S. watershed has an areal extent of 47.1 square miles with elevation ranging from 1568 to 4869 feet above mean sea level. The soils, vegetation, and vegetative cover densities vary widely in the area. The soil association for the area is Trement-Ebon-Pinamt. The land is dominantly gently sloping to moderately steep with gravelly and very gravelly, loamy, and clayey soils. The area is generally found on fan terraces. See Table 2.1 for a listing of the major soils located in the area. The vegetation is predominantly desert brush and cactus, with cover densities ranging from 10 to 40 percent.

2.3 Methodology

The U.S. Army Corps of Engineers HEC-1 (Revised 31, Jan 85) computer program was used to simulate rainfall runoff response in the Powerline F.R.S. watershed. The model simulates the basin as an interconnected system of hydrologic and hydraulic components. The watershed was divided into 20 subbasins ranging in size from .37 to 11.85 square miles. See Plate 1 for the subbasin delineations. The HEC-1 results were calibrated to the 1985 SCS Results produced using TR-20 based on the 100 year, 24 hour storm.

Table 2.1

Soils found in the Powerline F.R.S. Watershed

Soil Series	Soil No.	Location Found	Ave. Slope	H.S. Group	Soil Class	Infil.Rate (in/hr)*	Comments
Antho-Carrizo-Maripo complex	3	floodplains & drainageways	0-3%	B	SL & VGL	.3 - .4	alluvium soils, high permeability, Sandy Bottom range site
Brios-Carrizo complex	10	floodplains	1-5%	A	LS & VGS	.4	alluvium soils, high permeability, Sandy Bottom range site
Carefree cobbly clay loam	12	fan terrace	1-8%	D	CCL	.1	slow permeability, Clay Upland
Eba-Pinaleno complex	41	fan terrace	20-40%	B/C	VGL & VGCL	.3	alluvium soils, Loamy Hills
Ebon very gravelly loam	44	fan & stream terrace	1-8%	C	VGL	.3	slow permeability, Clay Loam Upland
Gachado-Lomitas-Rock outcrop	52	rock outcrop	7-55%	D	VGL, VGCL, RO	.1	slow permeability, high runoff, Volcanic Hills range site
Gunsight-Cipriano complex	68	fan terrace	1-7%	B/D	VGL & VGSL	.2 - .4	moderate permeability, Limy Upland
Mohall c.l. calcareous solum	78	fan terrace	0-3%	B	Calcareous Loam	.2	moderate to slow perm., Limy Fan
Pinamt-Tremant complex	98	fan terrace	1-10%	B	EGSL & GL	.2 - .3	moderately slow perm., Loamy Upland
Torriorthents	111	fan & stream terrace	15-40%	-	Calcareous soil LS to Clay	.2	high permeability, highly erodable, Breaks range site
Tremant gravelly loams	113	fan terrace	0-3%	B	GL	.3	moderately slow perm., Loamy Upland
Tremant-Antho complex	115	fan & stream terrace and floodplains	1-5%	B	GSL with SL in floodplain	.3 - .4	moderately slow to rapid perm., Sandy Loam Upland range site

CCL : Cobbly clay loam
EGSL : Extremely gravelly sandy loam
GL : Gravelly loam
GSL : Gravelly sandy loam
LS : Loamy sand
RO : Rock outcrop

SL : Sandy loam
VGL : Very gravelly loam
VGS : Very gravelly sand
VGCL : Very gravelly clayey loam
VGSL : Very gravelly sandy loam

Reference 11 for table information except *.

* Reference 15.

2.4 Precipitation Parameters

The 100 year, 24 hour storm and the Probable Maximum Precipitation (PMP) were used in modeling. An assumption used in hydrologic interpretation of rainfall-runoff models is that the 100 year storm produces the 100 year flood and the PMP produces the Probable Maximum Flood (PMF). The Arizona State Department of Water Resources, Division of Safety of Dams (State Dam Safety) requires that the Inflow Design Flood (IDF) for a specific spillway be determine by the runoff hydrograph selected primarily on the basis of the size and hazard classifications assigned to the dam. The Guidelines for the Determination of Spillway Capacity Requirements suggests that the current IDF for the Powerline F.R.S. is the 1/2 PMF, while future land use conditions could raise the IDF to a PMF. By State Dam Safety definition, the 1/2 PMF is not a result of half of the Probable Maximum Precipitation (PMP), but rather is 50% of the hydrograph from the full PMF.

2.4.1 100 Year Precipitation

The total storm, basin-average precipitation for the 100 year, 24 hour duration was computed using the 1973 publication NOAA Atlas No. 2, Precipitation - Frequency Atlas of the Western United States, Vol. VIII - Arizona (Reference 7). The precipitation value for the 100 year, 24 hour storm is 3.85 inches (see Appendix A). Areal reduction for point rainfall was not used, for comparison to, and to be consistent with the SCS study.

However, an analysis was conducted to determine the effect that areal reduction would have on the watershed and Powerline F.R.S.. Two methods were analyzed. The first was the method as described in the 1973 National Weather Service NOAA Atlas 2. The second method used was described by Osborn, Lane and Myers in their article entitled Rainfall/Watershed Relationships for Southwestern Thunderstorms (Reference 8) from the 1980 Transactions of the ASAE. Even though the Osborn method was developed specifically for the Southwest, the depth-area curves are just gaining recognition. Table 2.2 shows the areal reductions for each method. The Osborn method has a significant reduction for this watershed, while the NOAA method is not as dramatic. Table 2.3 indicates the differences in water surface elevations at the Powerline F.R.S.. There is less than 0.2 feet difference between no areal reduction and the NOAA method of areal reduction but there is nearly 4 feet of difference between the Osborn method the NOAA method and no areal reduction.

TABLE 2.2

Comparison of Conversion Ratios
for the Different Areal Reduction Methods

AREA (sq. miles)	OSBORN METHOD		NOAA METHOD	
	Conversion Ratio	Rainfall (inches)	Conversion Ratio	Rainfall (inches)
.01	1.00	3.85	1.00	3.85
10	.879	3.38	.988	3.80
30	.764	2.94	.965	3.72
50	.692	2.66	.953	3.67

TABLE 2.3

Comparison of Effects of Areal Reductions
on Powerline F.R.S. (100 year-24 hour storm)

	No Areal Reduction	NOAA	Osborn
Maximum Flood Reservoir Water Surface elevation (ft. a.m.s.l.)	1584.25	1584.07	1580.54
Discharge (cfs)	1700	1200	150
Inflow Volume (ac-ft)	5628	5248	3193

The SCS Type II rainfall distribution was used to distribute the 3.85 inches of rain over the 24 hour period. This distribution is generally accepted for the Southwestern Region of the United States. The time increment used in the distribution was 15 minutes.

At this time, the acceptable areal reduction method is the NOAA method. Because the difference in maximum reservoir water surface elevation between the NOAA method of areal reduction and No Areal Reduction is only about two inches, and to be consistent with the 1985 SCS study, areal reduction was not used in the modeling.

The Osborn method was specifically developed for the Southwest region of the United States. The data for the depth-area curves were gathered from dense recording raingage networks in Arizona and New Mexico. In the future the Osborn method is expected to become more widely accepted and will most likely become the preferred method for areal reduction in the Southwest.

2.4.2 Probable Maximum Precipitation

The precipitation amount for the Probable Maximum Precipitation (PMP) was generated to determine the effects on the Powerline F.R.S., and in compliance with Arizona Department of Water Resources (ADWR) regulations. The PMP was calculated using the procedure outlined in the U.S. Department of Commerce's 1977 publication <u>Hydrometeorological Report No. 49 - Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages</u>. With this procedure an accumulated rainfall depth, incremental rainfall depth, and a rainfall distribution pattern was developed for 6-hour time increments over a 72-hour period. The 6-hour PMP was also analyzed and compared to the 72-hour PMP. The 6-hour PMP with 1-hour time increments produces the higher inflow peak which produces a higher stage for the Powerline F.R.S., therefore it is controlling and is utilized in this study (see Table 2.5). The total 6-hour PMP is 11.08 inches.

TABLE 2.4

Probable Maximum Precipitation Data

Time Increment (hours)	1	2	3	4	5	6
Accumulated Rainfall Depth (in.)	7.02	8.73	9.54	10.29	10.76	11.08
Incremental Rainfall Depth (in.)	7.02	1.71	.81	.75	.47	.32
Synthetic 6 hour Rainfall Distribution (inches)	.47	.81	7.02	1.71	.75	.32

Table 2.5

Comparison of the 72 hour PMP and the 6 hour PMP
on the Powerline F.R.S.

	Peak Inflow (cfs)	Volume (ac-ft)	Maximum Flood Reservoir Surface elevation (ft.m.a.s.l.)	Maximum Outflow (cfs)
6 hour	80,000	21,100	1590.47	76,200
72 hour	32,800	31,400	1589.53	32,700

2.5 Rainfall Excess

In this study the SCS Curve Number method was used to estimate rainfall excess. The Curve Numbers (CN) for the calibration model were taken directly from the SCS Weekes Wash Study. The <u>SCS Technical Release 55 - Urban Hydrology for Small Watersheds</u> (Reference 13) was used to determine the CN for future land use on areas that are currently agricultural or undeveloped desert. The land use plans for the Tonto National Forest do not include development therefore there was no change in CN for the forest. The average lot size was estimated to be between 1/4 acre to 1/2 acre with 25% to 38% impervious area, and to have a hydrologic soil groups of mostly B and C. The CN's were based on antecedent soil moisture condition II.

2.6 Unit Hydrograph Development

The unit hydrograph technique is used in the subbasin runoff component of the HEC-1 models to transform rainfall excess to subbasin outflow. An SCS dimensionless unit hydrograph was used in this study. This method uses CN, rainfall, and lag time, defined as the lag (hours) between the center of mass of rainfall excess and the peak of the hydrograph. By definition lag time is equal to .6 times the time of concentration (Tc). The time of concentration for the individual subbasins was taken from the SCS Weekes Wash Study. For those subbasins that were further divided, the time of concentration was calculated by estimating the average velocity of the runoff over the hydraulic length of the basin. The estimated velocities were checked for reasonableness and used within the model.

2.7 Channel Routing

Both the Kinematic Wave method and the Muskingum method were used to simulate the channel flow through each subbasin. The washes in the study area are natural with little or no improvements and therefore significant storage and attenuation of the flood hydrographs was expected. The Muskingum method was chosen for the channel routing because it provides for attenuation of the flood wave and produces outflows that were similar to those of the SCS Weekes Wash Study.

The Muskingum 'X' coefficient (weighting factor) was set equal to 0.3. This coefficient can range in value from 0.0 to 0.5. Using an X of 0.5 results in translation of the routed hydrograph with minimal attenuation while an X of 0.0 would be used for a reservoir.

The Muskingum 'K' coefficient (storage constant) is the ratio of storage to discharge and has the dimension of time. The HEC-1 model assumes the K coefficient to be the travel time through the channel reach. The travel time was computed by estimating a velocity for a channel over the length of the channel reach. The travel times from the FCD study were checked with the travel times from the 1985 SCS study and were found to be within a reasonable percentage of difference.

2.8 Storage Routing

Reservoir routing was used in the HEC-1 model to simulate the two detention basins on the north side of the proposed Superstition Freeway, and at the series of culverts along the proposed Superstition Freeway east of Tomahawk Road. The Superstition Freeway was not modeled in the SCS study.

III. WEEKES WASH BREAKOUT AT JUNCTION DRIVE

Weekes Wash has a wide floodplain with braided, and moderately incised channels interrupted by many at grade road crossings. Typically, these road crossings provide dips which are insufficient to contain the flows. The potential for flows to breakout is great at many different points. Cella Barr Associates in a Flood Insurance Study for Weekes Wash, and A-N West in their hydrology study for the Superstition Freeway indicated a breakout in the vicinity between Superstition Road and Junction Road. The SCS Weekes Wash Study assumed that all the flow from Weekes Wash reached the Powerline F.R.S.. Lacking technical data to determine the extent of the breakout, the FCD took measures to quantify the breakout. Five cross- sections were surveyed along Weekes Wash by Z & H Engineering, Inc. of Phoenix, Arizona. The cross-sections were coded into the U.S. Army Corps of Engineers' HEC-2 Water Surface Profiles model to determine where the runoff would break out. By using the multi-profile option in the HEC-2 a rating curve was developed for the breakout. The breakout occurs at the point where Weekes Wash crosses Junction Drive. Junction Drive slopes toward the west and does not dip to allow the runoff to cross the road, but instead flow parallel to the road to the west. The rating curve for the breakout is shown in on the following table.

TABLE 3.1

Weekes Wash Breakout Rating Curve

Inflow (cfs)	Breakout (cfs)	
	Existing	Proposed*
1000	228	0
3000	1024	0
5000	2182	0
6000	2767	0
7000	3668	260
9000	4716	1300
10000	5382	1880

* The 100 year 24 hour flows are assumed to be channelized and contained within the Powerline F.R.S. watershed.

The "existing" rating curve was incorporated into the HEC-1 Powerline F.R.S. Study. The flow from the 100 year storm above the breakout is about 6000 cfs. The diversion into the East Maricopa County Area Drainage Master Study area from the 100 year storm computed to be approximately 2800 cfs while 3200 cfs stayed in the Weekes Wash system to the Powerline F.R.S..

It was assumed that culverts would eventually be placed under Junction Drive and the 100 year flows would not breakout into East Maricopa County. For all further runs the breakout of Weekes Wash at Junction Drive was assumed to be maintained and to reach the Powerline F.R.S..

IV. MODELING ASSUMPTIONS

4.1 Magnitude of Flood for Dam Safety Analysis

Using the Arizona Department of Water Resources, Dam Safety Guidelines, the Powerline F.R.S. currently has a "Hazard Potential Classification" of "Significant", e.g. limited urban development existing downstream from the structure. The "Size Classification" of Powerline F.R.S. is considered a medium dam. With these conditions the inflow design flood magnitude is 1/2 the PMF.

The area downstream of the Powerline F.R.S. is predicted to be a rapidly growing area within the next ten years. Therefore, the "Hazard Potential Classification" would increase to "High". The design flood magnitude would, therefore increase to PMF for these conditions. Because there are many factors to consider in the selection of the magnitude of the flood the "Guidelines for the Determination of Spillway Capacity Requirements" only provides ranges of flood magnitude. The Department of Water Resources would review all the factors and require a specific flood magnitude. Because the HEC-1 model allows for the easy determination of both the 1/2 PMF and the full PMF simultaneously, both flood magnitudes were analyzed.

4.2 Land Use Conditions:

The HEC-1 base model assumed existing land use conditions. However, as a result of an FCD In-House Progress Review (I.P.R.) is was decided that future land use conditions should be used to model the watershed. Private lands were assumed to be developed to 1/4 acre lots and 1/2 acre lots, which represents current residential and commercial development trends in the Apache Junction area. The Tonto National Forest land was assumed to remain undeveloped. Retention of onsite runoff was not assumed due to the physical difficulty of incorporating retention in mountainous watersheds, and the lack of a retention regulation in Pinal County. Table 4.1 show the comparison of Curve Numbers used in the 1985 SCS Weekes Wash Study, the FCD Study, and the A-N West Study for the Superstition Freeway.

4.3 Weekes Wash Breakout:

As discussed in Section III, the Weekes Wash breakout was assumed to be contained in the future land use conditions within the Weekes Wash watershed because of the likelihood of eventually channelizing the flow under Junction Drive. Flows over that magnitude would split as described in Section III.

TABLE 4.1

Curve Number Comparison
for the Powerline F.R.S. Analysis

Watershed	FCD [1] (future)	SCS [2] (existing)	A-N West [3] (future) Freeway)
10	79	--	--
17	79	79	87
16	82	82	88.5
15	80	80	87
14	79	79	86.5
13	83	83	90
12	86	86	92
11	89	82	91
2A	82	--	82
2BE	82	--	84
2BWW	82	--	83
2BWE	82	--	83
2BS	80	--	--
5	85	81	--
4	86	81	--
7	86	--	--
6	83	79	--

[1]The FCD analysis assumed future land use conditions.

[2]The 1986 SCS Weekes Wash Study assumed present land use conditions.

[3]The AN West Study for the Superstition Freeway (Upper Weekes Wash watershed).

4.4 Superstition Freeway:

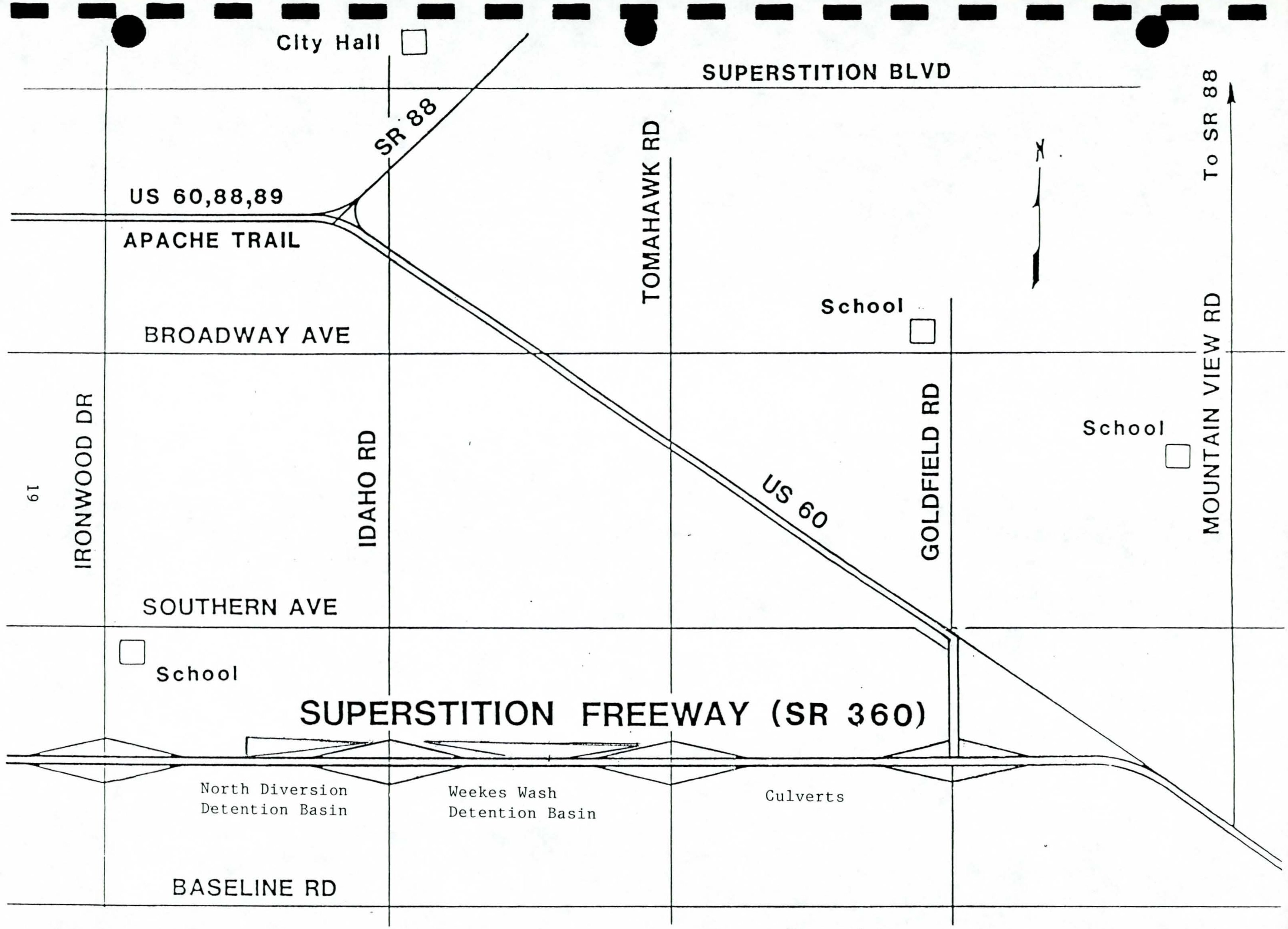
The proposed Superstition Freeway crosses the Powerline F.R.S. watershed between Southern Avenue and Baseline Road, and is approximately 2.25 miles north of the Powerline F.R.S. outlet (See Figure 4.1). There were three sources that were used to determine the flow under the Freeway: Superstition Freeway Comprehensive Offsite Drainage Plan, (Reference 1) dated February 1987, prepared by A-N West, Inc.; Final Drainage Study Report for Superstition Freeway (SR. 360) - Ironwood Drive to US 60, (Reference 2) dated November 1987, prepared for the Arizona Department of Transportation, by Tudor Engineering Company; and the final review plans for the Superstition Freeway, Plan and Profile of Proposed State Highway - Superstition Freeway, Maricopa - Pinal County, Power Road to US 60, Phase II, (Reference 3) dated July 1988, prepared by the Arizona Department of Transportation.

The section of Freeway from Ironwood Drive to Tomahawk has two detention ponds. Rating curves for each facility were found in both the Arizona Department of Transportation (ADOT) Final Study Report and The Offsite Drainage Plan by A-N West. Both sources had discrepancies in values for volume and discharge. The FCD slightly modified these curves to reflect the Plan and Profile sheets of ADOT.

4.4.1 ADOT Detention Basin at the North Diversion Structure

The purpose of the North Diversion Structure is to direct flows to the Powerline F.R.S. that would naturally flow to the north of the Powerline F.R.S.. The ADOT detention basin at the North Diversion Structure will be constructed parallel to the Freeway and will extend east from the North Diversion Structure toward the Idaho Road underpass (see Figure 4.1). The basin was designed to intercept all overland flows entering the Freeway right-of-way from the north and northeast between the stations. The basin has a 48 inch outlet pipe which discharges into an existing collector channel on the upstream side of the North Diversion Structure. The runoff then flows through two 10 x 6 foot box culverts.

The rating curves used in the A-N West design study indicates the invert of the detention outlet pipe to be 1628. ADOT's final plans show the invert of the pipe to be 1623.5. A-N West design study also indicates the box culverts to be three 6 x 8 foot box culverts. The rating curves for both the outlet pipe and the box culverts were recalculated.



TUDOR ENGINEERING COMPANY

AREA MAP

Figure 4.1
Superstition Freeway
Location Map

The top of the North Diversion Structure is at 1630. See Appendix E-II for a schematic of the North Diversion Structure at the Superstition Freeway. There appeared to be a possibility of overtopping of the North Diversion Structure in which the runoff would breakout from the watershed. A rating curve was constructed to estimate the flow that would breakout over the North Diversion. A rating curve for the two 10 x 6 foot box culverts was entered into the model and the flow over the maximum capacity of the culverts were diverted from the Powerline Dam. The 100 year flood was contained behind the North Diversion Dam and passed through the ADOT box culverts. Diverted flows occurred during the 1/2 PMF and the full PMF. The estimated diverted flows are 1050 cfs for the 1/2 PMF and 3360 cfs for the full PMF.

The road profile from ADOT's Final Plan and Profile for the Superstition Freeway was analyzed to determine the flow pattern of runoff if it overtopped the road. The road profile of the Superstition Freeway has a slope of 1.67% toward the east of the two box culverts and .86% toward the west of the box culverts. The slope of the road profile is greater than that of the runoff flows south, therefore most of the flow will follow the Freeway west 1000 feet to the low point in the profile. Since the North Diversion Structure is the west boundary for the watershed, any runoff that is diverted at this point will not enter the Powerline Dam, but might impact the Powerline Floodway.

4.4.2 ADOT Detention Basin at Weekes Wash

The ADOT detention basin for the Superstition Freeway at Weekes Wash is designed to intercept overland flows associated with Weekes Wash. See Appendix E-III for a schematic of the Weekes Wash detention basin at the Superstition Freeway. The plan shows the basin to be constructed parallel to the Freeway and will extend east from Idaho Road to just west of Tomahawk Road underpass. The surface area of the basin is approximately 24 acres and has a maximum storage volume of about 220 acre-feet. The outlet for the basin consists of two 12 x 12 concrete box culverts. The outlet channel for the basin will follow the Royal Palm Road alignment from the Freeway to Baseline Road, where the channel will connect into the existing Weekes Wash channel. Approximately 3,150 feet of the outlet channel from the basin will be lined with reinforced concrete and have a tapezoidal cross section. The bottom width of the channel will be 35 feet and have sideslopes with the ratio of one and one-half horizontal to one vertical.

The rating curves for the Weekes Wash detention basin and the outlet culverts were reviewed. The stage-storage relationship calculated in-house was not consistent with the curve prepared by A-N West. The average-end area method was used in-house to calculate the volume of the detention basin. The volume calculated by the FCD is approximately 1/2 of the A-N West values (see Appendix E-III for calculations). Independently the volumes were checked and verified by other FCD staff. The A-N West report does not specifically explain the methodology used to calculate the volume. It was assumed the A-N West study was done prior to the final design, and that the FCD lacked the final design curves. The road profile from ADOT's Final Plan and Profile for the Superstition Freeway was analyzed to determine the flow pattern of runoff if it overtopped the road. The profile is similar to that discussed for the North Diversion Structure detention basin. Most of the flow that overtops the road would travel 1000 feet west of the two 12 x 12 foot box culverts along the Freeway to a low point in the road. The runoff then flows south across the road entering a tributary of Weekes Wash which joins the main wash after the transition from a concrete lined channel to the natural wash. This system was modeled to more accurately simulate the actual flow pattern.

4.4.3 Culverts

Drainage from Tomahawk Road to U.S. 60 along the freeway will be passed through a series of culverts. Surface runoff entering the Freeway right-of-way east of Tomahawk Road is fairly well contained in a network of existing washes. The washes naturally concentrate runoff at locations where culverts can pass peak discharges through the Freeway embankment without impacting the upstream floodplain. A-N West developed a rating curve for the combined culverts. This rating curve was checked for reasonableness and used in the FCD study.

4.5 Powerline F.R.S.

4.5.1 Spillway Rating Curve

The discharge estimates for the principle and emergency spillways were taken from the SCS November 1967 notebook entitled: Powerline Dam, Hydrology and Hydraulics and Structure Design (Reference 12). The SCS 1985 Weekes Wash Study used this rating curve and adjusted the elevations as a result of the dam site being moved upstream 200 feet to accommodate the right-of-way for the Central Arizona Project Aqueduct. The parameters concerning the structure were nearly the same but only translated 1.1 foot higher in elevation. The documentation for the method that SCS used to determine the Emergency Spillway Rating Curve is not presented in the notebook and it is not clearly shown how the values were determined. The spillway crest is located within the emergency outlet channel. A simple weir calculation for estimating the flow through the emergency spillway would not accurately reflect the flow, since backwater in the outlet channel could "submerge" the weir. Therefore, the Army Corps of Engineer's HEC-2 Water Surface Profile computer model was used to develop the rating curve. See Volume II for the HEC-2 map, input, and output. The rating curve developed by the FCD indicates that the spillway has more capacity than what was represented in the SCS Weekes Wash Study. Table 4.2 shows the comparison of the discharges for each study. A cross check with a weir calculation was also made to determine reasonableness of the results.

The HEC-2 program documents a situation that is not apparent in the SCS study. The emergency spillway outlet channel has banks 4 feet in height, designed to prevent flows from flowing along the toe of the F.R.S. (see Appendix F for a schematic of the emergency outlet channel). At about 15,000 cfs the outflow begins to overtop these channel banks. The HEC-2 program indicates the height that the bank would need to be extended to contain the total outflow for emergency spills. For an outflow of 29,000 cfs over the emergency spillway the banks would need to be raised a minimum of 2 feet at the crest and 1 foot downstream. The outflow in the FCD study with future land use conditions for the 1/2 PMF is 20,100 cfs and the PMF is 76,200 cfs (including flows over the dam). It is estimated that the maximum flow through the emergency spillway without overtopping the dam is 22,000 cfs.

If flows were to overtop the banks of the emergency spillway then a hazardous situation would occur. Some of the overtopped flows would be trapped in the area between the toe of the dam and the banks of the emergency spillway. This could lead to possible erosion to the structures as water would then flow between the outlet-bank and the toe.

TABLE 4.2

Comparison of Discharge between
the 1985 SCS study and the FCD Study

		SCS Study	FCD Study	Weir** Values
Stage (msl)	Storage (ac-ft)	Outflow (cfs)	Outflow (cfs)	Outflow
1568.1	0	0	0	
1568.2	175	75	75	
1570.1	380	92	92	
1572.1	700	106	106	
1574.1	1100	119	119	
1576.1	1600	130	130	
1578.1	2175	141	141	
1580.1	2875	150	150	
1582.1	3675	159	159	
* 1583.3	4200	165	165	
1584.1	4600	668	1228	1060
1586.1	5525	4426	7360	7195
1588.1	6725	11,084	16,800	15,770
1590.1	7925	20,092	27,280	26,600

* Spillway Crest Elevation.

** $Q = Cd\ L\ h^{3/2}$ + 165(cfs) (165 is the flow through the principle spillway)

Cd = 2.5
L = 600 feet

V. RESULTS

5.1 Superstition Freeway Impact

The Superstition Freeway was modeled to determine the effect it would have on the Powerline F.R.S. and because it will be a significant local structure. The Superstition Freeway did not have any significant effects on Powerline F.R.S.. The breakout during the 1/2 PMF and the full PMF at the North Diversion Structure is not significant because Siphon Draw still dominates the inflow hydrograph to the Powerline F.R.S..

5.2 100 Year Flood

The 100 year 24 hour storm produces a peak outflow from the structure of about 1700 cfs, and a water surface elevation of 1584.25 at 20.67 hours. The emergency spillway is at 1583.3, therefore, there is almost 1 foot of runoff flowing through the emergency spillway. Since the difference between the top of the structure and the water surface elevation for the 100 year storm is greater than 3 feet, the structure is within ADWR dam safety guidelines for this event. Although there is enough freeboard, the emergency spillway is in operation. Technically, if the area downstream is to be protected from the 100 year flood, then there should be no outflow through the emergency spillway, which is not the case.

The Weekes Wash Breakout is assumed to be contained with no flows leaving the watershed.

The Siphon Draw watershed contributes a peak discharge of about 13,000 cfs, 13.67 hours after the start of the rainfall, while the Weekes Wash watershed contributes a peak discharge of 4800 cfs, 14.83 hours after the start of the rainfall. The combined hydrographs into the Powerline F.R.S. produces a peak discharge of about 14,300 cfs at 13.67 hours.

Although the FCD study did not use an areal reduction for the final computer model, an analysis was done to determine the effects on the Powerline F.R.S.. The NOAA method had a volume of 5250 ac-ft of runoff entering the storage area behind the Powerline F.R.S., a maximum reservoir water surface elevation of 1584.07, and a maximum outflow of 1200 cfs. The Osborn method had a volume of 3200 ac-ft of runoff entering the storage area behind the Powerline F.R.S., a maximum reservoir water surface elevation of 1580.54, and a maximum outflow of 152 cfs which does not spill into the emergency spillway channel.

5.3 Probable Maximum Flood

During the 1/2 Probable Maximum Flood, approximately 5000 cfs will breakout at the intersection of Junction Drive and Weekes Wash while 930 cfs will breakout from the intersection of the Superstition Freeway and the North Diversion Structure detention basin. The total volume of runoff being diverted from the watershed is 450 acre feet. The Siphon Draw watershed contributes a peak discharge of about 33,170 cfs, 4.33 hours after the start of the rainfall, while the Weekes Wash watershed contributes a peak discharge of 11,000 cfs, 5.33 hours after the start of the rainfall. The combined hydrographs into the Powerline F.R.S. produces a peak inflow of about 40,300 cfs at 4.67 hours. The 1/2 PMF produces an outflow from the structure of about 20,060 cfs, and a water surface elevation of 1588.72 at 6.33 hours. Although the water surface elevation for this storm does not overtop the structure there is less than five inches of freeboard.

During the Probable Maximum Flood approximately 13,500 cfs will breakout at the intersection of Junction Drive and Weekes Wash while 3100 cfs will breakout from the intersection of the Superstition Freeway and the North Diversion Structure detention basin. The total volume of runoff being diverted from the Powerline F.R.S. is 1690 acre-feet. The Siphon Draw watershed contributes a peak discharge of about 66,340 cfs, 4.33 hours after the start of the rainfall, while the Weekes Wash watershed contributes a peak discharge of 19,070 cfs, 5.17 hours after the start of the rainfall. The combined hydrographs into the Powerline F.R.S. produces a peak discharge of about 80,000 cfs at 4.50 hours. The PMF storm produces an outflow from the structure of about 76,200 cfs, and a water surface elevation of 1590.47 at 5.00 hours. The PMF will overtop the structure by 1.4 feet for a duration of just over 2.83 hours.

VI. ALTERNATIVES

The results from the FCD study indicates that the Powerline F.R.S. emergency spillway will spill during the 100 year, 24 hour storm, and that the PMF will overtop the structure for future land use conditions. Since these are the two criteria for which the Powerline F.R.S. will be based as safe or not, then alternatives must be considered to meet the criteria.

Because of the spillway type and associated discharge relationships, it was felt that modifying the emergency spillway was not a valid option. Therefore, the other alternatives had to deal with increasing the volume behind the structure.

Alternative G is the analysis with no changes from the final assumed conditions. All alternatives will be compared to Alternative G for the "Total Volume Leaving the Powerline F.R.S. Watershed (including Weekes Wash Breakout and Breakout at North Diversion Dam)"; "Peak Flood Reservoir Water Surface Elevation"; "Inflow to Powerline F.R.S. (cfs)"; and "Outflow from Powerline F.R.S. (cfs)". See Table 6.1 for a comparison of all the alternatives.

6.1 Alternative A : Excavate More Volume

The first alternative simulated an increase in the available storage behind the structure. It was assumed that the bottom of the storage area would be graded flat (realistically there would be a slight slope toward the principle spillway) at an elevation of 1570. This would result in the maximum volume of excavation behind the structure. The grading would begin at the spillway crest elevation of 1583.3 and go toward the structure at a 10:1 sideslope until the elevation of 1570 was reached. The stage-storage-discharge rating curve was recalculated and entered into the models.

During the 100 year storm the water surface elevation dropped from 1584.35 to 1582.52 (no flow over the emergency spillway), and the maximum outflow dropped from 1703 cfs to 161 cfs. When the new rating curve was entered into the PMF simulation the water surface elevation dropped from 1588.72 to 1587.65 for the 1/2 PMF, and remained the same at 1590.4 for the full PMF. There would be a little more freeboard during the 1/2 PMF but the full PMF would still overtop the structure by 1.4 feet.

Table 6.1

Comparison of Alternatives

	100 year, 24 hour				1/2 PMF				PMF			
Alternative	1	2	3	4	1	2	3	4	1	2	3	4
A	-	1582.52	14,300	161	454	1587.65	40,300	15,600	1687	1590.40	80,000	72,700
* B	-	1584.25	14,300	1703	454	1588.72	40,300	20,100	1687	1590.49	80,000	36,500
* C	-	1582.52	14,300	161	454	1587.65	40,300	15,600	1687	1590.38	80,000	34,900
D	-	1583.79	14,050	812	57	1588.15	36,400	17,100	1068	1590.32	74,800	68,300
E	1036	1583.66	14,000	641	1606	1588.18	36,800	17,200	3017	1590.41	77,500	73,200
F	1036	1583.66	14,000	641	2225	1587.75	36,200	15,100	4591	1590.28	74,200	65,800
G	-	1584.25	14,300	1703	454	1588.72	40,300	20,100	1687	1590.47	80,000	76,200

A - Excavation
B - Raise structure
C - Combine A and B
D - Weekes Wash Dam
E - Diversion of 100 year, 24 hour flows at Rt. 80 to Apache F.R.S.
F - Diversion of all flows at RT. 80 to Apache F.R.S.
G - Do Nothing

Top of Spillway : 1583.3
Top of Dam : 1589.1
* Top of Dam : 1590.6

1 - Total Volume leaving the Powerline F.R.S. Watershed (includes Weekes Wash Diversion at Rt. 80 and Breakout at North Diversion Dam)
2 - Peak Flood Reservoir Water Surface Elevation
3 - Inflow to Powerline F.R.S. (cfs)
4 - Outflow from Powerline F.R.S. (cfs)

6.2 Alternative B : Raise Powerline F.R.S.

The second alternative creates more volume by raising the height of the structure. In this simulation the Powerline F.R.S. was raised 1.5 feet to an elevation of 1590.60, which would contain all the runoff behind the structure.

The water surface elevation for the PMF would be 1590.49 with outflow at about 36,500 cfs, and a storage volume of 12,000 acre-feet. With the structure raised by 1.5 feet, the PMF would not overtop the structure, but there would be about .11 feet of freeboard.

The existing emergency spillway only has 4 foot high banks with a capacity of about 15,000 cfs, as previously discussed. If the structure was raised, then the spillway banks would also need to be raised to allow for the spillway to work effectively. In the model, the emergency spillway banks were raised by approximately 3 feet at the spillway crest and about 2 feet downstream.

6.3 Alternative C : Combination of Alternatives A and B

Alternative A will only solve the problem of emergency spills during the 100 year flood, while Alternative B will only solve the problem of flows overtopping the structure during the Probable Maximum Flood. Therefore, a combination of Alternative A, excavation of an additional 1540 ac-ft; and Alternative B, raising the structure by 1.5 feet to an elevation of 1590.6 was modeled. The combination of the two alternatives will allow the Powerline F.R.S. to meet all the Dam Safety criteria for future conditions.

The peak flood reservoir water surface elevation for the PMF at the Powerline F.R.S. would drop about an inch to 1590.38. The peak flood reservoir water surface elevation for the 100 year, 24 hour storm would be 1582.52. Therefore there would not be spills through the emergency spillway channel.

6.4 Alternative D : Proposed Weekes Wash Dam in Place

The stage-storage-discharge curve was taken directly from the 1985 SCS Weekes Wash Study. The data was developed by Rex Stone and S.M. Krahenbuhl for the SCS during the watershed work plan development. The SCS checked the data with the December 1983 Design Hydrology Analysis and found the data to be satisfactory and was therefore used in their analysis.

The location of the proposed Weekes Wash Dam is just north of Apache Trail.

The proposed Weekes Wash Dam did not make a significant difference in the peak flood reservoir water surface elevation at Powerline F.R.S. for any of the storms. The water surface elevation for the 100 year, 24 hour storm was reduced by 0.45 feet which still indicates spills through the emergency spillway. The 1/2 PMF and the PMF have a reduction of water surface elevations at Powerline F.R.S. of about 0.6 feet and 0.17 inches, respectively. The 1/2 PMF has an increase of freeboard to 0.95 inches, while the PMF will still overtop the Powerline F.R.S. by 1.2 feet for a duration of 3 hours.

6.5 Alternative E : Diversion of 100 year, 24 hour flows at Apache Trail (Rt. 80) to Apache F.R.S.

One of the alternative analyzed by SCS was the diversion of flows at the proposed Weekes Wash Dam site to Apache F.R.S.. Alternative E assumes that the diversion structure would be designed to carry only the 100 year, 24 hour storm.

During the 100 year, 24 hour storm there would be 1036 ac-ft of runoff leaving the Powerline F.R.S. watershed. The peak flood reservoir water surface elevation at Powerline F.R.S. would be 1583.66. There would still be spills through the emergency spillway.

During the PMF and the 1/2 PMF, 3017 ac-ft and 1606 ac-ft, respectively, of runoff would leave the Powerline F.R.S. watershed. The peak flood reservoir water surface elevation at Powerline F.R.S. would be 1590.41 and 1588.18 for the PMP and 1/2 PMF, respectively. Flows would still overtop the structure during the PMF.

6.6 Alternative F : Diversion of All Flows at Apache Trail (Rt. 80) to Apache F.R.S.

The total diversion of flows from the Upper Weekes Wash subbasin at Rt. 80 to Apache F.R.S. would result in the peak flood reservoir water surface elevation at Powerline F.R.S. for the PMF of 1590.28. Overtopping of the structure would still occur.

6.7 Alternative G : Do Nothing

This alternative suggests that no structural modification would be done. This alternative could be considered feasible if the dam safety criteria were based on the current land use conditions downstream. The required design flood magnitude for this alternative would be the 1/2 PMF. The model shows that the 1/2 PMF would not overtop the structure, then the Powerline F.R.S. would be within the criterion for the design flood magnitude. There would still be the question of the 100 year spills through the emergency spillway. In the future this structure would likely not meet ADWR Dam Safety criteria.

VII. CONCLUSIONS

This study was initiated in response to the 1985 SCS Weekes Wash Study. The SCS has different criteria for design storms than the Arizona Department of Water Resources (ADWR). Because the governing agency in Arizona is ADWR, Department of Dam Safety, their dam safety criteria are the controlling criteria. If this study determines that during the design storms Dam Safety Criteria is not met, then alternatives will be suggested to allow the Powerline F.R.S. to meet the criteria.

This study shows that for future land use conditions upstream of Powerline F.R.S. there will be emergency spills during the 100 year 24 hour storm, the PMF will overtop the structure, and the 1/2 PMF will only have 4.6 inches of freeboard from the Peak flood reservoir water surface elevation and the top of the dam.

Powerline F.R.S. was designed to protect the downstream areas from the 100 year, 24 hour storm. Therefore, there should be no spills through the emergency spillway during this frequency storm. Alternative A and C are the only two alternatives that allow Powerline F.R.S. to meet this criterion.

The recommended spillway design flood for current downstream land use, using future land use conditions upstream is the 1/2 PMF. All the alternatives meet this criterion.

According to downstream land use trends, the area will develop rapidly within the next ten years. This conditions would change the hazard category according to Dam Safety Criteria to "High", which in turn would raise the design flood magnitude to 1/2 PMF - PMF. Since this criteria is only a recommendation by the Department of Dam Safety, they would look at other factors to specify the exact design flood magnitude. Alternatives B and C are the only alternatives that allow Powerline F.R.S. to meet this criterion.

We would recommend using the PMF for the design flood magnitude. This would allow for the maximum protection of future downstream development. The Flood Control District currently has the financial ability to bring Powerline F.R.S. to future standards. It is not known if in five to ten years from now we will still be in that financial situation.

Alternative C is the preferred alternative for the Flood Control District of Maricopa County. Alternative C is the only alternative that meets all the Dam Safety Criteria for future land use upstream and downstream of Powerline F.R.S.. The excavation of storage behind Powerline F.R.S. was an estimate. The excavation assumed, showed the peak flood reservoir water surface elevation to be 1582.52, over 9 inches below the emergency spillway crest. Further refinement of the excavation will eventually need to be made.

A meeting of the Flood Control District of Maricopa County, the Soil Conservation Service, the Arizona Department of Water Resources - Dam Safety Division, an other pertinent entities should be held to discuss possible alternatives for Powerline F.R.S.. Since ADWR will be the governing agency over the structure, their input will be important in the final decision for establishing the criteria (future or present) and choosing the alternative which meets all the criteria.

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APPENDIX A

PRECIPITATION DATA

APPENDIX A-I

100 YEAR STORM

ADDENDUM to "HYDROLOGIC DESIGN FOR HIGHWAY DRAINAGE IN ARIZONA" April 1975

Steps to be used to determine precipitation values for various durations and return periods.

STEP 1. From the precipitation maps in the manual "Hydrologic Design for Highway Drainage in Arizona", determine the precipitation values for the 6 and 24 hour duration storms for return periods of 2, 5, 10, 25, 50 and 100 years. Tabulate these values in Table 1 in the column headed 'Map Values'

Table A-1

100 Year Precipitation for

Powerline F.R.S. Apache Junction - Gilbert Watershed

(Reference 4)

Return Period (Years)	Precipitation Values (inches)			
	6 hour duration		24 hour duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2			1.6	1.6
5			2.2	2.15
10			2.6	2.5
25			3.0	3.0
50			3.5	3.4
100			3.8	3.85

NOTE: There is a possibility of making an error while reading the maps because, (1) a site is not easy to locate precisely on a series of 12 maps, (2) there may be some slight registration differences in printing, and (3) precise interpolation between isolines is difficult. In order to minimize any errors in reading the maps, these values should be plotted on the diagram "Precipitation Depth versus Return Period" Fig. 1.

Project No. Powerline F.R.S.   Station ____________

Apache Junction - Gilbert Watershed

(Reference 4)

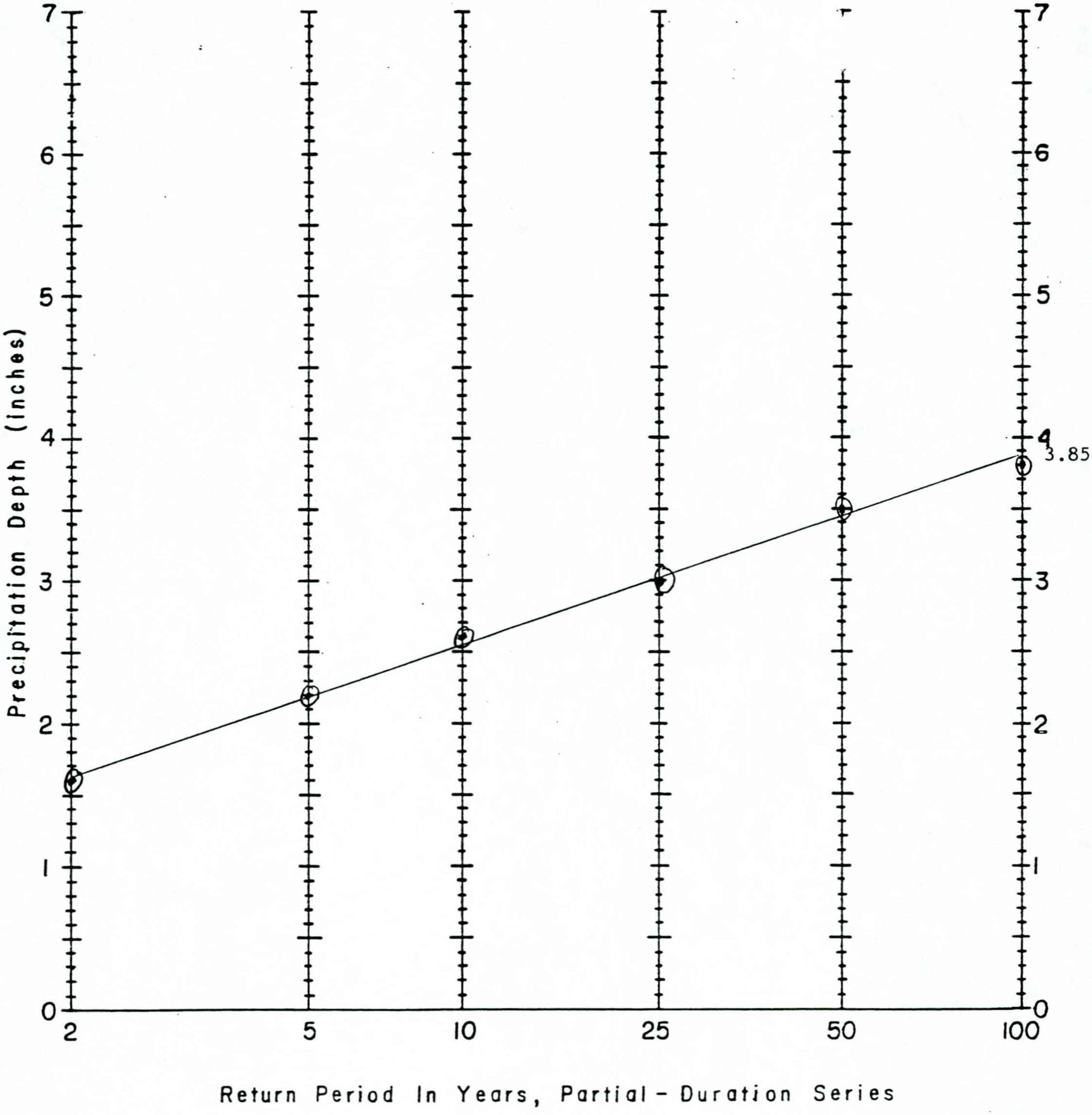


Figure A-1 Precipitation Depth Versus Return Period for Partial – Duration Series for the Apache Junction - Gilbert Watershed

Table A-2

SCS Type II Distribution

PC	0	.002	.005	.008	.011	.014	.017	.020	.023	.026
PC	.029	.032	.035	.038	.041	.044	.048	.052	.056	.060
PC	.064	.068	.072	.076	.080	.085	.090	.095	.100	.105
PC	.110	.115	.120	.126	.133	.140	.147	.155	.163	.172
PC	.181	.191	.203	.218	.236	.257	.283	.387	.663	.707
PC	.735	.758	.766	.791	.804	.815	.825	.834	.842	.849
PC	.856	.863	.869	.875	.881	.887	.893	.898	.903	.908
PC	.913	.918	.922	.926	.930	.934	.938	.942	.946	.950
PC	.953	.956	.959	.962	.965	.968	.971	.974	.977	.980
PC	.983	.986	.992	.995	.998	1.000	1.000	1.000	1.000	1.000

APPENDIX A-II

PROBABLE MAXIMUM PRECIPITATION

Table A-3 --General-storm PMP computations for the Powerline F.R.S. Watershed (Reference 16)

Drainage Powerline F.R.S. Area 46.3 mi^2 (km^2)

Latitude 33°20', Longitude 111° of basin center

Month Sept.

Step	6	12	18	24	48	72	
	Duration (hrs)						
A. Convergence PMP							
1. Drainage average value from one of figures 2.5 to 2.16 13.8 in. (mm)							
2. Reduction for barrier-elevation [fig. 2.18] 86 %							
3. Barrier-elevation reduced PMP [step 1 X step 2] 11.87 in. (mm)							
4. Durational variation [figs. 2.25 to 2.27 and table 2.7].	76	90	96	100	111	115	%
5. Convergence PMP for indicated durations [steps 3 X 4]	9.02	10.68	11.40	11.87	13.18	13.65	in. (mm)
6. Incremental 10 mi^2 (26 km^2) PMP [successive subtraction in step 5]	9.02	1.66	0.72	0.47	1.31	0.47	in. (mm)
7. Areal reduction [select from figs. 2.28 and 2.29]	89	97	98	99	100	100	%
8. Areally reduced PMP [step 6 X step 7]	8.03	1.61	0.71	0.47	0.47	0.47	in. (mm)
9. Drainage average PMP [accumulated values of step 8]	8.03	9.64	10.35	10.82	11.29	11.76	in. (mm)
B. Orographic PMP							
1. Drainage average orographic index from figure 3.11a to d. (REVISED) 2.0 in. (mm)							
2. Areal reduction [figure 3.20] 93 %							
3. Adjustment for month [one of figs. 3.12 to 3.17] 100%							
4. Areally and seasonally adjusted PMP [steps 1 X 2 X 3] 1.86 in. (mm)							
5. Durational variation [table ~~3.6~~ 3.9]	36	63	84	100	140	159	%
6. Orographic PMP for given durations [steps 4 X 5]	0.66	1.17	1.56	1.86	2.60	2.96	in. (mm)
C. Total PMP							
1. Add steps A9 and B6	8.69	10.81	11.91	12.68	13.89	14.72	in. (mm)

2. PMP for other durations from smooth curve fitted to plot of computed data.

3. Comparison with local-storm PMP (see sec. 6.3).

Table A-4 --Local-storm PMP computation for the Powerline F.R.S. Watershed (Reference 16)

Drainage Powerline F.R.S. Area 46.3 mi^2 (km^2)
Latitude 33°20' Longitude 111°30 Minimum Elevation ____ ft (m)

Steps correspond to those in sec. 6.3A.

1. Average 1-hr 1-mi^2 (2.6-km^2) PMP for drainage [fig. 4.5]. — 11.5 in. (mm)

2. a. Reduction for elevation. [No adjustment for elevations up to 5,000 feet (1,524 m): 5% decrease per 1,000 feet (305 m) above 5,000 feet (1,524 m)]. — 100 %

   b. Multiply step 1 by step 2a. — 11.5 in. (mm)

3. Average 6/1-hr ratio for drainage [fig. 4.7]. — 1.32

	Duration (hr)									
	1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio of step 3 [table 4.4].	72	88	95	100	115	122	126	130	132	%

5. 1-mi^2 (2.6-km^2) PMP for indicated durations [step 2b X step 4].
   8.28 10.12 10.92 11.50
   13.23 14.03 14.49 14.95 15.18 ___ in. (mm)

6. Areal reduction [fig. 4.9].	48	56	58	61	66	68	71	72	73	%

7. Areal reduced PMP [steps 5 X 6].
   3.97 5.67 6.33 7.02
   8.73 9.54 10.29 10.76 11.08 ___ ___ in. (mm)

8. Incremental PMP [successive subtraction in step 7].
   7.02 1.71 0.81 0.75 0.47 0.32 in. (mm)
   3.97 1.70 0.66 0.69 } 15-min. increments

9. Time sequence of incremental PMP according to:

   Hourly increments [table 4.7]. — 0.47 0.81 7.02 1.71 0.75 0.32 in. (mm)

   Four largest 15-min. increments [table 4.8]. — 3.97 1.70 0.69 0.66 in. (mm)

Table A-5

72 hour Distribution of the PMP for the Powerline F.R.S. Watershed
using the McMicken Dam Hydrology (TES, JMR 1987)

IN	360									
PB	14.72									
PI	1.045	2.157	6.751	1.220	0.609	0.609	0.609	0.609	0.282	0.282
PI	0.282	0.282								

Table A-6

6 hour Distribution of the PMP for the Powerline F.R.S. Watershed
(Reference 16)

IN	60					
PB	11.08					
PI	0.47	0.81	7.02	1.71	0.75	0.32

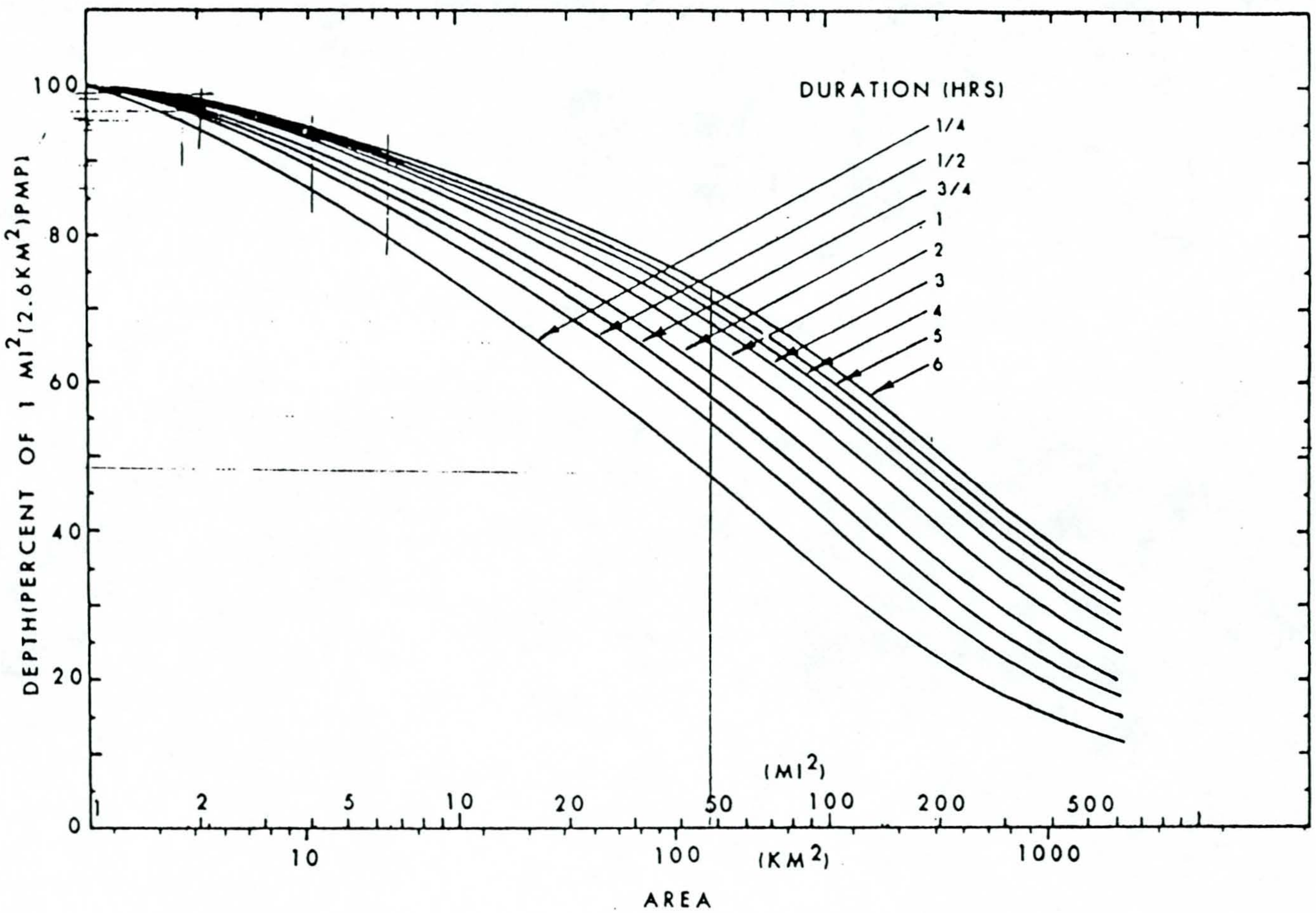


Figure A- 2 --*Adopted depth-area relations for local-storm PMP.* for Powerline F.R.S. Watershed (Reference 16)

APPENDIX B

REVISED SUBBASIN AREAS

Table B-1

Revised Drainage Areas for
the Powerline F.R.S. Watershed

arm length of planimeter: 25.0
scale of map: 1:24,000
factor: 27.5

Watershed	Reading	Area (sq. mi.)	Adjusted Area	SCS Study
Upper Weekes Wash Watershed:				
	254.7	9.26		
10	15.3	.56	.54	.57
11	20.0	.73	.70	.63
12	37.7	1.37	1.32	1.43
13	36.8	1.34	1.29	1.35
14	31.5	1.15	1.11	1.16
15	50.0	1.82	1.76	1.71
16	61.6	2.24	2.16	2.24
17	10.4	.38	.37	.21
		------	------	------
		9.59	9.25	9.30
Lower Weekes Watershed:				
	148.0	5.38		
2A	30.0	1.09		1.08
2BE	33.5	1.22		1.35
2BWW	12.4	.45		.88
2BWE	19.3	.70		.46
2BS	52.5	1.91		2.10
		------		------
		5.37		5.88
Total Weekes Wash Watershed Area:			$\boxed{14.62}$	
Siphon Draw Watershed:				
	733.8	32.45	(arm length - 30.0	factor - 22.61)
3N	78.8	3.15	2.89	2.85
3S	65.0	2.60	2.39	3.07
3A	32.8	1.31	1.20	1.19
4	323.0	12.92	11.85	13.11
5	153.9	6.16	5.65	5.52
6	214.3	8.57	7.86	7.17
7	16.4	.66	.61	.57
		------		------
		35.37	$\boxed{32.45}$	33.48
Total Powerline F.R.S. Watershed Area:			**47.07 sq. miles**	46.03 sq. miles

APPENDIX C

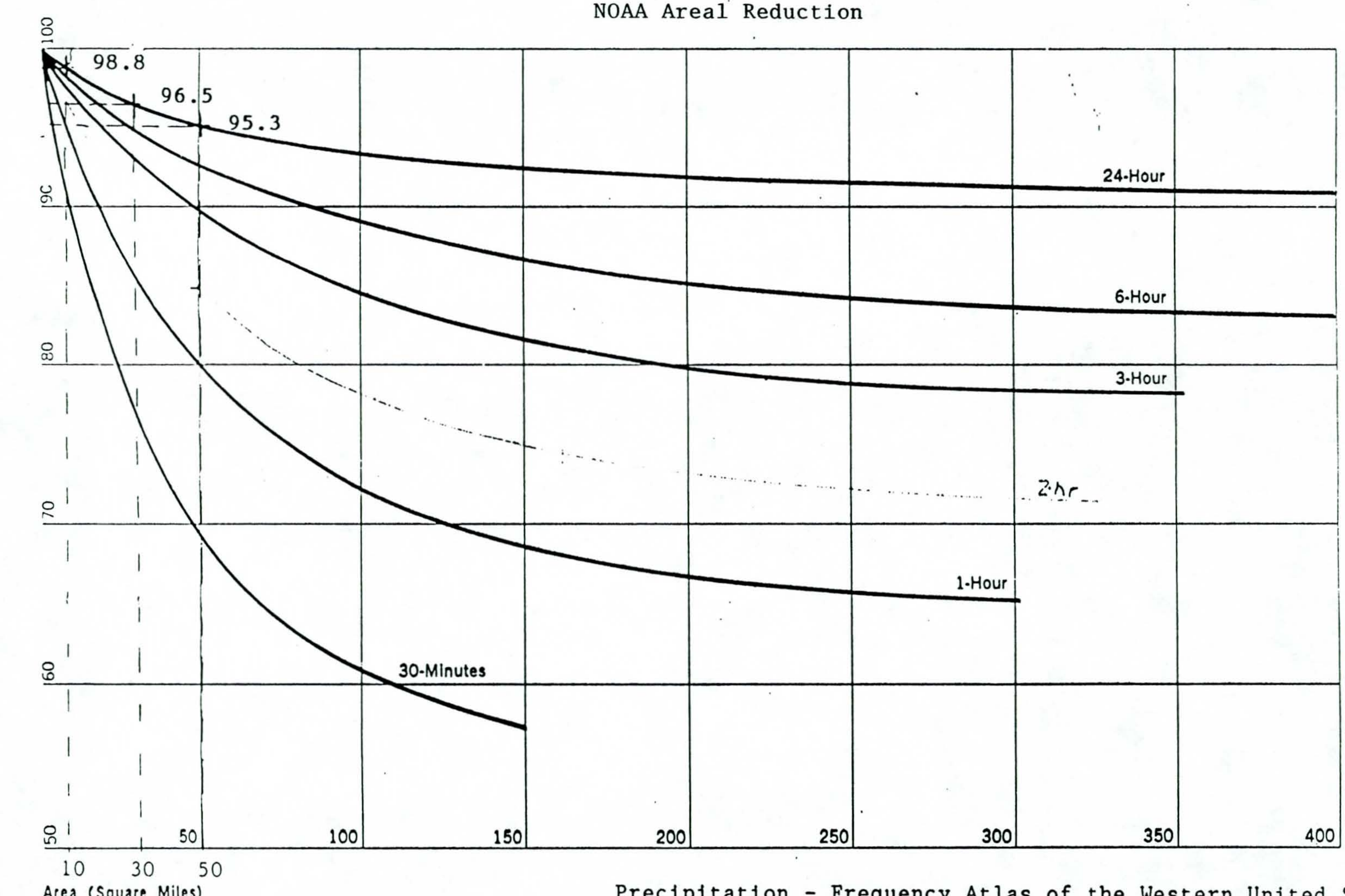
AREAL REDUCTION CALCULATIONS

APPENDIX C-I

NOAA METHOD

Figure C-1

NOAA Areal Reduction



Area sq.mi.	%	Inches
.01	100	3.85
10	98.8	3.80
30	96.5	3.72
50	95.3	3.67

Precipitation - Frequency Atlas of the Western United States, VIII:Arizona, NOAA Atlas 2

(Reference 7)

APPENDIX C-II

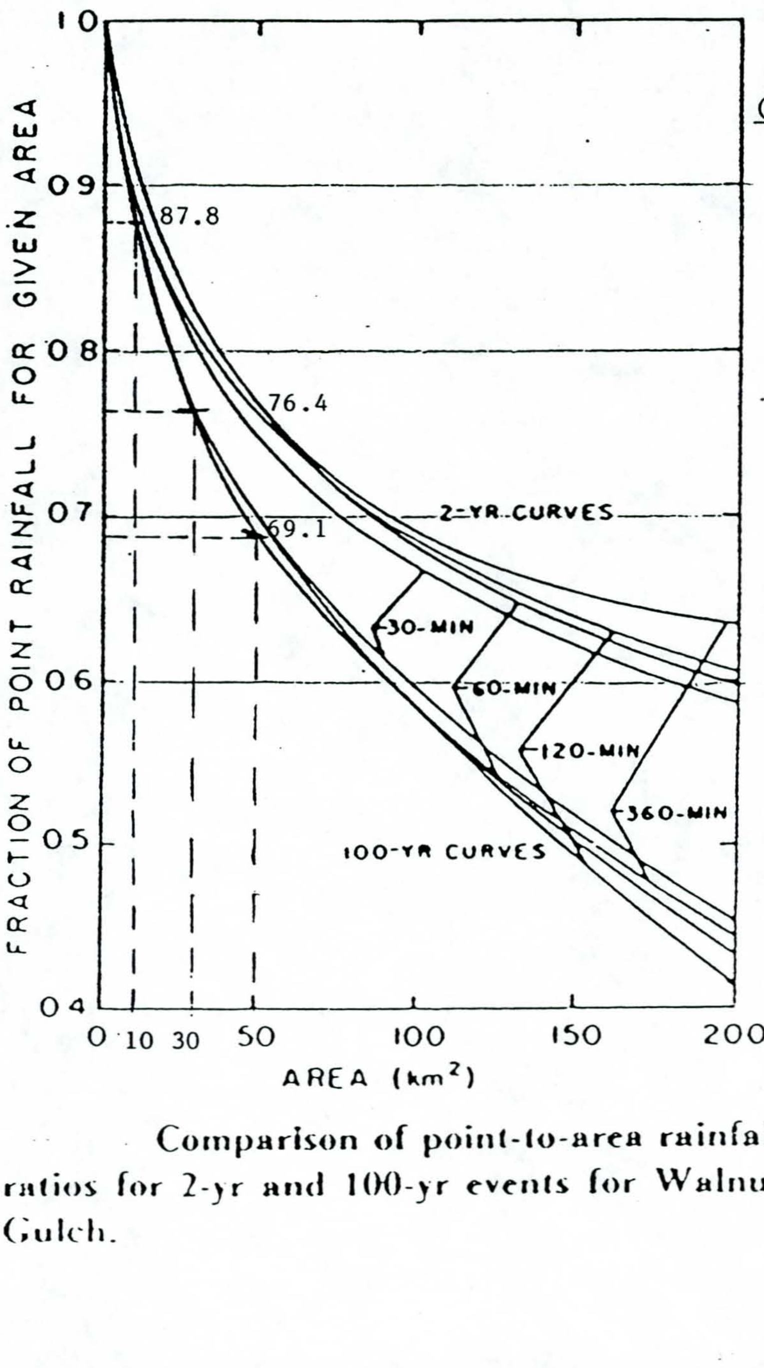
OSBORN METHOD

Figure C-2

Osborn Areal Reduction

Rainfall/Watershed Relationships for Southwestern Thunderstorms, Osborn, Lane, Myers

(Reference 8)



Area (sq. mi.)	%	Inches
.01	100	3.85
10	87.8	3.38
30	76.4	2.94
50	69.1	2.66

Comparison of point-to-area rainfall ratios for 2-yr and 100-yr events for Walnut Gulch.

APPENDIX D

WEEKES WASH BREAKOUT RATING CURVE

Table D-1

Comparison of the Breakout Flows
for the Upper Weekes Wash
North of Highway 60

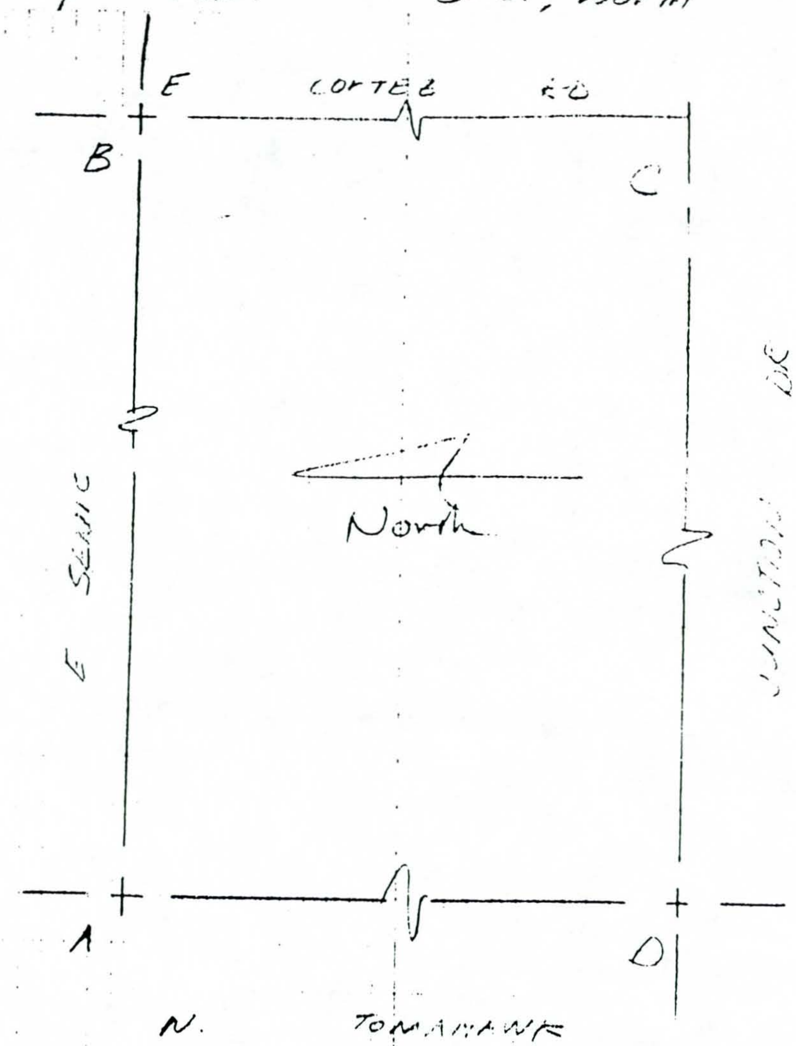
A-N West Engineering		Flood Control District	
DI	DQ	DI	DQ
0	0	0	0
100	0		
		1000	228
1065	580		
		3000	1024
		5000	2182
		6000	2767
		7000	3668
7500	3750		
		9000	4716
10000	4000	10000	5382
12500	5000		

MCFCD CROSS SECTION SURVEY APACHE JT.

88044
DISK TAH-1

T.L.H. 3-24-88 44
GJ.E. Clear, warm

Pt.	description
A	Fd. cotton picker spindle flush w/pvmt at inter. N. Tomahawk & E. Senic
B	set PK nail in pvmt. approx 15' west of inter. Cortez & Senic
C	set PK nail in pvmt approx, west of inter. Cortez & Junction
D	Fd B/C 0.1' below pvmt. at inter. Tomahawk & Junction Dr.
E	Fd. cotton picker spindle flush w/pvmt. at intersection of Cortez & senic Rd.



App-20

APR 12 1988

⊼ @ B B/S A

/S E
∡ Rt. HD= 14.64
179 59 03

/S C
∡ Rt.
275 26 33 HD= 1324.38

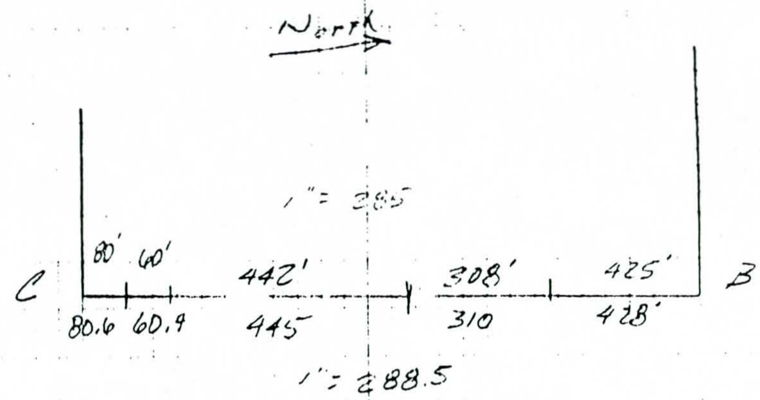
Z
M

⊼ @ D B/S C HD= 2504.61

/S A
Rt 270 02 41 ~~HD=~~

@ A B/S B HD= 2629.54

/S D
∡ Rt 90 01 50 HD= 1318.87



WEST END X-SECTION LINE 46

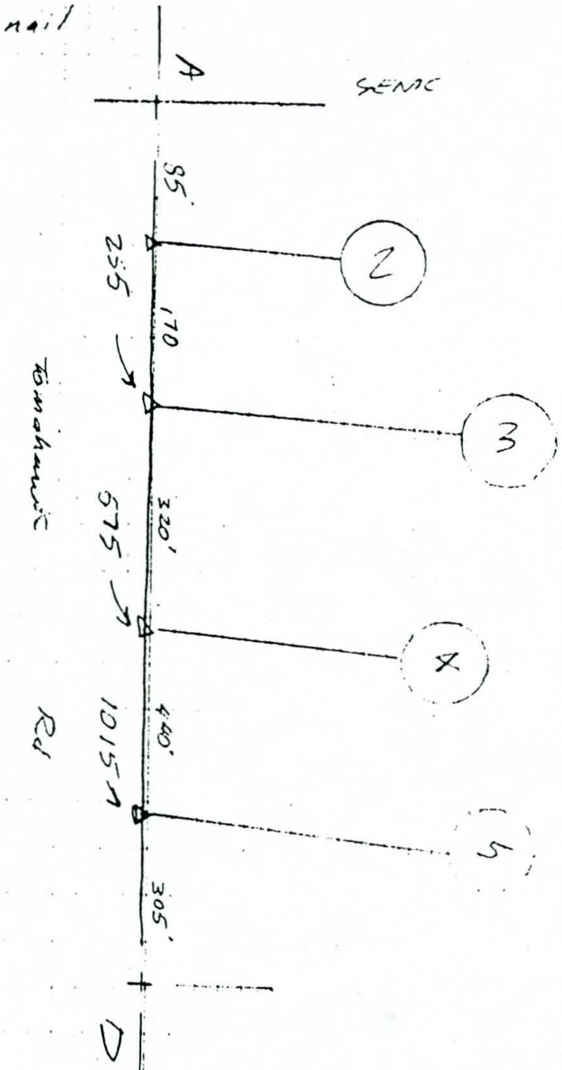
⊼ @ C   B/S D   HD = ~~250.458~~

F/S B

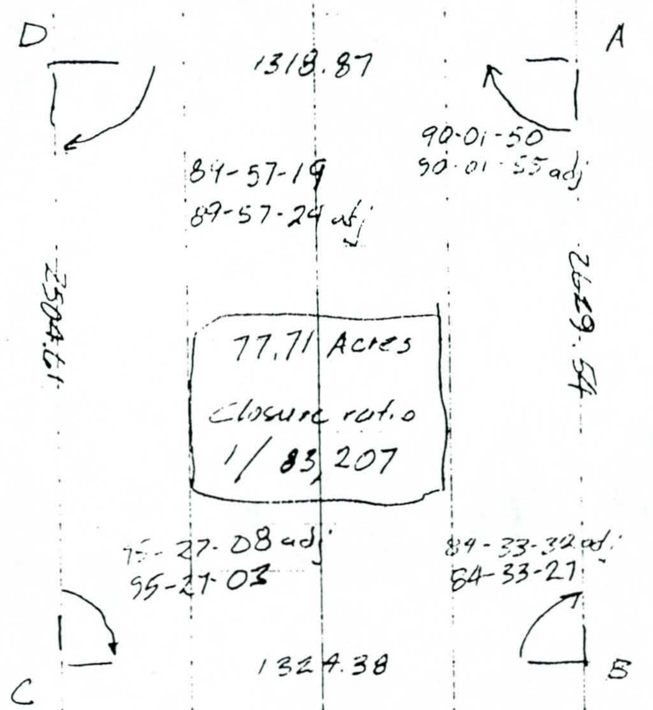
∡ Rt

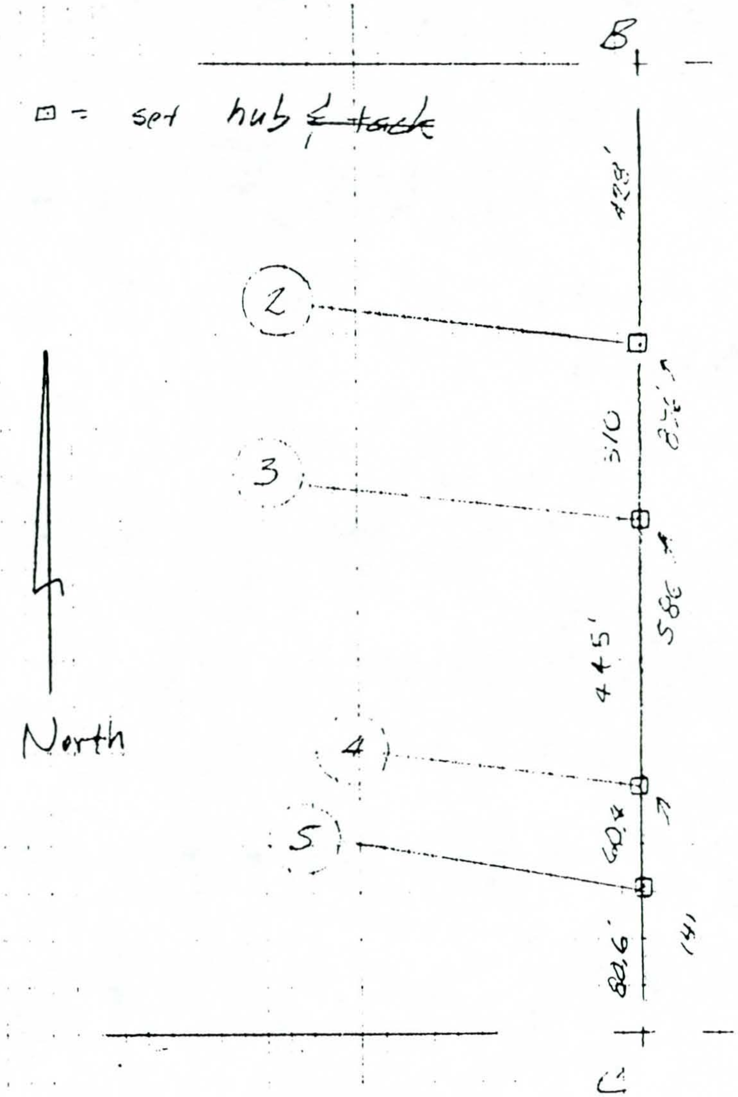
95 27 03 ~~HD=~~

Δ = set PK nail



∡ closure 359-59-39 = +00-00-05/∡





Bench loop

STA	+	HI	-	ELEV.
TBM D				1000.00
	5.52	05.52		
#5W			4.60	00.92
	8.20	09.12		
#4W			2.73	06.39
	12.52	18.91		
#3W			0.70	18.21
	6.72	24.93		
#2W			4.57	20.36
	5.01	25.37		
TBM A			4.52	20.85
	2.58	23.43		
#11			5.60	17.83
	4.71	22.54		
#12			3.93	18.61
	4.89	23.50		
#13			0.38	23.12
	7.90	31.02		
#14			0.92	30.10
	8.79	38.89		
TBM B			6.33	32.56
TBM E			6.63	32.26
	0.63	32.89		
TP			12.49	20.40
	4.37	24.77		

Pt. D see page 44 (relative ELEV.)

set PK nail in pvmt along ℄ Tomahawk

" " " " " " "

" " " " " " " "

" " " " " " " "

Pt. A see pg. 44

set PK nail in pvmt along ℄ Seneca Cr.

" " " " " " " "

" " " " " " "

" " " " " " " "

Pt. B see pg. 44
Pt. E " " 44

STA	+	HI	−	ELEV
		24.77		
2E			6.40	18.37
	6.40	24.77		
3E			11.12	13.65
	5.28	18.93		
4E			5.99	12.94
	7.78	20.72		
5E			3.82	16.90
	2.05	18.95		
TBM C			6.63	12.32
	6.82	19.14		
#16			4.39	14.75
	1.33	16.08		
#17			7.54	08.54
	4.57	13.11		
#18			8.76	04.35
	2.52	06.87		
TBM D			06.87	00.00

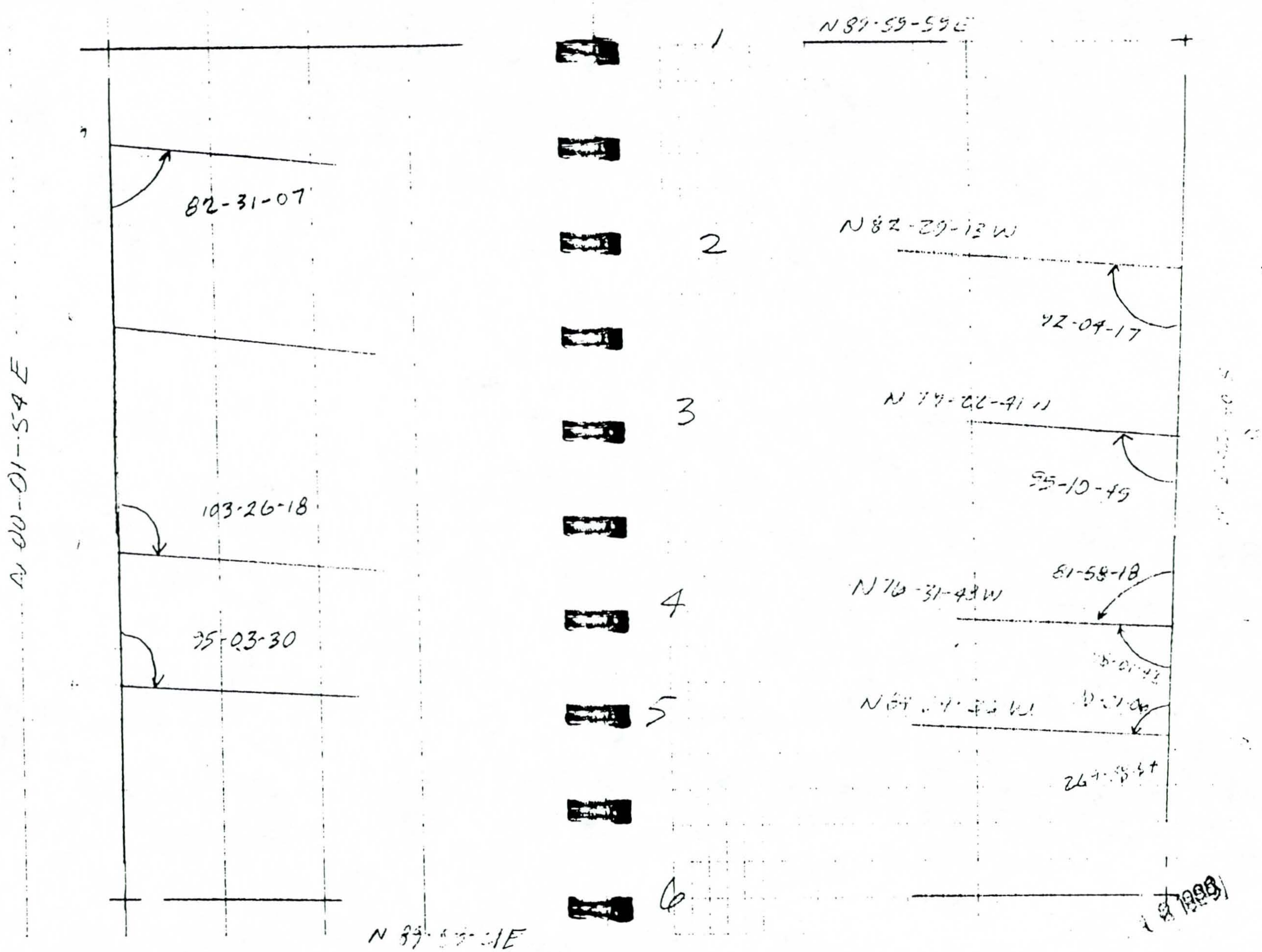
1x2 stake east end line 2
1x2 " " line 3
1x2 " " " line 4
1x2 " " " " 5.

Pt C see page 44

PK nail in pvmt along ℄ Junction Rd.
" " " " " " " "
" " " " " " "

see page 48 closure FLAT

X-SECTION BEARINGS



MCFCD WEEKS WASH

X-SECTION LINE 2 (2611.35)

⊼ @ 2E ELEV. = 18.37

HI = ~~+4.70~~ 4.73

F/S#	HR	HDist	Vert Diff.
1	-5.23	-6.8	0.0
2	-5.15	-18.34	+0.39
3	-5.15	-53.22	+4.17
4	-5.15	-97.20	+4.46
5	-8.00	-125.44	+6.68
6	-8.00	-152.58	+8.97
7	-8.00	-180.17	+10.20
8	-8.00	-249.85	+10.83
9	-8.00	-279.75	+11.76
10	-8.40	-307.33	+10.85

54 1/2

Note: X-Section distances begin at the west line along Tomahawk Rd. and increase going East.

⊼, ⊼ T.L.H. 5-25-88 5

ϕ GLE. Clear, cool

calced. X-sections → ELEV.

17.87 / 2604.55

18.34 / 2593.01

22.12 / 2548.13

22.41 / 2514.15

21.78 ~~24.63~~ / 2485.91

24.07 / 2458.77

25.30 / 2431.18

25.93 / 2361.50

26.86 / 2331.60

25.55 / 2304.02

6

LINE #3 (2604.25)

⊼ @ 3E ELEV. 1013.65

HI = +5.00

F/S#	HR	H. Dist	Vert Diff		
					1013.65 / 2604.25
1	-6.00	-23.39	+2.83	15.48	15.48 / 2580.86
2	-6.00	-126.20	+8.94	21.59	21.59 / 2478.05
3	-8.40	-194.72	+13.88	24.13	24.13 / 2409.53
4	-8.40	-220.04	+14.59	24.84	24.84 / 2384.21

LINE #4 (2588.815)

⊼ @ 4E ELEV. 1012.94

HI = +4.95

F/S#					
					1012.94 / 2588.82
1	-5.00	-16.24	+4.56	17.45	17.45 / 2542.58
2	-5.00	-99.00	+6.87	19.76	19.76 / 2481.82

App-28

APR 1 1988

Line 2

⊼@ 2W — ELEV. 1020.36 — HI = + 5.70

F/S #	HR	HD	Vert. Diff.	ELEV.
1	-4.85	33	0.00	21.21
2	~~-16~~ -16.65	122.30 ~~167.70~~	+1.25 ~~+1.77~~	10.66
3	-16.65	167.70	+1.77	11.18
4	-16.65	239.85	+1.50	10.91
5	-16.65	289.20	+3.75	13.16
6	-16.65	347.24	+6.68	16.09
7	-16.65	387.04	+7.75	17.16
8	-16.65	437.48	+8.24	17.65
9	-20.33 ~~-16.65~~	459.69	+5.44	11.17
10	-20.33 ~~-16.65~~	489.26	6.26	11.99
11	-20.23 ~~-20.33~~	536.87	+5.41	11.24

see pg. 46 - 48 ~~pg~~ book FB-1

1020.36 / 0.0

1021.21 / 33

10.66 / 122.30

11.18 / 167.70

10.91 / 239.85

13.16 / 289.20

Beg Mobile Home 16.09 / 347.24

End " " 17.16 / 387.04

17.65 / 437.48

11.17 / 459.69

11.99 / 489.26

11.24 / 536.87

APR 12 1988

Line 2 cont.

F/S #	HR	HD	Vert. Diff.	ELEV		
12	-20.50	+608.47	+7.48	13.04		13.04 / 608.47
13	-20.50	+704.44	+7.12	12.68		12.68 / 704.44
14	-20.50	835.44	+10.16	15.72		15.72 / 835.44
15	-20.50	855.56	+12.19	17.75	Beg House Blk constr.	17.75 / 855.56
16	-20.50	961.59	+10.14	15.70	End House	15.70 / 961.59
17	-20.50	1054.87	+10.30	15.86		15.86 / 1054.87
18	-20.50	1073.60	+11.70	17.26		17.26 / 1073.60
19	-20.50	1102.54	+9.96	15.52		15.52 / 1102.54
20	-20.50	1165.59	+10.72	16.28		16.28 / 1165.59
21	-~~20.50~~ 20.40	1185.60	+10.79	16.45	Beg Mobile Home	16.45 / 1185.6
22	-~~20.50~~ -20.40	1310.46	+12.25	17.91	End " "	17.91 / 1310.46

⊼ @ 22 continue east

HI = 1023.77 from ⊼

HI = 1023.61 from 22 } 1023.69

Line 2 cont.

16

F/S#	HR	HD	Vert.	Diff.	ELEV.
23	-12.70	+51.95	+5.85		1016.84
24	-12.70	+88.31	+5.16		16.15
25	-12.70	191.76	+6.72		17.71
26	-12.70	254.86	+5.38		16.37
27	-12.70	263.92	+4.64		15.63
28	-12.70	272.28	+5.52		16.51
29	"	327.43	+7.81		18.80
30	-12.70	345.11	+7.76		18.75
31	"	360.50	+8.52		19.51
32	-12.70	389.10	+8.20		19.19
33	"	457.49	+9.15		20.14
34	-12.70	532.64	+7.99		18.98

1016.84	1362.41
16.15	1398.77
17.71	1502.22
16.37	1565.32
15.63	1574.38
16.51	1582.74
18.80	1637.89
18.75	1655.57
19.51	1670.96
19.19	1699.56
20.14	1767.95
18.98	1843.10

1989

Line 2 cont.

F/S #	HR	HD	Vert Diff	ELEV
35	~~-12.70~~ -16.48	645.04	+9.45	1016.66
36	-20.45	692.43	+12.17	15.41
37	-20.45	739.91	+14.14	17.38
38	-20.45	779.07	+12.57	15.81
39	-20.45	795.86	+10.04	13.28
40	-20.45	833.14	+11.91	15.15
41	"	880.41	+15.16	18.40
42	-20.45	940.07	+16.77	compute 20.01
43				

.7

16.66	1955.50
15.41	2002.89
17.38	2050.37
15.81	2089.53
13.28	2105.53
15.15	2143.60
18.40	2190.87
20.01	2250.53

App-32

Auxilary Pts set along Lines 3, 4, 5

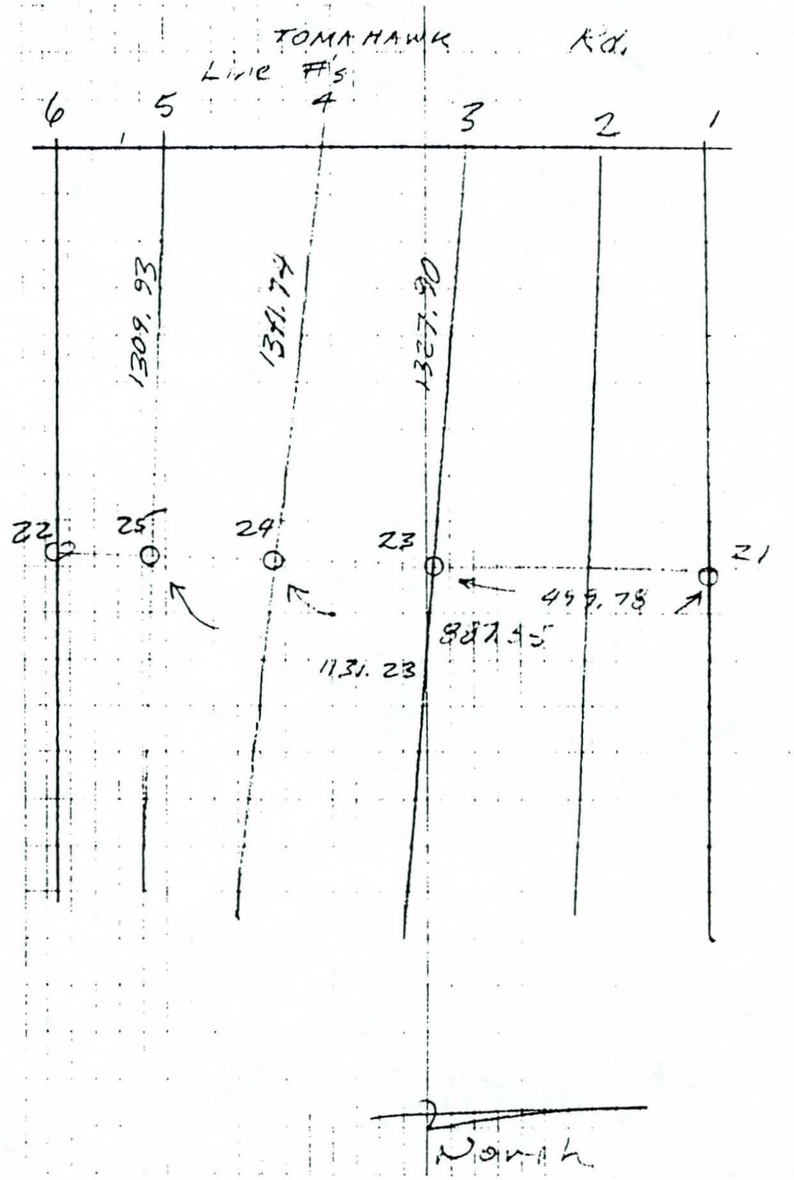
ΔI, ⊼ T.L.H
φ G.L.E.

3-21-88
Clear, cool,

18

STA TBM #	+	HI	-	ELEV
12				18.61
	5.90	24.51		
23			10.43	14.08
	4.29	18.37		
24			7.96	10.41
	5.94	16.35		
25			3.85	12.50
	3.12	15.62		
17			7.13	08.49 08.54

closure 0.05' low



LINE 3

⊼@ 3W   ELEV. 1018.21

HI= +5.50

F/S #	HR	HD	Vert Diff.	ELEV.
1	-5.40	+33	0.00	18.31
2	-8.80	+127.00 ~~169.00~~	-6.00	08.91
3	-8.80	+169.00	-6.00	08.91
4	"	+199.33	-6.64	08.27
5	"	+265.91	-6.08	08.83
6	"	+308.85	-2.61	12.30
7	"	+350.22	-0.10	14.81
8	"	+395.57	-2.91	12.00
9	"	+465.18	-5.82	09.09
10		+510.20	-4.67	10.24
11		+627.05	-3.52	11.39

18.21 / 0.0

18.31 / 33

08.91 / 127

08.91 / 169

08.27 / 199.33

08.83 / 265.9

12.30 / 308.85

14.81 / 350.22

12.00 / 395.57

09.09 / 465.18

10.24 / 510.20

11.39 / 627.05

LINE 3 cont.

F/S #	HR	HD	Vert. Diff.	ELEV.
12	-8.80	724.44	-2.64	12.27
13	"	795.54	-1.09	13.82
14	"	831.24	-1.95	12.96
15	"	852.38	-0.94	13.97
16	"	890.84	-0.02	14.89
17	"	899.03	-3.98	10.93
18	"	916.12	-2.36	12.55
19	"	933.01	-3.79	11.12
20	-8.80	955.90	-1.44	13.47
21	-8.80	986.46	-0.73	14.18
22	-12.55	1014.55	+1.47	12.63
23	-12.55	1047.28	+3.24	14.40

20

1012.27 / 724.44 –

13.82 / 795.54 –

12.96 / 831.24 –

13.97 / 852.38 –

14.89 / 890.84 –

10.93 / 899.03 –

12.55 / 916.12 –

11.12 / 933.01 –

13.47 / 955.90 –

14.18 / 986.46 –

12.63 / 1014.55 –

14.40 / 1047.28 –

2 1989

LINE 3 Cont.

F/S#	HR	HD	Vert. Diff.	ELEV.	
24	-12.55	1191.06	+2.96	14.12	1014.12 / 1191.06 -
25	-16.80	1353.16	+7.28	14.19	14.19 / 1353.16 -
26	-16.80	1449.64	+8.11	15.02	15.02 / 1449.64 -
27	-16.80	1527.84	+7.80	14.71	14.71 / 1527.84 -
28	"	1570.76	+9.08	15.99	along S. edge ~~House~~ Mobile Home 15.99 / 1570.76 -
29	"	1611.22	+11.40	18.31	18.31 / 1611.22 -
30	-16.80	1675.69	+10.99	17.90	17.90 / 1675.69 -
31	-16.80	1785.00	+9.36	16.27	16.27 / 1785.00 -
32	"	1840.27	+9.36	16.27	S.W. cor mobile home 16.27 / 1840.27
33	-16.80	1864.30	+9.35	16.26	S.E. Cor. " 16.26 / 1864.30
34	-20.55	1906.85	+10.26	13.42	13.42 / 1906.85 -
35	-20.55	1944.95	+9.06	12.22	12.22 / 1944.95 -

LINE 3 Cont.

F/S#	HR	HD	Vert. Diff.	
36	-20.55	1958.14	+7.86	11.02
37	-20.45	1996.48	+9.45	12.71
38	-20.45	2127.01	+16.92	20.18
39	-20.45	2213.09	+21.90	25.16
40	-20.45	2336.19	+21.40	24.66

Line 3 complete

$\frac{11.02}{1958.14}$ –

$\frac{12.71}{1996.48}$ –

$\frac{20.18}{2127.01}$ –

$\frac{25.16}{2213.09}$ –

$\frac{24.66}{2336.19}$ –

APR 1 2 1989

LINE 4

π @ 4W ELEV. 1006.39

HI= +5.53

F/S #	HR	HD	Vert. Diff	
1	-5.91	33.25	0.00	06.01
2	-12.80	49.64	+5.48	04.60
3	4.90	218.95	+1.78	09.30
π @ 3	HI= 5.70			09.30
4	-12.30	+43.80	+5.65	07.85
5		+82.54	+4.49	06.64
6	-3.80	+84.39	+3.47	05.67
7	"	+100.37	+2.50	04.70
8	-12.80	+111.37	+1.67	03.87
9	"	+130.47	+4.37	06.57
10	-12.80	+253.01	+6.80	09.09

continued on page 29

1006.39	0
1006.01	33.25
04.60	49.64
09.30	218.95
07.85	262.75
06.64	301.31
05.67	303.82
04.70	319.92
03.87	330.32
06.57	349.42
09.09	471.96

Line 5 from pg. 8

HS #	HR	HD	Vert. Diff.		ELEV.
∆ @ 25 Traverse mid Pt Line 5					HI = +5.62
					[illegible]
16	-20.80	-674.20	+9.92	635.73	1001.62
17	"	-564.07	+9.83	745.86	1001.58
18	"	-451.40	+10.56	858.53	1002.[illegible]
19	"	446.24	[illegible]	863.69	[illegible]
20	-20.30	-438.15	+3.60	871.78	1000.30
21	"	433.67	+10.84	876.26	1002.54
22	"	395.40	+11.37	914.53	1003.07
23	"	365.[illegible]	+11.[illegible]	944.7	1002.98
24		323.06	+11.49	986.87	1003.19
25	-20.80	286.50	+11.17	1053.43	1002.84
26	8.50	217.50	-1.31	1092.43	1008.31

[illegible] TCM GLE [illegible] 24

see page 18 ELEV. 1012.50 / 1301.93

335.73

[illegible]

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APR 12 1988

FS #	rR	HD	Vert	Diff.	Elev
27	-8.50	-150.25	-1.36	1159.68	1008.26 -
28	-8.50	-70.84	-0.04	1239.09	1009.58 -
29	-	-12.15	+2.92	1297.78	1012.54 ~~15.20~~ -
—	x @ Station		—		
30	-8.50	+13.63	+3.35	1328.61	07.13 ~~15.63~~ ~~[illegible]~~ -
31	-4.70	+75.22	-2.41	1389.15	1007.21 -

06.70
1297.78
12.50
1309.53

07.13
1328.61

Dog House

Continue Line 5 from F/S #3 page 7

26

F/S #	HR	HD	Vert. Diff	ELEV.
~~3~~ T@3			HI = +5.61	
4	-8.35	-39.34	+1.35	08.62
5	"	-68.32	+0.79	08.06
6	"	-76.48	0.00	07.27
7	"	-93.72	+0.96	08.23
8	"	-108.91	+3.23	10.50
9	-8.35	-133.35	+4.41	11.68
10	See Next			
11		PAGE		
12				
13	-24.80	-328.47	+18.99	15.42

15.62 / 2240.13

08.62 / 2200.79

08.06 / 2171.81

07.27 / 2163.65

08.23 / 2146.41

10.50 / 2131.22

11.68 / 2101.78

15.42 / 1911.66

APR 12 1988

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Sta	HR	HD	Vert Dist	Elev
⊼@13				15.42
12	-20.70	+62.41	+14.18	14.47
	-20.70	+120.88	+16.80	17.09
		(191.66)		
14	+4.35 ~~20.85~~	-109.75	-1.22	15.42
15	-12.75	-170.91	+6.53	14.77
16	"	-236.96	+4.30	12.54
17	+12.75	-295.76	+4.99	13.23
18	"	-362.72	+3.91	12.15
19	"	-387.77	+3.29	11.53
20	"	-409.20	-0.30	07.94
21	"	-430.43	-2.45	05.79
22	"	-442.14	-4.43	03.81

HI = 5.57

$\frac{15.42}{191.66}$

Bay House

F/SH	HIS	HD	Vert Diff.	Elev.	
23	-12.75	-450.08	-4.29	03.95	
24	"	-457.14	-2.14	06.10	
25	-12.75	-474.00	-1.84	06.40	End House

Line 5 Now complete

LINE 4 continued from pg. 23

F/S #	HR	HD	Vert Diff	ELEV.		
		(218.95)				
11	12.30	+338.91	+7.66	09.86	1009.86	547.91 –
12		+360.07	+7.94	10.14	10.14	579.02 –
13	"	+364.64	+8.65	10.85	10.85	583.59 –
14	"	+370.04	+7.61	09.81	09.81	588.99 –
15	12.80	+376.60	+ 8.41	10.61	10.61	595.55 –
16	"	+[illegible].70	+[illegible]	09.91	09.91	630.65 –
17	"	+415.31	+ 6.96	09.16	09.16	634.26 –
18	"	+424.39	+ 6.64	08.84	08.84	643.34 –
19	"	+428.30	+5.76	07.96	07.96	647.25 –
20	"	+447.77	+5.58	07.78	07.78	666.72 —
21	"	+460.27	+7.97	10.17	10.17	679.22 —
22		+470.65	+9.22	11.42	11.42	689.60 —

Line 4 continued

F/S #	HR	HD	Vert. Diff.	ELEV.
23	-16.60 ~~+2.80~~	+456.11	+11.26	09.66
24	-16.60	462.90	+10.72	09.12
25	-16.60	+545.75	+12.04	10.44
26	-20.60	+526.80	+14.63	09.03
27	"	+534.19	+16.42	10.82
28		+547.73	+15.82	10.22
29		+553.99	+15.14	09.54
30	"	+561.85	+16.77	11.17
31	"	+586.18	+16.16	10.56
32	"	+605.39	+15.78	10.18
33	"	+624.70	+16.44	10.34
34	"			

?

09.66 / 675.06

09.12 / 681.85

10.44

09.03 / 745.68 –

10.82 / 753.14 –

10.22 / 766.68 –

09.54 / 772.94 –

11.17 / 780.80 –

10.56 / 805.13 –

10.18 / 824.34 –

10.34 / 843.05 –

Line 4 cont.

31

F/S#	HR	HD	Vert. Diff.	ELEV.
⊼ @ 24 Traverse Pt.			HI = 15.71	
34	20.00	-456.20	+15.40	10.92
35	"	-470.02	+13.96	09.48
36	"	-433.66	+13.94	09.46
37	20.00	-436.38	+15.40	10.92
38	16.50	-411.88	+12.05	11.37
39	"	-353.62	+11.87	11.19
40	16.80	-330.01	+11.83	11.15
41		-308.20	+11.24	10.56
42	"	-281.51	+11.83 ~~+12.24~~	11.15
43	16.80	-261.69	+12.01	11.33
44	5.00	-121.20	+0.68	11.80

10.41
1341.74
10.92
885.5
09.48
891.72
09.46
908.08
915.36
930.36
982.12
11.15
1011.73
1033.54
1060.51
1080.05
11.80
1220.37
1341.77

LINE 4 cont.

FS#	HL	HD	Vert. Diff	EL
⊼@24		(1391.74)		
45	-16.80	+131.79	+12.89	12.21
46	-16.80	+204.53	+12.30	11.62
47	-20.60	+236.48	+12.68	08.20
48	-20.60	+[illegible]	+2351	19.03
⊼ @ 48		HI = 15.09 (1952.91)		
49	~~8.00~~ -7.60	+81.4	0.0	16.52
50	-5.50	+123.6	0.0	18.62
51	-8.00	+236.2	0.0	16.12
52	-10.81	-151.6	0.0	13.31
53	-13.80	-205.7	0.0	10.32
54	-18.11	-267.4	0.0	06.01

12.21 / 1473.53 -

11.62 / 1546.27 -

08.20 / 1578.22 -

19.03 / 1952.91 -

16.52 / 2034.31 -

18.62 / 2076.5 -

16.12 / 2189.1 -

13.31 / 1801.3 -

10.32 / 1747.2 -

06.01 / 1685.5 -

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2 1988

LINE 4 Cont.

F/S#	HR	HD	Vert. D	ELEV.
		(1952.91)		
55	-16.50	-286.0	0.0	07.62
56	-16.60	-320.0	0.0	07.57
57	-20.10	+317.06	+13.22	17.24

$\frac{07.62}{1666.9}$

$\frac{07.57}{1632.9}$

$\frac{17.24}{2269.97}$

App-48

Figure D-2

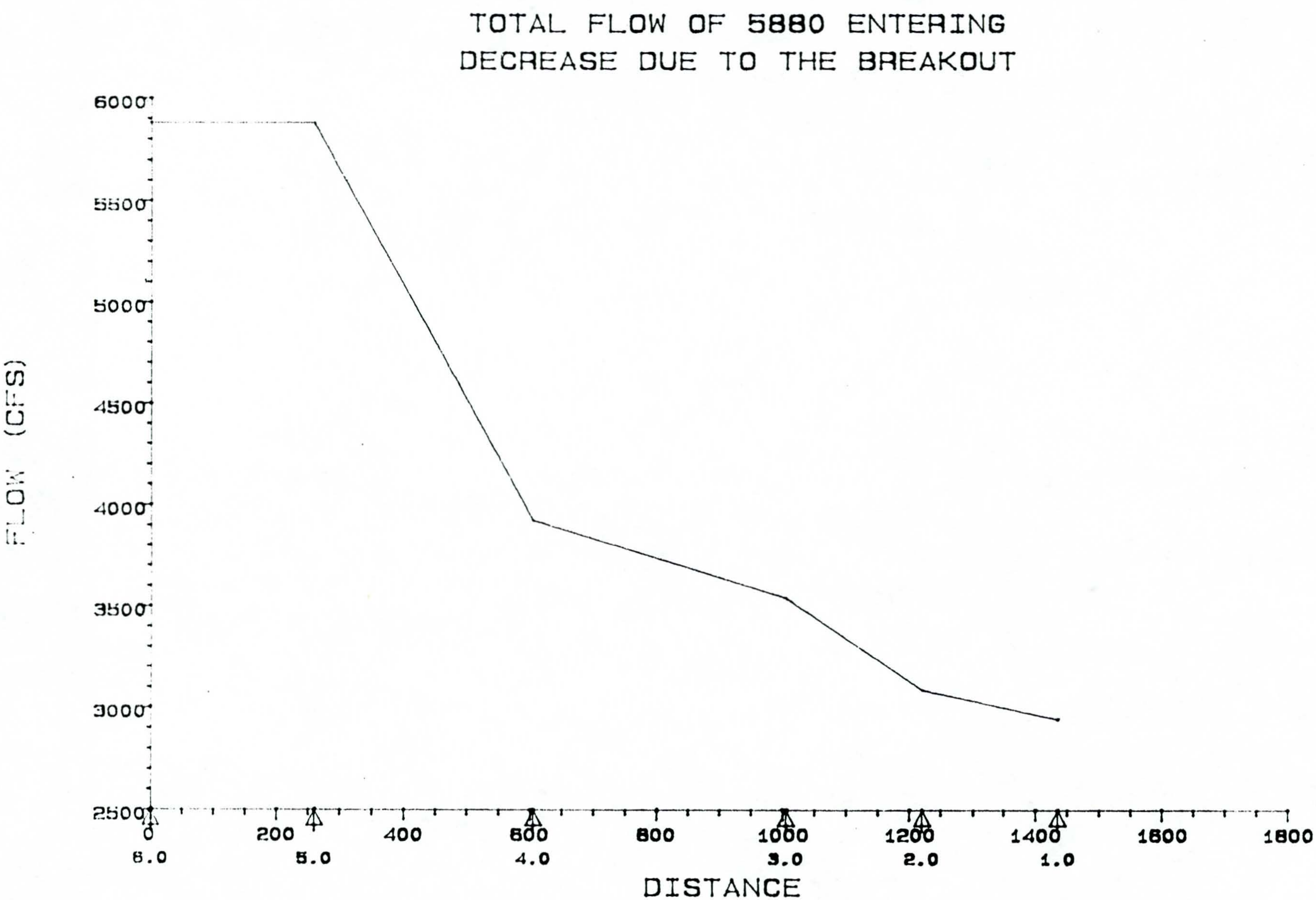
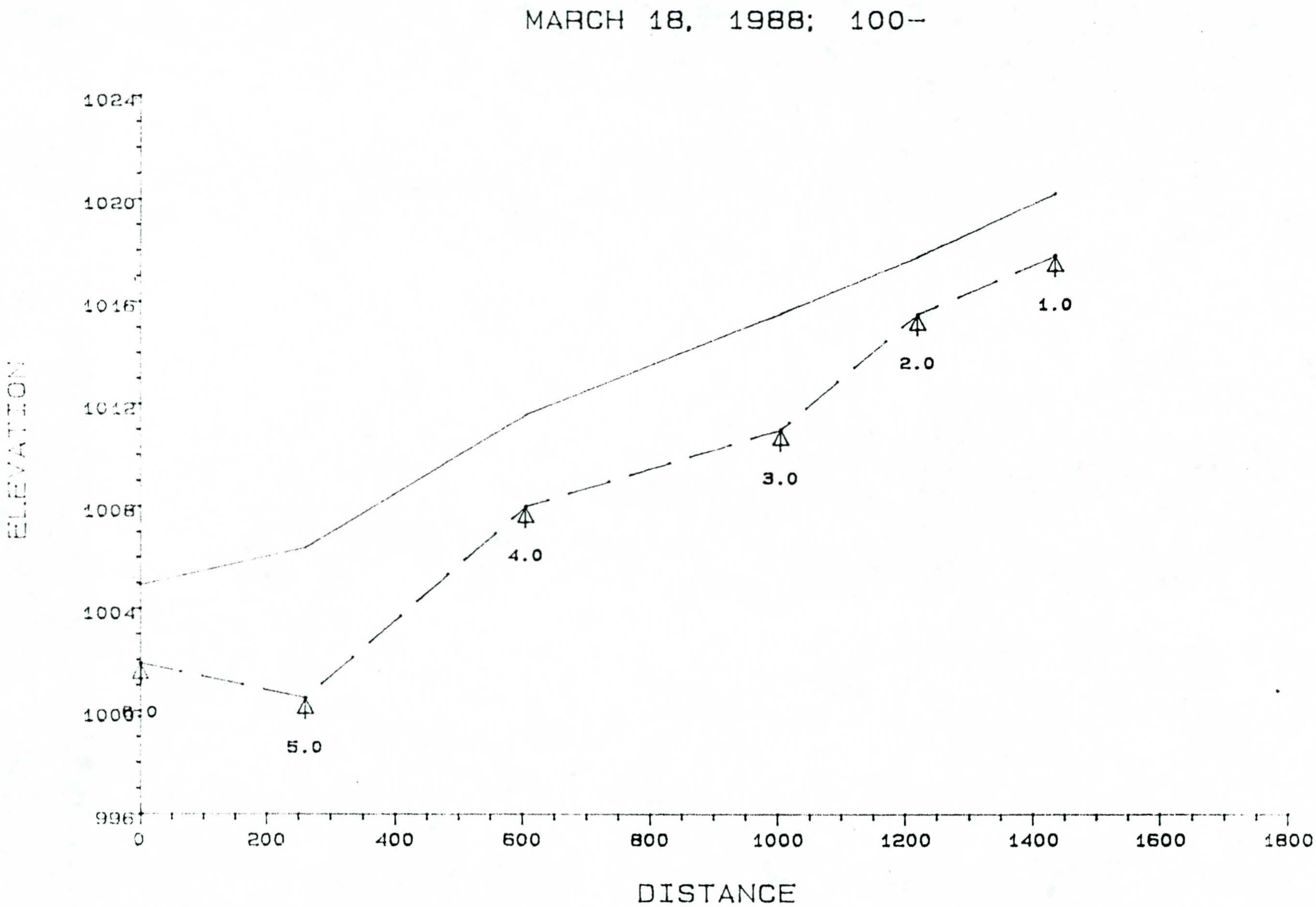

TOTAL FLOW OF 5880 ENTERING
DECREASE DUE TO THE BREAKOUT
FLOW (CFS)
6000
5500
5000
4500
4000
3500
3000
2500
0
200
400
600
800
1000
1200
1400
1600
1800
6.0
5.0
4.0
3.0
2.0
1.0
DISTANCE

Figure D-3



MARCH 18, 1988; 100-
1024
1020
1016
1012
1008
1004
1000
996
ELEVATION
0
200
400
600
800
1000
1200
1400
1600
1800
DISTANCE
1.0
2.0
3.0
4.0
5.0

Figure D-4

WEEKES WASH BREAKOUT AREA
BETWEEN SCENIC AND JUNCTION DRIVE.
CROSS-SECTION NO. 1.000

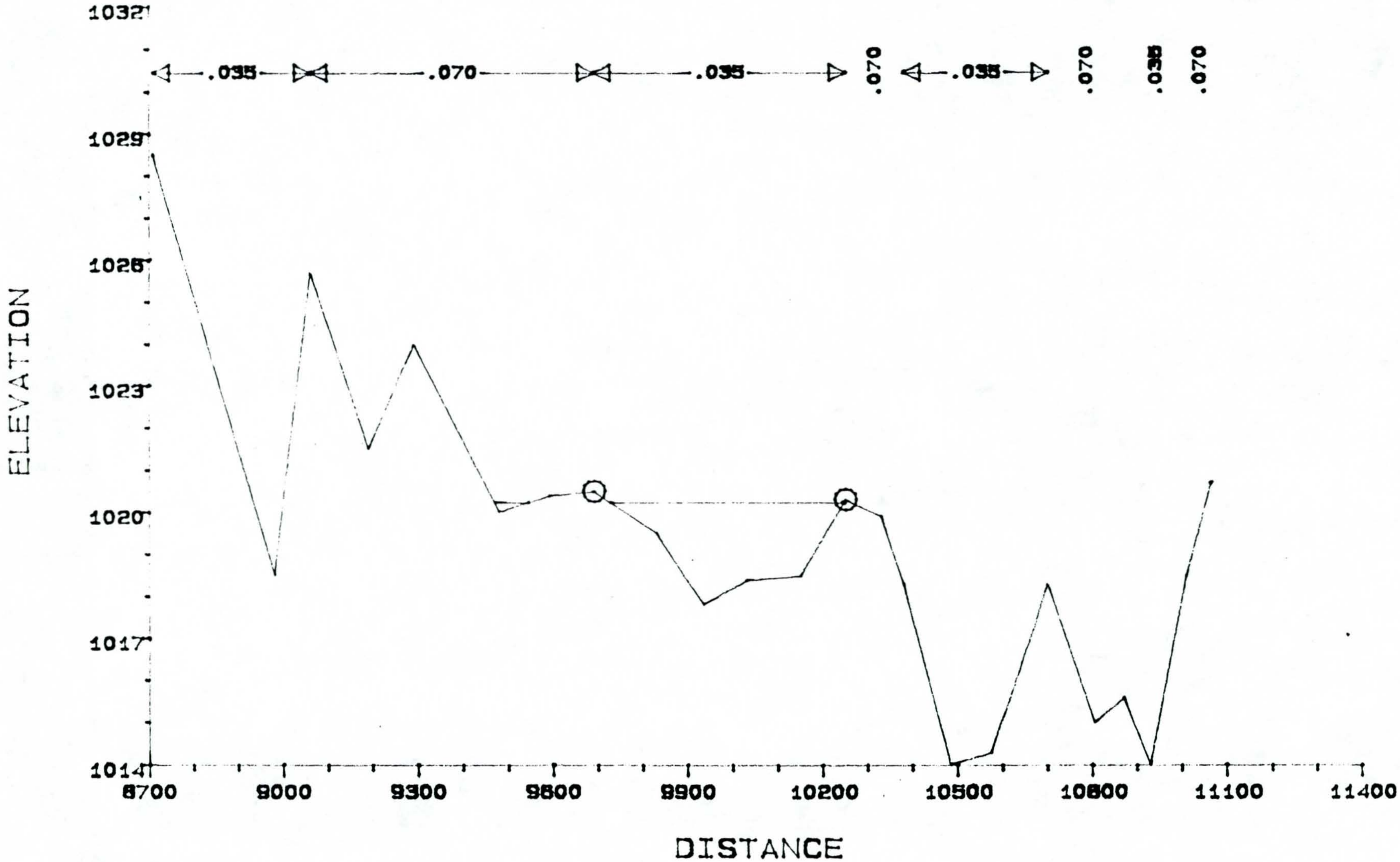


Figure D-5

WEEKES WASH BREAKOUT AREA
BETWEEN SCENIC AND JUNCTION DRIVE.
CROSS-SECTION NO. 2.000

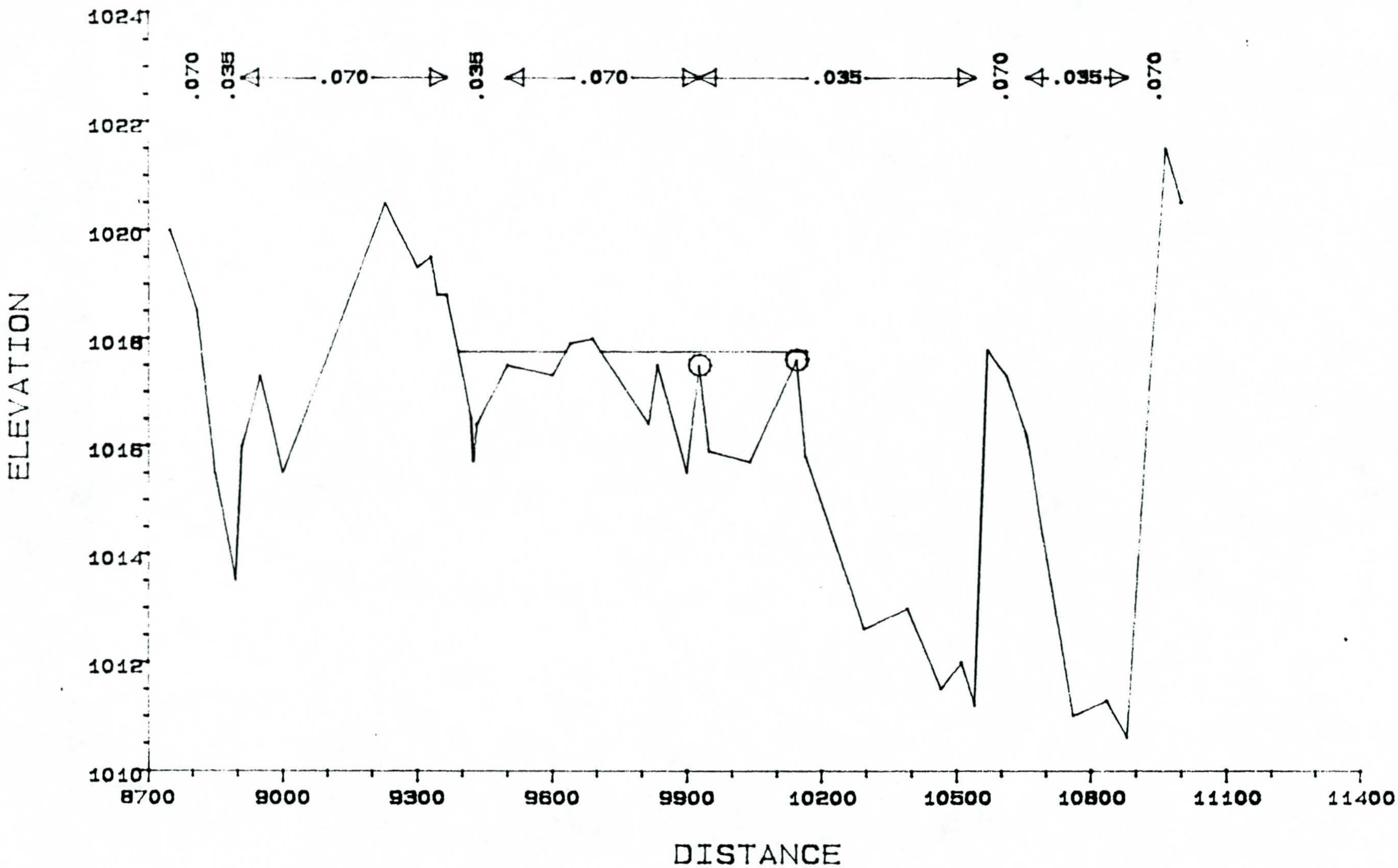


Figure D-6

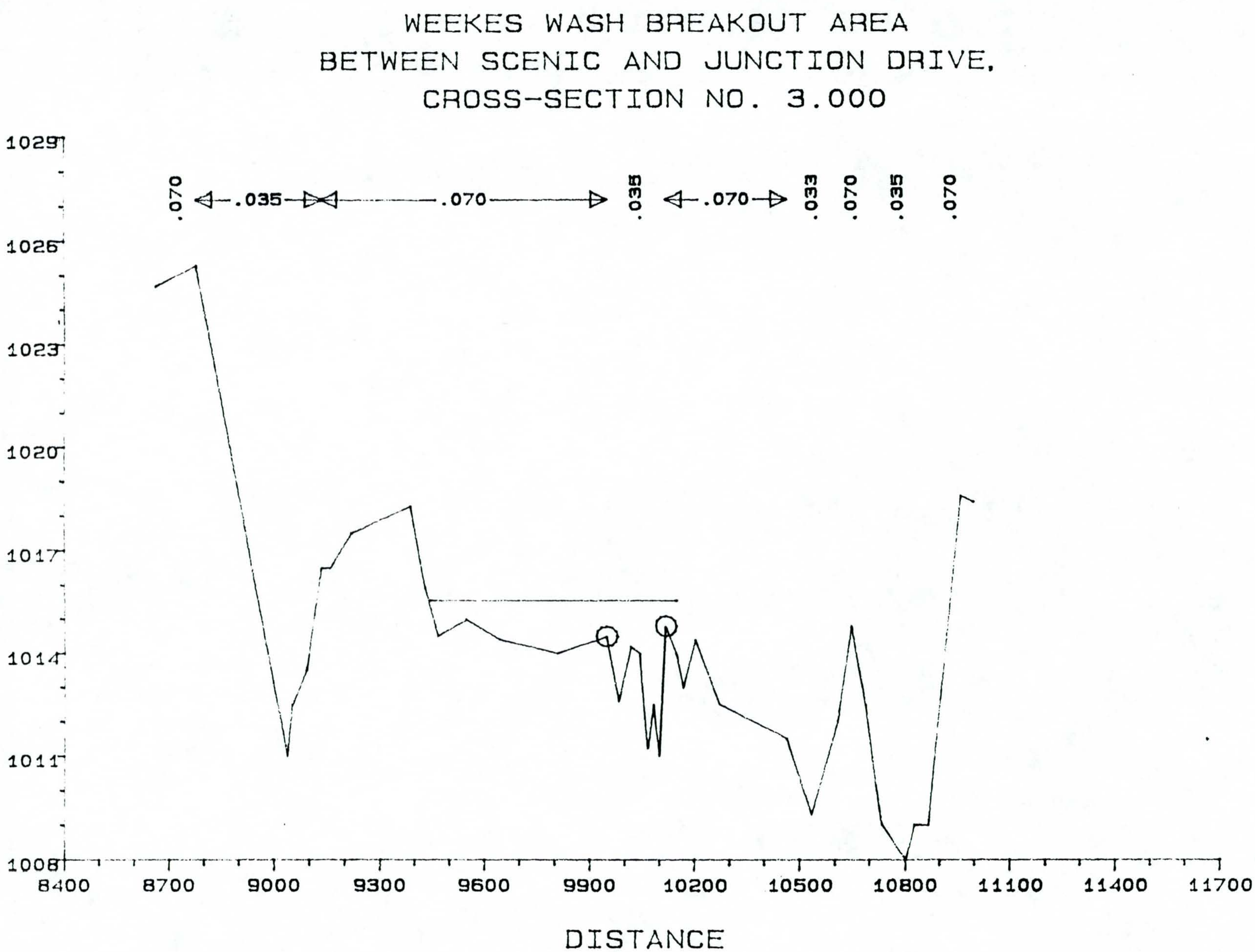

WEEKES WASH BREAKOUT AREA
BETWEEN SCENIC AND JUNCTION DRIVE.
CROSS-SECTION NO. 3.000
.070
.035
.070
.035
.070
.033
.070
.035
.070
ELEVATION
1029
1026
1023
1020
1017
1014
1011
1008
DISTANCE
8400
8700
9000
9300
9600
9900
10200
10500
10800
11100
11400
11700

Figure D- 7

WEEKES WASH BREAKOUT AREA
BETWEEN SCENIC AND JUNCTION DRIVE.
CROSS-SECTION NO. 4.000

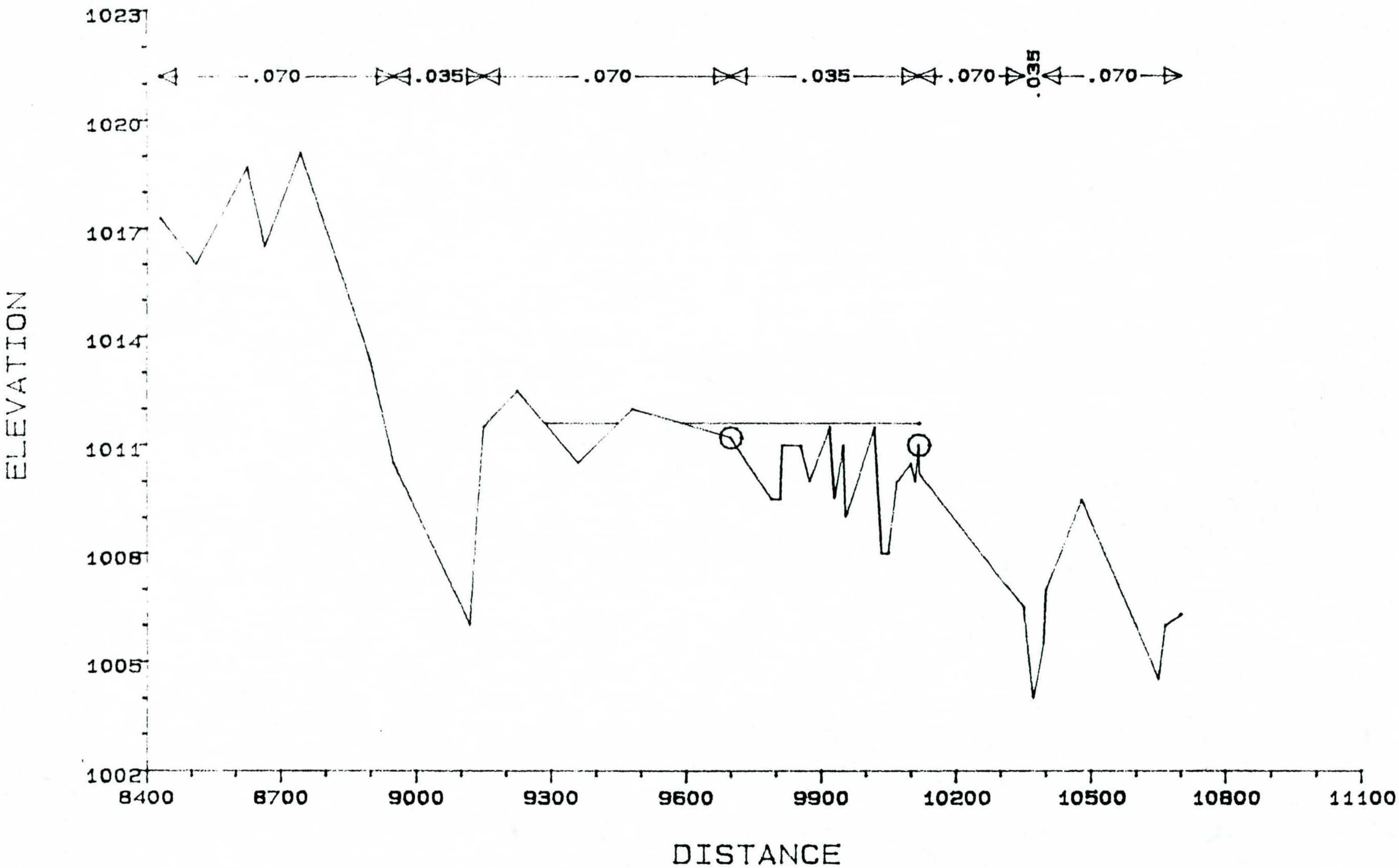


Figure D-8

WEEKES WASH BREAKOUT AREA
BETWEEN SCENIC AND JUNCTION DRIVE.
CROSS-SECTION NO. 5.000

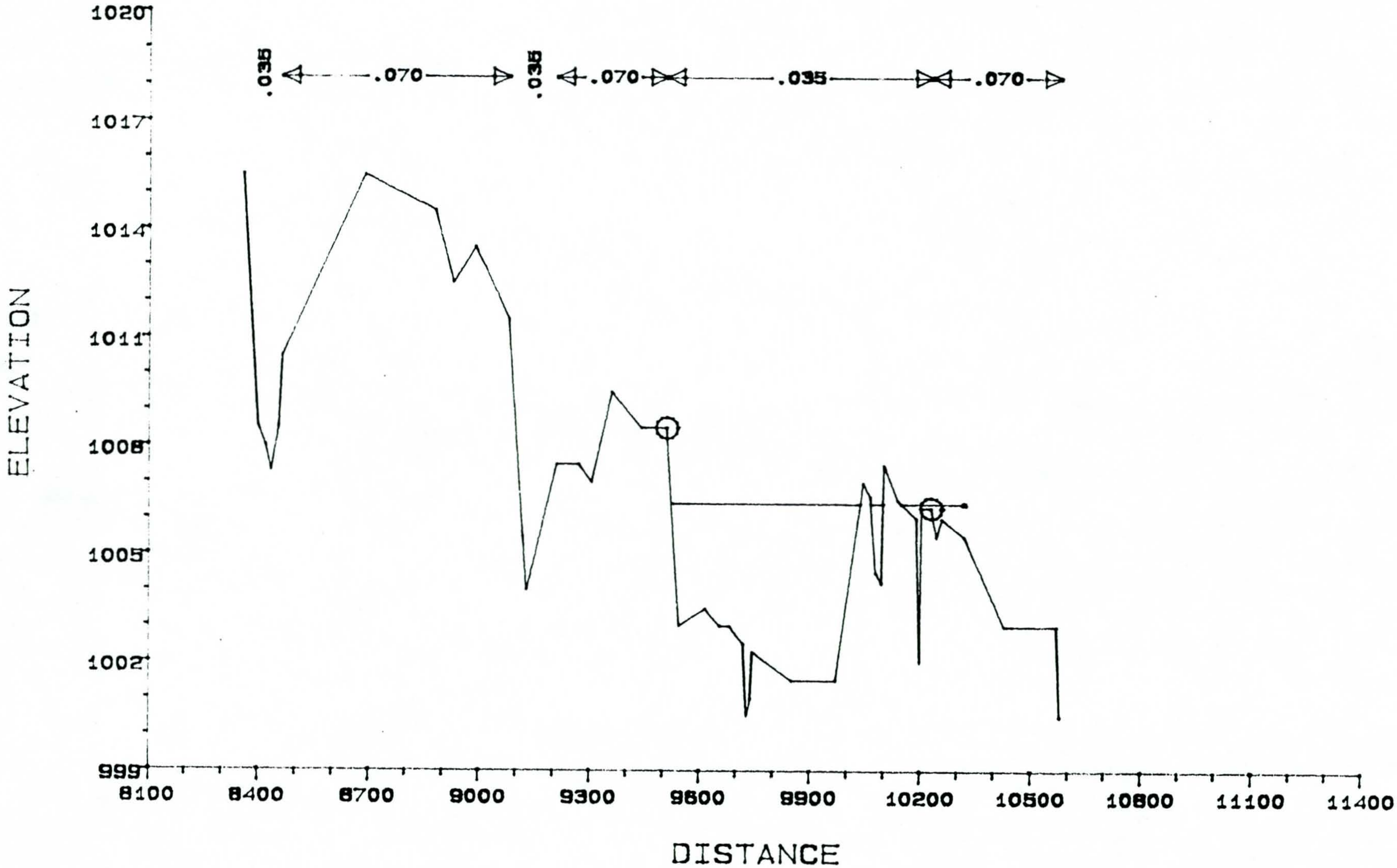
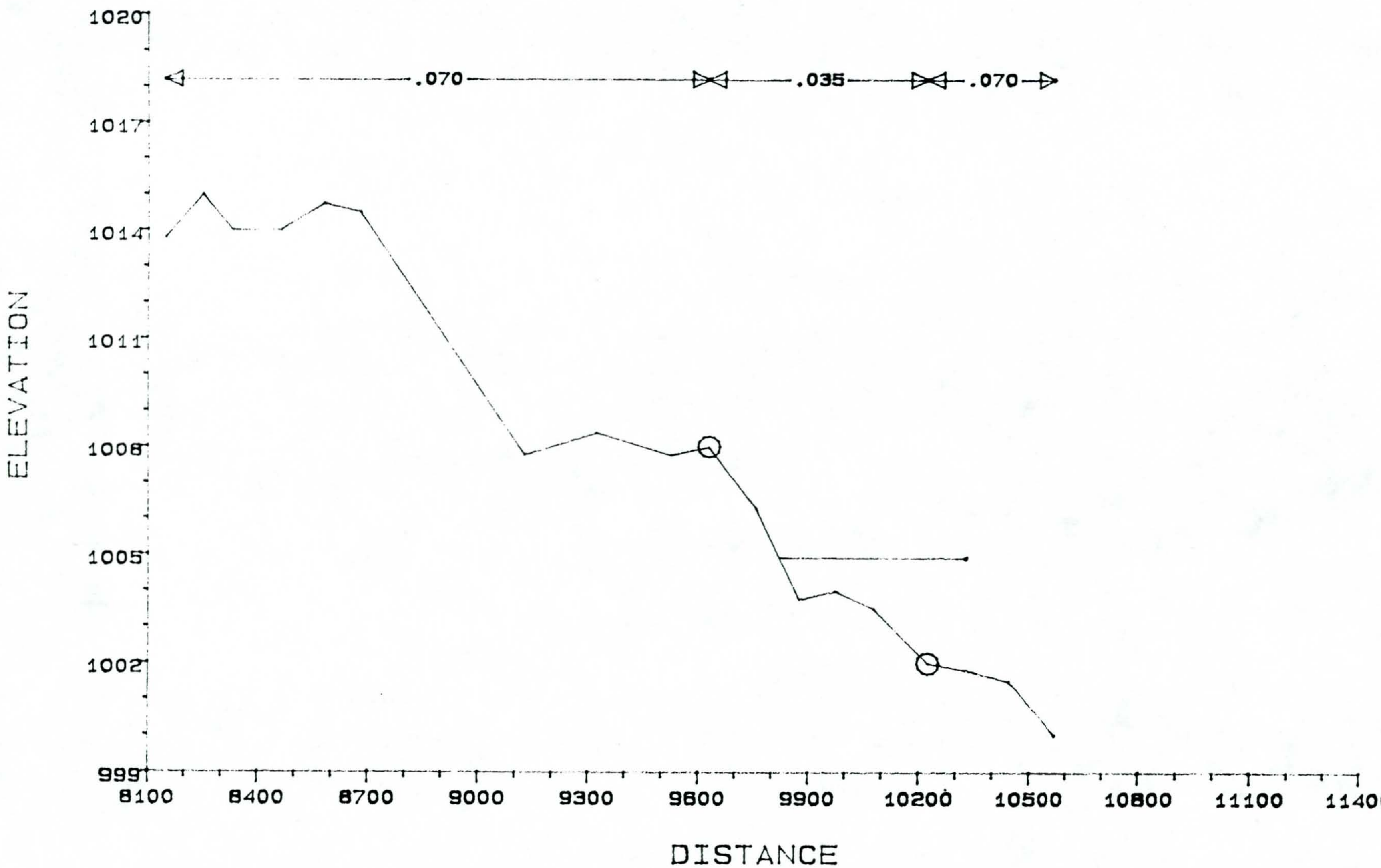


Figure D- 9

WEEKES WASH BREAKOUT AREA
BETWEEN SCENIC AND JUNCTION DRIVE.
CROSS-SECTION NO. 6.000



APPENDIX E

REVISED SUPERSTITION FREEWAY
DETENTION BASIN RATING CURVES

APPENDIX E-I

NORTH DIVERSION BASIN

Table E-1

North Diversion Detention Basin
Storage Calculations

Elevation	Reading	Acres	Ave. End Area	Depth (ft)	*Vol. (ac-ft)
23.5	-				
24	32.0	2.26	1.13	.5	.59
26	76.6	5.41	3.84	2	8.26
28	87.4	6.17	4.22	4	17.46
30	98.5	6.96	4.61	6	28.24
32	109.4	7.73	4.43	8	36.02
34	120.0	8.48	4.80	10	48.62

* The assumed bottom for calculations is 24.00, therefore the storage between 23.5 and 24 (which is .59) is added to all other volumes.

Planimeter arm set at 30.00

factor = .0706

Acres = Reading x factor

$$\text{Average-End Area} = \frac{\text{top area (acres) - bottom area (acres)}}{2}$$

Table E-2.

Flow for the 48 Inch Outlet Pipe
from the North Diversion Detention Basin

```
*********************
*  CULVERT PROGRAM  *
*********************

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT     NO. : POWERLINE     DATE : 03-28-1980   TIME : 08:38:44
PROJECT   NAME : N.DIVERSION DETENTION
COMPUTED    BY : VAR                      CHK BY :
**********************************************************************

      HYDRAULIC ANALYSIS OF  1 -  4.0 FT. X  92.00 FT.  P.C.
                                n = 0.0150

                          OUTLET CONTROL GOVERNS

                         HEADWATER   COMPUTATION
```

INLET CONT.			OUTLET CONTROL HW = H+ho-LSo							
Q	HW/D	HW	Ke	H	Dc	(Dc+D)/2	TW	ho	LSo	HW
31.00	0.58	2.32	0.20	0.17	1.65	2.82	1.80	2.82	0.50	2.50

CONTROL HW= 2.50   OUTLET VEL= 5.65   DHW= 102.50   AHW=   36.00   DN= 1.61

(Reference 5)

Table E-3

Flows for the 48 Inch Outlet Pipe
from the North Diversion Detention Basin

```
**********************
*  CULVERT PROGRAM   *
**********************


FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT     NO. : POWERLINE     DATE : 03-28-1980   TIME : 08:38:27
PROJECT    NAME : N.DIVERSION DETENTION
COMPUTED     BY : VAR                    CHK BY :
***************************************************************************


    HYDRAULIC ANALYSIS OF   1 -  4.0 FT. X  92.00 FT.  P.C.
                                n = 0.0150


                          INLET CONTROL GOVERNS


                        HEADWATER   COMPUTATION

      INLET CONT.                   OUTLET CONTROL HW = H+ho-LSo
   Q      HW/D   HW      Ke     H     Dc   (Dc+D)/2  TW     ho     LSo    HW
------- ------ ----    ---- ----- ----- ------   ----- ----- ----- ------

  93.00  1.13  4.51   0.20  1.53  2.92   3.46   1.80  3.46  0.50   4.50


CONTROL HW= 4.51  OUTLET VEL= 9.45  DHW= 104.51  AHW=  36.00  DN= 3.33
```


(Reference 5)

Table E-4

Flows for the 48 Inch Outlet Pipe
from the North Diversion Detention Basin

****
* CULVERT PROGRAM *
****

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT NO. : POWERLINE   DATE : 03-28-1980   TIME : 08:38:12
PROJECT NAME : N.DIVERSION DETENTION
COMPUTED BY : VAR   CHK BY :

****

HYDRAULIC ANALYSIS OF 1 - 4.0 FT. X 92.00 FT. P.C.
n = 0.0150

OUTLET CONTROL GOVERNS

HEADWATER COMPUTATION

	INLET CONT.				OUTLET CONTROL HW = H+ho-LSo					
Q	HW/D	HW	Ke	H	Dc	(Dc+D)/2	TW	ho	LSo	HW
136.00	1.61	6.46	0.20	3.28	3.47	3.74	1.80	3.74	0.50	6.52

CONTROL HW= 6.52 OUTLET VEL=11.74 DHW= 106.52 AHW= 36.00 DN= 4.00

  (Reference 5)

Table E-5

Flows for the 48 Inch Outlet Pipe
from the North Diversion Detention Basin

```
*********************
*  CULVERT PROGRAM  *
*********************
```

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

```
PROJECT     NO. : POWERLINE     DATE : 03-28-1980   TIME : 08:37:49
PROJECT    NAME : N.DIVERSION DETENTION
COMPUTED     BY : VAR                       CHK BY :
```

```
*************************************************************************
```

HYDRAULIC ANALYSIS OF 1 - 4.0 FT. X 92.00 FT. P.C.
n = 0.0150

INLET CONTROL GOVERNS

HEADWATER COMPUTATION

INLET CONT.			OUTLET CONTROL HW = H+ho-LSo							
Q	HW/D	HW	Ke	H	Dc	(Dc+D)/2	TW	ho	LSo	HW
169.00	2.13	8.50	0.20	5.06	3.72	3.86	1.80	3.86	0.50	8.43

CONTROL HW= 8.50 OUTLET VEL=13.87 DHW= 108.50 AHW= 36.00 DN= 4.00

Table E-6

Flows for the 48 Inch outlet Pipe
from the North Diversion Detention Basin

****
* CULVERT PROGRAM *
****

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT NO. : POWERLINE DATE : 03-28-1980 TIME : 08:37:29
PROJECT NAME : N.DIVERSION DETENTION
COMPUTED BY : VAR CHK BY :

****

HYDRAULIC ANALYSIS OF 1 - 4.0 FT. X 92.00 FT. P.C.
n = 0.0150

INLET CONTROL GOVERNS

HEADWATER COMPUTATION

	INLET CONT.				OUTLET CONTROL HW = H+ho-LSo					
Q	HW/D	HW	Ke	H	Dc	(Dc+D)/2	TW	ho	LSo	HW
208.00	2.87	11.49	0.20	7.67	3.87	3.93	1.80	3.93	0.50	11.11

CONTROL HW=11.49 OUTLET VEL=16.72 DHW= 111.49 AHW= 36.00 DN= 4.00

 (Reference 5)

Table E-7

Comparison of Rating Curve Values
for
the North Diversion Basin
at
the Proposed Superstition Freeway
(48 inch Outlet Pipe)

A-N WEST			TUDOR*			FCD*		
SV	SE	SQ	SV	SE	SQ	SV	SE	SQ
0	28	0	0	23.5	0	0	23.5	0
8.5	30	0	.33	24	3.80	.6	24	0
18.1	32	0	9.04	26	.82	8.3	26	31
28.4	34	0	21.29	28	6.84	17.5	28	93
40.0	36	33	29.31	29.21	106.80	28.2	30	135
52.7	38	65				** 36.0	32	169
						** 48.6	34	208

* Both Tudor and FCD took into account tailwater conditions from the North Diversion Channel when developing the outflow through the 48 inch pipe.

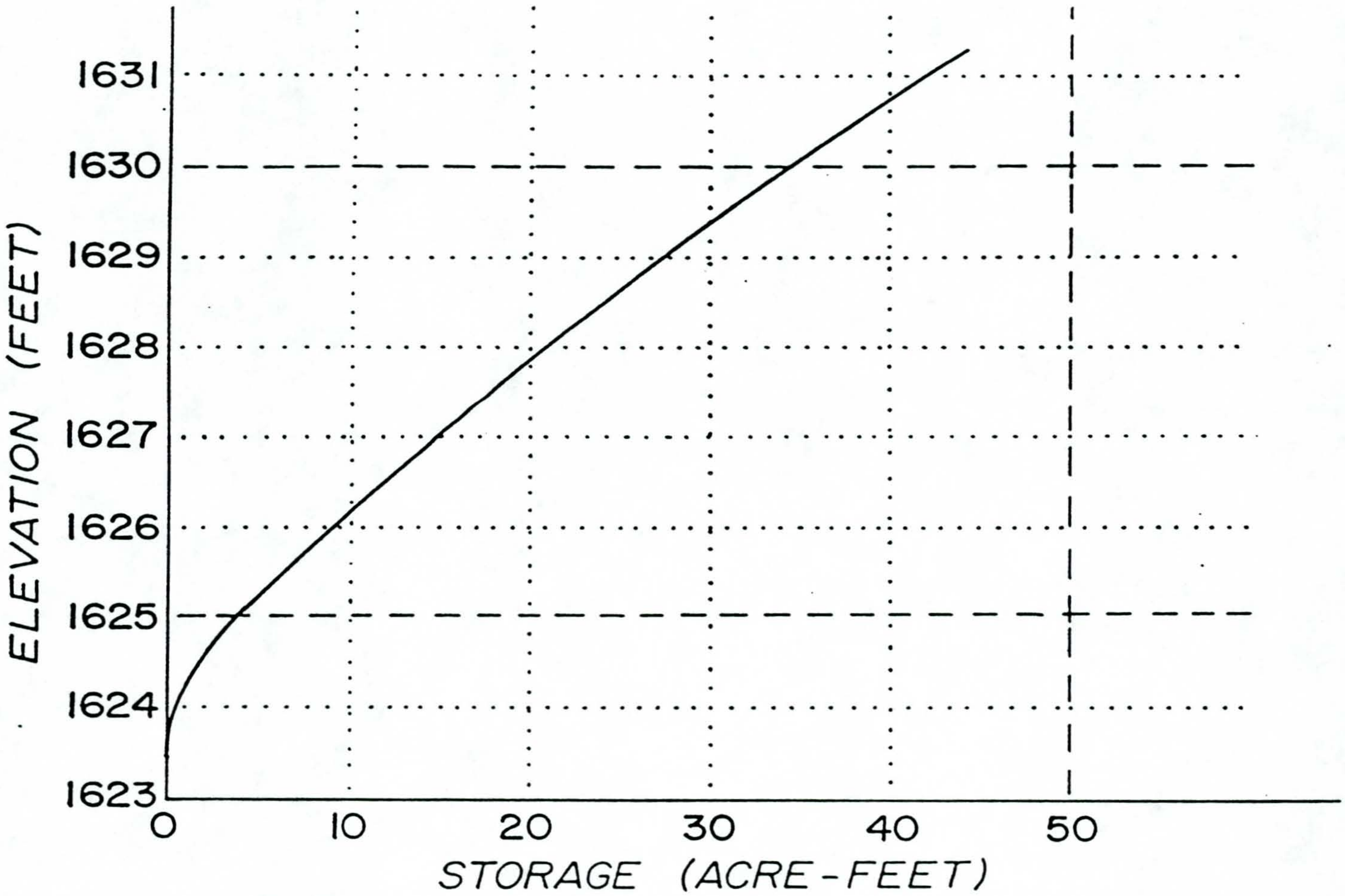
** These values were estimated for flows that would overtop the basin. The spillway elevation for the the basin is 1630.0.

Figure E-1

Storage-Elevation Curve

for the North Diversion Detention Basin at Superstition Freeway

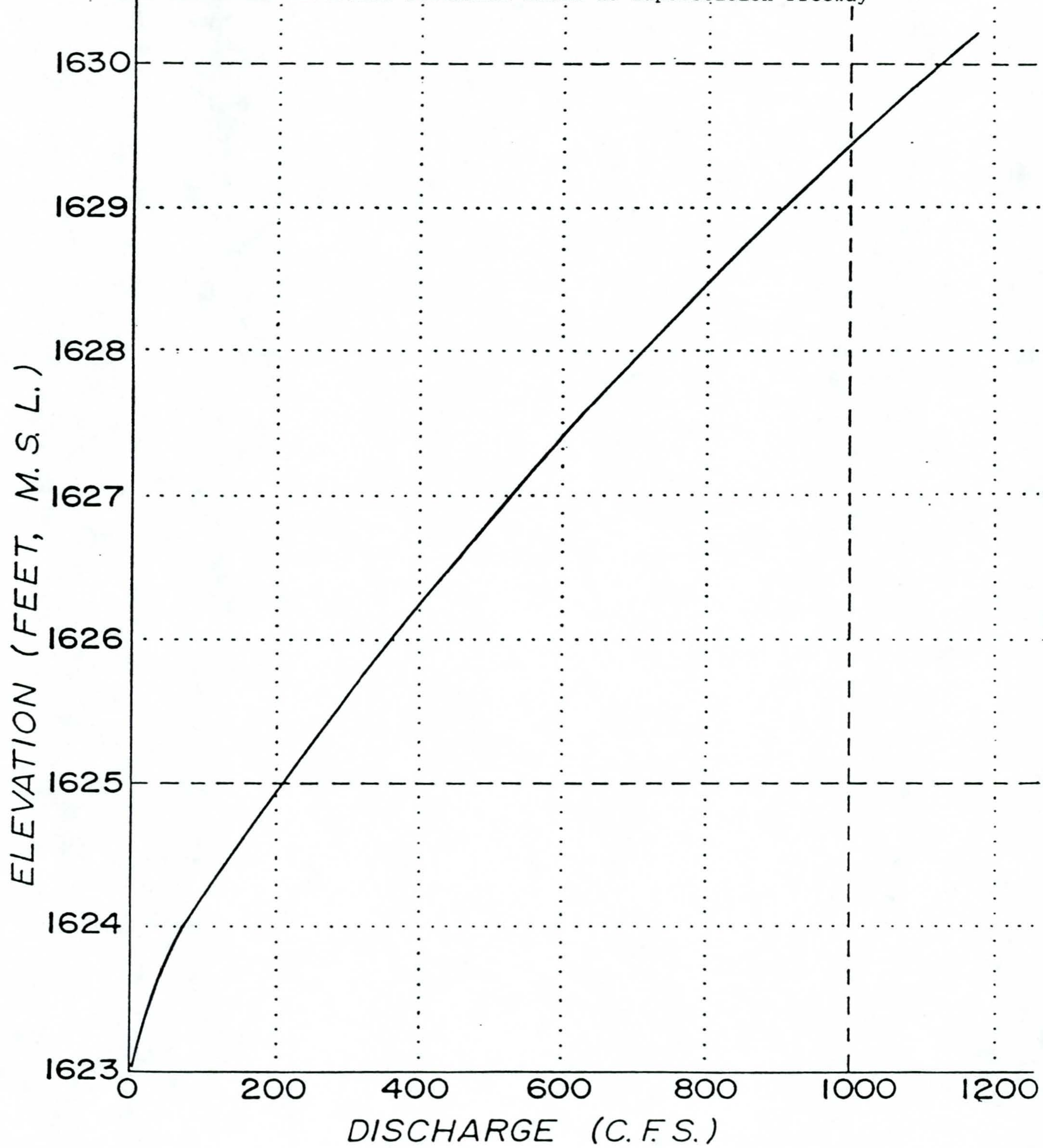
(Reference 3)



SUPERSTITION FREEWAY PROJECT

NORTH DIVERSION DAM DETENTION BASIN
STORAGE vs ELEVATION CURVE

Figure E- 2
Discharge-Elevation Curve
for the North Diversion Detention Basin at Superstition Freeway



(Reference 3)

SUPERSTITION FREEWAY PROJECT
NORTH DIVERSION DAM DRAINAGE CHANNEL STAGE-DISCHARGE CURVE AT DETENTION BASIN OUTLET

SUPERSTITION FREEWAY PROJECT
JOB NO: 1154.01
NORTH DIVERSION DAM DETENTION BASIN
100-YEAR FLOOD ROUTING
48-INCH DIAMETER OUTLET PIPE
100-YEAR IN-CHANNEL HYDROGRAPH
RUN DATE : 08-24-1987

Table E-8 (Reference 3)

A-N West Calculations of Flows from the North Diversion Detention Basin through the 48 Inch Outlet Pipe at Superstition Freeway

TIME (HR)	ACCUM. INFLOW (AC-FT)	ASSUME Q_{AVE} FOR DT (CFS)	ASSUME ACCUM. OUTFLOW (AC-FT)	BASIN STORAGE VOLUME (AC-FT)	BASIN SURFACE ELEVATION (FEET)	TAILWATER DISCHARGE (CFS)	TAILWATER SURFACE ELEVATION (FEET)	BASIN INST. (CFS)	OUTFLOW AVE (CFS)	COMPUTE OUTFLOW FOR DT (AC-FT)	OUTFLOW TOTAL (AC-FT)
8.00	0.00	0.0	0.00	0.00	1623.51	1	1623.02	0.1	0.0	0.00	0.00
8.20	0.02	0.1	0.00	0.02	1623.53	2	1623.03	0.2	0.2	0.00	0.00
8.40	0.05	0.2	0.00	0.04	1623.57	4	1623.05	0.6	0.4	0.01	0.01
8.60	0.09	0.6	0.01	0.07	1623.61	5	1623.07	0.9	0.8	0.01	0.02
8.80	0.14	1.2	0.03	0.10	1623.66	6	1623.09	1.3	1.1	0.02	0.04
9.00	0.20	1.6	0.06	0.14	1623.72	9	1623.12	1.7	1.5	0.03	0.07
9.20	0.27	2.0	0.09	0.18	1623.77	10	1623.15	2.2	2.0	0.03	0.10
9.40	0.36	2.5	0.13	0.22	1623.85	13	1623.18	2.8	2.5	0.04	0.14
9.60	0.46	3.2	0.19	0.27	1623.92	14	1623.21	3.5	3.1	0.05	0.19
9.80	0.58	3.8	0.25	0.33	1624.00	18	1623.26	4.1	3.8	0.06	0.25
10.00	0.71	4.1	0.32	0.40	1624.02	20	1623.29	4.3	4.2	0.07	0.32
10.20	0.88	4.3	0.39	0.49	1624.05	23	1623.32	4.4	4.3	0.07	0.40
10.40	1.08	4.6	0.46	0.62	1624.09	27	1623.38	4.7	4.6	0.08	0.47
10.60	1.31	4.9	0.54	0.77	1624.13	31	1623.44	5.1	4.9	0.08	0.55
10.80	1.59	5.3	0.63	0.96	1624.19	37	1623.54	5.5	5.3	0.09	0.64
11.00	1.94	5.8	0.73	1.21	1624.26	45	1623.64	6.1	5.8	0.10	0.73
11.20	2.36	6.4	0.83	1.52	1624.35	54	1623.77	6.7	6.4	0.11	0.84
11.40	2.90	7.2	0.95	1.95	1624.47	70	1624.00	7.7	7.2	0.12	0.96
11.60	3.64	8.4	1.09	2.55	1624.65	98	1624.19	9.1	8.4	0.14	1.10
11.80	5.71	4.5	1.17	4.55	1625.15	261	1625.26	0.0	4.5	0.07	1.17
12.00	11.21	0.0	1.17	10.04	1626.19	612	1627.33	0.0	0.0	0.00	1.17
12.20	19.33	0.0	1.17	18.16	1627.52	859	1628.66	0.0	0.0	0.00	1.17
12.40	26.16	14.3	1.40	24.75	1628.53	766	1628.15	28.9	14.4	0.24	1.41
12.60	30.36	58.7	2.37	27.99	1629.02	590	1627.21	86.9	57.9	0.96	2.37
12.80	33.14	96.9	3.97	29.17	1629.19	454	1626.45	106.3	96.6	1.60	3.97
13.00	35.05	106.3	5.73	29.31	1629.21	354	1625.86	107.3	106.8	1.77	5.73
13.20	36.48	107.3	7.50	28.97	1629.16	292	1625.47	106.7	107.0	1.77	7.50
13.40	37.64	106.1	9.26	28.38	1629.07	253	1625.21	105.5	106.1	1.75	9.25
13.60	38.61	104.4	10.98	27.62	1628.96	226	1625.04	103.4	104.5	1.73	10.98
13.80	39.48	101.1	12.66	26.82	1628.84	204	1624.89	98.8	101.1	1.67	12.65
14.00	40.24	97.6	14.27	25.97	1628.71	188	1624.79	96.5	97.6	1.61	14.27
14.20	40.95	95.1	15.84	25.11	1628.58	178	1624.72	93.9	95.2	1.57	15.84
14.40	41.62	92.6	17.37	24.24	1628.45	169	1624.66	91.3	92.6	1.53	17.37
14.60	42.24	90.1	18.86	23.38	1628.33	163	1624.62	88.9	90.1	1.49	18.86
14.80	42.84	87.6	20.31	22.53	1628.20	156	1624.58	86.4	87.6	1.45	20.31
15.00	43.40	85.1	21.71	21.68	1628.07	150	1624.53	83.8	85.1	1.41	21.71
15.20	43.53	82.1	23.07	20.46	1627.88	80	1624.07	80.4	82.1	1.36	23.07
15.40	43.53	78.0	24.36	19.17	1627.68	76	1624.04	75.8	78.1	1.29	24.36
15.60	43.53	73.3	25.57	17.96	1627.49	71	1624.01	71.0	73.4	1.21	25.58
15.80	43.53	68.7	26.71	16.82	1627.31	67	1623.95	66.6	68.8	1.14	26.71
16.00	43.53	64.4	27.77	15.75	1627.15	62	1623.89	62.4	64.5	1.07	27.78
16.20	43.53	60.5	28.77	14.76	1626.99	59	1623.84	58.7	60.6	1.00	28.78
16.40	43.53	56.7	29.71	13.82	1626.83	55	1623.78	54.7	56.7	0.94	29.72
16.60	43.53	52.8	30.58	12.95	1626.68	51	1623.73	51.0	52.9	0.87	30.59
16.80	43.53	49.4	31.40	12.13	1626.54	48	1623.68	47.6	49.3	0.82	31.41
17.00	43.53	46.0	32.16	11.37	1626.41	45	1623.64	44.6	46.1	0.76	32.17
17.20	43.53	43.0	32.87	10.66	1626.29	42	1623.59	41.6	43.1	0.71	32.88
17.40	43.53	40.2	33.54	9.99	1626.18	39	1623.55	38.8	40.2	0.66	33.55

Table E-8 (continued)

SUPERSTITION FREEWAY PROJECT
JOB NO: 1154.01
NORTH DIVERSION DAM DETENTION BASIN
100-YEAR FLOOD ROUTING
48-INCH DIAMETER OUTLET PIPE
100-YEAR IN-CHANNEL HYDROGRAPH
RUN DATE : 08-24-1987

TIME (HR)	ACCUM. INFLOW (AC-FT)	ASSUME Q_{AVE} FOR DT (CFS)	ASSUME ACCUM. OUTFLOW (AC-FT)	BASIN STORAGE VOLUME (AC-FT)	BASIN SURFACE ELEVATION (FEET)	TAILWATER DISCHARGE (CFS)	TAILWATER SURFACE ELEVATION (FEET)	BASIN INST. (CFS)	OUTFLOW AVE (CFS)	COMPUTE OUTFLOW FOR DT (AC-FT)	OUTFLOW TOTAL (AC-FT)
17.60	43.53	37.4	34.15	9.37	1626.08	36	1623.52	36.1	37.4	0.62	34.16
17.80	43.53	35.0	34.73	8.80	1625.97	34	1623.48	33.8	35.0	0.58	34.74
18.00	43.53	32.6	35.27	8.26	1625.87	31	1623.45	31.2	32.5	0.54	35.28
18.20	43.53	30.0	35.77	7.76	1625.77	29	1623.41	28.8	30.0	0.50	35.78
18.40	43.53	28.8	36.24	7.28	1625.68	27	1623.39	26.5	27.7	0.46	36.23
18.60	43.53	25.5	36.67	6.86	1625.60	25	1623.35	24.5	25.5	0.42	36.66
18.80	43.53	23.5	37.05	6.47	1625.52	23	1623.32	22.6	23.6	0.39	37.05
19.00	43.53	21.7	37.41	6.12	1625.46	21	1623.30	20.9	21.8	0.36	37.41
19.20	43.53	20.3	37.75	5.78	1625.39	20	1623.28	19.7	20.3	0.34	37.74
19.40	43.53	19.3	38.07	5.46	1625.33	19	1623.27	18.8	19.3	0.32	38.06
19.60	43.53	18.4	38.37	5.16	1625.27	18	1623.26	18.0	18.4	0.30	38.36
19.80	43.53	17.6	38.66	4.87	1625.21	17	1623.25	17.2	17.6	0.29	38.66
20.00	43.53	16.8	38.94	4.59	1625.16	16	1623.23	16.4	16.8	0.28	38.93
20.20	43.53	16.1	39.21	4.32	1625.11	16	1623.22	15.7	16.1	0.27	39.20
20.40	43.53	15.3	39.46	4.07	1625.06	15	1623.21	15.0	15.3	0.25	39.45
20.60	43.53	14.7	39.70	3.83	1625.01	14	1623.20	14.3	14.7	0.24	39.69
20.80	43.53	13.9	39.93	3.60	1624.95	13	1623.19	13.5	13.9	0.23	39.92
21.00	43.53	13.0	40.15	3.38	1624.89	13	1623.18	12.6	13.0	0.22	40.14
21.20	43.53	12.1	40.35	3.18	1624.83	12	1623.17	11.7	12.2	0.20	40.34
21.40	43.53	11.3	40.53	2.99	1624.78	11	1623.16	11.0	11.4	0.19	40.53
21.60	43.53	10.6	40.71	2.82	1624.73	10	1623.15	10.2	10.6	0.18	40.70
21.80	43.53	10.2	40.88	2.65	1624.68	10	1623.14	9.7	10.0	0.16	40.87

MAXIMUM BASIN WATER SURFACE ELEVATION : 1629.21 Feet PEAK BASIN DISCHARGE : 107.3 Cfs @ 13.0 Hours
MAXIMUM TAILWATER ELEVATION : 1628.66 Feet PEAK TAILWATER DISCHARGE : 859.0 Cfs @ 12.2 Hours

**** BASIN OPERATION CYCLE = 13.80 Hours ****

NOTES:
1) Basin Inflow Hydrograph : FILE "NDBA100"
2) Basin Storage-Elevation Data : FILE "NDBASIN"
3) Tailwater Inflow Hydrograph : FILE "NDCH100"
4) Tailwater Stage-Discharge Data : FILE "NDCHAN"
5) Spillway Discharge Data : FILE "NDSPILL"
6) OUTLET CONDUIT : 48" Diam Pipe
Inlet Control Headwater-Discharge Data : FILE "48INRCP"
Conduit Length = 102 Inlet Invert = 1623.5 Outlet Invert = 1623
Headloss Coefficient [k] =0.0002114; where $H_{Loss} = k * Q^2$
Full Flow Discharge Coefficient [C] = 68.80; where $Q_{Outlet} = C * H_{eff}^{0.5}$
Maximum Flapgate Loss (Submerged) = .2064 Ft

APPENDIX E-II

NORTH DIVERSION CHANNEL

Table E-9

North Diversion Structure (Channel)
Storage Calculation

Elevation	Reading	Acres	Ave. End Area	Depth (ft)	*Vol.(ac-ft)
22.6	-				
23	1.4	.10	.05	.4	.02
24	2.6	.18	.14	1	.16
26	9.0	.64	.37	3	1.12
28	22.1	1.56	.83	5	4.17
30	101.3	7.16	3.63	7	25.42

* The assumed bottom for calculations is 23.00, therefore the storage between 22.6 and 24 (which is .02) is added to all other volumes.

Planimeter arm set at 30.00

factor = .0706

Acres = Reading x factor

$$\text{Average-End Area} = \frac{\text{top area (acres)} - \text{bottom area (acres)}}{2}$$

Table E-10

Flow Through the Two 10 x 6 Foot Box Culverts

at the North Diversion under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1. No of Barrels	=	2.0000
2. Width of Box (ft)	=	10.0000
3. Depth of Box (ft)	=	6.0000
4. Discharge (cfs)	=	101.0000
5. Tailwater (ft)	=	0.5000
6. Entrance Loss Coefficient	=	0.4000
7. Slope of Box (ft/ft)	=	0.0038
8. Length of Box (ft)	=	156.0000
9. Inlet Invert Elev.	=	22.6000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 10.00 X 6.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.23	1.40	0.40	0.02	0.93	3.46	0.50	3.46	0.59	2.89

CONTROL HW = 2.89 OUTLET VEL. = 5.46 DHW = 25.49 DN = 0.83

 (Reference 5)

Table E-11

Flows Through the Two 10 x 6 Foot Box Culverts
at the North Diversion under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1.	No of Barrels	=	2.0000
2.	Width of Box (ft)	=	10.0000
3.	Depth of Box (ft)	=	6.0000
4.	Discharge (cfs)	=	372.0000
5.	Tailwater (ft)	=	0.5000
6.	Entrance Loss Coefficient	=	0.4000
7.	Slope of Box (ft/ft)	=	0.0038
8.	Length of Box (ft)	=	156.0000
9.	Inlet Invert Elev.	=	22.6000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 10.00 X 6.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.57	3.40	0.40	0.25	2.21	4.10	0.50	4.10	0.59	3.76

CONTROL HW = 3.76 OUTLET VEL. = 8.43 DHW = 26.36 DN = 1.93

(Reference 5)

Table E-12

Flows Through the Two 10 x 6 Foot Box Culverts

at the North Diversion under the Proposed Supstition Freeway

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**************************
*  BOX CULVERT PROGRAM   *
**************************
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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

```
PROJECT     NO. : N.DIVERSION  DATE : 03-28-1980   TIME : 08:26:46
PROJECT   NAME : POWERLINE F.R.S
COMPUTED     BY : VAR                    CHK BY :
```

****

HYDRAULIC ANALYSIS OF A 2 - 10.00 X 6.00 X 156.00 C.B.C.

INLET CONTROL GOVERNS

INLET CONT.			OUTLET CONTROL HW = H+ho-LSo							
Q	HW/D	HW	Ke	H	Dc	(Dc+D)/2	TW	ho	LSo	HW
738.00	0.90	5.40	0.40	0.99	3.48	4.74	0.50	4.74	0.59	5.14

CONTROL HW = 5.40   OUTLET VEL. = 11.81   DHW = 28.00   DN = 3.12

 (Reference 5)

Table E- 13

Flows Through the Two 10 x 6 Foot Box Culverts
at the North Diversion under the Proposed Superstition Freeway

****
* BOX CULVERT PROGRAM *
****

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT   NO. : N.DIVERSION   DATE : 03-28-1980   TIME : 08:26:05
PROJECT   NAME : POWERLINE F.R.S
COMPUTED   BY : VAR   CHK BY :

****

HYDRAULIC ANALYSIS OF A 2 - 10.00 X 6.00 X 156.00 C.B.C.

INLET CONTROL GOVERNS

INLET CONT.			OUTLET CONTROL HW = H+ho-LSo							
Q	HW/D	HW	Ke	H	Dc	(Dc+D)/2	TW	ho	LSo	HW
1111.00	1.23	7.40	0.40	2.24	4.58	5.29	0.50	5.29	0.59	6.94

CONTROL HW = 7.40   OUTLET VEL. = 13.24   DHW = 30.00   DN = 4.20

 (Reference 5)

Table E-14

Comparison of Rating Curve Values
for
North Diversion Structure
under
the Proposed Superstition Freeway
(2 - 10 x 6 foot Box Culverts)

A-N West			FCD		
SV	SE	SQ	SV	SE	SQ
			0	22.6	0
0	23	0	.02	23	0
.42	24	75	.16	24	101
.86	25	175			
1.35	26	312	1.12	26	372
1.88	27	480			
2.45	28	696	4.17	28	738
3.06	29	1163			
3.37	29.5	1491			
3.71	30	1848	25.42	30	1111

The top of road is at an elevation of 1655.25.

APPENDIX E-III

WEEKES WASH BASIN

Table E-15

Weekes Wash Detention Basin
Storage Calculations

Elevation	Reading	Acres	Ave. End Area	Depth (ft)	Vol (ac-ft)
* 36	16.0	.86	-	-	-
37	195.4	11.33	6.09	1	6
38	210.6	12.21	6.54	2	13
40	235.0	13.63	7.25	4	29
42	255.5	14.82	7.84	6	47
44	282.7	16.40	8.63	8	69
46	306.0	17.75	9.30	10	93
48	336.2	19.50	10.18	12	122
50	362.3	21.01	10.94	14	153
52	382.2	22.17	11.51	16	184
54	400.0	23.20	12.03	18	217
55	400.0	23.20	12.03	20	239

* Assume bottom 1636.00

Planimeter arm set at 25.0

factor = .058

Acres = reading * factor

Average-End Area = $\dfrac{\text{top area (acres) - bottom area (acres)}}{2}$

Table E-16

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1. No of Barrels	=	2.0000
2. Width of Box (ft)	=	12.0000
3. Depth of Box (ft)	=	12.0000
4. Discharge (cfs)	=	22.0000
5. Tailwater (ft)	=	0.0000
6. Entrance Loss Coefficient	=	0.4000
7. Slope of Box (ft/ft)	=	0.0020
8. Length of Box (ft)	=	204.3000
9. Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.08	1.00	0.40	0.00	0.30	6.15	0.00	6.15	0.41	5.74

CONTROL HW = 5.74 OUTLET VEL. = 3.09 DHW = 1641.74 DN = 0.36

 (Reference 5)

Table E-17

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1.	No of Barrels	=	2.0000
2.	Width of Box (ft)	=	12.0000
3.	Depth of Box (ft)	=	12.0000
4.	Discharge (cfs)	=	194.0000
5.	Tailwater (ft)	=	0.0000
6.	Entrance Loss Coefficient	=	0.4000
7.	Slope of Box (ft/ft)	=	0.0020
8.	Length of Box (ft)	=	204.3000
9.	Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.17	2.00	0.40	0.01	1.27	6.63	0.00	6.63	0.41	6.24

CONTROL HW = 6.24 OUTLET VEL. = 6.38 DHW = 1642.24 DN = 1.37

Table E-18

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1.	No of Barrels	=	2.0000
2.	Width of Box (ft)	=	12.0000
3.	Depth of Box (ft)	=	12.0000
4.	Discharge (cfs)	=	584.0000
5.	Tailwater (ft)	=	0.0000
6.	Entrance Loss Coefficient	=	0.4000
7.	Slope of Box (ft/ft)	=	0.0020
8.	Length of Box (ft)	=	204.3000
9.	Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.33	4.00	0.40	0.10	2.64	7.32	0.00	7.32	0.41	7.01

CONTROL HW = 7.01 OUTLET VEL. = 9.22 DHW = 1643.01 DN = 2.84

Table E-19

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1.	No of Barrels	=	2.0000
2.	Width of Box (ft)	=	12.0000
3.	Depth of Box (ft)	=	12.0000
4.	Discharge (cfs)	=	1050.0000
5.	Tailwater (ft)	=	0.0000
6.	Entrance Loss Coefficient	=	0.4000
7.	Slope of Box (ft/ft)	=	0.0020
8.	Length of Box (ft)	=	204.3000
9.	Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.50	6.00	0.40	0.33	3.90	7.95	0.00	7.95	0.41	7.87

CONTROL HW = 7.87 OUTLET VEL. = 11.21 DHW = 1643.87 DN = 4.30

Table E-20

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1. No of Barrels	=	2.0000
2. Width of Box (ft)	=	12.0000
3. Depth of Box (ft)	=	12.0000
4. Discharge (cfs)	=	1604.0000
5. Tailwater (ft)	=	0.0000
6. Entrance Loss Coefficient	=	0.4000
7. Slope of Box (ft/ft)	=	0.0020
8. Length of Box (ft)	=	204.3000
9. Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.67	8.00	0.40	0.77	5.18	8.59	0.00	8.59	0.41	8.95

CONTROL HW = 8.95 OUTLET VEL. = 12.91 DHW = 1644.95 DN = 5.86

Table E-21

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1.	No of Barrels	=	2.0000
2.	Width of Box (ft)	=	12.0000
3.	Depth of Box (ft)	=	12.0000
4.	Discharge (cfs)	=	2236.0000
5.	Tailwater (ft)	=	0.0000
6.	Entrance Loss Coefficient	=	0.4000
7.	Slope of Box (ft/ft)	=	0.0020
8.	Length of Box (ft)	=	204.3000
9.	Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

OUTLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
0.83	10.00	0.40	1.50	6.46	9.23	0.00	9.23	0.41	10.32

CONTROL HW = 10.32 OUTLET VEL. = 14.42 DHW = 1646.32 DN = 7.54

For a Printout Hit P

Table E-22

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1. No of Barrels	=	2.0000
2. Width of Box (ft)	=	12.0000
3. Depth of Box (ft)	=	12.0000
4. Discharge (cfs)	=	2900.0000
5. Tailwater (ft)	=	0.0000
6. Entrance Loss Coefficient	=	0.4000
7. Slope of Box (ft/ft)	=	0.0020
8. Length of Box (ft)	=	204.3000
9. Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

INLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
1.00	12.00	0.40	2.52	7.68	9.84	0.00	9.84	0.41	11.95

CONTROL HW = 12.00   OUTLET VEL. = 15.73   DHW = 1648.00   DN = 9.24

For a Printout Hit P

(Reference 5)

Table E-23

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1.	No of Barrels	=	2.0000
2.	Width of Box (ft)	=	12.0000
3.	Depth of Box (ft)	=	12.0000
4.	Discharge (cfs)	=	3534.0000
5.	Tailwater (ft)	=	0.0000
6.	Entrance Loss Coefficient	=	0.4000
7.	Slope of Box (ft/ft)	=	0.0020
8.	Length of Box (ft)	=	204.3000
9.	Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

INLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
1.17	14.00	0.40	3.74	8.77	10.38	0.00	10.38	0.41	13.71

CONTROL HW = 14.00 OUTLET VEL. = 16.80 DHW = 1650.00 DN = 10.83

For a Printout Hit P

(Reference 5)

Table E-24

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway
** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1. No of Barrels	=	2.0000
2. Width of Box (ft)	=	12.0000
3. Depth of Box (ft)	=	12.0000
4. Discharge (cfs)	=	4108.0000
5. Tailwater (ft)	=	0.0000
6. Entrance Loss Coefficient	=	0.4000
7. Slope of Box (ft/ft)	=	0.0020
8. Length of Box (ft)	=	204.3000
9. Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

INLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
1.33	16.00	0.40	5.05	9.69	10.85	0.00	10.85	0.41	15.49

CONTROL HW = 16.00 OUTLET VEL. = 17.66 DHW = 1652.00 DN = 12.00

For a Printout Hit P

(Reference 5)

Table E-25

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1. No of Barrels	=	2.0000
2. Width of Box (ft)	=	12.0000
3. Depth of Box (ft)	=	12.0000
4. Discharge (cfs)	=	4622.0000
5. Tailwater (ft)	=	0.0000
6. Entrance Loss Coefficient	=	0.4000
7. Slope of Box (ft/ft)	=	0.0020
8. Length of Box (ft)	=	204.3000
9. Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

INLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
1.50	18.00	0.40	6.40	10.48	11.24	0.00	11.24	0.41	17.23

CONTROL HW = 18.00 OUTLET VEL. = 18.37 DHW = 1654.00 DN = 12.00

For a Printout Hit P

 (Reference 5)

Table E-26

Weekes Wash Detention Basin Flows
Through Two 12 x 12 Foot Box Culverts
Under the Proposed Superstition Freeway

** BOX CULVERT PROGRAM **

** 3. Revise Analysis Information **

1.	No of Barrels	=	2.0000
2.	Width of Box (ft)	=	12.0000
3.	Depth of Box (ft)	=	12.0000
4.	Discharge (cfs)	=	4860.0000
5.	Tailwater (ft)	=	0.0000
6.	Entrance Loss Coefficient	=	0.4000
7.	Slope of Box (ft/ft)	=	0.0020
8.	Length of Box (ft)	=	204.3000
9.	Inlet Invert Elev.	=	1636.0000

Select Item To Be Revised (1- 9) ===> ?

** BOX CULVERT PROGRAM **

** 5. Perform Analysis **

HYDRAULIC ANALYSIS OF A 2 - 12.00 X 12.00 C.B.C.

INLET CONTROL GOVERNS

INLET CONTROL		OUTLET CONTROL							
HW/D	HW	KE	H	DC	DC+D/2	TW	HO	LSO	HW
1.58	19.00	0.40	7.07	10.84	11.42	0.00	11.42	0.41	18.08

CONTROL HW = 19.00 OUTLET VEL. = 18.68 DHW = 1655.00 DN = 12.00

For a Printout Hit P

Table E-27

Comparison of Rating Curve Values
for
Weekes Wash Detention Basin
at
the Proposed Superstition Freeway

A-N West			Tudor			FCD		
SV	SE	SQ	SV	SE	SQ	SV	SE	SQ
0	35	0						
13.0	36	74	0	36	0	0	36	0
						6	37	22
41.1	38	383	17.73	38	226.7	13	38	194
72.1	40	823	43.96	40	477.4	29	40	584
107.6	42	1364	73.24	42	1199.2	47	42	1050
147.5	44	1988	105.40	44	1373.7	69	44	1604
190.5	46	2687	140.16	46	1943.9	93	46	2236
236.5	48	3452	177.93	48	2847.8	122	48	2900
273.3	49.5	3857						
			226.00	50	3475.0	153	50	3534
						* 184	52	4408
						* 217	54	4622
						*+ 239	55	4820

* These values were estimated for flows that would overtop the basin. The basin will start to spill over the basin at an elevation of 1650.00.

+ The elevation of the top of the road is 1655.00.

SUPERSTITION FREEWAY PROJECT
JOB NO: 1154.01
WEEKES WASH DETENTION BASIN
100-YEAR FLOOD ROUTING
2- 12' X 12' BOX CULVERT OUTLET
RUN DATE : 08-24-1987

Table E-28 (Reference 3)

A-N West Calculations of Flows from the Weekes Wash Detention Basin Through the Two 12 x 12 Foot Box Culverts

TIME (HR)	ACCUM. INFLOW (AC-FT)	ASSUME Q_{AVE} FOR DT (CFS)	ASSUME ACCUM. OUTFLOW (AC-FT)	BASIN STORAGE VOLUME (AC-FT)	BASIN SURFACE ELEVATION (FEET)	BASIN INST. (CFS)	OUTFLOW AVE (CFS)	COMPUTE OUTFLOW FOR DT (AC-FT)	OUTFLOW TOTAL (AC-FT)
7.20	0.01	0.0	0.00	0.01	1636.00	0.2	0.1	0.00	0.00
7.40	0.03	0.2	0.00	0.03	1636.01	0.7	0.5	0.01	0.01
7.60	0.07	1.1	0.02	0.05	1636.01	1.3	1.0	0.02	0.03
7.80	0.14	1.8	0.05	0.09	1636.02	2.1	1.7	0.03	0.05
8.00	0.23	2.8	0.10	0.13	1636.02	3.2	2.7	0.04	0.10
8.20	0.35	4.0	0.16	0.18	1636.03	4.4	3.8	0.06	0.16
8.40	0.50	5.4	0.25	0.25	1636.04	6.1	5.2	0.09	0.25
8.60	0.70	7.2	0.37	0.33	1636.06	8.0	7.0	0.12	0.36
8.80	0.95	9.1	0.52	0.43	1636.08	10.3	9.1	0.15	0.51
9.00	1.26	11.7	0.72	0.55	1636.10	13.2	11.8	0.19	0.71
9.20	1.64	14.8	0.96	0.68	1636.12	16.5	14.9	0.25	0.96
9.40	2.11	18.3	1.26	0.84	1636.15	20.4	18.4	0.30	1.26
9.60	2.68	22.6	1.64	1.04	1636.19	25.1	22.7	0.38	1.64
9.80	3.42	28.3	2.10	1.32	1636.24	31.8	28.4	0.47	2.11
10.00	4.40	36.1	2.70	1.69	1636.30	40.9	36.3	0.60	2.71
10.20	5.63	46.3	3.47	2.16	1636.39	52.2	46.5	0.77	3.47
10.40	7.14	58.8	4.44	2.70	1636.48	65.2	58.7	0.97	4.44
10.60	8.95	72.6	5.64	3.31	1636.59	79.9	72.6	1.20	5.64
10.80	11.12	88.6	7.10	4.02	1636.72	97.0	88.5	1.46	7.11
11.00	13.69	106.7	8.87	4.82	1636.86	116.3	106.7	1.76	8.87
11.20	16.67	126.3	10.95	5.72	1637.01	136.4	126.4	2.09	10.96
11.40	20.19	143.6	13.33	6.86	1637.11	149.4	142.9	2.36	13.32
11.60	24.50	158.8	15.95	8.55	1637.25	168.4	158.9	2.63	15.95
11.80	34.12	203.3	19.31	14.81	1637.77	238.9	203.6	3.37	19.31
12.00	53.13	312.9	24.48	28.65	1638.86	386.9	312.9	5.17	24.48
12.20	77.45	487.9	32.55	44.90	1640.07	588.2	487.5	8.06	32.54
12.40	99.62	680.1	43.79	55.83	1640.84	771.4	679.8	11.24	43.78
12.60	117.09	805.0	57.09	60.00	1641.13	839.8	805.6	13.32	57.10
12.80	132.57	850.9	71.16	61.41	1641.22	862.4	851.1	14.07	71.16
13.00	154.92	919.7	86.36	68.56	1641.70	976.6	919.5	15.20	86.36
13.20	200.02	1199.9	106.19	93.82	1643.32	1421.8	1199.2	19.82	106.18
13.40	265.99	1756.1	135.22	130.77	1645.49	2090.7	1756.2	29.03	135.21
13.60	333.73	2337.5	173.85	159.87	1647.07	2584.5	2337.6	38.64	173.85
13.80	393.21	2703.4	218.54	174.67	1647.83	2821.7	2703.1	44.68	218.53
14.00	443.53	2847.3	265.60	177.93	1648.00	2873.8	2847.8	47.07	265.60
14.20	486.00	2838.1	312.51	173.49	1647.77	2802.7	2838.2	46.91	312.51
14.40	521.95	2729.1	357.62	164.33	1647.30	2655.9	2729.3	45.11	357.62
14.60	552.59	2559.8	399.93	152.65	1646.69	2464.3	2560.1	42.32	399.94
14.80	578.98	2357.8	438.90	139.98	1646.00	2251.3	2357.8	38.97	438.91
15.00	601.58	2140.7	474.29	127.29	1645.30	2029.9	2140.6	35.38	474.29
15.20	621.47	1923.1	506.07	115.40	1644.61	1816.1	1923.0	31.79	506.08
15.40	639.07	1717.0	534.45	104.62	1643.98	1617.8	1716.9	28.38	534.46
15.60	654.69	1529.2	559.73	94.96	1643.39	1441.1	1529.4	25.28	559.74
15.80	668.50	1365.8	582.31	86.19	1642.85	1290.4	1365.8	22.57	582.31
16.00	680.98	1221.8	602.50	78.48	1642.35	1153.0	1221.7	20.19	602.51
16.20	692.38	1093.2	620.57	71.81	1641.92	1033.4	1093.2	18.07	620.58
16.40	702.63	983.9	636.83	65.80	1641.52	932.5	982.9	16.25	636.82
16.60	712.21	891.3	651.56	60.65	1641.17	850.3	891.4	14.73	651.56
16.80	721.39	815.1	665.04	56.35	1640.88	780.2	815.2	13.47	665.03

Table E-28 (continued)

SUPERSTITION FREEWAY PROJECT
JOB NO: 1154.01
WEEKES WASH DETENTION BASIN
100-YEAR FLOOD ROUTING
2- 12' X 12' BOX CULVERT OUTLET
RUN DATE : 08-24-1987

TIME (HR)	ACCUM. INFLOW (AC-FT)	ASSUME Q_{AVE} FOR DT (CFS)	ASSUME ACCUM. OUTFLOW (AC-FT)	BASIN STORAGE VOLUME (AC-FT)	BASIN SURFACE ELEVATION (FEET)	BASIN OUTFLOW INST. (CFS)	BASIN OUTFLOW AVE (CFS)	COMPUTE OUTFLOW FOR DT (AC-FT)	OUTFLOW TOTAL (AC-FT)
17.00	730.07	749.1	677.42	52.65	1640.62	718.1	749.1	12.38	677.41
17.20	738.33	691.5	688.85	49.48	1640.39	665.0	691.6	11.43	688.84
17.40	746.26	642.5	699.47	46.80	1640.20	620.0	642.5	10.62	699.46
17.60	753.87	600.9	709.40	44.47	1640.04	580.9	600.5	9.93	709.39
17.80	761.14	563.2	718.71	42.43	1639.89	545.5	563.2	9.31	718.70
18.00	768.00	528.9	727.45	40.55	1639.75	512.4	528.9	8.74	727.44
18.20	774.36	500.2	735.72	38.64	1639.61	487.9	500.1	8.27	735.71
18.40	780.31	477.4	743.61	36.71	1639.47	468.5	478.2	7.90	743.61
18.60	786.10	459.5	751.20	34.89	1639.33	450.4	459.5	7.59	751.21
18.80	788.99	429.5	758.30	30.69	1639.02	408.3	429.3	7.10	758.30
19.00	788.99	375.8	764.52	24.48	1638.54	342.9	375.6	6.21	764.51
19.20	788.99	315.6	769.73	19.26	1638.13	287.8	315.3	5.21	769.72
19.40	788.99	263.8	774.09	14.90	1637.77	239.9	263.8	4.36	774.09
19.60	788.99	219.6	777.72	11.27	1637.47	199.0	219.4	3.63	777.71
19.80	788.99	182.0	780.73	8.26	1637.22	165.1	182.0	3.01	780.72
20.00	788.99	151.0	783.23	5.76	1637.01	137.0	151.0	2.50	783.22
20.20	788.99	114.9	785.13	3.86	1636.69	93.3	115.1	1.90	785.12
20.40	788.99	77.6	786.41	2.58	1636.46	62.3	77.8	1.29	786.41
20.60	788.99	51.9	787.27	1.72	1636.31	41.6	52.0	0.86	787.26
20.80	788.99	35.0	787.84	1.15	1636.20	27.7	34.6	0.57	787.84
21.00	788.99	23.0	788.23	0.77	1636.14	18.5	23.1	0.38	788.22
21.20	788.99	14.8	788.47	0.52	1636.09	12.6	15.5	0.26	788.48
21.40	788.99	10.6	788.64	0.35	1636.06	8.4	10.5	0.17	788.65
21.60	788.99	6.7	788.76	0.24	1636.04	5.7	7.0	0.12	788.76
21.80	788.99	5.7	788.85	0.14	1636.03	3.4	4.6	0.08	788.84
22.00	788.99	2.7	788.89	0.10	1636.02	2.3	2.9	0.05	788.89

MAXIMUM BASIN WATER SURFACE ELEVATION : 1648.00 Feet PEAK BASIN DISCHARGE : 2873.8 Cfs @ 14.0 Hours

**** BASIN OPERATION CYCLE = 14.80 Hours ****

NOTES: 1) Basin Inflow Hydrograph : FILE "WKWS100"
2) Basin Storage-Elevation Data : FILE "WKBASIN"
3) Spillway Discharge Data : FILE "WKSOUT"

Figure E-5 (Reference 3)

Storage-Elevation Curve for the Weekes Wash Detention Basin at the Proposed Superstition Freeway

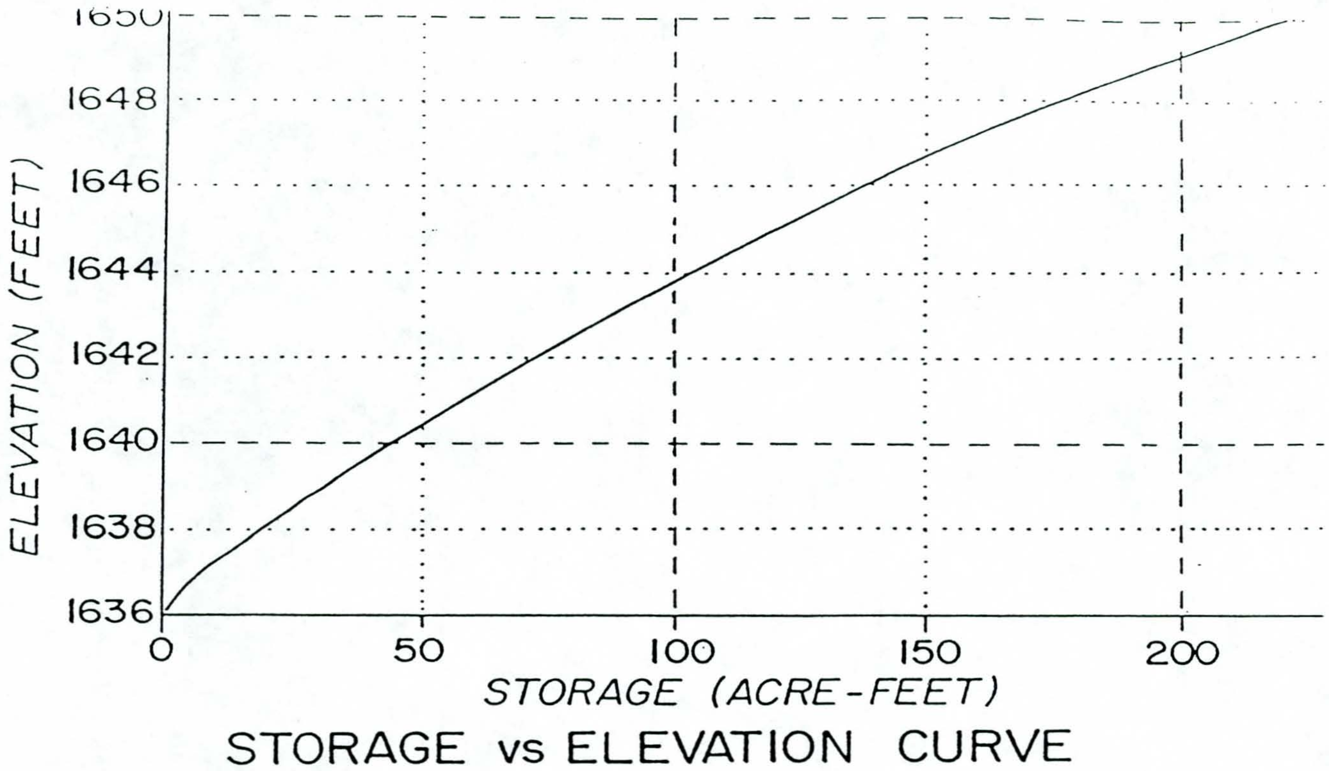
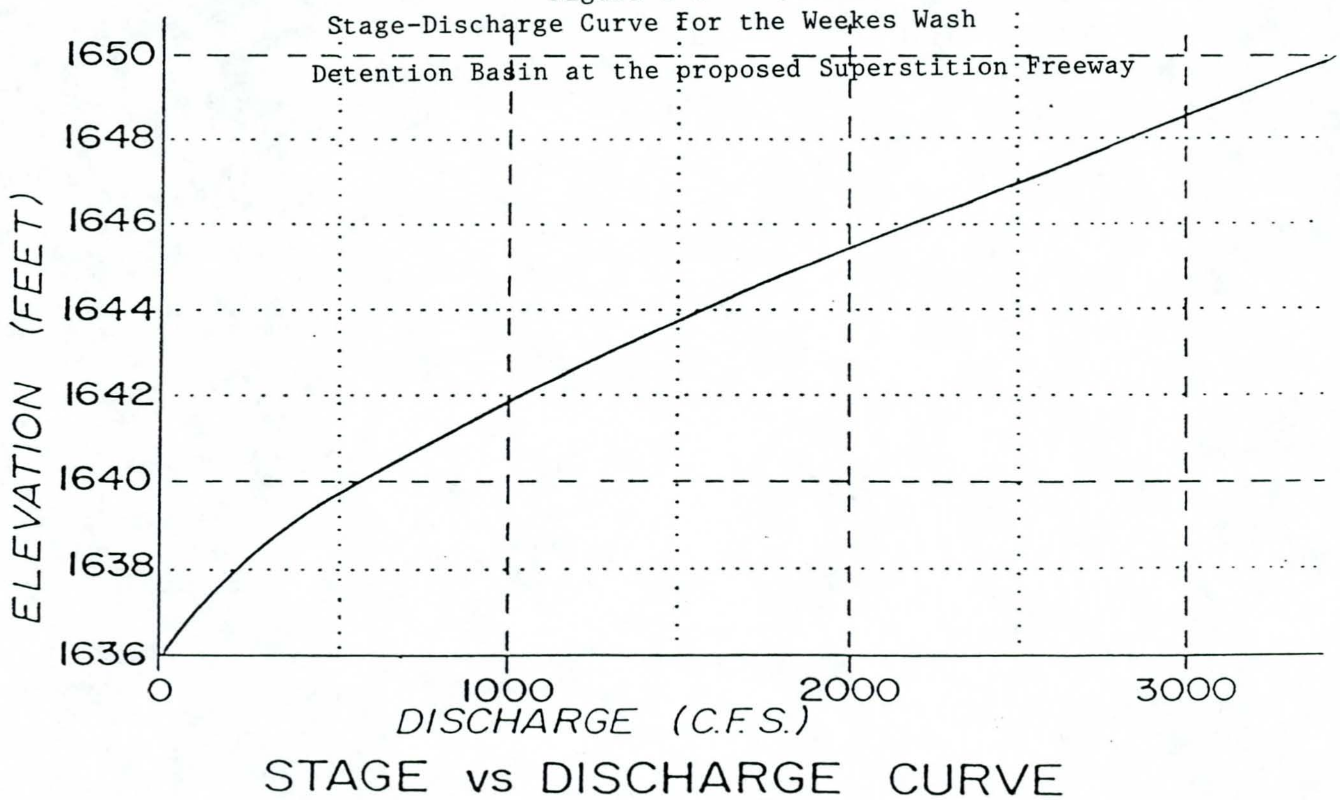


Figure E-6 (Reference 3)

Stage-Discharge Curve for the Weekes Wash Detention Basin at the proposed Superstition Freeway



SUPERSTITION FREEWAY PROJECT

WEEKES WASH DETENTION BASIN

APPENDIX F

POWERLINE EMERGENCY SPILLWAY RATING CURVES

Table F-1

Spillway Flows
for
Powerline F.R.S.

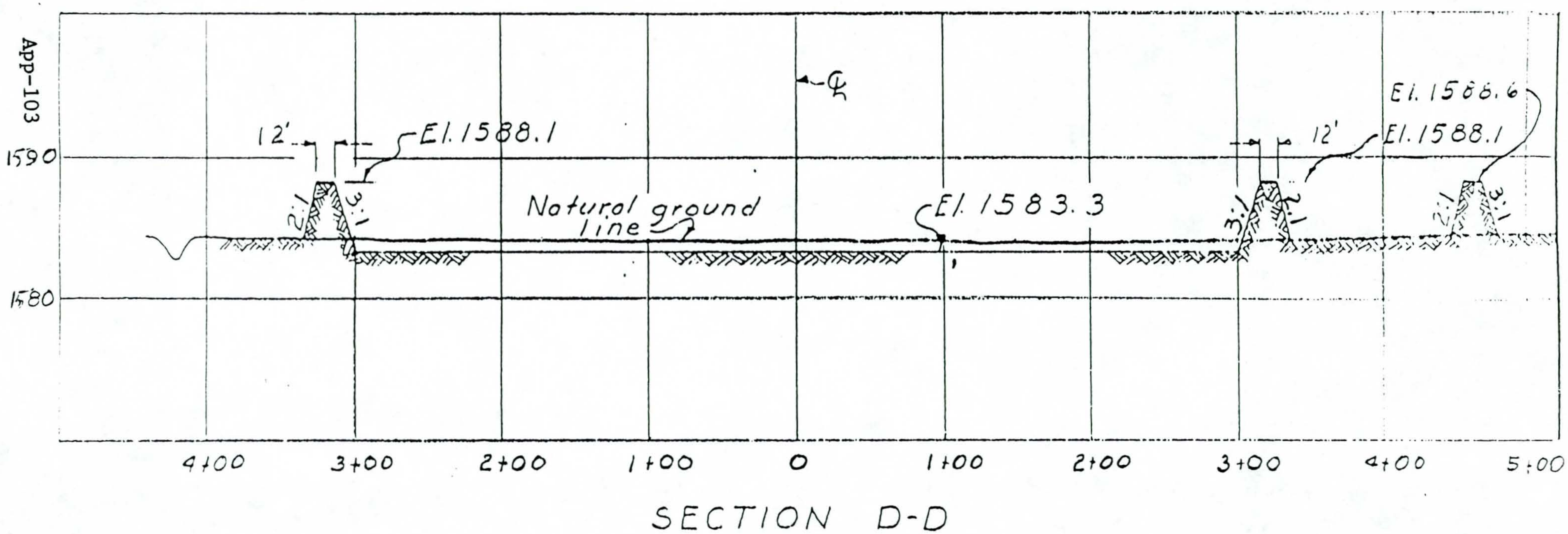
SE	Principle Spillway (cfs)	Emergency Spillway (cfs)	Combined (cfs)	Emergency Spillway (cfs)	Combined (cfs)
68.1	75	-	75	-	75
70.1	92	-	92	-	92
72.1	106	-	106	-	106
74.1	119	-	119	-	119
76.1	130	-	130	-	130
78.1	141	-	141	-	141
80.1	150	-	150	-	150
82.1	159	-	159	-	159
83.3	165	-	165	-	165
84.1	168	500	668	1,030	1,122
86.1	176	4,250	4,426	7,184	7,360
88.1	184	10,900	11,084	16,616	16,800
90.1	192	19,900	20,092	27,088	27,280

Figure F-1

Powerline F.R.S.

Emergency Spillway

Typical Channel Cross-Section



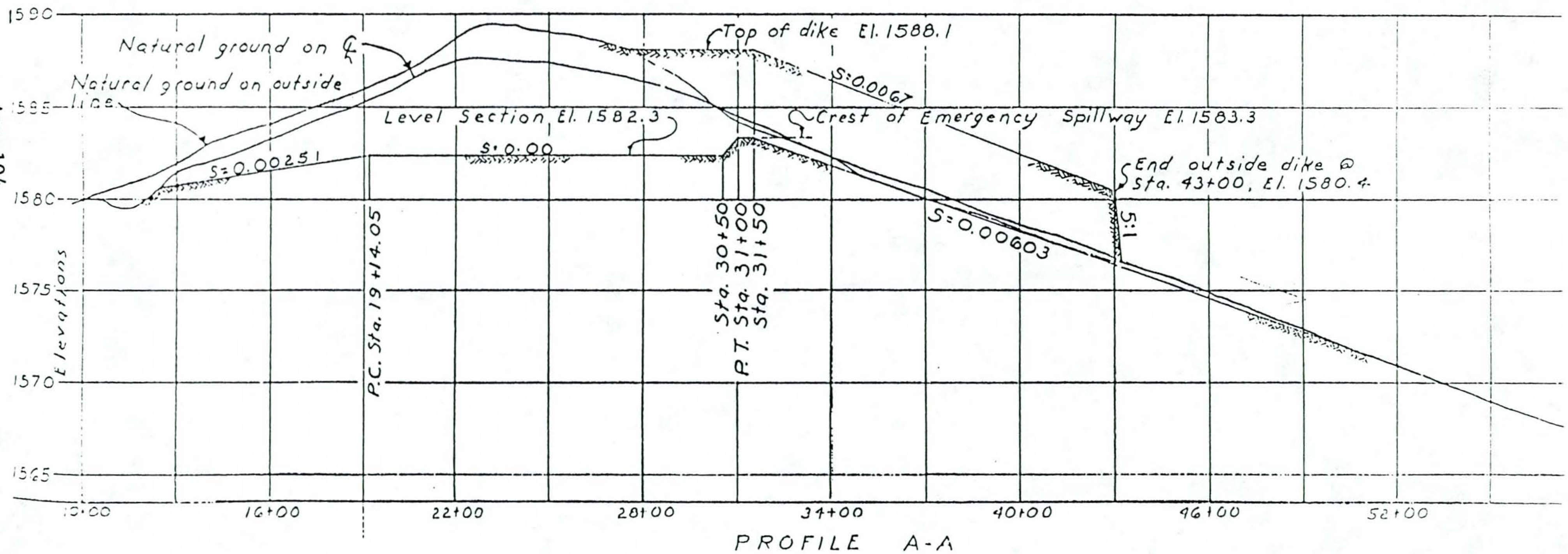
Powerline Dam, Hydrology and Hydraulics and Structure, SCS 1967.

(Reference 12)

Figure F-2

Powerline F.R.S.

Emergency Spillway Profile



Powerline Dam, Hydrology and Hydraulics and Structure Design, SCS 1967.

(Reference 12)

APPENDIX G

ARIZONA DAM SAFETY CLASSIFICATION FOR POWERLINE F.R.S.

Table G-1

DAM CLASSIFICATIONS

(Reference 6)

POWERLINE F.R.S.

I. Downstream Hazard Potential Classification:

Existing Conditions:

Significant	No urban development and no more than a small number of habitable structures.
	Appreciable economic loss (notable agriculture, industry or other structures).

Future Conditions:

High	Urban development with more than a small number of habitable structures.
	Excessive economic loss (extensive community, industry, agriculture).

II. Size Classification:

Height: (measured from the lowest elevation of the outside limit of the dam to the spillway crest or the top of the spillway gates.

Capacity: (Measured to the spillway crest or the top of the spillway gates.

CATEGORY	VALUE	RATING FACTOR
Existing Conditions:		
Height	18.2 feet	0
Capacity	4194 ac-ft	3
Possible future conditions:		
Height	>25 feet	1
Capacity	>10,000 ac-ft	5

Table G-1 (continued)

(Powerline F.R.S.)

Total Rating Factors:

Existing conditions:

Total of rating factors: 0 + 3 = 3

Size classification: Medium (3-7)

Future conditions:

Total of rating factors: 1 + 5 = 6

Size classification: Medium (3-7)

III. RECOMMENDED SPILLWAY DESIGN FLOODS

Hazard Category	Size Designation	Inflow Design Flood Magnitude
Significant (existing conditions)	Medium	1/2 PMF
High (future conditions)	Medium	1/2 PMF to PMF

STRUCTURE DATA

Top elevation of structure	1589.1
Spillway crest elevation	1583.3
Storage capacity	4194 ac-ft

DAM SAFETY CRITERIA

Total freeboard: (distance between the top of the dam and the spillway crest elevation)

top of dam	1589.1	
spillway crest	- 1583.3	
	5.8´ > 4´	therefore, total freeboard is within the limitations of Dam Safety.

APPENDIX H

POWERLINE F.R.S. - PROPOSED EXCAVATION FOR
THE INCREASE IN STORAGE (ALTERNATIVE A)

Table H-1

Powerline F.R.S.

Storage Rating Curve

(Existing vs. Proposed Excavation)

Elevation	SCS (existing) Storage (acre-feet)	Proposed Storage (acre-feet)
1568.1	0	0
1568.2	175	175
1570	380	380
1572	700	
1574	1100	1768
1576	1600	
1578	2175	3465
1580	2875	
1582	3675	5268
1583.3	4200	5941
1584	4600	
1586	5525	7196
1588	6725	
1590	7925	9465

EXECUTIVE CONTROL CARD 97 OPERATION LIST

LISTING OF DATA IN CORE

0 -POWERLINE DAM ANALYSIS, SEPT 1985

		VELOCITY INCREMENT			
1 CTABLE		.2000			
8	.0000	.0800	.1800	.2500	.3200
8	.3700	.4100	.4500	.4900	.5100
8	.5400	.5700	.5900	.6100	.6300
8	.6500	.6600	.6700	.6900	.7000
8	.7100	.7200	.7300	.7400	.7500
8	.7600	.7700	.7700	.7800	.7900
8	.7900	.8000	.8100	.8100	.8200
8	.8200	.8300	.8300	.8400	.8400
8	.8400	.8500	.8500	.8600	.8600
8	.8600	.8600	.8700	.8700	.8700
8	.8800	.8800	.8800	.8900	.8900
8	.8900	.8900	.8900	.8900	.9000
8	.9000	.9000	.9000	.9000	.9100
8	.9100	.9100	.9100	.9100	.9100
8	.9200	.9200	.9200	.9200	.9200
8	.9200	.9200	.9200	.9300	.9300
9 ENDTBL					

STRUCT NO.
3 STRUCT 1

	ELEVATION	DISCHARGE	STORAGE
8	1568.2000	.0000	175.0000
8	1570.1000	92.0000	380.0000
8	1572.1000	106.0000	700.0000
8	1574.1000	119.0000	1100.0000
8	1576.1000	130.0000	1600.0000
8	1578.1000	141.0000	2175.0000
8	1580.1000	150.0000	2875.0000
8	1582.1000	159.0000	3675.0000
8	1583.3000	165.0000	4199.9999
8	1584.1000	668.0000	4599.9999
8	1586.1000	4426.0000	5524.9999
8	1588.1000	11084.0000	6724.9999
8	1590.1000	20092.0000	7924.9999
9 ENDTBL			

STRUCT NO.
3 STRUCT 7

Table H-2

Powerline F.R.S.

Rating Curve

Weekes Wash - Powerline Dam Analysis, SCS 1985.

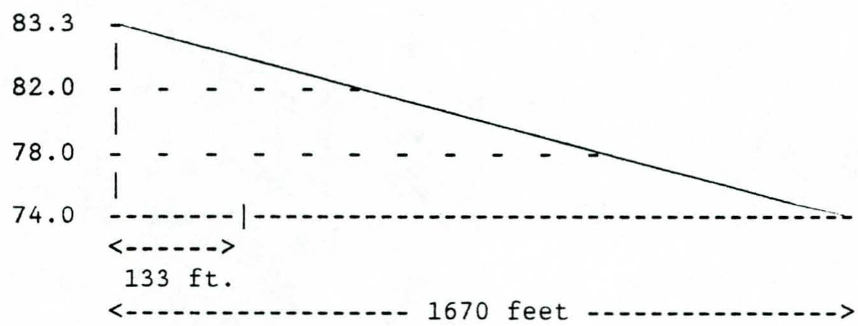
(Reference 14)

Table H-3

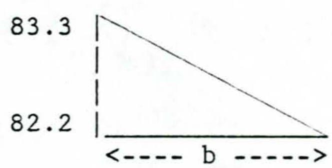
Powerline F.R.S.

Increase in Storage

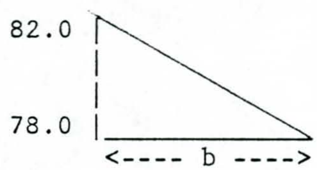
(Alternative A)



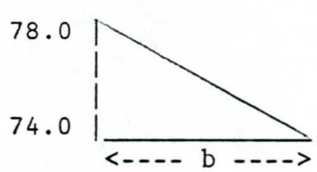
193 acres
below 1974
(196 ac-ft of
storage)



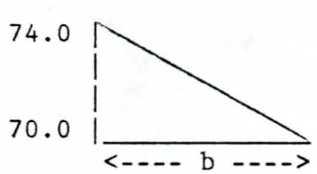
ave. b = 170 ft
length = 7200 ft
ave vol = 18.3 ac-ft



ave. b = 475 ft
length = 6900 ft
ave vol = 150.5 ac-ft



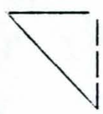
ave. b = 500 ft
length = 6300 ft
ave vol = 144.6 ac-ft



ave. b = 525 ft
length = 5600 ft
ave vol = 135 ac-ft

Table H-3 (continued)

1574 :

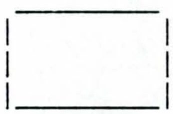


area = 80 sq. ft.

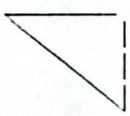
$\bar{l}$ = 5600 ft

193 acres x 4 ft	=	772
198 acres x 4 ft	=	790
80 sq.ft x 5600 ft	=	10
		196

		1768 ac-ft



1578 :

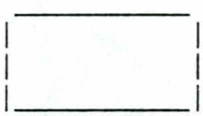


area = 320 sq.ft.

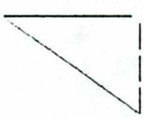
$\bar{l}$ = 5950 ft.

193 acres x 8 ft	=	1544
210 acres x 8 ft	=	1681
320 sq.ft. x 5950 ft	=	44
		196

		3465 ac-ft



1582 :



area = 720 sq.ft.

$\bar{l}$ = 6270 ft.

193 ac. x 12 ft	=	2316
221 ac. x 12 ft	=	2652
720 sq.ft. x 6270 ft.	=	104
		196

		5268 ac-ft

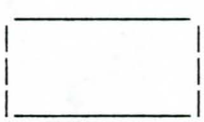


Table H-3 (continued)

1583.3 : 133 ft; 13.3 ft

area = 884 sq.ft.

$\bar{l}$ = 6500 ft

13.3 ft; 1537 ft

193 ac. x 13.3 ft =	2567
229 ac. x 13.3 ft =	3046
884 sq.ft. x 6500 ft =	132
	196
	5941 ac-ft

1586.0 : 333 ft; 2.7 ft.

area = 450 sq.ft.

$\bar{l}$ = 7600 ft.

2.7 ft; 1670 ft

450 sq.ft x 7600 ft =	79
4509 sq.ft x 7600 ft =	787
193 ac. x 16 ft =	3088
	196
	7196 ac-ft

1590.0 : 365 ft; 4 ft

area = 365 sq.ft.

$\bar{l}$ = 8800 ft.

4 ft; 1670 ft

6680 sq.ft. x 8800 ft =	1350
730 sq.ft. x 8800 ft =	147
193 ac. x 4 ft =	772
	7196
	9465 ac-ft

PLATE 1
POWERLINE F.R.S. WATERSHED