Economic Optimization of the Agua Fria River Levees

JOHN SVECHOVSKY

Flood Control District of Maricopa County, Phoenix, Arizona 85009

1992

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As part of a comprehensive plan for flood control for the metropolitan Phoenix area, the Flood Control District of Maricopa County has been the local sponsor for the construction of dams to reduce the flood peaks of urban washes and the interception and diversion of storm drainage to the lower Agua Fria River via Skunk Creek and New River. The added burden to the Agua Fria and its tributaries necessitated the acquisition of flowage easements, channelization and levee construction to gain positive control of the floodplains. Federal participation by the Corps of Engineers mandated that many of the structural and nonstructural improvements be designed to accommodate the standard project flood. At the present time, most of the major facilities have been constructed, including the Agua Fria River levees. An approximate one-mile portion of these levees, will be examined as to their cost, relative to the property they protect. The analysis will determine the optimal level of protection based on the value of the average annual flood loss prevented.

PROBLEM STATEMENT

This report will investigate the level of protection that is most appropriate for the Agua Fria River given the local conditions of property values. Of the 10.1 miles of structural and nonstructural improvements made to the Agua Fria, this paper will focus on the 1¼ miles between Thomas Road and McDowell Road [Plate 1]. Consideration will be given to the role that the levees have, not only in protecting property, but also as part of the comprehensive plan for flood control.

BACKGROUND

As authorized by the Flood Control Act of 1965, Public Law 89-298, the U. S. Army Corps of Engineers undertook studies to examine the flood hazards of waterways in the Gila River Basin. The comprehensive flood control plan in the metropolitan Phoenix area can be summarized [Plate 2] as having five components: (1) construction of four earthfill dams to control the standard project flood (SPF) of major washes before they drain through urbanized portions of the metropolitan Phoenix area, (2) construction of approximately twenty miles of channelization, mainly for the interception and diversion of storm drainage, (3) acquisition and management of flowage easements along Skunk Creek, New River and Agua Fria River from the Arizona Canal Diversion Channel (ACDC) to the Gila River, (4) management of the floodplains from Cave Buttes and Adobe Dams to the ACDC and from New River Dam to Skunk Creek, and (5) providing recreational development at the dam sites and along channels and flowage easements.

By the mid 1980s, the construction of the four flood control dams had been completed; Dreamy Draw Dam (1973) controlling Dreamy Draw, Cave Buttes Dam (1980) controlling Cave Creek Wash, Adobe Dam (1982) controlling Skunk Creek, and New River Dam (1985) controlling New River. Pertinent physical data for the dams is shown in Table 1.

Structures like the 38-mile Arizona Canal, constructed in the 1890s, has long been a drainage barrier. It serviced agricultural land above the Salt River. The natural drainage patterns of Cave Creek Wash and Cudia City Wash have been disrupted by the Canal. As agriculture gave way to urbanization, the flood hazard associated with a potential breach of the Canal became a growing concern. The Arizona Canal intercepted drainage until its banks were overtopped. At the SPF condition, a 50,500 acre [*U. S. Army Corps of Engineers,* 1986] floodplain resulted from breaches to the Arizona Canal predominantly at Cave Creek and Cudia City Washes. Half of this area was in downtown Phoenix and the surrounding urbanized areas.

The ACDC was designed to intercept 100-year flows from 40th Street near Cudia City Wash to Skunk Creek along a path parallel to the Arizona Canal, a distance of 16.5 miles [Plate 2]. Design 100-year flows along the ACDC are 6,700 cubic feet per second (cfs) at 40th Street, 10,000 cfs at the Dreamy Draw confluence near 10th Street, 14,000 and 25,000 cfs upstream and downstream, respectively, of the Cave Creek Wash confluence, and 29,000 cfs at the confluence with Skunk Creek.

The interception and diversion of flows along the Arizona Canal Diversion Channel significantly alleviated the flooding hazard in central Phoenix, but it put a new burden on the lower Skunk Creek, New River and the Agua Fria River.

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portions of Yavapai and Maricopa Counties with a total watershed of 2340 square miles [*Willdan Associates*, 1981]. Waddell Dam, holding back Lake Pleasant, 34 miles from the confluence with the Gila, is the river's only dam. It was constructed in 1927, for water conservation and recreation. The reservoir capacity and spillway discharge were 176,000 acre-ft and 225,000 cfs, respectively. With the completion of construction of New Waddell Dam in 1992, the SPF can safely be impounded by Lake Pleasant. McMicken Dam [Plate 2] was constructed in 1956 to provide flood control for Luke Air Force Base and the west portion of metropolitan Phoenix. Pertinent hydrologic data for these dams is also listed in Table 1.

TABLE 1. Physical Hydrologic Data for Flood Control Dams Protecting Phoenix Arizona and Vicinity [*U. S. Army Corps of Engineers,* 1986].

Dam	Drainage Area (sq-mi)	Reservoir Capacity (acre-ft)	SPF Peaks* Inflow (cfs)	SPF Peaks* Outflow (cfs)
Dreamy Draw	1.3	317	3,600	220
Cave Buttes	191	46,600	54,000	486
Adobe	89.6	18,350	66,000	1,890
New River	164	43,560	45,000	2,665
New Waddell†	1,460	1,145,506	493,400	325,200
McMicken‡	322	32,800	35,000	4,450

* Based on the 6-hour duration storm.
† *Bureau of Reclamation,* 1986.
‡ Wittmann, Arizona - Area Drainage Master Study, *WLB Group, Inc.,* 1989.

As discussed in the previous section, a peak 100-year discharge of 29,000 cfs enters Skunk Creek from the ACDC. At the 100-year event, the peak inflow into New River from Skunk Creek is 35,000 cfs. The inflow from New River to the Agua Fria is 39,000 cfs. The peak 100-year flow in the Agua Fria downstream of the New River confluence is 95,000 cfs. This peak attenuates to 85,000 cfs at the Gila River [*U. S. Army Corps of Engineers,* 1986].

Inflows into the Agua Fria below New River are all local urban drainage. The major inflow is at the Interstate-10 crossing where a collector channel has been constructed to drain a 45 sq-mi urban area extending from the river east to 27th Avenue [*Willdan,* 1981]. Since this drainage area is urbanized and relatively small, the time of concentration will be short enough that the peak will enter and leave the Agua Fria well ahead of upstream flood peaks. Other minor flows enter the lower Agua Fria; however, they also have little effect on the peak.

Table 1 indicates that with the exception of the potentially large releases from New Waddell, the majority of 100-year peaks into Skunk Creek, New and Agua Fria Rivers are from intercepted urban storm drainage.

DESIGN CONCEPT FOR DIVERTED STORM WATER

The diverted flows put an unnatural hydrologic and hydraulic burden on the lower Agua Fria River. Design concepts included realignment of the main channel and taking conservative measures to consider the new hydrologic conditions of flow frequency and durations. Structural and nonstructural improvements to Skunk Creek and New and Agua Fria Rivers were designed for the standard project flood (SPF). The design concept consisted of three components, that collectively managed the floodplains; acquisition and management of SPF flowage easements, channelization that reduced the floodplain, and levees built to protect against the SPF. Below is a brief description of the alternatives chosen for the affected portions of the three waterways [*U. S. Army Corps of Engineers,* 1986]. The associated costs for the Agua Fria River levees are summarized in Table 2.

Skunk Creek Improvements to the 1.8 miles between the ACDC and New River was limited to the acquisition of flowage easements and minor grouted stone channel stabilization.

New River Improvements to the 5.6 miles of New River from Skunk Creek to the Agua Fria confluence consisted of the replacement of a railroad bridge, deepening and realignment of the channel with soil cement stabilized sides.

Agua Fria River Improvements to the 10.1 miles of the Agua Fria from its confluence with New River to the Gila consists of channelization and realignment, construction of 12 to 15-foot high levees, protecting a sewage treatment plant, urbanized areas, and electrical transmission lines.

PROCEDURE OF ANALYSIS

The process employed [*Helweg,* 1985] will be to calculate the direct flood control benefits as the flood losses averted; that is, the prevention of losses by the project is a benefit. The flood control benefits are based on the average annual flood loss (AAFL). The steps incorporated to establish the AAFL are as follows:

Step 1: Perform a *flood flow-frequency* analysis. This will relate the Agua Fria's discharge and its probability of being exceeded [Table 5];

Step 2: Determine how much damage is caused to property and a function of flood water depth. This analysis will result in a *depth-damage* curve [Figure 1];

Step 3: Construct a *rating curve* for the Agua Fria River at the vicinity of the study area. This relates the depth of flow to discharge [Figure 2];

Step 4: Combine the curves of steps 2 and 3 to determine the *discharge-damage* curve [Figure 3]; and

Step 5: Combine steps 3 and 4 to determine the *frequency-damage* relationship [Table 9].

FLOOD FLOW-FREQUENCY ANALYSIS

The recording of floods on the Agua Fria has occurred since 1889. The largest floods occurred in 1916 and 1919, both estimated at 105,000 cubic feet per second (cfs) [*Willdan Associates*, 1981]. Since the construction of

Waddell Dam in 1927, there have been five floods, measured at the outlet of the Dam; 20,000 cfs in December 1967, 20,600 cfs in August 1970, 47,000 in March 1978, 60,000 in December 1978, and 73,300 in February 1980. It was the magnitude of the floods in 1978 and 1980 and their regional severity that initiated the construction of the portion of the comprehensive plan described in the early sections of this paper. In Table 3 are shown the annual maximum discharges recorded on the Agua Fria at Avondale Arizona, from 1960 to 1982.

TABLE 2. Flowage Easement and Project Construction Costs for the Agua Fria River, Phoenix, Arizona [*Flood Control District of Maricopa County*, 1988].

FLOWAGE EASEMENTS		
River Reach Length	(miles)	10.1
Average Easement Width	(ft)	3,205
Total Easement Area	(acres)	3,925
Total Land Acquisition Costs		$ 10,810,052
Unit Land Acquisition Costs	($/acre)	2,754
DESIGN & CONSTRUCTION (incl. construction mgt.)		
Design		$ 713,881
Construction		$ 29,006,958
Utility Relocation (estimated)		$ 3,260,000
Corps of Engineers*		$ 4,059,082
TOTAL PROJECT COST (incl. land acq.)		$ 47,849,973
COST-SHARES AND CONTRIBUTIONS		
Local Interests†		$ 13,267,302
Previous Corps of Engineers Project		$ 4,059,082
PROJECT COST (Flood Control District)		$ 30,523,589
STUDY AREA COSTS‡		
Design		$ 198,003
Construction		$ 8,230,364
Construction Management		$ 668,098
Land Acquisition		$ 2,615,980
TOTAL		$ 11,712,445

* Separate project financed solely by the Corps of Engrs.
† $ 5,028,270 (Az. Dept. of Trans.), $ 5,773,134 (Mar. Cnty. Dept. of Trans.), $ 617,873 (Roosevelt Irr. District), and $ 1,848,025 (sand and gravel operators).
‡ Thomas Road to McDowell Road, 1¼ miles.

To determine the flood flow-frequency, the method recommended by the U. S. Water Resources Council [*WRC*, rev. 1981] will be used. In the method, a log-Pearson Type III base distribution is used.

Since the releases for the lower Agua Fria River has been heavily regulated since 1927, there are many years in the historical record of no flow. The Water Resources Council cited "...[if] the hydrologic design of a project is governed by the critical hydrologic conditions only, e.g., floods and droughts, not all available data are necessary in producing significant results from the frequency analysis." [1] Since the levees were constructed for the critical hydrologic conditions of flood control, only the 12 measured flows of the 23-year historic record will be used in the analysis.

TABLE 3. Annual Maximum Discharges on the Agua Fria River at Avondale Arizona [*U. S. Geological Survey*, 1989].

Water Year	Date of Peak	Annual Peak Discharge, x (cfs)
1960	12-25-59	4,700
1961	----------	0
1962	----------	0
1963	08-00-63	63
1964	08-01-64	3,000
1965	04-05-65	460
1966	12-23-65	800
1967	----------	0
1968	12-20-67	20,000
1969	----------	0
1970	08-06-70	20,600
1971	08-21-71	8,200
1972	07-17-72	5,180
1973	10-07-73	5,000
1974	----------	0
1975	----------	0
1976	----------	0
1977	----------	0
1978	03-02-78	13,100
1979	12-19-78	29,300
1980	----------	0
1981	----------	0
1982	----------	0

The process to establish the flood flow-frequency begins by transforming each sample flow data, x_i to its logarithmic value y_i, such that $y_i = \log x_i$. The transformed values are used to determine the sample statistics. The mean of the log-transformed values, $\bar{y}$ the standard deviation, s_y and the sample skew coefficient, C_s as shown in Table 4. The mean of the log-transformed values and the standard deviation are

$$\bar{y} = \frac{1}{n}\sum_{i=1}^{n} y_i \qquad (1)$$

$$s_y = \sqrt{\frac{1}{(n-1)}\sum_{i=1}^{n} (y_i - \bar{y})^2} \qquad (2)$$

where n is the number in the data sample.

The *symmetry* of a distribution is measured by the *skewness*, which is the third moment about the mean [*Chow, et al*, 1988]. For a log-Pearson Type III distribution of a

sample, the estimate of the coefficient of skewness, C_s, is represented by

$$C_s = \frac{n\sum_{i=1}^{n} (y_i - \bar{y})^3}{(n-1)(n-2)s_y^3} \tag{3}$$

TABLE 4. Calculation of Statistics for Logarithms of Annual Maximum Discharges on the Agua Fria River at Avondale Arizona.

Year	Discharge x, (cfs)	$y = \log_{10} x$	$(y - \bar{y})^2$	$(y - \bar{y})^3$
1960	4,700	3.6721	0.0073	0.0006
1963	63	1.7993	3.1947	-5.7101
1964	3,000	3.4771	0.0120	-0.0013
1965	460	2.6628	0.8537	-0.7888
1966	800	2.9031	0.4673	-0.3195
1968	20,000	4.3010	0.5102	0.3645
1970	20,600	4.3139	0.5288	0.3845
1971	8,200	3.9138	0.1070	0.0350
1972	5,180	3.7143	0.0163	0.0021
1973	5,000	3.6990	0.0126	0.0014
1978	13,100	4.1173	0.2815	0.1493
1979	29,300	4.4669	0.7747	0.6818
Totals		43.0406	6.7661	-5.2004
Equations	(1)	(2)		(3)
$n = 12$	$\bar{y} = 3.5867$	$s_y = 0.7843$		$C_s = -1.1759$

The Water Resources Council recommends the use of a *generalized* skewness coefficient, C_w because the value given by C_s is sensitive to sample size and is difficult to estimate accurately, especially from small samples [*Chow, et al,* 1988].

C_w is an estimate which uses a weighing factor, W on the calculated sample skewness coefficient, C_s and an established map skewness coefficient, C_m, as read from Figure A1. C_w is determined as

$$C_w = WC_s + (1-W)\,C_m \tag{4}$$

The sample skewness coefficient and map skewness coefficient are assumed to have the same mean, but different variances, $V(C_s)$ and $V(C_m)$, respectively. The variance of the weighted skew is given by

$$V(C_w) = W^2 V(C_s) + (1-W)^2 V(C_m) \tag{5}$$

To determine the weight that minimizes $V(C_w)$, (5) is differentiated with respect to W and set to 0, to obtain

$$W = \frac{V(C_m)}{V(C_s) + V(C_m)} \tag{6}$$

The second derivative is shown to be positive, demonstrating that the weight given by (5) minimizes $V(C_m)$. To determine W, requires knowledge of $V(C_m)$ and $V(C_s)$. From the map of skew coefficients, $V(C_m)$ is estimated to be 0.3025 in the United States [*Chow, et al,* 1988]. By substituting (6) into (4), C_w can be written in terms of the variances as

$$C_w = \frac{V(C_m)\,C_s + V(C_s)\,C_m}{V(C_m) + V(C_s)} \tag{7}$$

To determine $V(C_s)$, [*Wallis, et al,* 1974] approximated that

$$V(C_s) = 10^{A - B\log_{10}(n/12)} \tag{8}$$

where

$$A = -0.33 + 0.08\,|C_s| \quad \text{if} \quad |C_s| \le 0.90 \tag{9a}$$
$$A = -0.52 + 0.30\,|C_s| \quad \text{if} \quad |C_s| > 0.90 \tag{9b}$$
$$B = 0.94 - 0.26\,|C_s| \quad \text{if} \quad |C_s| \le 1.50 \tag{9c}$$
$$B = 0.55 \quad \text{if} \quad |C_s| > 1.50 \tag{9d}$$

From the sample statistics in Table 4, and by (9b) and (9c), $A = -0.1672$ and $B = 0.6342$.

From (8), $V(C_s) = (10)^{-0.1672 - 0.6342\log(12/10)} = 0.6062$. By (6), the proper weight to apply to the generalized skewness coefficient is $W = 0.3025/[0.6062 + 0.3025] = 0.3329$. The distribution of the weight to the map skewness is 0.6671. From Figure A1, the map skewness is -0.1 and by (4), C_w is $0.3329(-1.1760) + 0.6671(-0.1) = -0.4582$.

For the logarithmic data used, the magnitude of the hydrologic event of return period T, is

$$y_T = \bar{y} + K_T s_y \tag{10}$$

where K_T is termed a *frequency factor.* The frequency factors are linearly interpolated for the value of C_w from Table A1 as 0.076, 0.856, 1.222, 1.583, 1.801, 1.986 and 2.147 for return periods of 2-, 5-, 10-, 25-, 50-, 100-, and 200-years. For a particular return period, the flood estimates are obtained by taking the antilogarithm of (10) as shown in column 4 of Table 5 written to three significant digits.

The Water Resources Council recommends that adjustments be made for *outliers.* Outliers are the annual maximum discharges that significantly depart from remaining data. To detect outliers, the following threshold equations are used

$$y_H = \bar{y} + K_n s_y \tag{11}$$

$$y_L = \bar{y} - K_n s_y \tag{12}$$

where y_H and y_L are the log values of the high and low outliers, respectively and K_n is a factor for one-sided tests to detect outliers from normally distributed data at the 10-percent level of significance [*Water Resources Council,* rev. 1981]. For a sample size of 12, $K_n = 2.134$.

By (11), the log value for the threshold for high outliers is $3.5867 + 2.134(0.7843) = 5.2604$, from which $Q_H =$

182,126 cfs. Since no data are larger than Q_H, there are no high outliers. By (12), y_L = 1.9131 and Q_L = 82 cfs. The peak discharge of 63 is below the threshold, is removed from the data, and the process is repeated, with n = 11. The new values are,

$\bar{y}$ = 3.7492, s_y = 0.5728, C_s = −0.7394,
A = − 0.2709, B = 0.7478, $V(C_s)$ = 0.4991,
W = 0.3774, C_w = − 0.3413,

and the map skew, C_m and variance to the map skewness, $V(C_m)$ remain at − 0.1 and 0.3025, respectively.

For C_w = − 0.3413, the interpolated values for K_T are 0.057, 0.854, 1.239, 1.628, 1.867, 2.073, and 2.256 for return periods of 2-, 5-, 10-, 25-, 50-, 100-, and 200-years, respectively. The flood estimates, $\bar{Q}_T$ without the low outlier are shown in column 5 of Table 5, written to three significant digits.

TABLE 5. Flood Flow-Frequency Results Using the Water Resources Council Method for the Agua Fria River, Avondale, Arizona.

Column 1	2	3	4	5	6
Return Period, T (years)	Frequency Factor K_T	log Q_T	Flood Estimates Q_T (cfs)	$\bar{Q}_T$ (cfs)	Q† (cfs)
2	0.076	3.6467	4,400	6,050	--------
5	0.856	4.2581	18,100	17,300	--------
10	1.222	4.5451	35,100	28,800	27,000
25	1.583	4.8282	67,300	48,100	47,000
50	1.801	4.9992	99,800	65,900	67,000
100	1.986	5.1443	139,000	86,400	85,000
200	2.147	5.2706	186,000	110,000	--------
SPF‡	--------	--------	---------	131,000	131,000

† Source: *Phoenix City Streams, Arizona, Agua Fria River, Hydrology*, U. S. Corps of Engineers, 1981.
‡ In the vicinity of the study reach, the SPF is associated with a return period of approximately 350 years.

The values in the last column of the Table 5 were developed by the Corps of Engineers. The design flow for the Agua Fria River levees was the SPF. This discharge ranged from 140,000 cfs at Indian School Road to 130,000 cfs at the Gila River.

The standard project flood is not generally associated with a return period, but can nonetheless be established from the sample statistics for the annual maximum discharges and a given value for the SPF.

In determining the return period, the frequency factor, K_T in (10) is the same as the standard normal deviate, z for a skewness coefficient of zero. For a non-zero coefficient, the value of z corresponding to an exceedence probability of p (p = 1/T) that can be calculated by the intermediate variable w [*Chow, et al,* 1988] as

$$w=\sqrt{\ln\left(\frac{1}{p^2}\right)} \tag{13}$$

z can then be approximated by

$$z=w-\frac{2.516+0.803w+0.0103w^2}{1+1.433w+0.189w^2+0.00131w^3} \tag{14}$$

The frequency factor for a log-normal distribution is derived from [*Kite,* 1977]

$$K_T=z+(z^2-1)k+\frac{(z^3-6z)k^2}{3}-(z^2-1)k^3+zk^4+\frac{k^5}{3} \tag{15}$$

where $k=C_s/6$. [2]

The process to determine the return period of the SPF, is iterative using (13) through (15) to determine K_T and substituting the value into (10) to determine $\bar{Q}_T$. For a first trial, (with the low outlier removed) let T = 300 years,

w = 3.3775 z = 2.7132 K_t = 2.3565
log $\bar{Q}_T$ = 5.0990 $\bar{Q}_T$ = 125,600 cfs

The process is repeated until the proper return period is found. For T = 350 years,

w = 3.4228 z = 2.7638 K_t = 2.3924
log $\bar{Q}_T$ = 5.1195 $\bar{Q}_T$ = 131,700 cfs

The SPF near the study reach has an associated return period of approximately 350 years.

DEPTH-DAMAGE RELATIONSHIP

With the comprehensive plan to divert flows to Skunk Creek and the New and Agua Fria Rivers, the Flood Control District proceeded to plan and design the improvements to those waterways. Initial study efforts were directed for the Agua Fria [*Willdan,* 1981]. The study compiled a land inventory of land use types, zoning, value, acreage, and property identification for 1,321 parcels for the Agua Fria from the Gila River to the Beardsley Canal flume, a 29-mile reach. Within the 1¼ mile study reach, six land use types were identified; one-story residential, two-story residential, mobile home residential, vacant residential, agricultural, and industrial. Since, data on flood damages for the 1978 and 1980 floods was not specific to the study area, the following assumption will be made in the depth-damage analysis:

(1) based on averages for data for the 29-mile reach, the average annual flood loss (AAFL) to agriculture (under cultivation or fallow) is estimated at $2,400 per acre,

(2) parcels identified as residential vacant will be

TABLE 6. Land Use Property Values and Parameters to the Determine the *Percent of Total Damage* Curves [*Helweg*, 1981].

		LAND USE				
		One-story House dwelling	Two-story House dwelling	Mobile Home† dwelling	Agricultural/ Vacant Residential acre	Industry acre
Unit						
Quantity		4	2	47	312	21
Average Value		$ 28,900*	$ 51,600*	$ 11,500*	$ 2,400	$ 3,500
Depth of Water (feet)	Value of Damages	PERCENT DAMAGED				
0	$ 0	0	0	0	0	0
2	211,645	10	15	20	10	15
4	495,760	40	30	50	20	30
6	746,110	70	45	80	25	50
8	923,990	80	60	100	30	70
10	1,078,050	90	75	100	45	90
12	1,224,760	100	90	100	60	100
14	1,309,960	100	100	100	70	100

†Value per dwelling determined from one assessment of entire mobile home park.
*Unit values include 30% for furnishings.

valued equal to agricultural parcels,

(3) based on averages for data for the 29-mile reach, the AAFL for sand and gravel sites, (the only industrial use identified in the study area) is $3,500 per acre.

Typical AAFL analyses calculate the depth-damage relationship by considering the elevation of the first floor of each structure. This is done to take into account the often varying relationship of the ground and first floor elevations between adjacent dwellings. For a given water depth two structures of the same type, one-story houses, for example, will have different damages if their first floor elevations are different.

In the study reach, the standard project flood limits were relatively constant in width before the construction of the levees [Plate 1]. This would indicate, that the water surface and ground surface were roughly parallel in the reach. Field visits indicated that the structures were built nearly at-grade, and therefore, the depth-discharge analysis need not consider first floor elevations.

By assumption 2, the six land use types are reduced to five, as shown in Table 6. For each land use, parameters are presented from which a the depth-damage curve can be developed. At a 2-foot water depth, the combined total damage from the five land uses is 0.10(4)($28,900) + 0.15(2)($51,600) + 0.20(47)($10,500) + 0.10(312)($2400) + 0.15(21)($3500) = $ 211,645. The remaining values are shown in Table 6. Figure 1 plots the depth-damage curve with the damage costs rounded to three significant digits.

THE RATING CURVE

The rating curve for the study area is determined from a Flood Control District stream gauge at Buckeye Road as shown in Table 7. The gauge is several miles downstream of the study area but represents the best available data. For stage S, the least-squares fourth-order approximation for discharge is $Q=0.586S + 0.444S^2 + 0.0304S^3 - 4.56\times10^{-4} S^4$ The curve is plotted as Figure 2.

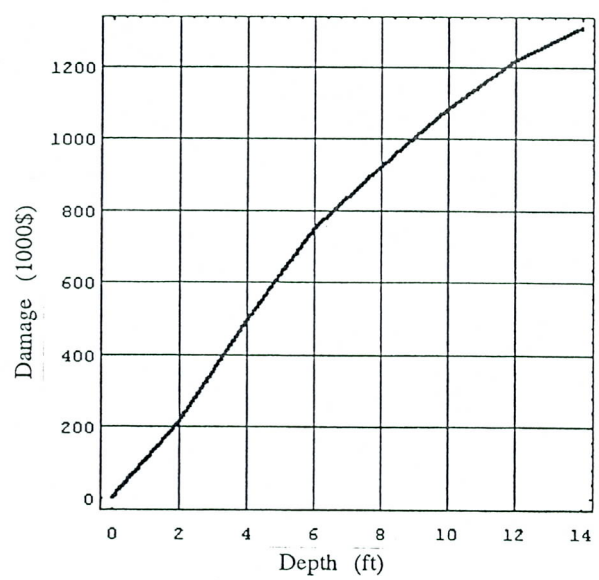

FIGURE 1. Depth-Damage Curve for the Agua Fria River (Thomas Road to McDowell Road).

DISCHARGE-DAMAGE RELATIONSHIP

The discharge-damage curve is derived by replacing depth by discharge on the abscissa in Figure 1. The data in Table 8 is used to generate the curve as plotted in Figure 3.

TABLE 7. Stream flow data for the Agua Fria River at Buckeye Road [*Flood Control District of Maricopa County,* 1993].

Stage (ft)	Discharge (cfs)	Stage (ft)	Discharge (cfs)	Stage (ft)	Discharge (cfs)
0	0	5	17,500	10	78,000
1	600	6	25,000	11	95,000
2	3,500	7	34,500	12	112,250
3	7,000	8	46,500	13	136,000
4	11,500	9	60,500	14	162,000

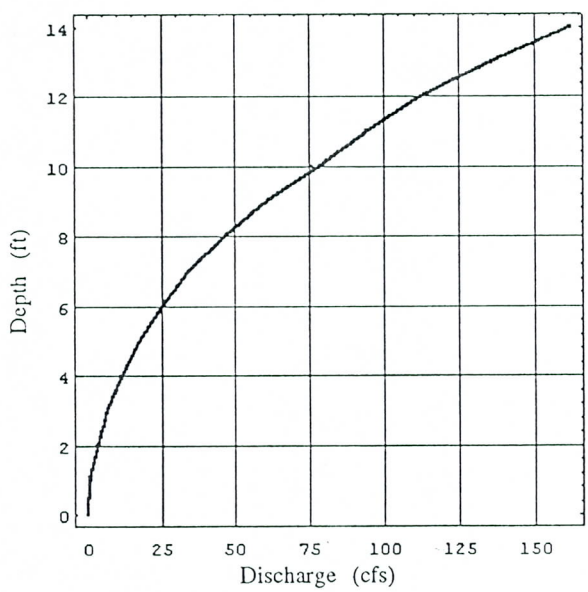

FIGURE 2. Rating Curve for the Agua Fria River at Buckeye Road [Compiled from Flood Control District Stream Gauge Data].

TABLE 8. Discharge-Damage Data for the Agua Fria River, Thomas Road to McDowell Road.

Discharge (cfs)	Damage (1000$)	Discharge (cfs)	Damage (1000$)	Discharge (cfs)	Damage (1000$)
0	0	25,000	746	112,500	1,220
3,500	212	46,500	924	162,000	1,310
11,500	496	78,000	1,080		

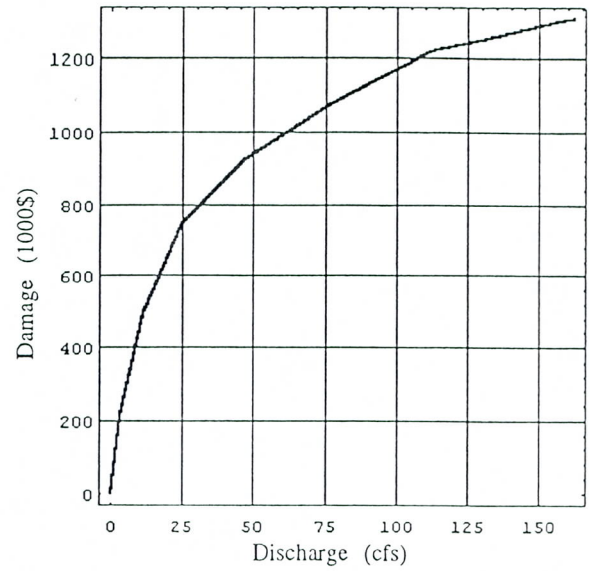

FIGURE 3. Discharge-Damage Curve for the Agua Fria River (Thomas Road to McDowell Road).

FREQUENCY-DAMAGE RELATIONSHIP

The final step in the average annual flood loss analysis is to establish the relationship of flood flow-frequency to the value of the damages caused by flooding. In Figure 3, discharge is plotted along the abscissa. Combining Figure 3 with column 5 of Table 5, will establish frequency-damage relationship.

To illustrate, the second and third data points in Figure 3, as found in Table 8, are 3,500 cfs, $212,000 and 11,500 cfs, $496,000. Between those two discharges, lies the discharge for the 2-year flood, 6,050 cfs. By interpolating between the data points, the 2-year flood has a corresponding damage of

$$212{,}000+\frac{6{,}050-3{,}500}{11{,}500-3{,}500}(496{,}000-212{,}000)=\$302{,}525$$

The remaining discharges and damages are calculated for Table 9. To calculate the AAFL, the area under the curve of the damage-frequency curve is determined. The areas between successive exceedence probabilities, the corresponding damages, and the frequency-damage curve form trapezoids. The area between the 2- and 5-year events has a base of 1/2 - 1/5 = 3/10. The heights of the trapezoid are $303,000 and $603,000. The area therefore, is

$$0.3\,\frac{(303{,}000+603{,}000)}{2}=\$135{,}900$$

RESULTS OF ANALYSES

Table 9 clearly shows that the AAFL does not appreciably increase beyond the 50-year flood. Moreover, the benefit of

the levees, (flood losses averted) is extremely low when compared to the study area's total cost of $11,712,445 as shown in Table 3.

TABLE 9. Depth-Damage Calculations for the Agua Fria River, Thomas Road to McDowell Road.

Return Period, T (years)	Exceedence Probability p = 1/T	Flood Estimate Q_T (cfs)	Value of Damages (1000$)	Cum. Avg. Annual Flood Loss (AAFL) (1000$)
2	0.5000	6,050	303	----
5	0.2000	17,300	603	136
10	0.1000	28,800	777	205
25	0.0400	48,100	933	256
50	0.0200	65,900	1,020	276
100	0.0100	84,600	1,114	286
200	0.0050	110,000	1,121	292
SPF	0.0029	131,000	1,250	295

CONCLUSIONS

This paper has attempted to determine the most appropriate level of protection for the Agua Fria River levees based on an analysis of a 1¼ mile reach of the levees. It has been demonstrated that the cost of the levees versus the benefits of flood losses averted to the area was not justified.

Justification for the project must come from other than economical sources.

Political: The severity of regional flooding in 1978 through 1980 gained a strong public awareness and support for flood control. Political response to and public acceptance contributed to the comprehensive plan as outlined in this paper. Since the Agua Fria River levees are a portion of that plan, they are partially justified for political reasons.

Social: The Agua Fria River Levees are on the downstream end of a flood control comprehensive plan. They receive drainage from heavily urbanized areas as well as diverted drainage and dam release flows. From a public concern perspective, a lesser more economical system would likely not have the confidence of the public as a major flood control facility.

Institutional: At the project conception stage, projections on population growths in the area were used as justification. Indeed, in the vicinity of upstream portions of the Agua Fria, as well as New River and Skunk Creek there are noted developments. The reaches where the levees were constructed, Indian School Road to Buckeye Road are not developed nearly to the extent projections as far back as 15 years would have predicted.

Federal mandates: The participation by the U. S. Army Corps of Engineers, mandated that dam spillways pass the SPF, levees protect from the SPF, and that flowage easements be purchased to the SPF delineation limit. The results in Table 9 suggest that levee heights, although not economical for any height, did not appreciably benefit the community for heights above the 50-year storm. A significant cost savings would have been realized had the levees been built to even a 100-year height. In easement costs, there would have been a 48-percent savings of approximately $1,250,000 in the study reach, if flowage easements were purchased to the 100-year floodplain and not the SPF delineation limit.

In the Corps of Engineers' cost benefit analysis, such factors as replacement of bridges and other public facilities, costs of emergency services averted, loss of income averted were all used. Those concerns were not analyzed in this paper because public policies, taxes, and insurance exist to account for these losses apart from flood control protection.

The entire picture: Of the over four miles of the total Agua Fria River levees, only 1¼ miles were studied in this paper. The benefits to upstream portions are undoubtedly greater than in the study reach; however, it is not likely that justification would have been close to being achieved solely by economic reasons.

Studies in the early 1980s indicated that the project was economically feasible, and socially and politically necessary. This analysis made since their construction in the mid-1980s has shown that aside from mandates and public outcry, the levees are quite difficult to justify.

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Tung, Y. -K., and L. W. Mays, Optimal Risk-Based Design for Water Resources Engineering Projects, *Technical Report CRWR-171,* Center for Research in Water Resources, Univ. of Texas, Austin, Texas, 1980.

[1] Guidelines for Determining Flood Flow Frequency, *Bulletin 17 C,* Committee on Water Data, U. S. Water Resources Council, rev. Sep. 1981, §13.4.

[2] The generalized skewness coefficient, C_w, is used instead of C_s, as referenced from Kite (1977), because C_s is difficult to estimate, especially for small sample sizes.

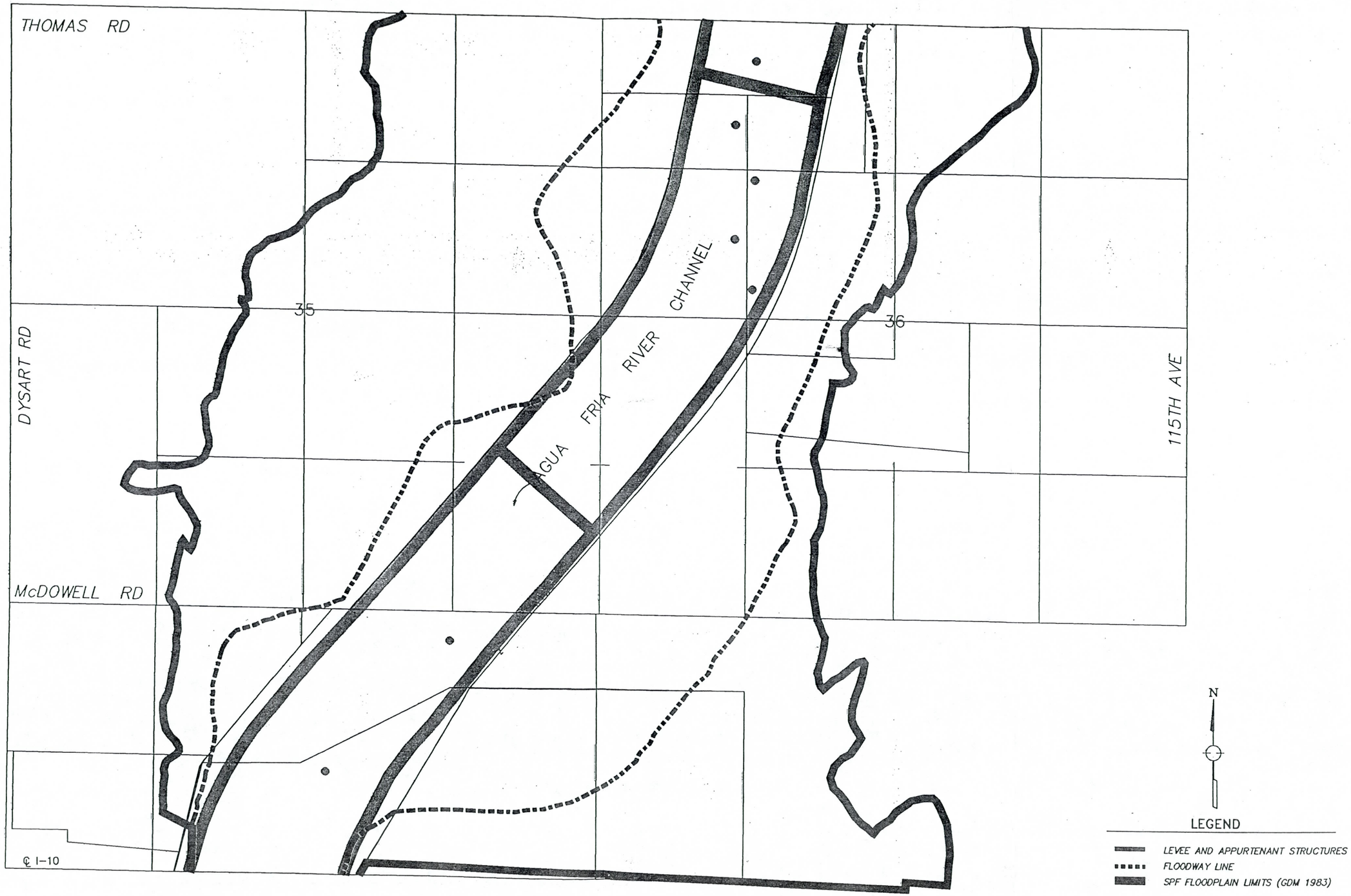
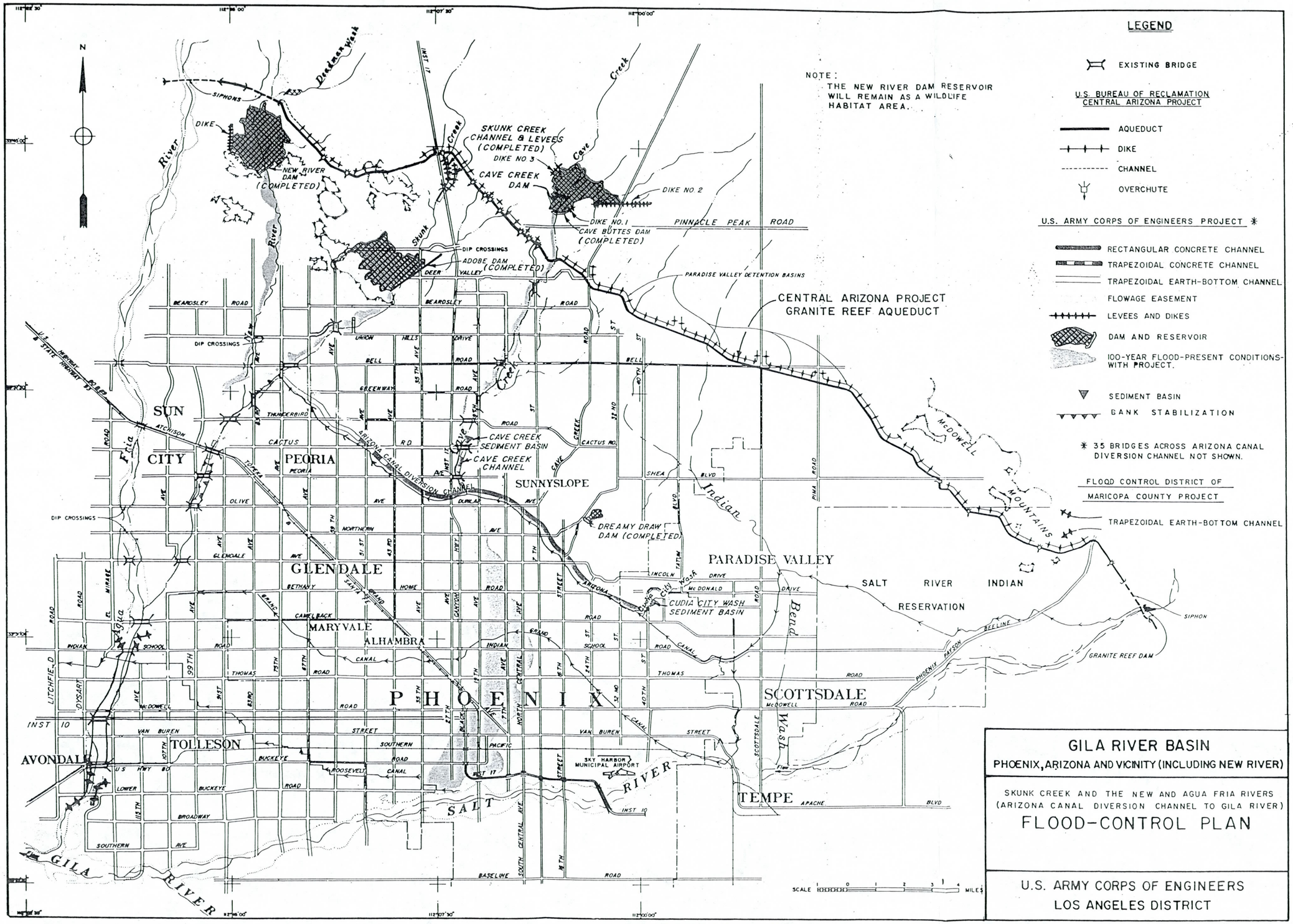

THOMAS RD
DYSART RD
McDOWELL RD
115TH AVE
35
36
AGUA FRIA RIVER CHANNEL
℄ I-10
N
LEGEND
LEVEE AND APPURTENANT STRUCTURES
FLOODWAY LINE
SPF FLOODPLAIN LIMITS (GDM 1983)

PLATE 1



GILA RIVER BASIN

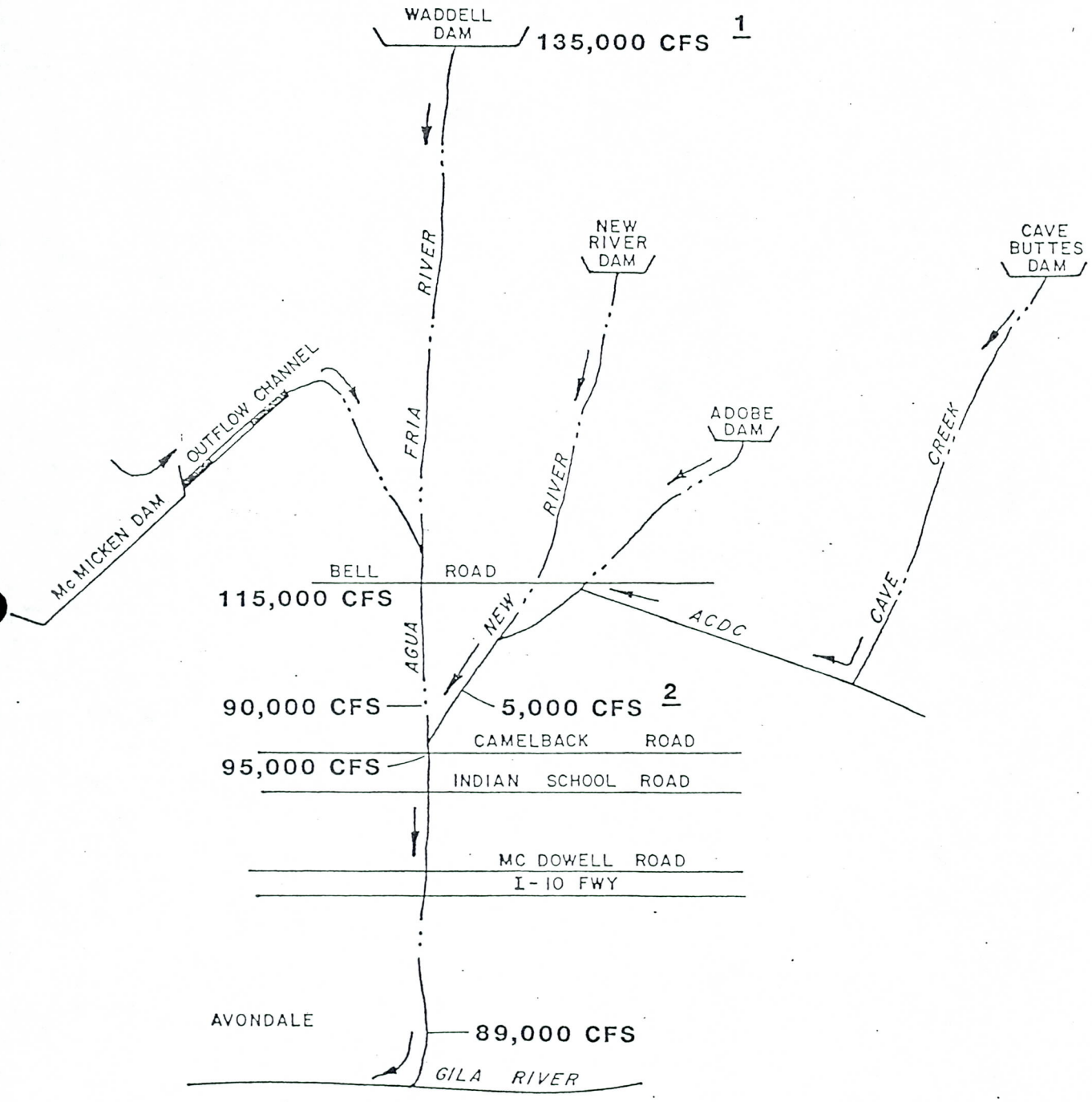
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

SKUNK CREEK AND THE NEW AND AGUA FRIA RIVERS (ARIZONA CANAL DIVERSION CHANNEL TO GILA RIVER)

FLOOD-CONTROL PLAN

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 2



1 FLOWS SHOWN ARE FOR A 100 YEAR STORM

2 FLOW SHOWN IS CONTRIBUTION TO PEAK FLOW DURING A 100 YEAR STORM

Source:
U.S. Army Corps of Engineers

AGUA FRIA RIVER
SCHEMATIC FLOW DIAGRAM
PLATE 3

TABLE 12.3.1
K_T values for Pearson Type III distribution (positive skew)

Skew coefficient C_s or C_w	Return period in years: 2	5	10	25	50	100	200
	Exceedence probability: 0.50	0.20	0.10	0.04	0.02	0.01	0.005
3.0	−0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	−0.390	0.440	1.195	2.277	3.134	4.013	4.909
2.8	−0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	−0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	−0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	−0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	−0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	−0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	−0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	−0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	−0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	−0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	−0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	−0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	−0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	−0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	−0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	−0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	−0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	−0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	−0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	−0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	−0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	−0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.6	−0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.5	−0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.4	−0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	−0.050	0.824	1.309	1.849	2.211	2.544	2.856
0.2	−0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	−0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.0	0	0.842	1.282	1.751	2.054	2.326	2.576

TABLE 12.3.1 (*cont.*)
K_T values for Pearson Type III distribution (negative skew)

Skew coefficient C_s or C_w	Return period in years: 2	5	10	25	50	100	200
	Exceedence probability: 0.50	0.20	0.10	0.04	0.02	0.01	0.005
−0.1	0.017	0.846	1.270	1.716	2.000	2.252	2.482
−0.2	0.033	0.850	1.258	1.680	1.945	2.178	2.388
−0.3	0.050	0.853	1.245	1.643	1.890	2.104	2.294
−0.4	0.066	0.855	1.231	1.606	1.834	2.029	2.201
−0.5	0.083	0.856	1.216	1.567	1.777	1.955	2.108
−0.6	0.099	0.857	1.200	1.528	1.720	1.880	2.016
−0.7	0.116	0.857	1.183	1.488	1.663	1.806	1.926
−0.8	0.132	0.856	1.166	1.448	1.606	1.733	1.837
−0.9	0.148	0.854	1.147	1.407	1.549	1.660	1.749
−1.0	0.164	0.852	1.128	1.366	1.492	1.588	1.664
−1.1	0.180	0.848	1.107	1.324	1.435	1.518	1.581
−1.2	0.195	0.844	1.086	1.282	1.379	1.449	1.501
−1.3	0.210	0.838	1.064	1.240	1.324	1.383	1.424
−1.4	0.225	0.832	1.041	1.198	1.270	1.318	1.351
−1.5	0.240	0.825	1.018	1.157	1.217	1.256	1.282
−1.6	0.254	0.817	0.994	1.116	1.166	1.197	1.216
−1.7	0.268	0.808	0.970	1.075	1.116	1.140	1.155
−1.8	0.282	0.799	0.945	1.035	1.069	1.087	1.097
−1.9	0.294	0.788	0.920	0.996	1.023	1.037	1.044
−2.0	0.307	0.777	0.895	0.959	0.980	0.990	0.995
−2.1	0.319	0.765	0.869	0.923	0.939	0.946	0.949
−2.2	0.330	0.752	0.844	0.888	0.900	0.905	0.907
−2.3	0.341	0.739	0.819	0.855	0.864	0.867	0.869
−2.4	0.351	0.725	0.795	0.823	0.830	0.832	0.833
−2.5	0.360	0.711	0.771	0.793	0.798	0.799	0.800
−2.6	0.368	0.696	0.747	0.764	0.768	0.769	0.769
−2.7	0.376	0.681	0.724	0.738	0.740	0.740	0.741
−2.8	0.384	0.666	0.702	0.712	0.714	0.714	0.714
−2.9	0.390	0.651	0.681	0.683	0.689	0.690	0.690
−3.0	0.396	0.636	0.666	0.666	0.666	0.667	0.667

Source: U. S. Water Resources Council (1981).

Chow, et al, 1988.

Table A1

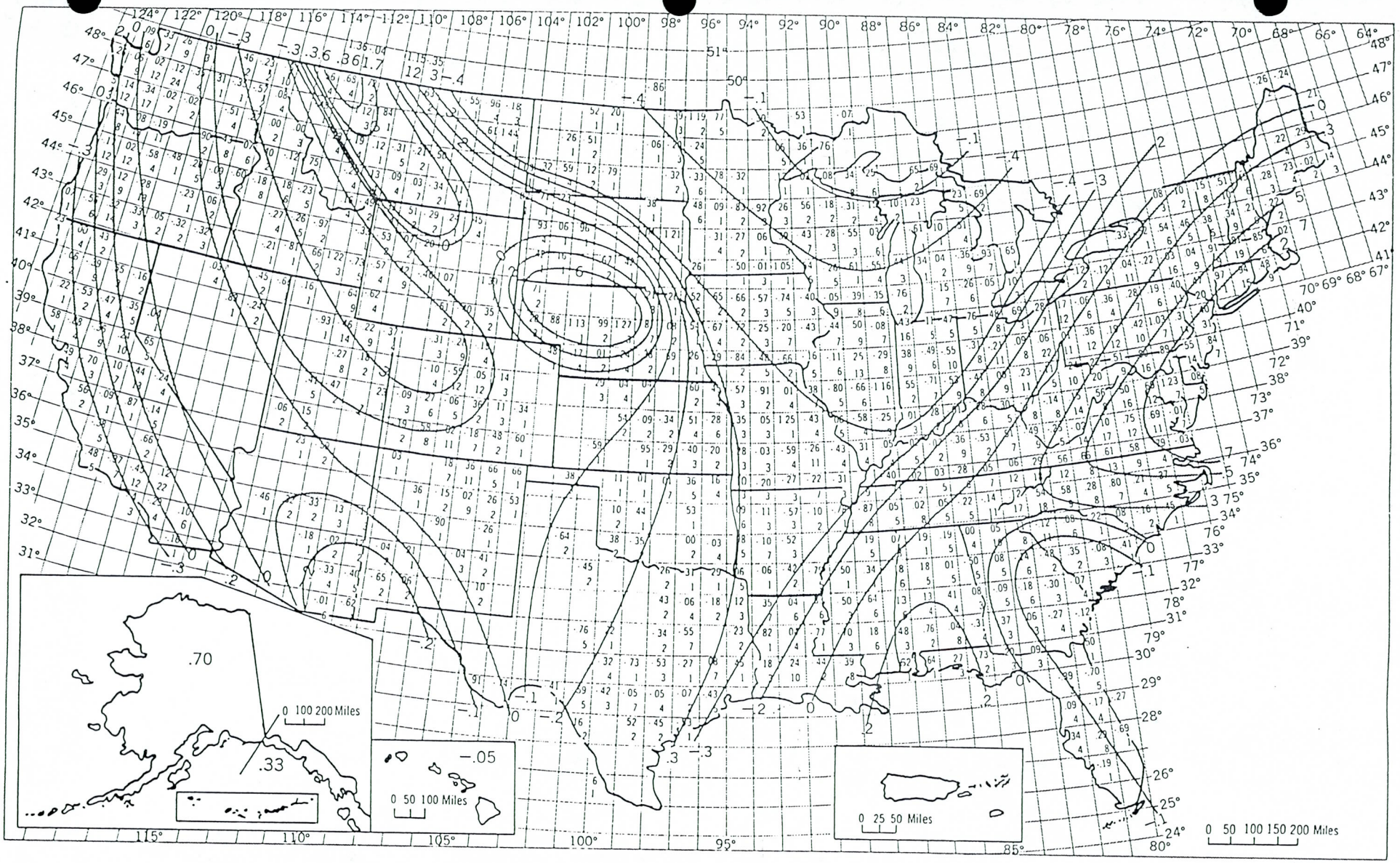


Figure 6-4 Generalized skew coefficients of logarithms of annual maximum stream flow. Average skew coefficient by 1° quadrangles. The lower number in each quadrangle is the number of stream-gauging stations for which the average shown above it was computed. (Source: WRC Guidelines.)

Helweg, 1985

Figure A1