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CITY OF PHOENIX
ARIZONA

PHOENIX STORM DRAINAGE REPORT

Mayor
JACK WILLIAMS

Council

V. A. CORDOVA
DR. JOSEPH MADISON GREER
G. WESLEY JOHNSON
DAVID P. JONES
MRS. FAITH I. NORTH
CLARENCE H. SHIVVERS

RAY W. WILSON, *City Manager*
K. K. KING, *Public Works Director*
R. GAIL BAKER, *City Engineer*

YOST AND GARDNER ENGINEERS
PHOENIX, ARIZONA
NOVEMBER 1, 1956

Nº 122

A026.301

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NOVEMBER 1, 1956

YOST AND GARDNER ENGINEERS
MAYER-HEARD BUILDING
PHOENIX, ARIZONA

November 1, 1956

Honorable Mayor and Council
City of Phoenix, Arizona

Submitted herewith is the Storm Drainage Report for Phoenix, in accordance with Agreement No. 3738 authorized by council action dated June 5, 1956.

The report consists of this volume and eight map exhibits bound separately. The report covers Phoenix in particular, and generally all adjacent areas draining to the Salt River.

We wish to thank Public Works Director K.K. King, City Engineer R. Gail Baker, the County Engineering Department, City and County Planning Departments, the State Highway Department, and others for information and assistance given. J.E. Schaefer of our office prepared the manuscript. The text includes many individual credits and acknowledgments for which our thanks are hereby extended.

Respectfully,

Leigh O. Gardner

Leigh O. Gardner

YOST AND GARDNI

Chelsea 2nd Fl 8/8/18/03 @ City Clerk's Office

15th Floor City Clerk 3rd & Washington

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I	Drainage Areas - Vicinity of Phoenix
II	Existing Street System
III	Contour Map
IV	Present Zoning
V	Future Land Use
VI	Runoff Factors and Expected Flows
VII	Flows for Suggested Improvements
VIII	Existing Storm Sewer System

I. SYNOPSIS

This section presents a brief summary of the material covered in detail in the following five sections. The reader is referred to those sections for amplification of the points merely mentioned here and for the reasoning behind the conclusions here stated.

A. The Drainage Area

Amount and rate of runoff are functions of certain qualities of the drainage area. There are several distinct basins which lie in the area covered by this report, the largest being the Cave Creek drainage. The areal extent, slope, surface characteristics, land use, and pattern of drainage channels of the basins and the effect of these qualities on runoff are described. The possibilities of the area in regard to flood control reservoir sites are evaluated. Certain artificial obstructions to the drainage of storm water are mentioned.

B. Hydrology

Runoff quantity and rate are also influenced by amount and intensity of rainfall and the areal extent of storms. Summer storms impose the most severe requirements on a storm drainage system so the characteristics of local thunderstorms are discussed in some detail. Attention is given to rainfall that does not become runoff and the effects of infiltration into the soil are evaluated. Finally a formula is derived which is used in computing the amounts of runoff to be expected throughout the area covered by the report.

C. Design Criteria

The proposed storm drainage system should have a consistency of design affording equivalent protection to comparable areas. It is shown to be economically impractical to provide for the worst possible storm. Recommended construction below the Grand Canal provides for storms such as occur no oftener than once a year on the average and can be augmented later by drains handling storms such as can be expected only once every two years. Facilities for Cave Creek and mountain runoff should handle hundred-year storms.

City areas presently subject to flooding are delineated and the probable causes discussed. Incorporation of irrigation and waste ditches in a storm drainage system is considered. The capacity of street pavements as drains is appraised. Finally, certain suggestions are made concerning division of responsibility for, and allocation of costs of drainage improvements.

D. Proposed Storm Drainage System

Work recommended to be done is divided into the two categories of immediately necessary work and future work some of which is to be done by the city and some by others. The immediate program includes lines on 24th and 7th Streets and 7th and 19th Avenues (approximate alignments) from the Grand Canal to the Salt River. Also included are branch lines to

areas presently prone to flooding, certain improvement to the Old Cross-cut Canal, and the purchase of right-of-way for a future floodway north of the Arizona Canal. Future work includes the construction of this floodway and lines on 27th Avenue, and other north-south section line streets. County participation in the major projects outside city limits is recommended. It is recommended that the Mayor appoint a committee representing the City, County, and Salt River Valley Water Users Association to study methods of financing, constructing and operating major flood protection works to the benefit of the community as a whole.

E. Cost Estimates

The cost of the work proposed for immediate construction is estimated as follows:

1.	Twenty fourth Street Drain - Grand Canal to Salt River	$ 713,000
2.	Seventh Street Drain - Grand Canal to Salt River	1,451,000
3.	Seventh Avenue Drain - Grand Canal to Salt River	1,665,000
4.	Nineteenth Avenue Drain - Grand Canal to Salt River	1,438,000
5.	Miscellaneous Collectors	578,000
6.	Old Cross-cut Canal Improvements *	228,000
7.	Arizona Canal Floodway, Right-of-way Costs*	360,000
	TOTAL COST	$6,433,000

*It is recommended that the County participate in expenditures for these items.

The cost of future work is not estimated except the Arizona Canal Floodway which will cost about $3,750,000.

F. Restatement of Principal Recommendations

The city should take the following action:

1. Proceed with plans and financing by bond issue or other means for the construction of drains on 24th and 7th Streets, 7th and 19th Avenues, and appurtenant lines.
2. Propose to the Maricopa County Board of Supervisors that joint action be taken with the City for the improvement of the Old Cross-cut Canal and for acquisition of right-of-way for a floodway north of the Arizona Canal. These improvements require the participation and cooperation of the Salt River Valley Water Users Assocation.
3. Appoint a Flood Protection Improvement Committee charged with the responsibility to study permanent means to finance, construct, and operate flood control works beyond the scope of City action. This work has the possibility of being of greatest future benefit to the community.
4. Request the Maricopa County Planning and Zoning Commission to act to keep all natural waterways unobstructed and to keep section, midsection, and quarter section lines open, at least in the direction of greatest land slope, to facilitate drainage.

II THE DRAINAGE AREA

"Those who look out for water must also observe the nature of the ground" Vitruvius, "On Architecture" Book VIII, Chapter I.

Vitruvius speaks here of surface indications of subterranean water but the precept is applicable in a larger sense than this eminent Roman engineer intended. It is certainly worth following by one who would look out for storm water. Some of the effects of the nature of the ground upon the amount of run-off are obvious, others more subtle, but in the aggregate they are profound. They must be properly evaluated in any logical storm drainage design.

It is largely the effect of meteorological factors the determines for any given locality what its maximum rainfall intensity may be and thus sets a ceiling, so to speak, on the possible runoff rate per acre. Actual runoff rates may be far below this ceiling or may come very close to it. How close they come is determined by the nature of the ground as it affects the efficiency of the drainage system. This depends on such things as the shape of the drainage area, the location of the center of area relative to its outlet, the slope and frictional characteristics of its waterways, and the texture and permeability of its surfaces. This

section will discuss in a general way the nature of the drainage area in which Phoenix is located and its subsidiary internal and adjoining drainages.

A. Boundaries of the Phoenix Basin

From the point of view of susceptibility to floods much of Phoenix is very well located, being near the divide which circumscribes its basin. This divide begins on the north bank of the Salt River near the Tempe bridge and runs northerly through Papago Park, past the new City reservoirs on Thomas Road to the crest of Camelback Mountain. It continues northerly to Mummy Mountain and then westerly to Squaw Peak in the Phoenix Mountains and on into the hills north of Sunnyslope. At this point however, it ceases to remain near the city making a sweeping loop northeastward enclosing Cave Creek and its tributaries. From a point about 43 miles north of Phoenix it returns along the Cave Creek channel and about 2 miles west of it into Deer Valley where it almost loses itself in the flat agricultural lands. It continues as a barely distinguishable ridge running just southeast of Glendale and west of Tolleson to the junction of the Gila and Agua Fria Rivers.

South Phoenix is located in a much smaller drainage area. The divide to the south is the crest line of the Salt River Mountains in South Mountain Park. The easterly limit is a ridge in the neighborhood of 56th Street. The westerly boundary runs diagonally northwestward from the western foothills of the Salt River Mountains toward the confluence of the Salt and the Gila. There are also small independent

drainage areas south of Tempe which fall into the scope of this report. Paradise Valley is not naturally tributary to the Phoenix drainage area. Principal drainage is thru the Indian Bend Wash or wasteway with outlet into the Salt River about two miles east of the Tempe bridge.

Figure 1 shows all of these areas in their relation to one another. The areas in the basins and sub-basins shown in Figure 1 are approximately as follows:

Upper Cave Creek Drainage Area (above Cave Creek Dam)	186 square miles
Lower Cave Creek Drainage Area (Cave Creek Dam to Shaw Butte)	60 square miles
Phoenix Drainage Area (exclusive of Cave Creek areas which are tributary)	180 square miles
South Phoenix Drainage Area	61 square miles
Paradise Valley Drainage Area	214 square miles

B. Slopes

There is a good deal of variation in the gradients of surfaces and drainage channels occurring in the basins that are here considered. Elevations vary from 5285 at Rover Peak near the head of Cave Creek to 925 at the Agua Fria-Gila junction. In the Salt River Mountains the highest point lies at an elevation of 2612 feet. Channel slopes are usually very steep in the upper reaches. Upper Cave Creek has a slope of about 300 feet per mile. In the lower portions the channels flatten to slopes of approximately 15 feet per mile and frequently lose their identity as waterways. Figure 2 shows profiles of some of the important natural drainage ways in this area.

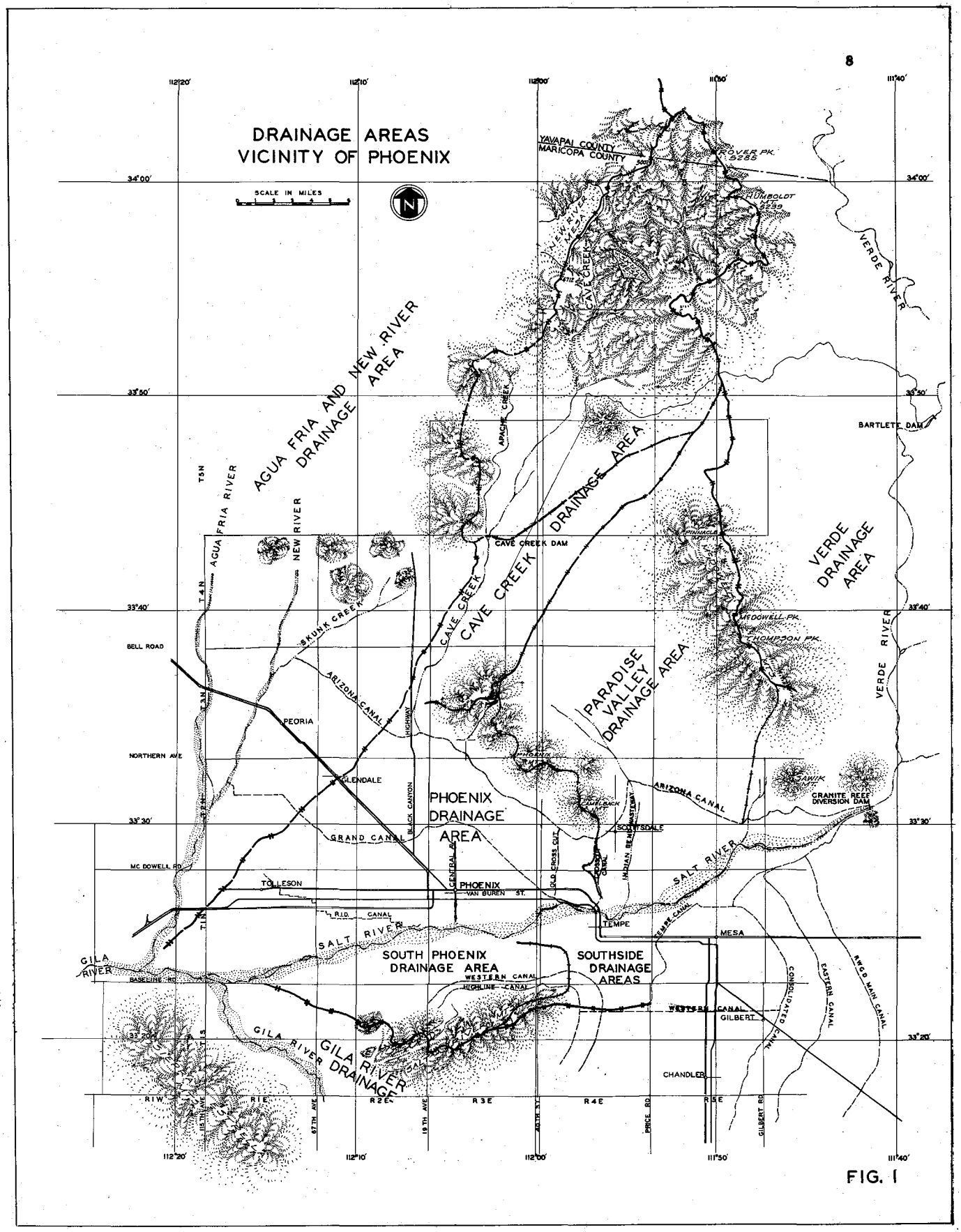
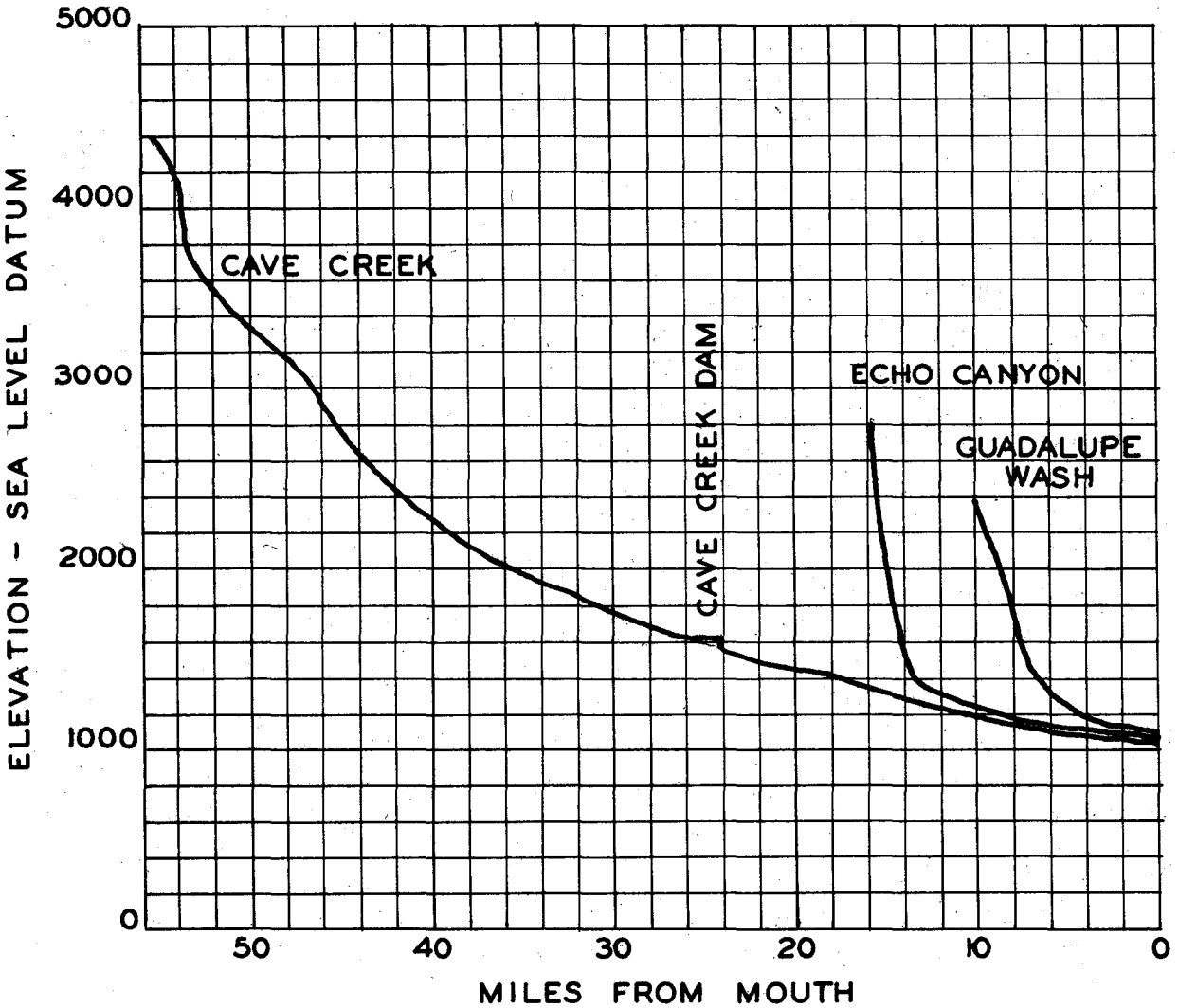

DRAINAGE AREAS
VICINITY OF PHOENIX
SCALE IN MILES
N
YAVAPAI COUNTY
MARICOPA COUNTY
HOVER PK.
HUMBOLDT MT.
VERDE RIVER
AGUA FRIA AND NEW RIVER DRAINAGE AREA
AGUA FRIA RIVER
NEW RIVER
CAVE CREEK
CAVE CREEK DRAINAGE AREA
CAVE CREEK DAM
BARTLETT DAM
VERDE DRAINAGE AREA
MEDOWELL PK.
THOMPSON PK.
SKUNK CREEK
BELL ROAD
ARIZONA CANAL
PEORIA
NORTHERN AVE
GLENDALE
PARADISE VALLEY DRAINAGE AREA
PHOENIX DRAINAGE AREA
GRANITE REEF DIVERSION DAM
SCOTTSDALE
GRAND CANAL
MC DOWELL RD
TOLLESON
PHOENIX
VAN BUREN ST.
R.I.D. CANAL
SALT RIVER
TEMPE
MESA
GILA RIVER
BASELINE RD
SOUTH PHOENIX DRAINAGE AREA
SOUTHSIDE DRAINAGE AREAS
WESTERN CANAL
HIGHLINE CANAL
GILBERT
CHANDLER
GILA RIVER DRAINAGE
EASTERN CANAL
CONSOLIDATED CANAL
AWGD MAIN CANAL
T3N
T4N
T2N
T1N
T1S
R1W
R1E
R2E
R3E
R4E
R5E
112°20'
112°10'
112°00'
111°50'
111°40'
34°00'
33°50'
33°40'
33°30'
33°20'

FIG. 1



PROFILES OF PRINCIPAL CHANNELS

FIG. 2

C. Surface Characteristics

The nature of the ground surface has an important effect on the amount of rainfall which becomes runoff and upon the rate at which runoff develops. Natural surfaces may be much more capable of absorbing rainfall than surfaces which have been artifically altered. Even desert soils in their natural state are very permeable compared to bare earth surfaces which have been subjected to traffic. On the other hand, desert soils which have been leveled and enclosed with berms for the retention of irrigation water and upon which a heavy cover of grass is growing are much more permeable to moisture in their present state than they were originally.

In addition to the state of the surface there is the intrinsic ability of the soil itself to absorb moisture. This varies with the type of soil occurring in the area and in this aspect there is also a considerable variation. It may be almost zero in the case of bare rock or as high as 6 to 8 inches per hour for open gravelly soils. Infiltration capacity is discussed in detail later in this report.

The nature of the soil is of no effect if it is covered with an impervious pavement or by a roof. The portion of the total surface of an area which is impervious depends of course upon the use to which the area is put. For the purposes of this study the surfaces comprising the drainage areas being considered are classified according to the following system.

Classification	Percent Pervious
1. Unimproved areas (essentially in their natural state)	
A. Rocky outcrops	100
B. Talus and alluvium	100
2. Improved areas	
A. Parks and parklike areas (including farm lands and groves)	90
B. Residential areas	
(1) Large lot (acre lots and over)	70-80
(2) Ordinary single family	60
(3) Multiple family	50-65
(4) Residential areas considering street pavement only	80-85
C. Industrial areas	10-30
D. Commercial areas	5

The figures indicating the degree of perviousness are based on inspection of aerial photographs and measurements of typical areas in each category of use. There are wide variations within each category of course, and considerable overlapping between categories. The figures are useful only as general indicators. In final storm sewer design they should be tempered by the judgment of the designer based upon his actual observation of the area in question.

Land use in the future is estimated to be about as shown on the map entitled Future Land Use which is Exhibit V of the group of large maps submitted separately with this report. The assumptions of future land use

are based on the supposition that industries will tend to develop in the areas that are relatively less desirable for other purposes and where transportation, power and water are readily available. It is presumed that there will be more intensive development of the present industrial area between the railroad and the river and that some residential zones in this area will be taken over by industry. The present downtown commercial area will probably expand to include most of the area between McDowell Road and Madison Street from 15th Avenue to 16th Street. The present pattern of scattered local commercial centers in the residential areas will probably continue in the future but with the commercial centers larger and farther apart than is presently the case. There will probably also be some scattered light industry of an inoffensive type in some areas that are primarily residential.

For the purposes of estimating future runoff from presently undeveloped areas which are expected to develop as shown in Exhibit V the following distribution of pervious and impervious area is assumed.

Classification	Percent Pervious	Percent Impervious
Park and parklike	90	10
Commercial	5	95
Industrial	10	90
Large lot residential including 3 percent commercial area	75	25
Residential including 5 percent commercial and 5 percent light industry	60-65	40-35
Residential including 7 percent commercial	60-65	40-35

It has been mentioned that the practice of irrigation has its effect upon storm runoff. Residential lots which are leveled and

surrounded by berms may retain so much water that there will be no run-off for any storm likely to occur. There are large areas where irrigation by flooding is practiced and this will very likely continue in the future. It is true that in the older residential sections of Phoenix most watering is done by sprinkling but parks and large lot areas such as in the Los Olivos District and in Country Club Manor are still flooded. The trend is toward lots which can be more economically watered by flooding than by sprinkling, minimum lot widths having increased from the once standard 50 feet to 65 feet at present. Continued use of flood irrigation in subdivisions is much more prevalent in the northeast than in the northwest however. A study of all sections having numbers divisible by three and lying between the Arizona, Old Crosscut, and Grand Canals, and east of Lateral 18 (59th Avenue) reveals that east of 7th Avenue 77 percent of the subdivided area continues to be irrigated, whereas this is true of only 12 percent of the subdivided area west of 7th Avenue. The area east of 7th Avenue is more intensively developed, 52 percent of the area being subdivided against 15 percent west of 7th Avenue.

It should be mentioned in this connection that while the effect of very severe storms may be alleviated by the conversion of desert areas into parks, golf courses, and lawns, yet the attendant increase in paved areas will make the one, two, and three year storms much more of a nuisance than formerly.

D. Drainage Channels

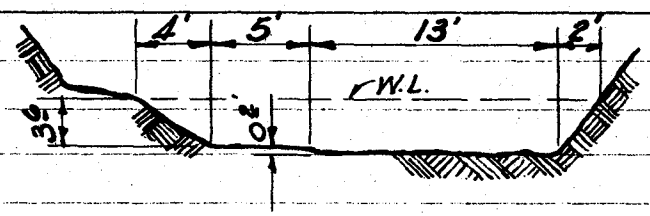
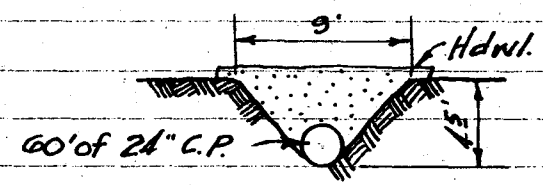
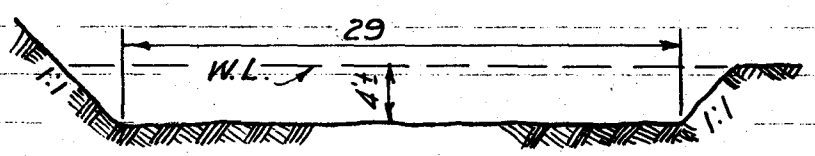
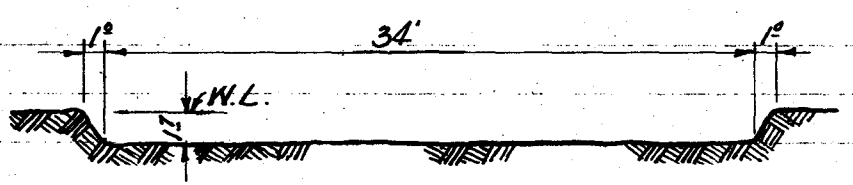
Natural storm drainage channels still exist in the upper undeveloped portions of the watershed. In the lower reaches where gradients are flat, runoff occurs as sheet flow over wide areas. In developed portions the street system does a large part of the work of carrying runoff. A system of storm drains to serve the area between 7th Avenue and 7th Street lying south of Roosevelt Street was constructed in 1935. This system served the area for which it was intended very well until it began to be overloaded by surface flow from points a mile or more away. The system utilizes collector lines in the alley between Madison and Jackson Streets and in 7th Avenue. It discharges to the Salt River through a 54- to 57-inch outfall in 7th Avenue. A 30-24-inch line was installed along 1st Street, between Polk Street and McDowell a few years ago to alleviate overloading of the line in Central Avenue. The State Highway Department has installed a 42-inch line along 19th Avenue to carry water from Grand Avenue. There is also a 36-inch Highway Department drain in 16th Street. This has been extended northward as far as the Grand Canal and east to 30th Street under various street paving projects. A 24-inch line runs westward on Indian School Road from 16th Street and discharges into the 19th Avenue waste ditch at Encanto Blvd. This line and others contributing to it were installed largely in connection with street paving projects. There are many local systems for storm drainage, especially north of the Grand Canal, which discharge into irrigation or waste ditches. Usually these have been installed under paving projects. The existing storm drainage system is shown in Exhibit VIII.

Natural channels on good slopes tend to become as large as is necessary to carry any flood likely to occur without spilling over their banks. Cave Creek where it leaves the Union Hills for example could carry 17,000 cubic feet per second when it is running full. Where slopes flatten water velocity drops below the minumum needed for channel cutting so sheet flow takes place. If this happens in a developed area nuisance and damage result. Capacities of some of the natural channels near Phoenix are shown in the following twelve pages.

Street pavements have their limitations as storm drainage channels. Capacities are limited by slopes and street cross section. Street systems are frequently not laid out from the point of view of providing proper drainage. Often a street which is well located as a drainage way for a short distance is not continuous, at least along the most favorable route from the drainage stand-point. Cul-de-sac and L-shaped streets are sometimes laid out with the dead end or angle lower than the entrance, requiring the acquisition of easements over private property for drainage. The biggest part of the job of carrying runoff out of built-up areas is done by the street system however, in spite of its inherent limitations. Figure 3 shows typical street pavement cross-sections.

AREA DESCRIPTION **SOUTH MOUNTAIN DRAINAGE**

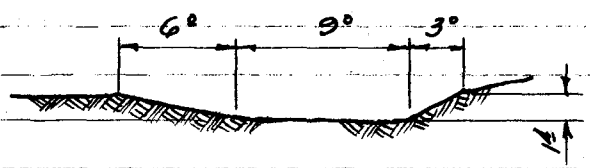
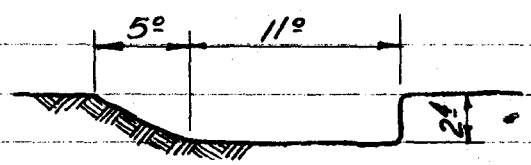
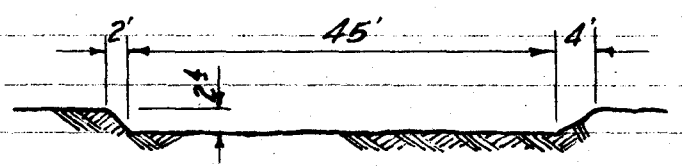
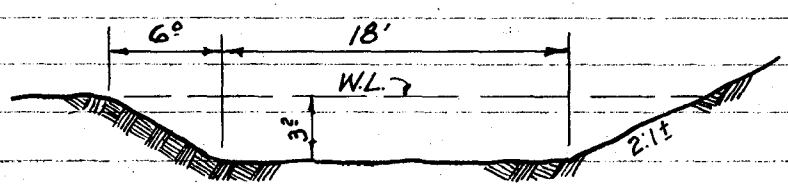
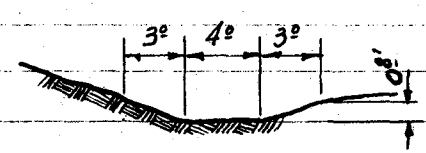
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth - Best
n·0.030 Earth - Brushy - Poor

Sta. or Location From-To	Waterway Description		Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p = Wet Per. r = A/p	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Near 48th St. Guadalupe Rd. Wash			0.025	20	78.5	2.91	19.5	1530
44th St. & High Line Canal		Ditch	0.025	30	20.3	2.4	25	508 @ 4.5'
		Pipe	0.012	30	3.1	0.5	13.5	42
41st St. S. of Baseline Road			0.025	27	132	3.47	24	3170
19th St. & Dobbins Road			0.025	17	59.5	1.54	11.2	665

1/12

AREA DESCRIPTION **SOUTH MOUNTAIN DRAINAGE**

n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth - Best
n·0.030 Earth - Brushy - Poor

Sta. or Location From-To	Waterway Description	Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p = Wet Per. r = A/P	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
12th. St. & Dobbins Road		0.025	9	18.9	1.03	6.2	117 Full
10th. St. & Dobbins Road.		0.025	11	32.4	1.70	9.7	314 Full
South of McNeil St. & 2nd. Ave.		0.025	11	115.2	2.18	11.8	1360 Full
Montezuma & S. Mount. Road		0.025	13	77.4	2.38	13.2	1020
1500' E. Alg. Wstrn. Canal from 27th. Ave.		0.030	.16	5.6	0.53	3.6	20.2

2/12

AREA DESCRIPTION **SOUTH MOUNTAIN DRAINAGE**

n·0.011 Big Concrete Culverts
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n·0.015 Street Paving
Concrete Lined Waterway
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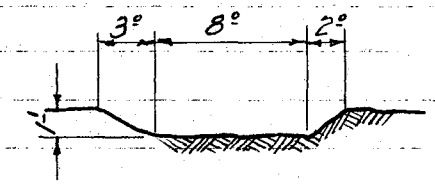
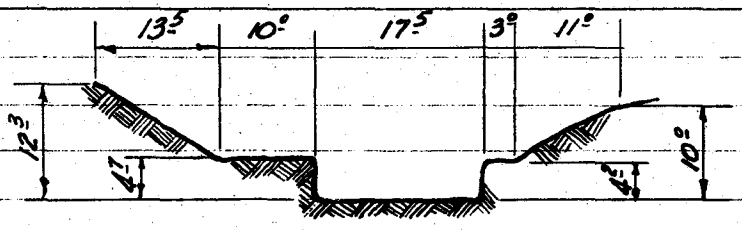
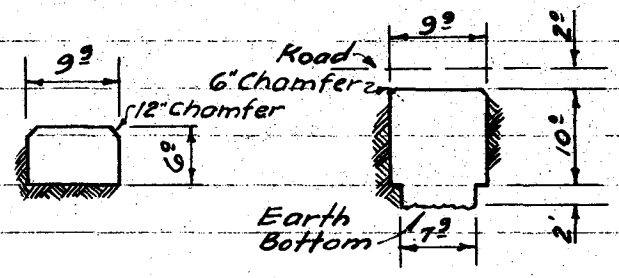
n·0.030 Earth - Brushy - Poor

Sta. or Location From-To	Waterway Description	Roughness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p = Wet Per. r = A/P	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
400' N. & 200' W. of S. End of 35 th. Ave		0.025	5	11.6	0.85	4.0	46.4

3/12

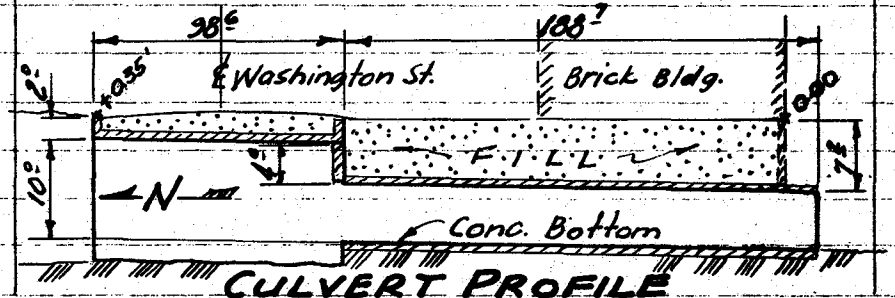
AREA DESCRIPTION **OLD CROSS CUT CANAL**

n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth·Best
n·0.030 Earth·Brushy·Poor

Sta. or Location From-To	Waterway Description	Roughness n	Slope Ft. Per 1000	Area Sq. Ft = A	p = Wet Per. $r = \frac{A}{P}$	Vel. Ft./Sec. = V	Quant. C.F.S. = Q
365' S. of Culvert @ Canal & Washington		0.030	3	164.1	3.33	6.0	985 @ 7'
Culvert at X-Cut Canal & Washington Street		0.012	5.4	59	1.9	19.6	1155 *

* Assume Culvert to have a head of 1.5' in addition to slope. Total Slope = 10.6 ft/1000.

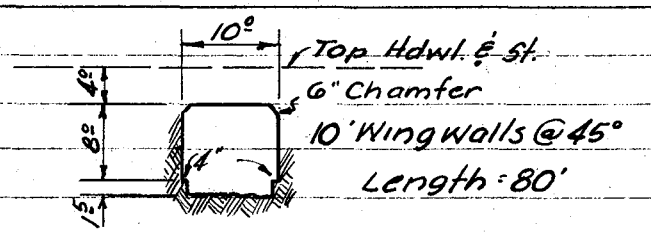
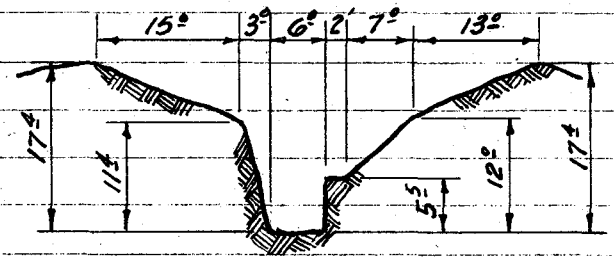
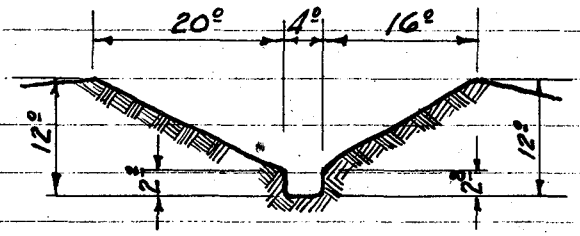
SOUTH END NORTH END



CULVERT PROFILE

AREA DESCRIPTION **OLD CROSS CUT CANAL**

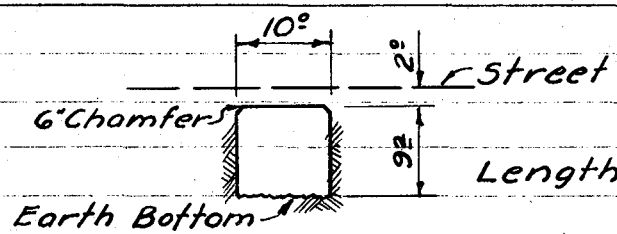
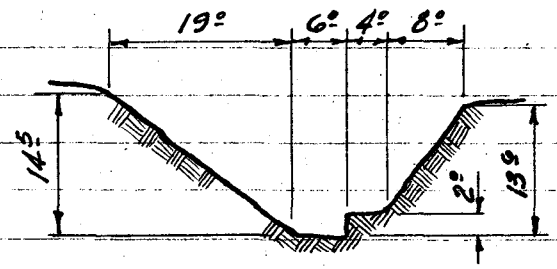
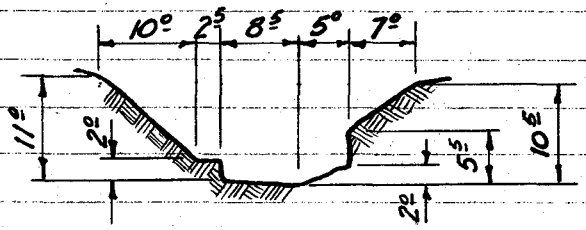
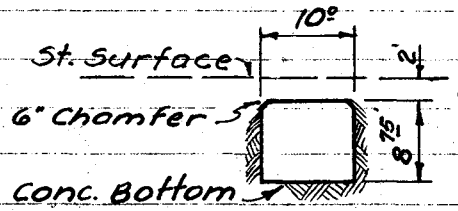
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth - Best
n·0.030 Earth - Brushy - Poor

Sta. or Location From-To	Waterway Description	Roughness n	Slope Ft. Per 1000	Area Sq. Ft. = A	p = Wet Per. r = A/p	Vel. Ft./Sec. = V	Quant. C.F.S. = Q
Box Culvert @ Van Buren St. & Canal		0.011	Level	95	2.44	13.9*	1320
	* Velosity Limited by a 3' Max Velosity Head or 13.9 ft./sec.						
Canal 300' N. of Van Buren Street		0.030	6	150	3.66	9.3	1400 @ 13.5'
Canal 0.45 Miles South of McDowell Road		0.025	8	111	3.08	12.8	1420 @ 9'

5/12

AREA DESCRIPTION **OLD CROSS CUT CANAL**

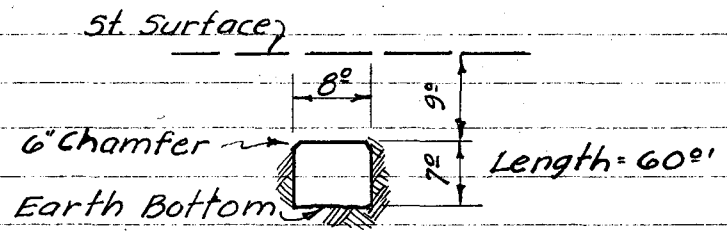
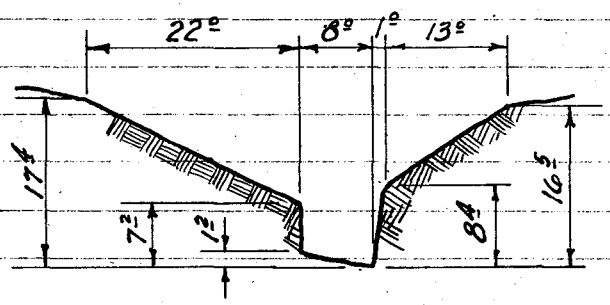
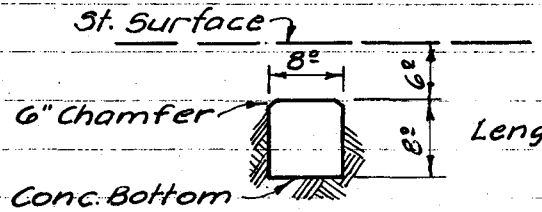
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth·Best
n·0.030 Earth·Brushy·Poor

Sta. or Location From-To	Waterway Description	Roughness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p=Wet Per. r=A/p	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Culvert @ McDowell Road		0.011	Level	91.8	2.37	9.6*	880
	*Limited by 1.5 ft. Vel. Hd. or 9.6 ft/sec.						
Canal @ Hubbell Street.		0.030	4	147.5	4.20	8.3	1225 @ 9º'
Canal @ Virginia Street		0.025	4	141	4.06	10.8	1520 @ 8º'
Culvert @ Thomas Road	 10º Wing walls @ 45º Length - 80⅝	0.011	Level	87.2	2.33	9.6*	835
	*Limited by 1.5 ft. Vel. Hd. or 9.6 ft/sec.						

6/12

AREA DESCRIPTION **OLD CROSS CUT CANAL**

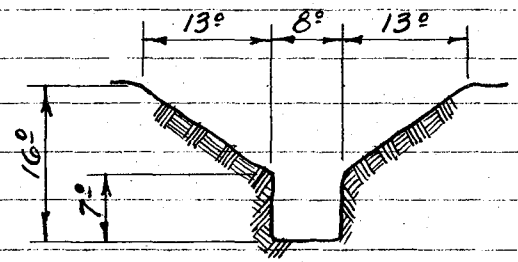
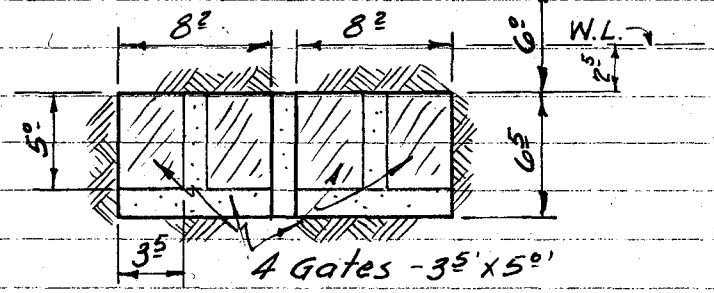
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21"ϕ Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth·Best
n·0.030 Earth·Brushy·Poor

Sta. or Location From-To	Waterway Description	Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p=Wet Per. $r=\frac{A}{P}$	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Culvert @ Osborn Road	 Length = 60''	0.011	Level	55.8	1.86	17.9 *	1000
	*Limited by 5' Vel. Head or 17.9 ft/sec.						
100' S. of Osborn Road Culvert		0.025	6.4	92.3	2.84	10.8	1000 @ 10'
Culvert @ Indian School Road	 Length = 51'	0.011	20	63.8	1.99	15.0*	960
	*Limited by 3.5' Vel. Hd. or 15.0 ft/sec.						

7/12

AREA DESCRIPTION **OLD CROSS CUT CANAL**

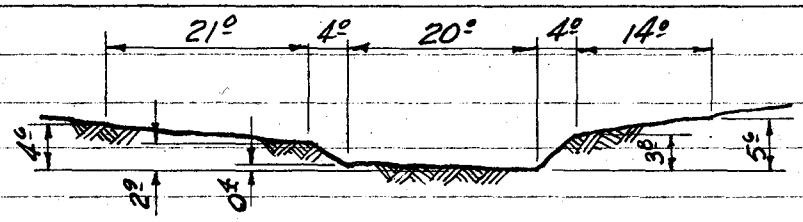
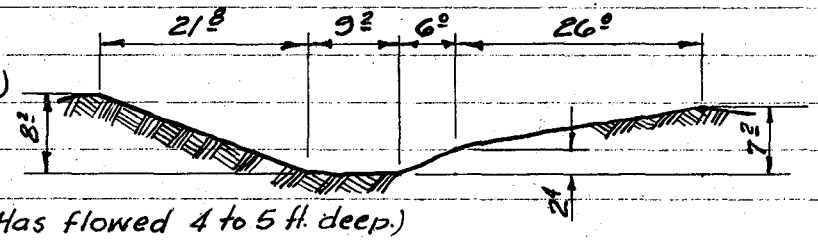
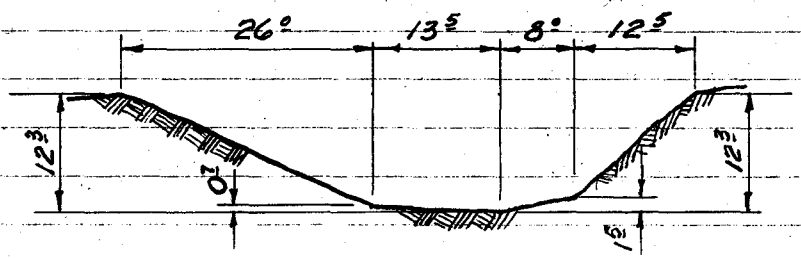
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth - Best
n·0.030 Earth - Brushy - Poor

Sta. or Location From-To	Waterway Description	Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p = Wet Per. r = A/p	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Canal 400' N. of Indian School Road		0.030	15	92	2.87	12.0	1100 @ 10'
Inlet to X-Cut at Arizona Canal	Top Hdwl. (1' over Road) 	$Q = 0.61 A\sqrt{2gh}$		$A = 17.5 \times 4$	$h = 5'$		
		$Q = 0.61 \times 70\sqrt{64.4 \times 5} =$					765

8/12

AREA DESCRIPTION **NORTH OF ARIZONA CANAL**

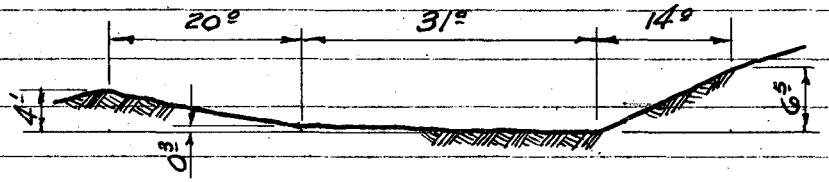
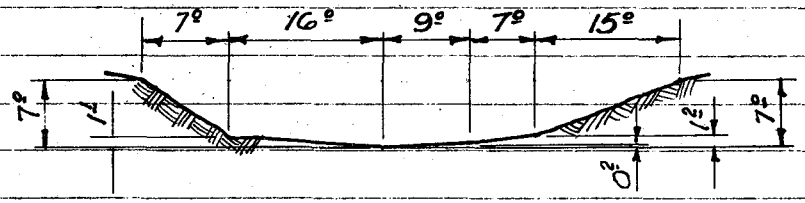
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth·Best
n·0.030 Earth·Brushy·Poor

Sta. or Location From·To	Waterway Description	Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p·Wet Per. r·A/P	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
50' E. of 40th. St. Near Missouri		0.030	9	94.8	2.43	8.3	790 @ 4°
Kitache Wash. (E. Br.) 50' W. of 35th. st. (Has flowed 4 to 5 ft. deep.)		0.025	18	103.2	2.52	16.8	1655 @ 5.9'
				218.4	3.47	20.4	4450 @ 7.2'
Kitache Wash (W. Br.) @ Bethany Home Rd. E. of 32nd. St.		0.030	15	91.7	2.70	13.2	1210 @ 4.9'
				242.5	4.96	18.7	4550 @ 8.9'

9/12

AREA DESCRIPTION **NORTH OF ARIZONA CANAL**

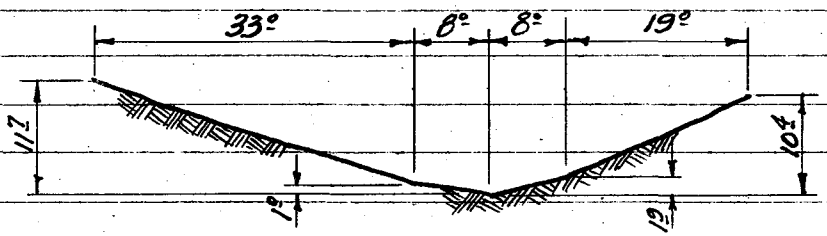
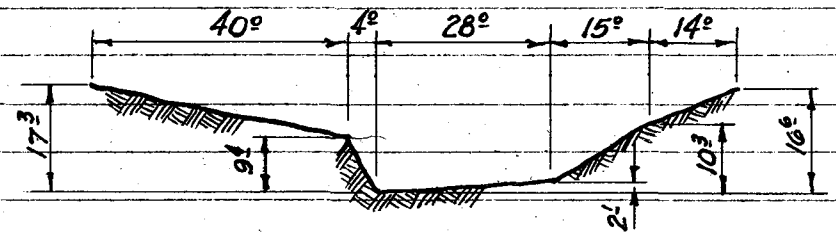
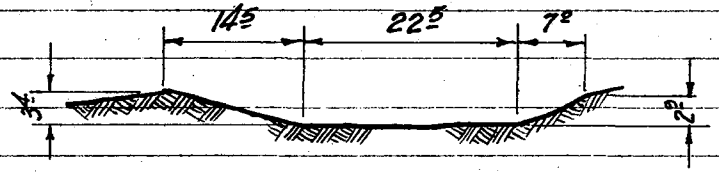
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth·Best
n·0.030 Earth·Brushy·Poor

Sta. or Location From·To	Waterway Description	Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p=Wet Per. r=A/p	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Wash Entering Canal @ Squaw Peak Rd.		0.025	10	118	2.32	11.8	1390 @ 3º'
Biltmore Dam Wash 1/4 Mile N. of Hotel		0.030	23	129	2.90	15.5	2000 @ 4º'

10/12

AREA DESCRIPTION **SUNNYSLOPE**

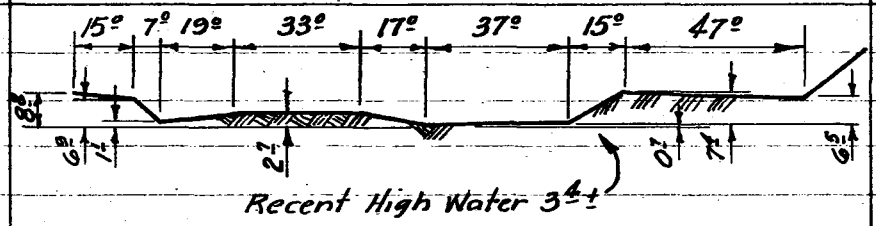
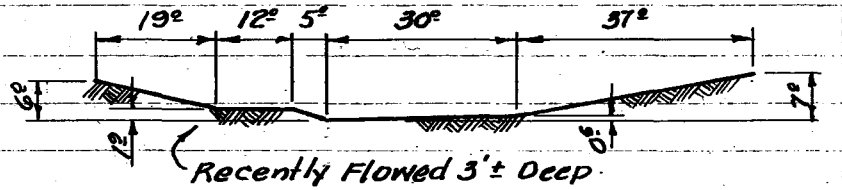
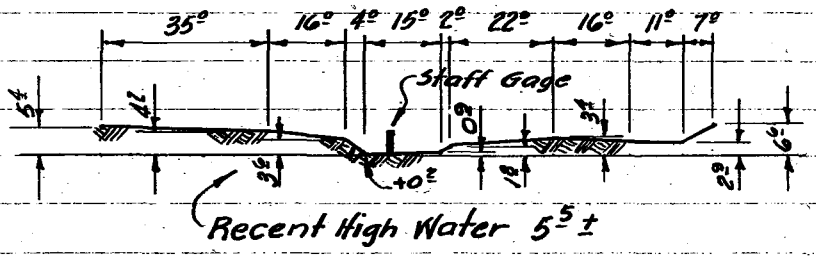
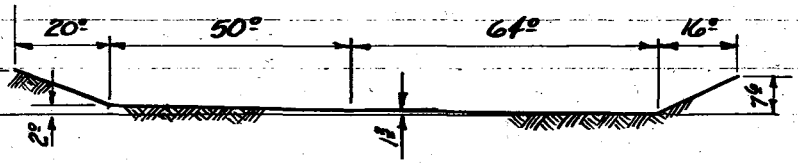
n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth - Best
n·0.030 Earth - Brushy - Poor

Sta. or Location From-To	Waterway Description	Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p = Wet Per. r = A/p	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Dreamy Drow @ Martin Drive		0.030	23	140.3	3.42	17.2	2420 @ 6°'
18th. St. 600' N. of Northern Exterded		0.025 35	23	161.4	3.84	25.0 17	4040 @ 6°' 2244
7th. St. Wash 1/4 Mi. E of 7th. St. at Butler Drive		0.025	5	90.5	2.21	8.0	725 @ 2°'

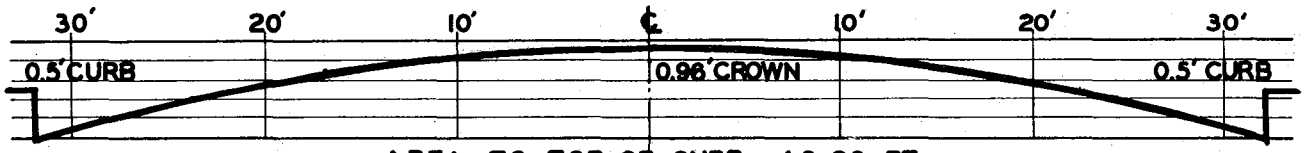
11/12

AREA DESCRIPTION **CAVE CREEK**

n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth · Best
n·0.030 Earth · Brushy · Poor

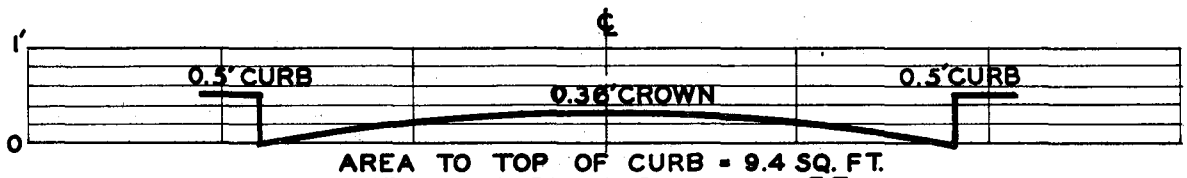
Sta. or Location From-To	Waterway Description	Roughness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p = Wet Per. r = A/P	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Greenway Rd. at N. End of Moonhill		0.030	6	210	2.28	6.6	1390 @ 3.4'
450' S.W. of 15th. Ave. (3/4 Mi. S. of Bell Rd.)		0.025	6	404	4.12	11.8	4760 Full
				137.5	2.12	7.56	1040 @ 3'
Staff Gage at Bell Road		0.030	7	242	1.92	6.2	1500 @ 5.4'
Cave Creek at 7th. St. Extended		0.025	10	881	5.95	19.5	17,200 Full

12/12



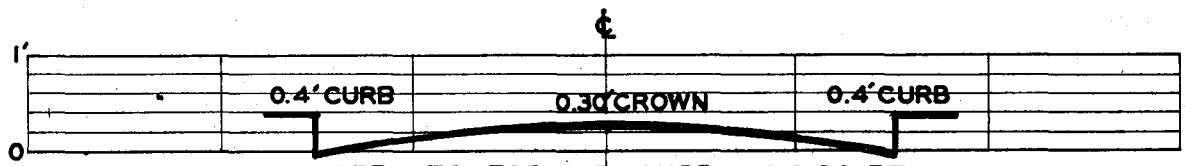
AREA TO TOP OF CURB = 4.8 SQ. FT.
WETTED PERIMETER = 20.3 FT.
HYDRAULIC RADIUS = 0.236 FT.

64 FT. ARTERIAL STREET



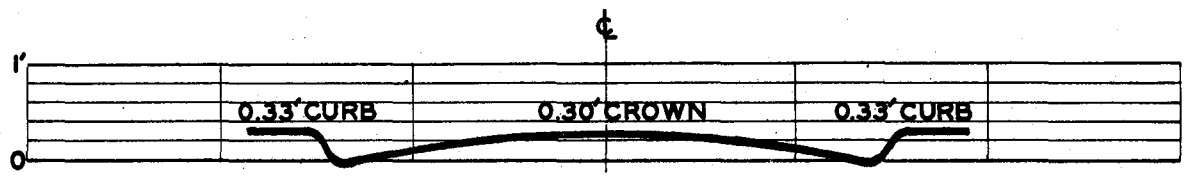
AREA TO TOP OF CURB = 9.4 SQ. FT.
WETTED PERIMETER = 37.0 FT.
HYDRAULIC RADIUS = 0.254 FT.

36 FT. RESIDENTIAL AND COLLECTOR STREETS



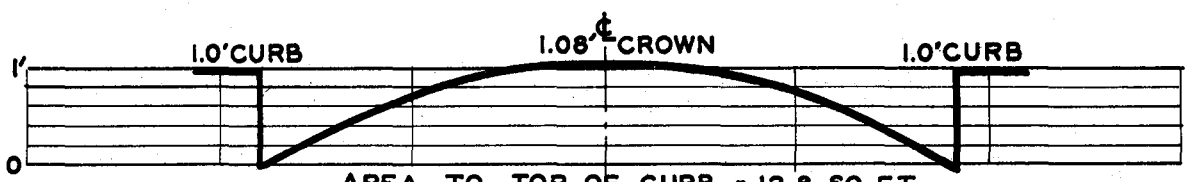
AREA TO TOP OF CURB = 6.0 SQ. FT.
WETTED PERIMETER = 30.8 FT.
HYDRAULIC RADIUS = 0.195 FT.

30 FT. RESIDENTIAL STREET



AREA TO TOP OF CURB = 4.1 SQ. FT.
WETTED PERIMETER = 31.2 FT.
HYDRAULIC RADIUS = 0.131 FT.

30 FT. RESIDENTIAL STREET WITH ROLL CURB



AREA TO TOP OF CURB = 12.8 SQ. FT.
WETTED PERIMETER = 38.0 FT.
HYDRAULIC RADIUS = 0.337 FT.

36 FT. STREET (OLDER RES. AREAS)

TYPICAL PAVED STREET CROSS-SECTIONS

FIG. 3

E. Storage Possibilities

Where the terrain is suitable an economical method of dealing with flood waters is to impound them and release them slowly after the storm is over. This is presently the practice at Cave Creek Dam which is designed for a 30,000 second foot maximum inflow and provides for a controllable outflow up to 1,600 second feet. The total storage available is 13,500 acre feet. [1] Cave Creek Dam has probably paid for itself several times in damages saved and it is constantly becoming more valuable.

The question of the possibility of applying the storage principle at other points has been investigated by the Soil Conservation Service. In a preliminary study made in March of 1954 this agency considered 17 dam sites in the Phoenix Mountains. The largest of these possible projects was the damming of Cave Creek at Moon Hill. Two dams would be required, one across the present channel at the north end of Moon Hill and another across an old channel between Moon Hill and Shaw Butte. The reservoir site would comprise 1580 acres. Other sites were investigated on Echo Canyon Wash, Katiche Wash, the Biltmore washes, Dreamy Draw, and the washes in the Sunnyslope area. Reservoir capacities obtainable are in the range from 25 to 375 acre feet with a cost estimated in 1954 to average about $600 per acre foot of capacity. The lower Cave Creek reservoir was of a different class having a capacity of 8800 acre feet provided at an estimated cost of $480,000, or about $54 per acre foot.

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[1] Statistics from Vic Householder of Phoenix, Resident Engineer on the Cave Creek Dam project.

There are few good natural reservoir sites. The channels in the mountainous areas are steep and narrow requiring high dams yielding little storage. The foothill zone is practically non-existent, the mountain washes forming broad alluvial fans immediately where they leave the rocky slopes. The few exceptions capable of development are Guadalupe Wash at the city gravel pit in the east end of South Mountain Park, and the washes leaving the park at South 7th Street and South Central Avenue where again there are gravel pits affording good storage possibilities. In the northern part of the valley where the Soil Conservation Service work was done, the few naturally good sites are rapidly being rendered unusable by suburban development and rising land prices. The cost of the Cave Creek reservoir site for example was estimated to be $50 per acre in 1954 but several large tracts were recently sold in this area for prices from $700 and over per acre.

F. Artificial Barriers

The natural pattern of drainage channels has been changed to a considerable extent by the works of man. This has been done in spite of the principle that the one who diverts drainage from its normal channel makes himself liable for damages suffered by others because of the diversion.

The canal system, especially the Arizona Canal on the north and the Highline Canal on the south intercept many natural channels. At the larger washes a spillway is usually provided to permit excessive floods to pass over the canal. This is not true for many minor channels, nor do the spillways provided pass all the water that comes into the canal from the uphill side. There has been little or no discharge of storm water from

the spillways on the south bank of the Arizona Canal in the Phoenix drainage area since 1943 but there has been runoff into the canal on many occasions since that time. There is no doubt that the canals do act as barriers to divert a great deal of runoff from its natural channels.

Street pavements also have this effect, especially in the flat areas where channels were indistinct to begin with. Some street pavements have crowns as much as a foot above the gutter, an appreciable amount where the general slope of the land is only about ten feet per mile. The Santa Fe railroad embankment is also a distinct barrier in some places although not nearly so much so as it was at one time.

III HYDROLOGY

"Are there any among the vanities of the Gentiles
that can cause rain?"

Jeremiah 14:22

Whether or not it is possible to cause rain by "seeding" the clouds is still the subject of debate in meteorological circles. There is general agreement however, that rain making is not possible if there is not ample moisture already present in the atmosphere. So far man has not devised a way to assist the Creator in bringing this necessary condition about.

Neither has it been possible for engineers to look into the future to measure the severity of its storms. Not being endowed with prescience, the designer who attempts to provide for future acts of nature must fall back on a resource he usually does have, thanks to the dedicated persons who provide such things, a record of what has already happened. Then, by leaning heavily on the principle that the past foreshadows the future, he attempts to winnow from this record harvest of many years a few kernels of what he believes to be truth and calls them patterns, trends, or if he is foolhardy, laws. He uses these to guide him in his work, a poor substitute for the more exact knowledge of the structural or machine designer, yet all he is ever likely to have.

It is the purpose of this portion of this report to set forth these guides, to show how they were derived, and to illustrate how they are used

later in computing the storm water flows for which a drainage system should be designed.

A. Precipitation

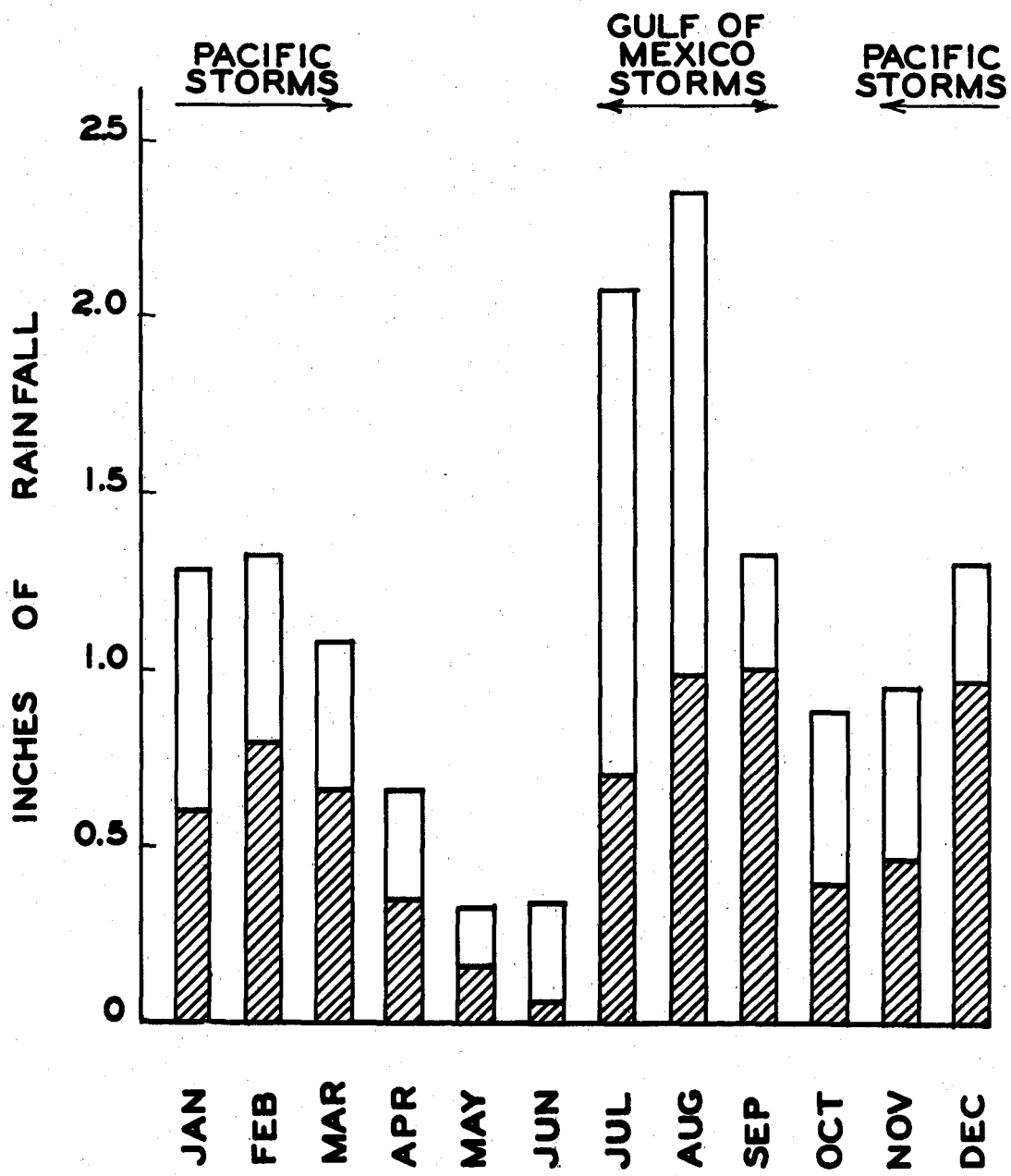
By the standards of other regions Arizona has very little rainfall, yet it does have two relatively rainy periods in the year, clearly shown in Figure 4 in which average monthly rainfall is plotted.[1] The two rainy periods are characterized by storms of distinctly different type. The dry transition periods are not always completely devoid of rainfall although they often are. Storms occurring during these periods may be of either of the two types but they are generally weak both in intensity and total precipitation.

Winter storms occur from November through March. These storms yield about half the total annual rainfall. They are usually of low intensity and often last for several days. The moisture comes from the Pacific in low lying stratiform clouds.

During July, August, and September air currents bring warm, moist air from the Gulf of Mexico. Mountain ranges and cold air fronts act on these streams to produce thunderstorm conditions characterized by the towering cummulus clouds seen almost daily this time of year. These storms often produce rainfall of high intensity, short duration, and sharply limited areal extent.

Storm drainage systems are designed to conduct water away rather than to store it. For this reason a storm of high intensity imposes more severe demands on the drains than a more gentle storm even if the latter results in

1. L.R. Jurwitz, "Rainfall in Arizona" Bulletin of the Arizona Sewage & Waterworks Assn. 1952, page 104



AVERAGE MONTHLY RAINFALL
SHADED BARS FOR CITY OF PHOENIX
UNSHADED BARS FOR STATE OF ARIZONA

FIG. 4

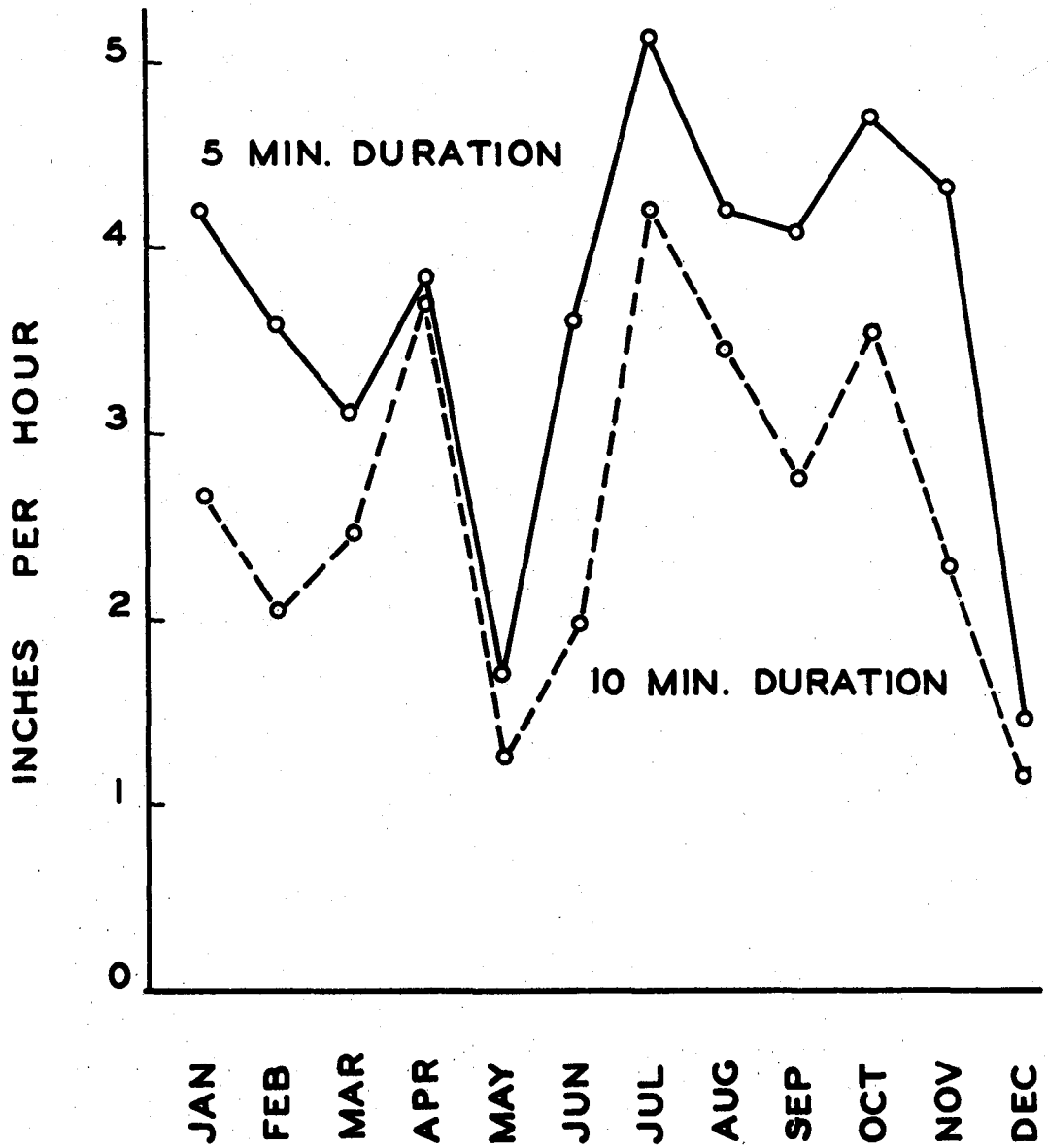
greater total rainfall. Figure 5 shows that the summer storms are considerably more intense than winter storms. Consequently this report will concern itself primarily with the summer thunderstorm.

1. Rainfall Intensity

The highest intensity of rainfall ever recorded at the Phoenix Post Office Weather Bureau station occurred on July 26, 1936 when 0.43 inches of rain fell between 7:38 and 7:43 p.m., a rate of 5.16 inches per hour. The record for the most rain in a ten minute period was set between 2:12 and 2:22 a.m. on July 26, 1952 when 0.70 inches fell, a rate of 4.20 inches per hour. Such high rates do not (in this location) continue for very long, there being an inverse relation between rainfall intensity and storm duration. This is shown in Figure 6 which represents the experience of a 52-year period at the Phoenix Post Office.

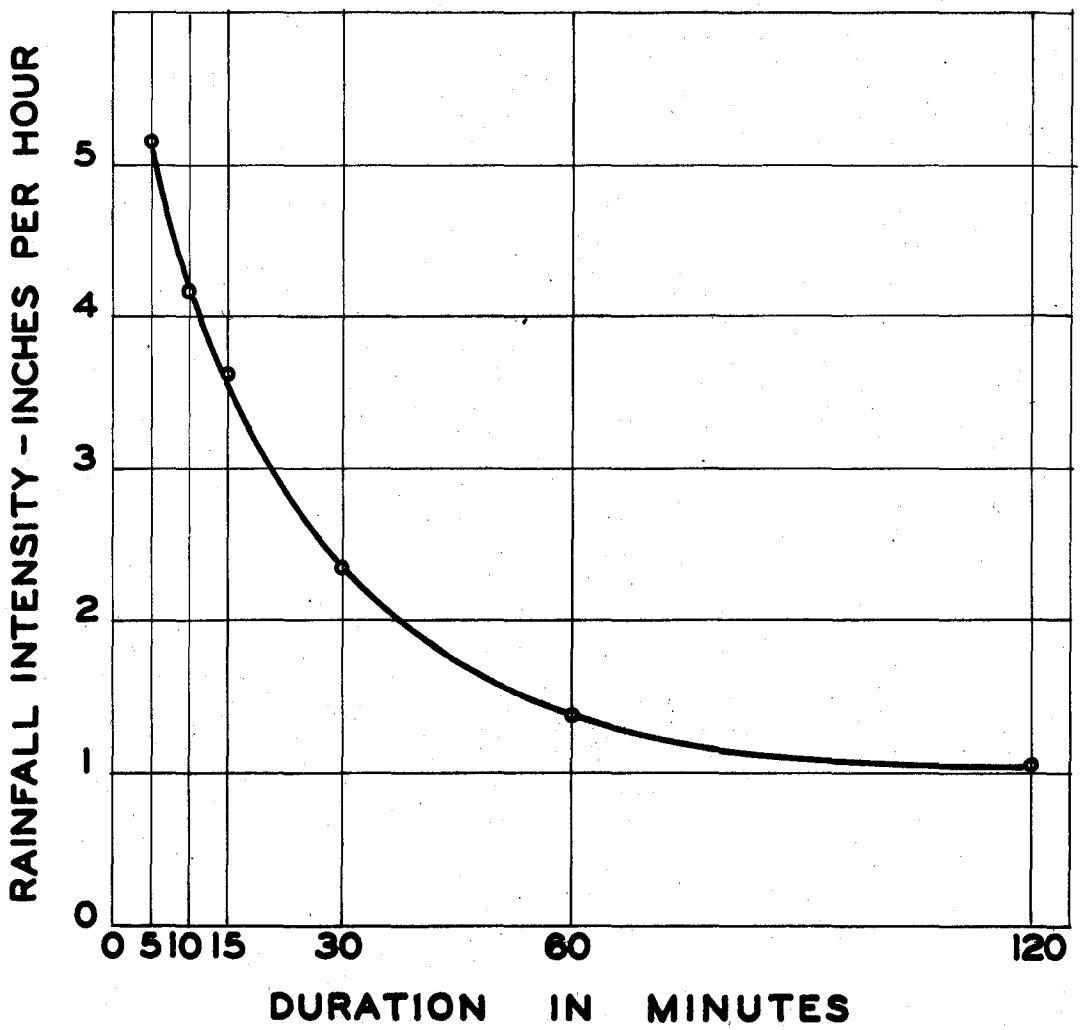
Figure 6 also indicates that there seems to be a limitation on the rain producing capacity of the atmosphere as far as rates are concerned. It is not known just what the limiting factors are but they would seem to include such things as the distance from the sea by possible paths of moisture-bearing air currents, depths of such currents as affected by atmosphere depth and structure and by ground elevation, temperature differentials, and probably other factors of lesser influence and more difficult to evaluate but probably just as general in effect as those mentioned. The net effect of all these factors is to produce a variation of rainfall intensity with geographical location. Maps have been prepared by the Weather Bureau showing this variation.[1] These maps indicate that for the relatively flat lands of the

1. Weather Bureau Technical Paper No.24 Rainfall Intensities for Local Drainage Design in the United States Part II



MAXIMUM RECORDED RAINFALL INTENSITIES
BY MONTHS AT PHOENIX POST OFFICE
PERIOD 1903 - 1955

FIG. 5



RELATION BETWEEN MAXIMUM RECORDED RAINFALL INTENSITY AND DURATION FROM PHOENIX WEATHER BUREAU RECORDS PERIOD 1903 - 1955

FIG. 6

lower Salt River Valley the variation is small and gradual. It appears quite proper therefor to predicate design intensities for local storm drainage work upon the records of the Phoenix Post Office Weather Bureau Station.

Storms of great intensity do not occur every year. The average random 20-year period will contain more severe storms than the average random 2-year period. In other words, the storm of intensity that occurs on the average of once in 20 years is more severe than on which occurs on the average of every other year. This point is of interest because it is generally not practical to design for the maximum possible storm. The question then arises how great is the risk if the design is for some lesser storm? The risk of flooding is of course directly related to the frequency of occurence of storms greater than the capacity of the drainage system. The relation between rainfall intensity, duration, and frequency of occurence have been studied by the Weather Bureau for many localities including Phoenix.[1] Figures 7, 8, and 9 show these relations as developed using statistical methods by the Weather Bureau and others. Figure 9 is the family of curves used subsequently in the computation of storm flows.

1. Weather Bureau Technical Paper No.25 Rainfall Intensity Duration Frequency Curves

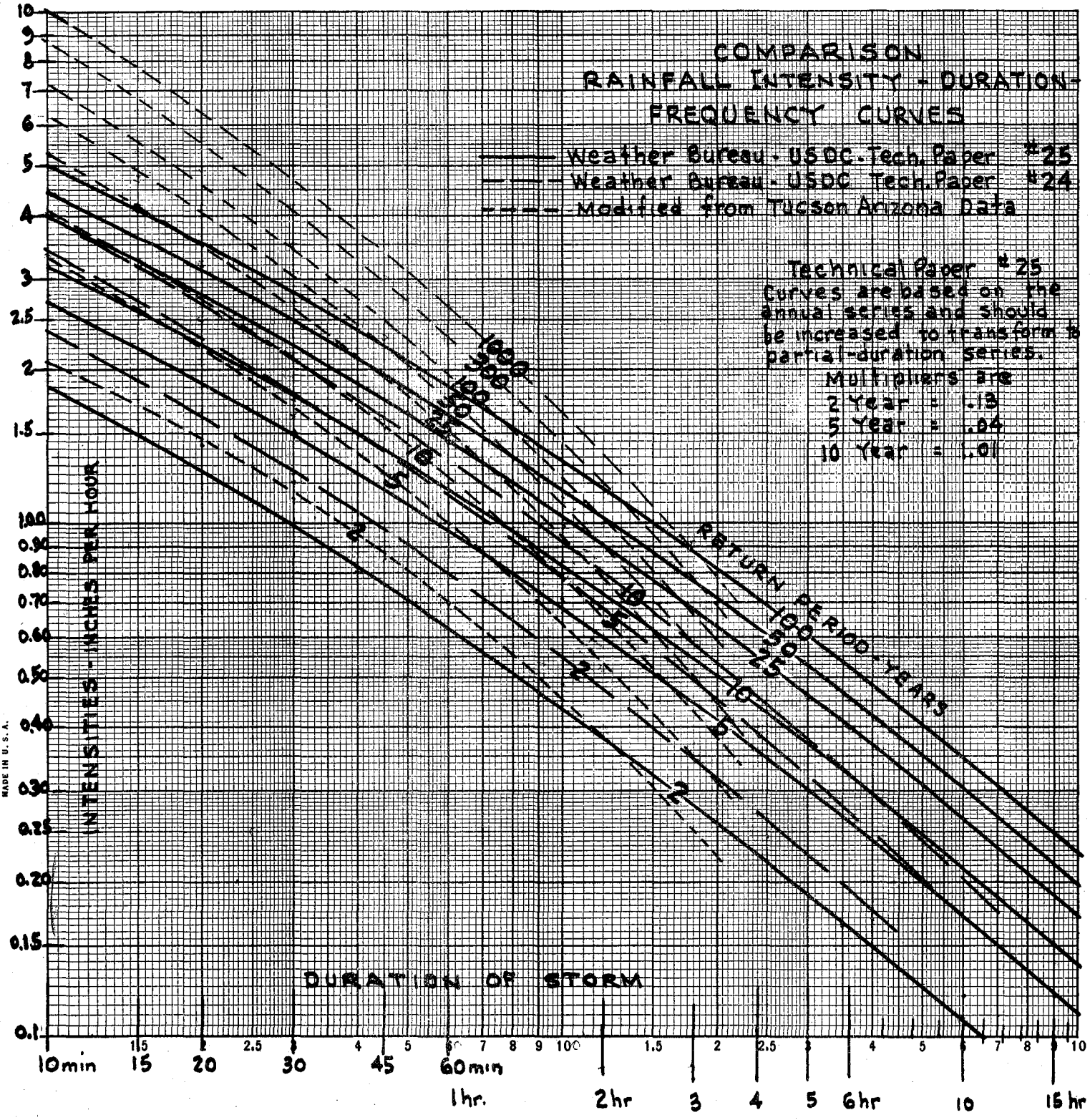

COMPARISON
RAINFALL INTENSITY - DURATION - FREQUENCY CURVES
Weather Bureau - USDC. Tech. Paper #25
Weather Bureau - USDC Tech. Paper #24
Modified from Tucson Arizona Data
Technical Paper #25
Curves are based on the annual series and should be increased to transform to partial-duration series.
Multipliers are
2 Year = 1.13
5 Year = 1.04
10 Year = 1.01
INTENSITIES - INCHES PER HOUR
RETURN PERIOD - YEARS
DURATION OF STORM
10min
15
20
30
45
60min
1hr.
2hr
3
4
5
6hr
10
15hr
359-110 KEUFFEL & ESSER CO.
Logarithmic, 2 X 2 Cycles.
MADE IN U.S.A.

FIG. 7

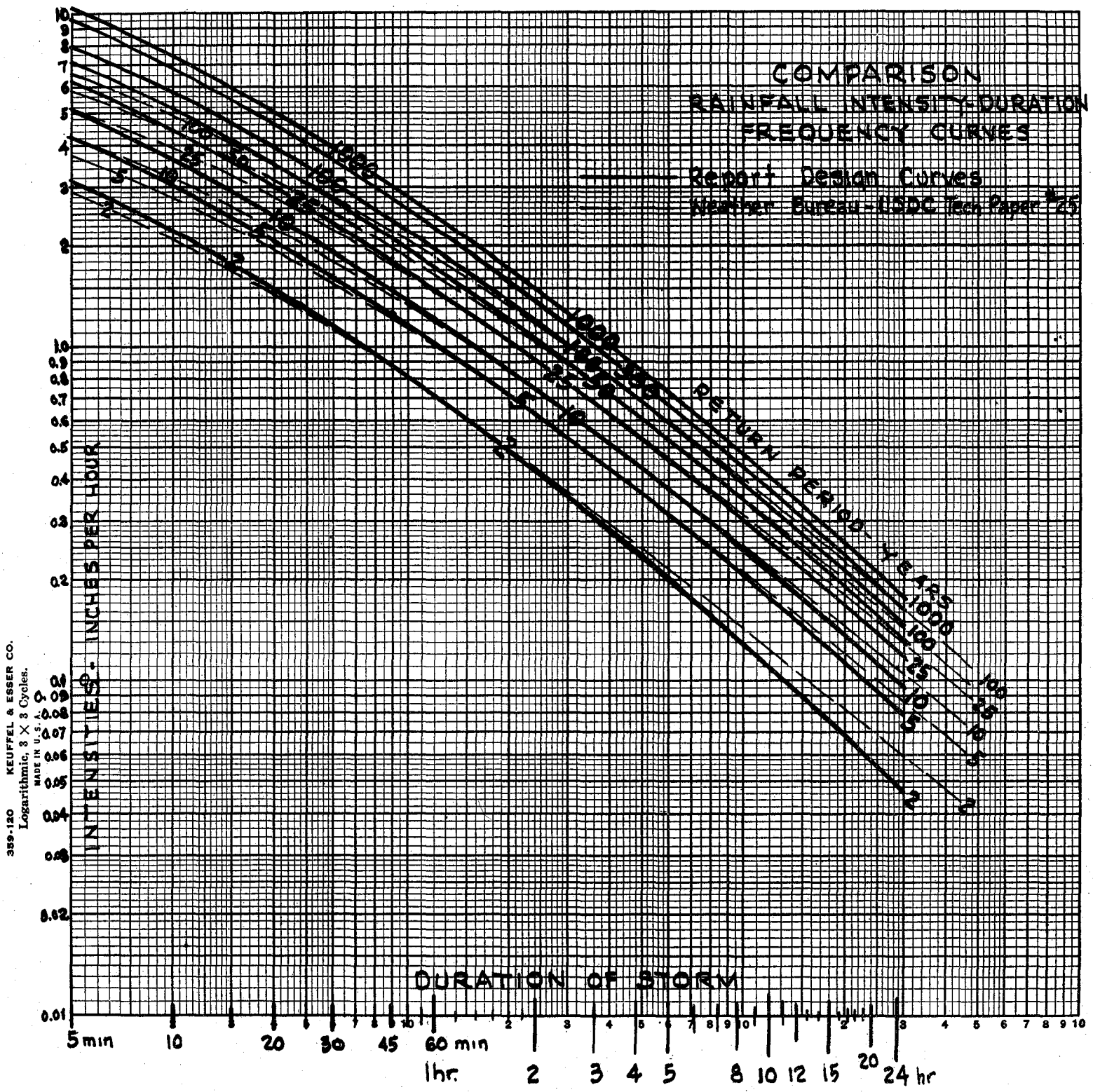

COMPARISON
RAINFALL INTENSITY-DURATION
FREQUENCY CURVES
Report Design Curves
Weather Bureau-USDC Tech Paper #25
RETURN PERIOD
INTENSITIES - INCHES PER HOUR
DURATION OF STORM
5 min
10
20
30
45
60 min
1hr.
2
3
4
5
8
10
12
15
20
24 hr
KEUFFEL & ESSER CO.
Logarithmic, 3 X 3 Cycles.
MADE IN U.S.A.

FIG. 8

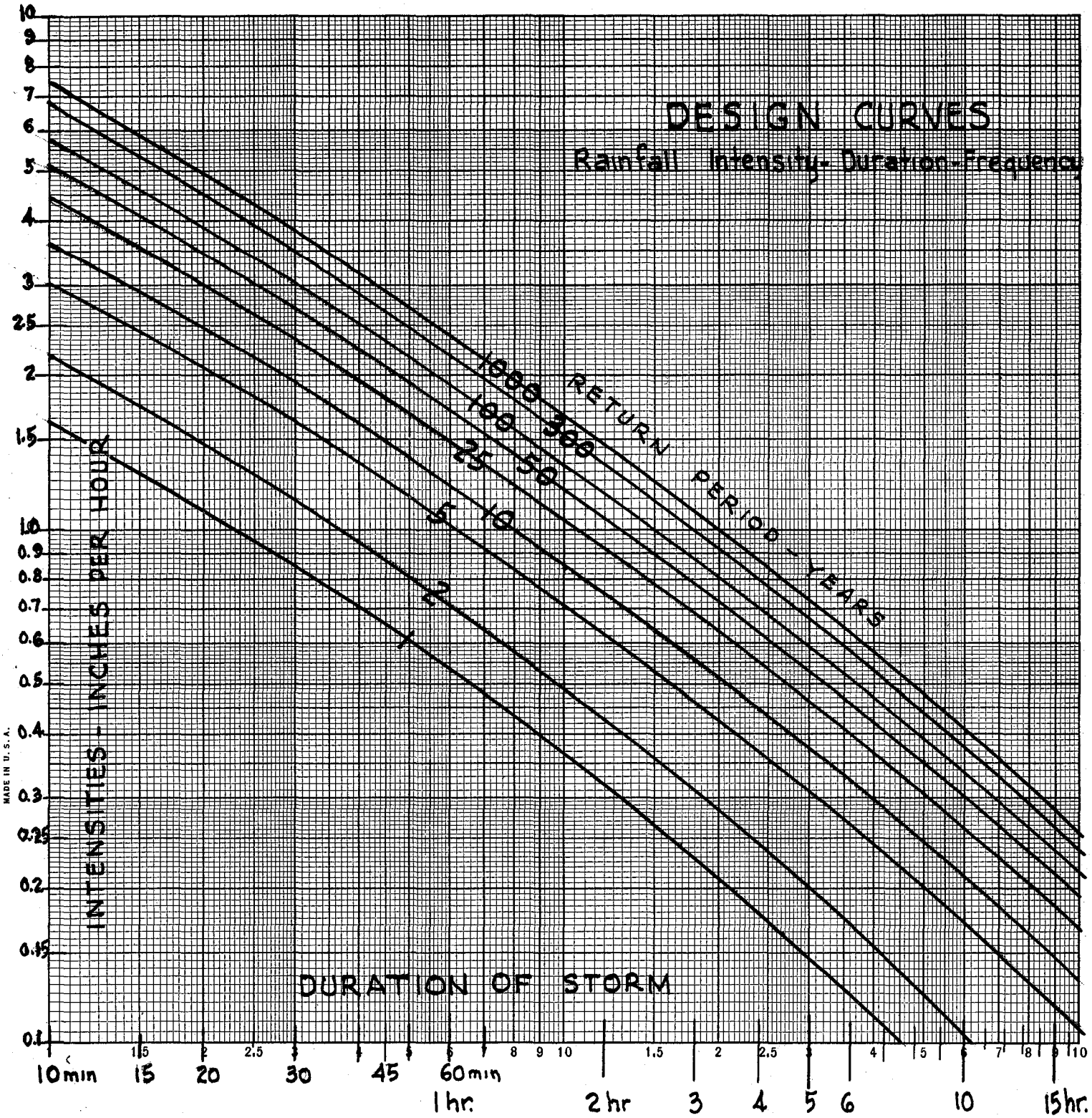

DESIGN CURVES
Rainfall Intensity-Duration-Frequency
RETURN PERIOD - YEARS
1000
500
100
50
25
10
5
2
1
INTENSITIES - INCHES PER HOUR
DURATION OF STORM
10 min
15
20
30
45
60 min
1 hr.
2 hr
3
4
5
6
10
15 hr.
359-110 KEUFFEL & ESSER CO.
Logarithmic, 2 X 2 Cycles.
MADE IN U.S.A.

FIG. 9

2. Amount of Rainfall

It has been previously pointed out that intensity, or rate of rainfall is of primary importance and that total amount of rainfall per storm is not of direct interest in designing the average municipal storm drainage system. It is only when the system utilizes storage to prolong the runoff period and reduce peak flows that total storm rainfall becomes a factor.

While runoff rates to be expected in the Phoenix area may seem very large, the total quantities of runoff are surprisingly low. This is due to the short duration of the storms which produce the most intense rainfall. The storm of August 3, 1943 for example was estimated to produce a maximum runoff rate of 30,000 cubic feet per second into the Arizona Canal but the total runoff was probably less than 10,000 acre feet.[1]

In the mountainous and undeveloped areas it may be possible to obtain dam and reservoir sites to take advantage of this fact and provide flood protection at a relatively low cost. In the city however, the topography is such that there would be no suitable storage sites even if the state of development were not already so far advanced as to make the cost prohibitive.

---

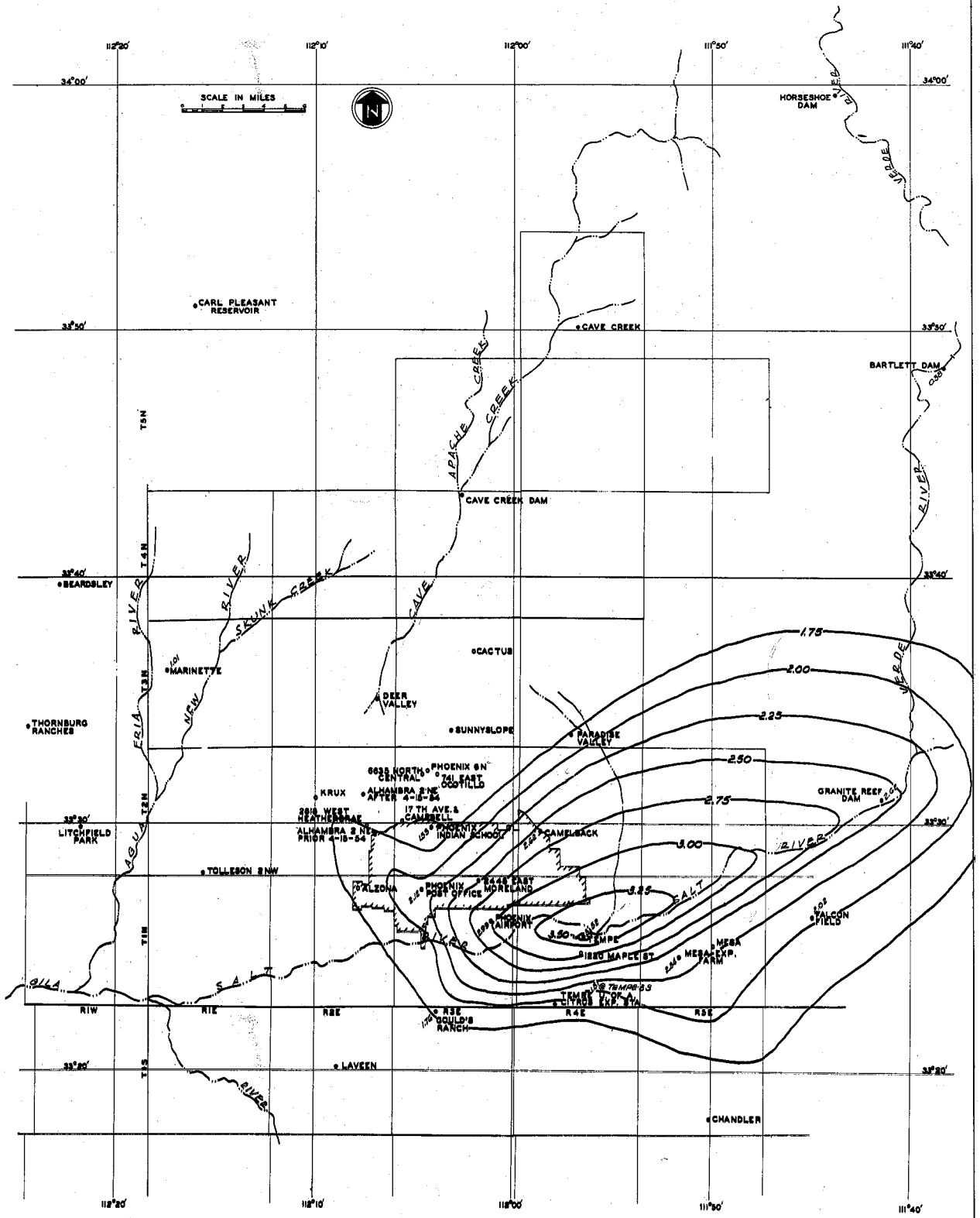
1. T. R. Neiswander, Report to Gen. Mgr. Lawson, SRVWA, dated Aug. 30,1943

3. Areal Extent of Storms

The phrase "scattered showers" is often used in summertime weather forecasts. It is often possible to see several intense downpours occurring simultaneously with wide spaces of sunlit landscape intervening. Thunderstorms are typically of small areal extent and when large areas receive rain from such storms it is because there are very many of them, perhaps enough that several may pass over any given point on the earth's surface. Isohyetal maps plotted for thunderstorms show many isolated points of high rainfall separated by relatively large areas receiving much less. Winter storms show much less variation in rainfall and that variation which does occur is more due to ground elevations than to atmospheric conditions.

A study has been made of rain gauge readings in the Phoenix area for several local storms. Total rainfall reported has been plotted in Figures 10 to 16. The area inside each closed isohyetal line in these figures was measured by planimeter. The relation between the area receiving rainfall and the average total depth of rain for that area was plotted for these storms in Figure 17. The relation in Figure 18 is from a curve used by the Corps of Engineers in a report entitled Interim Report on Survey-Flood Control for Tucson, Arizona, dated Nov. 20, 1945. It is the curve that will be used hereinafter in computing storm runoff rates.

STORM OF AUGUST 3, 1943



LEGEND
- ● WEATHER BUREAU STATION
- ○ COOPERATIVE STATION
- PHOENIX CITY LIMITS

FIG. 10

STORM OF AUGUST 15, 1943

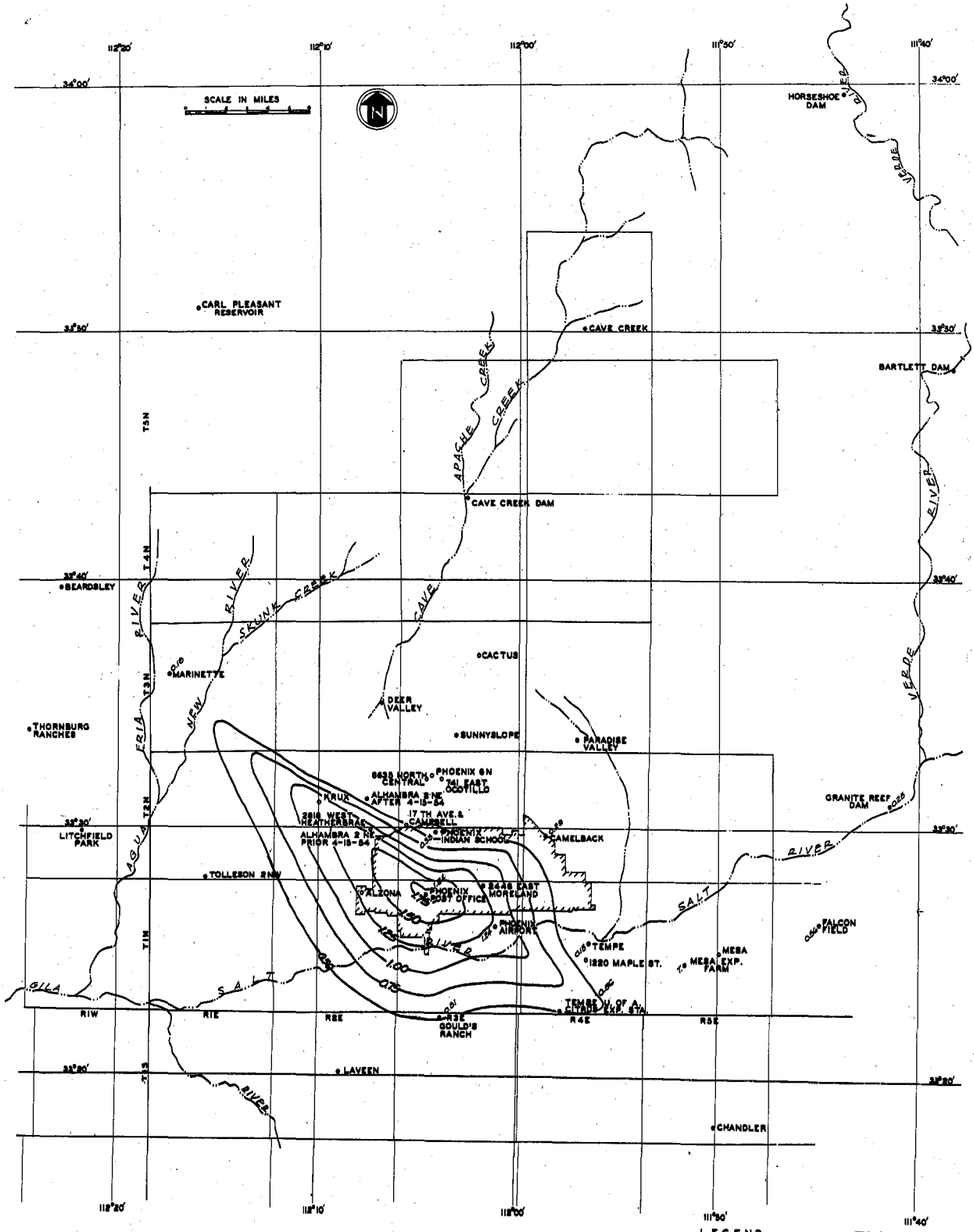


FIG. 11

STORM OF APRIL 19, 1951

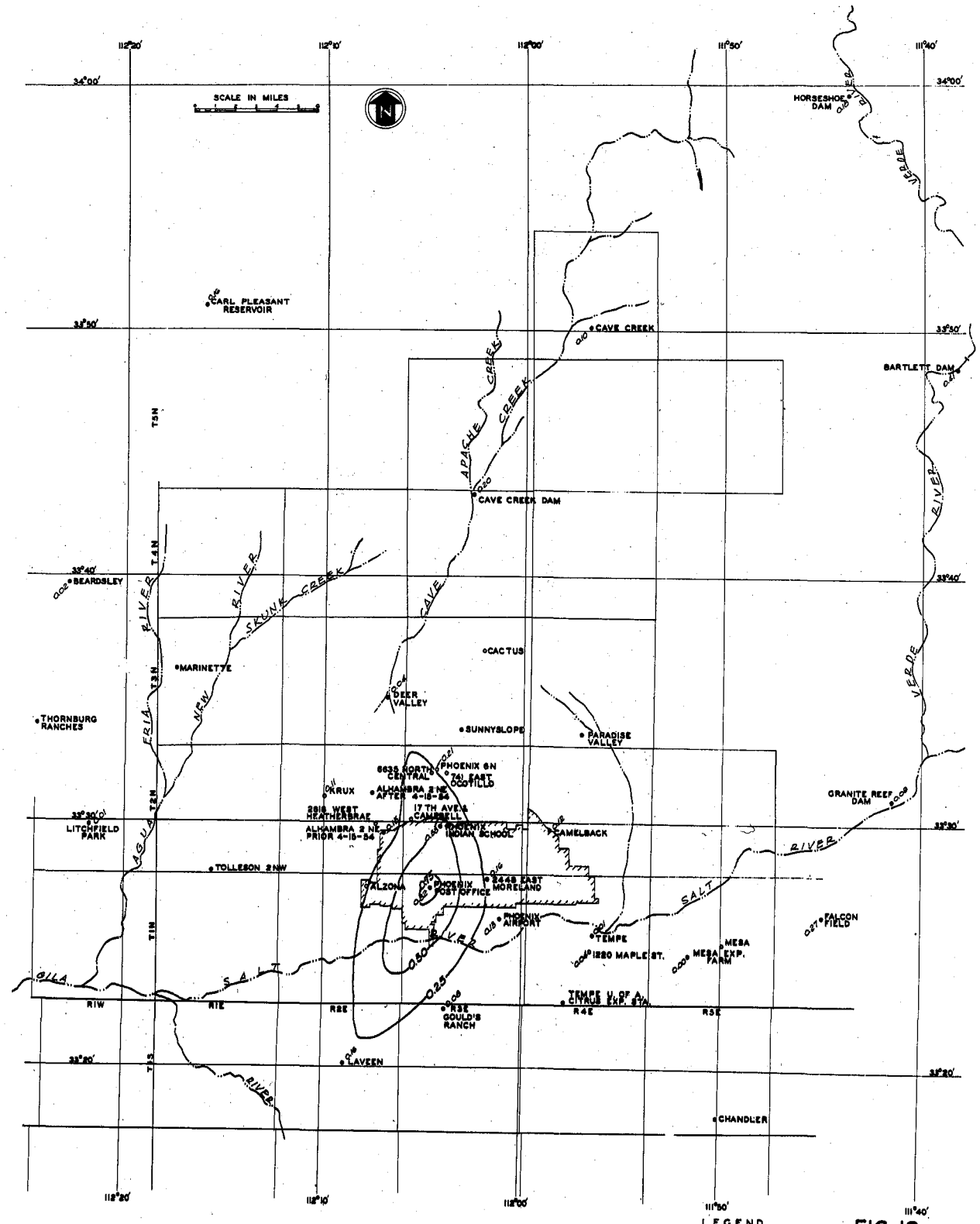


FIG. 12

STORM OF JULY 6, 1952

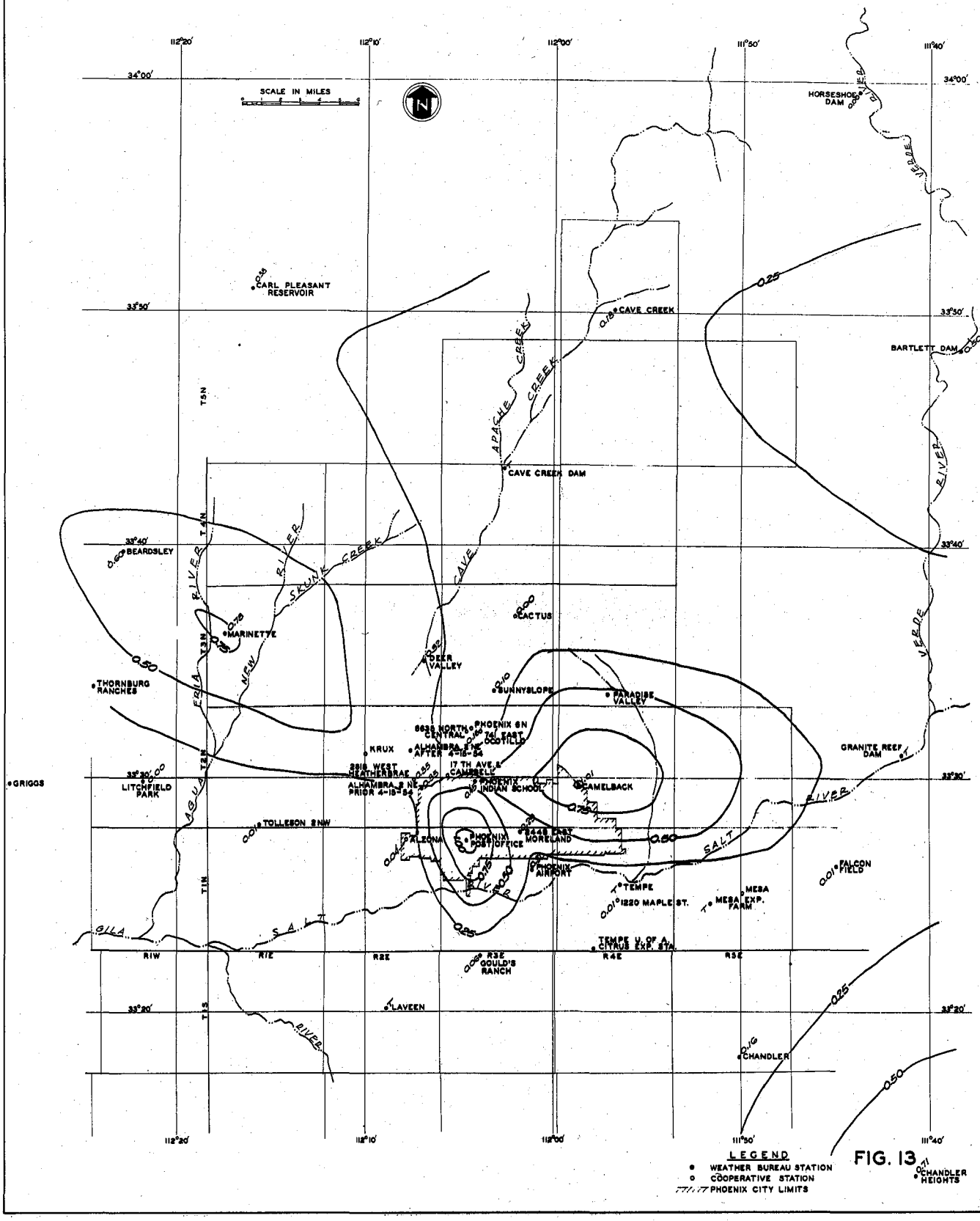
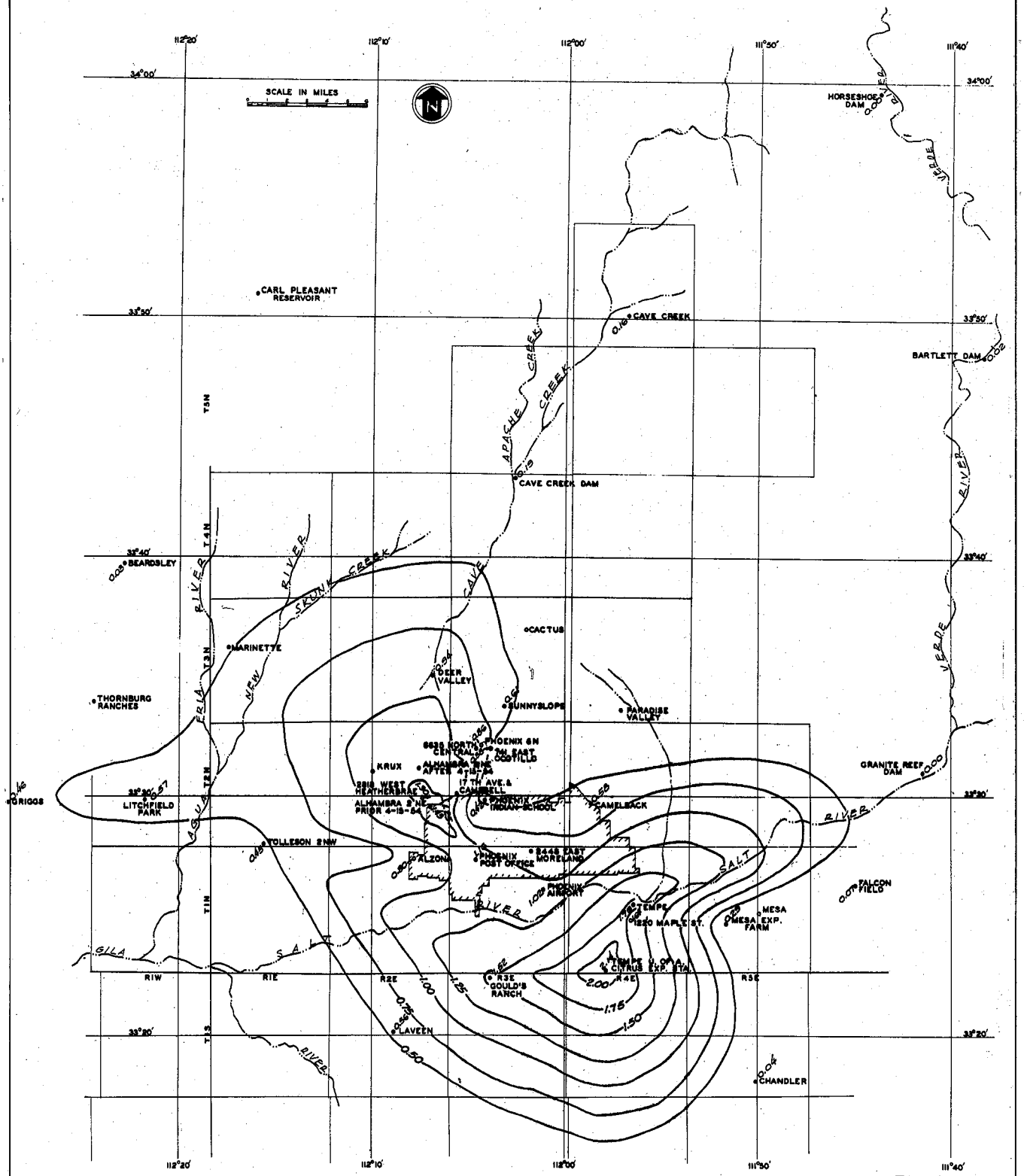


FIG. 13

STORM OF AUGUST 24, 1952



LEGEND

- ● WEATHER BUREAU STATION
- ○ COOPERATIVE STATION
- ////// PHOENIX CITY LIMITS

FIG. 14

STORM OF JUNE 12,1955

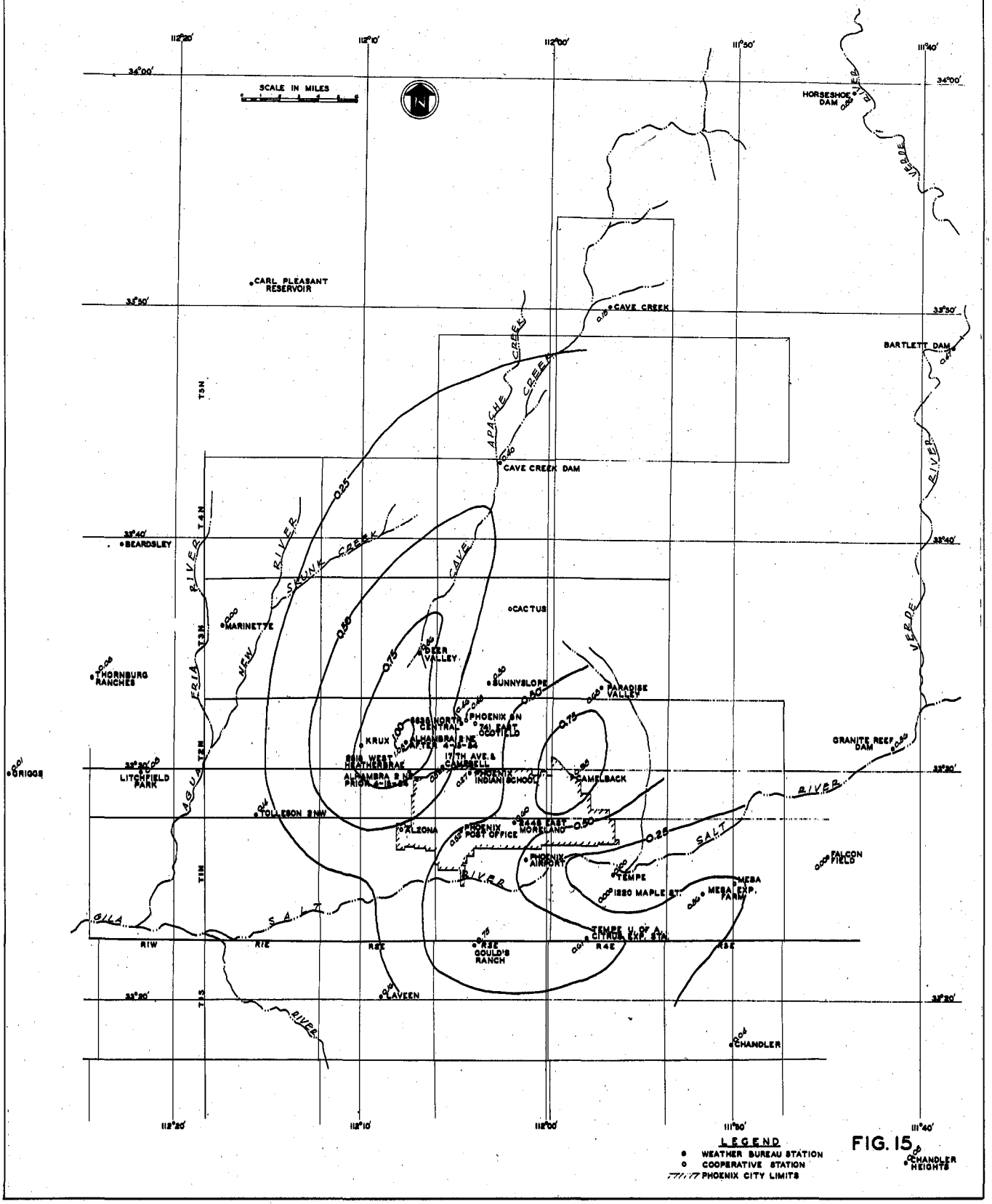
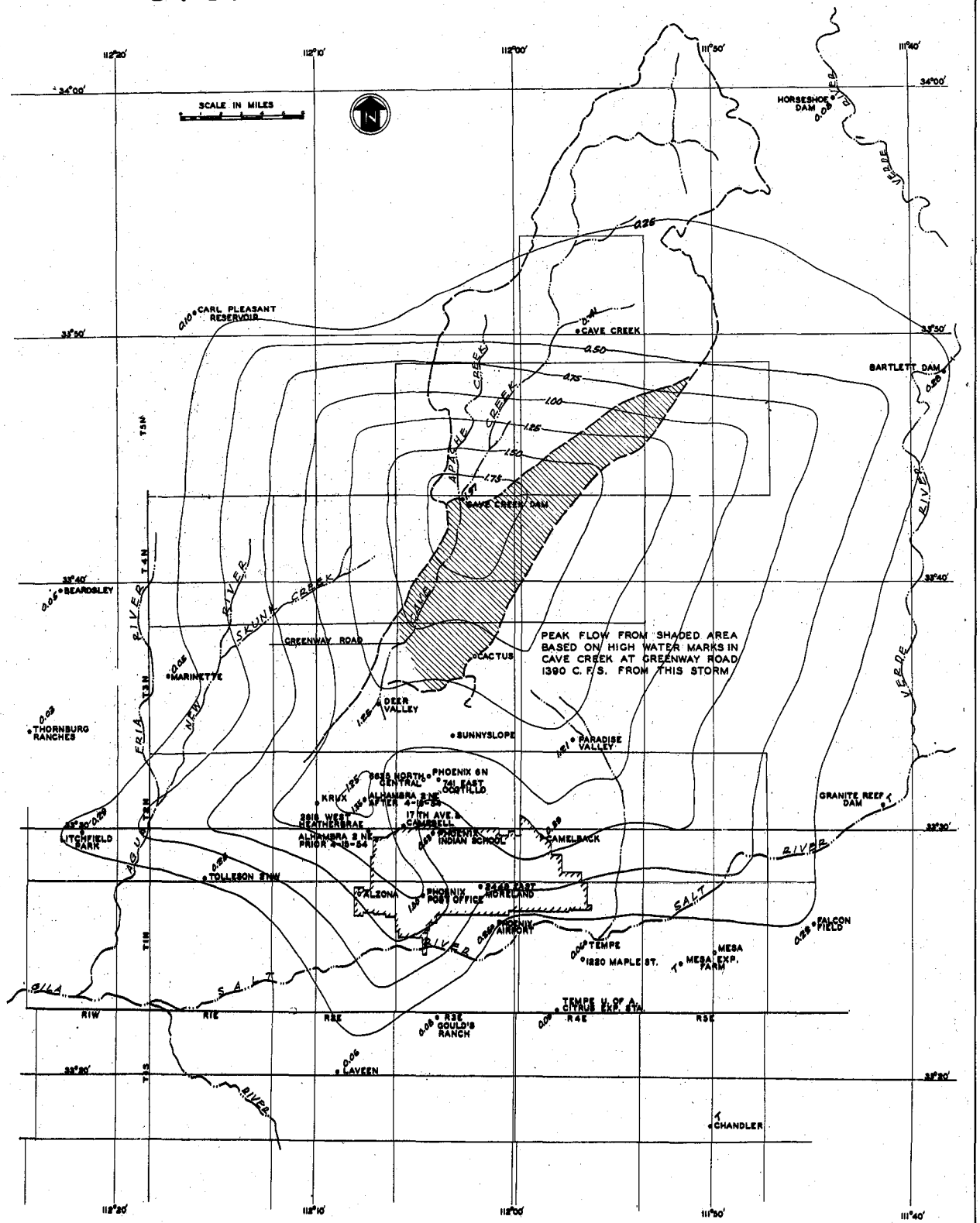


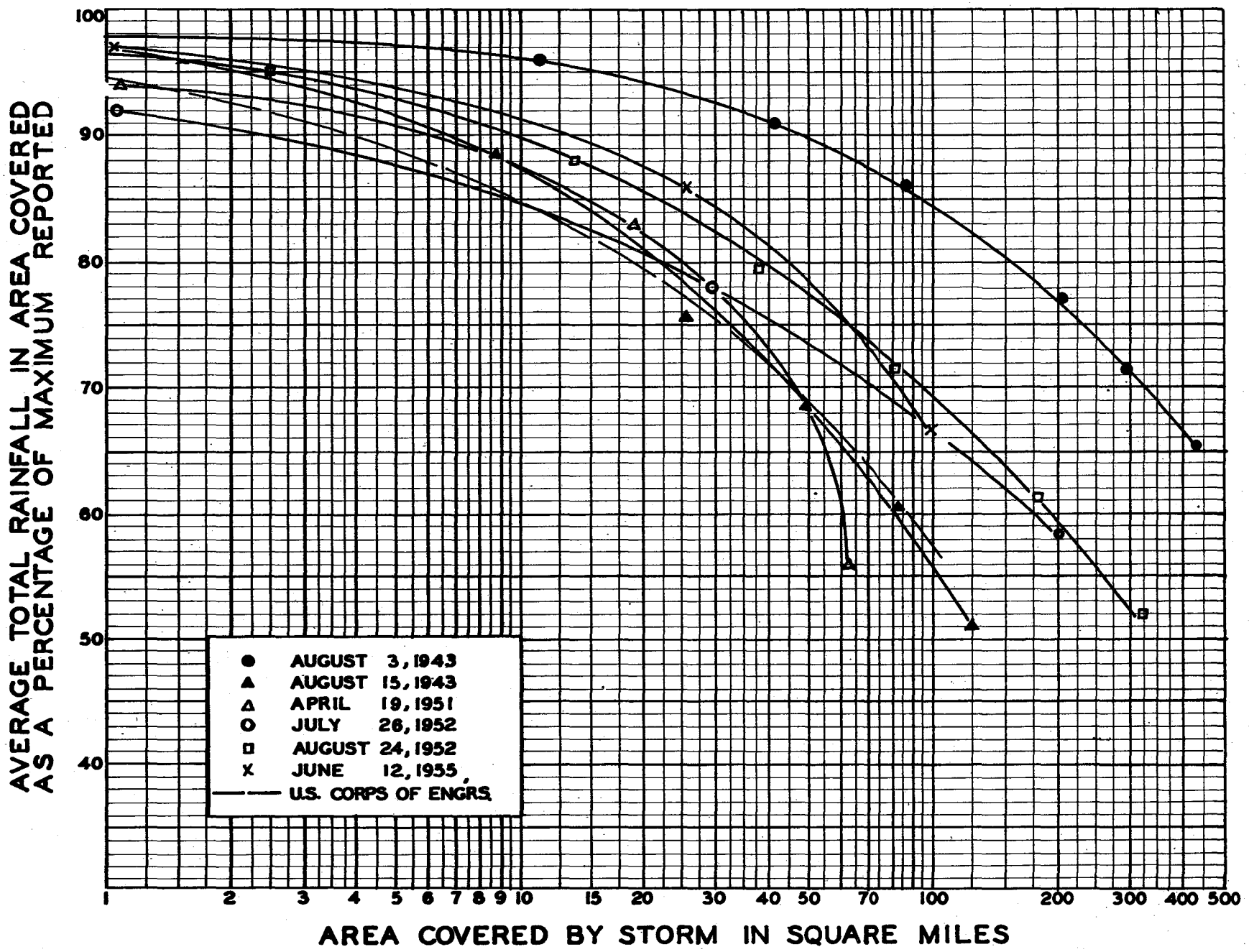
FIG. 15

STORM OF AUGUST 12 1956



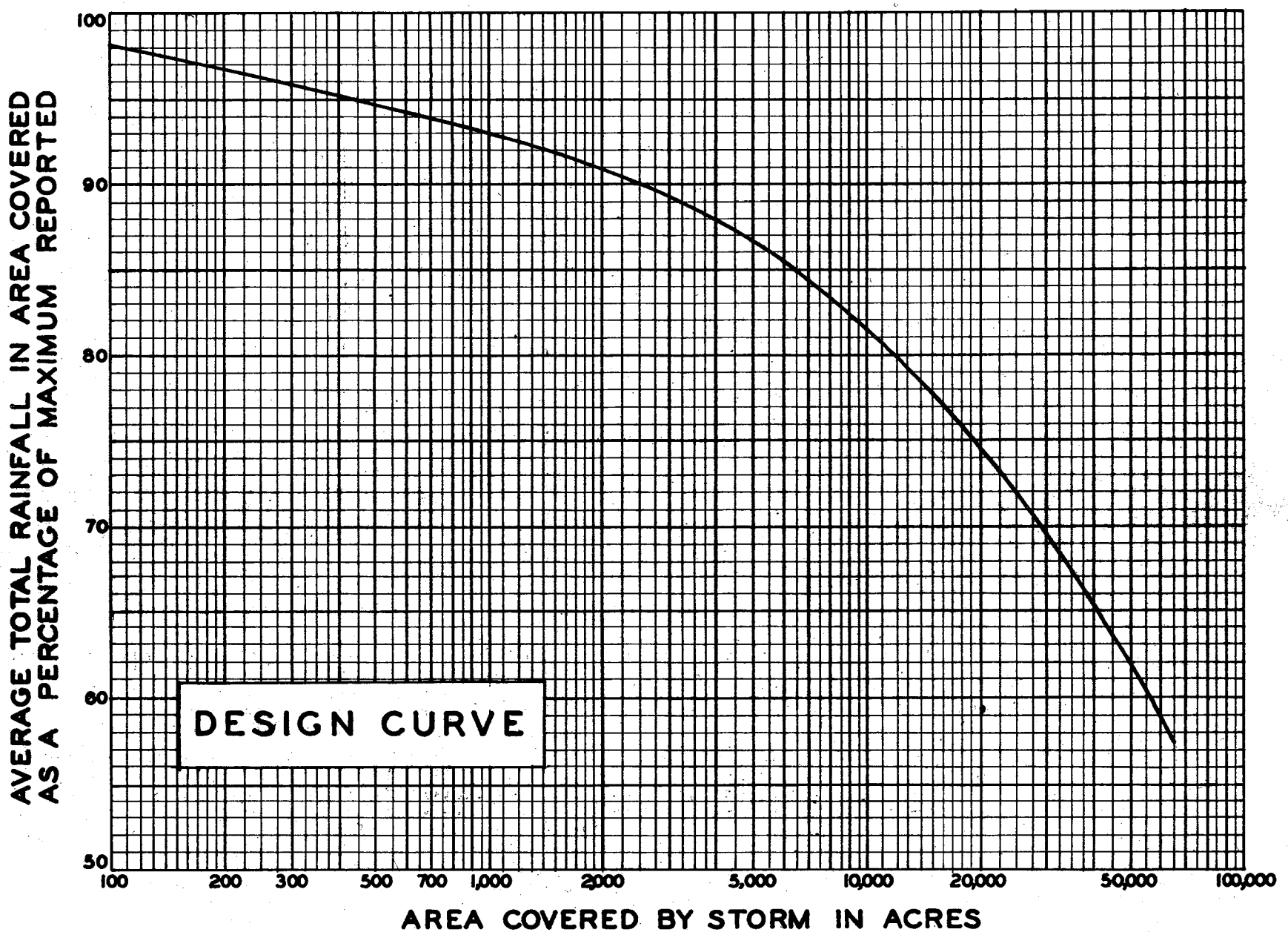
LEGEND
- ● WEATHER BUREAU STATION
- ○ COOPERATIVE STATION
- ////// PHOENIX CITY LIMITS

FIG. 16



AREA - DEPTH CURVES FOR LOCAL STORMS

FIG. 17



DESIGN AREA - DEPTH CURVE

FIG. 18

B. Infiltration and Other Losses

Not all the rain that reaches the earth becomes runoff. Some is returned immediately to the atmosphere by evaporation, some is retained in depressions to evaporate later, and some filters into the soil. A considerable portion which ultimately becomes runoff must be temporarily retained in channels and on surfaces to produce the hydrostatic conditions necessary to make flow take place. The actual losses are advantageous because they reduce the requirements of the drainage system. Temporary storage can also be helpful if it is allowable but it is usually this water that creates nuisance or does damage. The system should therefor be designed to keep this water within very definite limits.

1. Infiltration in Natural Soils

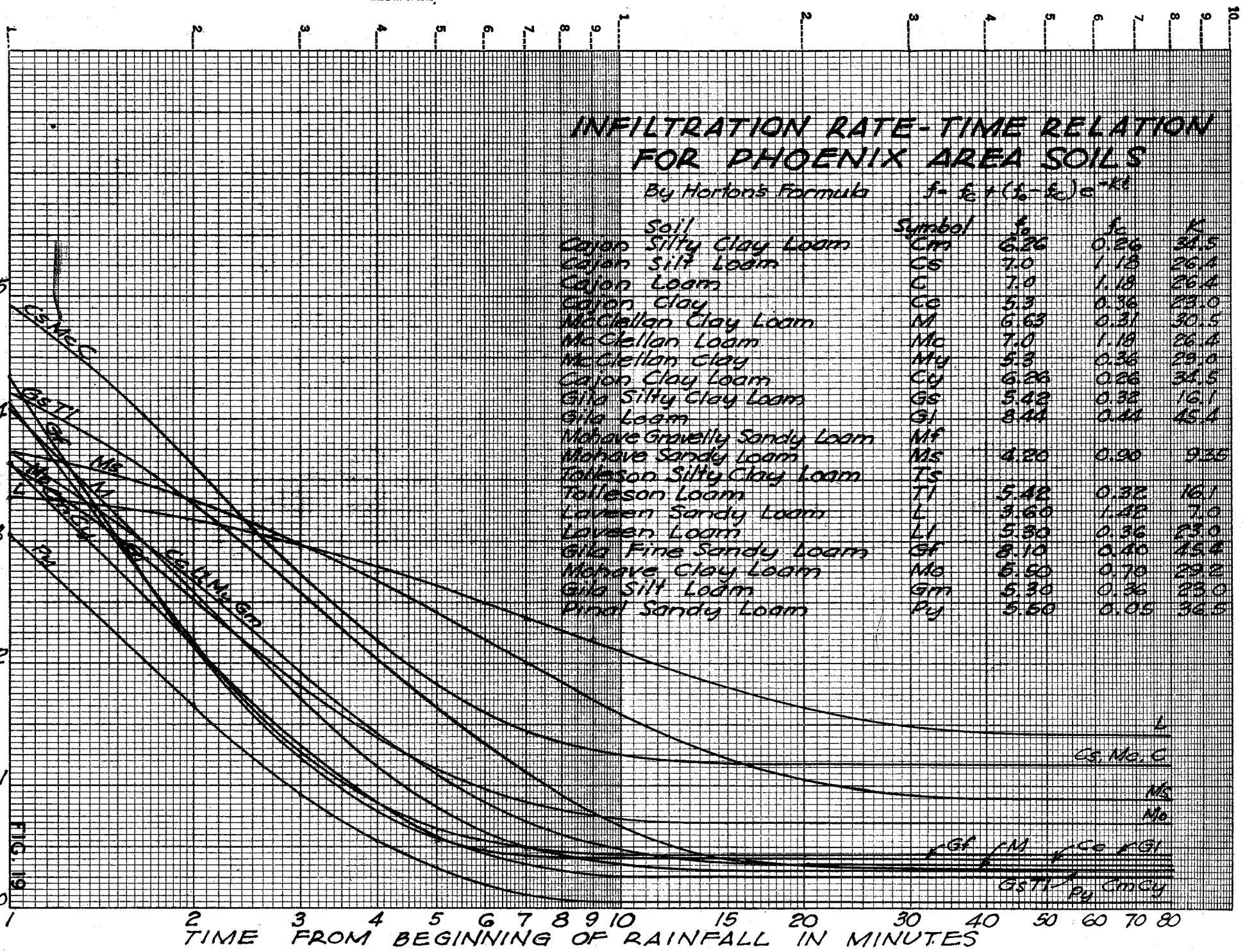
A good deal of experimental work has been done to measure the infiltration capacity of natural soils and to determine its variation with previous infiltration and time. A formula has been developed by Dr. R. E. Horton showing this relationship.[1] The constants in this formula have been evaluated for Arizona soils by the Soil Conservation Service in cooperation with the Arizona Agricultural Experiment Station.[2] Horton's formula and the constants are shown in Figure 19. The geographical location of the soils to which the constants apply is shown in Figure 20

---

[1] "Analysis of Runoff Plat Experiments with Varying Infiltration Capacity" Robert E. Horton, Transactions, American Geophysical Union, 1939, pp 693 to 711

[2] "Sprinkled Plat Runoff and Infiltration Experiments on Arizona Desert Soils", E. L. Beutner, R. R. Gaebe, and Robert E. Horton, Soil Conservation Service TP - 38, September 1940

5th lines accented.
MADE IN U. S. A.



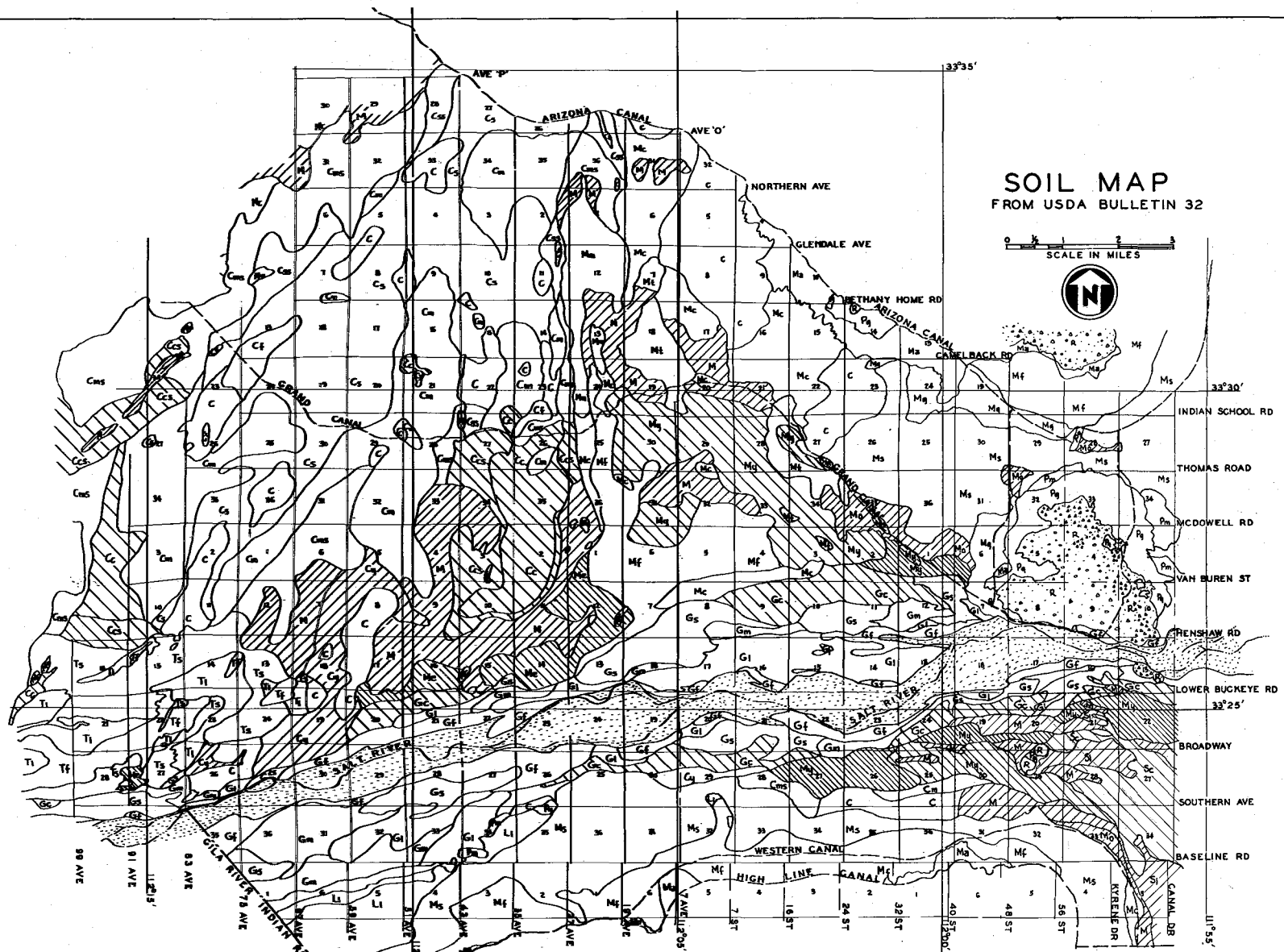

SOIL MAP
FROM USDA BULLETIN 32
SCALE IN MILES
0 ½ 1 2 3
N
AVE 'P'
AVE 'O'
ARIZONA CANAL
GRAND CANAL
NORTHERN AVE
GLENDALE AVE
BETHANY HOME RD
CAMELBACK RD
INDIAN SCHOOL RD
THOMAS ROAD
MCDOWELL RD
VAN BUREN ST
RENSHAW RD
LOWER BUCKEYE RD
BROADWAY
SOUTHERN AVE
BASELINE RD
SALT RIVER
WESTERN CANAL
HIGH LINE CANAL
GILA RIVER INDIAN RESERVATION
KYRENE DR
CANAL DR
33°35'
33°30'
33°25'
111°45'
111°40'
111°35'
111°30'
111°25'
111°5'
59 AVE
61 AVE
63 AVE
75 AVE
83 AVE
43 AVE
35 AVE
27 AVE
19 AVE
11 AVE
7 AVE
7 ST
16 ST
24 ST
32 ST
40 ST
48 ST
56 ST
CLAY
CLAY LOAM
LOAM OR LOAM TYPE
RIVER WASH AND FINE SAND
ROUGH BROKEN AND STONY LAND
Cl Cajon loamy sand
Cls Cl shallow phase
Cf Cajon fine sandy loam
Cfs Cf shallow phase
C Cajon loam
Cs Cajon silt loam
Css Cs shallow phase
Cy Cajon clay loam
Cm Cajon silty clay loam
Cms Cm shallow phase
Cc Cajon clay
Ccs Cc shallow phase
Gf Gila fine sandy loam
Gl Gila loam
Gm Gila silt loam
Gs Gila silty clay loam
Gc Gila clay
L Laveen sandy loam
Ll Laveen loam
Lc Laveen clay loam
Ls Laveen silty clay loam
Mc McClellan loam
M McClellan clay loam
Mt McClellan silty clay loam
My McClellan clay
Me McClellan clay adobe
Mf Mohave gravelly sandy loam
Ms Mohave sandy loam
Ml Mohave fine sandy loam
Ma Mohave stony loam
Mg Mohave fine gravelly loam
Mm Mohave loam
Mo Mohave clay loam
Mh Mohave silty clay loam
Pg Pinal gravelly fine sandy loam
Pm Pinal loam
Sl Sunrise clay loam
Sc Sunrise silty clay
Tf Tolleson fine sandy loam
Tl Tolleson loam
Ts Tolleson silty clay loam

FIG. 20

which has been adapted from a U. S. Dept. of Agriculture report.[1]

The plotted curves of Horton's formula in Figure 19 show that the initial infiltration capacity of a soil is fairly high, that it falls quickly as rainfall continues, and that it approaches a much lower rate which remains constant regardless of how long rainfall is continued. One would be inclined to think that the soil would finally become saturated and in effect impervious to further infiltration but generally the subsoils are so much higher in permeability that this condition never occurs. This is substantiated by experimental work done in 1950 by Karl Harris and H.B. Peterson of the Phoenix office of the Soil Conservation Service. The following is quoted from their unpublished report.[2] It is to be noted that the infiltration rates shown in the table are all after one hour of preliminary sprinkling, hence they represent final, or sustained, infiltration rates, the f_c in Horton's notation.

MC CLELLAN CLAY LOAM

> (The following table) "gives infiltration tests under several conditions. The columns marked "Spaded Lawn", "Lawns on which Boys Had Played", and "Vacant Lot" are on East Alvarado Street in Phoenix. The column marked "Vegetable Garden" is on the campus of Phoenix College, and the "Sorghum Field" is just across a fence from the vegetable garden.
>
> This table shows considerable differences in infiltration rates from 12.23 to 0.047 inches per hour. It will be noted that the rate for the second foot is about the same for every field.
>
> The difference in infiltration rates between the vegetable garden and the sorghum field is of particular interest. These tests were made not more than one hundred feet apart and bring

1. "Soil Survey of the Salt River Valley Area, Arizona" by W.G. Harper. F.O. Younger, A.T. Strahorn, S.W. Armstrong, and H.C. Schwalen. U.S.D.A. Bureau of Chemistry and Soils Bulletin 32, Series 1926

2. "A Progress Report on Factors that Effect Infiltration and Permeability Rates of Water into and through Soil" by Karl Harris and H.B. Peterson, S.C.S. 1950, pp32 & 33.

out the fact that cultural methods may change the rate of water infiltration into the soil a great deal; that is, from 0.047 inches per hour to 1.057 inches. The 1.057 rate was after the sorghums had been planted and irrigated twice.

	Spaded Lawn		Lawn on which boys had played	Vacant Lot Puddled by rain		Vegetable Garden			Sorghum
	Left rough	Raked Off		Surface	1 foot below surface	Surface	1 foot below surface	Surface	1 foot below surface
	12.23	6.77	0.456	0.425	2.510	0.029	2.900	1.390	2.270
	12.23	2.52	0.310	0.481	1.890	0.039	3.190	0.760	2.480
	12.23	4.32	0.339	0.347	2.740	0.081	2.690	1.020	2.450
	12.23	2.56	0.379	0.559	2.360	0.047	2.950	1.060	2.670
	12.23	4.04	0.200	0.371	1.890				
AVG.	12.23	4.04	0.337	0.436	2.278	0.047	2.933	1.057	2.468

In comparing the infiltration rates obtained by Harris and Johnson with the value of f_c shown for McClellan Clay Loam in Figure 19 it should be borne in mind that the former values are for soils that have been manipulated in various ways whereas the Figure 19 values are for soils in their natural state with naturally occurring vegetative cover. Even so, Harris' value for "Lawn on Which Boys Had Played" of 0.337 inches per hour is fairly close to the Figure 19 value of f_c of 0.31 for the same soil. Harris shows very plainly how greatly the natural capacity of the soil can be increase by certain types of cultivation.

The question arises- What value of infiltration rate should be used in

the computation of storm runoff if it is such an extremely variable quantity? The initial infiltration rate is so high that it affects the average for a much longer period than is required for the instantaneous rate to reach the final constant value. It is very tempting to the designer to use the average of instantaneous values determined from Horton's formula but some allowance must be made for the possiblilty that a storm may occur a short time prior to the design storm. The fact that this does happen is graphically illustrated in Figure 21. It is also true that in many residential areas the soil is constantly kept moist by lawn sprinkling. For these reasons the basic value of infiltration rate used for runoff computations will be the experimental value for f_c but this should be modified by the designer's judgment based on his actual knowledge of the particular area under consideration. For example a residential area where flood irrigation is practiced and where berms are in place to confine irrigation water will not produce runoff for any storm likely to occur in this area. Allowance may be made for this condition by arbitrarily increasing the value of f_c.

The area contributing storm water to a single inlet in the drainage system may comprise soils of several different types. When this is the case the value of f_c should be a weighted average, again adjusted according to the judgment of the designer. Some average values of the sustained infiltration rate for large areas are shown in the following table.

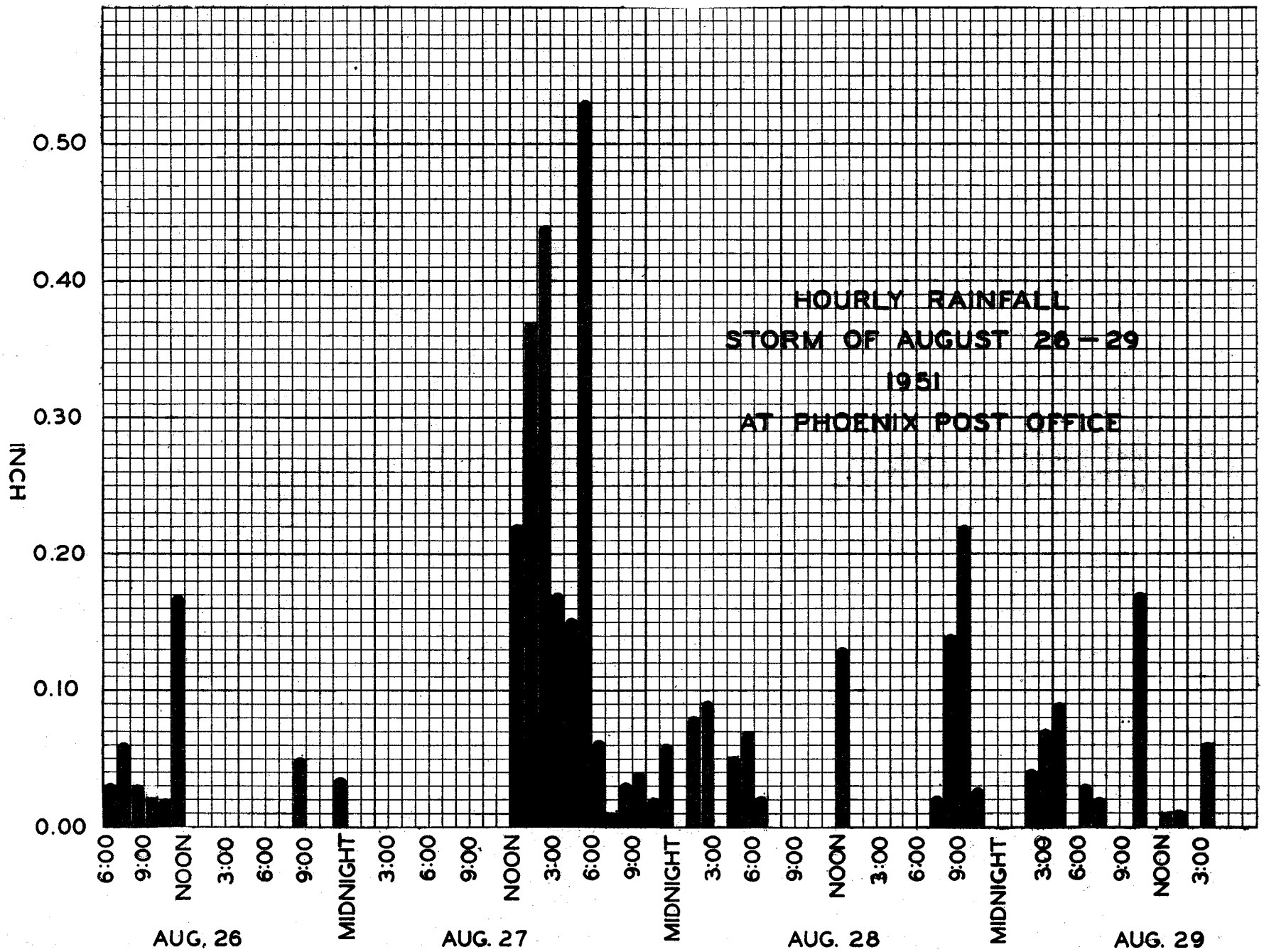

HOURLY RAINFALL
STORM OF AUGUST 26 – 29
1951
AT PHOENIX POST OFFICE
INCH
0.50
0.40
0.30
0.20
0.10
0.00
6:00
9:00
NOON
3:00
6:00
9:00
MIDNIGHT
3:00
6:00
9:00
NOON
3:00
6:00
9:00
MIDNIGHT
3:00
6:00
9:00
NOON
3:00
6:00
9:00
MIDNIGHT
3:00
6:00
9:00
NOON
3:00
AUG. 26
AUG. 27
AUG. 28
AUG. 29

FIG. 21

Boundaries	Area Sq. Mi.	Overall Avg. f_c in./hr.
Arizona Canal to Grand Canal Old Crosscut Canal to 7th St.	17	0.88
Grand Canal to Salt River Papago Park to 7th St.	14	0.41
Arizona Canal to Grand Canal 7th St. to 27th Ave.	12	0.76
Grand Canal to Salt River 7th St. to 27th Ave.	13	0.52
Arizona Canal to Grand Canal 27th Ave. to Agua Fria Drainage Divide	22	0.90
Grand Canal to Salt River 27th Ave. to 67th Ave.	26	0.35
Grand Canal to Salt River 67th Ave. to Agua Fria Drainage Divide	28	0.48
Highline Canal to Salt River near 48th St. to 7th St.	14	0.61
Highline Canal to Salt River 7th St. to Gila River Drainage Divide	14	0.59

This table indicates that the soils found between the Arizona and Grand Canals are approximately twice as capable of absorbing rainfall as the soils between the Grand Canal and the Salt River.

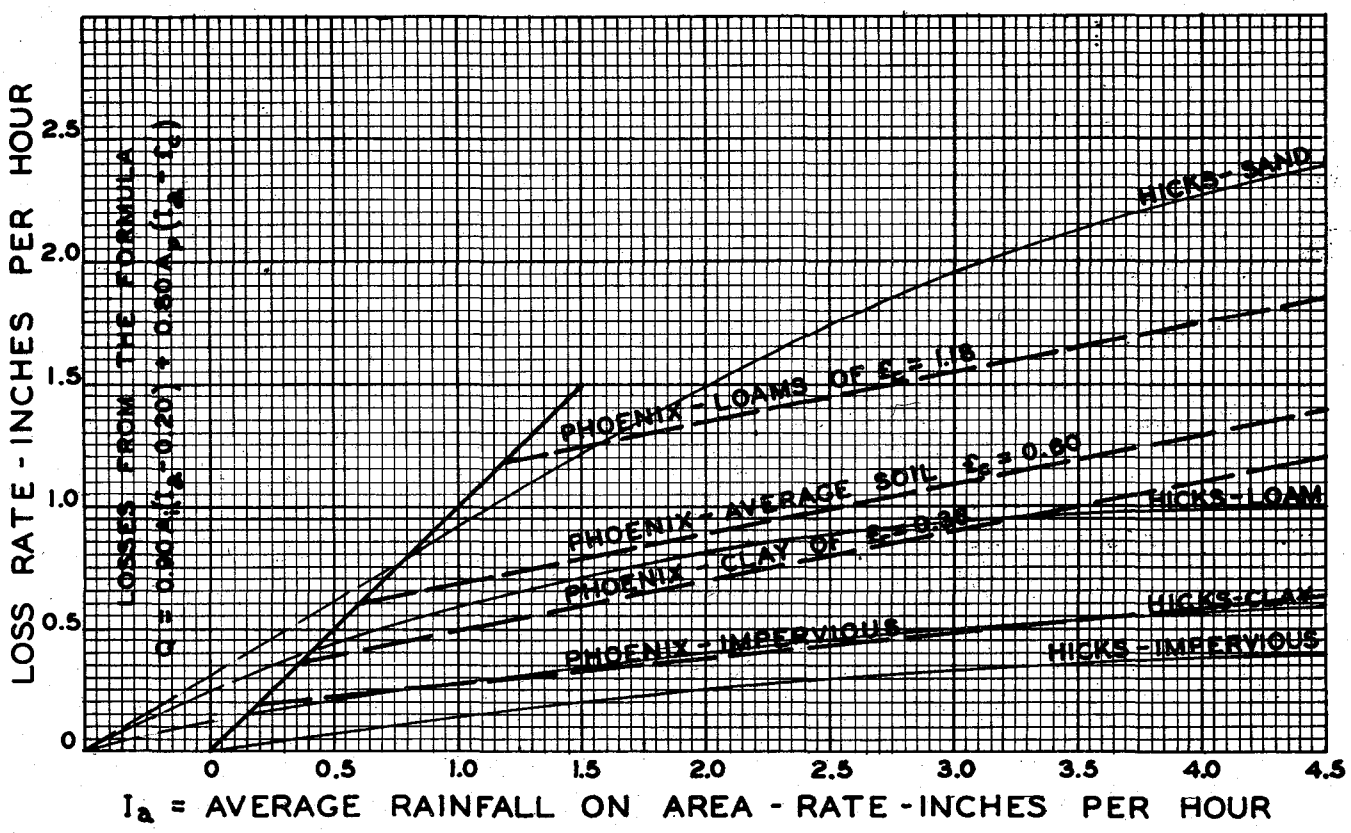
There is also an appreciable water loss on surfaces normally regarded as impervious. Even concrete will allow some water to pass through cracks in its surface as many a do-it-yourself swimming pool builder has discovered to his sorrow. A good deal of rain falling on hot pavement is immediately evaporated. There is an appreciable loss in the interception of rainfall by foliage and on the surfaces of buildings. Some water always finds its way into sanitary sewer, gas, telephone and electrical manholes. Some water is

trapped in surface hollows and depressions. There is a film of water that refuses to drain off of paved surfaces and will finally be removed only by evaporation after the storm. Finally there is the water that must be present on all water carrying surfaces and in all channels to provide the proper hydrostatic conditions for flow to take place at all. This water is constantly being replaced with new water and it all drains off after the storm, but the effect on required capacity of the drainage system is equivalent to that of an actual loss.

All of these effects would seem to be of very minor importance but the cumulative result is well worth taking advantage of in the design of storm drains. To do this it is necessary to evaluate these effects, preferably in the aggregate. It would seem that water lost in this way would be a fixed amount, more or less independent of the storm rainfall, but W.I. Hicks has found a relation between rate of rainfall and rate of loss on impervious surface in his work in Los Angeles.[1] It will be assumed for the purposes of this report that this loss on impervious surfaces amounts to ten percent of the rainfall. This is also the value Hicks uses.

Figure 22 shows the relation between rainfall and losses for various soils and impervious surfaces both as derived by Hicks for the Los Angeles area and as recommended by this report for Phoenix. These curves will be used as described in the following section.

1. A Method of Computing Urban Runoff. W.I. Hicks Transactions of the American Society of Civil Engineers, Vol. 109, (1944) p.1265



INFILTRATION AND DEPRESSION STORAGE LOSS COMPARISONS

LOS ANGELES DESIGN CURVES - PER W. I. HICKS REPORTS ASCE

PHOENIX VALUES FROM FORMULA INCLUDE LOSSES DUE TO SURFACE DETENTION AND CHANNEL STORAGE WHICH HICKS ANALYSES SEPARATELY BY HYDROGRAPH METHODS.

FIG. 22

C. Runoff

The first objective in a storm drainage system design is usually to arrive at the quantities of storm water it is necessary for the system to handle at various critical points along its length. These quanities are expressed as rates since it is rates rather than total quantities that affect the sizing of components of the drainage system.

The runoff process is a funneling process. Rain falls at a certain rate over a fairly large area and flows over surfaces and through a converging system of channels to a common low point where it discharges from the area. The rate of discharge cannot be greater than the rate of supply unless there is temporary storage and sudden release, a condition not likely to be found in storm drainage system. The rate of discharge can however be lower than the rate of supply, first, in case there is another outlet or, second, if there is temporary storage and gradual release after the supply ceases. Both of these conditions are likely to be found in any storm drainage system. The other outlet is provided by the various losses already discussed, namely infiltration, evaporation, and permanent storage. Temporary storage is provided by the surfaces and waterways themselves which must contain water in order to transport it.

If the rate of supply were equal to the rate of runoff, that is if there were no losses or temporary storage, the relation between supply and runoff could be expressed by the relation.

$$Q=IA \tag{1}$$

where Q is the rate of runoff, I the rate of rainfall, and A the drainage area. The equation needs no constant of proportionality if Q is

expressed in cubic feet per second, I in inches per hour, and A in acres.

In order to account for losses it is necessary to consider the relative amounts of pervious and impervious surface within the area and the loss by infiltration in the former. It is assumed that rain falls at the same rate on both types of surface. Equation 1 would then be expressed as

$$Q = I\ A_i + (I - f_c)\ A_p \qquad (2)$$

where A_i and A_p are the impervious and pervious areas, both in acres, and f_c is the sustained infiltration rate for the soil in the pervious area. In case of an area having several different soils f_c would be the weighted average. It would also be arbitrarily modified in accordance with the judgment of the designer based on his knowledge of the area as has been previously mentioned. Probably f_c should be at least 0.20 for even the most densely packed soil.

Equation 2 is still not representative of actual conditions. It has been pointed out that there is a considerable loss in impervious areas and that there are losses other than infiltration in the pervious areas. The following formula will therefor be used in computing flows.

$$Q = 0.9\ A_i\ (I_a - 0.20) + 0.8\ A_p\ (I_a - f_c) \qquad (3)$$

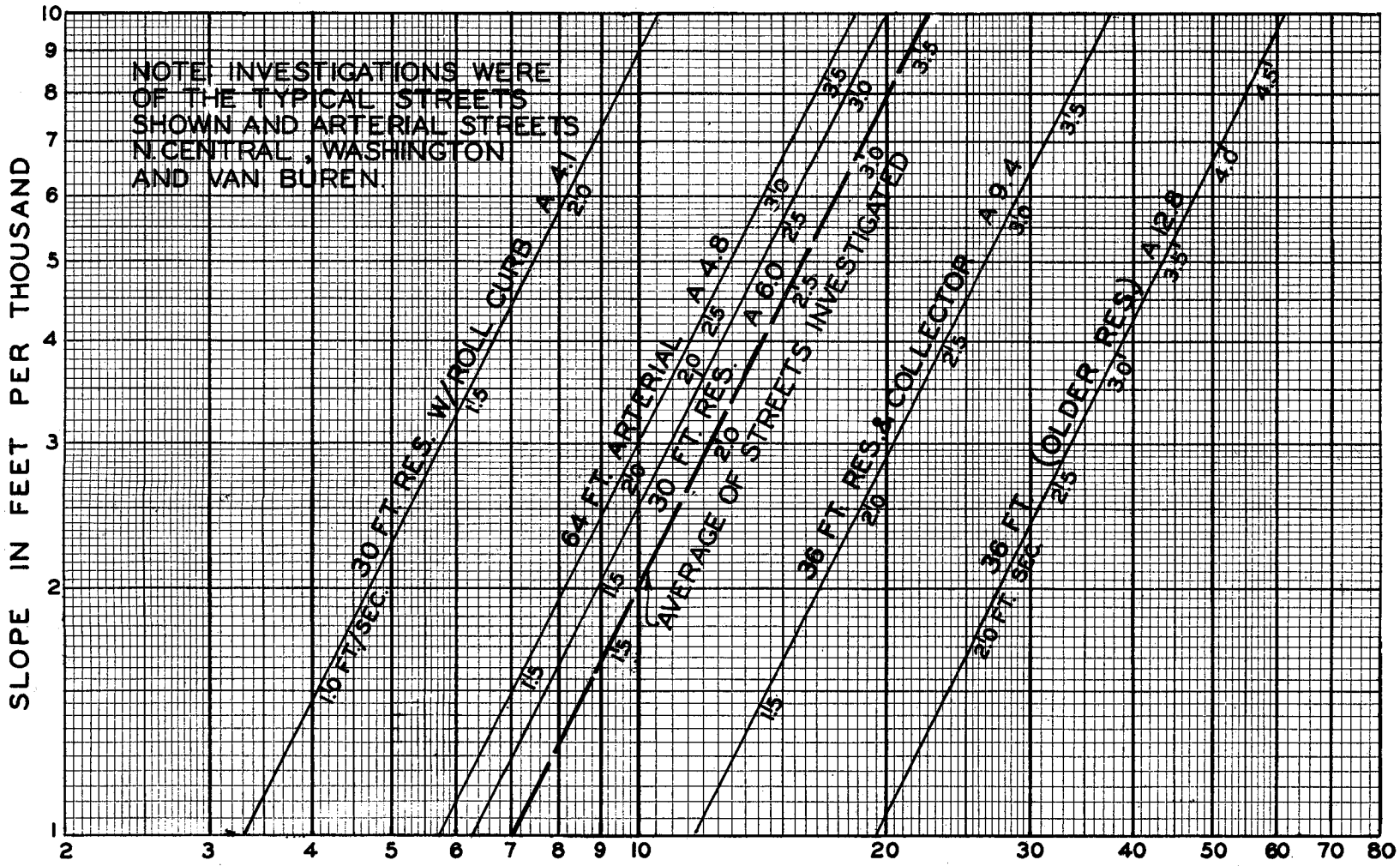
where

Q = design runoff rate in cubic feet per second
A_i = impervious portion of the drainage area in acres
A_p = pervious portion of drainage area in acres
I_a = average rainfall intensity over the area in inches per hour
f_c = final or sustained infiltration capacity of the soil in the pervious area in inches per hour

In the first term on the right hand side of Equation 3, I_a is arbitrarily

reduced by 0.2 inch per hour to account for depression storage, infiltration through cracks, and the other losses which exist even in impervious surfaces. The whole term is then multiplied by a factor of 0.90 to account for the fact that the loss is not a constant amount but is also a function of intensity as mentioned previously. This also allows for surface detention and channel storage. The second term on the right side is multiplied by a factor of 0.8 to make allowance for depression storage, for increased infiltration over the value used for f_c during the early minutes of the rain when infiltration capacity is high, and for surface detention presumed to be for flow over grassed areas.

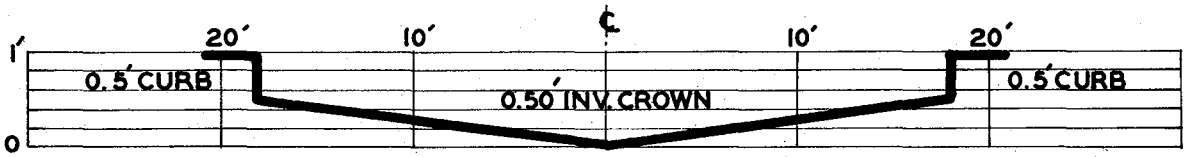
At this point it becomes necessary to discuss concentration time and to show how it affects the value of I_a in Equation 3. In any given drainage area a certain period of time is required for water to flow from the most remote point to the outlet. This depends of course upon the length of the flow path and the velocity of the water. The velocity in turn depends upon the channel slope, its roughness or frictional resistance to flow, and also to the quantity or depth of water flowing. Manning's formula was used to obtain velocities of flow in streets using an "n" of 0.015. Scobey's chart for the solution of Kutter's formula was used in obtaining velocities in pipes, using an "n" of 0.012 for pipe of 21" diameter and over. These are given in graphical form in Figure 23 and 24 for streets and Figure 25 for pipe lines. Both streets and pipe lines are assumed flowing full. Manning's formula is shown in Figure 23. It is usually solved for the velocity of the flowing water using slope in feet per foot and the hydraulic radius in feet. (The hydraulic radius is the measure



COMPUTED FROM MANNINGS FORMULA · $V = \frac{1.486}{n} R^{2/3} S^{1/2}$

TYPICAL STREET PAVING CAPACITY
FULL - NO FREEBOARD

FIG. 23

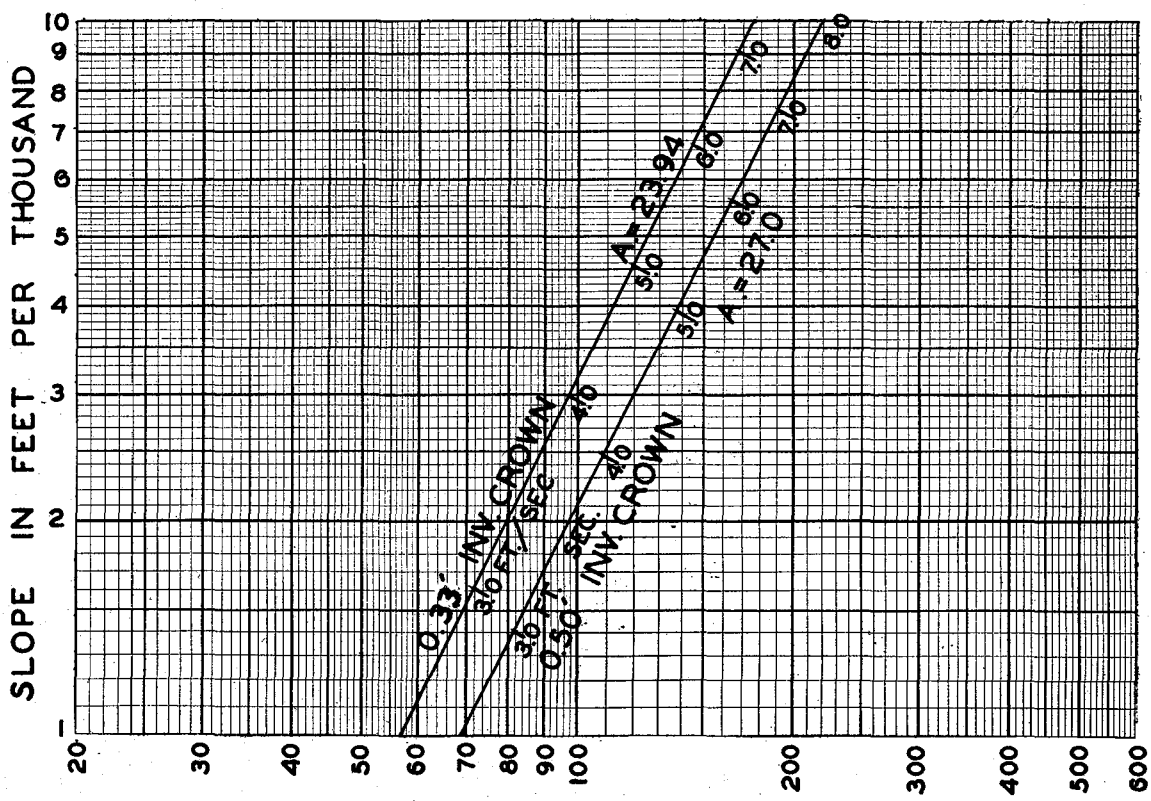


AREA TO TOP OF CURB = 27.00 SQ. FT.
WETTED PERIMETER = 37.01 FT.
HYDRAULIC RADIUS = 0.730 FT.

36 FT. RESIDENTIAL AND COLLECTOR STREETS INVERTED CROWN

NOTE: 36 FT. RESIDENTIAL AND COLLECTOR STREETS WITH 0.50' CURB AND 0.33' INVERTED CROWN

AREA TO TOP OF CURB = 23.94 SQ. FT.
WETTED PERIMETER = 37.01 FT.
HYDRAULIC RADIUS = 0.642 FT.

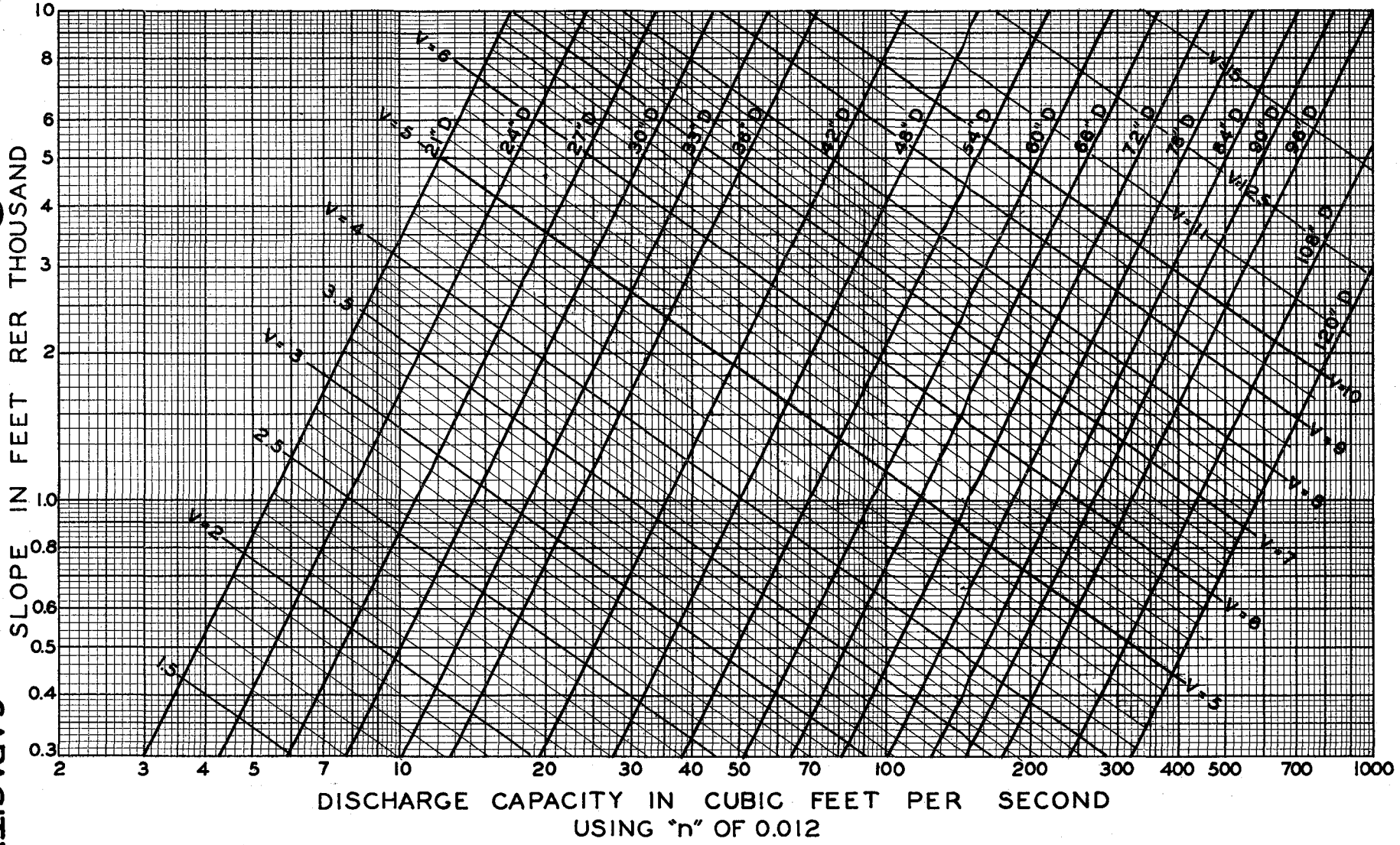


DISCHARGE CAPACITY IN CUBIC FEET PER SECOND
USING "n" OF 0.015

COMPUTED FROM MANNINGS FORMULA $V = \frac{1.486}{n} R^{2/3} S^{1/2}$

TYPICAL INVERTED CROWN STREET PAVING FULL NO FREEBOARD

FIG. 24



PIPE CULVERTS CAPACITY

FIG. 25

of the water quantity-depth relation and is equal to the cross-sectional area of the stream divided by the wetted perimeter.) The curves in Figures 23 and 24 give flow quantity directly but this is merely the product of velocity and the cross-sectional area of the stream. The term "n" in the formula relates to channel roughness. For gutters "n" was assumed equal to 0.015, for pipes 18 inches and under 0.013, and for pipes larger than 18 inches 0.012. Actual values of "n" for concrete pipe are lower than those chosen but these values allow for inlet disturbances, transition losses, and similar effets.

For flow over sodded areas or desert surfaces where water is carried by sheet flow rather than in channels there have been derived several formulas giving the relation between velocity, mean water depth, and slope of surface. The equation

$$V = 0.96\ d\ S^{\frac{1}{2}}$$

has been suggested for flow over turfed surfaces.[1] In this formula the velocity, V, is in feet per second; d is the water depth in inches; and S is slope in feet per foot. The coefficient of 0.96 was derived experimentally and a variation in this value from 0.6 to 1.2 is reported for different types of grassed surfaces. If it is assumed that land surface and street pavement slopes in an area are approximately equal, velocities in gutters will be from 32 to 16 times as large as velocities over sod computed by this formula for runoffs ranging from one fourth-inch to 2 inches.

1. W.W. Horner and S.W. Jens, "Surface Runoff from Rainfall Without Using Coefficients", Transactions, ASCE, No. 107 (1943) p. 1059

W.I. Hicks in his work in Los Angeles[1] has developed formulas for rates of flow on tar and sand, tar and gravel, and clipped sod surfaces giving ratios of approximately five for a tar and sand surface compared to clipped sod and about ten for gutter flow compared to clipped sod. In this report it will be assumed that velocities over sod will be approximately one tenth of street gutter velocities in the same area and that overland flow velocities in commercial and industrial areas will be one half of gutter velocities.

Actually for any sizeable area, say over 160 acres, the portion of the total runoff time attributable to overland flow is small and it makes little difference in the final result how this time is computed. In the computations done for this report the practice of computing flow times based on gutter and pipe flows only and arbitrarily adding four to five minutes in residential areas and two minutes in commercial and industrial has been followed.

In the hilly areas where drainage takes place through natural channels such as in the Phoenix Mountain and Salt River Mountain areas the following formula is useful for computing concentration time:

$$t_c = 5.75 \left(\frac{L^2}{S}\right)^{1/3}$$

where t_c is concentration time in minutes,
L is channel length in miles, and
S is slope in feet per foot.

Concentration time where flow occurs primarily on city streets and in culverts can be computed quickly by use of Figures 23 through 25.

1. Loc. Cit., pp. 1097 to 1102

Once the concentration time for a given drainage area has been determined this time is used to find the rainfall intensity to be used in Equation 3. It has been pointed out that the longer a storm lasts the less intense it is likely to be, the relation being shown in Figure 9. In any given drainage area (which has a concentration time characteristic of that area) the maximum runoff rate condition is likely to occur when the area undergoes a storm of duration equal to the concentration time of the area. A storm of longer duration would be less intense and its effect would be a lower peak rate sustained for a longer time. A storm of shorter duration than the concentration time of the area is not likely to produce such a high runoff rate even though the rainfall intensity is apt to be higher because the entire drainage area does not begin to contribute at the point of discharge by the time the rain has stopped. A storm of duration less than the concentration time of the drainage area may however, depending upon the shape of the area, actually produce higher runoff rates than one lasting as long as the concentration time and this possibility should always be investigated. The rainfall intensity is taken from the curves in Figure 9 for the concentration time or a shorter time if a shorter time produces higher runoff. The return period curve used is chosen in accordance with the degree of protection desired. (This aspect is discussed later in this report.) The intensity thus obtained from Figure 9 may be reduced by a factor which varies with the size of the drainage area. This factor is taken from Figure 18. The reduced value is the I_a, or average rainfall intensity, of Equation 3.

It is by application of the methods described above that the quantities of storm water shown in Exhibit VI were derived. The calculations were

carried out in tabular form and required sixty sheets. Because of their bulk they are not included in this report but one typewritten copy is submitted for the City's files.

IV. DESIGN CRITERIA

After an appraisal of the topography and hydrology of a drainage area and the computation of flood flows for various frequencies of occurrence it is proper to begin choosing routes for storm sewers and determine the necessary sizes of components of the system. The purpose of this section of this report is to recommend certain policies and standards of design for use in planning the system. These will again be of a general nature and judgment should be used in applying them to the design of any particular portion of the system.

A. Design Storm Frequency

It has been pointed out previously that there is an inverse relation between the severity of a storm and the average frequency of occurrence of others like it. In establishing the size of a storm sewer it then becomes necessary to decide if the sewer should be made large enough to handle the worst storm likely to occur once a year, every five years, or some other period. It is seldom practical to design for the worst condition ever likely to occur although this must be done where the hazard to life and property is great as in the case of some dams. Storm sewers are usually constructed to alleviate a nuisance however, not to protect life and limb.

There is therefore seldom any justification for designing a drainage system for the maximum probable storm or for any very low frequency of occurrence. In some locations it is possible to justify building for a ten, twenty-five, or fifty year return period on the basis of potential flood damages. In general the higher the value of the district the better its flood protection should be. For most of the Phoenix area however, quantities of water to be handled are great, but the past record of damages is so low that it is not economically justifiable to construct storm sewers for any very severe storm. This means simply that when such storms come, as they surely will, the drainage system will be overtaxed and there will still be ponding of water, blocked streets, stalled cars and flooded buildings, but not as frequently nor so severely as before. The less often flooding is allowed to happen the more such protection costs the community and there is a point beyond which the cost of flood prevention exceeds the cost of letting floods occur. This point is soon reached in the Phoenix area. In Sunnyslope and other communities near the mountains damage is more frequent and more severe, the cost of protection is less and hence more easily justified.

The potentialities of flood damage are rapidly increasing as the city grows. The flood of August 3, 1943 did damage estimated at $145,000 but the inundated areas were principally agricultural lands. This land is now about 60 percent occupied by homes and, in the lower portions, expensive commercial and industrial installations. It can hardly be denied that a similar storm tomorrow would result in many times greater damage.

Where economic justification cannot be shown the so-called intangible factors are usually brought into play. It is difficult to place a value on the alleviation of a nuisance, the increase in property values resulting from the installation of storm drains, or the extent of the obligation the community has for increasing runoff by paving up to 20 percent and more of the drainage area. These factors are very real and important even if it is impossible to evaluate them objectively and often they are sufficiently weighty to warrant construction of drainage systems that cannot be justified on the basis of diminution of flood damage alone. It is seldom possible to show economic justification for parks, monuments, or ornamentation on public buildings yet these things are provided and the costs willingly borne in the interest of building a better city. Similar reasoning may be applied to the case of storm sewers.

The storm water flows to be expected in the Phoenix area are shown in Exhibit VI. The flows in the developed areas are shown as if they were concentrated along the north-south section and midsection lines. These flows are computed on the assumption that the area has reached its ultimate development. Quantities shown on Exhibit VI are for a storm such as might be expected once every two years in the developed areas and once every hundred years in the natural channels draining mountainous areas. Calculation of flows for one-year storms in Phoenix was also made. The basis for this choice of design frequencies is simply that it does not seem reasonable to install any work providing less than one-year protection while, in the built-up areas, any greater degree of protection would generally be prohibitively expensive. In the mountainous areas flows are

concentrated in narrow channels and hence more dangerous. Because slopes are steeper and land generally less expensive it is practicable to provide a higher degree of protection than in developed areas. This is the reason for choosing the hundred-year storm to provide for in these areas. Final design of the system should of course give due consideration to special local conditions which would warrant a greater or lesser degree of protection.

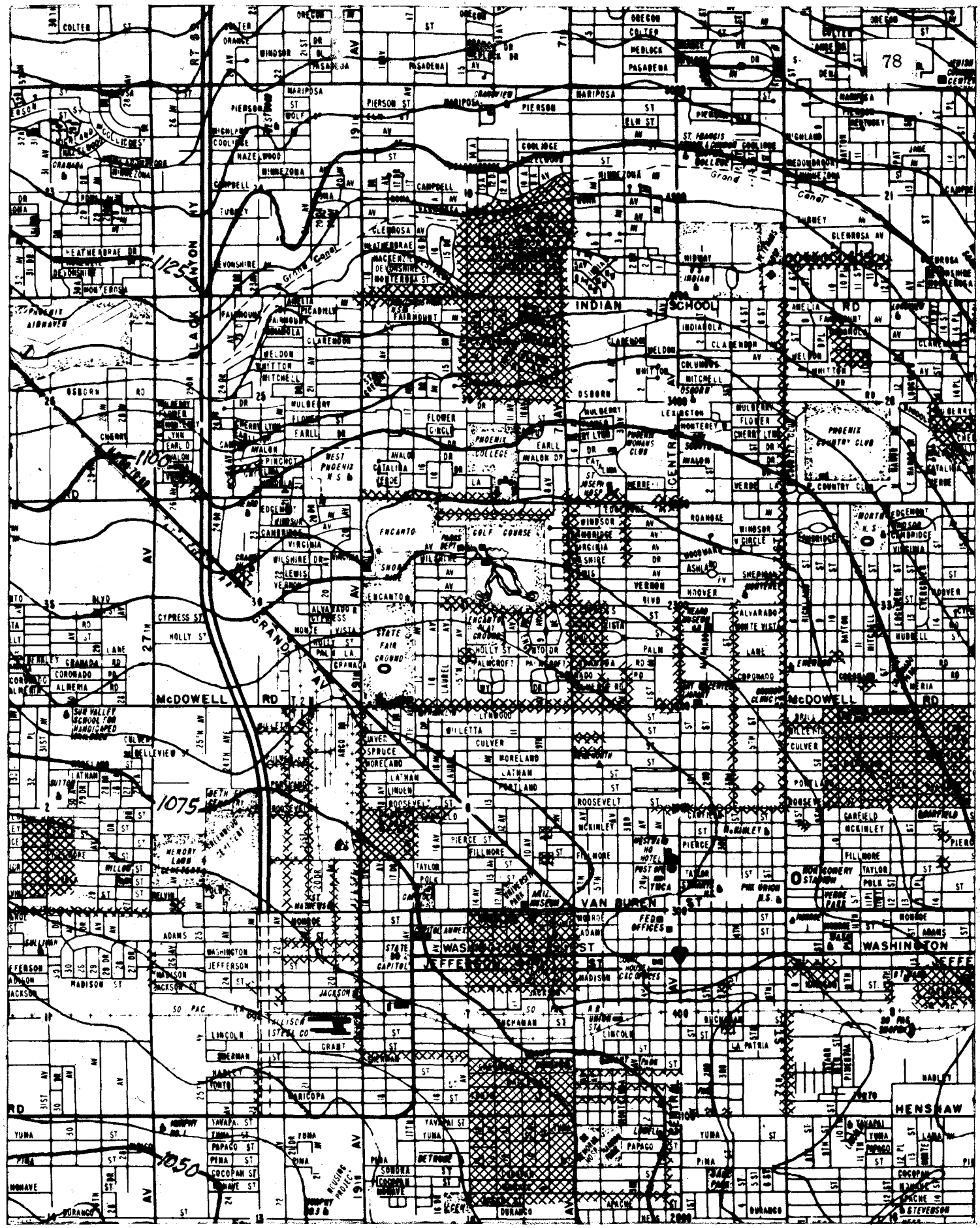
At present construction costs even two-year protection is beyond the means of the community. It is recommended however that the two-year standard be the goal and that lines designed for one-year flows be installed only in the most critical places along the section line streets. Later as more intensive development occurs and as more funds become available the mid-section drainage lines can be installed which will achieve nearly the 2-year standard sought.

B. Present Trouble Spots

The Superintendent of Streets receives complaints of flooding from certain areas after practically every storm. The streets, intersections, and general areas which are most persistently troublesome are plotted in Figure 26. Some of these troubles are due to purely local conditions such as the flooding above the old Maricopa Canal between 10th and 14th Streets and at Verde Lane west of the Black Canyon Highway. Other areas have troubles of a more deep seated nature because they are situated in natural swales and are areas of concentration for water arriving from two or three directions. A glance at the contours in Figure 26 will show that there is a ridge running through downtown Phoenix on an axis through the intersections of 7th Avenue

and Washington Street and 7th and Van Buren Streets. The small swale south of this ridge is the center of quite a bit of drainage trouble. The ground slope in this area is to the west rather than southward toward the river.

There is another ridge which runs north and south near the alignment of the Black Canyon Highway. The fan-shaped area between this ridge and the one previously mentioned is one broad swale with no sharply defined channels. Trouble spots exist throughout this area indicating the general nature of the problem.



AREAS PRESENTLY SUBJECT TO FLOODING

FIG. 26

C. Utilization of Existing Irrigation and Waste Ditches

Many of the street pavement projects constructed in suburban areas under the Improvement Act utilize irrigation and waste ditches operated by the Salt River Valley Water Users Association for the disposal of storm water. This has usually been done with Water User's permission under a revocable license. This arrangement has been fairly satisfactory in the past but the point is approaching where this can no longer be done.

There is little reserve capacity in the irrigation system when it is delivering its design quantities of water. Many of the larger canals operate brimfull almost continuously. The Water Users also have the practice of spilling water down laterals when there is danger of heavy runoff entering the main canals. It is quite likely therefor that there will be no capacity available at the very time when it is required.

Even if the canal system could be depended upon to be empty when storm runoff began to enter it, it would still be inadequate for any really severe storm. The capacity of the Arizona Canal above Arizona Falls (52nd Street) is about 2,000 second feet. Between Arizona Falls and 48th Street, it is 1,400 second feet, and it diminishes by steps to 400 second feet at 75th Avenue. The total capacity of the laterals occurring about once per mile is about 1,200 second feet, and the two Crosscuts together can safely handle another 1,200 second feet. The total peak rate of inflow into the Arizona Canal during the storm of August 3, 1943, was estimated at 30,000 second feet or about 15 times the capacity of the system.

This storm was severe enough to warrant some further description. The data given are from accounts in the Arizona Republic of August 4 and 5, 1943, from U. S. Weather Bureau records, and from an excellent report on the storm in the files of the Salt River Valley Water Users Association.

Storm conditions really began on August first. Late on August second, a heavy downpour occurred in the vicinity of Tempe.[1] Enough rain from this storm fell in Paradise Valley to cause the SRVWUA to empty the Arizona Canal in preparation for the influx of expected flood water. The inflow was not especially severe, however, and by 7:00 p.m. of the second, the spillway gates were closed, and the canal refilled. The storm wasn't over. It began raining again about 3 a.m. of August third. The intensity of rainfall reached its peak about 7 a.m. and finally stopped altogether at 11 a.m.

Total rainfall for the storm of August third at the Phoenix Post Office was 2.12 inches. It is likely that more rain than this fell in the desert areas to the north, but no records are available. The co-operative Weather Bureau stations at Carl Pleasant Dam, Cave Creek Dam, and Deer Valley were not in existence at the time of the storm.

Heavy inflow into the Arizona Canal began at Indian Bend by 8 a.m. on the third. The Indian Bend Spillway had a capacity estimated at 8,000 second feet, but this was insufficient to protect the canal. The first break in the levee occurred at 11 a.m.

1. In this storm 2.11 inches fell at the Tempe Date Gardens between 3:15 and 3:45 p.m. This is a rate of 4.22 inches per hour sustained for 30 minutes. By figure 9 this would rate as more severe than a thousand-year storm.

A series of 22 separate breaks occurred in this area before the water began to subside. High water marks showed a depth of about four feet for a distance of more than three-fourths of a mile in Indian Bend Wash immediately north of the Arizona Canal. Peak flow at Indian Bend was estimated by the Water Users to be about 15,000 cubic feet per second.

Peak inflow conditions arrived at different times along the Arizona Canal. At Lateral 6, the head of the Old Cross Cut Canal, the peak occurred at noon. At Lateral 10, there were two peaks, one at 10:30 a.m., due to water out of the Phoenix Mountains, and another later due to water coming down the canal from the east. At Cave Creek, there was inflow into the canal during the night of August 2-3 and by 6:00 a.m. on the third, it amounted to 175 second feet. At 6:15 a.m. a sudden rise in inflow was reported. Water ponded to a depth of $3\frac{1}{2}$ feet north of the canal. The inundated area extended from about 700 feet west of the Cave Creek channel to east of 19th Avenue. There was no break in the south bank of the Arizona Canal in the Cave Creek area until between noon and 1 p.m. on the third. When the break did occur, the water level in the canal dropped four feet in less than two hours. It was this mass of water released in the old Cave Creek channel now obliterated by farms and homes that probably did the greatest damage. The water arrived at the Grand Canal between 4 and 5 p.m. and began overflowing the south bank by 5:15 p.m. Nine breaks in the Grand Canal occurred within one quarter mile east of 19th Avenue. The water continued southward reaching 17th Avenue and Encanto by 7:55 p.m., Roosevelt and Grand Avenue by 10:02 p.m., and 15th Avenue and Van Buren at 11:50 p.m. The water depth on the ground in front of the Arizona Highway

Department Building on Jackson Street was 18 inches at 11:55 p.m.

The peak flow in Cave Creek above the Arizona Canal was estimated by high-water marks to be 9,000 cubic feet per second. The discharge from Cave Creek Dam was regulated at 400 cubic feet per second during the storm, the reservoir rising to a gage reading of 29.4 feet at the peak, within six feet of the top according to the Arizona Republic account. Undoubtedly the functioning of the dam prevented a great deal of damage.

Returning to consideration of dependence upon the irrigation system for storm drainage, there is also the point that there are only infrequent places where it is possible to discharge street drainage into the canals since the irrigation system is purposely built high so it can deliver water to the adjoining land. The waste ditch system is not nearly so complete as the supply system, and in many places it has practically disappeared due to non-use. Finally the irrigation system diminishes in capacity going downstream, which is the opposite of what a storm drainage system should do.

These considerations all weigh against the use of the irrigation system for storm drainage, but there are a few exceptions. The most notable is the Old Cross-Cut Canal which runs between the Arizona and Grand Canals near 48th Street. This canal is in a position to intercept all the drainage east of it and can readily be improved to give it adequate capacity for this purpose. It is still utilized by the Water Users for shunting water from the Arizona Canal to the Grand, but it would be relatively simple to install works permitting it to be used for both purposes.

D. Surface Drainage on Street Pavements

Street pavements have an important role in storm drainage, but it is a sharply limited one. The capacities of streets as water carriers are shown in Figure 23. It will be seen that pavements with high crown and low "roll-type" curbs so commonly being built today have a poor rating as water carriers. Streets with flat cross-sections or inverted crowns have their disadvantages in other respects, but they do make good waterways and should certainly be considered in some cases. Figure 24 illustrates the capabilities of inverted crown streets.

In general, street pavement should be depended upon for drainage only in the outer reaches of a drainage area, say for the first half mile or so, depending upon the slope obtainable and the pavement cross section. As soon as cumulative runoff along the drainage channel exceeds the capacity of the street, pipelines should be utilized. Pipe, because of its smoothness and low cross-sectional perimeter to area ratio is a much more efficient water carrier than a natural channel or street pavement. Pipe is obtainable in many sizes up to 120 inches in diameter. Parallel lines of pipe in smaller sizes may sometimes be more advantageous than a single large line, if there are space limitations, though normally increase in pipe size gives more capacity for less dollars per unit of flow.

E. Division of Responsibility for Providing Storm Drainage Facilities

It is usually difficult for the favorably situated landowner to see that he is partially responsible for the plight of the person whose property is frequently subjected to floods. There is, however, a well-established precedent for considering flood protection a community responsibility. This precedent has a dual basis. First, there is the principle that the benefits resulting from a local improvement are not confined to that locality but are felt to a lesser degree throughout the community if in no other way at least by the increase in or maintenance of taxable values. Secondly, the paving of streets and construction of buildings high in a watershed must definitely increase the amount and rate of runoff in the lower portions. The benefit of storm drainage or flood control lies in the alleviation of a nuisance or the mitigation of an evil and is in this sense a negative sort of thing. Like fire insurance or national defense, however, it may still be considered worthwhile.

If storm drainage is a matter of both local and community interest, the question of allocation of the costs of drainage facilities arises. In attempting to establish a policy by which such an allocation may be made, it is convenient to make a classification of lands as follows:

I. Areas not presently and never likely to be flooded.

   A. Inside the city.

   B. Outside the city.

II. Areas in danger of flooding.

A. Inside the city.

1. Where no adequate provision for local storm drainage has ever been made.

2. Where adequate local drainage has been provided but this has been, or is likely to be overloaded by new areas contributing from above.

3. Areas adequately drained.

B. Outside the city.

1. Where no adequate provision for local storm drainage has ever been made.

2. Where adequate local drainage has been provided but this has been, or is likely to be overloaded by new areas contributing from above.

3. Areas adequately drained.

The term "adequate drainage", as used in this classification will be understood to mean facilities for removal of storm water having capacity sufficient for a one-year storm from urban or future urban areas and for a 25 to 100 year storm from mountainous areas. The term "local drainage" will be construed as meaning facilities for conducting storm runoff for the first half mile of its travel in streets, easements, or other public rights of way. The reason for adopting the half-mile limit is because generally facilities for carrying storm water this far can be installed under the Improvement Act without throwing unduly heavy assessments on the property owners; because, in the first half mile, two-year flows are often low

enough that they can be carried in the street pavements; and finally, because there is the precedent established in the canal system of the Salt River Project, which was built to bring water to the high point of each quarter section, that is, within one half mile of every property.

It is suggested that a policy something like the following be adopted for allocating costs of storm drainage works to the lands previously classified:

For the costs of local drainage as defined above

Class IIA1 lands - by City Improvement District.

Class IIB1 lands - by County Improvement District. All costs to be assessed against benefitting property in the district.

For the major works which serve as the link between local drainage works and final disposal to the Salt River:

Class IA and IIA lands - by City general obligation financing.

Class IIB1 lands which utilize City outfall lines - by County Improvement District providing for cash payment to the City for rights to the use of City lines.

For the costs of relief drains in City area once but no longer adequately drained, that is for Class IIA2 lands, allocated to Class IA and IIA lands by City general obligation bond issue or City budgeted tax funds.

This scheme provides no relief for Class IIB2 lands nor does it provide means of assessing part of the cost of storm drainage against land in Class IB. Neither of these categories is a City responsibility,

however, and it appears that one practicable means of incorporating these areas in a storm drainage program might be the formation of a City-County Flood Control District. This is a problem to be studied by a Flood Protection Improvement Committee as discussed later in this report. If, as is recommended, the County participates in the improvement of the Old Cross-Cut Canal and in the purchase of right of way for an Arizona Canal floodway a means is thereby provided for allocation of part of the cost to land falling into Class IB.

V. THE PROPOSED STORM DRAINAGE SYSTEM

The previous three sections have dealt with the bases for computation of storm water runoff rates for any particular drainage area. It is the purpose of this section to present the results of the computations that were made in accordance with the principles discussed; to show the cumulative flows resulting from the succession of drainage areas which contribute runoff along the various major drainage channels; and to recommend steps to be taken immediately and at later dates, to handle these flows in a manner which will reduce property damage and nuisance resulting from them.

A. Design Flows

In Section III the following formula was developed for the computation of the runoff rate to be expected from any individual drainage area:

$$Q = 0.9A_i\ (I_a - 0.20) + 0.8A_p\ (I_a - f_c)$$

Where

Q = surface water runoff rate in cubic feet per second (sometimes simply called second-feet).

A_i = impervious portion of the drainage area in acres.

I_a = rainfall intensity in inches per hour (average for the area).

A_p = pervious portion of drainage area in acres.

f_c = infiltration rate in inches per hour.

The value used for rainfall intensity is taken from the design intensity, duration, frequency relationship, Figure 9, entering it with the frequency consistent with the degree of protection decided upon and with a duration equal to the time of concentration. The value from Figure 9 can be reduced for large areas by a factor taken from Figure 18 because of the fact that total depth of rainfall (and intensity by inference) has been found to vary inversely with the area covered by the storm. The value of f_c is chosen from Figure 19 by taking into consideration the soil or soils occurring in Figure 20. It may, as has been pointed out, be modified arbitrarily to allow for the effect of the character of the district as to irrigation practices and other factors.

By use of this formula and the methods discussed, the flows to be expected in Phoenix were computed and are plotted on a flow sheet, Exhibit VI. For the flat lands where flow does not follow well defined channels, it has been assumed it will follow artificial waterways generally spaced one-half mile apart. In mountainous areas where definite channels exist, the assumption is that they will be allowed to continue to function.

In Exhibit VI the runoff factors assigned to each drainage area (or each quarter section in the flat lands) are tabulated in small figures in the following order.

1. total area of basin in acres (not stated in flat areas).
2. impervious area in acres.
3. pervious area in acres.
4. average infiltration rate in inches per hour.

In the mountainous areas a fifth figure follows which is the peak runoff rate at the outlet of the drainage area in cubic feet per second. In the flat lands this quantity is shown in heavy figures along the assumed flow path. Here it represents the cumulative amount to be expected at the point where the figure is written.

These runoff factors represent conditions as they are expected to be under ultimate development. Exhibit V shows what has been assumed in this respect for the purposes of these calculations. It is not represented to be an accurate prognostication for any particular area but generally representative of what the Phoenix area will be if present trends of growth are continued.

In computing concentration time for the purpose of arriving at the optimum storm duration the flow charts, Figures 23, 24, and 25, and the Contour Map, Exhibit III, were used. Velocities obtained in collection by the street or pipe system at the slopes existing in the area under consideration yield the concentration time used in the tables.

The calculation of runoff rates shown in Exhibit VI was done in tabular form. These calculations show the flows to be expected from a storm such as is likely to occur on the average of once every two years and collected by a system of drains spaced one half mile apart covering the area between the Arizona Canal and the Salt River, and between 48th Street and the Agua Fria divide. They also show flows from a two-year storm as might occur in the street system of South Phoenix flowing from the Highline Canal north to the Salt River. Finally, they show runoff rates from hundred-year storms on the drainage areas east of the Old

Cross-cut Canal, north of the Arizona Canal out of the Phoenix Mountains, and south of the Highline Canal out of the Salt River Mountains. The 100-year flows out of the mountainous areas are shown in Exhibit VI only for each individual basin but the calculation sheets show certain possible combined flows.

The typewritten calculations comprise 60 pages and are too bulky for inclusion in this report. They have been given to the City Engineer for reference purposes. Certain sheets are incorporated in this report, however. The sheet immediately following this page is the index sheet of the calculations. It is included to show the scope of the work done and to indicate what comparative analyses were made with changes of criteria. For example, the variation of flows with different recurrence intervals was investigated, also the effect on flows of a change in concentration time due to collection and transmission of flood waters on street pavements rather than in pipe lines. Other investigations were carried out to discover the effect of draining strips one mile wide rather than the half-mile widths shown in Exhibit VI, and to determine flows to be expected under existing conditions of surface development. These are all contained in the calculations filed with the City Engineer but are not shown on Exhibit VI. The computation sheets for the 7th Street line are included herein to illustrate the method used.

The procedure of computing runoff is fairly clear from the headings on the computation sheets. The method is to begin at the upper end of a proposed drainage line entering the acreages of impervious and pervious surface in the first drainage area considered in the proper columns.

INDEX OF FLOW COMPUTATION SHEETS

All computation sheets are based upon 'Draining through Collector Pipe, ½ mile width, 2 year Storm' unless otherwise noted.

1. 44th Street - Arizona Canal to River.
2. 40th Street - Arizona Canal to River.
3. 36th Street - Arizona Canal to River.
4. 32nd Street - Arizona Canal to River.
5. 32nd Street - Arizona Canal to River. Trials of 1 year and 5 year Rainfall Intensity for comparison Purposes.
6. 32nd Street - Arizona Canal to River. Alternate Presuming 5 thru streets were constructed to carry water to River - Total Design flow shown.
7. 28th Street - Arizona Canal to River.
8. 24th Street - Arizona Canal to River.
9. 20th Street - Arizona Canal to Salt River.
10. 20th Street - Arizona Canal to Salt River (Existing Conditions - Collecting on streets principally)
11. 16th Street - Arizona Canal to River.
12. 12th Street- Arizona Canal to River.
13. 7th Street - Arizona Canal to River.
14. 7th Street - Grand Canal to River - Alternate Calculation.
15. 7th Street - Grand Canal to River (Existing Conditions - Collecting on Streets)
    7th Street - Ariz. Canal to River - Alternate Calculation based on Draining 1 mile width, or to 16th Street.
16. 7th Street - Ariz. Canal to River - Alternate Calculation based on Draining 1 mile width, or to 16th Street (Cont'd.)
17. 7th Street - Draining 1 mile to East - and Running from Arizona Canal to River, Rainfall of 1 year intensity.
18. 7th Street - Draining 1 mile to East - from Grand Canal to River (1 yr. rainfall intensity)
19. Central Ave. - Arizona Canal to River. (7th Ave. at River via 5th & 3rd to Thomas & Central)
20. Central Ave. (Cont'd.)
21. 7th Ave. Arizona Canal to River (via 11th Ave. from River to VanBuren, via 9th Ave. to McDowell)
22. 7th Ave. (Cont'd.)
23. 15th Ave. Bethany Home Road to River - Branches on 11th and 17th Avenues.
24. 15th Avenue (Cont'd.)
25. 19th Ave. - Arizona Canal to River (North of Bethany Home Road Collects 11th & 17th Ave.) Branches on 11th and 17th Avenue.
26. 19th Ave. (Cont'd.)
27. 23rd Ave. - Grand Canal to River (via 25th near River to Roosevelt)
28. 27th Ave. - Arizona Canal to River (North of Grand Canal Collects from 19th & 23rd Ave.) Branches on 19th Ave. & 23rd Ave.
29. 27th Avenue (Cont'd.)
30. 31st Ave. - Arizona Canal to River (via 31st., 29th, 31st, 27th, and 29th Avenues)
31. 31st Ave. (Cont'd.)
32. 35th Ave. Orangewood To River.
33. 35th Ave. (Cont'd.)
34. 39th Ave. - Arizona Canal to River.
35. 39th Ave. (Cont'd.)
36. 43rd Ave. Divide to River.
37. 43rd Ave. (Cont'd.)
38. 47th Ave. Divide to River
39. 47th Ave. (Cont'd.), Trial for 47th Ave. from Grand Canal to River.
40. 47th Ave. Grand Canal to River (Cont'd.)
41. 51st Ave. - Divide to River.
42. 51st Ave. (Cont'd.)
43. 55th Ave. - Divide to River.
44. 55th Ave. - (Cont'd.)
45. 59th Ave. - Divide to River.
46. 59th Ave. (Cont'd.)
47. 67th Ave. - Divide to River.
48. 67th Ave. (Cont'd.)
49. 75th Ave. - Divide to River.
50. 75th Ave. - (Cont'd.)
51. 83rd Ave. - Divide to River.
52. 91st Ave. - Divide to River.
53. 91st Ave. to 131st Ave. - Divide to River. The Maximum Expected Flows for Mile Roads from Divide to River.
54. Areas East of old Cross-Cut Canal (48th St.) Comparison with Present Day or Present Conditions Flow)
55. Areas North of Arizona Canal - (Future conditions - 100 yr. Storm) Assumed carried west in New Channel)
    Cave Creek Drainage (Estimates) Regulated Discharge from Dam (186 square miles)
56. Areas North of Arizona Canal (Present Conditions - Assumed Carried West at 20 minutes per mile - 2 yr. Rain unless shown)
    Cave Creek Drainage - Estimates - Present Conditions - Regulated Discharge from Dam (186 square miles.)
57. South Phoenix Drainage Areas (So. Mts.) 100 yr. Rainfall Intensity - Cumulative Flows Based on Drainage being carried West from 48th St. in a Channel along the Highline Canal at a Rate of 16 minutes per mile. Individual Flows are without added Channel Collection Time.
58. South Phoenix - Assumed Carried in Streets - Each ½ mi., Inverted Crown Roads - So. 24th St. Western Canal to River. South 7th Street.
59. General Trials. 1 Year Rainfall Intensity - Average Street - 1 mile long.
60. Existing System - Heart of Phoenix or Downtown Area. 7th Avenue at McDowell.

DEMONSTRATIONS AND COMPARISONS

Page	
3	36th St. Comparison 5 year flows.
5	32nd St. Comparison 1 & 5 year flows, including pipe.
6.	32nd St. Comparison - Drainage by Street system only.
10.	20th St. Flows for existing conditions.
14.	7th St. Comparison or alternate - Grand Canal to River.
15.	7th St. Flows Grand Canal to River on Existing Streets.
15.	7th St. Comparison draining 1 mi. East instead of ½ mi. Arizona Canal to River.
17.	7th St. Comparison draining 1 mi. East instead of ½ mi. 1 yr. rainfall - Both Ariz. Canal to River & Grand Canal to River.
20.	Central - Approximate loading or flows from impervious areas adjacent to Streets only.
39.	47th Ave. Trial for 47th Ave. from Grand Canal to River.
54.	Areas East of old Cross-Cut - Comparison with Present Day or Present Conditions - Various Storms.
56.	Areas North of Arizona Canal - Present Conditions - Various Storms.
59.	General Trials , 1 Yr. Rainfall Intensity - Average Street 1 mi. long.
60.	Existing System, Heart of Phoenix or Downtown Area - 7th Ave. above McDowell.

Sheet 1 of 6 sheets.

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$IaxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$IaxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
7TH ST — ARIZ CANAL TO SALT RIVER															
Northern Ave.		35	15	20	1.5	.008	15	1.73	1.70	0.16	3	1.35	20	23	20
Orangewood		95	35	60	1.0	.005									
	Sum.	130	50	80	1.1		20	1.47	1.44	0.27	22	1.12	56	78	80
Glendale		160	50	110	1.6	.007									
	Sum.	290	100	190	1.4		26	1.25	1.20	0	(10)	0.90	90	100	100
Maryland		160	60	100	1.7	.006									
	Sum.	450	160	290	1.5		32	1.10	1.04	0	-	0.76	121	121	130
Bethany Home Road		160	60	100	1.8	.006									
	Sum.	610	220	390	1.6		37	1.00	0.94	0	—	0.67	146	146	160
Missouri		160	70	90	1.8	.006									
	Sum.	770	290	480	1.6		42	0.92	0.85	0	-	0.58	168	168	175
Camelback		160	70	90	1.8	.006									
	Sum.	930	360	570	1.6		47	0.84	0.78	0	-	0.52	187	187	195
Campbell		170	70	100	1.4	.0045									
	Sum.	1100	430	670	1.6		52	0.78	0.72	0	-	0.47	202	202	215
Indian School		150	60	90	0.5	.003									
	Sum.	1250	490	760	1.4		57	0.74	0.68	0	-	0.43	212	212	225 (Minor increases due to small areas of low infiltration affect remaining values)
Osboun		160	60	100	0.5	.003									
	Sum.	1410	550	860	1.3		62	0.70	0.64	0	-	0.40	220		230
Thomas		160	40	120	0.8	.003									
	Sum.	1570	590	980	1.3		68	0.65	0.60	0-	—	0.36	212		235
Encanto		160	50	110	0.5	.0025									
	Sum.	1730	640	1090	1.2		74	0.61	0.56	0	-	0.33	212		240
McDowell		160	60	100	0.6	.0015									
	Sum.	1890	700	1190	1.2		81	0.56	0.51	0	-	0.28	196		245
Roosevelt		160	150	10	0.9	.0015									
	Sum.	2050	850	1200	1.2		89	0.53	0.48	0	-	0.25	212		250
Van Buren		160	140	20	1.0	.001									
	Sum.	2210	990	1220	1.2		97	0.50	0.45	0	0	0.23	228	228	255
Harrison		160	150	10	0.4	.0008									
	Sum.	2370	1140	1230	1.2		104	0.47	0.42	0	0	0.20	228	228	260

$$Q = .0.9 A_i (I_a - 0.20) + 0.8 A_p (I_a - f_c)$$

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

Sheet 2 of 6 sheets.

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
7TH ST - ARIZ CANAL TO RIVER- CONT.															
Buckeye Road		160	140	20	0.4	.0008									
	Sum.	2530	1280	1250	1.2		111	0.45	0.40	0	0	0.18	231	231	265
Durango		160	140	20	0.5	.0008									
	Sum.	2690	1420	1270	1.2	[illegible]	118	0.43	0.39	0	0	0.17	242	242	270
7TH ST - GRAND CANAL TO RIVER - ALTERNATE CALCULATION															
Indian School		150	60	90	0.5		30	1.15	1.11	0.49					
Osborn		160	60	100	0.5										
	Sum.	310	120	190	0.5	[illegible]	35	1.03	0.99	0.39	74	0.71	85	159	160
Thomas		160	40	120	0.8										
	Sum.	470	160	310	0.6		41	0.93	0.88	0.22	68	0.61	97	165	180
Encanto		160	50	110	0.5										
	Sum	630	210	420	0.6		47	0.84	0.72	0.15	63	0.53	111	174	
	Trial Maximum	330	100	230	0.6		26	1.3	1.25	0.52	120	0.94	94	214	210
McDowell		160	60	100	0.6										
	Sum.	790	270	520	0.6		54	0.77	0.72	0.10	52	0.47	127	179	220
Roosevelt		160	150	10	0.9										
	Sum.	950	420	530	0.6		62	0.70	0.65	0.04	21	0.40	168	189	225
Van Buren		160	140	20	1.0										
	Sum.	1110	560	550	0.6		70	0.63	0.58	0	-	0.34	191	191	230
Harrison		160	150	10	0.4										
	Sum.	1270	710	560	0.6		77	0.58	0.53	0	-	0.30	213	213	235
Buckeye Road		160	140	20	0.5										
	Sum.	1430	850	580	0.6		84	0.55	0.50	0	-	0.27	230	230	240
Durango		160	140	20	0.5										
	Sum.	1590	990	600	0.6		91	0.52	0.48	0	-	0.25	247	247	245

The above calculation would indicate saving of 3"+ on pipe Diameter (Grand Canal to river) if we could utilize the Grand Canal as storm drain.

Sheet 3 of 6 sheets.

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$IaxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$IaxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
TH ST -- GRAND CANAL TO RIVER (EXISTING CONDITIONS - COLLECTING ON STREETS)															
Indian School Road		150	40	110	0.5	.003	35	1.03	1.00	0.48	44	0.72	29	73	
Osborn		160	60	100	0.5	.003									
	Sum.	310	100	210	0.5		57	0.74	0.71	0.17	36	0.46	46	82	
Thomas		160	20	140	1.0	.003									
	Sum.	470	120	350	0.7		79	0.58	0.55	0		0.31	37	37	
Encanto		160	50	110	0.5	.0025									
	Sum.	630	170	460	0.65		94	0.51	0.48	0		0.25	42	42	
McDowell		160	60	100	0.6	.0015									
	Sum.	790	230	560	0.64		125	0.41	0.38	0		0.16	37	37	
Roosevelt		160	60	100	0.9	.0015									
	Sum.	950	290	660	0.68		155	0.35	0.33	0		0.12	35	35	
Van Buren		160	60	100	0.9	.001									
	Sum.	1110	350	760	0.71		185	0.31	0.29	0		0.08	28	28	
Harrison		160	70	90	0.6	.0008									
	Sum.	1270	420	850	0.7		220	0.26	0.24	0		0.04	17	17	
Buckeye Road		160	60	100	0.5	.0008									
	Sum.	1430	480	950	0.68		250	0.24	0.22	0		0.02	10	10	
Durango		160	60	100	0.5	.0008									
	Sum.	1590	540	1050	0.67		280	0.21	0.19	0		-	-		
7TH ST - ARIZ CANAL TO RIVER -- ALTERANTE CALCULATION BASED ON DRAINING 1 MILE WIDTH, OR TO 16TH ST+															
Northern Ave.		35	15	20	1.5		15	1.72	1.68	0.15	3	1.35	20	23	20
Orangewood		95	35	60	1.0		20	1.47	1.44	0.35	21	1.12	39	60	60
Glendale		270	85	185	1.4										
	Sum.	400	135	265	1.3		26	1.25	1.18	0	-	0.88	119	119	120 (Concentration time near same as per ½ mile width - heretofore.)
Maryland		320	115	205	1.35										
	Sum.	720	250	470	1.3		31	1.12	1.05	0	-	0.77	192	192	195
Bethany Home Road		320	115	205	1.7										
	Sum.	1040	365	675	1.45		36	1.02	0.95	0	-	0.68	249	248	250
Missouri		320	130	190	1.8										
	Sum.	1360	495	865	1.5		41	0.94	0.86	0	-	0.59	282	282	285

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

Sheet 4 of 6 sheets.

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
7TH ST — ARIZ CANAL TO RIVER — ALTERNATE CALCULATION BASED ON DRAINING 1 MILE WIDTH, OR TO 16TH ST — CONTINUED															
Camelback		320	135	185	1.8										
	Sum.	1680	630	1050	1.55		46	0.85	0.78	0	-	0.52	327	327	330
Campbell		320	135	195	1.6										
	Sum.	2010	765	1245	1.6		51	0.80	0.73	0	—	0.48	368	368	370
Indian School		310	115	195	0.5										
	Sum.	2320	880	1440	1.4		55	0.76	0.69	0	—	0.44	387	387	390
Osborn		320	125	195	0.6										
	Sum.	2640	1005	1635	1.3		60	0.71	0.64	0	-	0.40	402	402	405
Thomas		320	100	220	0.7										
	Sum.	2960	1105	1855	1.2		66	0.66	0.58	0	—	0.34	375	375	410
Encanto		320	105	215	0.6										
	Sum.	3280	1210	2070			72	0.63	0.56	0	-	0.32	388	388	415
Mc Dowell		320	115	205	0.6										
	Sum.	3600	1325	2275			79	0.57	0.50	0	—	-0.27	358	358	420
Roosevelt		320	300	20	0.8										
	Sum.	3920	1625	2295			86	0.54	0.47	0	-	0.24	390	390	425
Van Buren		320	280	40	1.0										
	Sum.	4240	1905	2335			94	0.52	0.45	0	-	0.22	420	420	430
Harrison		320	250	70	0.4										
	Sum.	4560	2155	2405			100	0.49	0.42	0	-	0.20	431	431	435
Buckeye		320	270	50	0.4										
	Sum.	4880	2425	2455			107	0.46	0.40	0	-	0.18	437	437	440
Durango		320	280	40	0.5										
	Sum.	5200	2705	2495			114	0.44	0.38	0	—	0.16	433	433	445

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	Pervious $InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	Impervious $InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
7th ST. - DRAINING 1 MILE TO EAST - AND RUNNING FROM ARIZONA CANAL TO SALT RIVER. Rainfall of 1 year intensity. Collecting entire side area by pipe system														
Northern Ave.	35	15	20	1.5		15	1.3	1.28	0	-	0.97	15		20
Orangewood	95	35	60	1.0										
Sum	130	50	80	1.2		21	1.05	1.03	0	-	0.75	38		40
Glendale Ave.	270	85	185	1.4										
Sum	400	135	265	1.3		27	0.90	0.84	0	-	0.58	78		80
Maryland	320	115	205	1.3										
Sum	720	250	470	1.3		32	0.81	0.76	0	-	0.50	125		120
Bethany Home Road	320	115	205	1.7										
Sum	1040	365	675	1.45		37	0.74	0.69	0	-	0.44	160		160
Missouri	320	130	190	1.8										
Sum	1360	495	865	1.5		42	0.68	0.63	0	-	0.39	193		190
Camelback	320	135	185	1.8										
Sum	1680	630	1050	1.55		47	0.63	0.58	0	-	0.34	214		210
Campbell (Grand Canal)	330	135	195	1.										
Sum	2010	765	1245	1.6		52	0.58	0.52	0	-	0.29	222		225
Indian School Road	310	115	195	0.5										
Sum	2320	880	1440	1.4		56	0.56	0.51	0	-	0.28	247		250
Osborn	320	125	195	0.6										
Sum	2640	1005	1635	1.3		62	0.53	0.48	0	-	0.25	251		255
Thomas	320	100	220	0.7										
Sum	2960	1105	1855	1.2		67	0.49	0.44	0	-	0.22	243		260
Encanto	320	105	215	0.5										
Sum	3280	1210	2070	- -		73	0.46	0.41	0	-	0.19	230		265
McDowell Road	320	115	205	0.6										
Sum	3600	1325	2275	- -		79	0.43	0.38	0	-	0.16	212		270
Roosevelt	320	300	20	0.8										
Sum	3920	1625	2295	- -		86	0.41	0.36	0	-	0.14	228		275
Van Buren	320	280	40	1.0										
Sum	4240	1905	2335	- -		93	0.38	0.33	0	-	0.12	229		280

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$IaxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$IaxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
7TH ST. - DRAINING 1 MILE TO EAST (Cont'd.)								(One Year)							
Harrison		320	250	70	0.4										
	Sum	4560	2155	2405	- -		100	0.37	0.32	0	-	0.11	236		285
Buckeye Road		320	270	50	0.4										
	Sum	4880	2425	2455	- -		107	0.35	0.30	0	-	0.09	218		290
Durango		320	280	40	0.5										
	Sum	5200	2705	2495	- -		114	0.33	0.28	0	-	0.07	189		295
To River															
7TH ST. - DRAINING 1 MILE TO EAST - FROM GRAND CANAL TO RIVER. One Year Rainfall Intensity.															
Indian School		120	90	130	0.5		26	0.93	0.91	0.33	43	0.64	57	100	100
Osborn		320	125	195	0.6		- -								
Thomas		320	100	220	0.7		- -								
	Trail Max. Sum	540	190	350	0.6		20	1.10	1.04	0.35	122	0.76	144	266	260
McDowell Road		640	220	420	0.6										
	Sum	1180	410	770	0.6		30	0.84	0.78	0.14	57	0.52	213	270	270
Van Buren		640	580	60	0.9										
	Sum	1820	990	830	0.6		52	0.59	0.53	0	-	0.30	297	297	300
Buckeye Road		640	520	120	0.4										
	Sum	2460	1510	950	0.6		67	0.50	0.45	0	-	0.22	332	332	330
Durango		320	280	40	0.6										
	Sum	2780	1790	990	0.6		75	0.46	0.41	0	-	0.19	331	331	335

This comparison indicates we would get identical values of flow below Thomas whether starting at Grand Canal or Arizona Canal. The showing of less flow for the alternate starting at Arizona Canal would be increased to same values as from Grand Canal had we used correct trials for maximum area draining in least time.

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$IaxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$IaxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
AREAS EAST OF OLD CROSS-CUT CANAL (48TH ST.)														
North of Arizona Canal that could drain to X- Cut	1330	430	900	1.0		40	100 yr. 2.5	2.30	1.04	936	1.89	830	1766	Assume made ditch along canal.
Part of Secs. 28, 29, 33	1260	380	880	1.3	2¼ .007	52	100 yr. 2.1	1.94	0.51	450	1.56	590	1040	
Sum	2590	810	1780	1.2		52	100 yr. 2.1	1.89	0.55	980	1.52	1230	2210	
Part of Secs. 32, 33	550	110	440	0.6+	1.3 .015	28	100 yr. 3.2	3.02	1.93	850	2.54	280	1130	
Sum	3140	920	2220	1.1		60	100 yr. 1.9	1.7	0.48	1060	1.35	1240	2300	
Sec. 5 and adjacent	1180	240	940	0.6+	2¼ .016	40	100 yr. 2.5	2.3	1.36	1270	1.89	450	1720	
Sum	4320	1160	3160	1.0-		72	100 yr. 1.67	1.47	0.38	1200	1.14	1320	2520	
COMPARISON WITH PRESENT DAY OR PRESENT CONDITIONS FLOW														
Areas as above.	1330	200	1130	1.0		50	1 yr. 0.6	0.55	0	0	0.32	64	64	1 yr.
	1260	120	1140	1.3		60	1 yr. 0.53	0.49	0	0	0.26	31	31	1 yr.
Sum	2590	320	2270	1.1		60	1 yr. 0.53	0.48	0	0	0.25	80	80	1 yr.
							2 yr. 0.71	0.64	0	0	0.40	128	128	2 yr.
	550	50	500	0.3		30	1 yr. 0.84	0.79	0.39	195	0.54	27	222	1 yr.
Sum	3140	370	2770	0.9		65	1 yr. 0.5	0.44	0	0	0.22	82	82	1 yr.
							1000 yr. 2.25	2.00	0.88	2440	1.62	600	3040	1000 yr.
	1180	70	1110	0.3		45	1 yr. 0.65	0.60	0.24	265	0.36	25	290	1 yr.
							1000 yr. 2.9	2.7	1.92	2130	2.0	140	2270	1000 yr.
Sum	4320	440	3880	0.75		75	1 yr. 0.45	0.40	0	0	0.18	79	79	
							2 yr. 0.60	0.53	0	0	0.30	132	132	2 yr.
							100 yr. 1.67	1.47	0.57	2210	1.14	500	2710	100 yr.
							1000 yr. 2.02	1.78	0.82	3180	1.42	625	3805	1000 yr.

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
AREAS NORTH OF ARIZONA CANAL - (FUTURE CONDITIONS - 100 YEAR STORM) ASSUMED CARRIED WEST IN NEW CHANNEL)														
48th to 40th Streets	1080	260	820	1.0		50	2.17	2.02	0.81	660	1.63	423	1083	100 yr.
40th to 37th St. (Tatum)	3020	620	2400	1.2		65	1.81	1.61	0.33	790	1.27	786	1576	100 yr.
Sum	4100	880	3220	1.15		65	1.81	1.61	0.37	1190	1.27	1120	2310	100 yr.
37th to 32nd	870	170	700	0.6		36	2.7	2.5	1.52	1060	2.07	350	1410	100 yr.
Sum	4970	1050	3920	1.05		80	1.57	1.38	0.26	1040	1.06	1110	2150	100 yr.
32nd to 21st	760	150	610	0.6		45	2.35	2.2	1.28	780	1.80	270	1050	100 yr.
Sum	5730	1200	4530	1.05		110	1.25	1.07	0.02	90	0.78	936	1026	100 yr.
21st to 16th	790	200	590	0.6		45	2.35	2.2	1.28	755	1.80	360	1115	100 yr.
Sum	6520	1400	5120	0.9+		130	1.12	0.95	0.03	150	0.68	950	1000	100 yr.
16th to 12th	1020	310	710	0.7		40	2.52	2.34	1.31	930	1.93	600	1530	100 yr.
Sum	7540	1710	5830	0.9-		141	1.04	0.87	0	-	0.60	1030	1030	100 yr.
12th to 10th (Shea)	1860	620	1240	0.8		50	2.17	1.97	0.94	1170	1.59	990	2160	100 yr.
Sum	9400	2330	7070	0.9-		159	0.95	0.78	0	-	0.53	1230	1230	100 yr.
10th St. from East and North	1710	600	1110	0.9		56	2.00	1.82	0.74	820	1.46	870	1690	100 yr.
(Cave Creek Road) Sum	11110	2930	8180	0.9-		160	0.95	0.77	0	-	0.51	1490	1490	100 yr.
10th St. to 3rd Avenue	1280	380	900	0.9		60	1.91	1.76	0.69	620	1.40	530	1150	100 yr.
(Sunnyslope) Sum	12,390	3310	9080	0.9-		185	0.86	0.68	0	-	0.43	1420	1420	100 yr.
3rd Avenue to 19th Avenue	1620	560	1060	1.0		60	1.91	1.75	0.60	636	1.39	780	1416	100 yr.
Sum	14,010	3870	10,140	0.9		205	0.79	0.62	0	--	0.38	1450	1450	100 yr.
CAVE CREEK DRAINAGE (ESTIMATES) REGULATED DISCHARGE FROM DAM (186 SQUARE MILES)													1600	
Lower 3 miles	7 mi.^{2}	1500	3000	1.2		60	100 yr 1.9	1.67	0.37	1110	1.32	1980	3090	100 yr.
Lower 6 miles (Moon Valley)	14 mi.^{2}	3000	6000	1.2		100	100 yr 1.33	1.10	0	-	0.81	2430	2430	100 yr.
Area below Dam	30 mi.^{2}	6000	13,300	1.2		140	100 yr 1.05	0.79	0	-	0.53	3180	3180	100 yr.
Same but 2 yr. Storm	30 mi.^{2}	6000	13,300	1.2		140	2 yr 0.38	0.29	0	-	0.08	480	480	2 yr.
Only future paving	30 mi.^{2}	3000		1.2		140	100 yr 1.05	0.79	0	-	0.53	1590	1590	100 yr.

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	Pervious IxA_p =CFS	Impervious $(Ia-0.2)0.9$ = Inches	Impervious IxA_i =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
AREAS NORTH OF ARIZONA CANAL (PRESENT CONDITIONS - ASSUMED CARRIED WEST AT 20 MINUTES PER MILE - 2 YEAR RAIN UNLESS SHOWN)														
48th to 40th Street	1080	160	920	0.8	1.3 .025	50	0.81	0.75	0	0	0.50	80	80	
40th to 37th St. (Tatum Blvd.)	3020	300	2720	0.8	3.4 .008	65	0.67	0.60	0	0	0.36	108	108	
Sum	4100	460	3640	0.8+		65	0.67	0.59	0	0	0.36	165	165	
37th to 32nd St.	870	10	860	0.5	1.5 .025	36	100 yr 1.81 1.0	1.61 0.93	0.64 0.34	2330 290	1.27 0.66	580 6	2910 296	100 yr.
Sum	4970	470	4500	0.72		80	0.58	0.51	0	0	0.28	132	132	
32nd to 21st St.	760	10	750	0.4	1.0 .03	45	0.87	0.82	0.33	246	0.56	6	252	
Sum	5730	480	5250	0.70		110	0.45	0.38	0	0	0.16	77	77	
21st to 16th Street	790	40	750	0.5	1.4 .025	45	0.87	0.82	0.26	195	0.56	22	217	
Sum	6520	520	6000	0.65		130	0.40	0.34	0	0	0.13	67	67	
Same - 100 yr. Storm	790	40	750	0.5		45	100 yr 2.15	2.02	1.21	900	1.64	66	966	100 yr.
Same - 100 yr. Storm Sum	6520	520	6000	0.65		130	100 yr 1.12	0.95	0.24	1440	0.68	352	1790	100 yr.
16th to 12th St.	1020	100	920	0.6	2.0 .030	40	0.92	0.85	0.20	184	0.58	58	242	
Sum	7540	620	6920	0.65		141	0.38	0.32	0	-	0.11	68	68	
12th to 10th St. (Shea Blvd.)	1860	100	1760	0.6	2.8 .014	50	0.81	0.74	0.11	193	0.49	49	242	
Sum	9400	720	8680	0.63		159	0.34	0.28	0	-	0.07	50	50	
Same - 100 yr. Sum	9400	720	8680	0.63			100 yr 0.95	0.78	0.12	1040	0.52	375	1415	100 yr.
10th St. from East & North	1710	150	1560	0.7	3.0 .010	56	0.74	0.67	0	-	0.43	64	64	
(Along Cave Creek Road) Sum	11,110	870	10,240	0.64		160	0.34	0.27	0	-	0.06	52	52	
10th St. to 3rd Ave. (Sunny-slope)	1280	250	1030	0.9	2.0 .015	60	0.71	0.65	0	-	0.41	103	103	
Sum	12,390	1120	11,270	0.68		185	0.30	0.24	0	-	0.04	45	45	
3rd Ave. to 19th Ave.	1620	160	1460	0.8	1.8 0.15	60	0.71	0.65	0	-	0.41	66	66	
Sum	14,010	1280	12,730	0.7		205	0.28	0.22	0	-	0.02	26	26	
							100 yr 0.78	0.61	0	-	0.37	470	470	100 yr.
CAVE CREEK DRAINAGE - ESTIMATES - PRESENT CONDITIONS - Regulated Discharge from Dam (186 sq. miles)													1600	
Lower 3 miles	70 mi.	200	4300	1.4	3 .005	65	100 yr. 1.8	1.58	0.15	650	1.24	248	898	
Lower 6 miles (Moon Valley)	140 mi.	300	8700	1.0	6 .005	110	100 yr. 1.26	1.04	0.03	261	0.75	225	486	
Area below Dam	300 mi.	300	19,000	1.2	11 .006	160	100 yr 0.95	0.71	0	0	0.46	138	138	

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

SOUTH PHOENIX DRAINAGE AREAS (SOUTH MOUNTAINS) - 100 YEAR RAINFALL INTENSITY
CUMULATIVE FLOWS ARE BASED ON DRAINAGE BEING CARRIED WEST FROM 48TH STREET IN A CHANNEL
ALONG THE HIGHLINE CANAL AT A RATE OF 16 MINUTES PER MILE. INDIVIDUAL FLOWS ARE WITHOUT ADDED CHANNEL COLLECTION TIME.

LOCATION	AREA IN ACRES: Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time: Street Slope	Min. t_c	RAIN: Point Intensity I	Average Intensity Ia	RUNOFF: Pervious $(Ia-f_c)0.8$ = Inches	Pervious IxA_p =CFS	Impervious $(Ia-0.2)0.9$ = Inches	Impervious IxA_i =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
East of 48th Street	1510	-	-	0.5	4 0.05	43-	2.4	2.2	1.36		-	2060		
Near 40th Street	180			0.5	1.2 0.05	22-	3.57	3.47	2.37		-	427		
48th to 40th Street	620			0.5		22-	3.57	3.36	2.29		-	1420		
Sum	500			0.5		34+	2.75	2.6	1.68		-	840		
40th to 28th Street	700			0.5	0.8 0.16	14	4.8	4.51	3.21		-	2247		
Sum	1200			0.5		60	1.92	1.77	1.02		-	1220		
28th to 14th Street	1700			0.6	2.3 0.10	26	3.35	3.06	1.97		-	3350		
Sum	2900			0.6		86	1.48	1.32	0.58		-	1680		
14th to 8th Street	1120			0.5	1.7 0.11	21	3.75	3.47	2.37		-	2660		
Sum	4020			0.5		98	1.35	1.19	0.55		-	2210		
7th St. Area	560			0.4	1.8 0.12	21	3.75	3.54	2.51		-	1400		
Sum	4580			0.5		100	1.32	1.15	0.52		-	2380		
7th St. to 11th Ave. (Central)	1970			0.7	2.6 0.026	41	2.47	2.24	1.23		-	2420		
Sum	6550			0.6		130	1.12	0.95	0.28		-	1830		
11th Ave. to 19th Ave.	550			0.7	1.2 0.13	17	4.25	4.0	2.64		-	1450		
Sum	7100			0.6		140	1.06	0.89	0.23		-	1630		
19th Ave. to 27th	710			0.7	1.2 0.15	16	4.40	4.1	2.72		-	1930		
Sum	7810			0.6		158	0.96	0.80	0.16		-	1250		
27th Ave. to 35th	940			0.7	1.5 0.16	18	4.15	3.85	2.52		-	2370		
Sum	8750			0.6		180	0.87	0.72	0.10		-	875		
35th Ave. to 51st.	480			0.3	0.4 0.15	10	5.8	5.5	4.16		-	2000		
	9230			0.6		180	0.87	0.71	0.09			830		
Trial 11th Ave. to 35th	2200			0.7	-	55	2.02	1.81	0.89		-	1960		
Trial 7th St. to 19th Ave.	3080			0.7	-	57	1.97	1.73	0.83		-	2560		

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.6$ = Inches	$InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
SOUTH PHOENIX - ASSUMED CARRIED IN STREETS - EACH ½ MILE, INVERTED CROWN ROADS SO. 24TH ST. WESTERN CANAL TO RIVER														
Canal to Southern Ave.	220	70	150	0.9	.008	30	1.14	1.10	0.16	24	0.81	57	81	
Trial	120	40	80	0.9		17	1.62	1.58	0.55	44	1.28	51	95	100
Southern to Roeser	160	50	110	0.9	.006									
Trial Sum	330	100	230	0.9		27	1.22	1.16	0.21	48	0.86	86	134	130
Roeser to River	240	70	170	0.6	.002									
Sum	620	190	430	0.8		47	0.84	0.79	0	-	0.53	100	100	140
IF STORM SEWERS ARE BUILT THE ABOVE CONCENTRATION TIMES WILL BE REDUCED AND FLOWS INCREASED. IF IT IS POSSIBLE TO BUILD INVERTED CROWN ROADWAYS EVERY ¼ MILE (OR CLOSER) THESE STREETS SHOULD HANDLE THE EXPECTED 2 YEAR FLOWS.														
THE LOW AREA ALONG BROADWAY FROM 7TH AVENUE TO 24TH STREET GENERALLY WILL HAVE TO BE DRAINED WESTERLY BY OTHER MEANS SINCE THE RIVER BANKS ARE AS HIGH AS THE LAND.														
SOUTH 7TH STREET.														
Canal to Southern Avenue	250	80	170	0.9	.011	30	1.15	1.11	0.17	29	0.82	65	94	100
Trial	115	35	80	0.9		20	1.46	1.43	0.42	34	1.11	39	73	-
Southern to Roeser	160	30	130	0.9	.008									
Trial Sum	205	55	150	0.9		23	1.35	1.30	0.32	48	0.99	54	102	120
Roeser to Broadway	160	60	100	0.6	.004									
Trial Sum	365	115	250	0.8		36	1.02	0.97	0.13	32	0.69	79	111	125
On Broadway - 7th to 5th	40	20	20	0.6	.001+									
Trial Sum	405	135	270	0.8		40	0.95	0.90	0.08	22	0.63	85	107	130
5th St. - Broadway to River	160	60	100	0.6	.001+									
Sum	565	195	370	0.7		60	0.71	0.67	0	-	0.42	82	82	135

EXPECTED FLOWS - 2 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$IaxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$IaxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
GENERAL TRIALS														Street Capacity
Average Street ½ mile long	30	10	20	0.8	.0005	59	0.72	0.71	0	-	0.47	5	5	5
					.001	42	0.91	0.90	0.06	1	0.64	6	7	7
					.002	31	1.12	1.11	0.25	5	0.83	8	13	10
					.003	26	1.25	1.23	0.34	7	0.94	9	16	12
					.005	21	1.42	1.40	0.48	10	1.10	11	21	16
					.010	16	1.67	1.64	0.67	13	1.32	13	26	22
Average Street 1 mile long	60	20	40	0.8	.001	80	0.56	0.55	0	-	0.32	6	6	7
					.002	58	0.73	0.72	0	--	0.48	10	10	10
					.003	48	0.83	0.82	.02	1	0.57	11	12	12
					.005	38	0.97	0.95	0.12	5	0.69	14	19	16
					.010	28	1.20	1.18	0.30	12	0.90	18	30	30

ABOVE INDICATES STREETS SHOULD HAVE CAPACITY TO DRAIN ½ MILE, 2 YR. FLOW. WOULD NOT BE TOO BAD AT 1 MILE. UNFORTUNATELY STREET CONSTRUCTION DOES NOT DIVIDE EQUALLY AND ALSO LOW AREAS MUST BE DRAINED BY PIPE. LAYOUT OF STREETS IS SUCH THAT ½ MILE STREETS ARE USUALLY CONTINOUS AND SYSTEM DESIGN BASED ON ½ MILE SPACING IS USED THROUGHOUT THE 2 YEAR FLOW CALCULATION SHEETS.

1 YEAR RAINFALL INTENSITY - AVERAGE STREET - 1 MILE LONG

Average Street	60	20	40	0.8	.001	80	0.43	0.42	0	-	0.20	4	4	
					.002	58	0.54	0.53	0	-	0.30	6	6	
					.003	48	0.62	0.61	0	-	0.37	7	7	
					.005	38	0.73	0.72	0	-	0.47	9	9	
					.010	28	0.88	0.86	0.05	2	0.59	12	12	

ABOVE INDICATES STREETS SHOULD HAVE CAPACITY TO DRAIN ONE YEAR STORM FOR 1 MILE.

Then follows assigning and entering a value of f_c, and computation of flow time from the most remote point along the shortest possible flow path along the street system. In these computations four minutes were added to street flow times to make allowance for overland flow in the most remote block and the total was entered in the column headed t_c in the form. Using this value of t_c as the duration, a value of intensity is taken from Figure 9, corrected to I_a by the factor from Figure 18, and the indicated multiplications are made to get runoff from both the pervious and impervious areas which are then added to get total flow which is entered in the next-to-last column on the right. The data for the next increment of drainage area downstream from the first are then entered in the first six columns of the form on the next line, sums of the areas and a weighted average for infiltration rate are taken, and the entire process is carried through again as if this new area were the first one in the basin.

Then this procedure is carried out down the length of a channel it frequently will happen that the computed total flow will reach a maximum and then actually fall off as the computation proceeds down-stream. This is due to the longer concentration time introducing lower rainfall intensity into the formula over-shadowing the effect of the increasing area. Once a maximum has been reached, a continuing slight increase in flow is still indicated in the column headed Design Flow even though the calculatons show a diminution. It is also true that a greater flow may develop from a portion of a drainage area under more intense rainfall than is indicated in Figure 9, for the concentration time (duration) of the entire area. This is apt to happen in heavily commercial or industrial areas and is also

likely to happen when the pipe or street flow time from areas upstream is about equal to the concentration time of the latest increment of drainage area. This is checked for in the computation wherever the words "Trial maximum @ t_c = ____ " occur. If it develops that the shorter concentration time yelds higher flows for a partial area than for the cumulative area, the shorter time and the smaller area are carried forward in the computations.

There is little information available which may be utilized to check the reasonableness of the flows shown in Exhibit VI. A few hydrographs which could be correlated with rain intensity gage records would have been invaluable and would have permitted an entirely different and more accurate method of analysis. The Water Users have recently set staff gages in Cave Creek and Indian Bend and appointed observers for them. It is to be hoped that this will result in the availability of runoff records from these streams. Recording gages should also be installed at the outlets of major city drains. A few of these records together with rainfall intensity records will be very helpful in designing future work.

There is some guidance to be obtained from the work of others. The 1935 storm drainage project utilized a 54-inch outfall in a system designed to accommodate the downtown square mile. The airport system utilizes two 60-inch lines to drain an area of about 1200 acres. The Water Users' reports on the storm of Aug. 3, 1943 and the estimates of flows from the desert washes for that storm by T.R. Neiswander are very valuable. Finally there is the work of William Anderson of the Soil Conservation Service which, although of a preliminary nature, contains estimates of 100-year flows from Cave Creek and the washes out of the Phoenix Mountains and evaluates

the potentialities of numerous storage dam sites.

Based on a consideration of the aforementioned computations, guides and preliminary estimates of costs, the following conclusions are reached:

(a) The 100-year recurrence interval should be used in design to take care of storm runoff for areas north of the Arizona Canal, east of the Old Cross-cut Canal, and south of the Highline Canal.

The potential damage to the city from a flood out of the mountains is enormous. This is true especially for Cave Creek. It is also true of Indian Bend Wash if water from this source follows the Arizona Canal westward over the divide, something which has already occurred and which might happen again.

The Old Cross-Cut Canal could readily be enlarged and improved to provide protection to the city from 100-year storms to the east of it without impairing its usefulness to the Water Users.

As the foothill zone of South Phoenix develops increasing amounts of runoff will be thrown on the flat lands. The situation here is better than in Phoenix proper because grades are steeper and there are few swales where water can collect in sufficient depth to be damaging. The only exceptions are the swale paralleling Broadway and other more minor ones nearer the Salt River.

Phoenix has flatter slopes and more places where water tends to collect. These swales are obvious from the contour map. They are aggravated by artificially created collection points behind the Grand Canal, the Black Canyon Highway, the railroads, and certain streets.

(b) A one-year recurrence interval should be the minimum used for the design of street system drainage. It is hoped that later installations on approximately the $\frac{1}{2}$-mile lines may work toward furnishing capacities sufficient to handle flows from the two-year recurrence interval storm.

It is impossible to show economic justification for designing for any longer recurrence interval, and no matter what reasonable interval is used there will always be big storms that will overtax the system, flooding streets and low-lying lands. Runoff from one and two-year storms comes principally from the street system itself plus immediately adjacent commercial areas which are largely paved, the intensity of rainfall being low enough from these storms that little or no runoff develops from unpaved areas.

The cost of building drains adquate for a one-year storm in the area between the Arizona Canal and the Salt River west of 48th Street and east of the Agua Fria divide would be between 25 and 30 million dollars. This is based on parallel collecting lines one mile apart. Drains to handle two-year storms would be spaced one-half mile apart and the total cost of protecting the area mentioned would be about 60 million dollars.

The estimates which are given in detail later are high enough to include the extension of drains into relatively small areas, even though this is not economically justified, for the purpose of abating a nuisance. There will of course be some saving in property damage and some in street cleaning and maintenance costs, but there will be the additional maintenance cost of the drains.

(c) The system should use pipe collectors north of the Salt River between 48th Street and 67th or 75th Avenue

The reason for this conclusion is that the street system is already established and not generally suited for adequate drainage to the river. There is little or no room left for open ditches in this area. The only possible exceptions are at the approaches to the Salt River where there are still some areas where the street system or open ditches might be used. In general ditches have high maintenance costs compared to pipe lines.

d) The north-south street system should provide through routes to the river wherever possible.

There is great economy in this measure. In many cases it would obviate the need of pipelines altogether because street pavements could be made to carry the smaller amounts of water. It would also lessen the cost of construction of new lines by diminishing conflict with other underground utilities of which there is an ever increasing number and which all seek out through routes themselves. Keeping at least the quarter-mile streets open can still be of some help in South Phoenix (where some pipe would still be necessary near the river because of adverse grades) and would eliminate the need for pipe drains altogether west of 67th Avenue.

e) Every old or natural waterway should be preserved.

Wherever it is still possible to do so, old waterways should be designated as such by the appropriate authority and any development which would impair their function prevented. This would not preclude the use of natural channels as street locations if the water carrying capacity were preserved. The opportunity for City action in this direction

has largely passed but the County can still do a lot of good. The Indian Bend Wasteway and the Sunnyslope washes are examples.

B. Recommendations for Immediate Work

The lines proposed for immediate construction are shown in Exhibit VII. The basic premise is to provide capacity ample only for runoff from the street system and adjacent commercial areas from the one-year storm with lines one mile apart. The computations upon which the selection of pipe sizes was based are given in pages 114 through 118. The computations were made for lines on 24th Street, 16th Street, 7th Street, 7th Avenue, and 19th Avenue. Calculations for other lines would follow the same pattern. The 7th and 19th Avenue lines collect runoff from drainage areas more than one mile wide, the limits of the drainage areas being shown on Exhibit VII.

It is proposed to build only the lines on 24th and 7th Streets and on 7th and 19th Avenues at this time. Sixteenth Street is omitted because it has been recently paved over much of its length. The 24th Street line is not yet urgently needed although it could advantageously be installed with a widening program. The 27th Avenue line will be required soon, but it may be possible to construct this line in conjunction with the Black Canyon Highway under a Highway Department project. The sizes shown for the various pipe lines are the very smallest that can be conscientiously recommended.

For the storm water flows north of the Arizona Canal, it is proposed that the right-of-way for future construction be acquired at this time

while it is still relatively undeveloped and obtainable at a reasonable cost. The following page shows cross-sections and hydraulic properties of an adequate floodway illustrating the width of right-of-way necessary. The actual construction of the drainage canal and appurtenant works may be deferred or left to others. Estimates of the cost of such work are presented later. Some construction to enlarge the Old Cross-Cut-Canal and make it suitable for a drainage way should be done now. This is shown in Exhibit VII and discussed later with the cost estimates. Work adjacent to the Arizona Canal and on the Old Cross-Cut-Canal would of course require cooperation with the Water Users.

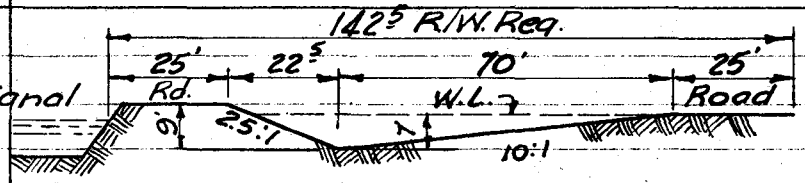
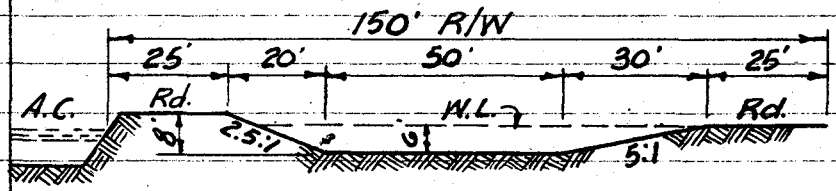
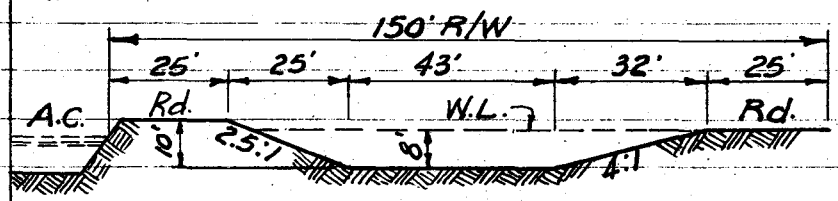
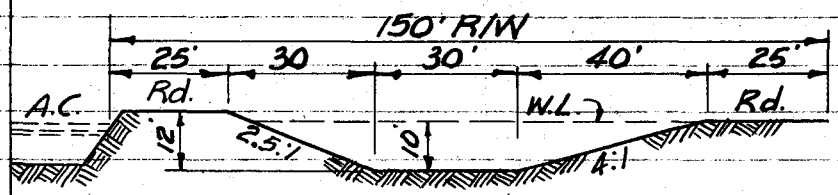
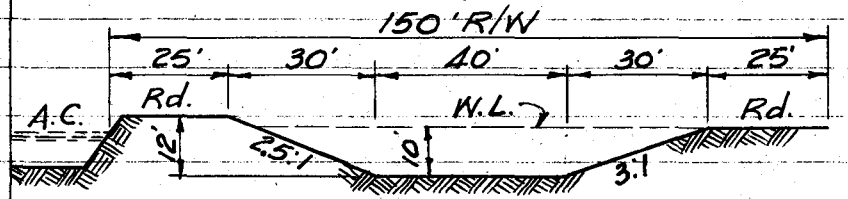
This report does not attempt to provide a detailed answer to the problem of handling runoff in the Sunnyslope, South Phoenix or South Mountain areas. It is in these areas that the storage solution is most likely to be the economical one. Existing arroyos, if preserved, can protect the area to the north without endangering downstream property if they can empty into an outlet such as the proposed floodway north of the Arizona Canal.

This report does not answer all the area's drainage problems and final solution can only be accomplished by community effort. It is therefor recommended that the Mayor appoint a committee having City, County, and Water Users representation. This Committee should study methods of financing, constructing, and operating major flood protection projects. The Committee could well consider alternatives such as

AREA DESCRIPTION **NORTH OF ARIZONA CANAL**

POSSIBLE DRAINAGE CHANNEL SECTIONS
PARALLELING THE ARIZONA CANAL

n·0.011 Big Concrete Culverts
n·0.012 Pipe Culverts 21" & Larger
n·0.015 Street Paving
Concrete Lined Waterway
n·0.020 Earth · Best
n·0.030 Earth · Brushy · Poor

Sta. or Location From-To	Waterway Description	Rough-ness =n	Slope Ft. Per 1000	Area Sq. Ft. =A	p = Wet Per r = A/P	Vel. Ft./Sec. =V	Quant. C.F.S. =Q
Ariz. Canal		0.015	0.4	306	3.4	4.4	1340
A.C.		0.015	0.4	435	4.7	5.5	2390
A.C.		0.015	0.4	562	5.7	6.0	3370
A.C.		0.015	0.4	625	6.3	6.5	4060
A.C.		0.015	0.4	675	6.8	7	4730

improvement districts, cooperatives, or other organizations having permanent status and the authority to finance and accomplish such work.

The County is in a position to take certain steps immediately which would have important favorable results. These are,

1. Declare Indian Bend a drainageway from the Cave Creek divide through the Scottsdale area to the Salt River and prohibit its obstruction.

2. Declare certain other waterways untenable.

3. Continue the present policy of setting building grades high enough to minimize flood damage.

4. Require subdividers to keep continuity of streets on quarter and midsection lines.

5. Participate in the work of the Flood Protection Improvement Committee.

6. Help finance work on the Old Cross-cut Canal and in obtaining right-of-way along the Arizona Canal.

EXPECTED FLOWS – 1 year rainfall intensity and duration unless noted

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	Pervious IxA_p =CFS	Impervious $(Ia-0.2)0.9$ = Inches	Impervious IxA_i =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
24th Street - Arizona Canal to River - Based on 1 year rainfall intensities - draining one mile width by street flow - and collector piping draining the street system and adjacent commercial (impervious) areas only.															
Camelback Road		535	70				27	0.90	0.85			0.58	41		40
Indian School Road		640	120												
	Sum	1175	190				41	0.70	0.65			0.40	76		80
Thomas Road		640	130												
	Sum	1815	320				53	0.57	0.52			0.29	93		95
McDowell Road		640	120												
	Sum	2455	440				66	0.50	0.45			0.22	97		105
Van Buren St.		640	120												
	Sum	3095	560				78	0.44	0.39			0.17	96		115
Buckeye Rd.		640	180												
	Sum	3735	740				93	0.39	0.34			0.13	96		125
Trial at Buckeye Draining in 30 min.		400	150				30	0.84	0.80			0.54	81		Not a maximum
Durango St.		160	30												
	Sum	3895	770				101	0.36	0.32			0.11	85		130
To River															
16th Street - Arizona Canal to River - Criteria as for 24th St.															
Bethany Home Rd.		330	60				22	1.02	0.98			0.70	42	42	40
Camelback Rd.		640	120												
	Sum	970	180				32	0.81	0.75			0.58	104	104	100
Indian School Rd.		640	140												
	Sum	1610	320				42	0.68	0.62			0.38	122	122	120
Thomas Rd.		640	130												
	Sum	2250	450				54	0.57	0.52			0.29	131		130
McDowell Rd.		640	130												
	Sum	2890	580				67	0.49	0.44			0.22	128		140

EXPECTED FLOWS - 1 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
16th Street - Continued														
Van Buren St.	640	300												
Sum	3530	880				80	0.44	0.39			0.17	150		160
Buckeye Rd.	640	460												
Sum	4170	1340				95	0.38	0.33			0.12	161		180
Trial at Buckeye Draining in 50 min.	960	570				50	0.60	0.56			0.32	182		A maximum
Durango St.	320	230												
Sum	4490	1570				103	0.36	0.31			0.10	157		185
To River														
7th Street - Arizona Canal To River - Criteria as for 24th St.														
Glendale Ave.	400	70				30	0.84	0.80			0.54	38		40
Bethany Home	640	120												
Sum	1040	190				42	0.68	0.63			0.39	74		80
Camelback	640	130												
Sum	1680	320				53	0.58	0.53			0.30	96		100
Indian School	640	140												
Sum	2320	460				65	0.50	0.45			0.22	101		110
Thomas	640	120												
Sum	2960	580				77	0.44	0.39			0.17	98		120
Mc Dowell	640	120												
Sum	3600	700				91	0.39	0.35			0.14	98		140
Van Buren	640	500												
Sum	4240	1200				106	0.35	0.31			0.10	120		200
Buckeye	640	500												
Sum	4880	1700				122	0.31	0.27			0.06	102		230 (See below)
Durango	320	230												
Sum	5200	1930				130	0.30	0.26			0.06	116		235
To River														
Trial At Buckeye Draining in 50 min.	900	700				50	0.60	0.56			0.32	224		A maximum

EXPECTED FLOWS - 1 year rainfall intensity and duration unless noted

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
7th Ave. - Arizona Canal To River - Based on 1 Year Rainfall intensities - draining 1½ mile widths above the Grand Canal and less widths below the canal by means of street flow and collector piping - draining the street system and adjacent commercial (impervious) areas only.															
Northern Ave.		740	150				34	0.78	0.73			0.48	72		70
Glendale Ave.		960	190												
	Sum	1700	340				46	0.64	0.59			0.35	119		120
Bethany Home		960	190												
	Sum	2660	530				58	0.54	0.48			0.25	132		130
Camelback		960	200												
	Sum	3620	730				69	0.47	0.42			0.20	146		145
Indian School		960	200												
	Sum	4580	930				80	0.43	0.38			0.16	148		155
Thomas		800	220												
	Sum	5380	1150				92	0.39	0.34			0.13	149		165
McDowell		640	200												
	Sum	6020	1350				104	0.35	0.30			0.09	122		175
Van Buren		160	130												
	Sum	6180	1480				120	0.32	0.27			0.06	89		185
Buckeye Rd.		160	130												
	Sum	6340	1610				134	0.29	0.25			0.05	81		195
Durango		80	60												
	Sum	6420	1670				141	0.28	0.24			0.04	67		200
To River															
Trial at Mc Dowell draining in 60 min.		1300	340				60	0.53	0.49			0.26	88		Not a maximum

EXPECTED FLOWS - 1 year rainfall intensity and duration unless noted

LOCATION		AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	$InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	$InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
19th Ave. - Arizona Canal To River - Based on 1 year rainfall intensities - draining 1¼ mile widths above the Grand Canal and 1 to 1 3/4 miles widths below the canal by means of street flow and collector piping - draining the street system and adjacent commercial (impervious) areas only.															
Northern Ave.		1080	200				36	0.76	0.70			0.45	90		90
Glendale		800	160												
	Sum	1880	360				48	0.61	0.56			0.32	115		115
Bethany Home		800	150												
	Sum	2680	510				60	0.53	0.48			0.25	128		130
Camelback		800	160												
	Sum	3480	670				72	0.47	0.42			0.20	134		145
Indian School		800	150												
	Sum	4280	820				83	0.41	0.36			0.14	115		160
Thomas		960	180												
	Sum	5240	1000				94	0.38	0.33			0.12	120		175
McDowell		1120	220												
	Sum	6360	1220				104	0.36	0.31			0.10	122		190
Van Buren		800	440												
	Sum	7160	1660				116	0.33	0.28			0.07	116		200 (See below)
Buckeye Road		720	320												
	Sum	7880	1980				128	0.31	0.26			0.05	99		210
Durango		320	200												
	Sum	8200	2180				135	0.29	0.24			0.04	87		215
To River															
Trial at Van Buren draining in 40 min.		720	500				40	0.70	0.65			0.40	200		A maximum

EXPECTED FLOWS - 1 year rainfall intensity and duration unless noted

LOCATION	AREA IN ACRES Total Area A	Imperv's Area A_i	Pervious Area A_p	Infiltr'n (final) in/hr f_c	Concentration Time Street Slope	Min. t_c	RAIN Point Intensity I	Average Intensity Ia	RUNOFF Pervious $(Ia-f_c)0.8$ = Inches	Pervious $InxA_p$ =CFS	Impervious $(Ia-0.2)0.9$ = Inches	Impervious $InxA_i$ =CFS	Total Flow CFS	DESIGN FLOW AND REMARKS
27th Avenue - Arizona Canal To River - Based on 1 year rainfall intensities - draining normally from 31st Avenue to Black Canyon Highway (3/4 mile width) by means of street flow and collector piping - draining the street system and adjacent commercial (impervious) areas only.														
Northern Avenue	560	100				40	0.70	0.66			0.41	41		40
Glendale Avenue	760	140												
Sum	1320	240				52	0.59	0.54			0.31	75		80
Bethany Home	480	100												
Sum	1800	340				64	0.51	0.46			0.23	78		110
Camelback	480	90												
Sum	2280	430				76	0.45	0.40			0.18	78		140
Indian School	480	90												
Sum	2760	520				88	0.40	0.36			0.14	72		160
Thomas	560	250												
Sum	3320	770				98	0.37	0.33			0.12	92		180
McDowell	520	300												
Sum	3840	1070				110	0.34	0.30			0.09	96		210 (See below)
Van Buren	320	130												
Sum	4160	1200				122	0.31	0.27			0.06	72		230
Buckeye	400	160												
Sum	4560	1360				135	0.29	0.25			0.05	68		240
Durango	240	70												
Sum	4800	1430				142	0.28	0.24			0.04	57		245
To River														
Trial at McDowell draining in 30 min.	640	400				30	0.85	0.80			0.54	216		A maximum

C. Future Work by City and by Others

The City should prepare for the installation of adequately sized collector lines on section line streets in connection with paving programs even if this results in temporarily unusable portions of the drainage system. It should also take the initiative in the formation of improvement districts for the purpose of providing connection to the new system from city areas which have never had adequate drainage.

The County should continue working on the following matters:

1. The Arizona Canal Floodway,
2. The definition of a spillway from Cave Creek Dam,
3. Channelization of the Salt River by cooperation with the Corps of Engineers, and
4. Further improvement of lower Cave Creek and of the mountain washes which menace developed areas.

D. Legal Considerations

It is recognized that there are legal questions involved wherever there is a diversion of storm water out of its normal drainage path. This applies to the utilization of the Old Cross-cut Canal as a floodway and particularly to the diversion of Phoenix Mountain runoff into the proposed Arizona Canal floodway. While legal problems are beyond the scope of this report, it is likely that a reasonable solution protecting the rights of all parties involved can be worked out.

VI COST ESTIMATES

Estimates presented in this section are based on present day prices for labor and materials. They include allowances for all appurtenances necassary to make a complete and working installation even though these items are not specifically mentioned. Moderate contractor's overhead and profit are assumed.

A. Collector Piping in City Areas

Estimates of cost per foot were made on pipe ranging in size from 21 to 96 inches. Unit costs are shown on the tabulation following. The column headed "Best Cost per Foot" represents the cost of laying pipe in streets where no pavement replacement is required and where there is no unusual conflict with other utilities. The usual condition in city streets will require pavement cut and replacement and moving a water main, gas line, or sewer, to permit installation of the storm drain. It is presumed that part of the repavement will require a concrete base. The very worst conditions which require cutting through concrete pavement and moving of parallel utility lines will cost more per foot than the cost shown in the column headed "Total Cost in Built-up Areas." In the tabular estimates which follow for the lines in 24th and 7th Streets, and 7th and 19th Avenues, some adjustment in unit costs was made to allow for the effect of local

peculiarities on construction costs.

Allowances must also be made for small lines to drain local low spots to these three major collectors. Much of this can be financed by local improvement districts and under street improvement programs. Costs of such work could vary widely, but the estimate includes money for some such projects in troublesome areas where construction under improvement district programs or paving projects is not to be considered.

The 27th Avenue line is not included in the estimate but this and other section-line drains should be installed prior to permanent pavement on these streets.

PIPE SIZE INCHES I.D.	EXCAVATION AND BACKFILL					PIPE COST PER LIN. FT.	INSTALLATION COST PER LIN. FT.	INTERSECTION COSTS		BEST TOTAL COST PER LIN. FT.		PARALLELING COST PER LIN. FT.		TOTAL COST IN BUILT-UP AREAS PER LIN. FT.		PIPE SIZE INCHES I.D.
	TRENCH WIDTH FT.	TRENCH DEPTH FT.	CU. YD. PER LIN. FT.	COST PER CU. YD.	COST PER LIN. FT.			INLET COST PER LIN. FT.	UTILITY X-ING COST PER LIN. FT.	TOTAL	USE	PAVEMENT CUT & REPL.	UTILITY RE-LOCATION	TOTAL	USE	
21	4.2	7.2	1.11	0.50	0.55	3.88	1.10	1.60	0.20	7.33	7.50	2.64	2.00	11.97	12.00	21
24	4.4	7.4	1.21	0.50	0.61	4.25	1.26	1.60	0.20	7.92	8.50	2.73	2.20	12.85	13.00	24
27	4.8	7.8	1.36	0.50	0.68	5.00	1.42	1.60	0.25	8.95	9.50	2.85	2.40	14.20	14.50	27
30	5.0	8.0	1.48	0.50	0.74	5.65	1.57	1.60	0.25	9.81	10.50	2.93	2.60	15.34	15.50	30
33	5.2	8.2	1.60	0.50	0.80	6.51	1.73	1.60	0.30	10.94	11.50	3.02	2.80	16.76	17.00	33
36	5.5	8.5	1.73	0.50	0.87	6.80	2.27	1.60	0.30	11.84	12.50	3.11	3.00	17.95	18.00	36
42	6.2	9.2	2.09	0.50	1.05	9.43	3.08	1.80	0.35	15.71	16.00	3.34	3.20	22.25	22.50	42
48	6.7	9.7	2.38	0.52	1.24	11.30	3.78	1.80	0.35	18.47	19.00	3.51	3.40	26.38	26.00	48
51											21.00				28.00	51
54	7.2	10.2	2.75	0.52	1.43	14.53	4.53	1.80	0.40	22.69	23.00	3.71	3.60	30.00	30.00	54
57											25.00				32.00	57
60	7.8	10.8	3.13	0.55	1.72	17.20	5.04	1.80	0.40	26.16	27.00	3.91	3.80	33.87	34.00	60
63											30.00				37.00	63
66	8.5	11.5	3.62	0.55	1.99	21.05	6.24	1.80	0.50	31.58	32.00	4.15	4.00	39.73	40.00	66
69											35.00				43.00	69
72	9.0	12.0	4.00	0.57	2.28	24.20	7.56	1.80	0.60	36.44	37.00	4.33	4.20	44.97	45.00	72
75											40.00				48.00	75
78	9.7	12.7	4.53	0.57	2.58	26.90	8.95	2.00	0.70	41.13	42.00	4.57	4.40	50.10	51.00	78
81											45.50				54.00	81
84	10.3	13.3	5.11	0.60	3.07	30.40	10.60	2.00	0.80	46.87	47.50	4.72	4.60	56.19	57.00	84
87											51.00				60.00	87
90	11.0	14.0	5.70	0.62	3.53	34.40	12.30	2.00	0.90	53.13	54.00	4.92	4.80	62.85	63.00	90
93											57.00				67.00	93
96	11.5	14.5	6.18	0.65	4.02	38.40	14.10	2.00	1.00	59.52	60.00	5.09	5.00	69.61	70.00	96

B. Work in Other Areas

No estimates were made for work in the South Phoenix and South Mountain areas.

The estimate for work on the Old Cross-cut Canal includes construction of a new outlet to the Salt River and the rebuilding of culverts at street crossings. The cost of work expected to be done by the Water Users for the benefit of the area and of the Water Users is not included. High water velocities in the canal may require drop structures or lining but the cost is not included. The Water Users may assume some or all of this cost since they would undeniably benefit. This matter would fall into the scope of inquiry and action of a Flood Protection Committee. The cost of the suggested improvements to the Old Cross-cut Canal is $228,000 which should be divided between City and County.

The estimate for work north of the Arizona Canal is $3,750,000 for the completed project. It is not suggested that this work be included in the immediate project or that funds for it be included in the bond issue. It is recommended that right-of-way be acquired at this time. The cost of land is estimated to be $360,000 which again should be divided between the City and County.

C. Summary of Costs

The cost estimate is given in some detail in the following pages but a summary follows.

1.	Pipe - 24th Street from the Salt River to the Grand Canal	$ 713,000
2.	Pipe - 7th Street from the Salt River to the Grand Canal	1,451,000
3.	Pipe - 7th Avenue, Salt River to the Grand Canal	1,665,000
4.	Pipe - 19th Avenue, Salt River to the Grand Canal	1,438,000
5.	Miscellaneous Collectors and Laterals	578,000
6.	Improvements to Old Cross-cut Canal*	228,000
7.	Right-of-way cost for Arizona Canal Floodway*	360,000
	TOTAL COST	$6,433,000

* County participation in expenditures for these items is recommended.

COLLECTOR DESIGN AND COSTS

LOCATION	Remarks	Design Freq'ncy	Design Flow-CFS	Slope Ft/thou	Size and Kind	Length	Unit Cost	Total Cost
24th STREET - Salt River to Grand Canal								
River to Durango	(Open ditch below pipe)	1 Yr.	130	0.8	72" Pipe	1500	$40.00	$ 60,000
Durango to Buckeye			125-130	0.8	72"	2700	42.00	113,400
Buckeye to Harrison	Done with paving program		125-120	0.8	69"	2700	37.00	99,900
Harrison to Van Buren	" " " "		120-115	0.8	69"	2700	39.00	105,300
Van Buren to Roosevelt	" " " "		115-110	2.5	54"-57"	2700	28.00	75,600
Roosevelt to McDowell	" " " "		110-105	2.5	54"	2700	27.00	72,900
McDowell to Encanto	" " " "		105-100	2.5	54"	2700	28.00	75,600
Encanto to Grand Canal	" " " "		100	2.5	51"	1800	25.00	45,000
Collectors primarily part of paving and improvement districts						-	-	-
						Subtotal		$647,700
					Engr. and contingencies			65,300
						Total		$713,000

COLLECTOR DESIGN AND COSTS

LOCATION	Remarks	Design Freq'ncy	Design Flow-CFS	Slope Ft/thou	Size and Kind	Length	Unit Cost	Total Cost
7th STREET - Salt River to Grand Canal								
River to Durango	(May ditch portion)	1 Yr.	235	1.0	87" Pipe	2700	$54	$145,800
Durango to Buckeye			230-235	0.8	90"	2700	57	153,900
Buckeye to Harrison			215-230	0.8	87"	2700	60	162,000
Harrison to Van Buren			200-215	0.8	84"-87"	2700	60	162,000
Van Buren to Roosevelt			175-200	0.8	81"-84"	2700	57	153,900
Roosevelt to McDowell			140-175	1.1	69"-75"	2700	48	129,600
McDowell to Encanto	Done with repaving job		130-140	1.0	69"	2700	39	105,300
Encanto to Thomas	" " " "		120-130	1.5	63"	2700	34	91,800
Thomas to Osborn	" " " "		115-120	2.6	54"	2700	26	70,200
Osborn to Indian School	" " " "		110-115	3.0	54"	2700	26	70,200
Indian School to Canal	" " " "		105-110	3.5	51"	3000	25	75,000
						Subtotal		$1,319,700
						Engr. and contingencies		131,300
						Total		$1,451,000

(Alignment may be 5th St. below Roosevelt to south of tracks and 3rd St. to River.)

COLLECTOR DESIGN AND COSTS

LOCATION	Remarks	Design Freq'ncy	Design Flow-CFS	Slope Ft/thou	Size and Kind	Length	Unit Cost	Total Cost
7th AVE. LINE (11th AVE. FROM RIVER TO VAN BUREN, 9th AVE. TO ENCANTO, 7th AVE. THEREON) SALT RIVER TO GRAND CANAL								
River to Durango		1 Yr.	200	0.7	87"	4000	$53	$ 212,000
Durango to Buckeye			195-200	0.7	87"	2700	55	148,500
Buckeye to Harrison			190-95	3.0	66"	2700	36	97,200
Harrison to Van Buren			185-90	0.8	81"	2700	57	153,900
11th Ave. to 9th			185	0.8	81"	700	57	39,900
Van Buren to Roosevelt			180-85	0.8	81"	2700	57	153,900
Roosevelt to McDowell			175-80	1.0	78"	2700	54	145,800
McDowell to Encanto			170-75	1.2	75"	2700	50	135,000
9th Ave. to 7th			170	1.2	72"	700	45	31,500
Encanto to Thomas			165-70	2.5	63"	2700	34	91,800
Thomas to Osborn			160-65	2.5	63"	2700	39	105,300
Osborn to Indian School			155-60	3.3	60"	2700	36	97,200
Indian School to Canal			150-55	3.0	60"	3000	34	102,000
					Subtotal			$1,514,000
					Engr. and Contingencies			151,000
					Total			$1,665,000

COLLECTOR DESIGN AND COSTS

LOCATION	Remarks	Design Freq'ncy	Design Flow-CFS	Slope Ft/thou	Size and Kind	Length	Unit Cost	Total Cost
19th AVE. - SALT RIVER TO GRAND CANAL								
River to Durango	(Same ditching below pipe)	1 Yr.	215	0.8	81"	1500	$48	$ 72,000
Durango to Buckeye			210-15	0.8	78"	2700	50	135,000
Buckeye to Harrison			205-10	2.5	69"	2700	43	116,100
Harrison to Van Buren			200-05	2.2	69"	2700	45	121,500
Van Buren to Roosevelt			195-200	1.2	78"	2700	51	137,700
Roosevelt to McDowell			190-95	1.6	72"	2700	45	121,500
McDowell to Encanto			180-190	2.6	66"	2700	40	108,000
Encanto to Thomas			175-80	2.8	66"	2700	40	108,000
Thomas to Osborn			165-75	3.0	63"	2700	37	99,900
Osborn to Indian School			160-65	2.5	63"	2700	34	91,800
Indian School to Canal			155-60	2.5	63"	1800	34	61,200
Allow for possible 21st and 17th Ave. alignment.						2700	50	135,000
						Subtotal		$1,307,700
					Engr. and Contingencies			130,300
						Total		$1,438,000

COLLECTOR DESIGN AND COSTS

LOCATION	Remarks	Design Freq'ncy	Design Flow-CFS	Slope Ft/thou	Size and Kind	Length	Unit Cost	Total Cost
MISCELLANEOUS COLLECTORS TO 7th ST., 7th AVE., 19th AVE. LINES								
7th ST. LINE								
Devonshire						1300'		
Whitton	Fix by paving the area					-		
Osborn						3000'		
Roosevelt						2600'		
Jefferson						1300'		
					30" to 36" Pipe	8200'	$14	$114,800
7th AVE. LINE								
McDowell to Encanto Area (connecting east of 7th)						2700'		
Thomas to West						2700'		
Osborn to West*						1500'		
Indian School to West*						1300'		
					30" to 36" Pipe	8200'	$17	139,400
19th AVE. LINE								
Sherman area*						700'		
Adams to West						700'		
Fillmore to West						2700'		
Fillmore to East						2700'		
Thomas to East						1300'		
Indian School to East						1300'		
					30" to 36" Pipe	9400'	14	131,600
Reserve for miscellaneous lines						10000'	14	140,000
						Subtotal		$525,800
					Engr. and Contingencies			52,200
						Total		$578,000

*Additional work as necessary to be done by improvement district.

COLLECTOR DESIGN AND COSTS

LOCATION	Remarks	Design Freq'ncy	Design Flow-CFS	Slope Ft/thou	Size and Kind	Length	Unit Cost	Total Cost
OLD CROSS CUT CANAL								
Right-of-way from SRVWUA	- except near River					3000'		$ 5,000
River to Grand Canal		100 Yr.	2800	1'	Earth canal	3000'	$13	39,000
Grand Canal Structure	Siphon and Spillway	"	-	-	Concrete	-		40,000
Grand Canal to Jefferson		"	2500	2'	Canal	900'		
Jefferson thru Washington		"	2500	4'	Add Box Culvert	400'	105	42,000
Washington - Van Buren			2500	2'	Canal	1200'		
At Van Buren			2500	4'	Add Box Culvert	80'	92	7,200
Van Buren to McDowell			2500	7'	Canal	5200'		
At McDowell Rd.			2500	5'	Add Box Culvert	140'	100	14,000
McDowell to Thomas			2500	3'	Canal	5200'		
At Thomas Rd.			2500	4'	Add Box Culvert	80'	110	8,800
Thomas to Osborn			2500	4'	Canal	2600'		
At Osborn Rd.			2500	5'	Add Box Culvert	60'	105	6,300
Osborn to Indian School			2500	6'	Canal	2700'		
At Indian School			2500	6'	Add Box Culvert	50'	95	4,700
Indian School to Ariz.Canal			2500	10'	Canal	700'		
Siphon under Ariz. Canal			1500	10'	Box Culverts	200'	84	16,800
Rebuild Ariz.Canal Spill	To Spill Ariz.Canal Capacity		1400	-	Concrete			18,000
						Subtotal		$201,800
					Engr. and Contingencies			26,200
						Total		$228,000

Canal work or drop structures by SRVWUA.

COLLECTOR DESIGN AND COSTS

LOCATION	Remarks	Design Freq'ncy	Design Flow-CFS	Slope Ft/thou	Size and Kind	Length	Unit Cost	Total Cost
NEW CANAL PARALLELING THE ARIZONA CANAL								
Right-of-way - Old X-Cut Canal to New River - SRVWUA (50') and purchased (100')						94,000'	$1500 acre	$ 324,000
47th to 39th Streets		100 yr.	To 1000	0.35	Earth Canal	8,000'	9	72,000
Crossings	(or adjacent to homes)	"	1000	1.0	Culverts	500'	118	59,000
39th to 24th Streets			2300	0.30	Earth Canal	11,000	16	176,000
Crossings	(or adjacent to Biltmore)		2300	1.0	Culverts	1,200'	250	300,000
24th to 16th Streets			2700	0.30	Earth Canal	8,200	17	139,000
Crossings			2700	1.0	Culverts	200	300	60,000
16th to 8th Streets			3000	0.30	Earth Canal	7,400	18	133,000
Crossings			3000	1.0	Culverts	120	320	38,000
8th St. to 3rd St.	(adjacent to subdivisions)		3300	0.35	Lined Canal	2,100	110	231,000
3rd St. to 7th Ave.			3300	0.3	Earth Canal	5,400'	19	103,000
Crossings-8th St. to 7th Ave.			3300	1.0	Culverts	200'	350	70,000
7th. Ave. to 23rd Ave.			3500	0.3	Earth Canal	7,500'	20	150,000
Crossings			3500	1.0	Culverts	160'	360	58,000
23rd Ave. to Skunk Creek			4500	0.35	Earth Canal	44,000'	23	1,012,000
Crossings			4500	1.0	Culverts	600'	400	240,000
Skunk Creek to New River			5000	7.0	River Channelling	10,000'	10	100,000
						Subtotal		$3,265,000
					Engr. and Contingencies			485,000
						Total		3,750,000
			Allow for land acquisition only at this time					$ 360,000

Note. Canal lengths are total including structure lengths.