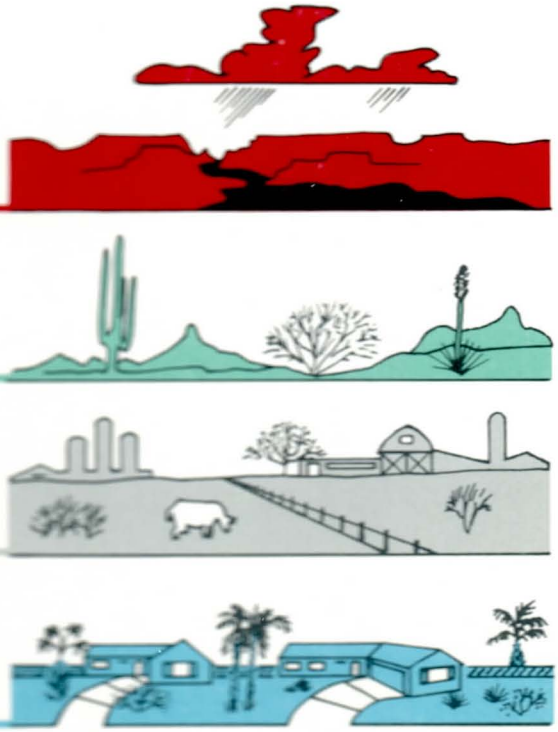

FLOOD CONTROL
DISTRICT OF
MARICOPA COUNTY

HYDRO LIBRARY

ANALYSIS OF W.S. GOOKIN
REPORT ON CUDIA CITY WASH

Brian C. Dietterick
Hydrologist

Timothy E. Sutko
Hydrologist

MAR 12 1987

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

24-9-08-1-02/87

Property of
Flood Control District of MC Library
Please Return to
2801 W. Durango
Phoenix, AZ 85009

ANALYSIS OF W.S. GOOKIN
REPORT ON CUDIA CITY WASH

Brian C. Dietterick
Hydrologist

Timothy E. Sutko
Hydrologist

MAR 12 1987

INTRODUCTION

W.S. Gookin & Associates, Consulting Engineers (WSG), on behalf of the Citizens Against Reach Four, completed a study in October, 1986 entitled "Cudia City Wash Runoff Analysis". The purpose of this study was to determine the peak discharge on Cudia City Wash at the Arizona Canal resulting from a 100-year storm event and evaluate the effect of a detention basin on the outflow into the proposed Arizona Canal Diversion Channel. The Watershed Hydrology branch of the Flood Control District of Maricopa County was directed to review this report and offer comments on it. Watershed Hydrology was further directed to perform its own separate analysis to determine what it believed the peak discharge from the 100-year storm centered on the Cudia City Wash area would be. The following describes the findings of that review and analysis.

The Cudia City Wash watershed is located in the northeast section of the City of Phoenix, and is generally bounded by Camelback Mountain, Mummy Mountain, and the Phoenix Mountains. The watershed is characterized by mild to steep slopes and sparse to moderate vegetation. Historically, this area has been subject to intense thunderstorms that have caused damaging floods.

WSG MODEL

Design Storm

The storm used for the WSG analysis had a total depth of 3.80 inches and a Soil Conservation Service (SCS) Type IIA temporal distribution. The rationale given by WSG for the use of this design storm was that "this is the standard storm distribution agreed upon...for Scottsdale." The total storm depth was determined using the isohyetal maps contained in Hydrologic Design for Highway Drainage in Arizona (ADOT,1975).

The selection of a storm for use in developing the hydrology for the design of a drainage facility is, to some extent, arbitrary. It is common and accepted practice to use one storm for the design of an entire drainage facility, as opposed to using different storms over each of the various areas contributing runoff to that facility. The purpose of this practice is to insure continuity of the assumptions and of the design. Thus, it is inappropriate, when the entire Arizona Canal Diversion Channel (ACDC) project is considered, to use a different design storm over the Cudia City Wash area than was used for the remainder of the project's contributing area.

However, inasmuch as the intent of the WSG analysis was to determine the 100-year peak discharge on Cudia City Wash at the Arizona Canal, and not to design the ACDC, a case might be made for using a design storm and methodology other than those used by the U.S. Army Corps of Engineers (COE) in their design of ACDC. In this instance it is common practice to use, when available, the storm used by the design engineers of the municipality within whose political boundaries the drainage area lies. The purpose again is to insure continuity with other drainage facilities.

The entire Cudia City Wash drainage area is contained within the political boundaries of the City of Phoenix and the Town of Paradise Valley. The 100-year 24-hour design rainfall depth used by the engineers for these municipalities is 4.04 inches with a time distribution developed by the City of Phoenix.

The use of the Phoenix storm distribution rather than the SCS Type IIA storm would affect the timing of the peak runoff, but not significantly alter its magnitude. However, the greater storm depth would increase the magnitude of the peak discharge.

FCD Model Simulating WSG Model

To determine the increase in the magnitude of the peak discharge using the Phoenix design storm criteria, a computer model was created using the COE Flood Hydrograph Package (HEC-1) and the WSG modeling parameters as listed in their report. These parameters include: storm depth of 3.80 inches, Type IIA storm distribution, curve numbers, lag times, and channel lengths.

Using the SCS hydrograph method, combined with the kinematic wave channel routing technique, the FCD simulated hydrographs for each subbasin. The FCD estimates of the peak discharges from each of the subbasins range from 19%-51% lower than the WSG estimates. The final routed and combined FCD hydrograph estimates the peak discharge at the Arizona Canal and matches WSG's estimate of the peak discharge to within 13%. However, at this point the FCD estimate of the peak discharge is higher than the the WSG estimate. Peak discharges for the individual subbasins and for the total basin are listed in Table 1. Appendix A contains a copy of this HEC-1 input deck and output summary.

The change from low comparative subbasin estimates to a high combined estimate suggests that differences in the routing techniques may be the cause. The FCD modeling scheme used an accepted, technically sound routing method (kinematic wave) in its HEC-1 model. The WSG model used the Lotus Spreadsheet to combine and route the subbasin hydrographs. If WSG had used the kinematic wave routing technique, their combined peak discharge would probably have been greater than the FCD estimate.

It was noted on the output from the FCD version of the WSG modeling scheme that a number of warnings were given concerning the length of the computation interval relative to the lag time used. If the computation interval is not acceptably small compared to the lag time, there is a good possibility that

Table 1. Comparison of subbasin discharges (cfs) and total basin discharges between the FCD model and the WSG model.

Subbasin	FCD model	WSG model	% Greater(WSG v FCD)
1	639	760	19
2	69	86	25
3	27	34	26
4	371	455	23
5	21	27	29
6	150	195	30
7	41	54	32
8	286	345	21
9	517	780	51
10	299	415	39
11	638	800	25
12	1238	*	
13	538	*	
14	883	*	
Total Routed Q	4835	4422	

* not calculated individually for these subbasins

peak discharges will be miscalculated. The peak discharge at the Arizona Canal from the FCD version of the WSG model, using a 6 minute computation interval, was 4835 cfs. This value rose to 5094 cfs when the computation interval was reduced to 2 minutes. These values are to be compared with a peak discharge of 4422 cfs in the WSG report. Appendix B contains a copy of the HEC-1 input deck and output summary for the computer model using a 2 minute computation interval.

Sensitivity of FCD Simulation Model to Storm Depth and Distribution

Assuming that WSG correctly modeled the Cudia City Wash area and that the FCD version was a reasonable re-creation of that model, FCD then determined the effect a different design storm on the peak discharge. The City of Phoenix design storm and distribution, as described earlier, were used as input to the model. The result is a peak discharge of 5271 cfs at the Arizona Canal. This result was based on a 6 minute computation interval. Appendix C contains a copy of the HEC-1 input deck and output summary for this computer model. Using a 2 minute computation interval the peak discharge is 5402 cfs.

WSG Report Conclusions

WSG evaluated the results of their model using estimates from a historic storm that occurred June 21-22, 1972. WSG stated that this storm "approximated a 100-year storm". However, FCD does not believe that a recurrence interval was ever assigned to this storm. The limited data available on this storm suggests that the average storm depth over the Cudia City Wash area was approximately 3.48 inches. This storm is probably not representative of the 100-year 24-hour, and any direct comparisons of the two are invalid.

The COE released a Report on Flood of 22 June 1972 in October, 1972. This report presents estimates of the storm depth and distribution, and of the subsequent runoff and the damages which it caused. The peak discharge from this storm at a point on Cudia City Wash 1000 feet north of McDonald Drive was estimated to have been 4200 cfs from a contributing area of 2.16 square miles in the COE report. This estimate was determined by the U.S. Geological Service using the slope-area method. The WSG report cites SRP as estimating the peak

discharge into the Arizona Canal during that storm as being 3375 cfs from a contributing area of 5.12 square miles. SRP typically estimates storm water inflow to their canal systems by estimating the peak outflow and a change in canal storage (principle of continuity). While both of these methods are useful, the large disparity in their estimated values diminishes the use of either as a point for comparison.

INDEPENDENT FCD MODEL

The SCS Method has long been considered a reasonable means for generating storm runoff. However, this method was developed for natural catchments and its appropriateness for estimating the discharge from urban areas of this type is questionable.

Kinematic wave modeling is a physically-based, conceptual approach for generating runoff from areas where the primary flow regime is sheetflow. The input parameters are easily measured from topographic maps. Urbanized areas are particularly amenable to this type of modeling scheme.

FCD hydrology staff considered a combination of the kinematic wave and the SCS methods to be the most valid technique for generating runoff from this study area. The staff used HEC-1 to develop an independent model for this area. The kinematic wave technique was used on the urbanized subbasins having relatively less steep slopes. The SCS method was used on the two subbasins having steep slopes over a significant portion of the total area. The subbasin delineations were essentially the same as those used in the WSG analysis.

After measuring the model's physical input parameters, numerous differences between FCD estimates and the estimates WSG used in their model were noted. Differences were found in subbasin areas, channel slopes, and times of concentration. Table 2 presents FCD's and WSG's estimates of these parameters. Lag time, referred to as the time of concentration by WSG, is a function of both channel length and channel slope. Thus, the lag times used in the WSG study are also suspect and are capable of changing the results significantly.

A 100-year 24-hour storm depth of 4.04 inches with the City of Phoenix storm distribution was used. Rainfall excess was determined using SCS curve numbers. Because two overland flow planes (describing pervious and impervious surfaces) were used, two sets of curve numbers were required as input for each

Table 2. Differences between the measured physical parameters.

	Area (mi^2)		Channel length (ft)		Slope (%) (ft/ft)	
Sub	FCD	WSG	FCD	WSG	FCD	WSG
1	0.37	0.40	6100	6706	0.043	0.147
2	0.05	0.05	1200	2376	0.050	0.143
3	0.03	0.02	1000	1056	0.050	0.076
4	0.21	0.25	3400	4330	0.059	0.138
5	0.03	0.02	800	1320	0.013	0.043
6	0.12	0.13	3200	3854	0.026	0.028
7	0.04	0.04	2000	2904	0.030	0.036
8	0.17	0.19	4200	5280	0.048	0.116
9	0.38	0.41	5200	6494	0.042	0.144
10	0.50	0.55	5800	5280	0.024	0.175
11	0.63	0.55	9800	9610	0.168	0.113
12	1.30	1.25	4800	11510	0.007	0.122
13	0.38	0.38	4600	5755	0.045	0.111
14	0.71	0.88	5400	7181	0.008	0.029

subbasin. The weighted average of these curve number pairs agrees with the curve numbers used by WSG.

The estimated peak discharge for the combination kinematic wave/SCS model is 6570 cfs.

SUMMARY

Numerous problems with the WSG modeling effort became evident during our review. These include 1) the apparent specification of too long a computation interval, 2) the use of a design rainfall depth and distribution inappropriate to the study area, 3) the contrasting results in hydrographs for individual subbasins and for hydrographs resulting from combining and routing flows from various subbasins, 4) the apparent measurement errors of channel length, channel slope, and subbasin areas, and 5) the suspect times of concentration, or lag times. The WSG report indicates that the peak discharge from Cudia City Wash at the Arizona Canal resulting from a 100-year 24-hour storm event is 4422 cfs. Assuming that the WSG modeling efforts provide a reasonable description of the study area, FCD ran that same model using a shorter computation interval and the City of Phoenix design storm. This model indicates that the peak discharge is 5400 cfs at the same location. FCD then created an independent computer model of the area using what we consider to be the most appropriate methodologies and design rainfall. The peak discharge from this model at the Arizona Canal is 6570 cfs.

WSG attempted to validate their model by comparing their results to estimated discharges at the Arizona Canal resulting from the June 21-22, 1972 storm. We do not believe that direct comparisons between the historic storm and the 100-year design storm, or between their resulting discharges, are valid.

APPENDIX A

```
                    LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

                                  *DIAGRAM
*** FREE ***
                      1           ID   CUDIA CITY WASH
                      2           ID   RUNOFF ANALYSIS USING THE 100YR 24HR STORM
                      3           IT       6  1JAN87    1200     250
                      4           IO       5

                      5           KK    SUB8
                      6           IN      15
                      7           PB    3.30
                      8           PC     .00     .00     .01     .02     .02     .03     .04     .05     .05     .06
                      9           PC     .07     .08     .08     .09     .10     .11     .13     .14     .15     .18
                     10           PC     .21     .24     .42     .87    2.07    2.77    2.83    2.89    2.94    3.00
                     11           PC    3.04    3.07    3.09    3.12    3.15    3.17    3.20    3.22    3.24    3.26
                     12           PC    3.28    3.29    3.31    3.32    3.34    3.36    3.37    3.39    3.40    3.41
                     13           PC    3.42    3.43    3.44    3.45    3.47    3.48    3.49    3.50    3.51    3.52
                     14           PC    3.53    3.55    3.56    3.57    3.58    3.59    3.60    3.61    3.61    3.62
                     15           PC    3.63    3.64    3.64    3.65    3.66    3.67    3.67    3.68    3.69    3.70
                     16           PC    3.70    3.71    3.72    3.73    3.74    3.74    3.75    3.76    3.77    3.77
                     17           PC    3.78    3.78    3.78    3.79    3.79    3.80    3.80    3.80    3.80    3.80
                     18           BA    0.19
                     19           LS       0    87.2
                     20           UD    0.29

                     21           KK   ROUTE
                     22           RS       1    STOR       0
                     23           SV       0    2.05
                     24           SE       1      10
                     25           SQ       0     115
                     26           ST      10      95   0.611     1.5

                     27           KK    R8-9
                     28           RK    4700   0.016   0.030            TRAP       5       2

                     29           KK    SUB7
                     30           BA     .04
                     31           LS       0      86
                     32           UD    0.57

                     33           KK   ROUTE
                     34           RS       1    STOR       0
                     35           SV       0    0.78
                     36           SE       1      10
                     37           SQ       0      28

                     38           KK    R7-9
                     39           RK    2482   0.018   0.030            TRAP       5       2

                     40           KK    SUB9
                     41           BA    0.41
                     42           LS       0    85.4
                     43           UD    0.37
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  44          KKCB9(7&8)
  45          HC       3

  46          KK    ROUTE
  47          RK     2500   0.008   0.030            TRAP      5       2

  48          KK     SUB3
  49          BA     0.02
  50          LS        0      86
  51          UD     0.33

  52          KK    R3-12
  53          RK     3696   0.016   0.030            TRAP     10       2

  54          KK     SUB4
  55          BA     0.25
  56          LS        0    87.8
  57          UD     0.32

  58          KK    R4-12
  59          RK     3590   0.014   0.030            TRAP     10       2

  60          KK     SUB5
  61          BA     0.02
  62          LS        0      86
  63          UD     0.52

  64          KK    ROUTE
  65          RS        1    STOR       0
  66          SV        0    0.42
  67          SE        1      10
  68          SQ        0      14

  69          KK    R5-12
  70          RK     3590   0.014   0.030            TRAP      5       2

  71          KK     SUB6
  72          BA     0.13
  73          LS        0      89
  74          UD     0.56

  75          KK    ROUTE
  76          RS        1    STOR       0
  77          SV        0    0.69
  78          SE        1      10
  79          SQ        0     156

  80          KK    R6-12
  81          RK     3590   0.009   0.030            TRAP     15       2
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  82          KK   SUB10
  83          BA    0.55
  84          LS       0    88.4
  85          UD    1.47

  86          KK  R10-12
  87          RK    5016   0.008   0.030            TRAP      15       2

  88          KK   SUB11
  89          BA    0.55
  90          LS       0    87.5
  91          UD    0.51

  92          KK  R11-12
  93          RK    3696   0.009   0.030            TRAP      15       2

  94          KK   SUB12
  95          BA    1.25
  96          LS       0    88.7
  97          UD    0.70

  98          KK COMBINE
  99          HC       8

 100          KK  R12-13
 101          RK    2006   0.007   0.030            TRAP      20       2

 102          KK   SUB13
 103          BA    0.38
 104          LS       0    88.8
 105          UD    0.39

 106          KK CB12&13
 107          HC       2

 108          KK  R13-14
 109          RK    5597   0.009   0.030            TRAP      25       2

 110          KK    SUB1
 111          BA    0.40
 112          LS       0      89
 113          UD    0.30

 114          KK   ROUTE
 115          RS       1    STOR       0
 116          SV       0    2.05
 117          SE       1      10
 118          SQ       0      13
 119          ST      10      95   0.611     1.5
```

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*            FEBRUARY 1981             *
*          REVISED 31 JAN 85           *
*                                      *
*  RUN DATE 3/11/1987   TIME14:12:14 * 
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*  THE HYDROLOGIC ENGINEERING CENTER   *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*  (916) 440-3285 OR (FTS) 448-3285    *
*                                      *
****************************************
```

```
                         CUDIA CITY WASH
                         RUNOFF ANALYSIS USING THE 100YR 24HR STORM

   4 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

     IT          HYDROGRAPH TIME DATA
                        NMIN           6  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1JAN87  STARTING DATE
                       ITIME        1200  STARTING TIME
                          NQ         250  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      2JAN87  ENDING DATE
                      NDTIME        1254  ENDING TIME

                  COMPUTATION INTERVAL     .10 HOURS
                       TOTAL TIME BASE   24.90 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

 WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

 WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

 WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

 WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

 WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  120         KK   R1-14
  121         RK    7498   0.020   0.030            TRAP      25       2

  122         KK    SUB2
  123         BA    0.05
  124         LS       0      86
  125         UD    0.32

  126         KK   ROUTE
  127         RS       1    STOR       0
  128         SV       0    3.28
  129         SE       1      10
  130         SQ       0       8

  131         KK   R2-14
  132         RK    7603   0.016   0.030            TRAP      25       2

  133         KK   SUB14
  134         BA    0.88
  135         LS       0      89
  136         UD    0.70

  137         KKCB14&13(3,4,5,11,6,7,8,9,10 & 12)
  138         HC       4
  139         ZZ
```

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	SUB8	286.	6.30	42.	13.	12.	.19		
ROUTED TO	ROUTE	288.	6.30	42.	13.	12.	.19	11.88	6.30
ROUTED TO	R8-9	280.	6.40	42.	13.	12.	.19		
HYDROGRAPH AT	SUB7	41.	6.60	8.	3.	2.	.04		
ROUTED TO	ROUTE	32.	6.80	8.	3.	2.	.04	11.24	6.80
ROUTED TO	R7-9	32.	6.90	8.	3.	2.	.04		
HYDROGRAPH AT	SUB9	517.	6.40	85.	26.	25.	.41		
3 COMBINED AT	CB9(7&8)	811.	6.40	134.	41.	39.	.64		
ROUTED TO	ROUTE	788.	6.40	134.	41.	39.	.64		
HYDROGRAPH AT	SUB3	27.	6.30	4.	1.	1.	.02		
ROUTED TO	R3-12	27.	6.50	4.	1.	1.	.02		
HYDROGRAPH AT	SUB4	371.	6.30	57.	17.	16.	.25		
ROUTED TO	R4-12	368.	6.40	58.	17.	17.	.25		
HYDROGRAPH AT	SUB5	21.	6.50	4.	1.	1.	.02		
ROUTED TO	ROUTE	16.	6.80	4.	1.	1.	.02	11.47	6.80
ROUTED TO	R5-12	16.	7.00	4.	1.	1.	.02		
HYDROGRAPH AT	SUB6	150.	6.60	31.	9.	9.	.13		
ROUTED TO	ROUTE	150.	6.60	31.	9.	9.	.13	9.64	6.60
ROUTED TO	R6-12	146.	6.70	31.	9.	9.	.13		
HYDROGRAPH AT	SUB10	299.	7.50	124.	38.	37.	.55		
ROUTED TO	R10-12	298.	7.60	123.	38.	36.	.55		
HYDROGRAPH AT	SUB11	638.	6.50	123.	37.	36.	.55		
ROUTED TO	R11-12	635.	6.60	123.	37.	36.	.55		
HYDROGRAPH AT	SUB12	1238.	6.70	292.	88.	84.	1.25		
8 COMBINED AT	COMBINE	3080.	6.60	768.	232.	224.	3.41		
ROUTED TO	R12-13	3068.	6.60	767.	232.	223.	3.41		
HYDROGRAPH AT	SUB13	538.	6.40	90.	27.	26.	.38		
2 COMBINED AT	CB12&13	3490.	6.60	855.	259.	249.	3.79		

ROUTED TO	R13-14	3476.	6.60	852.	258.	249.	3.79		
HYDROGRAPH AT	SUB1	639.	6.30	95.	28.	27.	.40		
ROUTED TO	ROUTE	639.	6.30	92.	28.	27.	.40	14.84	6.30
ROUTED TO	R1-14	618.	6.40	91.	28.	27.	.40		
HYDROGRAPH AT	SUB2	69.	6.30	11.	3.	3.	.05		
ROUTED TO	ROUTE	9.	7.20	7.	3.	3.	.05	10.68	7.20
ROUTED TO	R2-14	8.	8.10	7.	3.	3.	.05		
HYDROGRAPH AT	SUB14	883.	6.70	208.	62.	60.	.88		
4 COMBINED AT	CB14&13(	4835.	6.60	1155.	351.	338.	5.12		

APPENDIX B

```
                 LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

                               *DIAGRAM
*** FREE ***
                    1          ID   CUDIA CITY WASH
                    2          ID   RUNOFF ANALYSIS USING THE 100YR 24HR STORM
                    3          IT       2  1JAN87    1200     800
                    4          IO       5

                    5          KK    SUB8
                    6          IN      15
                    7          PB    3.80
                    8          PC     .00     .00     .01     .02     .02     .03     .04     .05     .05     .06
                    9          PC     .07     .08     .08     .09     .10     .11     .13     .14     .15     .18
                   10          PC     .21     .24     .42     .87    2.07    2.77    2.83    2.89    2.94    3.00
                   11          PC    3.04    3.07    3.09    3.12    3.15    3.17    3.20    3.22    3.24    3.26
                   12          PC    3.28    3.29    3.31    3.32    3.34    3.36    3.37    3.39    3.40    3.41
                   13          PC    3.42    3.43    3.44    3.45    3.47    3.48    3.49    3.50    3.51    3.52
                   14          PC    3.53    3.55    3.56    3.57    3.58    3.59    3.60    3.61    3.61    3.62
                   15          PC    3.63    3.64    3.64    3.65    3.66    3.67    3.67    3.68    3.69    3.70
                   16          PC    3.70    3.71    3.72    3.73    3.74    3.74    3.75    3.76    3.77    3.77
                   17          PC    3.78    3.78    3.78    3.79    3.79    3.80    3.80    3.80    3.80    3.80
                   18          BA    0.19
                   19          LS       0    87.2
                   20          UD    0.29

                   21          KK    ROUTE
                   22          RS       1    STOR       0
                   23          SV       0    2.05
                   24          SE       1      10
                   25          SG       0     115
                   26          ST      10      95   0.611     1.5

                   27          KK    R8-9
                   28          RK    4700   0.016   0.030            TRAP      5       2

                   29          KK    SUB7
                   30          BA     .04
                   31          LS       0      86
                   32          UD    0.57

                   33          KK    ROUTE
                   34          RS       1    STOR       0
                   35          SV       0    0.78
                   36          SE       1      10
                   37          SQ       0      28

                   38          KK    R7-9
                   39          RK    2482   0.018   0.030            TRAP      5       2

                   40          KK    SUB9
                   41          BA    0.41
                   42          LS       0    85.4
                   43          UD    0.37
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  44          KKCB9(7&8)
  45          HC        3

  46          KK    ROUTE
  47          RK     2500   0.008   0.030            TRAP      5       2

  48          KK     SUB3
  49          BA     0.02
  50          LS        0      86
  51          UD     0.33

  52          KK    R3-12
  53          RK     3696   0.016   0.030            TRAP     10       2

  54          KK     SUB4
  55          BA     0.25
  56          LS        0    87.8
  57          UD     0.32

  58          KK    R4-12
  59          RK     3590   0.014   0.030            TRAP     10       2

  60          KK     SUB5
  61          BA     0.02
  62          LS        0      86
  63          UD     0.52

  64          KK    ROUTE
  65          RS        1    STOR       0
  66          SV        0    0.42
  67          SE        1      10
  68          SQ        0      14

  69          KK    R5-12
  70          RK     3590   0.014   0.030            TRAP      5       2

  71          KK     SUB6
  72          BA     0.13
  73          LS        0      89
  74          UD     0.56

  75          KK    ROUTE
  76          RS        1    STOR       0
  77          SV        0    0.69
  78          SE        1      10
  79          SQ        0     156

  80          KK    R6-12
  81          RK     3590   0.009   0.030            TRAP     15       2
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  82          KK   SUB10
  83          BA    0.55
  84          LS       0    88.4
  85          UD    1.47

  86          KK  R10-12
  87          RK    5016   0.008   0.030            TRAP      15       2

  88          KK   SUB11
  89          BA    0.55
  90          LS       0    87.5
  91          UD    0.51

  92          KK  R11-12
  93          RK    3696   0.009   0.030            TRAP      15       2

  94          KK   SUB12
  95          BA    1.25
  96          LS       0    88.7
  97          UD    0.70

  98          KK COMBINE
  99          HC       8

 100          KK  R12-13
 101          RK    2006   0.007   0.030            TRAP      20       2

 102          KK   SUB13
 103          BA    0.38
 104          LS       0    88.8
 105          UD    0.39

 106          KK CB12&13
 107          HC       2

 108          KK  R13-14
 109          RK    5597   0.009   0.030            TRAP      25       2

 110          KK    SUB1
 111          BA    0.40
 112          LS       0      89
 113          UD    0.30

 114          KK   ROUTE
 115          RS       1    STOR       0
 116          SV       0    2.05
 117          SE       1      10
 118          SQ       0      13
 119          ST      10      95   0.611     1.5
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 120          KK   R1-14
 121          RK    7498   0.020   0.030            TRAP      25       2

 122          KK    SUB2
 123          BA    0.05
 124          LS       0      86
 125          UD    0.32

 126          KK   ROUTE
 127          RS       1    STOR       0
 128          SV       0    3.28
 129          SE       1      10
 130          SQ       0       8

 131          KK   R2-14
 132          RK    7603   0.016   0.030            TRAP      25       2

 133          KK   SUB14
 134          BA    0.88
 135          LS       0      89
 136          UD    0.70

 137          KKCB14&13(3,4,5,11,6,7,8,9,10 & 12)
 138          HC       4
 139          ZZ
```

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	SUB8	300.	6.30	42.	13.	11.	.19		
ROUTED TO	ROUTE	299.	6.30	42.	13.	11.	.19	11.96	6.30
ROUTED TO	R8-9	293.	6.40	42.	13.	11.	.19		
HYDROGRAPH AT	SUB7	42.	6.57	8.	3.	2.	.04		
ROUTED TO	ROUTE	33.	6.83	8.	3.	2.	.04	11.63	6.83
ROUTED TO	R7-9	33.	6.90	8.	3.	2.	.04		
HYDROGRAPH AT	SUB9	545.	6.37	85.	26.	23.	.41		
3 COMBINED AT	CB9(7&8)	850.	6.40	135.	41.	37.	.64		
ROUTED TO	ROUTE	846.	6.43	135.	41.	37.	.64		
HYDROGRAPH AT	SUB3	29.	6.33	4.	1.	1.	.02		
ROUTED TO	R3-12	28.	6.47	4.	1.	1.	.02		
HYDROGRAPH AT	SUB4	390.	6.33	57.	17.	15.	.25		
ROUTED TO	R4-12	389.	6.37	57.	17.	15.	.25		
HYDROGRAPH AT	SUB5	23.	6.53	4.	1.	1.	.02		
ROUTED TO	ROUTE	17.	6.80	4.	1.	1.	.02	11.82	6.80
ROUTED TO	R5-12	17.	6.97	4.	1.	1.	.02		
HYDROGRAPH AT	SUB6	158.	6.57	31.	9.	8.	.13		
ROUTED TO	ROUTE	156.	6.60	31.	9.	8.	.13	10.02	6.60
ROUTED TO	R6-12	156.	6.70	31.	9.	8.	.13		
HYDROGRAPH AT	SUB10	309.	7.50	124.	38.	34.	.55		
ROUTED TO	R10-12	308.	7.60	124.	38.	34.	.55		
HYDROGRAPH AT	SUB11	668.	6.50	123.	37.	33.	.55		
ROUTED TO	R11-12	667.	6.57	123.	37.	33.	.55		
HYDROGRAPH AT	SUB12	1285.	6.70	292.	88.	79.	1.25		
8 COMBINED AT	COMBINE	3197.	6.57	768.	232.	209.	3.41		
ROUTED TO	R12-13	3197.	6.57	768.	232.	209.	3.41		
HYDROGRAPH AT	SUB13	565.	6.40	90.	27.	24.	.38		
2 COMBINED AT	CB12&13	3669.	6.53	856.	259.	233.	3.79		

ROUTED TO	R13-14	3664.	6.60	853.	259.	233.	3.79		
HYDROGRAPH AT	SUB1	669.	6.30	95.	28.	26.	.40		
ROUTED TO	ROUTE	668.	6.30	92.	28.	25.	.40	15.00	6.30
ROUTED TO	R1-14	641.	6.43	91.	28.	25.	.40		
HYDROGRAPH AT	SUB2	73.	6.33	11.	3.	3.	.05		
ROUTED TO	ROUTE	9.	7.17	7.	3.	3.	.05	10.73	7.17
ROUTED TO	R2-14	9.	8.00	7.	3.	3.	.05		
HYDROGRAPH AT	SUB14	916.	6.70	208.	62.	56.	.88		
4 COMBINED AT	CB14&13(	5094.	6.57	1156.	352.	317.	5.12		

APPENDIX C

```
          LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

                        *DIAGRAM
*** FREE ***
             1          ID   CUDIA CITY WASH
             2          ID   RUNOFF ANALYSIS USING THE 100YR 24HR, TYPE 2A STORM
             3          IT       6  1JAN87    1200     250
             4          IO       5

             5          KK    SUB8
             6          IN      30
             7          PB    4.04
             8          PC     .00     .02     .03     .05     .07     .09     .11     .13     .14     .16
             9          PC     .18     .19     .21     .23     .25     .27     .29     .30     .32     .38
            10          PC     .43     .48     .57     .69    2.02    3.35    3.47    3.56    3.61    3.66
            11          PC    3.72    3.73    3.75    3.77    3.79    3.81    3.83    3.84    3.86    3.88
            12          PC    3.89    3.91    3.93    3.95    3.97    3.99    4.00    4.02    4.04    4.04
            13          BA    0.19
            14          LS       0    87.2
            15          UD    0.29

            16          KK   ROUTE
            17          RS       1    STOR       0
            18          SV       0    2.05
            19          SE       1      10
            20          SQ       0     115
            21          ST      10      95   0.611     1.5

            22          KK    R8-9
            23          RK    4700   0.016   0.030            TRAP       5       2

            24          KK    SUB7
            25          BA     .04
            26          LS       0      86
            27          UD    0.57

            28          KK   ROUTE
            29          RS       1    STOR       0
            30          SV       0    0.78
            31          SE       1      10
            32          SQ       0      28

            33          KK    R7-9
            34          RK    2482   0.018   0.030            TRAP       5       2

            35          KK    SUB9
            36          BA    0.41
            37          LS       0    85.4
            38          UD    0.37

            39          KKCB9(7&8)
            40          HC       3
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9.......10

  41          KK   ROUTE
  42          RK    2500   0.008   0.030            TRAP       5       2

  43          KK    SUB3
  44          BA    0.02
  45          LS       0      86
  46          UD    0.33

  47          KK   R3-12
  48          RK    3696   0.016   0.030            TRAP      10       2

  49          KK    SUB4
  50          BA    0.25
  51          LS       0    87.8
  52          UD    0.32

  53          KK   R4-12
  54          RK    3590   0.014   0.030            TRAP      10       2

  55          KK    SUB5
  56          BA    0.02
  57          LS       0      86
  58          UD    0.52

  59          KK   ROUTE
  60          RS       1    STOR       0
  61          SV       0    0.42
  62          SE       1      10
  63          SQ       0      14

  64          KK   R5-12
  65          RK    3590   0.014   0.030            TRAP       5       2

  66          KK    SUB6
  67          BA    0.13
  68          LS       0      89
  69          UD    0.56

  70          KK   ROUTE
  71          RS       1    STOR       0
  72          SV       0    0.69
  73          SE       1      10
  74          SQ       0     156

  75          KK   R6-12
  76          RK    3590   0.009   0.030            TRAP      15       2

  77          KK   SUB10
  78          BA    0.55
  79          LS       0    88.4
  80          UD    1.47
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  81          KK  R10-12
  82          RK    5016   0.008   0.030            TRAP      15       2

  83          KK   SUB11
  84          BA    0.55
  85          LS       0    87.5
  86          UD    0.51

  87          KK  R11-12
  88          RK    3696   0.009   0.030            TRAP      15       2

  89          KK   SUB12
  90          BA    1.25
  91          LS       0    88.7
  92          UD    0.70

  93          KK COMBINE
  94          HC       8

  95          KK  R12-13
  96          RK    2006   0.007   0.030            TRAP      20       2

  97          KK   SUB13
  98          BA    0.38
  99          LS       0    88.8
 100          UD    0.39

 101          KK CB12&13
 102          HC       2

 103          KK  R13-14
 104          RK    5597   0.009   0.030            TRAP      25       2

 105          KK    SUB1
 106          BA    0.40
 107          LS       0      89
 108          UD    0.30

 109          KK   ROUTE
 110          RS       1    STOR       0
 111          SV       0    2.05
 112          SE       1      10
 113          SQ       0      13
 114          ST      10      95   0.611     1.5

 115          KK   R1-14
 116          RK    7498   0.020   0.030            TRAP      25       2

 117          KK    SUB2
 118          BA    0.05
 119          LS       0      86
 120          UD    0.32
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 121          KK   ROUTE
 122          RS       1    STOR       0
 123          SV       0    3.28
 124          SE       1      10
 125          SQ       0       8

 126          KK   R2-14
 127          RK    7603   0.016   0.030            TRAP      25       2

 128          KK   SUB14
 129          BA    0.88
 130          LS       0      89
 131          UD    0.70

 132          KKCB14&13(3,4,5,11,6,7,8,9,10 & 12)
 133          HC       4
 134          ZZ
```

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*             FEBRUARY 1981            *
*           REVISED 31 JAN 85          *
*                                      *
*  RUN DATE 3/12/1987   TIME12:22:11 * 
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*  THE HYDROLOGIC ENGINEERING CENTER   *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*  (916) 440-3285 OR (FTS) 448-3285    *
*                                      *
****************************************
```

```
                         CUDIA CITY WASH
                         RUNOFF ANALYSIS USING THE 100YR 24HR, TYPE 2A STORM

  4 IO          OUTPUT CONTROL VARIABLES
                      IPRNT           5  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE

    IT          HYDROGRAPH TIME DATA
                       NMIN           6  MINUTES IN COMPUTATION INTERVAL
                      IDATE      1JAN87  STARTING DATE
                      ITIME        1200  STARTING TIME
                         NQ         250  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE      2JAN87  ENDING DATE
                     NDTIME        1254  ENDING TIME

                 COMPUTATION INTERVAL     .10 HOURS
                      TOTAL TIME BASE   24.90 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

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG
```

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	SUB3	260.	12.50	49.	14.	13.	.19		
ROUTED TO	ROUTE	260.	12.60	49.	14.	13.	.19	11.66	12.60
ROUTED TO	R8-9	258.	12.60	49.	14.	13.	.19		
HYDROGRAPH AT	SUB7	43.	12.80	10.	3.	3.	.04		
ROUTED TO	ROUTE	37.	13.00	10.	3.	3.	.04	12.89	13.00
ROUTED TO	R7-9	37.	13.10	10.	3.	3.	.04		
HYDROGRAPH AT	SUB9	510.	12.60	100.	28.	27.	.41		
3 COMBINED AT	CB9(7&8)	792.	12.60	160.	44.	43.	.64		
ROUTED TO	ROUTE	783.	12.70	160.	44.	43.	.64		
HYDROGRAPH AT	SUB3	26.	12.60	5.	1.	1.	.02		
ROUTED TO	R3-12	26.	12.80	5.	1.	1.	.02		
HYDROGRAPH AT	SUB4	341.	12.60	66.	18.	18.	.25		
ROUTED TO	R4-12	339.	12.60	67.	19.	18.	.25		
HYDROGRAPH AT	SUB5	23.	12.70	5.	1.	1.	.02		
ROUTED TO	RCUTE	19.	13.00	5.	1.	1.	.02	13.03	13.00
ROUTED TO	R5-12	19.	13.10	5.	1.	1.	.02		
HYDROGRAPH AT	SUB6	156.	12.70	36.	10.	10.	.13		
ROUTED TO	ROUTE	155.	12.80	36.	10.	10.	.13	9.94	12.80
ROUTED TO	R6-12	153.	12.90	36.	10.	10.	.13		
HYDROGRAPH AT	SUB10	361.	13.60	147.	41.	39.	.55		
ROUTED TO	R10-12	360.	13.70	147.	41.	39.	.55		
HYDROGRAPH AT	SUB11	657.	12.70	145.	40.	39.	.55		
ROUTED TO	R11-12	654.	12.80	145.	40.	39.	.55		
HYDROGRAPH AT	SUB12	1341.	12.80	342.	95.	92.	1.25		
8 COMBINED AT	COMBINE	3388.	12.80	905.	252.	242.	3.41		
ROUTED TO	R12-13	3381.	12.80	905.	251.	242.	3.41		
HYDROGRAPH AT	SUB13	511.	12.60	104.	29.	28.	.38		
2 COMBINED AT	CB12&13	3819.	12.80	1009.	280.	270.	3.79		

ROUTED TO	R13-14	3813.	12.80	1008.	279.	269.	3.79		
HYDROGRAPH AT	SUB1	568.	12.50	111.	31.	30.	.40		
ROUTED TO	ROUTE	568.	12.60	109.	30.	29.	.40	14.47	12.60
ROUTED TO	R1-14	560.	12.60	108.	30.	29.	.40		
HYDROGRAPH AT	SUB2	65.	12.60	13.	3.	3.	.05		
ROUTED TO	ROUTE	11.	13.30	8.	3.	3.	.05	13.42	13.30
ROUTED TO	R2-14	11.	14.00	8.	3.	3.	.05		
HYDROGRAPH AT	SUB14	953.	12.80	243.	68.	65.	.88		
4 COMBINED AT	CB14&13(	5271.	12.80	1365.	380.	366.	5.12		

APPENDIX D

```
          LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

*** FREE ***
                        *DIAGRAM
            1           ID   CUDIA CITY WASH
            2           ID   RUNOFF ANALYSIS USING THE 100YR 24HR STORM
            3           IT       2  1JAN87    1200     800
            4           IO       5

            5           KK    SUB8
            6           BA   0.169
            7           IN      30
            8           PB    4.04
            9           PC     .00     .02     .03     .05     .07     .09     .11     .13     .14     .16
           10           PC     .18     .19     .21     .23     .25     .27     .29     .30     .32     .38
           11           PC     .43     .48     .57     .69    2.02    3.35    3.47    3.56    3.61    3.66
           12           PC    3.72    3.73    3.75    3.77    3.79    3.81    3.83    3.84    3.86    3.88
           13           PC    3.89    3.91    3.93    3.95    3.97    3.99    4.00    4.02    4.04    4.04
           14           LS       0      98               0      81       0
           15           UK      75    .005    .011      35
           16           UK     200    .005     .10      65
           17           RK     300     .05    .020     .06    TRAP       5       2
           18           RK    4200     .05    .035            TRAP      15       2      NO

           19           KK   ROUTE
           20           RS       1    STOR       0
           21           SV       0    2.05
           22           SE       1      10
           23           SQ       0     115
           24           ST      10      95   0.611     1.5

           25           KK    R8-9
           26           RK    4699   0.016   0.030            TRAP      15       2

           27           KK    SUB7
           28           BA    .043
           29           LS       0      98               0      80       0
           30           UK     100    .005    .011      35
           31           UK     300    .005      .1      65
           32           RK     400     .04    .020    .013    TRAP       5       2
           33           RK    2000     .04    .035            TRAP      15       2      NO

           34           KK   ROUTE
           35           RS       1    STOR       0
           36           SV       0    0.78
           37           SE       1      10
           38           SQ       0      28

           39           KK    R7-9
           40           RK    2482   0.018   0.030            TRAP       5       2

           41           KK    SUB9
           42           BA   0.384
           43           LS       0      79
           44           UK     200    .005     .10     100
           45           RK     400     .04    .020     .14    TRAP       5       2
           46           RK    5200     .04    .035            TRAP      15       2      NO
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  47          KK CB7&8&9
  48          HC       3

  49          KK    ROUTE
  50          RK     2500   0.008    0.030              TRAP       5       2

  51          KK     SUB3
  52          BA    0.025
  53          LS        0      98                  0     80
  54          UK      100    .005     .011        35
  55          UK       75    .005      .1         65
  56          RK      200     .08     .020      .006   TRAP       5       2
  57          RK     1000     .05     .035            TRAP      15       2      NO

  58          KK    R3-12
  59          RK     3696   0.016    0.030              TRAP      10       2

  60          KK     SUB4
  61          BA    0.213
  62          LS        0      98                  0     82       0
  63          UK       75    .005     .011        35
  64          UK      200    .005      .10        65
  65          RK     2300     .05     .020      .08   TRAP       5       2
  66          RK     3800     .03     .035            TRAP      15       2      NO

  67          KK    R4-12
  68          RK     3590   0.014    0.030              TRAP      10       2

  69          KK     SUB5
  70          BA    0.028
  71          LS        0      98                  0     80
  72          UK       75    .005     .011        35
  73          UK      100    .005      .10        65
  74          RK      200     .04     .020      .006   TRAP       5       2
  75          RK      800     .07     .035            TRAP      15       2      NO

  76          KK    ROUTE
  77          RS        1    STOR        0
  78          SV        0    0.42
  79          SE        1      10
  80          SQ        0      14

  81          KK    R5-12
  82          RK     3590   0.014    0.030              TRAP       5       2

  83          KK     SUB6
  84          BA    0.123
  85          LS        0      98                  0     84
  86          UK       75    .005     .011        35
  87          UK      300    .005      .10        65
  88          RK      400     .05     .020      .04   TRAP       5       2
  89          RK     3200     .03     .035            TRAP      15       2      NO
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  90          KK   ROUTE
  91          RS       1    STOR       0
  92          SV       0    0.69
  93          SE       1      10
  94          SQ       0     156

  95          KK   R6-12
  96          RK    3590   0.009   0.030            TRAP      15       2

  97          KK   SUB10
  98          BA   0.495
  99          LS       0      98              0      83       0
 100          UK      75    .005    .011     35
 101          UK     200    .005     .10     65
 102          RK    2800    .050     .02    .18    TRAP       5       2
 103          RK    5880    .024    .035            TRAP      15       2      NO

 104          KK  R10-12
 105          RK    5016   0.008   0.030            TRAP      15       2

 106          KK   SUB11
 107          BA   0.625
 108          LS       0    87.5
 109          UD   0.212

 110          KK  R11-12
 111          RK    3696   0.009   0.030            TRAP      15       2

 112          KK   SUB12
 113          BA    1.30
 114          LS       0      98              0      84       0
 115          UK      75    .005    .011     35
 116          UK     200    .005     .10     65
 117          RK    2725    .035    .020    .41    TRAP      10       2
 118          RK    4800    .007    .035            TRAP      25       2      NO

 119          KK COMBINE
 120          HC       8

 121          KK  R12-13
 122          RK    2006   0.007   0.030            TRAP      20       2

 123          KK   SUB13
 124          BA   0.378
 125          LS       0      98              0      82       0
 126          UK      75    .005    .011     35
 127          UK     200    .005     .10     65
 128          RK    4300    .106    .020    .13    TRAP      10       2
 129          RK    4600     .05    .035            TRAP      35       2      NO
```

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 130          KK CB12&13
 131          HC        2

 132          KK   R13-14
 133          RK     5597    0.009    0.030             TRAP      25       2

 134          KK     SUB1
 135          BA     0.37
 136          LS        0       89
 137          UD     0.47

 138          KK    ROUTE
 139          RS        1     STOR        0
 140          SV        0     2.05
 141          SE        1       10
 142          SQ        0       13
 143          ST       10       95    0.611      1.5

 144          KK   R1-14
 145          RK     7498    0.020    0.030             TRAP      25       2

 146          KK     SUB2
 147          BA    0.054
 148          LS        0       98                 0       80
 149          UK       75     .005     .011       35
 150          UK      200     .005      .10       65
 151          RK      300       .1      .02      .02    TRAP       5       2
 152          RK     1200      .05     .035             TRAP      10       2      NO

 153          KK    ROUTE
 154          RS        1     STOR        0
 155          SV        0     3.28
 156          SE        1       10
 157          SQ        0        8

 158          KK   R2-14
 159          RK     7603    0.016    0.030             TRAP      25       2

 160          KK    SUB14
 161          BA    0.707
 162          LS        0       98                 0       84
 163          UK       75     .005     .011       35
 164          UK      200     .005      .10       65
 165          RK     5600      .02     .020      .29    TRAP       5       2
 166          RK     5400     .008     .035             TRAP      30       2      NO

 167          KK     CB12,13&14
 168          HC        4
 169          ZZ
```

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	SUB8	236.	12.50	41.	12.	11.	.17		
ROUTED TO	ROUTE	234.	12.50	41.	12.	11.	.17	11.44	12.50
ROUTED TO	R8-9	232.	12.57	41.	12.	11.	.17		
HYDROGRAPH AT	SUB7	59.	12.50	10.	3.	3.	.04		
ROUTED TO	ROUTE	48.	12.57	10.	3.	3.	.04	16.50	12.57
ROUTED TO	R7-9	48.	12.63	10.	3.	3.	.04		
HYDROGRAPH AT	SUB9	455.	12.53	70.	20.	18.	.38		
3 COMBINED AT	CB7&8&9	732.	12.53	122.	35.	31.	.60		
ROUTED TO	ROUTE	726.	12.60	122.	35.	31.	.60		
HYDROGRAPH AT	SUB3	36.	12.50	6.	2.	2.	.02		
ROUTED TO	R3-12	36.	12.60	6.	2.	2.	.02		
HYDROGRAPH AT	SUB4	301.	12.50	53.	15.	14.	.21		
ROUTED TO	R4-12	300.	12.53	53.	15.	14.	.21		
HYDROGRAPH AT	SUB5	40.	12.50	7.	2.	2.	.03		
ROUTED TO	ROUTE	34.	12.53	7.	2.	2.	.03	22.60	12.53
ROUTED TO	R5-12	33.	12.67	7.	2.	2.	.03		
HYDROGRAPH AT	SUB6	178.	12.50	32.	9.	8.	.12		
ROUTED TO	ROUTE	176.	12.53	32.	9.	8.	.12	11.14	12.53
ROUTED TO	R6-12	176.	12.60	32.	9.	8.	.12		
HYDROGRAPH AT	SUB10	707.	12.50	126.	36.	32.	.50		
ROUTED TO	R10-12	699.	12.53	126.	36.	32.	.50		
HYDROGRAPH AT	SUB11	911.	12.53	164.	46.	41.	.63		
ROUTED TO	R11-12	911.	12.57	164.	46.	41.	.63		
HYDROGRAPH AT	SUB12	1886.	12.50	338.	96.	87.	1.30		
8 COMBINED AT	COMBINE	4714.	12.50	847.	241.	217.	3.40		
ROUTED TO	R12-13	4714.	12.53	848.	241.	217.	3.40		
HYDROGRAPH AT	SUB13	534.	12.50	94.	27.	24.	.38		
2 COMBINED AT	CB12&13	5239.	12.53	942.	268.	241.	3.78		

ROUTED TO	R13-14	5234.	12.60	947.	269.	242.	3.78		
HYDROGRAPH AT	SUB1	484.	12.67	102.	28.	26.	.37		
ROUTED TO	ROUTE	484.	12.70	100.	28.	25.	.37	14.00	12.70
ROUTED TO	R1-14	476.	12.80	100.	28.	25.	.37		
HYDROGRAPH AT	SUB2	77.	12.50	13.	4.	3.	.05		
ROUTED TO	ROUTE	12.	12.90	9.	4.	3.	.05	14.51	12.90
ROUTED TO	R2-14	12.	13.63	9.	3.	3.	.05		
HYDROGRAPH AT	SUB14	985.	12.50	179.	51.	46.	.71		
4 COMBINED AT	CB1	6570.	12.57	1232.	351.	317.	4.91		

DRJ

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P.O. BOX 2711
LOS ANGELES, CALIFORNIA 90053-2325

FLOOD CONTROL DISTRICT
RECEIVED
MAR 1 7

	CH ENG	Z	PP & PM
			HYDRO
	ADMIN		LMGT
	FINANCE		FILL
	C & O		
	ENGR		

REMARKS Copy to: DES, DRJ, TES, SLS

REPLY TO
ATTENTION OF:

March 6, 1987

Office of the Chief
Hydrology and Hydraulics Branch

Mr. W. Scudder Gookin
W.S. Gookin Associates
4203 N. Brown Avenue
Scottsdale, Arizona 85251

Dear Mr. Gookin:

Thank you for the opportunity to review and comment on your report "Cudia City Wash Runoff Analysis" dated October 1986.

During our review, we found the following items worthy of noting:

a. The study was performed using the generalized SCS procedure to generate subarea hydrographs. The procedure was not calibrated to the Phoenix area. In the study it was assumed that the Type IIA storm distribution, used with NOAA 100-year, 24-hour rainfall, would yield 100-year peak discharges. There is no assurance that this is true unless the rainfall-runoff model is calibrated to discharge frequency relationships developed from streamflow data.

b. Information given in the report shows that using one subarea, without explicitly considering storage effects of flow restrictions, gives results that are only slightly different (less than 4 percent smaller) than results for 14 subareas, with or without considering storage effects of flow restrictions.

c. The report implies that the Corps of Engineers "Report on Flood of 22 June 1972" stated that the June 22, 1972 flood approximated a 100-year flood. We could find no such statement in that report or in any other Corps of Engineers report. Furthermore, Appendix 6 of "Design Memorandum No. 3, General Design Memorandum Phase I", dated March 1976, uses a peak discharge estimate for Cudia City Wash at the Arizona Canal of 4,000 cfs for the June 22, 1972 flood, stating that this discharge approximates a 50-year event. Your report calls the June 22, 1972 storm a 100-year storm. To our knowledge, the June 1972 storm has never been analyzed for depth-duration-frequency.

d. The report states that the "maximum discharge during the June 22, 1972 storm in the Arizona Canal was computed by SRP to be 3,375 cfs". The report assumes that 3,375 cfs is a 100-year peak discharge and then concludes that the study results are conservative. This totally ignores the USGS estimate of 4,200 cfs for a point on Cudia City Wash 1,000 feet upstream of McDonald Drive given in the Corps of Engineers 1972 flood report. The USGS estimate was for a drainage area of 2.16 square miles; the total Cudia City Wash area is about 5 square miles. The Arizona Canal at Cudia City Wash cannot hold 3,375 cfs;

thus, the 3,375 cfs estimate cannot be in the Arizona Canal. According to SRP staff, 3,375 cfs is an estimate of the flow from Cudia City Wash reaching the Arizona Canal, based on inflow-outflow-storage calculations for the Arizona Canal. Inflow was estimated from a gage in the Canal at 56 Street; outflow was estimated from a gage in the Canal at 32nd Street, a gage in Old Cross-Cut Canal, and gages for the overflow spillways downstream from 40th Street and upstream of 32nd Street; and storage was computed from the Canal configuration between 56th Street and 32nd Street.

From the above findings, we conclude that the study procedures, results, and conclusions are not sufficiently compelling to require us to change our methods or design flow rates for Cudia City Wash and the ACDC. One subarea for Cudia City Wash appears to be adequate, and the detailed effort required to break the watershed into 14 subareas unnecessary. The attempt to confirm the study results using the SRP estimate is unconvincing because there is no basis for stating that 3,375 cfs is a 100-year peak discharge at the mouth of Cudia City Wash.

We thank you again for the opportunity to review and comment on your report. More detailed observations are contained in the enclosed paper.

Sincerely,

Norman Arno
Chief, Engineering Division

Enclosure

Mr. Daniel Sagramoso
Chief Engineer and General Manager
Flood Control District of Maricopa County
Phoenix, Arizona 85009

CUDIA CITY WASH RUNOFF ANALYSIS

W.S. GOOKIN AND ASSOCIATES

October 1986

REVIEW COMMENTS BY CORPS OF ENGINEERS

February 27, 1987

The following comments on the report "Cudia City Wash Runoff Analysis" by W.S. Gookin and Associates dated October 1986 (hereinafter called the Gookin report or study) accompany a letter from the Corps of Engineers to Mr. W. Scudder Gookin dated March 6, 1987.

1. Although the explanation of the determination of parameter values used in the study was sometimes unclear or lacking and there was insufficient data given for an independent evaluation of the parameter values, we assume that the SCS procedure was applied correctly. We would mention, however, that the results using the SCS procedure are very sensitive to the time of concentration (tc) parameter. A relatively small change in tc can have a substantial impact on the computed peak discharge. Also, the increment of rainfall used is important. Even if a 5- or 10-minute computation interval is used, if the smallest increment of rainfall used is 30 minutes (i.e., the same intensity for three 10-minute computation intervals), the computed peak discharge may be much smaller than if 10-minute increments of rainfall were used. There is insufficient information in the report to evaluate this.

Encl

2. The SCS Method is an uncalibrated rainfall-runoff model. It is assumed that when using the SCS Type IIA rainfall distribution, with NOAA 100-year, 24-hour rainfall, along with the SCS dimensionless unit hydrograph and the SCS loss function, that 100-year peak discharges and hydrographs are produced. Thus, runoff frequency is assumed equivalent to rainfall frequency. There is no assurance that this is true unless the rainfall-runoff model is calibrated to discharge frequency relationships developed from streamflow data. In the Gookin study, an attempt was made to show that the SCS procedure did produce a 100-year peak discharge for Cudia City Wash by comparing the results with an SRP estimate of 3375 cfs for the June 22, 1972 flood peak discharge for the total Cudia City Wash watershed. However, the Corps believes the assumption that the SRP estimate is a 100-year peak discharge is unfounded. The Gookin study implies that the Corps of Engineers "Report on Flood of 22 June 1972" stated that the June 22, 1972 flood approximated a 100-year flood. We could find no such statement in that report or any other Corps of Engineers report. Furthermore, Appendix 6 of "Design Memorandum No. 3, General Design Memorandum Phase I", dated March 1976, uses a peak discharge estimate for Cudia City Wash at the Arizona Canal of 4000 cfs for the June 22, 1972 flood, stating that this discharge approximates a 50-year event. Moreover, there is uncertainty as to the actual magnitude of the peak discharge for the total watershed in view of the USGS peak discharge estimate of 4200 cfs for a point on Cudia City Wash 1000 feet upstream of McDonald Drive, less than half of the total watershed.

3. The volume difference between the Corps hydrograph for Cudia City Wash and the Gookin report hydrograph is 20 percent, 580 AF versus 698 AF. Because slightly different drainage area sizes were used, the difference in inches of runoff is only 15 percent, 2.22 inches for the Corps hydrograph and 2.57 inches

for the Gookin study hydrograph. It is obvious from the figure called "Comparison of Cudia City Hydrographs and Analysis of Peak Retention Effects" in the Gookin report that it is not the volume itself, but the distribution of the volume in the hydrograph, along with the detention basin storage and outflow capability that are important to the design of a detention basin. Given the detention basin capacity of 131 AF, the Corps' smaller volume comes into the basin quicker, filling the basin quicker and resulting in a higher flow downstream of the basin. The distribution of the volume in the hydrograph is greatly influenced by the distribution of rainfall used to compute the hydrograph. According to SCS Phoenix office staff, 59 percent of the 24-hour rainfall of the SCS Type IIA rainfall distribution, which was used in the Gookin study, occurs in the maximum hour of the storm. Using NOAA 100-year, 24-hour rain of 3.8 inches, which was used in the Gookin study, 2.24 inches falls in 1 hour. The 7-hour storm on which the Corps' hydrograph is based has about 80 percent of the 7-hour rainfall occurring in the maximum hour. If NOAA 100-year, 6-hour rainfall (3.0 inches) is used to approximate the 100-year, 7-hour rainfall (say 3.1 inches), about 2.48 inches of rainfall occurs in the maximum hour of the Corps' storm, about 11 percent more than the SCS Type IIA. It is typical for thunderstorms in Central Arizona to have all or nearly all precipitation occur in 6 hours, with a very large percentage occurring in 1 hour. This characteristic is observed in the June 22, 1972 storm, where 77 percent of the 6-hour rain fell in the maximum hour at rain gage number 13 given in the Corps' report on the 1972 flood. The August 28, 1986 flood event exhibited the same characteristics in that most of the rain fell in a 2-3 hour period, according to the Maricopa County Flood Control District's "Report of Flooding Near 32nd Street and the Arizona Canal, August 28-29, 1986." The probable reason why the 1986 flood caused less damage than the 1972 flood was

the difference in areal extent of the two storms. In the 1986 event, rainfall within 2 miles of the center of the storm, 40th Street and Montebello Avenue, had fallen off from 5.53 inches to 1.1-2.2 inches. The storm isohyets given in the Gookin report show high rainfall for a much larger areal extent.

With most of the rainfall occurring in such a short time, the hydrograph tends to have a "spiked" shape. Inspecting annual peaks versus 1-day volume for two available streamgages in small watersheds, "Agua Fria Tributary at Youngtown" (USGS No. 9-5137, drainage area 0.13 sq. mi.) and "Salt River Tributary in South Mountain Park" (USGS No. 9-5122, drainage area 1.75 sq. mi.) it was found that for events which produce the annual maximum peak discharge, the average 1-day flow rate was less than 3 percent of the peak discharge. (The 1-day volume is equal to the average 1-day flow rate multiplied by the 1-day duration.) This indicates that on small drainage areas, the flood events that produce maximum peak discharges are of very short duration. The design storm used to shape the design flood hydrographs for the ACDC is based on the August 1954 Queen Creek local storm. This design storm has a 7-hour duration with most of the rain occurring in the maximum 3 hours. The maximum 1-hour amount for a small areal extent is about 80 percent of the total storm amount. The ACDC hydrographs shaped with this storm have an average 1-day flow rate of about 7 percent of the peak, as compared with less than 3 percent for the streamgage records mentioned above. Volume-frequency analysis of the streamgage records for large watersheds, such as Indian Bend Wash and New River near Phoenix, produce much higher ratios of average 1-day flow rate to peak for high peak discharges (on the order of 15 percent), indicating the volume is distributed over a longer period of time and there is relatively more volume than in the events causing high peaks in small watersheds. Thus, we conclude

that the relationship between peak discharge and volume in the ACDC design hydrographs is adequate; the design storm properly shapes the hydrograph, i.e. appropriately distributes the volume within the hydrograph.

In summary, our judgment is that the regional discharge frequency relationships developed for Part 1 of DM No. 2, dated 1974, give appropriate estimates of peak discharge and the storm used to determine the amount and distribution of volume in the hydrograph is adequate for detention basin design on Cudia City Wash and the ACDC. The regional relationships were developed in 1973, fourteen years ago, but were re-evaluated using data through 1981 while preparing Part 2 of DM No. 2, which is dated 1982, and were found to still be valid. The peak discharge of 6700 cfs is approximately the same (within 5-10 percent) as that determined by at least two other consultants in recent years, one of the consultants being W. Scudder Gookin.

Review of WSG Report
March 3, 1987

Flood Coontrol District of Maricopa County
Watershed Hydrology Branch

Introduction

W.S. Gookin & Associates, Consulting Engineers (WSG), on behalf of the Citizens Against Reach Four, completed a study in October, 1986 entitled "Cudia City Wash Runoff Analysis". The purpose of this study was to determine the peak discharge on Cudia City Wash at the Arizona Canal resulting from a 100-year storm event and evaluate the effect of a detention basin on the outflow into the proposed Arizona Canal Diversion Channel. The report was submitted for review and comments to the the Watershed Hydrology branch of the Flood Control District of Maricopa County (FCD). The following describes the findings of that review.

The Cudia City Wash watershed is located in the northeast section of the City of Phoenix, and is generally bounded by Camelback Mountain, Mummy Mountain, and the Phoenix Mountains. The watershed is characterized by mild to steep slopes and sparse to moderate vegetation. Historically, this area has been subject to intense thunderstorms that have caused damaging floods.

WSG Model

WSG analyzed the runoff resulting from the 100-year 24-hour storm centered over the Cudia City watershed. The storm used for this analysis had a total depth of 3.80 inches and a Soil Conservation Service (SCS) Type IIA temporal distribution. The rationale given by WSG for the use of this design storm was that "this is the standard storm distribution agreed upon...for Scottsdale." However, no part of this study area is within the City of Scottsdale. Rather, the entire area is contained within the political boundaries of the City of Phoenix and the Village of Paradise Valley. The 100-year 24-hour design rainfall depth used by the engineers for these municipalities is 4.04 inches with a time distribution developed by the City of Phoenix.

FCD Model Simulating WSG Model

In order to evaluate the WSG modeling strategy, a computer model was created by FCD using HEC-1 and the WSG modeling parameters. These parameters include: storm depth of 3.80 inches, Type IIA storm distribution, curve numbers, lag times, and channel lengths estimated by WSG.

Using the SCS hydrograph method, combined with the kinematic wave channel routing technique, the FCD simulated hydrographs for each subbasin. The FCD estimates compare reasonably well with those given in the WSG report, although low by 13 to 30 percent. The final routed and combined FCD hydrograph estimates the discharge at the Arizona Canal and matches WSG's estimate of the peak discharge to within 13%. However, at this point the FCD estimate of the peak discharge is higher than that given by WSG. Discharges for the individual subbasins are listed in Table 1.

Table 1. Comparison of subbasin discharges between the FCD and the WSG models.

Subbasin	Q(FCD model)	Q(WSG model)	% Greater(WSG v FCD)
1	669	760	13
2	73	86	15
3	29	34	15
4	390	455	14
5	23	27	15
6	158	195	19
7	42	58	28
8	300	345	13
9	545	780	30
10	309	415	25
11	668	800	16
12		*	
13		*	
14		*	
Total Routed Q	5094	4422	

* not calculated individually for these subbasins

The change from low subbasin estimates to a high combined estimate suggests that differences in the routing techniques may be the cause. The FCD modeling scheme used an accepted, technically sound routing method (kinematic wave) in its HEC-1 model. The WSG model used the Lotus Spreadsheet to combine and route the subbasin hydrographs. It is suspected that differences in the routing techniques are the reason the FCD combined peak discharges at the Arizona Canal are higher in comparison. If WSG had used the kinematic wave routing technique, their combined peak discharge would probably have been greater than the FCD estimate. However, in order to evaluate different modeling schemes, the two models were assumed to be reasonably similar.

It was noted on the output from the FCD version of the WSG modeling scheme that a number of warnings were given on the length of the computation interval relative to the lag time which was input. If the computation interval is not acceptably small compared to the lag time, there is a good possibility that peak discharges could be miscalculated. The peak discharge at the Arizona Canal from the FCD version of the WSG model, using a 6 minute computation interval, was 4870 cfs. This value rose to 5115 cfs when the computation interval was reduced to 2 minutes. These values are to be compared with a peak discharge of 4422 cfs in the WSG report.

Sensitivity of FCD Simulation Model to Storm Depth and Distribution

Assuming that WSG correctly modeled the Cudia City Wash area, FCD then tried to determine what the effect of using a different design storm would have on the peak discharge. The City of Phoenix design storm and distribution, as described earlier, were used as input to the model. The result is a peak discharge of 5402 cfs at the Arizona Canal.

Independent FCD Model

The SCS Method has long been considered a reasonable means for generating storm runoff. However, this method was developed for natural catchments and its appropriateness for estimating the discharge from urban areas of this type is questionable.

Kinematic wave modeling is a physically-based, conceptual approach for generating runoff from areas where the primary flow regime is sheetflow. The input parameters are easily measured from topographic maps. Urbanized areas are particularly amenable to this type of modeling scheme.

FCD hydrology staff considered a combination of the kinematic wave and the SCS methods to be the most valid technique for generating runoff from this study area. The staff used HEC-1 to develop an independent, unbiased model for this area. The kinematic wave technique was used on the urbanized subbasins having relatively, less steep slopes. The SCS method was used on the two subbasins having steep slopes over a significant portion of the total area.

After measuring the model's physical input parameters numerous inconsistencies were evident between FCD estimates and the estimates WSG used in their SCS model. Inconsistencies were found with subbasin areas, channel slopes, and times of concentration. These discrepancies deserve more explanation and will be addressed later in this report.

It should be noted that because of the discrepancies in parameter measurements, the 100-year 24-hour storm depth, and the storm distribution, the FCD combination model represents the Hydrology section's best attempt to model the 100-year 24-hour storm runoff from the Cudia City watershed. The FCD model includes revised estimates of subbasin areas, channel lengths, channel slopes, and times of concentration. The loss rate function, input to the kinematic wave portion of the model, uses curve numbers to represent pervious and impervious areas of each subbasin. The average of each pair of subbasin curve numbers equals the curve numbers determined by WSG. The combination model uses 4.04 inches as the 100-year 24-hour storm depth and the City of Phoenix storm distribution. The estimated peak discharge for the combination kinematic wave/SCS model is 6520 cfs.

Model Calibration

Attempts were made to calibrate both of the FCD models using a historic storm that occured on June 22, 1972. A peak discharge was estimated during this

storm at a point near the outlet of the Cudia City watershed. However, insufficient precipitation measurements, necessary for accurate isohyetal mapping, have precluded the use of this storm for calibration.

Comparison of Physical Parameters

The differences in the measured physical parameters, mentioned earlier, were channel lengths, channel slopes, and subbasin areas. Table 2. presents FCD's and WSG's estimates of these parameters.

Table 2. Differences between the measured physical parameters.

	Area (mi^2)		Channel length (ft)		Slope (%)	
Sub	FCD	WSG	FCD	WSG	FCD	WSG
1	0.37	0.40	6100	6706	0.043	0.147
2	0.05	0.05	1200	2376	0.050	0.143
3	0.03	0.02	1000	1056	0.050	0.076
4	0.21	0.25	3400	4330	0.059	0.138
5	0.03	0.02	800	1320	0.013	0.043
6	0.12	0.13	3200	3854	0.026	0.028
7	0.04	0.04	2000	2904	0.030	0.036
8	0.17	0.19	4200	5280	0.048	0.116
9	0.38	0.41	5200	6494	0.042	0.144
10	0.50	0.55	5800	5280	0.024	0.175
11	0.63	0.55	9800	9610	0.168	0.113
12	1.30	1.25	4800	11510	0.007	0.122
13	0.38	0.38	4600	5755	0.045	0.111
14	0.71	0.88	5400	7181	0.008	0.029

Lag time, referred to as the time of concentration by WSG, is a function of both channel length and channel slope. Thus, the lag time is also suspect and is capable of changing the results significantly.

Summary

Numerous problems with the WSG modeling effort became evident during our review. These include 1) the apparent specification of too long a computation interval, 2) the use of a design rainfall depth and distribution inappropriate to the study area, 3) the contrasting results in hydrographs for individual subbasins and for hydrographs resulting from combining and routing flows from various subbasins, 4) the apparent measurement errors of channel length, channel slope, and subbasin areas, and 5) the suspect times of concentration, or lag times. The WSG report indicated that the peak discharge from Cudia City Wash at the Arizona Canal resulting from a 100-year 24-hour storm event is 4245 cfs. Assuming that the WSG modeling efforts provide a reasonable description of the study area, FCD ran that same model using a shorter computation interval and the City of Phoenix design storm. This model indicates that the peak discharge is 5400 cfs at the same location. FCD then created an independent, unbiased computer model of the area using what we considered to be the most appropriate methodologies and design rainfall. The peak discharge from this model at the Arizona Canal is 6520 cfs.