Updated Hydrology Analysis
Arizona Canal Drainage Channel

Pima Freeway
Via Linda Drive to Arizona Canal

Maricopa County, Arizona

ROBERT L. WARD, P.E. *Consulting Engineer*

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Via Linda Drive to Arizona Canal

Maricopa County, Arizona

prepared for:

Salt River Pima-Maricopa Indian Community
Route 1, Box 216
Scottsdale, Arizona 85256

prepared by:

Robert L. Ward, P.E.
Consulting Engineer
5345 East McLellan Road, #112
Mesa, Arizona 85205

August 20, 1996
Final Report

Robert L. Ward

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13505
ROBERT L.
WARD
8/20/96
Date Signed
ARIZONA, U.S.A.

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Robert B. Ward

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13506
ROBERT B.
WARD
Date Signed 8/20/96
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1 INTRODUCTION

The purpose of this report is to update the hydrology analysis that was prepared for that reach of the Pima Freeway extending from Via Linda Drive to the Arizona Canal. Previous hydrology analyses for this project are documented in the **Concept Design Summary, Arizona Canal Drainage Channel, Pima Freeway**, May 18, 1993, Robert L. Ward, P.E., Consulting Engineer. Figure 1.1 is a location map showing the watershed boundaries for this project.

The proposed Pima Freeway drainage system consists of an open channel along the east side of the Freeway. Floodwaters intercepted by this channel will flow south for discharge to the proposed Arizona Canal Drainage Channel (ACDC). The westerly flowing ACDC will be connected to the existing Indian Bend Wash (IBW) Interceptor Channel.

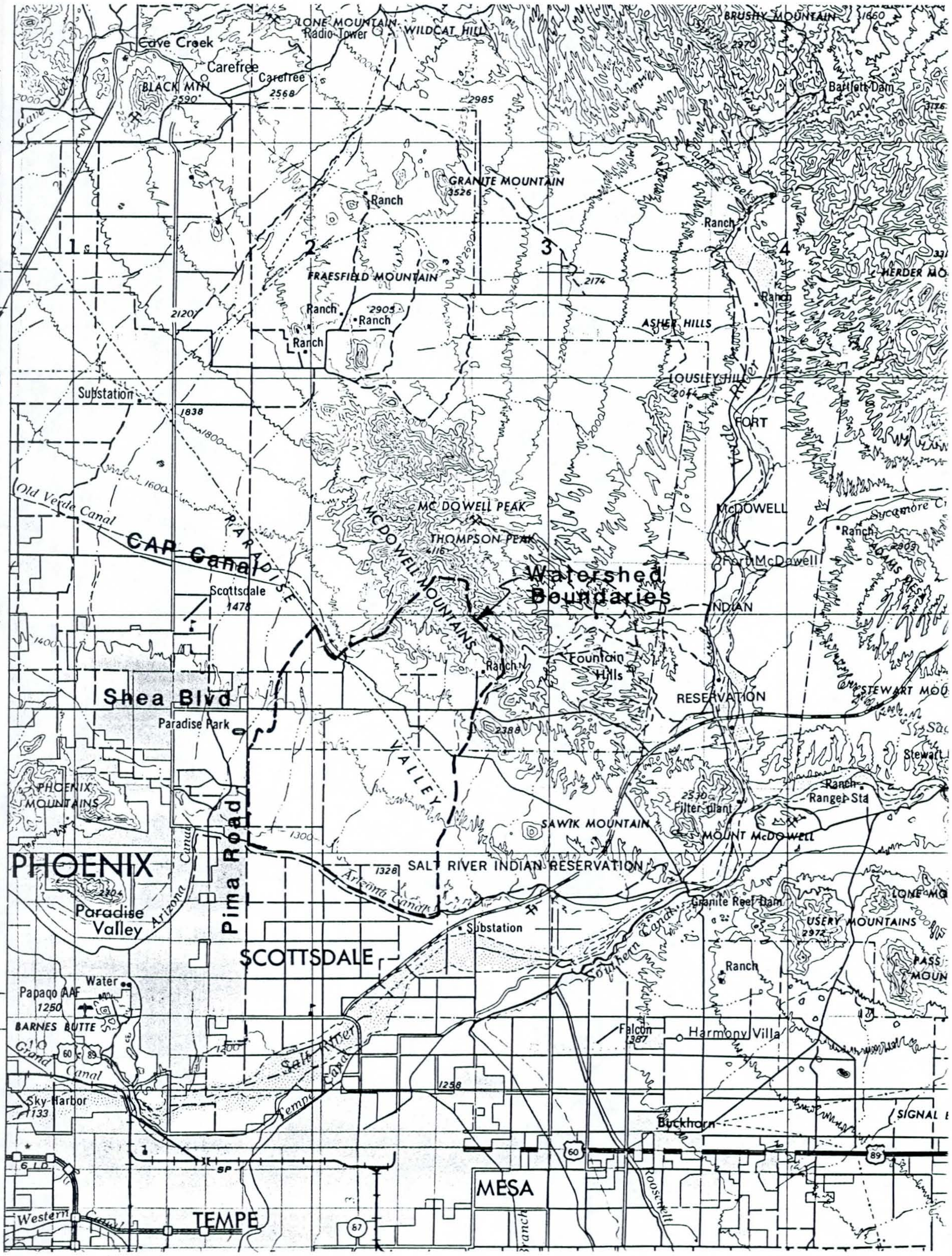
The ACDC was originally designed by ADOT to extend from Pima Road to the Pima Freeway. At the request of the Salt River Pima-Maricopa Indian Community (SRPMIC), the ACDC design was modified in 1992 to accommodate an easterly extension of the channel to 96th Street. This eastern channel extension, which was designed by Evans, Kuhn & Associates, will provide an outfall for the proposed 96th Street storm drain system.

During May 1993, ADOT turned the ACDC portion of this project over to SRPMIC for final design and construction. In April 1996, SRPMIC retained Robert L. Ward, P.E., (with the approval of ADOT) to make modifications and refinements to the previously developed HEC-1 models in order that SRPMIC could proceed with the Arizona Canal Drainage Channel design.

A technical coordination meeting was held at SRPMIC on May 30, 1996 to discuss hydrologic modeling criteria that would be acceptable for allowing the ACDC to outlet to the Indian Bend Wash Interceptor Channel, located on the west side of Pima Road. Representatives from the City of Scottsdale, the Flood Control District of Maricopa County, and SRPMIC attended this meeting.

It was agreed at the conclusion of this meeting that the 100-year runoff from the Pima Freeway drainage system (including the ACDC), as well as that from the associated SRPMIC and City of Scottsdale watershed, would not be allowed to exceed 8,000 cfs at the northeast corner of Pima Road and the Arizona Canal. It was further agreed that the following HEC-1 modeling refinements would be made prior to running the model to determine compliance with the 8,000 cfs criteria at Pima Road.

1. Recent HEC-1 models commissioned by the City of Scottsdale, for areas in and adjacent to the Stonegate development, would be used to replace those corresponding portions of the original Pima Freeway HEC-1 model. This replacement was considered justified to reflect new development that is presently in-place, but was not



Location Map

Miles

0 5

Arizona Canal Drainage Channel	
Pima Freeway Drainage System	
August 1996	Figure 1.1

in-place when the original Pima Freeway hydrology was developed in 1988 and 1989.

2. An "As-Built" stage/storage/discharge relationship would be developed for the Scottsdale Ranch Detention Basin. The IBW Interceptor Channel was designed on the assumption that the City of Scottsdale would provide 357 AF of flood control storage and limit the 100-year discharge to a maximum of 200 cfs at Corp's CP 103 (see page III-5, **Design Memorandum No. 4, Feature Design For Interceptor Channel**, U.S.A.C.O.E., January 1980).

   Although a detention basin has been built on Scottsdale Ranch, no technical documentation has been found to verify whether the basin will perform as assumed in the Corp's design for the Interceptor channel. The original Pima Freeway HEC-1 model used an outflow hydrograph from this basin that was taken from reports prepared in the 1970's by Water Resources Associates (WRA), Inc. and Collar, Williams & White Engineering (CWW), Inc. (see page 23, **Final Hydrology Report, Outer Loop Highway, Camelback Walk Channel to the Arizona Canal**, May 1989, Simons, Li & Associates, Inc. (SLA) This outflow hydrograph used a peak discharge of 336 cfs.

   Using 1993 topographic mapping provided by the City of Scottsdale, an As-Built stage/storage/discharge relation was developed for the Scottsdale Ranch Detention Basin. This data was combined with the City's recent HEC-1 model for the drainage area intercepted by this detention basin in order to create a more reliable outflow hydrograph from this basin than was used in the original SLA model.

3. If possible, refinements were to be made to the outflow hydrographs for the CAP overchutes that contribute runoff to the project watershed. The previous Pima Freeway HEC-1 model used peak discharges, provided by the Bureau of Reclamation, to create <u>assumed</u> hydrograph shapes (and timing) for these overchute locations.

   During recent years, detailed HEC-1 modeling has been performed for the upstream watersheds that drain to these overchutes. In some cases, the Bureau of Reclamation has also provided stage/storage/discharge data that can be used to develop reservoir routing operations to analyze the hydrograph attenuation effects that these overchutes are capable of producing. Accordingly, this new data has been used to refine the outflow hydrographs at the CAP overchutes contained within the project watershed.

The results of these modeling refinements indicate that the proposed Pima Freeway drainage system, in conjunction with the proposed SRPMIC 96th Street storm drain system and existing City of Scottsdale urbanization changes, should not cause the 100-year peak discharge at Pima Road and the Arizona Canal to exceed 8,000 cfs. The HEC-1 model developed for this discharge verification will be used as a baseline condition to measure the hydrologic impact of any future changes on both SRPMIC and City of Scottsdale lands. Future land-use changes must be implemented in a manner that will preserve this baseline discharge.

2 DISCUSSION OF MODELING PARAMETERS

The following sections of this report provide a technical discussion of the refinements that were made to the original Pima Freeway HEC-1 model referenced in the May 18, 1993 **Concept Design Summary**. A complete discussion of the HEC-1 parameters, that were used to create the original HEC-1 model for this watershed, is presented in the previously referenced 1989 SLA **Hydrology Report**.

Appendix A includes a copy of the peak discharge output summary for Model PF4.7I, which is the recommended HEC-1 file for demonstrating that the 8,000 cfs limit is not exceeded at the intersection of Pima Road and the Arizona Canal. Appendix B includes a magnetic disk with executable input files for the four HEC-1 models that are presented in this report.

2.1 HEC-1 Program Version

The original HEC-1 modeling for the Pima Freeway offsite drainage system was performed in the late 1980's with the 1985 program version of HEC-1. In order to preserve continuity for peak discharge comparisons, as modeling revisions were made into the 1990's, the 1985 program version has continued to be used.

However, since a secondary objective of this updated modeling effort is to provide SRPMIC with a detailed, baseline hydrologic model that can be used for future land planning purposes, the modeling refinements addressed in this report have been run under both the 1985 program version and the 1991 version 4.0.1E. Changes in the kinematic wave routing algorithms cause the 1991 version to produce slightly higher peak discharges than the 1985 version.

Use of the 1991 program version will provide SRPMIC with some additional modeling capabilities that are not available in the 1985 program version.

2.2 Rainfall Data

Rainfall depths for the project watershed were developed using isopluvial maps and regression equations presented in the **Precipitation-Frequency Atlas of the Western United States, Volume III - Arizona**. Table 3.1 of the May 1989 SLA report presents the point rainfall values for the 50-and 100-year events. For convenient reference, Table 2.1 of

Table 2.1
Summary Of Point Rainfall Data
Pima Freeway Drainage System at the Arizona Canal
Maricopa County, Arizona

Storm Return Interval (yrs)	Point Precipitation (inches)								
	5-min	15-min	30-min	1-Hr	2-Hr	3-Hr	6-Hr	12-Hr	24-Hr
2	0.24	0.47	0.66	0.83	0.96	1.03	1.19	1.38	1.58
5	0.36	0.71	0.98	1.24	1.40	1.51	1.71	1.95	2.20
10	0.44	0.87	1.20	1.52	1.70	1.82	2.06	2.33	2.61
25	0.54	1.06	1.47	1.87	2.08	2.22	2.49	2.81	3.13
50	0.63	1.25	1.73	2.19	2.43	2.59	2.90	3.26	3.62
100	0.72	1.42	1.97	2.50	2.77	2.95	3.29	3.68	4.08

Note: Rainfall values from NOAA Atlas 2, Vol. VIII, Arizona

File:SRPRAIN.WK4

this current report lists this same data, as well as rainfall depths for the 2-, 5-, 10- and 25-year events.

The initial ADOT rainfall design criteria for the Pima Freeway offsite drainage system was the 100-year, 24-hour hypothetical storm. This was subsequently reduced to a 12-hour storm because the HEC-1 results from a 12-hour storm simulation (with existing watershed conditions) were found to provide a nearly identical match to the Corp's 100-year discharge of 8,000 cfs at Pima Road and the Arizona Canal.

The 12-hour hypothetical storm distribution is being retained for this updated modeling analysis. However, the Corp's 7-hour distribution for the 1954 Queen Creek storm is also being analyzed in order to make this updated analysis as compatible as possible with the Corp's design assumption for the IBW Interceptor Channel. The Corp's 7-hour storm is also the only event that could be used to make a fair assessment of the design performance of the Scottsdale Ranch detention basin, i.e., the Corp's requirement for this detention basin was to insure that a 100-year, 7-hour storm would not produce more than 8,000 cfs at Pima Road and the Arizona Canal.

The Corp's 7-hour rainfall distribution was developed with the LAPRE1 program (*LAD Preprocessor For HEC-1*), using rainfall pattern curve 3.3. This rainfall pattern curve was taken from Plate 12, **Design Memorandum No. 1, General Design Memorandum - Phase 1, Indian Bend Wash**, U.S.A.C.O.E. October 1973. A 10-year, 6-hour rainfall value of 2.06" and a drainage area of 34.6 square miles were used to enter the chart in Plate 12. The 34.6 square mile drainage area is the sum of the sub-areas listed in Table 3.1, **Design Memorandum No. 4**, U.S.A.C.O.E., January 1980.

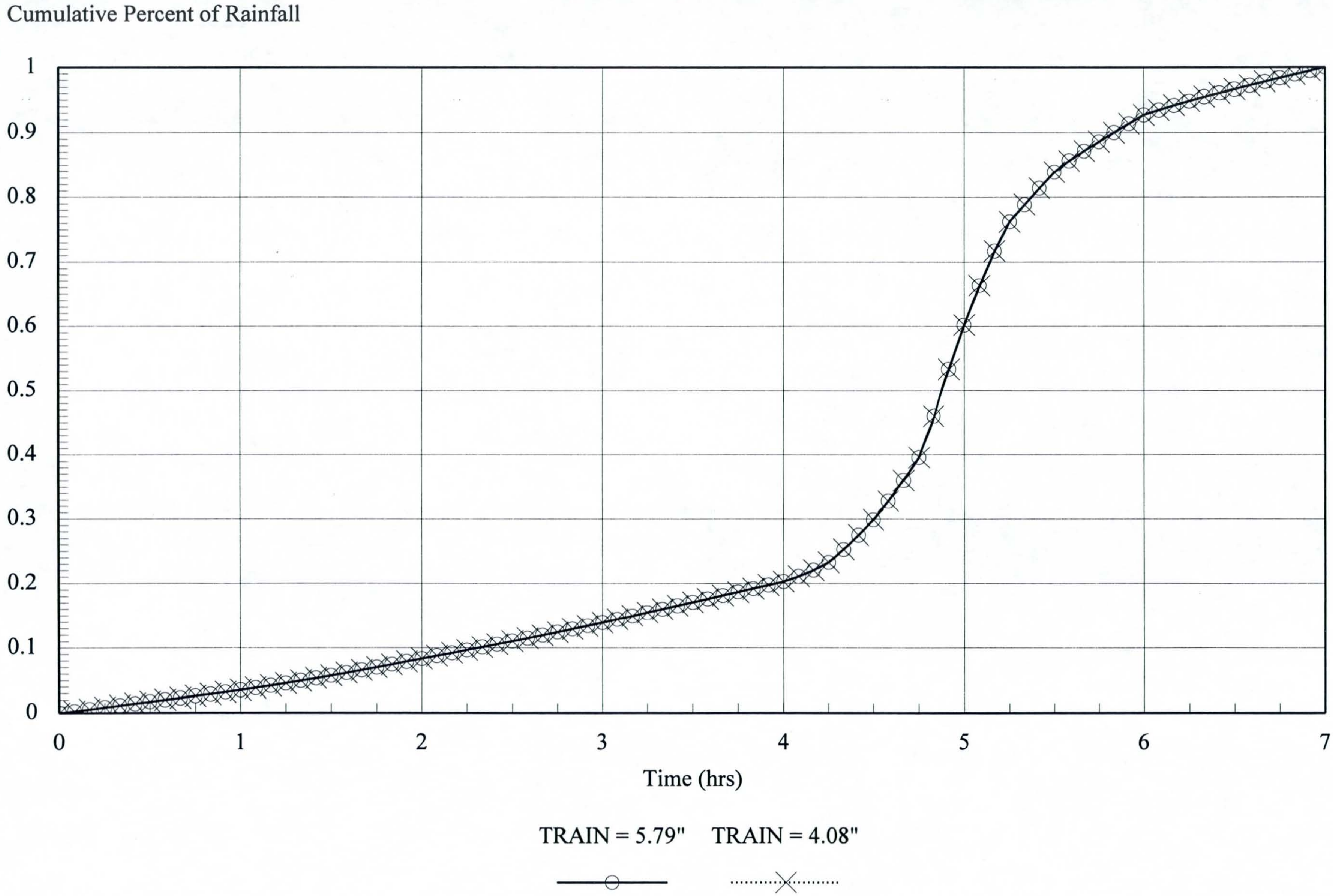
The LAPRE1 program produced a series of PI records that contained extraordinarily high rainfall depths. It was deduced that the LAPRE1 program had somehow generated rainfall <u>intensities</u> rather than <u>depths</u> on the PI records. By definition in the HEC-1 User's Manual, the PI records reflect incremental rainfall depths, not intensities.

The information on the PI records was converted into a dimensionless format that provided the cumulative percent of rainfall at any given time during the 7-hour storm duration. In order to check the accuracy of this procedure, the LAPRE1 program was run for a different total rainfall value and the data generated on the PI records was again converted to cumulative percent of rainfall. The resulting curves from these two runs were found to be identical (see Figure 2.1). Accordingly, the rainfall distribution shown in Figure 2.1 is considered to be a reliable replication of the Corps 7-hour Queen Creek storm distribution.

Table 2.2 lists the coordinates that were used to generate Figure 2.1. The "Cumulative Percent" column listed under "TRAIN" = 5.79" was used to code PC records for input to the HEC-1 model.

The 100-year, 7-hour point rainfall values were interpolated from Figure 2.2, which is a plot of the data in Table 2.1. Two curves are shown in Figure 2.2 . These two curves represent

Figure 2.1
COE 7-Hour Rainfall Distribution
August 1954 Queen Creek Storm



File: COE7HR.PRE

Table 2.2
Conversion of LAPRE1 Output For 7-Hour August 1954 Storm At Queen Creek To Cumulative Percent of Rainfall

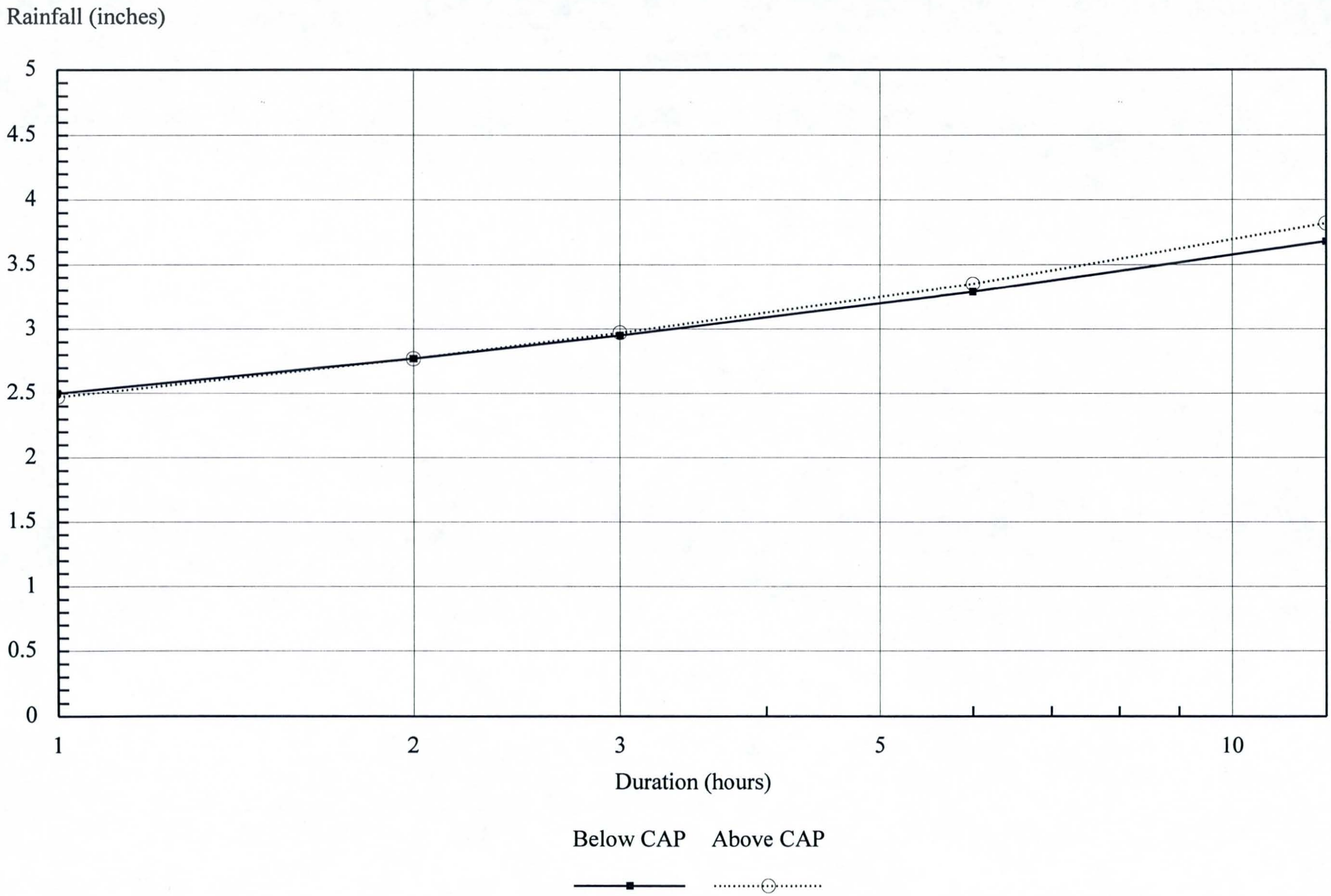
	"TRAIN" = 5.79"		"TRAIN" = 4.08"	
Time (hrs)	PI Record Increment (in.)	Cumulative Percent	PI Record Increment (in.)	Cumulative Percent
0	0	0	0	0
0.0833	0.194	0.00280	0.137	0.00280
0.1667	0.194	0.00559	0.137	0.00560
0.2500	0.199	0.00846	0.140	0.00847
0.3333	0.201	0.01136	0.141	0.01135
0.4167	0.201	0.01425	0.141	0.01424
0.5000	0.201	0.01715	0.141	0.01712
0.5833	0.206	0.02012	0.145	0.02009
0.6667	0.206	0.02309	0.145	0.02305
0.7500	0.208	0.02608	0.146	0.02604
0.8333	0.208	0.02908	0.146	0.02902
0.9167	0.213	0.03215	0.150	0.03209
1.0000	0.220	0.03532	0.155	0.03526
1.0833	0.229	0.03862	0.162	0.03858
1.1667	0.241	0.04209	0.170	0.04205
1.2500	0.253	0.04574	0.178	0.04570
1.3333	0.262	0.04952	0.185	0.04948
1.4167	0.271	0.05342	0.191	0.05339
1.5000	0.280	0.05746	0.197	0.05742
1.5833	0.287	0.06159	0.202	0.06155
1.6667	0.294	0.06583	0.207	0.06578
1.7500	0.296	0.07010	0.209	0.07006
1.8333	0.303	0.07446	0.213	0.07441
1.9167	0.305	0.07886	0.215	0.07881
2.0000	0.305	0.08325	0.215	0.08321
2.0833	0.310	0.08772	0.218	0.08767
2.1667	0.312	0.09222	0.220	0.09217
2.2500	0.312	0.09671	0.220	0.09667
2.3333	0.312	0.10121	0.220	0.10117
2.4167	0.314	0.10573	0.221	0.10569
2.5000	0.314	0.11026	0.221	0.11021
2.5833	0.319	0.11486	0.225	0.11481
2.6667	0.326	0.11955	0.230	0.11952
2.7500	0.333	0.12435	0.235	0.12432
2.8333	0.340	0.12925	0.239	0.12921
2.9167	0.347	0.13425	0.244	0.13420
3.0000	0.349	0.13928	0.246	0.13923
3.0833	0.356	0.14441	0.251	0.14437
3.1667	0.361	0.14962	0.254	0.14956
3.2500	0.368	0.15492	0.259	0.15486
3.3333	0.368	0.16022	0.259	0.16016
3.4167	0.368	0.16553	0.259	0.16546
3.5000	0.368	0.17083	0.259	0.17075
3.5833	0.368	0.17613	0.259	0.17605
3.6667	0.368	0.18144	0.259	0.18135
3.7500	0.370	0.18677	0.260	0.18667
3.8333	0.370	0.19210	0.260	0.19199
3.9167	0.374	0.19749	0.264	0.19739
4.0000	0.374	0.20288	0.264	0.20279
4.0833	0.563	0.21099	0.397	0.21091
4.1667	0.673	0.22069	0.474	0.22060
4.2500	0.832	0.23268	0.587	0.23261
4.3333	1.401	0.25287	0.988	0.25282
4.4167	1.542	0.27509	1.086	0.27503
4.5000	1.656	0.29896	1.167	0.29890
4.5833	2.031	0.32823	1.431	0.32817
4.6667	2.227	0.36032	1.569	0.36027
4.7500	2.483	0.39610	1.749	0.39604
4.8333	4.451	0.46025	3.136	0.46019
4.9167	5.075	0.53338	3.576	0.53333
5.0000	4.764	0.60204	3.357	0.60200
5.0833	4.245	0.66321	2.991	0.66318
5.1667	3.746	0.71720	2.640	0.71718
5.2500	3.138	0.76242	2.211	0.76240
5.3333	1.836	0.78888	1.294	0.78887
5.4167	1.775	0.81446	1.251	0.81446
5.5000	1.720	0.83924	1.212	0.83925
5.5833	1.192	0.85642	0.840	0.85643
5.6667	1.060	0.87170	0.747	0.87171
5.7500	1.004	0.88617	0.707	0.88617
5.8333	0.971	0.90016	0.684	0.90016
5.9167	0.953	0.91389	0.671	0.91389
6.0000	0.927	0.92725	0.653	0.92724
6.0833	0.523	0.93479	0.369	0.93479
6.1667	0.495	0.94192	0.349	0.94193
6.2500	0.468	0.94867	0.330	0.94868
6.3333	0.436	0.95495	0.307	0.95496
6.4167	0.416	0.96095	0.293	0.96095
6.5000	0.402	0.96674	0.283	0.96674
6.5833	0.395	0.97243	0.279	0.97245
6.6667	0.390	0.97805	0.275	0.97807
6.7500	0.390	0.98367	0.275	0.98370
6.8333	0.379	0.98913	0.267	0.98916
6.9167	0.377	0.99457	0.265	0.99458
7.0000	0.377	1.00000	0.265	1.00000
Total:	69.391		48.889	

File: COE7HR.WK4

Figure 2.2

Point Rainfall vs Storm Duration

Pima Freeway at Pima Road & Arizona Canal



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the rainfall data for contributing drainage areas above and below the CAP Aqueduct. The resulting 100-year, 7-hour point rainfall values were 3.38" below the CAP and 3.44" above the CAP. Based on a total drainage area of 34.6 square miles, Figure 14 from the NOAA Atlas was used to convert the point rainfall values to areally reduced values of 3.20" and 3.26" for below and above the CAP, respectively. The areal rainfall reduction factor was 0.948.

It should be noted that the HEC-1 models addressed in this report were run with and without areally reduced rainfall. The original Pima Freeway HEC-1 models were run in a conservative mode of having no areal reduction in rainfall.

2.3 Modeling Refinements

The following subsections provide detailed discussions of the modeling refinements listed in Section 1, as well as some additional refinements (not referenced in Section 1) that were concurrently made to the HEC-1 model.

2.3.1 Newly Urbanized Areas

The City of Scottsdale provided copies of recent (early 1996) HEC-1 models that were prepared for those portions of the project watershed located below the CAP and north of the SRPMIC reservation boundary. HEC-1 routing schematics and sub-basin boundary maps were also provided at a scale of 1" = 1000'. The information provided by the City was part of the City of Scottsdale *Storm Water Master Plan and Management Program.*

This new modeling data was used to replace portions of the original Pima Freeway HEC-1 model that were located generally east of 112th Street and north of the SRPMIC boundary. This area has undergone substantial urbanization since the original Pima Freeway model was prepared. Some minor sub-basin modifications were also made near the southwest corner of Shea Boulevard and 96th Street.

The Scottsdale HEC-1 models were rearranged in order to provide outflow points that would be captured by the remnants of the original HEC-1 sub-basins on the SRPMIC lands. The integration of the Scottsdale models with the original Pima Freeway model required that some of the SRPMIC basins be re-numbered and updated with revised drainage area sizes and channel routing parameters. The updated SRPMIC sub-basins are 1155A, 1165A, 1177A, 1187A, 1204A, and 1217A.

The revised HEC-1 routing schematic is shown on Plate 1. Technical documentation for the City of Scottsdale models, referenced on Plate 1, is available through the City.

2.3.2 Scottsdale Ranch Detention Basin Analysis

2.3.2.1 Basin Description

As constructed, the Scottsdale Ranch detention basin consists of two distinct storage areas. The first component of this detention basin consists of a permanent lake that provides about 303 AF of storage at elevation 1374.0-ft MSL. This lake outlets to a recreational park through a non-level weir that is constructed as a concrete jogging path. The estimated low point on this weir is at about elevation 1369.75-ft MSL. Based on topographic contour lines, the lake appears to be at about elevation 1368.0-ft MSL.

The recreational park provides an additional 74 AF of storage at elevation 1374.0-ft MSL. This park contains 1 - 8'W x 2.5'H CBC and 1-24" RCP to drain the park of floodwaters. The RCP is set at the bottom of the park area while the CBC is about 3-feet above the bottom of the park.

Table 2.3 lists the stage/storage data developed for the park and lake areas. This data is plotted in Figure 2.3.

Since the lake and park areas are hydraulically connected during flood events, a single, combined stage/storage relationship was used to model the detention basin performance. The associated stage/discharge relationship was based on inlet control nomographs for the CBC and RCP outlets in the park area.

The drainage area captured by the Scottsdale Ranch detention basin was not previously modeled in the original Pima Freeway HEC-1 model. As stated previously, an outflow hydrograph was simply inserted in the model to simulate the outflow as documented in the WRA and CWW reports.

This updated model uses the 1996 City of Scottsdale HEC-1 models to provide an inflow hydrograph to the detention basin. This drainage area is contained within City of Scottsdale Basin 28. As will be discussed in a subsequent section of this report, the CAP overchute hydrograph at CP 2110 in the Scottsdale model, has been replaced with a new HEC-1 model and reservoir routing operation for the contributing drainage area located above the CAP. This overchute point is identified as CP CACTUS.

2.3.2.2 Basin Performance

Under existing conditions, the Scottsdale Ranch detention basin was found to overtop the embankment elevation of 1374.0-ft MSL. The combined outflow from the existing CBC, RCP and embankment overtopping was found to exceed 200 cfs for all modeling scenarios. This situation occurs in spite of the fact that the City appears to have fulfilled their obligation to the Corps to provide a minimum of 357 AF of flood

Table 2.3
Stage/Storage Data For Scottsdale Ranch Detention Basin
Scottsdale, Arizona

Based on 1993 Scottsdale Topographic Mapping

Area Conversion Factor:
0.935027983143

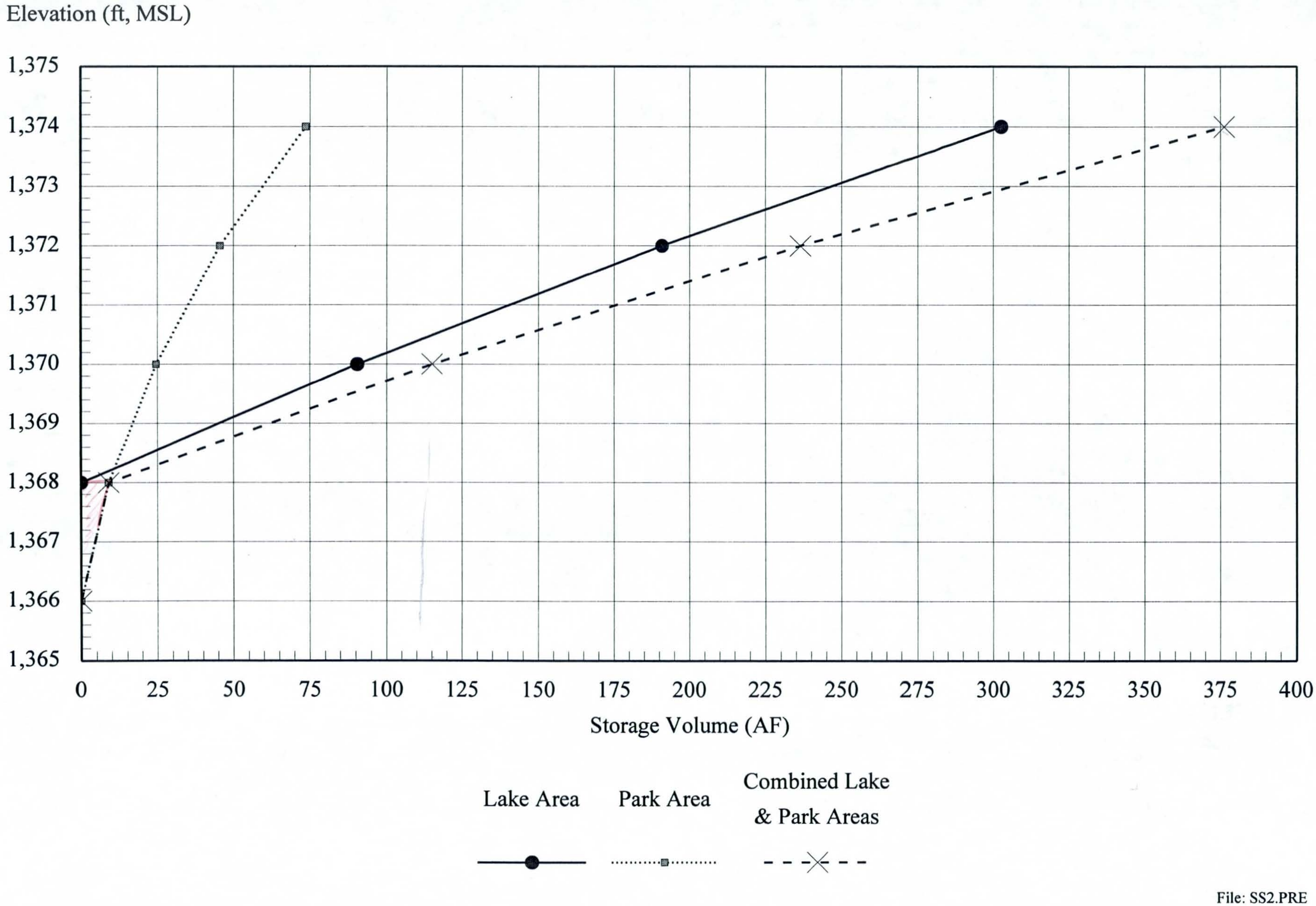
Elevation (ft, MSL)	Planimetered Area (sq in)	Surface Area (acres)	Average End Area (acres)	Elevation Increment (ft)	Incremental Volume (AF)	Total Volume (AF)
Lake Area						
1368	45.56	42.5999				0
			45.2460	2.0	90.4920	
1370	51.22	47.8921				90.4920
			50.2017	2.0	100.4033	
1372	56.16	52.5112				190.8953
			55.8726	2.0	111.7452	
1374	63.35	59.2340				302.6405 ✓ above 1368
Park Area						
1366	2.58	2.4124				0
			4.4601	2.0	8.9202	
1368	6.96	6.5078				8.9202
			7.8355	2.0	15.6711	
1370	9.80	9.1633				24.5912
			10.4957	2.0	20.9914	
1372	12.65	11.8281				45.5826
			13.9974	2.0	27.9947	
1374	17.29	16.1666				73.5774 ✓ above 1366
Combined Lake & Park Area (storage below Elevation 1368 is confined to the Park area)						
1366						0
			4.4601	2.0	8.9202	
1368	52.52	49.1077				8.9202
			53.0815	2.0	106.1631	
1370	61.02	57.0554				115.0833
			60.6973	2.0	121.3947	
1372	68.81	64.3393				236.4780
			69.8700	2.0	139.7399	
1374	80.64	75.4007				376.2179

File: SCTLK1.WK4

Figure 2.3

Stage - Storage Curves For Scottsdale Ranch Detention Basin

Based on 1993 City of Scottsdale Topography



control storage at this location. Using inlet control nomographs, the combined CBC and RCP outflow, at the top of embankment elevation of 1374.0-ft MSL, is 211 cfs, which is within 5.5% of the maximum 200 cfs outflow required by the Corps of Engineers.

As a matter of technical interest, it should be noted that the Corp's hydrograph for Sub-Area 3, **Design Memorandum No. 4** (Table 3-4), January 1980, was also routed through this detention basin and was found to cause slightly more overtopping (and a higher outlet discharge) than that from the City of Scottsdale hydrographs. Outflows from the CAP Cactus Road detention basin were added to the Corp's hydrograph in order to make an equivalent comparison with the City model.

Figure 2.4 presents a plot of the inflow hydrographs to the Scottsdale Ranch detention basin. This Figure provides a visual comparison of the hydrograph shapes developed from this study, as compared to the Corp's hydrograph from Sub-Area 3. The Corp's hydrograph plot in Figure 2.4 does <u>not</u> include the outflow from the Cactus Road Basin, however, the HEC-1 model <u>does</u> combine this outflow with the Corp's hydrograph.

Table 2.4 presents a summary of the detention basin performance for the different HEC-1 scenarios. The data in Table 2.4 is based on the 1991 version of HEC-1.

Several options may be available to correct the embankment overtopping problem. For example, the size of the outlet structures could be increased, in conjunction with lowering the invert of the box so that it would begin to evacuate the basin at an earlier point on the rising limb of the inflow hydrograph.

Based on a preliminary analysis, it was found that adding a second 8'W x 2.5'H CBC, and setting both box inverts 2-feet lower than the existing box, would provide a floodpool elevation of 1373.59-feet MSL during a 100-year, 7-hour storm. This would be below the 1374.0-ft MSL embankment elevation. The maximum basin outflow for this condition was 446 cfs, which still exceeds the 200 cfs limit prescribed by the Corps.

The outflow channel from the Scottsdale Ranch detention basin discharges onto SRPMIC lands at a point about 1,100' east of 96th Street. Assuming that the downstream outlet channel (through the City area) could adequately convey 446 cfs, without overtopping, such a discharge will not cause the 8,000 cfs limit to be exceeded (using HEC-1 Models PF2.12I or PF4.7I) at the intersection of Pima Road and the Arizona Canal.

However, as these flows move south, they will probably be captured by the 96th Street storm drain project that has been initiated by SRPMIC. Accordingly, unless the City of Scottsdale initiates action to decrease the outflow from the Scottsdale Ranch detention basin, the SRPMIC storm drain system design may have to be

Figure 2.4
100-Year Inflow Hydrograph
Scottsdale Ranch Detention Basin

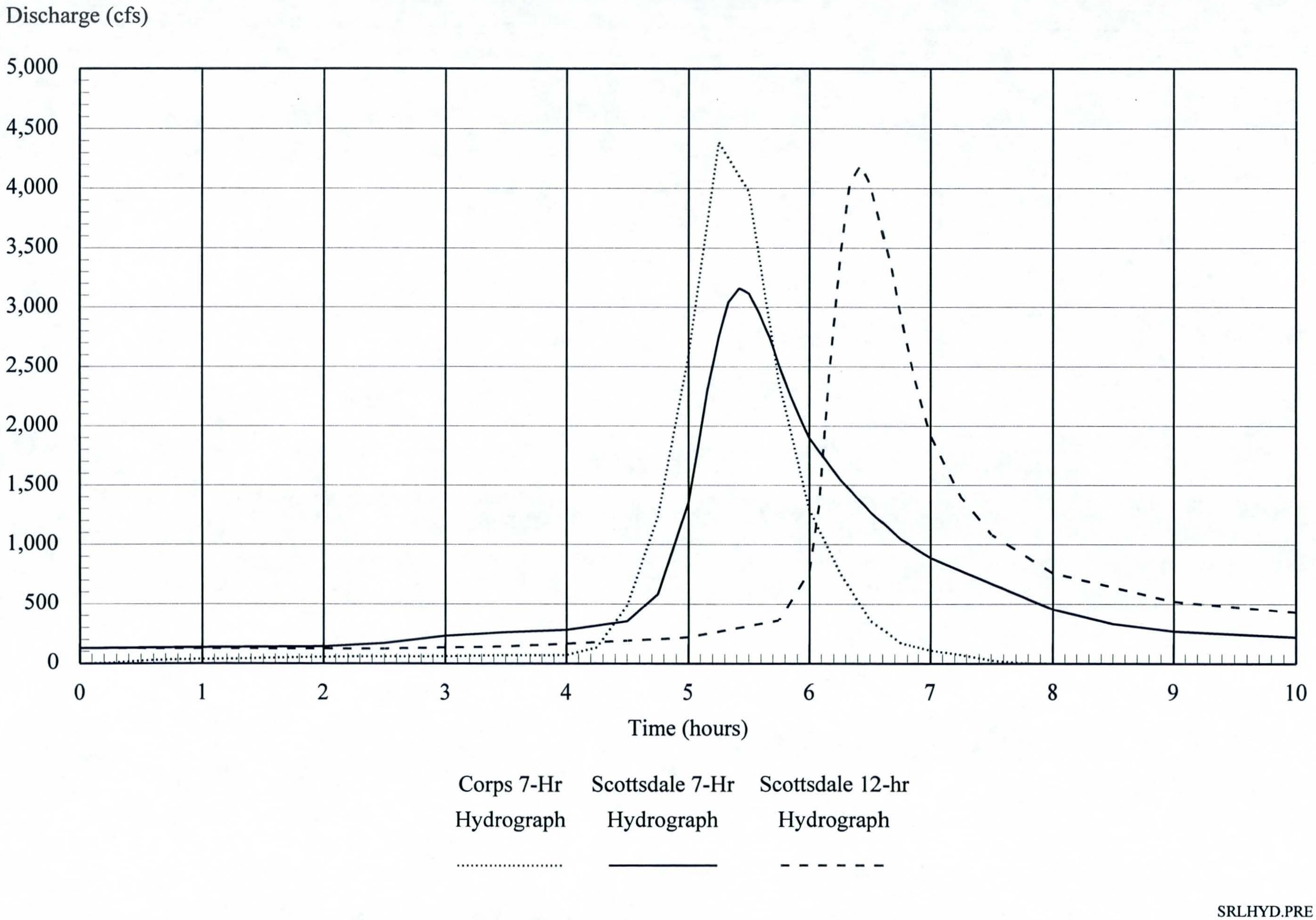


Table 2.4
Performance Summary
Scottsdale Ranch Detention Basin
Scottsdale, Arizona

All Data Based On HEC-1 Version 4.0.1E, May 1991

HEC-1 Model	Areal Reduction Used For Rainfall	Design Storm	Maximum Floodpool Elevation (ft, MSL)	Maximum Discharge (cfs)	Basin Overtop
PF1.12I	No	100-Year, 12-hour	1374.37	878	Yes, 0.37'
PF2.12I	Yes	100-Year, 12-hour	1374.26	606	Yes, 0.26'
PF3.7I	No	100-Year, 7-hour	1374.36	843	Yes, 0.36'
PF4.7I	Yes	100-Year, 7-hour	1374.27	621	Yes, 0.27'
COEHYD.7I (C.O.E. Hydrograph)	unknown	100-Year, 7-hour	1374.32	749	Yes, 0.32'

Note: All elevations referenced to 1993 City of Scottsdale topographic mapping.
Basin overtopping begins at about elevation 1374.0-ft MSL.
Existing outlet consists of 1-8'W x 2.5'H CBC & 1-24" RCP.

File: SRDB1.WK4

upgraded to insure that it can safely handle such flows. Section 2.3.2.3 provides a more detailed discussion of this potential problem.

There are numerous combinations of outlet structure geometry and invert elevations that could be evaluated in order to find an optimal solution that could correct the existing problem at the Scottsdale Ranch detention basin. The City of Scottsdale has been made aware of this problem and is actively committed to resolving this problem.

2.3.2.3 Detention Basin Overflows To 96th Street / Via Linda Intersection

As discussed in the preceding section, if the Scottsdale Ranch Detention Basin overtops, or if the capacity of the outlet channel is exceeded, the excess flows will be captured by Via Linda Drive and carried west. This potential flow diversion created by Via Linda raises a concern about possible impacts to the SRPMIC storm drain system at 96th Street, as well as possible impacts to the Pima Freeway drainage system at the intersection with Via Linda Drive.

In order to evaluate this potential problem, a modified HEC-1 model was created to divert a portion of the outflows from the Scottsdale Ranch Detention Basin to 96th Street. HEC-1 Model PF2.12I (100-year, 12-hour hypothetical storm, with areal rainfall reduction) was used for this analysis. The divert operation was inserted at CP 233, which is the location where the detention basin outlet channel crosses under Via Linda Drive. The divert ratio at this location was set to direct any flows above 200 cfs in a westerly direction along Via Linda to 96th Street. The 200 cfs limit was based on the maximum detention basin discharge assumed by the Corps of Engineers in the original design of the IBW Interceptor Channel.

This divert operation produced a 100-year, 12-hour peak discharge of 434 cfs at the intersection of 96th Street and Via Linda Drive. This peak discharge occurs 8.58-hours into the 12-hour storm. Without this divert operation in-place, the original PF2.12I HEC-1 model produced a peak discharge at this intersection of 422 cfs at 6.25-hours into the 12-hour storm. Since the diverted flows from the detention basin arrive over 2-hours later than the natural runoff that reaches this intersection, there is very little increase in the peak discharge at the intersection, i.e., there will be a double-peaked inflow hydrograph at this intersection, with the peaks separated by over two hours.

Based on site inspections and a review of 2-foot contour interval maps prepared in 1993, a simple hydraulic analysis was performed to evaluate the capacity of this intersection to cope with inflows in excess of 400 cfs. The contour maps, and field inspection, indicated that Via Linda Drive rises about 2.5-feet within a distance of about 675-feet west of the 96th Street intersection. Accordingly, there is a significant sag at the intersection which should prevent any flows from moving west of this intersection.

Field measurements of the four existing culverts in the detention basin outlet channel (from Mountain View Lake Drive to Via Linda Drive) revealed a minimum culvert capacity of 304 cfs occurs at Mountain View Drive. The culvert capacities were based on inlet control with a maximum headwater depth measured to a point on the culvert headwall where overtopping would occur into the street. No water surface profile was computed between culverts to verify the available channel capacity.

Under natural conditions (with no diversions from the Scottsdale Ranch Detention Basin), the majority of the inflow to this intersection will arrive in a drainage channel that parallels the east side of 96th Street. Immediately south of Via Linda Drive, this channel is about 5-feet deep with a bottomwidth of 12-feet. The bankfull capacity of this channel was estimated at about 980 cfs ($z = 3:1$, $S = .0033$ ft/ft, $n = .025$). The slope of this channel was difficult to measure from the available contour mapping. However, even at an extremely conservative flat slope of 0.0006 ft/ft, the bankfull capacity is 434 cfs. Accordingly, under natural conditions, the combined street and channel flow, along the 96th Street alignment, should easily be capable of conveying 422 cfs through this intersection without any threat of overtopping the elevated crest of Via Linda Drive that is located about 675-feet west of the intersection of 96th Street. These flows will then be delivered to the SRPMIC boundary, along the 96th Street alignment.

Under the "diverted flow" scenario from the Scottsdale Ranch Detention Basin, the second peak of the inflow hydrograph to this intersection will arrive as street flow. The hydraulics of passing this flow through the intersection will be complex. There are two 40.5" x 22" storm drain inlets at the north and south sides of Via Linda, at the east side of the 96th Street intersection. These storm drain inlets discharge to the previously discussed 96th Street. Water that is not captured by the storm drain inlets will flow into the street intersection. Some temporary ponding may occur before these flows begin moving south along 96th Street.

The east lane of 96th Street, south of Via Linda, has a minimum width of 26-feet and a 6-inch curb height. At an approximate slope of 0.0033 ft/ft ($n = 0.14$), the street can carry about 49 cfs before the curbs would be overtopped. Overtopping of the east curb would allow water to enter the 96th Street Channel. Overtopping of the west curb would allow water to enter the west lane of 96th Street. One 26-foot wide street lane can carry 434 cfs of water at a depth of 1.93-feet.

Accordingly, under the worst-case assumption that <u>all</u> of the Via Linda inflows would be confined to a single 26-foot wide street section, the flow depth would still be below the 2.5-foot rise on Via Linda that is located 675-feet of the 96th Street intersection. This calculation indicates that the Pima Freeway intersection with Via Linda Drive should be isolated from any additional inflows due to overtopping of the Scottsdale Ranch Detention Basin during a 100-year, 12-hour storm.

These diverted flows will cause only a slight increase in the flows delivered to the SRPMIC 96th Street storm drain system. As discussed previously, this minimal impact at the SRPMIC boundary is due to the late arrival time of the overflows from the Scottsdale Ranch Detention Basin to the 96th Street intersection with Via Linda Drive.

In conclusion, this analysis indicates that flow diversions from the Scottsdale Ranch Detention Basin should have no impact on the Pima Freeway drainage system at Via Linda and a very minimal impact to the upstream end of the proposed SRPMIC storm drain system along 96th Street. However, any detention basin overtopping could potentially cause major street flooding along Via Linda Drive and possible flood damage to adjacent residential structures.

2.3.3 CAP Overchute Hydrology

There are four CAP overchute locations that can contribute runoff to the intersection of Pima Road and the Arizona Canal. These overchute locations are listed as follows.

1. CAP Station 135+00 (MP 177.922): 1-48" CMP
2. CAP Station 234+10 (MP 179.801): 3-72" steel pipes.
3. CAP Station 243+00 (MP 179.970): 1-36" RCP
4. CAP Station 332+75 (MP 181.670): 2-66" steel pipes

The following subsections discuss the updated hydrology and reservoir routing operations that were used for each of these crossings.

2.3.3.1 CAP Station 135+00

The 48" CMP at this location drains a large detention basin. A detailed HEC-1 simulation of this basin was previously published as **Attachment No. 1, Technical Support Data For LOMR Request at CAP Cactus Road Detention Basin, Scottsdale, Arizona**, April 11, 1994, Robert L. Ward, P.E., Consulting Engineer. This report was approved by the City of Scottsdale and FEMA and was used to revise the floodplain maps for this area.

The rainfall data in this model was revised to reflect the 7-hour and 12-hour storm distributions referenced in Section 2.1 of this report. The resulting outflow hydrographs from the detention basin were input with QI records to the updated Pima Freeway HEC-1 model. QI records were used instead of attaching the entire model to the Pima Freeway model in order to prevent tracking more than 9 active hydrographs in the HEC-1 routing diagram for the Pima Freeway model.

The peak outflow from this basin was on the order of 180 to 190 cfs for the 7-hour and 12-hour storms. The outflow from this detention basin is routed to the Scottsdale Ranch detention basin.

2.3.3.2 CAP Station 234+10

The overchute structure at this location consists of 3-72" steel pipes. The contributing drainage area to this location was previously modeled as part of the **General Drainage Plan for North Scottsdale, Arizona**, June 7, 1989, Water Resources Associates, Inc.

The future land-use condition model from this 1989 study (File: GN60.24I) was combined with stage/storage/discharge data from Figures 4 and 5 published in the **Master Drainage Plan, Stonegate**, April 1988, Carter Associates, Inc., in order to develop an outflow hydrograph for this overchute location.

Only those sub-basins from HEC-1 file GN60.24I that contribute to this overchute location were used. All rainfall data was adjusted to match the 7-hour and 12-hour storm data discussed in Section 2.1 of this report.

For the same reasons stated for CAP Station 135+00, QI records were again used to input the outflow hydrograph from this overchute to the Pima Freeway model.

The peak outflow from this overchute was on the order of 1100 cfs to 1400 cfs for the 7-hour and 12-hour storms.

2.3.3.3 CAP Station 243+00

This location consists of a single 36" RCP which drains about 0.1 square miles. A single sub-basin was used to define the inflow hydrograph to this culvert.

Stage/storage/discharge data for this overchute was taken from Figures 6 and 7 in the April 1988 Carter Associates report. The peak discharges from this overchute location were on the order of 25 to 35 cfs.

2.3.3.4 CAP Station 332+75

This overchute location, which is near the eastern boundary of the project watershed, consists of 2-66" pipes. The location of this overchute raises a question as to whether it will actually contribute runoff to the Pima Road - Arizona Canal intersection.

Based on a review of aerial photographs, a decision was made to allocate 15% of the overchute outflow to the project watershed. This assumption is documented on page 4 of the May 1989 SLA **Hydrology Report**.

No stage/storage information was available for this site. The Bureau of Reclamation data for this site indicated a peak inflow of 1,107 cfs and a peak outflow of 437 cfs (telephone communication with Chris Brechler on 5/5/89). This information was used to manufacture a hydrograph that had a 17-hour time base and a constant 2-hour peak discharge of 437 cfs.

This artificial hydrograph was positioned (via QI records) so that the 2-hour peak outflow discharge occurred at about the same time as that from the watershed at CAP overchute Station 234+10, i.e., between hours 5 to 7 for both the 12-hour and the 7-hour storms.

2.4 Overflow Along Arizona Canal Embankment

As part of the IBW Interceptor Channel design, the Corps of Engineers analyzed the hydraulic capacity of the Arizona Canal and north overbank area between Alma School Road and Pima Road. The results of this analysis are documented on Plate 5 in the previously referenced January 1980 **Design Memorandum No. 4**.

The Corp's analysis indicates that the south bank of the Arizona Canal would be overtopped when the combined canal and overbank flow enters the 6,000 to 10,000 cfs range, east of 96th Street. For example, the maximum capacity about 500-feet west of Alma School Road is 6,000 cfs while the maximum capacity at a point midway between North Longmore Road and North Dobson Road is listed as greater than 10,000 cfs.

The information on Plate 5 was used to create a divert routine, in the updated HEC-1 model, that would eliminate any flow from the model that exceeds 6,500 cfs at North Longmore Road (CP 1987). This location corresponds to Corps Station 30+00 (on **D.M. No. 4**, Plate 5), which lists a maximum canal and overbank capacity of 6,500 cfs.

The proposed SRPMIC extension of the ACDC begins at 96th Street (1/2 mile west of North Longmore Road) and, therefore, eliminates the possibility of any additional overtopping of the south canal bank between 96th Street and Pima Road. The minimum canal and overbank capacity between North Longmore Road (CP 1987) and 96th Street is 7,000 cfs. The HEC-1 models in this report do not produce discharges that exceed 7,000 cfs between North Longmore Road and 96th Street.

By letter dated August 1, 1996, the Flood Control District of Maricopa County (FCDMC) notified SRPMIC that the flows reaching North Longmore Road must be limited to no more that 6,500 cfs in order to receive FCDMC approval for connecting the ACDC to the IBW Interceptor Channel. Accordingly, any future development on those SRPMIC properties located north of the Arizona Canal, between North Longmore Road and Alma School Road, must include sufficient hydrologic and hydraulic analyses to insure that this 6,500 cfs constraint is not exceeded. Such analyses might demonstrate a need to design and build new drainage structures that will insure that no more than 6,500 cfs is allowed to pass to the west of North Longmore Road. If this FCDMC criteria were not to be complied with, it is possible that the capacity of the ACDC might be exceeded, which could also cause a potentially unacceptable increase in the discharge to the IBW Interceptor Channel, west of Pima Road.

2.5 Additional Onsite Flows To Pima Freeway Channel

During late 1989, DeLeuw, Cather & Company (DCC, management consultant to ADOT) requested that some additional flows be piped into the Freeway offsite drainage channel near Via de Ventura (CP 503C). These flows, which originate north of the SRPMIC boundary, were to be routed to this location through 7,000 L.F. of 4.5-foot diameter pipe. DCC provided an outflow hydrograph from this pipe that had a peak discharge of 120.6 cfs occurring at 106 minutes after the start of runoff.

As part of this updated analysis, the current design consultants for the Pima Freeway were contacted in order to verify that this flow was still being piped into the offsite channel near Via de Ventura. Discussions with Wood, Patel & Associates, Inc. on June 21, 1996, verified that 125 cfs of onsite Freeway drainage is being delivered to an 84" pipe that starts near Via Linda. This 125 cfs originates from onsite Freeway runoff generated from Via Linda to 4,265-feet north of the Camelback Walk Channel. The outfall location for this onsite runoff is still at the Pima Freeway near Via de Ventura (CP 503B).

To reflect this updated information, the ordinates for the original 120.6 cfs hydrograph were increased by a factor of 125/120.6 = 1.036. The pipe routing length was increased from 7,000-feet to 12,365-feet and the routing pipe diameter was increased from 4.5-feet to 7.0-feet. This updated, onsite runoff hydrograph is identified in the HEC-1 model as CP 503A.

The resulting hydrograph was positioned in the model so that the peak inflow to the pipe occurred at the same time as that from nearby Sub-Basin 100, which has a similar peak discharge of 119 cfs. A kinematic wave routing operation (CP 503B) then routed the inflow hydrograph to the offsite drainage channel at Via de Ventura, where it is combined with other flows at CP 503C.

2.6 Onsite Retention/Detention Storage

Nearly all of the commercial and residential development within the City of Scottsdale portion of the watershed contains some level of onsite retention or detention storage for floodwater runoff. The modeling assumptions used to simulate this storage have been previously documented in Section 4.3.4 of the May 1989 SLA **Hydrology Report**.

Numerous reservoir routing operations are also included in the 1996 City of Scottsdale modeling that was incorporated into this updated HEC-1 analysis. Documentation for these storage routing operations are assumed to be on file with the City of Scottsdale.

Most of the storage basins were designed on the basis of storms of <u>lesser magnitude</u> than the 100-year, 12-hour or 7-hour events. Accordingly, such basins frequently overtop when subjected to these larger storms.

Due to limitations within the HEC-1 program code, embankment overtopping is simply computed as a function of the weir equation. The energy head used in the weir equation is interpolated from the stage/storage data that is input to the model for each reservoir routing operation. The program has no mechanism with which to check the computed weir flow discharge against the peak inflow discharge. As a result, the computed peak outflow rate can often exceed the peak inflow rate.

A review of the HEC-1 output data indicated that some of the small retention basins were experiencing this problem. Although there was a very small amount of error involved at any given detention basin (1 to 20 cfs), the 11 offending retention basins were de-activated in the HEC-1 model. The data records have been left in the model but de-activated by placing an asterisk and a blank space at the left of each record. Accordingly, should one ever want to model smaller storms that would not cause basin overtopping, the reservoir routing operations can easily be re-activated. It should be emphasized that the de-activation of these detention basin operations is a conservative assumption that will lead to slightly higher downstream discharges than those which might actually occur during a storm.

3 CONCLUSIONS

Table 3.1 presents a summary of the total 100-year discharges that reach the east side of Pima Road, at the Arizona Canal. These discharges, which include combined canal and north overbank flow, reflect all the modeling assumptions discussed in the preceding sections of this report.

Each of the HEC-1 models listed in Table 3.1 contain an activatable Concentration Point (CP) 548.1. Under normal conditions, this CP is inactive. When the asterisks are removed from each record at this CP, the model will combine the flow (at Pima Road) in the Arizona Canal with that in the proposed ADOT channel, thus providing a combined hydrograph for the total flow at this location. Accordingly, the discharges in Table 3.1 indicate whether the maximum 8,000 cfs criteria at Pima Road and the Arizona Canal is being complied with.

It should be noted that activating CP 548.1 will invalidate the remaining two operations in the model at "DIV 550" and "CP 552". Accordingly, these two remaining operations <u>must</u> be

Table 3.1
100-Year Peak Discharge Summary
Pima Road & Arizona Canal
Scottsdale, Arizona
(revised 8/14/96)

HEC-1 Model	Areal Reduction Used For Rainfall	Design Storm	HEC-1 Program Version	Peak Discharge at CP 548.1 (Pima Road & Arizona Canal) (cfs)
PF1.12I	No	100-Year, 12-hour	1985 1991, version 4.0.1E	8,004 8,282
PF2.12I	Yes	100-Year, 12-hour	1985 1991, version 4.0.1E	7,170 7,886
PF3.7I	No	100-Year, 7-hour	1985 1991, version 4.0.1E	7,727 8,263
PF4.7I	Yes	100-Year, 7-hour	1985 1991, version 4.0.1E	7,011 7,719
Corps of Engineers	unknown	100-Year, 7-hour	unknown	8,000

Note: Proposed Pima Freeway & 96th Street storm drain are assumed to be in-place.

File: SRPSUM1.WK4

de-activated (i.e., add an asterisk and a blank space in front of each HEC-1 input record) or the program will not run to a successful completion when invoking the "CP 548.1" option.

A review of the data in Table 3.1 indicates that 5 of the 8 modeling scenarios meet the "not to exceed" 8,000 cfs criteria at Pima Road and the Arizona Canal. The only three scenarios that do not meet this criteria use no areal rainfall reduction for the 35 square mile watershed.

The size of the drainage area would seem to justify the use of an areal reduction factor, as discussed in Section 2.1 of this report. Accordingly, it is recommended that HEC-1 model **PF4.7I** be accepted as documentation that the proposed Pima Freeway drainage system will not cause the 8,000 cfs limit to be exceeded at Pima Road.

In order to conform to previously accepted ADOT design criteria, it is further recommended that HEC-1 model **PF2.12I** be used for the design of the Pima Freeway drainage system.

This recommended approach is compatible with the Corp's use of a 7-hour storm for the design of the IBW Interceptor Channel, and ADOT's use of the 12-hour storm for the Pima Freeway drainage system.

Since the allowable inflow to the IBW Interceptor Channel is the critical design criteria for this watershed, it would seem logical to recommend that the hydrologic impact of all future land-use changes/improvements, within that portion of the watershed located within the City of Scottsdale and the SRPMIC boundaries, be based on model **PF4.7I** (Corp's design storm), using HEC-1 Version 4.0.1E, 1991. Under this recommendation, any future land-use changes in the project watershed would not be allowed to cause the 100-year, 7-hour peak discharge to exceed 8,000 cfs at CP 548.1 (Pima Road and the Arizona Canal) when using model PF4.7I. In conjunction with this 8,000 cfs limitation, it is also imperative that SRPMIC enforce the provision that no more than 6,500 cfs be allowed to pass west of North Longmore Road (along the Arizona Canal embankment) during a 100-year storm (see Section 2.4).

In summary, this report verifies that the proposed Pima Freeway drainage system, and associated SRPMIC 96th Street storm drain system, will not cause the 100-year, 7-hour peak discharge of 8,000 cfs to be exceeded at Pima Road and the Arizona Canal when using the HEC-1 model recommended in this report.

APPENDIX A

HEC-1 Output Summary

Pima Freeway
Arizona Canal Drainage Channel

Model: PF4.7I

HEC-1 Version 4.0.1E
May 1991

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****************************************                                       **************************************
*                                      *                                       *                                    *
*  FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                       *  U.S. ARMY CORPS OF ENGINEERS      *
*             MAY   1991               *                                       *  HYDROLOGIC ENGINEERING CENTER     *
*          VERSION 4.0.1E              *                                       *        609 SECOND STREET           *
*                                      *                                       *     DAVIS, CALIFORNIA 95616        *
* RUN DATE  08/20/96  TIME  14:36:26   *                                       *         (916) 551-1748             *
*                                      *                                       *                                    *
****************************************                                       **************************************

                  *************************************************************
                  *                                                           *
                  *                   ROBERT L. WARD, P.E.                    *
                  *                   CONSULTING ENGINEER                     *
                  *                                                           *
                  *                      PIMA FREEWAY                         *
                  *                (NORTH OF THE ARIZONA CANAL)               *
                  *                                                           *
                  *     SCOTTSDALE PAVILIONS & PIMA FREEWAY ARE IN-PLACE      *
                  *                                                           *
                  *           HYDROLOGY ANALYSIS FOR OFFSITE DRAINAGE         *
                  *                     100 YEAR EVENT                        *
                  *           7 HOUR COE QUEEN CREEK STORM DISTRIBUTION       *
                  *                        ARF=0.948                          *
                  *                                                           *
                  *     USES % IMPERVIOUS COVER TO SIMULATE DEVELOPMENT       *
                  *         INCLUDES RETENTION/DETENTION BASINS FOR           *
                  *              RESIDENTIAL & COMMERCIAL AREAS               *
                  *                                                           *
                  *                                                           *
                  *           THIS MODEL INCLUDES THE ADDITIONAL AREA         *
                  *         PREVIOUSLY MODELED BY THE CORP OF ENGINEERS       *
                  *                                                           *
                  * SEE SECTION 4.3.4 OF THE MAY 1989 SLA FINAL HYDROLOGY RPT. *
                  * OUTER LOOP HWY, CAMELBACK WALK CHANNEL TO ARIZONA CANAL.   *
                  * FOR DISCUSSION ON ONSITE RETENTION/DETENTION ASSUMPTIONS  *
                  *                                                           *
                  *                                                           *
                  *     ALL CURVE NUMBERS ARE FOR 24-HOUR STORM DURATION      *
                  *     NORMAL DEPTH STORAGE ROUTING ALONG ARIZONA CANAL      *
                  *       INCLUDES CAP CROSS-DRAINAGE AT EAST BOUNDARY        *
                  * BASE MODEL USES APRIL 1989 HYDROLOGIC REVISIONS BY R. WARD *
                  * MARCH 1989 HIGHWAY ALIGNMENT HAS BEEN REVISED TO MID 1989 *
                  *                    HIGHWAY ALIGNMENT.                     *
                  *                                                           *
                  *                      MODEL PF4.7I                         *
                  *                (Base Model Was OL1L.12I)                  *
                  *                                                           *
                  *      THIS MODEL SIMULATES THE IMPACT TO THE PIMA FWY      *
                  *      OF A NORTH/SOUTH CHANNEL ALONG AN EXTENSION OF       *
                  *      96th STREET FROM THE NORTH BOUNDARY OF THE SRPMIC    *
                  *      TO THE ARIZONA CANAL                                 *
                  *      THIS MODEL ALSO SIMULATES THE IMPACT OF THE PROPOSED *
                  *      EVANS-KUHN CHANNEL ALONG THE NORTH BANK OF THE ARIZONA*
                  *      CANAL BETWEEN 96th ST & THE PIMA FREEWAY             *
                  *                                                           *
                  *  DAM 1 ROUTING SEQUENCE HAS BEEN CORRECTED ON 7/29/96 TO  *
                  *  ONLY DETAIN RUNOFF FROM SUB-BASINS 185 & 195.            *
                  *                                                           *
                  *  ADOT BERM HAS BEEN REMOVED & THE EVANS-KUHN CHANNEL HAS  *
                  *  BEEN CONNECTED DIRECTLY TO THE ADOT CHANNEL              *
                  *                                                           *
                  *  DIVERT HAS BEEN ADDED AT CP 1987 TO REFLECT COE          *
                  *  CANAL SPILLS FROM PLATE 5 OF D.M. No. 4.                 *
```

```
*  SEPARATE CHANNEL & CANAL ROUTING OPERATIONS BEGIN AT      *
*  CP 546T TO REFLECT SPLIT-FLOW CALCULATIONS.               *
*  CANAL "n"=.021                                            *
*  DIVERT AT 545E HAS BEEN CHANGED TO PUT 50% TO NORTH-SOUTH *
*  ADOT CHANNEL & 50% TO CP 546T.                            *
*                                                            *
*  13 DETENTION BASINS HAVE BEEN REMOVED THAT WERE ADJACENT  *
*  TO THE OUTER LOOP.  MODIFIED PULS CHANNEL ROUTING         *
*  OPERATIONS ARE USED IN-PLACE OF THE BASINS.               *
*  CHANNEL GEOMETRY IS BASED ON AVERAGED. APPROXIMATE XSEC   *
*  GEOMETRY USED BY MKE FOR FINAL DESIGN                     *
*                                                            *
*               CP 103 IS ROUTED TO SUB 360.                 *
* ALL SUB-BASIN DATA & ROUTING OPERATIONS HAVE BEEN REVISED  *
* TO REFLECT THE HIGHWAY ALIGNMENT CHANGE MADE BY DCCO IN    *
* MID-1989.  ADDITIONAL ONSITE DRAINAGE HAS BEEN ROUTED      *
*THROUGH 12.365' OF PIPE AND INSERTED IN THE MODEL AT CP 503B *
*    WHICH IS AT THE NORTH SIDE OF VIA DE VENTURA            *
*                                                            *
* NORTH-SOUTH CHANNEL ROUTING OPERATIONS ARE REFERENCED TO   *
* OUTER LOOP HIGHWAY STATIONING FROM THE HIGHWAY GENERAL PLAN *
*                                                            *
* NORTH-SOUTH DRAINAGE CHANNEL GEOMETRY HAS BEEN REVISED TO  *
* REFLECT MKE FINAL DESIGN (SOIL-CEMENT)                     *
*                                                            *
* CITY OF SCOTTSDALE HEC-1 MODELS HAVE BEEN INSERTED FOR     *
* STONEGATE & FOR THE AREA CAPTURED BY SCOTTSDALE RANCH LAKE. *
*                                                            *
* HEC-1 MODELS FOR CAP CACTUS ROAD DETENTION BASIN AND THE   *
* TWO CAP PIPE OVERCHUTES SOUTH OF SHEA BLVD ARE INCLUDED IN *
* THE FORM OF HYDROGRAPHS WHICH ARE INPUT ON QI RECORDS.     *
**************************************************************

IT          HYDROGRAPH TIME DATA
                  NMIN          5  MINUTES IN COMPUTATION INTERVAL
                 IDATE    14JUN96  STARTING DATE
                 ITIME       0000  STARTING TIME
                    NQ        300  NUMBER OF HYDROGRAPH ORDINATES
                NDDATE    15JUN96  ENDING DATE
                NDTIME       0055  ENDING TIME
                 ICENT         19  CENTURY MARK

          COMPUTATION INTERVAL    0.08 HOURS
               TOTAL TIME BASE   24.92 HOURS

     ENGLISH UNITS
          DRAINAGE AREA          SQUARE MILES
          PRECIPITATION DEPTH    INCHES
          LENGTH, ELEVATION      FEET
          FLOW                   CUBIC FEET PER SECOND
          STORAGE VOLUME         ACRE-FEET
          SURFACE AREA           ACRES
          TEMPERATURE            DEGREES FAHRENHEIT
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	165	20.	5.08	4.	1.	1.	0.01		
ROUTED TO	156	20.	5.17	4.	1.	1.	0.01		
HYDROGRAPH AT	284310	27.	5.08	5.	1.	1.	0.02		
ROUTED TO	284310	0.	5.08	0.	0.	0.	0.02	4.10	12.17
ROUTED TO	284320	0.	6.92	0.	0.	0.	0.02		
HYDROGRAPH AT	4320A	90.	5.08	17.	4.	4.	0.06		
2 COMBINED AT	284320	90.	5.08	17.	4.	4.	0.08		
HYDROGRAPH AT	160	123.	5.17	24.	6.	6.	0.10		
ROUTED TO	161	122.	5.17	24.	6.	6.	0.10		
HYDROGRAPH AT	155	37.	5.08	8.	2.	2.	0.03		
3 COMBINED AT	157	178.	5.17	36.	9.	9.	0.14		
ROUTED TO	151	178.	5.17	36.	9.	9.	0.14		
HYDROGRAPH AT	150	62.	5.08	13.	3.	3.	0.04		
HYDROGRAPH AT	150A	28.	5.17	6.	1.	1.	0.02		
ROUTED TO	151.1	27.	5.25	6.	1.	1.	0.02		
3 COMBINED AT	152	267.	5.17	54.	14.	14.	0.20		
ROUTED TO	121	262.	5.17	54.	14.	14.	0.20		
HYDROGRAPH AT	70	8.	5.33	1.	0.	0.	0.01		
ROUTED TO	122	8.	5.33	1.	0.	0.	0.01		
HYDROGRAPH AT	120	21.	5.17	4.	1.	1.	0.01		
3 COMBINED AT	123	288.	5.17	60.	16.	15.	0.23		
HYDROGRAPH AT	75	290.	5.25	61.	16.	15.	0.23		
ROUTED TO	DAM75	290.	5.25	61.	16.	15.	0.23	102.39	5.25
HYDROGRAPH AT	85	293.	5.25	62.	16.	15.	0.23		
ROUTED TO	DAM85	292.	5.25	61.	16.	15.	0.23	102.39	5.25
HYDROGRAPH AT	80	10.	5.00	2.	1.	0.	0.01		
ROUTED TO	DAM80	10.	5.08	2.	0.	0.	0.01	102.04	5.08
ROUTED TO	87	10.	5.08	2.	0.	0.	0.01		

2 COMBINED AT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
ROUTED TO	96	299.	5.25	63.	16.	16.	0.24		
HYDROGRAPH AT	95	19.	5.17	4.	1.	1.	0.01		
2 COMBINED AT	97	317.	5.25	67.	17.	17.	0.25		
ROUTED TO	91	314.	5.25	67.	17.	17.	0.25		
HYDROGRAPH AT	90	19.	5.17	4.	1.	1.	0.02		
2 COMBINED AT	92	332.	5.25	71.	18.	18.	0.27		
HYDROGRAPH AT	110	24.	5.08	5.	1.	1.	0.02		
ROUTED TO	101	24.	5.17	5.	1.	1.	0.02		
HYDROGRAPH AT	100	48.	5.08	10.	2.	2.	0.00		
ROUTED TO	DAM100	48.	5.08	9.	2.	2.	0.00	102.11	5.08
3 COMBINED AT	102	394.	5.25	85.	22.	21.	0.29		
ROUTED TO	103	394.	5.25	85.	22.	21.	0.29		
DIVERSION TO	103B	0.	5.25	0.	0.	0.	0.29		
HYDROGRAPH AT	103A	394.	5.25	85.	22.	21.	0.29		
ROUTED TO	361	386.	5.25	85.	22.	21.	0.29	2.97	5.25
HYDROGRAPH AT	360	12.	5.83	3.	1.	1.	0.03		
2 COMBINED AT	362	393.	5.33	88.	23.	22.	0.32		
ROUTED TO	411	394.	5.33	88.	23.	22.	0.32	3.02	5.33
ROUTED TO	410B	395.	5.33	88.	23.	22.	0.32	3.02	5.33
HYDROGRAPH AT	410	22.	5.33	4.	1.	1.	0.03		
ROUTED TO	410A	22.	5.33	4.	1.	1.	0.03	0.24	5.33
2 COMBINED AT	410C	417.	5.33	92.	24.	23.	0.35		
ROUTED TO	421	411.	5.33	92.	24.	23.	0.35	3.04	5.33
HYDROGRAPH AT	140	37.	5.17	8.	2.	2.	0.03		
ROUTED TO	DAM140	38.	5.17	6.	2.	2.	0.03	102.10	5.17
HYDROGRAPH AT	130	79.	5.17	15.	4.	4.	0.06		
HYDROGRAPH AT	420	96.	5.33	25.	7.	6.	0.14		
ROUTED TO	420A	95.	5.33	25.	7.	6.	0.14	1.25	5.33
2 COMBINED AT	423	506.	5.33	116.	31.	29.	0.49		
ROUTED TO	401	505.	5.42	116.	31.	29.	0.49	3.37	5.42
HYDROGRAPH AT	240	66.	5.17	12.	3.	3.	0.06		
ROUTED TO	246	65.	5.25	12.	3.	3.	0.06		

2 COMBINED AT	247	89.	5.17	17.	4.	4.	0.09		
ROUTED TO	248	89.	5.25	17.	4.	4.	0.09		
HYDROGRAPH AT	284010	133.	5.17	26.	7.	6.	0.09		
HYDROGRAPH AT	255	31.	5.17	6.	2.	1.	0.03		
ROUTED TO	236	31.	5.25	6.	2.	1.	0.03		
HYDROGRAPH AT	235	15.	5.17	3.	1.	1.	0.01		
2 COMBINED AT	237	44.	5.25	9.	2.	2.	0.04		
ROUTED TO	181	44.	5.33	9.	2.	2.	0.04		
3 COMBINED AT	182	258.	5.17	52.	13.	13.	0.22		
ROUTED TO	191	257.	5.25	52.	13.	13.	0.22		
HYDROGRAPH AT	195	62.	5.00	12.	3.	3.	0.04		
ROUTED TO	186	62.	5.08	12.	3.	3.	0.04		
HYDROGRAPH AT	185	38.	5.00	7.	2.	2.	0.02		
2 COMBINED AT	187	99.	5.08	19.	5.	5.	0.06		
ROUTED TO	188	97.	5.08	19.	5.	5.	0.06		
ROUTED TO	DAM1	0.	0.08	0.	0.	0.	0.06	101.75	15.17
2 COMBINED AT	189	257.	5.25	52.	13.	13.	0.28		
HYDROGRAPH AT	190	13.	6.00	4.	1.	1.	0.03		
2 COMBINED AT	192	262.	5.25	55.	14.	14.	0.32		
ROUTED TO	211	258.	5.25	55.	14.	14.	0.32		
HYDROGRAPH AT	210	26.	5.17	5.	1.	1.	0.02		
2 COMBINED AT	212	282.	5.25	60.	16.	15.	0.34		
DIVERSION TO	429	282.	0.08	60.	16.	15.	0.34		
HYDROGRAPH AT	213	0.	0.08	0.	0.	0.	0.34		
HYDROGRAPH AT	405	10.	5.33	2.	0.	0.	0.35		
HYDROGRAPH AT	200	33.	5.33	6.	2.	2.	0.04		
ROUTED TO	DAM200	33.	5.33	6.	2.	1.	0.04	102.09	5.33
HYDROGRAPH AT	415	37.	5.33	7.	2.	2.	0.05		
HYDROGRAPH AT	135	34.	5.17	8.	2.	2.	0.03		
HYDROGRAPH AT	425	43.	5.25	9.	2.	2.	0.04		
3 COMBINED AT	426	87.	5.25	17.	5.	4.	0.44		
HYDPOGRAPH AT	400	133.	5.50	32.	8.	8.	0.54		

2 COMBINED AT	402	635.	5.42	148.	39.	38.	1.02		
ROUTED TO	431	625.	5.50	148.	39.	38.	1.02	3.71	5.50
HYDROGRAPH AT	430C	69.	5.50	16.	4.	4.	0.10		
2 COMBINED AT	432	694.	5.50	164.	43.	42.	1.12		
ROUTED TO	433	692.	5.50	164.	43.	42.	1.12	3.71	5.50
HYDROGRAPH AT	430A	62.	5.50	13.	3.	3.	0.08		
HYDROGRAPH AT	440A	41.	5.75	12.	3.	3.	0.07		
3 COMBINED AT	441	790.	5.50	188.	50.	48.	1.26		
ROUTED TO	501	790.	5.50	188.	50.	48.	1.26	3.99	5.50
HYDROGRAPH AT	500A	21.	5.50	5.	1.	1.	0.03		
HYDROGRAPH AT	503A	125.	6.67	15.	6.	6.	0.00		
ROUTED TO	503B	125.	6.92	15.	6.	6.	0.00		
3 COMBINED AT	503C	811.	5.50	206.	57.	55.	1.30		
ROUTED TO	500B1	811.	5.50	206.	57.	55.	1.30	2.36	5.50
HYDROGRAPH AT	500B	13.	5.42	3.	1.	1.	0.02		
2 COMBINED AT	500B2	824.	5.50	209.	58.	55.	1.31		
ROUTED TO	560A1	820.	5.50	209.	58.	55.	1.31	2.03	5.50
HYDROGRAPH AT	560A	16.	5.25	3.	1.	1.	0.02		
2 COMBINED AT	560A2	833.	5.50	212.	58.	56.	1.33		
ROUTED TO	560B1	834.	5.58	212.	58.	56.	1.33	3.40	5.58
HYDROGRAPH AT	560B	16.	5.33	3.	1.	1.	0.02		
2 COMBINED AT	560B2	848.	5.58	215.	59.	57.	1.36		
ROUTED TO	560C1	847.	5.58	215.	59.	57.	1.36	3.42	5.58
HYDROGRAPH AT	560C	25.	5.50	5.	1.	1.	0.04		
2 COMBINED AT	560C2	871.	5.58	221.	60.	58.	1.40		
ROUTED TO	551	856.	5.67	221.	60.	58.	1.40	3.44	5.67
HYDROGRAPH AT	495A	20.	5.42	4.	1.	1.	0.03		
HYDROGRAPH AT	550A	110.	5.83	30.	8.	8.	0.18		
2 COMBINED AT	552	961.	5.67	251.	68.	66.	1.58		
HYDROGRAPH AT	540A	49.	5.58	12.	3.	3.	0.07		
2 COMBINED AT	540A1	1008.	5.67	263.	72.	69.	1.65		
ROUTED TO	545A1	1010.	5.67	263.	72.	69.	1.65	3.63	5.67

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2 COMBINED AT	545A2	1014.	5.67	264.	72.	69.	1.66		
ROUTED TO	545B1	1017.	5.67	264.	72.	69.	1.66	4.07	5.67
HYDROGRAPH AT	545B	10.	5.25	2.	0.	0.	0.01		
2 COMBINED AT	545B2	1022.	5.67	266.	72.	70.	1.67		
ROUTED TO	545C1	1019.	5.67	266.	72.	70.	1.67	4.07	5.67
HYDROGRAPH AT	545C	10.	5.33	2.	1.	1.	0.01		
2 COMBINED AT	545C2	1026.	5.67	268.	73.	70.	1.68		
ROUTED TO	545D1	1021.	5.75	268.	73.	70.	1.68	3.84	5.75
HYDROGRAPH AT	545D	24.	5.42	5.	1.	1.	0.03		
2 COMBINED AT	545D2	1039.	5.75	274.	74.	72.	1.71		
ROUTED TO	545E1	1040.	5.75	274.	74.	72.	1.71	3.88	5.75
HYDROGRAPH AT	545E	25.	5.50	6.	2.	1.	0.03		
DIVERSION TO	545EDV	12.	5.50	3.	1.	1.	0.03		
HYDROGRAPH AT	545EW	12.	5.50	3.	1.	1.	0.03		
2 COMBINED AT	545E2	1050.	5.75	277.	75.	72.	1.74		
HYDROGRAPH AT	1270	9.	6.83	4.	1.	1.	0.03		
HYDROGRAPH AT	1280	18.	6.92	8.	2.	2.	0.06		
HYDROGRAPH AT	1350	27.	6.92	12.	3.	3.	0.09		
HYDROGRAPH AT	1360	36.	6.92	16.	5.	4.	0.12		
HYDROGRAPH AT	1440	44.	7.00	20.	6.	5.	0.15		
HYDROGRAPH AT	1450	53.	7.00	25.	7.	7.	0.18		
HYDROGRAPH AT	1520	61.	7.00	29.	8.	8.	0.22		
HYDROGRAPH AT	1530	69.	7.08	33.	9.	9.	0.25		
HYDROGRAPH AT	1600	86.	7.08	40.	11.	11.	0.31		
ROUTED TO	1591	85.	7.25	40.	11.	11.	0.31		
HYDROGRAPH AT	1260	9.	6.92	4.	1.	1.	0.03		
HYDROGRAPH AT	1290	18.	7.00	9.	2.	2.	0.06		
HYDROGRAPH AT	1340	27.	7.00	13.	4.	3.	0.10		
HYDROGRAPH AT	1370	36.	7.00	17.	5.	5.	0.13		
HYDROGRAPH AT	1430	45.	7.00	21.	6.	6.	0.16		
HYDROGRAPH AT	1460	53.	7.08	25.	7.	7.	0.19		
HYDROGRAPH AT	1510	62.	7.08	29.	8.	8.	0.22		

HYDROGRAPH AT	1590	87.	7.17	42.	12.	11.	0.32
2 COMBINED AT	1592	172.	7.25	82.	23.	22.	0.63
ROUTED TO	1581	171.	7.33	82.	23.	22.	0.63
HYDROGRAPH AT	1250	9.	6.83	4.	1.	1.	0.03
HYDROGRAPH AT	1300	18.	6.83	8.	2.	2.	0.06
HYDROGRAPH AT	1330	27.	6.92	12.	3.	3.	0.09
HYDROGRAPH AT	1380	35.	6.92	16.	4.	4.	0.12
HYDROGRAPH AT	1420	44.	6.92	20.	6.	5.	0.15
HYDROGRAPH AT	1470	52.	7.00	24.	7.	6.	0.18
HYDROGRAPH AT	1500	61.	7.00	28.	8.	8.	0.21
HYDROGRAPH AT	1550	69.	7.00	32.	9.	9.	0.24
HYDROGRAPH AT	1580	85.	7.00	40.	11.	11.	0.30
2 COMBINED AT	1582	252.	7.25	121.	34.	33.	0.93
ROUTED TO	1571	252.	7.33	121.	34.	33.	0.93
HYDROGRAPH AT	1240	9.	6.83	4.	1.	1.	0.03
HYDROGRAPH AT	1310	18.	6.83	8.	2.	2.	0.06
HYDROGRAPH AT	1320	28.	6.83	13.	4.	3.	0.09
HYDROGRAPH AT	1400	29.	6.83	14.	4.	4.	0.10
HYDROGRAPH AT	1390	8.	6.50	3.	1.	1.	0.02
ROUTED TO	1391	8.	6.58	3.	1.	1.	0.02
2 COMBINED AT	1392	37.	6.75	17.	5.	5.	0.13
HYDROGRAPH AT	1410	46.	6.75	22.	6.	6.	0.16
HYDROGRAPH AT	1480	55.	6.83	26.	7.	7.	0.19
HYDROGRAPH AT	1490	64.	6.83	30.	8.	8.	0.22
HYDROGRAPH AT	1560	73.	6.83	34.	9.	9.	0.25
HYDROGRAPH AT	1570	92.	6.92	43.	12.	11.	0.31
2 COMBINED AT	1572	331.	7.17	161.	46.	44.	1.25
ROUTED TO	1611	331.	7.25	161.	46.	44.	1.25
HYDROGRAPH AT	1610	67.	7.17	32.	9.	9.	0.25
2 COMBINED AT	1612	398.	7.25	194.	55.	53.	1.50
ROUTED TO	1621	398.	7.25	194.	55.	53.	1.50
HYDROGRAPH AT	1620	77.	7.08	36.	10.	10.	0.25

HYDROGRAPH AT	1640	54.	6.75	23.	6.	6.	0.14		
3 COMBINED AT	1622	521.	7.17	252.	72.	69.	1.89		
ROUTED TO	1887	412.	8.00	232.	71.	69.	1.89	387.24	8.00
HYDROGRAPH AT	1950	14.	6.50	6.	2.	1.	0.04		
HYDROGRAPH AT	1945	27.	6.83	12.	3.	3.	0.08		
ROUTED TO	1951	27.	6.92	12.	3.	3.	0.08		
2 COMBINED AT	1952	40.	6.75	18.	5.	5.	0.12		
HYDROGRAPH AT	1940	85.	6.67	38.	11.	10.	0.24		
HYDROGRAPH AT	1920	80.	6.75	34.	10.	9.	0.20		
ROUTED TO	1941	80.	6.83	34.	10.	9.	0.20		
2 COMBINED AT	1942	164.	6.75	73.	20.	19.	0.43		
HYDROGRAPH AT	1925	189.	6.83	84.	23.	23.	0.50		
HYDROGRAPH AT	1930	209.	7.08	96.	27.	26.	0.57		
HYDROGRAPH AT	1910	55.	6.75	24.	7.	6.	0.14		
HYDROGRAPH AT	1900	102.	6.75	46.	13.	12.	0.27		
HYDROGRAPH AT	1890	137.	6.92	63.	18.	17.	0.36		
ROUTED TO	1931	137.	6.92	63.	18.	17.	0.36		
2 COMBINED AT	1932	345.	7.00	159.	45.	43.	0.93		
HYDROGRAPH AT	1885	361.	7.25	169.	48.	46.	1.00		
HYDROGRAPH AT	1850	58.	6.67	25.	7.	7.	0.14		
HYDROGRAPH AT	1860	104.	6.75	46.	13.	12.	0.26		
HYDROGRAPH AT	1870	167.	7.08	78.	22.	21.	0.44		
ROUTED TO	1886	167.	7.08	78.	22.	21.	0.44		
HYDROGRAPH AT	1880	26.	6.42	10.	3.	3.	0.05		
4 COMBINED AT	1888	890.	7.50	483.	144.	139.	3.38		
ROUTED TO	1984	888.	7.67	482.	144.	139.	3.38	388.56	7.67
HYDROGRAPH AT	1840	12.	6.83	6.	2.	1.	0.03		
HYDROGRAPH AT	1670	28.	6.42	11.	3.	3.	0.08		
ROUTED TO	1681	28.	6.58	11.	3.	3.	0.08		
HYDROGRAPH AT	1680	28.	6.42	11.	3.	3.	0.08		
2 COMBINED AT	1682	56.	6.50	21.	6.	5.	0.15		
ROUTED TO	1751	56.	6.58	21.	6.	5.	0.15		

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DIVERSION TO	1228	371.	5.00	304.	113.	110.	0.00
HYDROGRAPH AT	1227	66.	5.00	54.	20.	19.	0.00
HYDROGRAPH AT	1225	128.	5.17	64.	23.	22.	0.08
HYDROGRAPH AT	1230	502.	5.50	146.	43.	42.	0.72
ROUTED TO	1221	499.	5.67	146.	43.	42.	0.72
HYDROGRAPH AT	310510	5.	5.33	1.	0.	0.	0.01
HYDROGRAPH AT	1217A	558.	5.25	96.	24.	23.	0.74
HYDROGRAPH AT	1220	959.	5.50	181.	46.	44.	1.41
ROUTED TO	1222	935.	5.58	181.	46.	44.	1.41
2 COMBINED AT	1223	1428.	5.58	326.	89.	85.	2.13
HYDROGRAPH AT	1750	1422.	5.75	336.	91.	88.	2.20
2 COMBINED AT	1752	1438.	5.75	357.	97.	93.	2.36
ROUTED TO	1761	1418.	5.83	357.	97.	93.	2.36
HYDROGRAPH AT	1760	22.	6.17	7.	2.	2.	0.05
2 COMBINED AT	1762	1434.	5.83	364.	99.	95.	2.41
ROUTED TO	1771	1428.	5.83	364.	99.	95.	2.41
HYDROGRAPH AT	1770	17.	6.08	5.	1.	1.	0.04
2 COMBINED AT	1772	1442.	5.83	370.	100.	97.	2.44
ROUTED TO	1841	1416.	5.92	369.	100.	96.	2.44
2 COMBINED AT	1842	1425.	5.92	375.	102.	98.	2.47
HYDROGRAPH AT	1830	1477.	6.00	395.	107.	103.	2.59
HYDROGRAPH AT	1660	31.	6.58	12.	3.	3.	0.09
ROUTED TO	1691	31.	6.75	12.	3.	3.	0.09
HYDROGRAPH AT	1690	31.	6.58	13.	3.	3.	0.09
2 COMBINED AT	1692	62.	6.67	25.	7.	7.	0.18
ROUTED TO	1741	62.	6.83	25.	7.	7.	0.18
HYDROGRAPH AT	1740	31.	6.58	13.	3.	3.	0.09
2 COMBINED AT	1742	92.	6.75	37.	10.	10.	0.27
ROUTED TO	1781	92.	6.92	37.	10.	10.	0.27
HYDROGRAPH AT	1780	37.	6.42	14.	4.	4.	0.09
2 COMBINED AT	1782	125.	6.75	51.	14.	13.	0.36
ROUTED TO	1831	125.	6.92	51.	14.	13.	0.36

HYDROGRAPH AT	1820	1538.	6.08	463.	126.	122.	3.06
HYDROGRAPH AT	1650	31.	6.58	12.	3.	3.	0.09
ROUTED TO	1701	31.	6.75	12.	3.	3.	0.09
HYDROGRAPH AT	1700	31.	6.58	13.	3.	3.	0.09
2 COMBINED AT	1702	62.	6.67	25.	7.	7.	0.18
ROUTED TO	1731	62.	6.83	25.	7.	7.	0.18
HYDROGRAPH AT	1730	31.	6.58	13.	3.	3.	0.09
2 COMBINED AT	1732	92.	6.75	37.	10.	10.	0.27
ROUTED TO	1791	92.	6.92	37.	10.	10.	0.27
HYDROGRAPH AT	1790	35.	6.50	14.	4.	4.	0.09
2 COMBINED AT	1792	123.	6.75	51.	14.	13.	0.36
ROUTED TO	1822	123.	6.92	51.	14.	13.	0.36
2 COMBINED AT	1823	1581.	6.08	513.	140.	135.	3.43
HYDROGRAPH AT	1810	1626.	6.08	534.	146.	140.	3.54
HYDROGRAPH AT	1210	490.	5.67	118.	30.	29.	0.93
ROUTED TO	1201	487.	5.83	118.	30.	29.	0.93
HYDROGRAPH AT	310210	25.	5.42	6.	2.	1.	0.04
HYDROGRAPH AT	310220	14.	5.33	3.	1.	1.	0.02
2 COMBINED AT	310220	39.	5.33	9.	2.	2.	0.06
HYDROGRAPH AT	310310	20.	5.33	5.	1.	1.	0.03
HYDROGRAPH AT	310410	4.	5.33	1.	0.	0.	0.01
3 COMBINED AT	31B	63.	5.33	14.	4.	4.	0.09
HYDROGRAPH AT	1204A	612.	5.25	117.	29.	28.	0.74
HYDROGRAPH AT	1200	900.	5.67	188.	47.	46.	1.31
2 COMBINED AT	1202	1331.	5.67	306.	78.	75.	2.24
ROUTED TO	1191	1330.	5.75	305.	78.	75.	2.24
HYDROGRAPH AT	300010	15.	5.33	3.	1.	1.	0.02
HYDROGRAPH AT	310010	31.	5.33	7.	2.	2.	0.05
HYDROGRAPH AT	310110	16.	5.33	4.	1.	1.	0.02
ROUTED TO	310120	15.	5.33	4.	1.	1.	0.02
ROUTED TO	310130	15.	5.42	4.	1.	1.	0.02
HYDROGRAPH AT	310130	21.	5.33	5.	1.	1.	0.03

3 COMBINED AT	31A	82.	5.33	19.	5.	5.	0.12		
HYDROGRAPH AT	1187A	217.	5.33	47.	12.	11.	0.33		
HYDROGRAPH AT	1190	575.	5.83	138.	35.	34.	1.05		
2 COMBINED AT	1192	1889.	5.83	443.	113.	109.	3.29		
ROUTED TO	1181	1870.	5.83	443.	113.	109.	3.29		
HYDROGRAPH AT	2530B	98.	5.17	17.	4.	4.	0.10		
ROUTED TO	1173	24.	6.17	19.	13.	13.	0.10	1500.13	6.17
ROUTED TO	300720	24.	6.25	20.	14.	14.	0.10		
HYDROGRAPH AT	300720	17.	5.33	4.	1.	1.	0.03		
2 COMBINED AT	300720	40.	5.33	24.	15.	15.	0.13		
ROUTED TO	300730	40.	5.42	24.	16.	16.	0.13		
HYDROGRAPH AT	300730	14.	5.33	3.	1.	1.	0.02		
2 COMBINED AT	300730	53.	5.42	27.	16.	16.	0.15		
ROUTED TO	300740	53.	5.42	27.	17.	17.	0.15		
HYDROGRAPH AT	300740	51.	5.08	11.	3.	3.	0.04		
2 COMBINED AT	300740	95.	5.25	37.	19.	19.	0.19		
HYDROGRAPH AT	300810	47.	5.33	11.	3.	3.	0.07		
ROUTED TO	300740	47.	5.42	11.	3.	3.	0.07		
2 COMBINED AT	300740	138.	5.33	48.	22.	22.	0.26		
ROUTED TO	300750	137.	5.33	48.	22.	22.	0.26		
HYDROGRAPH AT	300750	30.	5.08	6.	2.	1.	0.02		
2 COMBINED AT	300750	160.	5.33	54.	24.	23.	0.28		
HYDROGRAPH AT	300910	16.	5.33	4.	1.	1.	0.02		
ROUTED TO	300750	16.	5.50	4.	1.	1.	0.02		
2 COMBINED AT	300750	173.	5.33	57.	25.	24.	0.30		
ROUTED TO	300760	172.	5.42	57.	25.	24.	0.30		
HYDROGRAPH AT	300760	63.	5.08	13.	3.	3.	0.05		
2 COMBINED AT	300760	221.	5.25	70.	28.	27.	0.35		
ROUTED TO	300770	221.	5.33	70.	28.	27.	0.35		
HYDROGRAPH AT	300770	22.	5.25	5.	1.	1.	0.03		
2 COMBINED AT	300770	243.	5.33	74.	29.	29.	0.38		
HYDROGRAPH AT	301010	30.	5.33	7.	2.	2.	0.05		

ROUTED TO	301030	30.	5.42	7.	2.	2.	0.05		
HYDROGRAPH AT	301030	16.	5.42	4.	1.	1.	0.02		
2 COMBINED AT	301030	45.	5.42	11.	3.	3.	0.07		
ROUTED TO	301040	45.	5.50	11.	3.	3.	0.07		
HYDROGRAPH AT	301040	34.	5.33	8.	2.	2.	0.05		
2 COMBINED AT	301040	77.	5.50	18.	5.	5.	0.12		
ROUTED TO	300770	76.	5.50	18.	5.	5.	0.12		
2 COMBINED AT	300770	307.	5.33	93.	34.	33.	0.50		
ROUTED TO	300770	305.	5.42	87.	30.	29.	0.50	4.77	5.42
HYDROGRAPH AT	1177A	373.	5.42	105.	34.	33.	0.65		
HYDROGRAPH AT	1180	706.	5.92	189.	56.	54.	1.35		
HYDROGRAPH AT	1000	10.	5.33	2.	1.	1.	0.01		
ROUTED TO	1042	10.	5.33	2.	1.	1.	0.01		
HYDROGRAPH AT	1060A	38.	6.00	14.	4.	4.	0.06		
ROUTED TO	1051	38.	6.08	14.	4.	4.	0.06		
HYDROGRAPH AT	1050	18.	5.08	3.	1.	1.	0.01		
2 COMBINED AT	1052	43.	6.00	17.	5.	4.	0.07		
ROUTED TO	1041	43.	6.17	17.	5.	4.	0.07		
HYDROGRAPH AT	1040	41.	5.17	8.	2.	2.	0.03		
3 COMBINED AT	1043	80.	5.25	27.	7.	7.	0.11		
ROUTED TO	DAM8	78.	5.42	21.	5.	5.	0.11	102.16	5.42
ROUTED TO	1011	78.	5.50	22.	6.	5.	0.11		
HYDROGRAPH AT	1010	44.	5.17	8.	2.	2.	0.04		
2 COMBINED AT	1012	106.	5.50	29.	8.	7.	0.15		
HYDROGRAPH AT	1110	109.	5.50	31.	8.	8.	0.16		
HYDROGRAPH AT	1020	41.	5.08	8.	2.	2.	0.03		
HYDROGRAPH AT	1120	62.	5.25	12.	3.	3.	0.07		
2 COMBINED AT	1121	151.	5.50	42.	11.	11.	0.23		
DIVERSION TO	2223	60.	5.50	17.	4.	4.	0.23		
HYDROGRAPH AT	2222	90.	5.50	25.	7.	6.	0.23		
ROUTED TO	1131	87.	5.58	25.	7.	6.	0.23		
HYDROGRAPH AT	1070	43.	5.25	9.	2.	2.	0.04		

HYDROGRAPH AT	1030	34.	5.25	7.	2.	2.	0.03
2 COMBINED AT	1032	77.	5.25	15.	4.	4.	0.07
HYDROGRAPH AT	1130	118.	5.33	23.	6.	6.	0.13
2 COMBINED AT	1132	192.	5.50	48.	13.	12.	0.36
DIVERSION TO	3334	96.	5.50	24.	6.	6.	0.36
HYDROGRAPH AT	3333	96.	5.50	24.	6.	6.	0.36
ROUTED TO	1151	95.	5.58	24.	6.	6.	0.36
HYDROGRAPH AT	1080A	93.	5.17	18.	5.	4.	0.09
HYDROGRAPH AT	1090	148.	5.17	28.	7.	7.	0.14
HYDROGRAPH AT	1150	253.	5.50	57.	15.	14.	0.37
2 COMBINED AT	1152	348.	5.50	81.	21.	20.	0.74
DIVERSION TO	4445	174.	5.50	41.	11.	10.	0.74
HYDROGRAPH AT	4444	174.	5.50	41.	11.	10.	0.74
ROUTED TO	1161	173.	5.67	41.	11.	10.	0.74
HYDROGRAPH AT	1159	1117.	5.83	437.	111.	107.	2.25
ROUTED TO	300120	1117.	5.83	437.	111.	107.	2.25
HYDROGRAPH AT	300120	25.	5.33	6.	1.	1.	0.04
2 COMBINED AT	300120	1131.	5.83	442.	112.	108.	2.29
ROUTED TO	300130	1131.	5.83	442.	112.	108.	2.29
ROUTED TO	300140	1130.	5.83	442.	112.	108.	2.29
HYDROGRAPH AT	300140	17.	5.42	4.	1.	1.	0.03
2 COMBINED AT	300140	1141.	5.83	446.	113.	109.	2.31
ROUTED TO	300150	1140.	5.83	446.	113.	109.	2.31
ROUTED TO	300160	1138.	5.92	446.	113.	109.	2.31
HYDROGRAPH AT	300160	44.	5.33	10.	3.	2.	0.06
2 COMBINED AT	300160	1163.	5.83	455.	116.	112.	2.37
HYDROGRAPH AT	300310	8.	5.33	2.	0.	0.	0.01
ROUTED TO	300160	8.	5.58	2.	0.	0.	0.01
2 COMBINED AT	300160	1170.	5.83	457.	116.	112.	2.38
ROUTED TO	300170	1169.	5.83	457.	116.	112.	2.38
ROUTED TO	300180	1168.	5.92	457.	116.	112.	2.38
HYDROGRAPH AT	300180	6.	5.42	1.	0.	0.	0.01

2 COMBINED AT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
ROUTED TO	300190	1171.	5.92	458.	117.	113.	2.39		
ROUTED TO	300200	1170.	5.92	458.	117.	113.	2.39		
HYDROGRAPH AT	300200	56.	5.25	12.	3.	3.	0.06		
2 COMBINED AT	300200	1193.	5.92	467.	120.	116.	2.46		
HYDROGRAPH AT	300410	44.	5.42	10.	3.	3.	0.07		
ROUTED TO	300410	39.	5.58	8.	2.	2.	0.07	2.41	5.58
ROUTED TO	300420	39.	5.58	8.	2.	2.	0.07		
HYDROGRAPH AT	300420	27.	5.33	6.	2.	2.	0.04		
2 COMBINED AT	300420	62.	5.58	14.	4.	4.	0.11		
ROUTED TO	300420	0.	5.58	0.	0.	0.	0.11	3.23	24.92
ROUTED TO	300200	0.	5.67	0.	0.	0.	0.11		
2 COMBINED AT	300200	1193.	5.92	467.	120.	116.	2.56		
HYDROGRAPH AT	290410	83.	5.42	20.	5.	5.	0.13		
ROUTED TO	290420	83.	5.50	20.	5.	5.	0.13		
ROUTED TO	290430	82.	5.50	20.	5.	5.	0.13		
HYDROGRAPH AT	290430	28.	5.08	7.	2.	2.	0.04		
2 COMBINED AT	290430	104.	5.50	26.	7.	7.	0.17		
2 COMBINED AT	30A	1265.	5.83	492.	127.	122.	2.74		
HYDROGRAPH AT	1155A	1306.	5.92	512.	132.	127.	2.89		
HYDROGRAPH AT	1160	1406.	6.08	547.	141.	136.	3.20		
HYDROGRAPH AT	300510	29.	5.42	7.	2.	2.	0.04		
ROUTED TO	300520	29.	5.42	7.	2.	2.	0.04		
ROUTED TO	300530	28.	5.50	7.	2.	2.	0.04		
HYDROGRAPH AT	300530	49.	5.25	11.	3.	3.	0.06		
2 COMBINED AT	300530	71.	5.25	17.	4.	4.	0.10		
ROUTED TO	300540	70.	5.33	17.	4.	4.	0.10		
HYDROGRAPH AT	300540	28.	5.25	6.	2.	2.	0.03		
2 COMBINED AT	300540	96.	5.25	23.	6.	6.	0.13		
ROUTED TO	300550	96.	5.25	23.	6.	6.	0.13		
HYDROGRAPH AT	300550	50.	5.25	11.	3.	3.	0.06		
2 COMBINED AT	300550	146.	5.25	34.	9.	9.	0.19		
ROUTED TO	300560	143.	5.33	34.	9.	9.	0.19		

2 COMBINED AT	300560	176.	5.25	42.	11.	10.	0.22		
ROUTED TO	300560	135.	5.58	23.	6.	6.	0.22	4.32	5.58
HYDROGRAPH AT	300610	34.	5.25	7.	2.	2.	0.04		
ROUTED TO	300620	34.	5.25	7.	2.	2.	0.04		
HYDROGRAPH AT	300630	67.	5.25	15.	4.	4.	0.08		
ROUTED TO	300630	23.	6.08	5.	1.	1.	0.08	4.11	6.08
2 COMBINED AT	308	138.	5.58	28.	7.	7.	0.30		
HYDROGRAPH AT	1165A	191.	5.75	53.	13.	13.	0.48		
HYDROGRAPH AT	1170	293.	5.58	85.	22.	21.	0.74		
ROUTED TO	1162	288.	5.67	85.	22.	21.	0.74		
3 COMBINED AT	1163	1760.	6.17	672.	174.	167.	4.67		
DIVERSION TO	5556	880.	6.17	336.	87.	84.	4.67		
HYDROGRAPH AT	5555	880.	6.17	336.	87.	84.	4.67		
ROUTED TO	1182	878.	6.17	336.	87.	84.	4.67		
2 COMBINED AT -	1183	1569.	5.92	525.	143.	137.	6.02		
ROUTED TO	1184	1561.	6.00	525.	142.	137.	6.02		
2 COMBINED AT	1185	3398.	5.92	967.	255.	246.	9.31		
ROUTED TO	1646	3376.	5.92	968.	255.	246.	9.31		
HYDROGRAPH AT	1645	48.	6.08	16.	4.	4.	0.09		
2 COMBINED AT	1647	3421.	5.92	983.	259.	250.	9.40		
ROUTED TO	1711	3388.	6.00	984.	259.	250.	9.40		
HYDROGRAPH AT	1710	39.	6.42	15.	4.	4.	0.09		
2 COMBINED AT	1712	3419.	6.00	998.	263.	254.	9.49		
ROUTED TO	1721	3416.	6.00	999.	263.	254.	9.49		
HYDROGRAPH AT	1720	43.	6.33	16.	4.	4.	0.09		
2 COMBINED AT	1722	3454.	6.00	1015.	268.	258.	9.59		
ROUTED TO	1801	3435.	6.00	1015.	268.	258.	9.59		
HYDROGRAPH AT	1800	48.	6.25	18.	5.	5.	0.09		
2 COMBINED AT	1802	3480.	6.00	1032.	272.	262.	9.68		
ROUTED TO	1811	3453.	6.08	1033.	272.	262.	9.68		
2 COMBINED AT	1812	5079.	6.08	1566.	418.	403.	13.22		
ROUTED TO	1813	5063.	6.08	1566.	418.	403.	13.22		

2 COMBINED AT	1985	[illegible]	6.00	2037.	562.	542.	16.60		
ROUTED TO	1986	4710.	6.42	2029.	562.	541.	16.60	394.49	6.42
HYDROGRAPH AT	3334	96.	5.50	24.	6.	6.	0.00		
HYDROGRAPH AT	1135	165.	5.50	40.	10.	10.	0.11		
ROUTED TO	1136	164.	5.58	40.	10.	10.	0.11		
HYDROGRAPH AT	4445	174.	5.50	41.	11.	10.	0.00		
HYDROGRAPH AT	1140	253.	5.67	61.	16.	15.	0.14		
2 COMBINED AT	1141	412.	5.67	101.	26.	25.	0.25		
ROUTED TO	1971	410.	5.75	101.	26.	25.	0.25		
HYDROGRAPH AT	5556	880.	6.17	336.	87.	84.	0.00		
ROUTED TO	1972	879.	6.33	335.	87.	84.	0.00		
HYDROGRAPH AT	1970	255.	6.00	77.	20.	19.	0.51		
3 COMBINED AT	1973	1452.	6.00	511.	133.	128.	0.76		
HYDROGRAPH AT	1980	1658.	6.17	591.	155.	149.	1.23		
2 COMBINED AT	1987	6299.	6.33	2611.	717.	690.	17.83		
DIVERSION TO	1987B	0.	6.33	0.	0.	0.	17.83		
HYDROGRAPH AT	1987A	6299.	6.33	2611.	717.	690.	17.83		
ROUTED TO	1991	5933.	6.67	2605.	716.	690.	17.83	395.56	6.67
HYDROGRAPH AT	1960	235.	5.92	69.	18.	17.	0.38		
HYDROGRAPH AT	1990	443.	6.00	137.	36.	34.	0.76		
HYDROGRAPH AT	429	282.	5.25	60.	16.	15.	0.00		
HYDROGRAPH AT	430B	285.	5.33	62.	16.	16.	0.01		
HYDROGRAPH AT	440B	296.	5.42	67.	17.	17.	0.05		
ROUTED TO	495B1	296.	5.42	67.	17.	17.	0.05		
HYDROGRAPH AT	280010	10.	5.17	2.	1.	0.	0.01		
ROUTED TO	280020	10.	5.17	2.	1.	0.	0.01		
ROUTED TO	280030	10.	5.17	2.	1.	0.	0.01		
ROUTED TO	280040	10.	5.17	2.	1.	0.	0.01		
HYDROGRAPH AT	280040	30.	5.08	6.	1.	1.	0.02		
2 COMBINED AT	280040	40.	5.08	8.	2.	2.	0.03		
ROUTED TO	280050	40.	5.08	8.	2.	2.	0.03		
ROUTED TO	280060	39.	5.17	8.	2.	2.	0.03		
HYDROGRAPH AT	280060	2.	5.00	0.	0.	0.	0.00		

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
ROUTED TO	280070	40.	5.17	8.	2.	2.	0.03		
ROUTED TO	280080	40.	5.17	8.	2.	2.	0.03		
HYDROGRAPH AT	280080	22.	5.08	4.	1.	1.	0.01		
2 COMBINED AT	280080	62.	5.17	12.	3.	3.	0.04		
HYDROGRAPH AT	280410	14.	5.00	3.	1.	1.	0.01		
ROUTED TO	280420	14.	5.08	3.	1.	1.	0.01		
ROUTED TO	280080	14.	5.08	3.	1.	1.	0.01		
2 COMBINED AT	280080	75.	5.17	15.	4.	4.	0.05		
ROUTED TO	280090	75.	5.17	15.	4.	4.	0.05		
ROUTED TO	280100	75.	5.17	15.	4.	4.	0.05		
HYDROGRAPH AT	280100	35.	5.08	7.	2.	2.	0.02		
2 COMBINED AT	280100	108.	5.17	22.	6.	5.	0.07		
ROUTED TO	280110	108.	5.17	22.	6.	5.	0.07		
HYDROGRAPH AT	280510	44.	5.08	8.	2.	2.	0.03		
ROUTED TO	280520	43.	5.17	8.	2.	2.	0.03		
HYDROGRAPH AT	280520	38.	5.08	8.	2.	2.	0.03		
2 COMBINED AT	280520	81.	5.17	17.	4.	4.	0.06		
ROUTED TO	280530	81.	5.17	17.	4.	4.	0.06		
HYDROGRAPH AT	280530	9.	5.08	2.	0.	0.	0.01		
2 COMBINED AT	280530	90.	5.17	18.	5.	5.	0.06		
ROUTED TO	280540	90.	5.17	18.	5.	5.	0.06		
ROUTED TO	280110	89.	5.17	18.	5.	5.	0.06		
2 COMBINED AT	280110	197.	5.17	40.	10.	10.	0.14		
ROUTED TO	280120	194.	5.17	40.	10.	10.	0.14		
HYDROGRAPH AT	280120	12.	5.42	3.	1.	1.	0.02		
2 COMBINED AT	280120	203.	5.17	43.	11.	11.	0.16		
HYDROGRAPH AT	280610	41.	5.42	9.	2.	2.	0.06		
ROUTED TO	280610	10.	6.67	3.	1.	1.	0.06	4.17	6.67
ROUTED TO	280620	10.	6.75	3.	1.	1.	0.06		
HYDROGRAPH AT	280620	35.	5.33	8.	2.	2.	0.03		
2 COMBINED AT	280620	35.	5.33	10.	3.	3.	0.09		
HYDROGRAPH AT	280810	66.	5.08	13.	3.	3.	0.04		

ROUTED TO	280620	64.	5.17	13.	3.	3.	0.04
2 COMBINED AT	280620	96.	5.17	23.	6.	6.	0.13
ROUTED TO	280630	95.	5.17	23.	6.	6.	0.13
HYDROGRAPH AT	280910	3.	5.08	1.	0.	0.	0.00
ROUTED TO	280920	3.	5.08	1.	0.	0.	0.00
ROUTED TO	280630	3.	5.08	1.	0.	0.	0.00
2 COMBINED AT	280630	98.	5.17	23.	6.	6.	0.14
ROUTED TO	280640	97.	5.25	23.	6.	6.	0.14
HYDROGRAPH AT	281010	34.	5.08	7.	2.	2.	0.02
ROUTED TO	280640	34.	5.08	7.	2.	2.	0.02
2 COMBINED AT	280640	130.	5.17	30.	8.	8.	0.16
ROUTED TO	280650	128.	5.25	30.	8.	8.	0.16
HYDROGRAPH AT	280650	15.	5.50	3.	1.	1.	0.02
2 COMBINED AT	280650	140.	5.25	33.	9.	8.	0.18
ROUTED TO	280660	139.	5.25	33.	9.	8.	0.18
HYDROGRAPH AT	280660	9.	5.42	2.	1.	1.	0.01
2 COMBINED AT	280660	147.	5.25	35.	9.	9.	0.20
HYDROGRAPH AT	281110	30.	5.08	6.	2.	2.	0.02
ROUTED TO	281120	30.	5.17	6.	2.	2.	0.02
HYDROGRAPH AT	281120	29.	5.00	6.	1.	1.	0.02
2 COMBINED AT	281120	57.	5.08	12.	3.	3.	0.04
HYDROGRAPH AT	281210	43.	5.42	10.	3.	3.	0.07
ROUTED TO	281120	43.	5.42	10.	3.	3.	0.07
2 COMBINED AT	281120	89.	5.25	22.	6.	5.	0.11
HYDROGRAPH AT	281310	16.	5.50	4.	1.	1.	0.03
ROUTED TO	281320	16.	5.50	4.	1.	1.	0.03
ROUTED TO	281120	16.	5.58	4.	1.	1.	0.03
2 COMBINED AT	281120	100.	5.33	26.	7.	6.	0.14
ROUTED TO	281130	100.	5.33	26.	7.	6.	0.14
HYDROGRAPH AT	281130	55.	5.08	11.	3.	3.	0.04
2 COMBINED AT	281130	144.	5.25	36.	9.	9.	0.17
ROUTED TO	281140	144.	5.25	36.	9.	9.	0.17

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2 COMBINED AT	281140	165.	5.25	41.	11.	10.	0.19
ROUTED TO	280660	165.	5.25	41.	11.	10.	0.19
2 COMBINED AT	280660	311.	5.25	76.	20.	19.	0.39
2 COMBINED AT	280120	514.	5.25	119.	31.	30.	0.55
ROUTED TO	280120	512.	5.25	119.	31.	30.	0.55
ROUTED TO	280130	509.	5.25	119.	31.	30.	0.55
HYDROGRAPH AT	280130	17.	5.58	5.	1.	1.	0.04
2 COMBINED AT	280130	524.	5.25	124.	32.	31.	0.59
ROUTED TO	280140	522.	5.25	124.	32.	31.	0.59
ROUTED TO	280150	518.	5.33	124.	32.	31.	0.59
HYDROGRAPH AT	280150	18.	5.58	5.	1.	1.	0.05
2 COMBINED AT	280150	536.	5.33	130.	34.	32.	0.64
ROUTED TO	280160	534.	5.33	130.	34.	32.	0.64
HYDROGRAPH AT	280160	11.	5.25	3.	1.	1.	0.02
2 COMBINED AT	280160	544.	5.33	133.	34.	33.	0.66
HYDROGRAPH AT	281410	59.	5.08	11.	3.	3.	0.04
ROUTED TO	281420	57.	5.17	11.	3.	3.	0.04
HYDROGRAPH AT	281420	33.	5.08	6.	2.	2.	0.02
2 COMBINED AT	281420	89.	5.08	18.	4.	4.	0.06
ROUTED TO	281430	89.	5.17	18.	4.	4.	0.06
HYDROGRAPH AT	281430	55.	5.25	12.	3.	3.	0.06
2 COMBINED AT	281430	143.	5.17	30.	8.	7.	0.12
ROUTED TO	281440	142.	5.25	30.	8.	7.	0.12
HYDROGRAPH AT	281440	55.	5.25	12.	3.	3.	0.06
2 COMBINED AT	281440	197.	5.25	42.	11.	10.	0.19
ROUTED TO	281450	196.	5.25	42.	11.	10.	0.19
HYDROGRAPH AT	281450	38.	5.25	8.	2.	2.	0.04
2 COMBINED AT	281450	234.	5.25	50.	13.	12.	0.23
ROUTED TO	280160	233.	5.25	50.	13.	12.	0.23
2 COMBINED AT	280160	772.	5.33	183.	47.	46.	0.89
ROUTED TO	280170	771.	5.33	183.	47.	46.	0.89
ROUTED TO	280180	767.	5.33	183.	47.	46.	0.89

HYDROGRAPH AT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
ROUTED TO	281520	26.	5.17	5.	1.	1.	0.02		
ROUTED TO	281530	26.	5.17	5.	1.	1.	0.02		
HYDROGRAPH AT	281530	63.	5.17	13.	3.	3.	0.05		
2 COMBINED AT	281530	89.	5.17	18.	5.	4.	0.06		
HYDROGRAPH AT	281710	49.	5.08	9.	2.	2.	0.03		
ROUTED TO	281720	48.	5.08	9.	2.	2.	0.03		
ROUTED TO	281530	48.	5.17	9.	2.	2.	0.03		
2 COMBINED AT	281530	136.	5.17	27.	7.	7.	0.09		
ROUTED TO	281540	134.	5.25	27.	7.	7.	0.09		
HYDROGRAPH AT	281540	49.	5.08	10.	3.	2.	0.04		
2 COMBINED AT	281540	181.	5.17	37.	10.	9.	0.13		
ROUTED TO	281540	180.	5.25	35.	9.	8.	0.13	2.54	5.25
ROUTED TO	281550	180.	5.25	35.	9.	8.	0.13		
HYDROGRAPH AT	281550	14.	5.08	3.	1.	1.	0.01		
2 COMBINED AT	281550	193.	5.25	37.	9.	9.	0.14		
HYDROGRAPH AT	281810	7.	5.08	1.	0.	0.	0.00		
ROUTED TO	281810	0.	6.50	0.	0.	0.	0.00	1.85	10.67
ROUTED TO	281550	0.	7.75	0.	0.	0.	0.00		
2 COMBINED AT	281550	193.	5.25	37.	9.	9.	0.14		
ROUTED TO	281560	192.	5.25	37.	9.	9.	0.14		
HYDROGRAPH AT	281560	12.	5.17	3.	1.	1.	0.01		
2 COMBINED AT	281560	204.	5.25	40.	10.	10.	0.15		
HYDROGRAPH AT	281910	41.	5.25	9.	2.	2.	0.05		
ROUTED TO	281560	41.	5.33	9.	2.	2.	0.05		
2 COMBINED AT	281560	244.	5.25	49.	12.	12.	0.20		
ROUTED TO	281570	243.	5.25	49.	12.	12.	0.20		
HYDROGRAPH AT	281570	30.	5.17	6.	2.	2.	0.02		
2 COMBINED AT	281570	274.	5.25	55.	14.	14.	0.22		
ROUTED TO	281580	270.	5.33	55.	14.	14.	0.22		
HYDROGRAPH AT	281580	11.	5.33	3.	1.	1.	0.01		
2 COMBINED AT	281580	281.	5.33	58.	15.	14.	0.24		
ROUTED TO	280180	281.	5.33	58.	15.	14.	0.24		

2 COMBINED AT	[illegible]	[illegible]	[illegible]	241.	[illegible]	[illegible]	[illegible]		
ROUTED TO	280190	1044.	5.33	241.	62.	60.	1.13		
HYDROGRAPH AT	282010	74.	5.33	17.	4.	4.	0.09		
ROUTED TO	280190	74.	5.33	17.	4.	4.	0.09		
2 COMBINED AT	280190	1118.	5.33	258.	67.	64.	1.22		
ROUTED TO	280200	1103.	5.42	258.	67.	64.	1.22		
HYDROGRAPH AT	CACTUS	184.	7.75	183.	165.	164.	5.14		
HYDROGRAPH AT	282110	0.	5.58	0.	0.	0.	0.00		
2 COMBINED AT	282110	184.	7.75	183.	165.	164.	5.14		
ROUTED TO	282120	184.	7.83	183.	165.	164.	5.14		
HYDROGRAPH AT	282120	13.	5.25	3.	1.	1.	0.01		
2 COMBINED AT	282120	184.	7.83	183.	166.	164.	5.16		
ROUTED TO	282130	184.	7.83	183.	166.	164.	5.16		
ROUTED TO	282140	184.	7.92	183.	166.	164.	5.16		
HYDROGRAPH AT	282140	9.	5.25	2.	0.	0.	0.01		
2 COMBINED AT	282140	185.	7.92	183.	166.	165.	5.17		
ROUTED TO	282150	185.	7.92	183.	166.	165.	5.17		
HYDROGRAPH AT	282310	45.	5.08	10.	2.	2.	0.04		
ROUTED TO	282310	5.	7.00	1.	0.	0.	0.04	4.04	7.00
ROUTED TO	282320	5.	7.08	1.	0.	0.	0.04		
HYDROGRAPH AT	282320	34.	5.25	8.	2.	2.	0.04		
2 COMBINED AT	282320	34.	5.25	8.	2.	2.	0.08		
HYDROGRAPH AT	282410	33.	5.17	7.	2.	2.	0.03		
ROUTED TO	282320	33.	5.17	7.	2.	2.	0.03		
2 COMBINED AT	282320	67.	5.25	16.	4.	4.	0.12		
ROUTED TO	282320	65.	5.33	12.	3.	3.	0.12	2.36	5.33
ROUTED TO	282330	64.	5.33	12.	3.	3.	0.12		
HYDROGRAPH AT	282330	6.	5.25	1.	0.	0.	0.01		
2 COMBINED AT	282330	70.	5.33	13.	3.	3.	0.12		
ROUTED TO	282150	69.	5.33	13.	3.	3.	0.12		
2 COMBINED AT	282150	228.	5.33	193.	170.	168.	5.29		
ROUTED TO	282160	228.	5.42	193.	170.	168.	5.29		
HYDROGRAPH AT	282160	92.	5.25	20.	5.	5.	0.12		

2 COMBINED AT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
ROUTED TO	282170	310.	5.42	209.	175.	173.	5.41
ROUTED TO	282180	310.	5.42	209.	175.	173.	5.41
HYDROGRAPH AT	282180	5.	5.33	1.	0.	0.	0.01
2 COMBINED AT	282180	314.	5.42	210.	175.	173.	5.41
ROUTED TO	282190	314.	5.42	210.	175.	173.	5.41
HYDROGRAPH AT	282510	90.	5.42	22.	6.	5.	0.13
ROUTED TO	282520	89.	5.42	22.	6.	5.	0.13
ROUTED TO	282530	88.	5.50	22.	6.	5.	0.13
HYDROGRAPH AT	282530	36.	5.25	8.	2.	2.	0.05
2 COMBINED AT	282530	118.	5.42	29.	8.	7.	0.17
ROUTED TO	282190	118.	5.42	29.	8.	7.	0.17
2 COMBINED AT	282190	432.	5.42	237.	183.	181.	5.59
ROUTED TO	282200	427.	5.50	237.	182.	180.	5.59
HYDROGRAPH AT	282200	70.	5.33	15.	4.	4.	0.09
2 COMBINED AT	282200	493.	5.42	252.	186.	184.	5.68
HYDROGRAPH AT	282610	47.	5.25	10.	3.	2.	0.06
ROUTED TO	282620	47.	5.33	10.	3.	2.	0.06
HYDROGRAPH AT	282620	118.	5.33	27.	7.	7.	0.16
2 COMBINED AT	282620	165.	5.33	37.	9.	9.	0.22
ROUTED TO	282630	164.	5.33	37.	9.	9.	0.22
ROUTED TO	282640	163.	5.42	37.	9.	9.	0.22
HYDROGRAPH AT	282640	17.	5.25	4.	1.	1.	0.02
2 COMBINED AT	282640	178.	5.42	40.	10.	10.	0.24
ROUTED TO	282650	178.	5.42	40.	10.	10.	0.24
ROUTED TO	282200	177.	5.42	40.	10.	10.	0.24
2 COMBINED AT	282200	671.	5.42	291.	197.	194.	5.91
ROUTED TO	282210	668.	5.42	291.	197.	194.	5.91
ROUTED TO	282220	664.	5.50	291.	197.	194.	5.91
HYDROGRAPH AT	282220	82.	5.33	18.	5.	4.	0.10
2 COMBINED AT	282220	732.	5.50	307.	201.	199.	6.02
HYDROGRAPH AT	282710	160.	5.33	35.	9.	9.	0.20
ROUTED TO	282720	158.	5.42	35.	9.	9.	0.20

2 COMBINED AT	282720	271.	5.33	60.	15.	15.	0.35		
ROUTED TO	282220	267.	5.42	60.	15.	15.	0.35		
2 COMBINED AT	282220	998.	5.42	365.	217.	213.	6.37		
ROUTED TO	282230	987.	5.42	365.	217.	213.	6.37		
HYDROGRAPH AT	282230	202.	5.33	44.	11.	11.	0.26		
2 COMBINED AT	282230	1177.	5.42	407.	228.	224.	6.63		
ROUTED TO	282240	1174.	5.42	407.	228.	224.	6.63		
HYDROGRAPH AT	282810	52.	5.08	11.	3.	3.	0.05		
ROUTED TO	282820	52.	5.17	11.	3.	3.	0.05		
HYDROGRAPH AT	282820	75.	5.25	16.	4.	4.	0.09		
2 COMBINED AT	282820	126.	5.25	28.	7.	7.	0.14		
ROUTED TO	282830	126.	5.25	28.	7.	7.	0.14		
ROUTED TO	282840	126.	5.25	28.	7.	7.	0.14		
HYDROGRAPH AT	282840	17.	5.25	4.	1.	1.	0.02		
2 COMBINED AT	282840	142.	5.25	31.	8.	8.	0.16		
HYDROGRAPH AT	283010	29.	5.25	6.	2.	2.	0.03		
ROUTED TO	283010	25.	5.33	4.	1.	1.	0.03	4.19	5.33
ROUTED TO	283020	25.	5.42	4.	1.	1.	0.03		
HYDROGRAPH AT	283020	8.	5.25	2.	0.	0.	0.01		
2 COMBINED AT	283020	31.	5.42	6.	1.	1.	0.04		
ROUTED TO	283020	29.	5.50	5.	1.	1.	0.04	2.53	5.50
ROUTED TO	283030	28.	5.58	5.	1.	1.	0.04		
HYDROGRAPH AT	283030	96.	5.17	20.	5.	5.	0.08		
2 COMBINED AT	283030	96.	5.17	25.	7.	6.	0.12		
ROUTED TO	283040	96.	5.25	25.	7.	6.	0.12		
HYDROGRAPH AT	283040	32.	5.33	7.	2.	2.	0.04		
2 COMBINED AT	283040	127.	5.25	32.	8.	8.	0.16		
ROUTED TO	283050	127.	5.25	32.	8.	8.	0.16		
ROUTED TO	282840	126.	5.33	32.	8.	8.	0.16		
2 COMBINED AT	282840	269.	5.25	63.	16.	16.	0.32		
ROUTED TO	282840	113.	6.08	23.	6.	6.	0.32	20.96	6.08
ROUTED TO	282850	112.	6.08	23.	6.	6.	0.32		

2 COMBINED AT	282850	126.	6.08	31.	8.	8.	0.37		
ROUTED TO	282860	125.	6.17	31.	8.	8.	0.37		
HYDROGRAPH AT	282860	14.	5.33	3.	1.	1.	0.02		
2 COMBINED AT	282860	131.	6.17	34.	9.	9.	0.38		
HYDROGRAPH AT	283110	58.	5.25	13.	3.	3.	0.07		
ROUTED TO	283120	58.	5.33	13.	3.	3.	0.07		
HYDROGRAPH AT	283120	36.	5.25	8.	2.	2.	0.05		
2 COMBINED AT	283120	93.	5.25	20.	5.	5.	0.12		
ROUTED TO	283120	23.	6.42	6.	1.	1.	0.12	14.40	6.42
ROUTED TO	282860	23.	6.50	6.	1.	1.	0.12		
2 COMBINED AT	282860	134.	6.33	39.	10.	10.	0.50		
ROUTED TO	282870	133.	6.33	39.	10.	10.	0.50		
HYDROGRAPH AT	282870	24.	5.25	5.	1.	1.	0.03		
2 COMBINED AT	282870	140.	6.33	44.	12.	11.	0.54		
ROUTED TO	282880	140.	6.33	44.	12.	11.	0.54		
ROUTED TO	282890	140.	6.33	44.	12.	11.	0.54		
HYDROGRAPH AT	282890	98.	5.25	21.	5.	5.	0.13		
2 COMBINED AT	282890	170.	5.33	65.	17.	16.	0.66		
ROUTED TO	282900	169.	6.25	65.	17.	16.	0.66		
HYDROGRAPH AT	282900	37.	5.25	8.	2.	2.	0.05		
2 COMBINED AT	282900	204.	5.33	73.	19.	18.	0.71		
ROUTED TO	282910	200.	5.33	73.	19.	18.	0.71		
HYDROGRAPH AT	282910	54.	5.25	12.	3.	3.	0.07		
2 COMBINED AT	282910	253.	5.33	84.	22.	21.	0.78		
ROUTED TO	282230	251.	5.33	84.	22.	21.	0.78		
2 COMBINED AT	282230	1421.	5.42	490.	250.	246.	7.41		
ROUTED TO	282240	1419.	5.42	490.	250.	246.	7.41		
HYDROGRAPH AT	283210	14.	5.33	3.	1.	1.	0.02		
ROUTED TO	283220	14.	5.42	3.	1.	1.	0.02		
HYDROGRAPH AT	283220	41.	5.25	9.	2.	2.	0.05		
2 COMBINED AT	283220	53.	5.33	12.	3.	3.	0.07		
ROUTED TO	283230	53.	5.33	12.	3.	3.	0.07		

HYDROGRAPH AT	283240	55.	5.25	12.	3.	3.	0.07
2 COMBINED AT	283240	106.	5.33	24.	6.	6.	0.14
ROUTED TO	283250	106.	5.33	24.	6.	6.	0.14
ROUTED TO	283260	105.	5.42	24.	6.	6.	0.14
HYDROGRAPH AT	283260	119.	5.25	26.	7.	6.	0.15
2 COMBINED AT	283260	223.	5.33	49.	13.	12.	0.29
ROUTED TO	283270	222.	5.33	49.	13.	12.	0.29
ROUTED TO	283280	221.	5.42	49.	13.	12.	0.29
HYDROGRAPH AT	283280	8.	5.50	2.	1.	1.	0.02
2 COMBINED AT	283280	229.	5.42	52.	13.	13.	0.31
ROUTED TO	282240	228.	5.42	52.	13.	13.	0.31
2 COMBINED AT	282240	1647.	5.42	541.	264.	259.	7.71
ROUTED TO	280200	1634.	5.50	541.	263.	258.	7.71
HYDROGRAPH AT	280200	398.	5.08	79.	20.	20.	0.30
3 COMBINED AT	280200	2961.	5.42	867.	350.	342.	9.23
ROUTED TO	280210	2956.	5.42	867.	350.	342.	9.23
HYDROGRAPH AT	280210	95.	5.08	19.	5.	5.	0.07
2 COMBINED AT	280210	3013.	5.42	885.	355.	347.	9.30
HYDROGRAPH AT	283410	7.	5.58	2.	1.	1.	0.02
ROUTED TO	283420	7.	5.67	2.	1.	1.	0.02
HYDROGRAPH AT	283420	23.	5.58	7.	2.	2.	0.06
2 COMBINED AT	283420	30.	5.67	9.	2.	2.	0.08
ROUTED TO	283430	30.	5.67	9.	2.	2.	0.08
HYDROGRAPH AT	283430	30.	5.67	9.	2.	2.	0.08
2 COMBINED AT	283430	60.	5.67	18.	5.	5.	0.16
ROUTED TO	283440	60.	5.75	18.	5.	5.	0.16
HYDROGRAPH AT	283440	30.	5.17	8.	2.	2.	0.05
2 COMBINED AT	283440	82.	5.42	25.	7.	6.	0.21
HYDROGRAPH AT	283610	35.	5.08	9.	2.	2.	0.05
ROUTED TO	283620	35.	5.08	9.	2.	2.	0.05
ROUTED TO	283440	35.	5.08	9.	2.	2.	0.05
2 COMBINED AT	283440	113.	5.33	34.	9.	9.	0.26

HYDROGRAPH AT	283450	1.	5.00	0.	0.	0.	0.00		
2 COMBINED AT	283450	114.	5.33	34.	9.	9.	0.26		
ROUTED TO	283460	114.	5.33	34.	9.	9.	0.26		
ROUTED TO	283470	113.	5.33	34.	9.	9.	0.26		
HYDROGRAPH AT	283470	20.	5.17	4.	1.	1.	0.02		
2 COMBINED AT	283470	130.	5.33	39.	10.	10.	0.28		
ROUTED TO	283480	130.	5.33	39.	10.	10.	0.28		
ROUTED TO	280210	129.	5.33	39.	10.	10.	0.28		
2 COMBINED AT	280210	3142.	5.42	924.	365.	357.	9.58		
ROUTED TO	280220	3125.	5.42	924.	365.	357.	9.58		
HYDROGRAPH AT	280220	54.	5.08	11.	3.	3.	0.04		
2 COMBINED AT	280220	3161.	5.42	935.	368.	359.	9.62		
ROUTED TO	PARK	621.	7.58	326.	199.	191.	9.62	1374.27	7.58
ROUTED TO	291	620.	7.58	326.	198.	191.	9.62		
HYDROGRAPH AT	290	25.	5.00	5.	1.	1.	0.02		
ROUTED TO	DAM6	15.	5.33	2.	0.	0.	0.02	102.05	5.33
HYDROGRAPH AT	295	45.	5.08	9.	2.	2.	0.03		
ROUTED TO	291.1	45.	5.08	9.	2.	2.	0.03		
3 COMBINED AT	292	622.	7.58	327.	201.	194.	9.67		
ROUTED TO	331	622.	7.67	327.	201.	194.	9.67		
HYDROGRAPH AT	330	29.	5.08	6.	1.	1.	0.02		
ROUTED TO	DAM2	29.	5.25	3.	1.	1.	0.02	102.08	5.25
2 COMBINED AT	332	623.	7.67	327.	202.	194.	9.69		
ROUTED TO	231	623.	7.67	327.	201.	194.	9.69		
HYDROGRAPH AT	230	54.	5.08	10.	3.	3.	0.04		
2 COMBINED AT	232	624.	7.67	328.	204.	197.	9.73		
ROUTED TO	233	624.	7.67	328.	204.	197.	9.73		
HYDROGRAPH AT	220	625.	7.67	333.	208.	200.	9.81		
HYDROGRAPH AT	445	625.	7.75	336.	208.	201.	9.84		
HYDROGRAPH AT	310	23.	5.42	6.	2.	1.	0.03		
ROUTED TO	DAM4	0.	5.83	0.	0.	0.	0.03	101.66	12.17
ROUTED TO	312	0.	6.08	0.	0.	0.	0.03		

HYDROGRAPH AT	320	27.	5.00	5.	1.	1.	0.02		
2 COMBINED AT	322	27.	5.00	5.	1.	1.	0.05		
ROUTED TO	DAM3	21.	5.33	2.	1.	1.	0.05	102.06	5.33
HYDROGRAPH AT	260	58.	5.33	11.	3.	3.	0.09		
ROUTED TO	451	56.	5.33	11.	3.	3.	0.09		
HYDROGRAPH AT	450	71.	5.42	15.	4.	4.	0.12		
2 COMBINED AT	452	630.	7.67	348.	212.	204.	9.95		
HYDROGRAPH AT	455	632.	7.83	357.	213.	206.	10.02		
HYDROGRAPH AT	280	87.	5.17	18.	5.	4.	0.06		
ROUTED TO	DAM5	45.	5.58	7.	2.	2.	0.06	102.11	5.58
HYDROGRAPH AT	250	66.	5.67	16.	4.	4.	0.11		
HYDROGRAPH AT	460	122.	5.92	34.	9.	9.	0.24		
2 COMBINED AT	461	653.	7.75	390.	222.	214.	10.25		
HYDROGRAPH AT	495B	658.	7.92	409.	225.	217.	10.38		
2 COMBINED AT	495B2	842.	5.58	468.	243.	234.	10.42		
HYDROGRAPH AT	270	66.	5.17	13.	3.	3.	0.06		
HYDROGRAPH AT	470	107.	5.33	22.	6.	6.	0.13		
HYDROGRAPH AT	2223	60.	5.50	17.	4.	4.	0.00		
HYDROGRAPH AT	480	98.	5.67	26.	7.	7.	0.08		
2 COMBINED AT	481	190.	5.42	48.	13.	12.	0.21		
HYDROGRAPH AT	490	329.	5.67	85.	22.	21.	0.41		
2 COMBINED AT	491	1153.	5.67	553.	265.	255.	10.84		
HYDROGRAPH AT	550B	1189.	5.67	562.	267.	257.	10.89		
ROUTED TO	1992	1182.	5.75	562.	265.	256.	10.89		
2 COMBINED AT	1993	1570.	5.83	697.	301.	290.	11.65		
2 COMBINED AT	1994	6910.	6.58	3286.	1017.	980.	29.49		
ROUTED TO	2001	6879.	6.67	3282.	1015.	977.	29.49	10.21	6.67
HYDROGRAPH AT	2000	75.	5.75	21.	5.	5.	0.12		
2 COMBINED AT	2002	6913.	6.67	3300.	1020.	983.	29.60		
ROUTED TO	541	6894.	6.75	3298.	1018.	981.	29.60	10.22	6.75
HYDROGRAPH AT	540B	105.	5.67	27.	7.	7.	0.15		
2 COMBINED AT	542	6931.	6.75	3320.	1025.	988.	29.75		
ROUTED TO	546	6928.	6.75	3319.	1024.	986.	29.75	10.24	6.75

2 COMBINED AT	546T	6931.	6.75	3320.	1025.	987.	29.75		
DIVERSION TO	546CAN	2907.	6.75	679.	170.	164.	29.75		
HYDROGRAPH AT	546ADT	4024.	6.75	2641.	855.	824.	29.75		
2 COMBINED AT	547	4744.	6.08	2896.	930.	896.	31.49		
ROUTED TO	531	4631.	6.33	2886.	926.	892.	31.49	17.18	6.33
HYDROGRAPH AT	560W	52.	5.75	15.	4.	4.	0.09		
ROUTED TO	531A	52.	6.00	15.	4.	4.	0.09		
HYDROGRAPH AT	530AW	97.	5.67	26.	7.	7.	0.15		
3 COMBINED AT	532	4727.	6.33	2920.	937.	902.	31.72		
ROUTED TO	537	4700.	6.42	2914.	934.	899.	31.72	13.37	6.42
HYDROGRAPH AT	360W	35.	6.08	11.	3.	3.	0.09		
HYDROGRAPH AT	420W	12.	5.42	2.	1.	1.	0.02		
HYDROGRAPH AT	410W	15.	5.50	3.	1.	1.	0.02		
2 COMBINED AT	381	26.	5.42	6.	1.	1.	0.04		
HYDROGRAPH AT	380	36.	5.42	8.	2.	2.	0.06		
HYDROGRAPH AT	400W	38.	5.42	8.	2.	2.	0.05		
2 COMBINED AT	371	73.	5.42	16.	4.	4.	0.11		
HYDROGRAPH AT	370	100.	5.58	22.	6.	5.	0.16		
2 COMBINED AT	372	120.	5.67	33.	8.	8.	0.25		
ROUTED TO	373	118.	5.67	33.	8.	8.	0.25	8.76	5.67
ROUTED TO	501	115.	5.83	32.	8.	8.	0.25	9.05	5.83
HYDROGRAPH AT	510	58.	6.00	18.	5.	4.	0.14		
HYDROGRAPH AT	500W	179.	5.83	51.	13.	13.	0.28		
3 COMBINED AT	502	348.	5.83	101.	26.	25.	0.67		
ROUTED TO	516	345.	5.92	101.	26.	25.	0.67	10.16	5.92
HYDROGRAPH AT	515	20.	5.58	5.	1.	1.	0.03		
2 COMBINED AT	517	361.	5.92	106.	27.	26.	0.70		
ROUTED TO	518	360.	5.92	106.	27.	26.	0.70	10.79	5.92
ROUTED TO	519	359.	6.00	106.	27.	26.	0.70	10.95	6.00
HYDROGRAPH AT	520A	155.	5.75	43.	11.	11.	0.25		
2 COMBINED AT	519A	504.	5.92	149.	39.	37.	0.95		
ROUTED TO	521	493.	6.00	149.	39.	37.	0.95	10.33	6.00

ROUTED TO	DAMP1	194.	5.17	22.	6.	5.	0.13	110.88	5.17
2 COMBINED AT	522	539.	6.00	171.	44.	43.	1.08		
ROUTED TO	535A1	534.	6.08	171.	44.	43.	1.08	10.27	6.08
HYDROGRAPH AT	535A	154.	5.00	29.	7.	7.	0.09		
ROUTED TO	DAMP2	61.	5.50	9.	2.	2.	0.09	110.41	5.50
2 COMBINED AT	535A2	565.	6.08	180.	47.	45.	1.18		
ROUTED TO	535D1	564.	6.08	180.	47.	45.	1.18	10.32	6.08
HYDROGRAPH AT	535D	4.	5.08	1.	0.	0.	0.00		
2 COMBINED AT	535D2	565.	6.08	180.	47.	45.	1.18		
DIVERSION TO	CWRW	0.	6.08	0.	0.	0.	1.18		
HYDROGRAPH AT	CWRS	565.	6.08	180.	47.	45.	1.18		
ROUTED TO	535B1	566.	6.08	180.	47.	45.	1.18	10.73	6.08
HYDROGRAPH AT	535E	1.	5.00	0.	0.	0.	0.00		
ROUTED TO	535B2	1.	5.08	0.	0.	0.	0.00		
HYDROGRAPH AT	535B	18.	5.00	3.	1.	1.	0.01		
3 COMBINED AT	535B3	569.	6.08	183.	48.	46.	1.19		
ROUTED TO	535C1	562.	6.17	182.	48.	46.	1.19	10.72	6.17
HYDROGRAPH AT	535C	16.	5.42	4.	1.	1.	0.02		
3 COMBINED AT	548	5205.	6.42	3082.	982.	946.	32.93		
HYDROGRAPH AT	546CAN	2907.	6.75	679.	170.	164.	0.00		
ROUTED TO	549	2827.	6.92	679.	170.	164.	0.00	7.60	6.92
DIVERSION TO	551	2247.	6.92	621.	155.	150.	0.00		
HYDROGRAPH AT	550	580.	6.92	58.	14.	14.	0.00		
2 COMBINED AT	552	5477.	6.83	3140.	997.	960.	32.93		

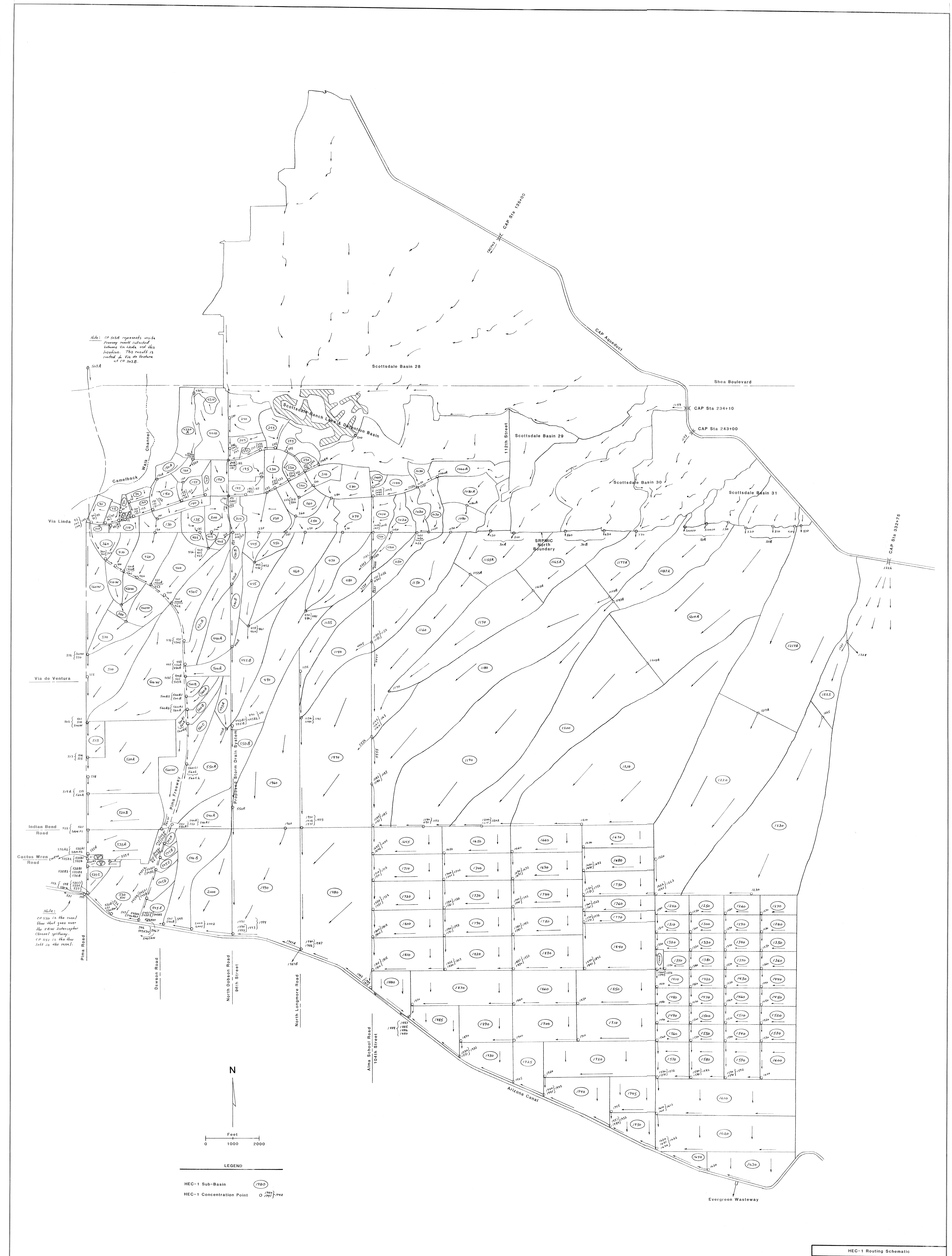
APPENDIX B

Magnetic Disk
HEC-1 Input Files

Pima Freeway
Arizona Canal Drainage Channel

Models:

PF1.12I
PF2.12I
PF3.7I
PF4.7I



HEC-1 Routing Schematic

Pima Freeway / SRPMIC Drainage Analysis

Robert L. Ward, P.E., Consulting Engineer

August 1996 | PLATE 1