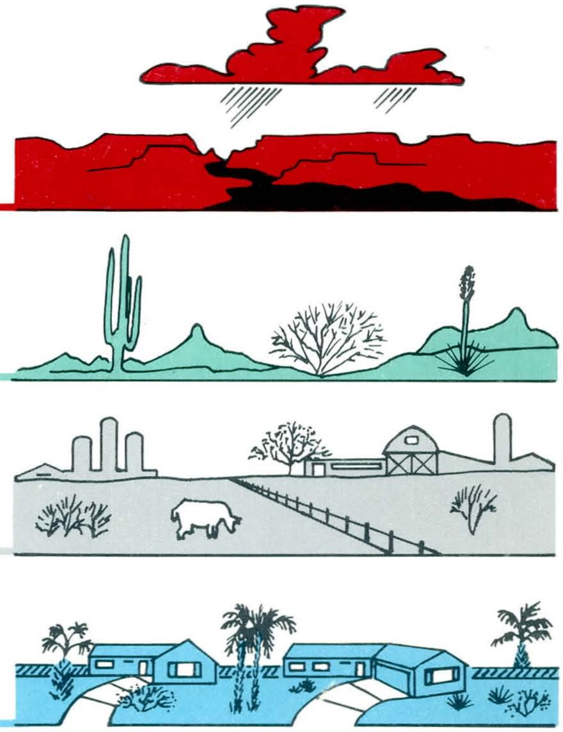

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

HYDROLOGIC AND HYDRAULIC ANALYSIS OF WHITE TANKS #4 F.R.S. SPILLWAY FOR THE 1/2 PROBABLE MAXIMUM FLOOD

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PREPARED BY:

MARICOPA COUNTY FLOOD CONTROL DISTRICT
HYDROLOGY DIVISION
WATERSHED MANAGEMENT BRANCH

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HYDROLOGIST

JULY 8, 1993

HYDROLOGIC AND HYDRAULIC ANALYSIS
OF WHITE TANKS #4 F.R.S. SPILLWAY
FOR THE 1/2 PROBABLE MAXIMUM FLOOD

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Hydraulic Modeling of White Tanks #4 F.R.S. Spillway

SPILLWAY GEOMETRY

White Tanks #4 FRS has two spillways. According to SCS and the data in the ERTEC Report, each spillway is 165 ft. wide and at an elevation of 1050.0 ft. On June 17, 1993, a survey team was sent from FCD to measure the dimensions of the two spillways. The survey notes are presented in an Annex to this report. The survey determined that the west spillway was 151.6 ft wide at the bottom of the crest and 227.2 ft wide at the top of the spillway embankment. The height of the western spillway was measured at 5.5 ft from the crest to the top of the embankment.

The east spillway was found to be 5.3 ft high from crest to top of embankment. The bottom width of the crest was measured at 130.0 ft, and the top width was 210.0 ft.

For hydraulic modeling of the spillway, an *idealized* composite spillway cross-section, representative of the *actual* cross-section, was analyzed using the US Army COE HEC-2 program for computing surface water profiles. The composite cross-section used was 281.6 ft wide at the crest, and 437.2 ft across the top of the crest embankment. These represent the sum of the east and west spillways as measured in the field. The height of the spillway was conservatively taken as 5.3 ft, even though the west spillway was measured at 5.5 ft. The idealized composite cross-section is shown below in Figure 1. The effective slopes on the sides of the embankment are taken as 14:1, horizontal:vertical.

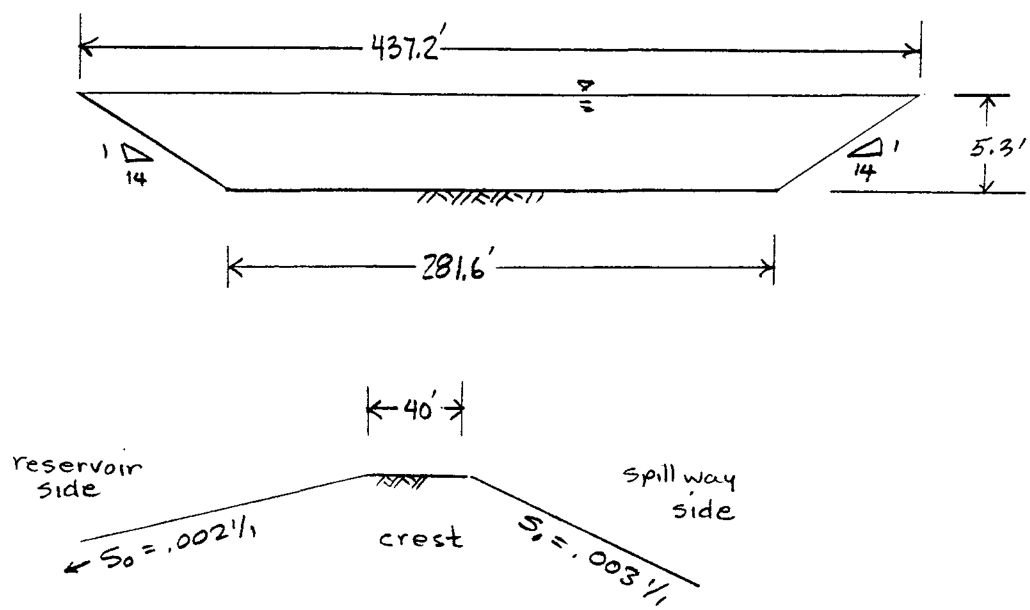


Figure 1: Idealized Spillway Dimensions as Modeled with HEC-2

As shown in Figure 1, the breadth of the spillway is 40 ft. The approach has an adverse slope of 0.002 ft/ft, and the spillway run has a slope of 0.003 ft/ft. The Manning's n value for the spillway is also conservatively estimated at 0.03.

HEC-2 Modeling of the White Tanks #4 Spillway

Computation of water surface profiles commenced 800 ft downstream of the spillway crest with normal depth as a starting water surface elevation. The invert elevation in the spillway at the beginning of the channel section is 1047.30 ft. Table 1 lists the starting water surface elevations at the various flow rates investigated.

Table 1
Starting Water Surface Elevations in HEC-2 Model at Station 0+00, 800 ft downstream from the Crest

Q (cfs)	d_n (ft)	Starting WSE (ft)
500	0.77	1048.07
1000	1.16	1048.46
2000	1.75	1049.05
3000	2.21	1049.51
4000	2.62	1049.92
5000	2.98	1050.28
6000	3.31	1050.61
7000	3.62	1050.92
8000	3.90	1051.20
9000	4.17	1051.47
10,000	4.43	1051.73
11,000	4.68	1051.98
12,000	4.91	1052.21
13,000	5.14	1052.44
14,000	5.35	1052.65

The HEC-2 analysis computes the water surface profile from the starting point, 800 ft downstream of the spillway crest, and every 100 ft up to the spillway crest. Cross

sections are placed at intervals of 5, 10 or 20 ft immediately downstream of the spillway crest and across the crest. Figure 2 shows the locations of cross sections in which water surface profiles were computed using the HEC-2 package.

The computation of water surface profiles across the spillway crest permits the development of a stage-storage-discharge relationship for the White Tanks #4 FRS. The stage-storage-discharge values are subsequently input to the HEC-1 program on SE, SV, and SQ cards. The HEC-1 program then performs a storage routing of the inflow hydrograph to determine the maximum water surface elevation during passage of the 1/2 PMF.

Table 2 presents the results of the HEC-2 analysis of the composite spillway under flow conditions from 500 cfs to 14,000 cfs.

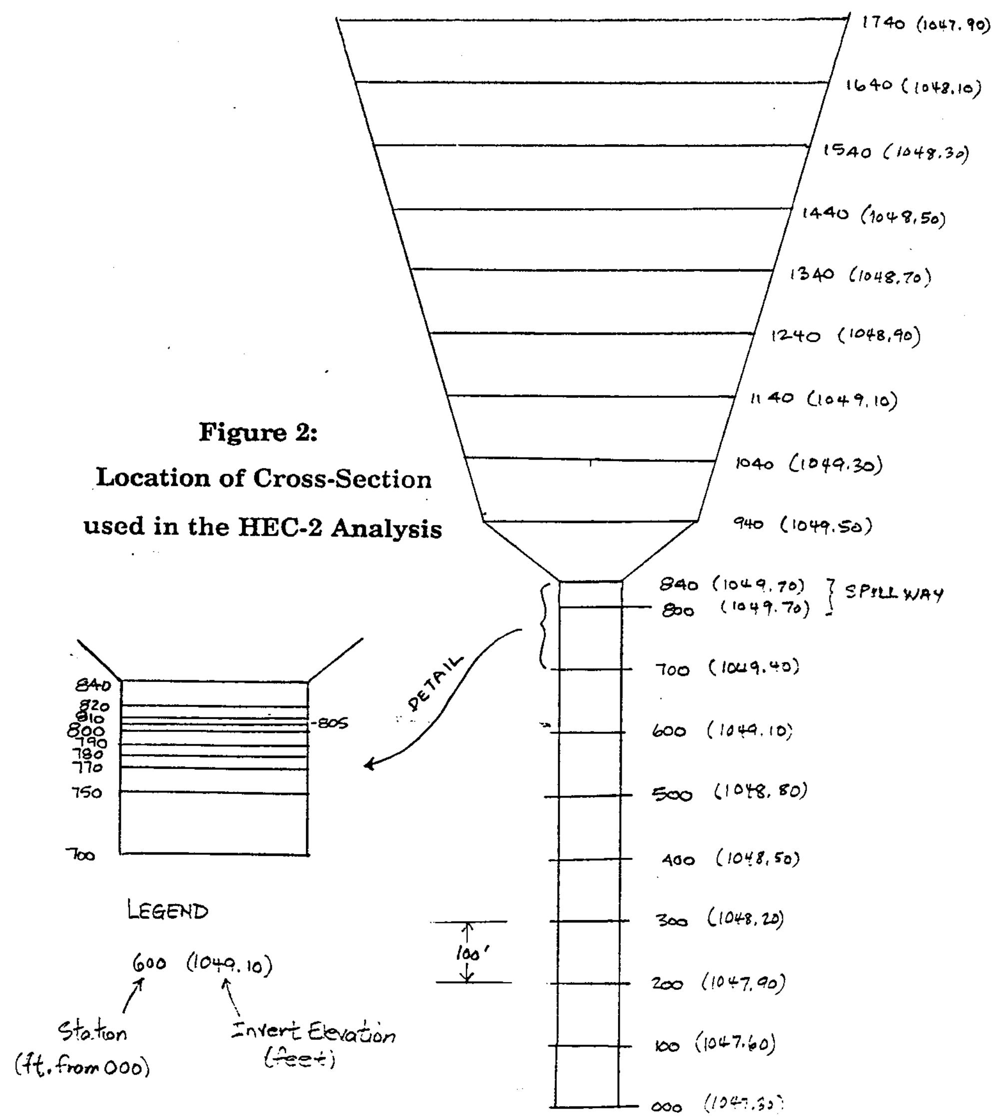


Figure 2:
Location of Cross-Section used in the HEC-2 Analysis

Table 2: Hydraulic Parameters from HEC-2 Analysis of the White Tanks #4 Idealized Spillway

1 Q (cfs)	2 Elev. EGL (ft)	3 $d_n+v^2/2g$	4 V_{ch} (fps)	5 L_{ave} (ft)	6 d_n (ft)	7 C	8 W.S.E. (ft)
500	1050.60	0.90	2.06	292.10	0.83	2.00	1050.67
1000	1051.10	1.40	2.61	297.71	1.29	2.03	1051.14
2000	1051.78	2.08	3.47	305.15	1.89	2.18	1051.84
3000	1052.33	2.63	4.08	311.05	2.37	2.26	1052.40
4000	1052.81	3.11	4.56	316.15	2.78	2.31	1052.89
5000	1053.23	3.53	4.93	320.87	3.16	2.35	1053.32
6000	1053.63	3.93	5.28	325.06	3.49	2.37	1053.72
7000	1053.99	4.29	5.59	328.94	3.81	2.39	1054.09
8000	1054.33	4.63	5.86	332.57	4.10	2.41	1054.44
9000	1054.66	4.96	6.11	336.01	4.38	2.43	1054.77
10,000	1054.97	5.27	6.35	339.27	4.64	2.44	1055.08
11,000	1055.26	5.56	6.56	342.38	4.89	2.45	1055.38
12,000	1055.54	5.84	6.76	345.36	5.13	2.46	1055.67
13,000	1055.82	6.12	6.96	348.22	5.36	2.47	1055.94

Notes:

2. *Energy Grade Line is measured at Sta 840, 40 ft upsteam of the spillway downstream edge.*
7. *$C = Q/[L_{ave} \cdot (d_n + v^2/2g)^{3/2}]$ as used in broad-crested weir equation: $Q = C\,L\,H^{3/2}$.*
8. *Water Surface Elevation is specified in the reservoir pool, 600 ft upstream of the spillway, at Sta 1740.00.*

The values in Table 3 from Column 1 and Column 8 provide the required information for the SQ and SE cards of the HEC-1 simulation. In Column 7, the C value of the broad-crested weir formula is back-calculated from the known values of discharge, total energy head, and average length of weir crest. For the White Tanks #4 spillway, the weir coefficient varies from 2.00 at 500 cfs to 2.47 at 13,000 cfs.

The HEC-2 computer print-out of the spillway analysis is included as an Annex to this report. A diskette with input and output files for HEC-2 is also included.

Top of Dam Elevations

The top of the dam embankment for White Tanks #4 F.R.S. was surveyed on July 2, 1993 by FCD personnel. The results of the survey are shown in Figure 3. As shown, the top of dam is not completely level, but varies by slightly more than a foot, between 1055.2 and 1056.3. The reference elevation used in the survey was the RM 209, a 1/2" open pipe in a hand hole at the intersection of Jackrabbit Trail and Van Buren Street. This is the same reference mark used in the WLB White Tanks ADMS.

Figure 3 indicates that the spillway elevation is lower on the western spillway (Sta 6600 to 6751.6) than for the eastern spillway (Sta -130 to Sta +00.) The western spillway crest is located at elevation 1048.7 (6.5 ft. below the top-of-dam elevation *assumed* at 1055.0.) The eastern spillway was found to be at an elevation of 1049.4 (5.8 ft. below the assumed 1055.0 elevation.)

These survey results of top-of-dam elevations indicate that for storage routing through the reservoir, it is unnecessary to represent the top-of-dam elevation as non-level, provided that the maximum water surface elevation in the reservoir pool does not exceed the elevation of 1055.0. This value is conservative, since the lowest elevation on the top of the dam is *actually* at 1055.2, as shown in Figure 3.

Potential Backwater Effects Downstream of the Spillways

The are no significant impedances to flow downstream of the two spillways which are likely to cause backwater effects during passage of the 1/2 PMF. The prevailing topographic gradient slopes 0.0088 ft/ft to the southeast. On the eastern spillway, a farm equipment yard with a chain-link security fence is located about 950 ft. downstream of the eastern spillway crest. At this distance from the spillway, the flow is no longer confined within the 130 ft. spillway width, but is free to cross Jackrabbit Trail and proceed southeasterly along the steeper prevailing topographic gradient. Hydraulic analysis of the spillway shows that as gradient changes from 0.003 ft/ft (spillway) to 0.0088 ft/ft (natural topography), and n value is increased from 0.03 (spillway) to 0.07 (farm equipment yard), an equivalent conveyance capacity is achieved by increasing the channel bottom width by only 120 ft. The spread-out of flows across Jackrabbit Trail provides more than this additional width of channel, thus backwater effects are not expected from the farm equipment sales outlet. The next nearest potential obstruction is the Roosevelt Irrigation District Canal, which lies more than a mile to the south of the White Tanks #4 FRS--too far to produce backwater effects that could affect spillway rating.

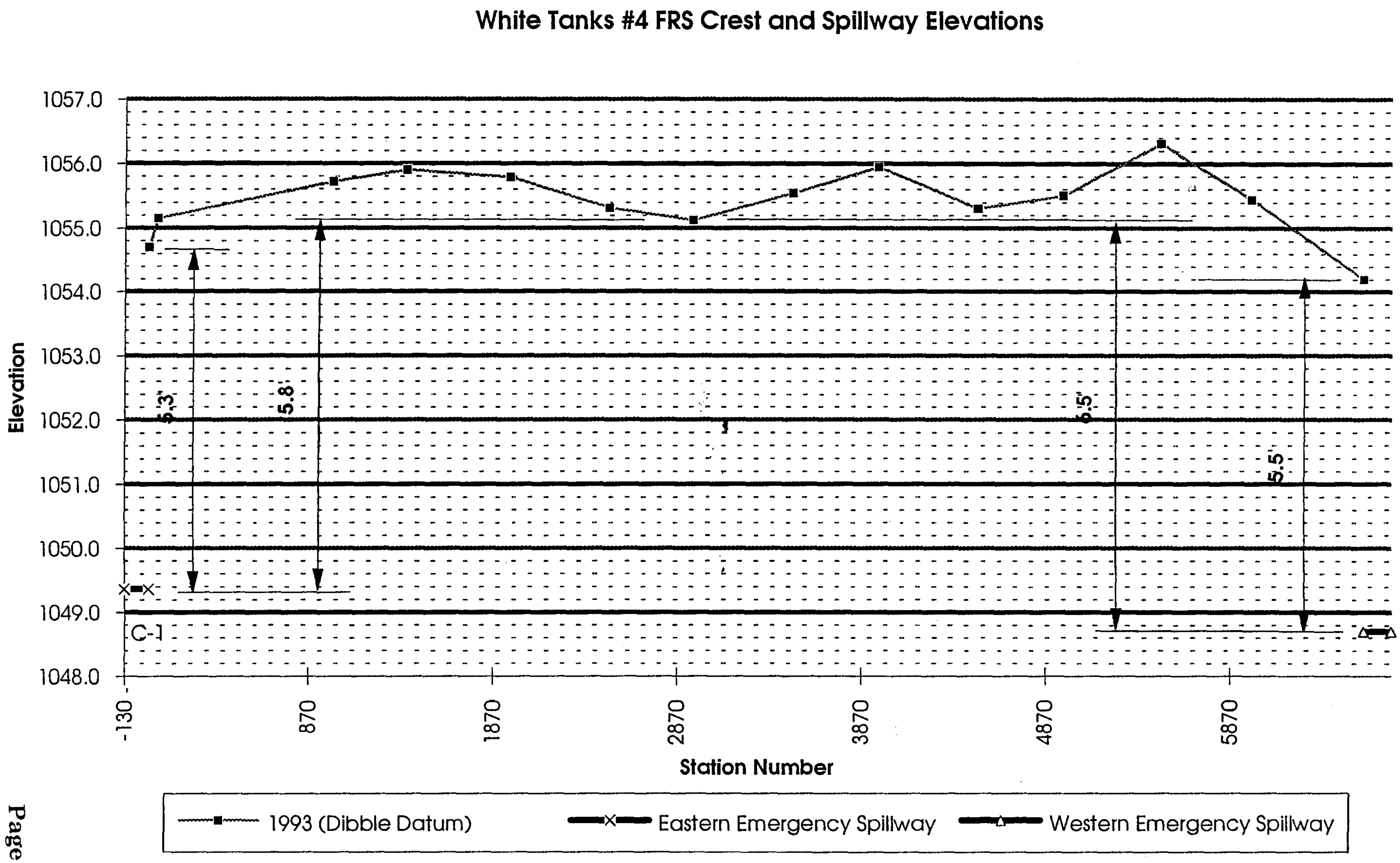
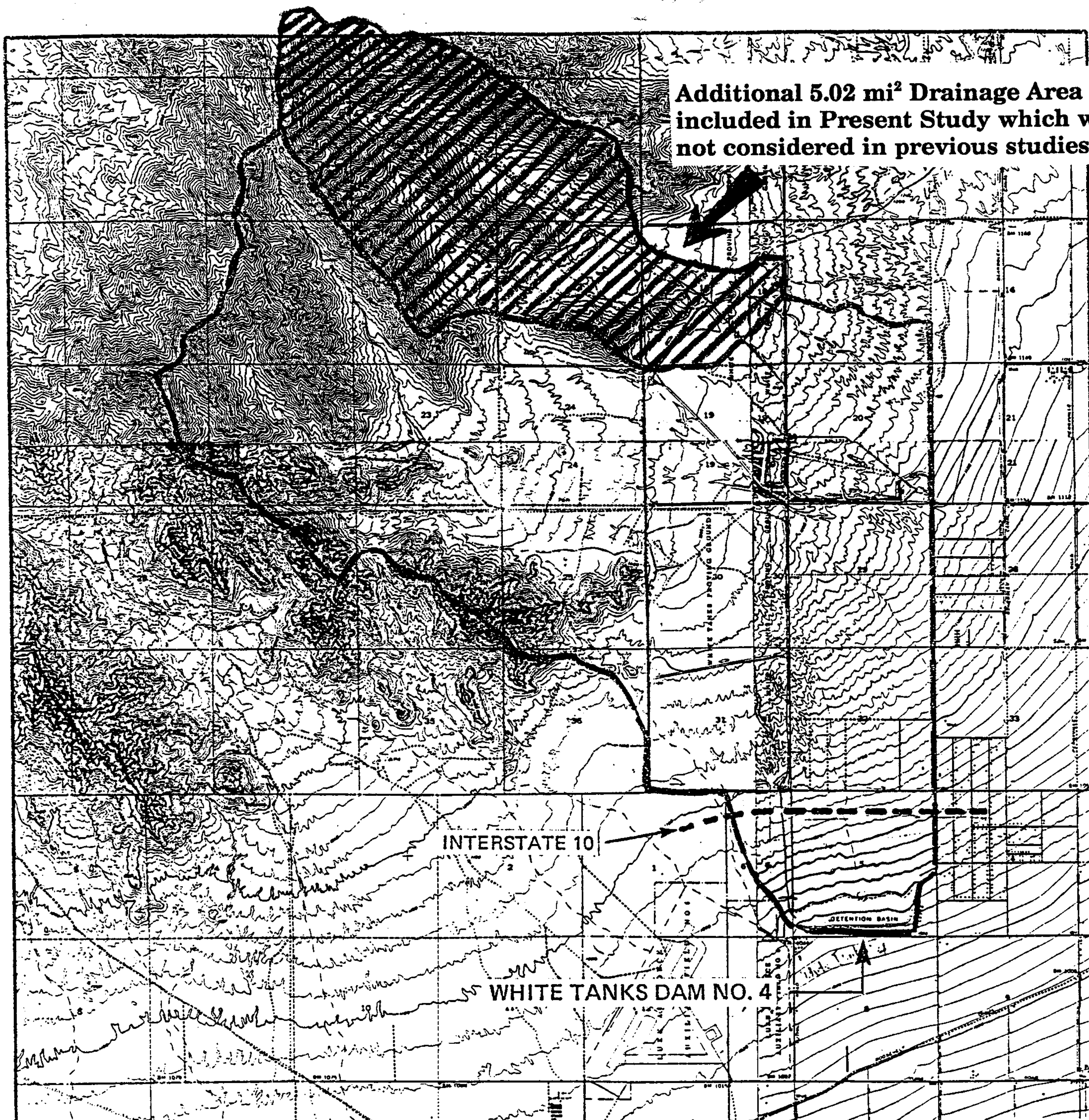

White Tanks #4 FRS Crest and Spillway Elevations
Elevation
1057.0
1056.0
1055.0
1054.0
1053.0
1052.0
1051.0
1050.0
1049.0
1048.0
5.3'
5.8'
6.5'
5.5'
C-1
-130
870
1870
2870
3870
4870
5870
Station Number
1993 (Dibble Datum)
Eastern Emergency Spillway
Western Emergency Spillway

Figure 3



NORTH

1 .5 0 1 MILES

1 .5 0 1 KILOMETERS

Figure 4:

Contributory Drainage Area of White Tanks #4 FRS As Modeled Previously and in Current Study

Ertec The Earth Technology Corporation	PROJECT NO.: 81-161 PHASE I SAFETY INSPECTION WHITE TANKS DAM NO. 4
WATERSHED MAP	
7-81	

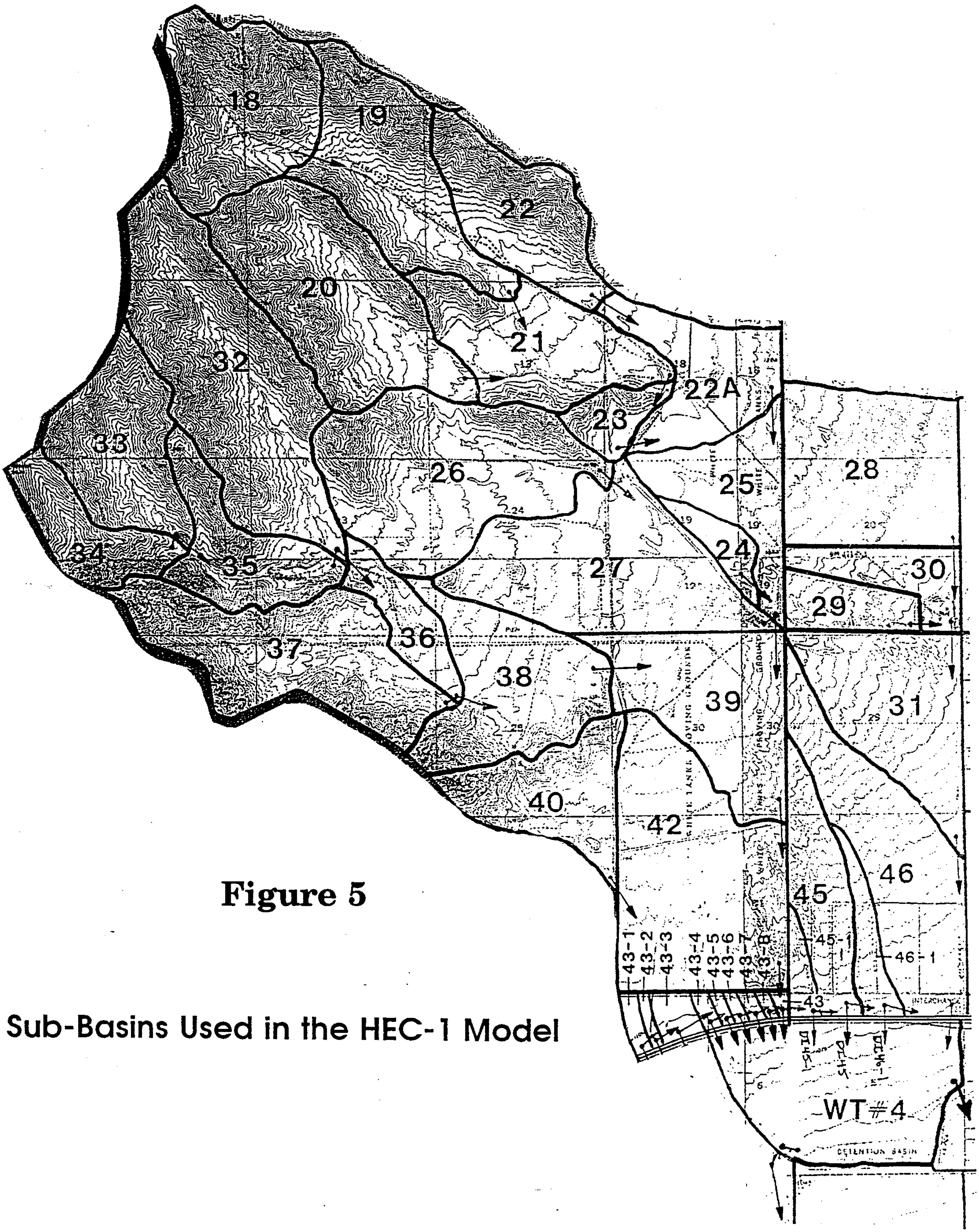


Figure 5

Sub-Basins Used in the HEC-1 Model

Hydrologic Modeling of the Contributing Watershed

DRAINAGE AREA

The contributing drainage area was modeled using the US Army Corps of Engineers HEC-1 computer model. The rainfall/runoff model was developed for the Flood Control District by the consultant WLB, Inc. as part of an Area Drainage Master Study (ADMS.) Photogrammetry of the area was completed in 1989, and study area maps of 400 scale with two foot contour intervals were used for delineating the drainage boundaries. The ADMS also included the establishment of base-flood elevations utilizing the 100-year 24-hour duration precipitation event. Boundaries of drainage areas were thus defined based upon the 100-year runoff. For precipitation events of greater intensity, such as the 1/2 PMF which is considered here, drainage boundaries could differ from those defined for the 100-year event; however, no attempt was made to re-delineate subbasin boundaries for the 1/2 PMF.

The contributing area consists of the west and southern slopes of the White Tanks Mountains, foothills, and southwesterly-sloping alluvial plains. Total catchment area as modeled comprises 20.25 square miles. A previous study of the catchment by Earth Technology Corporation (ERTEC) in 1981 modeled the catchment as 14.23 square miles. Previous studies apparently relied only upon USGS 7.5 minute quadrangle mapping with 10 foot contours. With the better 2-foot contour maps, WLB identified a dike running from Camelback Road 1 mile west of Tuthill Road northeast to Tuthill Road, one-half mile north of Camelback. This dike diverts runoff from the additional 5.02 square mile drainage area to Tuthill Road, where it flows south along the western side of Tuthill Road. Figure 4 shows the additional catchment area considered in the ADMS and in this study.

SUBBASIN DELINEATION

For the hydrologic modeling, the contributory watershed was divided into 37 subbasins by WLB Group for the White Tanks ADMS, which was designed to determine 100-yr base flood elevations. Figure 5 shows the subbasin boundaries as delineated by WLB.

For the estimation of the 1/2 PMF, the same subbasin boundaries were used. The contributory drainage area is bounded on the east by Jackrabbit Trail. For the 100-year stormwater discharge, it was assumed that new channel improvements would contain all flows to the west side of Jackrabbit Trail, and convey them to the White Tanks #4 FRS. During the 1/2 PMF; however, flows in excess of about 2000 cfs are expected to cross Jackrabbit Trail, and sheet flow along the prevailing southeasterly natural topographic gradient. The splitting of these flows is discussed later in this report.

Design Storm Event (PMP)

The calculations for the PMP were performed for a contributory drainage area of 19.3 sq. mi. according to the procedures outlined in HMR-49. The General and Local Storm Probable Maximum Precipitation calculations are contained in an Annex to this report. The Local Storm PMP of total depth of 12.3 inches over 6 hours was used for the design storm input to the HEC-1 model. The hourly depth of precipitation was determined from HMR-51 as follows:

Hour	Rainfall
first	0.4"
second	0.7"
third	8.5"
fourth	1.6"
fifth	0.7"
sixth	0.4"

The most intense hour of precipitation in the PMP occurs in the third hour. During that period, 5.4" falls during the first 15-minute interval, 1.6" in the second, 0.8" in the third, and 0.7" in the last.

Rainfall/Runoff Model

The rainfall/runoff modeling was performed with the HEC-1 Flood Hydrograph Package by the US Army Corps of Engineers. The unit hydrographs were generated from Phoenix valley and Phoenix mountain S-Graphs, according to the procedures outlined in FCD's *Hydrologic Design Manual.* The rainfall losses were computed using the Green-Ampt procedure, based upon local soils data and ground cover. The modeling procedures as presented in WLB's White Tanks ADMS are presented in an Annex to this report.

Hydrograph Routing

Modified Puls Routing

Runoff hydrographs from each subbasin are routed to combination points using normal depth routing. The routing procedure utilizes the HEC-1 input combination of RS, RC, RX, and RY input cards. The typical cross-section representing the geometry of the routing reach is input on the RX/RY cards. This cross-section data was developed for the ADMS by WLB, Inc. from 2 foot contour maps. When the 1/2

PMF is generated by 12.3" of precipitation, these cross sections are not sufficiently wide to accommodate the massive runoff flows generated. To accommodate the additional flows, the "ELMAX" field on the RC card is provided with an elevation sufficiently high, so that the HEC-1 program will automatically extend the cross section to the elevation input in the ELMAX field. This eliminates warning messages in the HEC-1 output.

Storage Routing

In routing reaches where significant ponding of water occurs, the reaches are defined using the SE, SV, SQ cards available in HEC-1. These situations occur at several storage pond locations within the Caterpillar property, at the White Tanks #4 FRS itself, and on the north side of Interstate 10. For storage ponds, the SV, SE relationship is determined by planimetering the areas at each contour and multiplying the area by the 2 foot contour interval. The SQ card for ponds is defined by a broad-crested weir equation (using C = 2.5) for the length of the spillway at lower elevations, and across the entire length of the dam embankment at higher elevations.

inflow

On the north side of Interstate 10, there ~~are numerous~~ culverts located between Tuthill Road and Jackrabbit Trail. When inflowing stormwater ponds behind I-10, it may exit in three potential directions. The initial outflow is to the south into the White Tanks #4 FRS through the culvert(s). Some model reaches have as many as nine culvert crossings. These flows are defined by use of a culvert analysis spreadsheet at various depths assuming inlet control. For the PMF, culvert ratings were extended by approximation of orifice flow, with discharge proportional to the square root of upstream head.

Along the north side of I-10, once sufficient head is built up at culvert inlets, an additional exit point for flow out of the storage routing reach is to the east along the north side of I-10. This flow can be represented as either weir flow, or as normal depth flow. ADOT constructed small embankments in a north-south alignment near some culverts in this area to facilitate build-up of head to increase flow south through the culverts. Thus, weir flow is used to represent these eastbound flows across the ADOT embankments at low elevations. Once depth of flow exceeds 3 or 4 feet; however, normal depth flow calculations are used to continue the rating with the Manning equation.

A third component of flows exiting the storage routing reach north of I-10 consists of flows overtopping I-10 and flowing to the south, entering the White Tanks #4 FRS. These flows occur at Tuthill Dike, and in two routing reaches between Tuthill Road and Jackrabbit Trail. They are quantified as weir flows considering the elevations of the I-10 freeway.

In order to extend the rating for storage routing reaches, topographic maps were produced with 2-foot contours at a scale of 1" = 200'. The topographic information was used to extend the routing cross sections and to determine the storage volumes at each contour interval. Culvert flows are diverted south into the White Tanks #4

FRS, while weir and normal depth flows eastward along the north side of I-10 are the remaining (undiverted) flow. Weir flows representing overtopping of I-10 are also diverted from the main routing reach and retrieved later to join with culvert flows entering the White Tanks #4 FRS.

Diversions Across Jackrabbit Trail

As mentioned previously, the confinement of flows west of Jackrabbit Trail, while realistic for the 100-year event, is not an appropriate representation for the 1/2 PMF. To represent the flow across Jackrabbit Trail during the 1/2 PMF, diversions are employed at three locations: 1. 800 feet south of CP31, 2. at the north end of the I-10/Jackrabbit Trail underpass, and 3. at the south end of the I-10/Jackrabbit Trail underpass.

At the first diversion (800 feet south of CP31) only 2000 cfs is allowed to continue south within the existing earthen channel on the west side of Jackrabbit Trail. Flows in excess of 2000 cfs are diverted out of the model.

On the north entrance of the Jackrabbit Trail/I-10 underpass, flow is diverted eastward along the north side of I-10. The rating is accomplished by representing weir flow south through the underpass to the south as $Q = C*L*H^{3/2}$, with $C = 2.5$. Flow continuing east along the north side of I-10 is assumed to behave as normal depth flow following the prevailing topographic gradient. Areas were determined for each contour interval between 1074 and 1080, and input to the storage routing reach as SV, SE, and SQ cards.

South of the underpass, flow was again diverted using a circular cross section 300 feet south of the underpass. Flows on the west side of Jackrabbit Trail centerline are assumed to enter the Flood Retarding Structure, while flows on the east side are assumed to leave the area to the southeast.

Sketches of each diversion point and computations of flows are presented in an annex to this report.

Inflow Design Flood (IDF) Hydrograph

The result of the hydrologic modeling of the contributory watershed is the Inflow Design Flood (IDF) hydrograph, which results from the 1/2 PMF. This hydrograph is subsequently routed through the White Tanks #4 Flood Retarding Structure. Figure 6 shows the IDF.

As shown in Figure 6, for the 1/2 PMF, the peak rate of inflow into the White Tanks #4 FRS is 17,738 cfs, which occurs 3 hours and 20 minutes after the beginning of the Probable Maximum Precipitation. The Probable Maximum Flood (PMF) is thus 35,476 cfs. In the HEC-1 model, the JR card is used to multiply the runoff from each individual subbasin by 0.5 to produce the 1/2 PMF. The HEC-1 input and output is included as an annex to this report. Refer to this annex for additional information on the IDF.

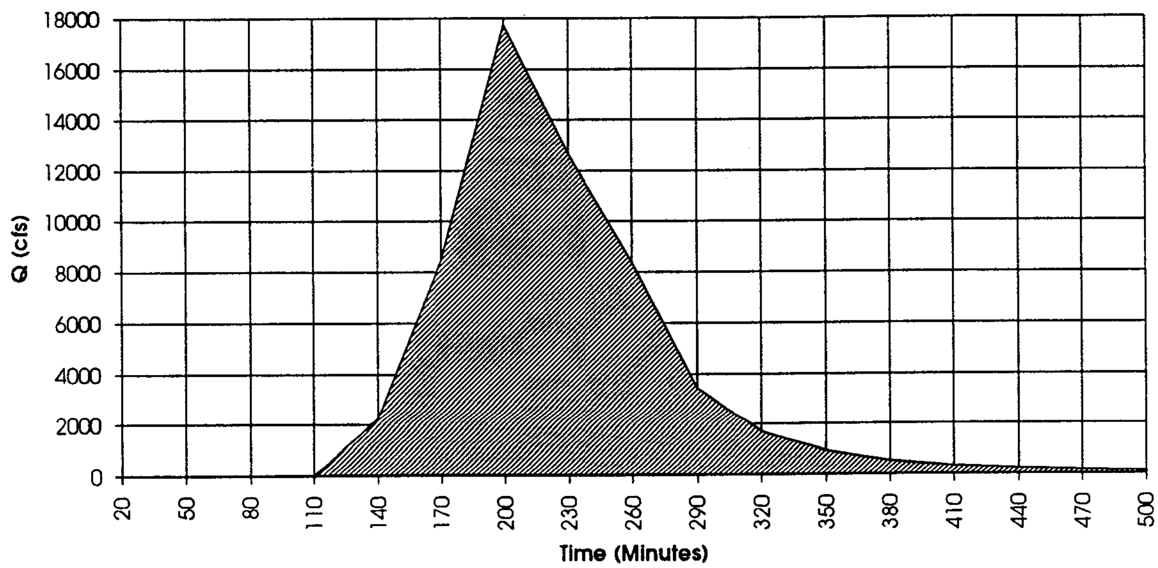


Figure 6 Inflow Design Flood Hydrograph to White Tanks #4 FRS

Routing of the 1/2 PMF Hydrograph

STORAGE ROUTING PROCEDURE

The HEC-1 model used to generate the Inflow Design Flood **(IDF) Hydrograph (Figure 6) is also used to route the** hydrograph through **the Flood Retarding Structure. Storage routing is employed** to accomplish **this task. The stage-storage-outflow table is input to the routing reach of the** HEC-1 input **via SV, SE, SQ cards, which contain, respectively,** volume, water surface **elevation, and spillway discharge information.**

The values on the SE cards represent elevations **obtained from the 2-foot-interval contour mapping of WLB.** Storage volumes are **obtained by planimetering the areas at each contour, then multiplying by the** 2-foot contour depth. **The lowest elevation behind the White Tanks** #4 FRS is 997.0 ft **above sea level, which represents the bottom of the pit** excavated by ADOT **for material used in construction of Interstate 10. Storage** volumes are computed **for selected elevations between 997.0 and 1055.0. The** 1055.0 elevation represents **the top of the dam. As mentioned in the first section of this** report, **the actual lowest elevation surveyed on the top of the dam is 1055.2; however,** 1055.0 is **used as the limit for volume and discharge rating. This value is adequate,** since the **1/2 PMF does not exceed a water surface elevation of 1055.0.**

difference?

The values on the SQ cards represent the discharge through the spillway at various elevations of water surface elevation in the reservoir pool. In the first section of this report, the hydraulic analysis of the spillway is detailed. Briefly, the HEC-2 Water Surface Profiles computer model was used to analyze an idealized composite spillway representative of the two spillways on the White Tanks #4 FRS to develop a relationship between water surface pool elevation and spillway discharge. This relationship is presented in Table 2, Columns 1 and 8. There is no discharge over the spillway at water surface elevations of 1049.7 or less--the modeled spillway crest elevation is 1049.4. The distance between spillway crest elevation (1049.7) and the top-of-dam elevation (1055.0) represents the minimum distance actually surveyed between spillway crest elevation and top-of-dam elevation as measured at the dam embankment adjacent to the eastern emergency spillway. (Refer to Figure 3.) The spillway is rated for flows between 0 and 13,000 cfs, which is more than adequate to contain the SDH. Table 3 shows the values on the SE, SV, and SQ cards of the HEC-1 storage routing reach for the White Tanks #4 FRS.

Table 3: HEC-1 SE, SV, SQ Cards Representing White Tanks #4 FRS

Remark:	SE	SV	SQ
Bottom of Pit	997.0	0	0
	1000.0	2.81	0
	1010.0	51	0
Top of Pit = 1024.0	1020.0	148	0
	1030.0	277	0
	1040.0	543	0
	1049.0	1269	0
Spillway Crest Elev = 1049.7	1050.0	1396	92.5
	1050.67	1494	500
	1051.14	1564	1000
	1051.80	1666	2000
	1052.40	1761	3000
	1052.89	1848	4000
	1053.32	1924	5000
	1053.72	1995	6000
	1054.09	2063	7000
	1054.77	2200	8000
Top of Dam = 1055.0	1055.08	2266	9000

Initial Reservoir Water Surface Elevation

Initial starting water content of the White Tanks #4 FRS could be assumed to be in three conceivable conditions:

1. **Starting with reservoir full to spillway crest level (1049.7);**
2. **Starting with basic design level of empty prior to borrow-pit excavation (1024);**
3. **Starting with basic design level as augmented by borrow-pit excavation (997.0).**

Of these three options, ADWR has recommend application of assumption #2, citing the possibility that the borrow-pit area may be either full from runoff from a previous less than 1/2 PMF event or that it might be silted up from sediment deposition.

Without comment to the appropriateness of the assumption, the storage routing was performed assuming that the initial contents of the pool were at 200 acre feet-- the approximate volume of the excavated borrow-pit area. The value is specified on the RS card of the HEC-1 input indicating an initial storage volume of 200 AF.

Spillway Design Hydrograph (SDH)

The results of the HEC-1 modeling are displayed in detail in an annex to this report. The routed hydrograph passing through the spillway resulting from the IDF is the Spillway Design Hydrograph (SDH.) Figure 7 shows the plot of this hydrograph, which is produced by the 1/2 PMF.

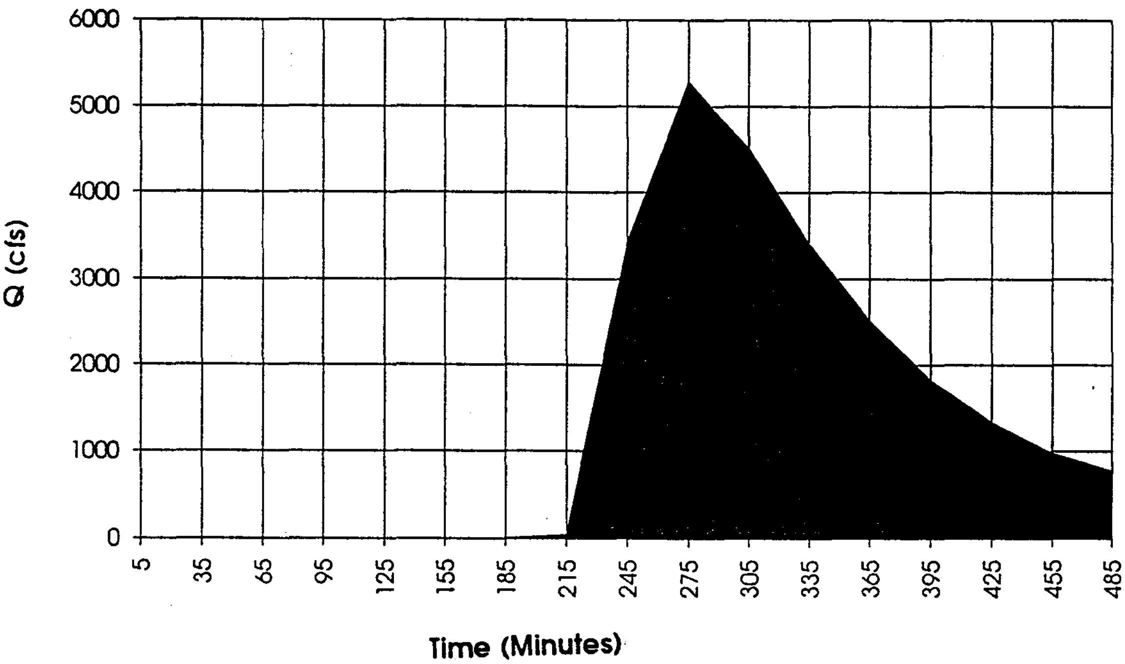


Figure 7: Spillway Design Hydrograph for White Tanks #4 FRS

As shown in Figure 7, the maximum rate of outflow through the emergency spillway is 5286 cfs, which occurs 4 hours and 35 minutes after the onset of the Probable Maximum Precipitation. The maximum water surface elevation at the spillway is 1053.4 ft.--1.6 feet below the top-of-dam elevation of 1055.0. This elevation is reached at the time of the peak outflow, 4.5 hours after the beginning of rainfall. Refer to the HEC-1 output in the annex for additional details of the simulation.

Conclusions

Based upon the hydraulic analysis of the White Tanks #4 FRS spillways, and the hydrologic modeling of the 1/2 Probable Maximum Flood (PMF), it has been shown that the maximum elevation of pooled water surface during the event is 1053.4 ft. This elevation is 1.6 feet below the assumed top-of-dam elevation of 1055.0. The 1/2 PMF thus passes the emergency spillway with 1.6 feet of freeboard to spare. If the top-of-dam elevation of 1055.2 is considered (as borne out by field survey), the freeboard is 1.8 feet.

According to the guidelines specific to the White Tanks #4 FRS provide by ADWR's David Creighton, "A one-foot minimum residual for IDF's between 100-yr. and 0.5 PMF is considered to be a reasonable guidance considering the difference between "minimum permissible total" of 4 feet and the "residual freeboard" of 3 feet, and the length of fetch." (See annex for complete text.)

Therefore, since the SDH resulting from the storage routing of the 1/2 PMF through the White Tanks #4 FRS (with an initial starting water content of 200 AF) results in a maximum water surface elevation 1.6 to 1.8 feet below the top-of-dam elevation, it can be concluded that the spillway capacity for this structure is safe for the existing condition. The dam should be immediately reclassified from "unsafe" to "safe" by ADWR, because, as shown in this report, the spillway capacity is adequate.

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WSMB INTEROFFICE MEMORANDUM

To : DBB
Cc : AMM

From : MRD MRD

Date:July 20, 1993

SUBJECT: COMMENTS ON WHITE TANK #4 REPORT DATED JULY 8, 1993

1. Normally the hydrologic portion precedes the hydraulic analysis. The report has the hydraulic analysis discussed first.

2. With the 1/2PMF, is the assumed storm size the same as that of the 100yr-24hr? Could this be the reason for possible different drainage basin boundaries? An expalnation to this effect can be added to the report.

3. With the 1/2PMF there may be possible submerged flow at some of the culverts crossing I-10 and Q would no longer be proportional to square root of head. Is there possibility for this scenario?

4. With the small embankments constructed by ADOT (north-south alignment) are the potential storage negligible to be included in the model?

5. At the first diversion only 2000cfs is allowed to continue. What structural measure is proposed to contain the excess of 2000cfs so that it will not rejoin the system?

6. Table 3 can be presented in graphical display to show clearer relationship of SV, SE and SQ.

7. In many RS records the ITYP is not specified. It seems it does not affect the results.

8. After KK RS47 in HEC-1 model, the initial storage of 200 AF does not appear in the SV record. Isnt the first value equal to the initial storage? no 1049.7

9. The invert elev. of spillway is 1047.3'. In the SE record after KK RS47 there is no outflow until SE becomes greater than 1049. Note that in HEC-2 model some 800'd/s of crest there is flow of 500cfs at 1048.7 elev.

10. Figures 4 and 5 should follow the report after page 9.

11. With max. Q of 5286, I think the velocity at the spillway channel should be checked if it is greater than allowable.

12. In the report of M. Bressor, the d/s spillway slope of eastern spillway is -0.4%. If this is correct flow will back up before it will be discharged.

SPILLWAY CREST ANALYSIS - HEC-2

Effect of Mannings "n" on. Crest Coefficient $= \frac{Q}{LH^{3/2}}$ 6

NC = 0.000, 0.001, 0.01, 0.015, 0.02, 0.025, 0.03, 0.035

H based on. (EG - ELMIN)