ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

State Standard

for

Stormwater Detention/Retention

Under the authority outlined in ARS 48-3605(a) the Director of the Arizona Department of Water Resources establishes the following standard for Stormwater Detention/Retention in Arizona.

Local community and county flood control districts developing designs for stormwater detention/retention facilities must use either the procedures outlined in State Standard Attachment 8-99 entitled, “Stormwater Detention/Retention,” or an alternative procedure accepted by the Director of the Department.

Application of these procedures will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, provides for sufficient and effective stormwater management.

This requirement is effective September 15, 1999.

Copies of this State Standard and State Standard Attachment can be obtained by contacting the Department's Flood Mitigation Section at (602) 417-2445.

NOTICE

This document is available in alternative formats. Contact the Department of Water Resources, Flood Mitigation Section at (602) 417-2445 or (602) 417-2455 (TDD).

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD MITIGATION SECTION

Stormwater Detention/Retention

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

State Standard Attachment
SSA8-99

August 1999

DISCLAIMER OF LIABILITY

The Arizona Department of Water Resources is not responsible for the application of the methods outlined in this publication and accepts no liability for their use. Sound engineering judgment is recommended in all cases.

The Arizona Department of Water Resources reserves the right to modify, update, or otherwise revise this document. Questions regarding information contained in this document and/or floodplain management should be directed to the local floodplain administrator or the office below:

Flood Mitigation Section
Arizona Department of Water Resources
500 North Third Street
Phoenix, Arizona 85004

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I. INTRODUCTION

1.1 Project Background

This report has been prepared to document the results and recommended standards and procedures developed in the Assessment and Development of State Standard for Stormwater Detention/Retention in Arizona project. The purpose of the project was to conduct a literature search and assessment of the practice of stormwater detention/retention in Arizona and the southwest; identify stormwater detention/retention methods and practices; and develop guidelines based on the information gathered. The project was performed under contract to the Arizona Department of Water Resources (ADWR) and under the direction of ADWR's State Standards Work Group (SSWG). The SSWG is a volunteer group of floodplain management officials from around the state working in conjunction with ADWR to make floodplain management throughout Arizona more uniform and efficient. Everyone in Arizona benefits from these standards.

The purpose of this report is to provide general information on stormwater detention/retention and to document the recommended standards and procedures for use in Arizona.

1.2 General

Stormwater Management through Detention/Retention is a widely used tool for mitigating the effects of urbanization on flood peak discharges and runoff volume. Increased runoff associated with urbanization is a widely documented phenomenon. Generally speaking, the principle of stormwater detention/retention is to store runoff from urbanized areas and release it at rates reflecting the natural or unurbanized condition which existed before development.

Proper implementation of stormwater detention/retention depends on a number of factors including the goals of the community, physical conditions within the area where stormwater detention/retention is proposed for use and design assumptions such as rainfall duration, intensity and frequency, and hydrograph shape. Different standards and procedures may be appropriate in different localities depending on the extent of urbanization occurring and the goals of the community. With this in mind, procedures were developed which can be applied on a broad but conservative basis or, alternatively, on a more site specific and detailed basis.

Background documentation on the research and findings leading up to the recommended standards and procedures contained herein are documented in the Phase I and Phase II reports for the project which are available through ADWR.

1.3 Limitations of Procedures

Standards and procedures were developed for stormwater detention/retention for use in Arizona using the Level 1, 2 and 3 format common to other state standards. Generally speaking, the lower the procedure level the simpler the evaluation; and the more conservative the resulting design parameters. The Level 1 procedure requires the least information and associated analysis but, because of the limited investigation, yields the most conservative result relative to the general goal of stormwater detention/retention. The Level 2 procedure results in a less conservative design but requires more information and analysis than the Level 1 procedure. The Level 3 procedure is the most in-depth approach and, in the case of stormwater detention/retention, reflects a more regional approach to the problem.

Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, provides for sufficient and effective stormwater management.

1.4 Use-Based Application of Procedures

Utilizing simplified standards and procedures involves uncertainty. Consequently, these standards recommend against applying low procedure levels to larger, more complex developments. The following matrix provides an index to the applicability of the various procedure levels to the types of uses for which they can be confidently applied.

Application[1]	**Procedure Level**		
	Level 1	**Level 2**	**Level 3**
Single Commercial Lot	Acceptable		
Small Subdivisions (<160 acres)	Acceptable	Recommended	
Large Subdivisions (>160 acres) and Planned Communities	Not recommended		Recommended

1.5 Criteria for Optional Waiver of Requirements

All requests for waivers must first refer to the local jurisdiction or agency for waiver criteria. If none exist, then the following are offered as waiver requirements. All waivers will require written approval from the local jurisdiction or governing agency prior to issuance.

[1] See waiver provisions of Section 1.5 also.

Waiver of stormwater detention/retention may be observed for:

- Single residential lots (i.e., not associated with a subdivision).
- Residential subdivisions with average lot areas $\geq$ 1 acre in area.
- Projects smaller than 160 acres which drain *directly* into a watercourse intercepting a drainage area of $\geq$ 100 square miles.

II. STORMWATER DETENTION/RETENTION PROCEDURES

2.1 Level 1

The Level 1 procedure is based on storage of the entire 1-hour, 100-year rainfall falling on the project site. The procedure for determining the required storage volume is as follows. A worksheet and design charts are contained in Appendix A.

1. Determine the area of the project site, A (acres).

2. Determine 100-year, 1-hour rainfall depth, $P_{100,1}$ (inches), by finding the 100-year, 6-hour, $P_{100,6}$, and 100-year, 24-hour, $P_{100,24}$, rainfall depths using Precipitation Maps 7 & 8, respectively, from the ADOT Hydrology Manual (1993) and using the 100-year, 1-hour Rainfall Depth Chart, all in Appendix A of this report.

3. Determine the developed condition runoff coefficient, C, for the project site using Figure 2-3 from the ADOT Hydrology Manual (1993) in Appendix A of this report. For purposes of using Figure 2-3 the following residential densities shall be assumed to apply:

   - Heavy Urban - > 4 units/acre
   - Moderate Urban - 2 – 4 units/acre

4. Determine the developed condition 100-year, 1-hour runoff volume, V_r (acre-feet) to be retained, as follows:

   $$V_r = (C \times P_{100,1} \times A)/12$$

Using the storage volume requirement determined above, a storage basin should be designed using the following general guidelines:

1. Design the basin to intercept site runoff, not offsite runoff. If necessary the storage can be accommodated by constructing more than one basin (e.g., to accommodate off-site drainage through the site, drainage divides through the site or grading constraints).

2. Keep basin ponding depths to three feet or less where possible.

3. Keep basin side slopes to 4:1 or flatter where possible. Basins with steeper side slopes should be properly stabilized if used.

4. Regardless of basin side slope, seeding of the basin to promote vegetation should be considered in the design to prevent rill and gully erosion.

5. Unauthorized access should be physically restricted (i.e., by fencing or other appropriate means) where basin depth is greater than three feet *and* any side slopes steeper than 4:1.

6. Provide a 6" to 8" diameter pipe outlet at the low point of the basin[2]. The pipe should be no longer than 30 feet in length to facilitate cleaning. In order to maintain a reasonable drain time, one 6" diameter pipe should be provided for every acre-foot of required storage volume (or fraction thereof) *or* one 8" diameter pipe should be provided for every two acre-feet of required storage volume (or fraction thereof). The inlet to the pipe should include a grate or riser for debris interception with a total open area as large or larger than the pipe area. The inlet should also be elevated slightly above the basin bottom (≤ 6") where sedimentation is likely to occur at the inlet. The pipe should outlet to a natural/ historic point of drainage outflow. The pipe outlet should include erosion protection to prevent scour at the outlet.[3]

7. Grade the basin bottom to provide a minimum of 0.2% grade toward the pipe outlet.

8. To the extent possible, avoid sharp angular shapes (e.g., squares or rectangles) in favor of gently curving lines for the basin geometry.

9. Vehicular access should be provided to the basin either around the perimeter or into the interior of the basin to allow adequate maintenance.

10. An inspection and maintenance plan should be developed which clearly specifies the party responsible for maintenance and the frequency and method of maintenance. The plan should insure that the original storage volume of the basin is maintained, including sediment removal as needed.

11. The basin should be designed with an emergency overflow level such that ponding in excess of the design level (i.e., due to outlet clogging or extreme/successive flow events) will not cause inundation of unintended areas or improvements. The emergency overflow should act as a weir with a minimum length (in feet) equal to three times the area of the project site (in acres), A, as defined above (e.g., for A = 10 acres, the emergency overflow control weir would be 30 feet long). The emergency overflow should drain to a natural/ historic point of drainage outflow.

12. Adjacent structures should be constructed at an elevation at least two feet above the emergency overflow level described above.

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[2] Alternatively, a larger pipe (e.g., 18" or larger) can be installed and a plate with a 6" to 8" orifice, or a grated or riser-type structure with equivalent flow capacity, can be placed over the inlet. Such an approach may be advisable in areas where significant debris accumulation is possible or where frequent cleaning of the pipe may otherwise be a concern.

[3] In cases where topographic or other physical constraints preclude application of the guidelines under this item, the user should consult the local floodplain management authority for design assistance to comply with drainage requirements.

2.2 Level 2

The Level 2 procedure is based on storage of a portion of the 1-hour, 100-year rainfall falling on the project site to maintain the 100-year pre-development runoff rate from the site. The procedure also includes an adjustment to the design basin outflow rate to account for the cumulative downstream effect of urbanization. The resulting procedure should provide a conservative measure of stormwater detention in the vast majority of applications. If the user or jurisdiction is concerned that application of this procedure in a particular situation will not accomplish the intended goal of downstream peak flow reduction, the user should refer to Level 3 procedures or procedures as directed by the jurisdiction.

The procedure for determination of the required storage volume and design outflow rate is as follows and can be performed using the worksheet and charts in Appendix B[4]:

1. Determine the area of the project site, A (acres).

2. Determine 100-year, 1-hour rainfall depth, $P_{100,1}$ (inches), by finding the 100-year, 6-hour, $P_{100,6}$, and 100-year, 24-hour, $P_{100,24}$, rainfall depths using Precipitation Maps 7 & 8, respectively, from the ADOT Hydrology Manual (1993) and the 100-year, 1-hour Rainfall Depth Chart, all in Appendix A of this report.

3. Determine the developed condition runoff coefficient, C, for the project site using Figure 2-3 from the ADOT Hydrology Manual (1993) in Appendix A of this report. For purposes of using Figure 2-3 the following residential densities shall be assumed to apply:

   - Heavy Urban - > 4 units/acre
   - Moderate Urban - 2 – 4 units/acre

4. Determine the developed condition 100-year, 1-hour runoff volume, V_r (acre-feet) to be retained, as follows:

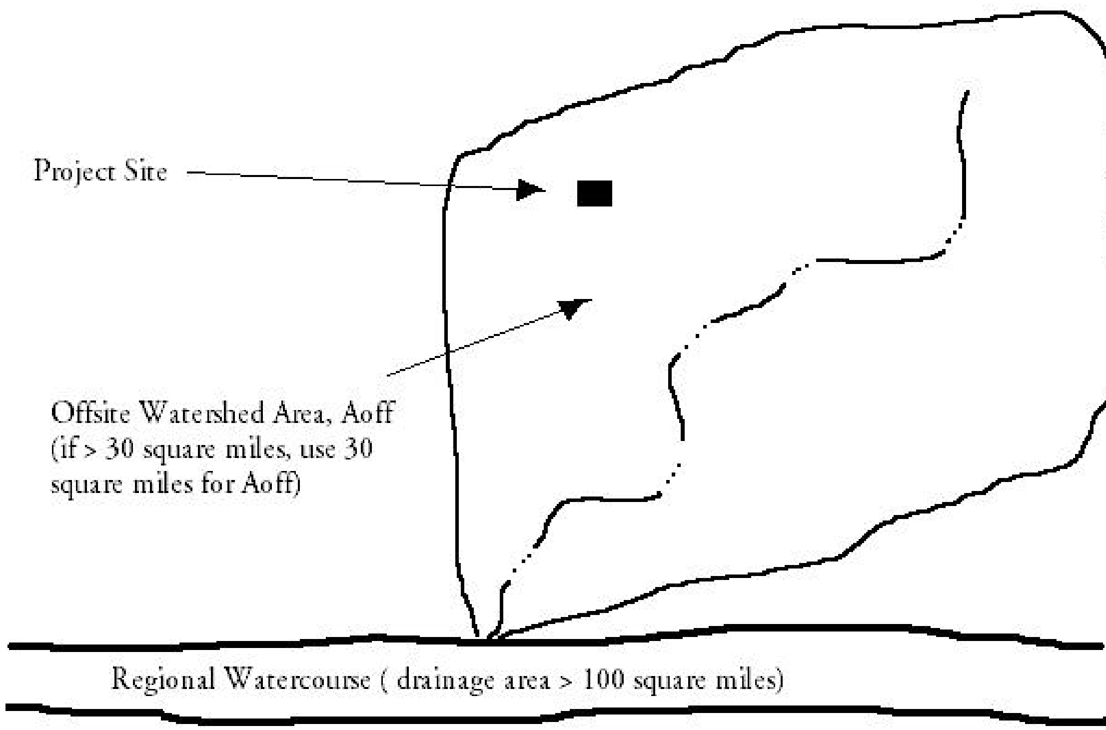
$$V_r = (C \times P_{100,1} \times A)/12$$

5. Determine the developed condition peak discharge, Q (cfs), for the site using the 1993 ADOT Hydrology Manual rational method procedure as outlined in the Level 2 worksheet in Appendix B of this report.

6. Determine the existing condition peak discharge contribution of the project site to the "offsite" watershed peak discharge, Q_{off} (cfs), as follows:

[4] The first four steps of the Level 2 procedure are identical to the first four steps of the Level 1 procedure. For this reason, design charts for the first four steps are contained in Appendix A.

- Determine the area (in square miles) of the offsite watershed, A_{off}, which the project is located in, at the point where it empties into a regional watercourse (see definition sketch below). Where the size of the offsite watershed exceeds 30 square miles, use 30 square miles for A_{off} (i.e., A_{off} cannot be more than 30 square miles).



- Determine the unit discharge for the offsite watershed, q_{off} (cfs/sq mi), using the 100-year Unit Discharge Chart in Appendix B for the appropriate region.
- Determine Q_{off} (cfs) as follows: $Q_{off} = A \times q_{off}/640$

Note: The calculation of Q_{off} as described above is intended to result in a design outflow which limits the 100-year post-development peak outflow from the project site to a rate which reflects runoff rates associated with natural conditions on a larger watershed scale. As such this adjustment is intended for areas where urbanization is expected to occur widely throughout the regional watershed. However, if it can be documented that existing and future urbanization can not potentially affect more than a total of 10% of the offsite watershed, then Q_{off} can be calculated as the existing condition discharge from the site using the rational method procedure as outlined in the 1993 ADOT Hydrology Manual[5].

7. Divide the value of Q_{off} (i.e., the design basin outflow) by Q (the design basin inflow) to determine the value of Q_{off}/Q and find the value of V_s/V_r using Q_{off}/Q vs. V_s/V_r Chart in Appendix B of this report. Be sure to use the plot that represents the type of outlet structure intended for the detention basin design (i.e., pipe vs. weir).

8. Determine the required detention storage volume for the project site, V_s (acre-feet) as follows: $V_s = V_r \times (V_s/V_r)$ (where Vr is as determined in step 4)

9. Determine an appropriate outflow structure based on the design outflow (Q_{off}) and the maximum depth of the basin (i.e., the maximum headwater, HW)[6].

   - For pipe outflow structures, most structures can be sized using the performance charts from HEC No. 10 “Capacity Charts for the Hydraulic Design of Highway Culverts”. For the convenience of the reader, Charts 11, 13, 19 and 22 of HEC-10 have been reproduced in Appendix B of this report along with select passages of text from HEC-10 explaining the use of the charts. For design types or conditions not covered by the charts included in Appendix B, the reader is referred to FHWA HEC-10, (Nov. 1972) or HEC-5 (reprinted June 1980).

   - For weir outflow structures, a simple rectangular weir should be sized/designed by solving for L (weir length) in the weir equation below knowing the other variables: $Q = CLH^{3/2}$

     where: Q = design outflow, Q_{off} (cfs)
     C = weir coefficient (use 3.1 for sharp-crest, 2.7 for broad-crest)
     L = length of the weir (ft)
     H = the head on the weir, HW as defined above, (ft)

Using the storage volume and outflow structure requirements determined above, a storage basin should be designed using the following general guidelines:

1. Design the basin to intercept site runoff, not offsite runoff. If necessary the storage can be accommodated by constructing more than one basin (e.g., to accommodate off-site drainage through the site, drainage divides through the site or grading constraints).

2. Keep basin ponding depths to three feet or less where possible.

[5] An example of such a situation would be development of a small in-holding in a national forest.

[6] In most instances, a pipe outflow structure will most likely provide the most cost-effective design. If a weir outflow structure is used, the weir crest should be set at the basin low-point to provide a design consistent with the assumptions in the detention volume sizing procedure and to facilitate complete drainage of the pond.

3. Keep basin side slopes to 4:1 or flatter where possible. Basins with steeper side slopes should be properly stabilized if used.

4. Regardless of basin side slope, seeding of the basin to promote vegetation should be considered in the design to prevent rill and gully erosion.

5. Unauthorized access should be physically restricted (i.e., by fencing or other appropriate means) where basin depth is greater than three feet *and* any side slopes steeper than 4:1.

6. The basin outlet should outlet to a natural/historic point of drainage outflow. The pipe outlet should include erosion protection to prevent scour at the outlet. The outlet should be designed/located so as to preclude submergence of the outlet by tailwater.

7. Grade the basin bottom to provide a minimum of 0.2% grade toward the outlet.

8. To the extent possible, avoid sharp angular shapes (e.g., square or rectangular) in favor of gently curving lines for the basin geometry.

9. Vehicular access should be provided to the basin either around the perimeter or into the interior of the basin to allow adequate maintenance.

10. An inspection and maintenance plan should be developed which clearly specifies the party responsible for maintenance and the frequency and method of maintenance. The plan should insure that the original storage volume of the basin is maintained, including sediment removal as needed.

11. The basin should be designed with an emergency overflow level such that ponding in excess of the design level (i.e., due to outlet clogging or extreme/successive flow events) will not cause inundation of unintended areas or improvements. The emergency overflow should act as a weir with a minimum length (in feet) equal to the 100-year discharge from the site, Q, divided by 2.7 (e.g., for Q = 27 cfs, the emergency overflow control weir would be 10 feet long). The emergency overflow should drain to a natural/ historic point of drainage outflow.

12. Adjacent structures should be constructed at an elevation at least two feet above the emergency overflow level described above.

2.3 Level 3

Generally speaking, Level 3 procedures are used to provide the most detailed and cost effective design based on evaluation of the most detailed information available.

For purposes of applying Level 3 procedures the following design criteria should be observed:

1. Where possible, stormwater detention/retention should be implemented on a regional basis by the governing authority/district. The stormwater detention/retention program should utilize regional detention/retention based on watershed-wide assessment of the effects of urbanization and planning and development of facilities at the most effective locations to minimize those effects. Such a watershed wide assessment should include an evaluation of the cumulative effects of urbanization such that the implementation of the stormwater detention/retention program addresses both localized increases in runoff and regional effects to the extent possible. Where such a plan can be implemented, on-site stormwater detention/retention should be avoided.

2. It is recognized that the criteria and goals outlined in (1) above are not always practical or attainable for institutional, legal, financial or other reasons. Where implementation of a regional program is not possible or practical, stormwater detention should be provided to the extent necessary to insure that post-development peak discharges from a project site are no greater than pre-development peak discharge rates for the 2-, 10- and 100-year events. Use of the multiple-event criteria described above will aid in minimizing the cumulative regional effects of urbanization on downstream areas. However, even under these circumstances the local jurisdiction should be consulted as to any input they may have on design criteria to address regional effects.

   In support of the application of this criterion, the procedures contained in the following publications are recommended for use in Arizona, *where jurisdictions do not already have adopted manuals or criteria*:

   - *Stormwater Detention/Retention Manual*, Pima County Department of Transportation and Flood Control District, 1987
   - *Drainage Design Manual for Maricopa County, Arizona*, Vol. II Hydraulics, Revised 1996
   - *Yavapai County Drainage Criteria Manual*, Yavapai County, 1998

2.4 Example Applications

A blank worksheet and design charts for use in application of the Level 1 procedure can be found in Appendix A of this report. Blank worksheets and design charts for use in application of the Level 2 procedure can be found in Appendix B of this report. Example applications of Level 3 procedures can be found in the references listed in Section 3.3, or within the references listed therein.

III. REFERENCES

1. *Highway Drainage Design Manual*, Arizona Department of Transportation, March 1993

2. *Preliminary Sizing of Detention Reservoirs to Reduce Peak Discharges*, McEnroe, Bruce, M., Journal of Hydraulic Engineering, ASCE, Vol. 118, No. 11, November 1992

3. *Stormwater Detention – Downstream Effects on Peak Flow Rates*, Lakatos, David F. and Kropp, Richard H., article within "Stormwater Detention Facilities; Planning Design Operation and Maintenance", William DeGroot, Editor, ASCE, 1983

4. *Stormwater Detention/Retention Manual*, Pima County Department of Transportation and Flood Control District, 1987

5. *Drainage Design Manual for Maricopa County, Arizona*, Vol. II Hydraulics, Flood Control District of Maricopa County, Revised 1996

6. *Yavapai County Drainage Criteria Manual*, Yavapai County, 1998

7. *Capacity Charts for the Hydraulic Design of Highway Culverts*, Hydraulic Engineering Circular No. 10, Federal Highway Administration, November 1972

8. *Assessment and Development of State Standard for Stormwater Management Through Detention/Retention in Arizona, Final Report on Phase I: Literature Search and Assessment of Current Practices*, ADWR, August 1998

9. *Assessment and Development of State Standard for Stormwater Management Through Detention/Retention in Arizona, Final Report on Phase II: Literature Review and Evaluation of Methods*, ADWR, October 1998

10. *Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States*, Thomas, B.E., Hjalmarson, H.W., and Waltemeyer, S.D., USGS Open File Report 93-419, 1994

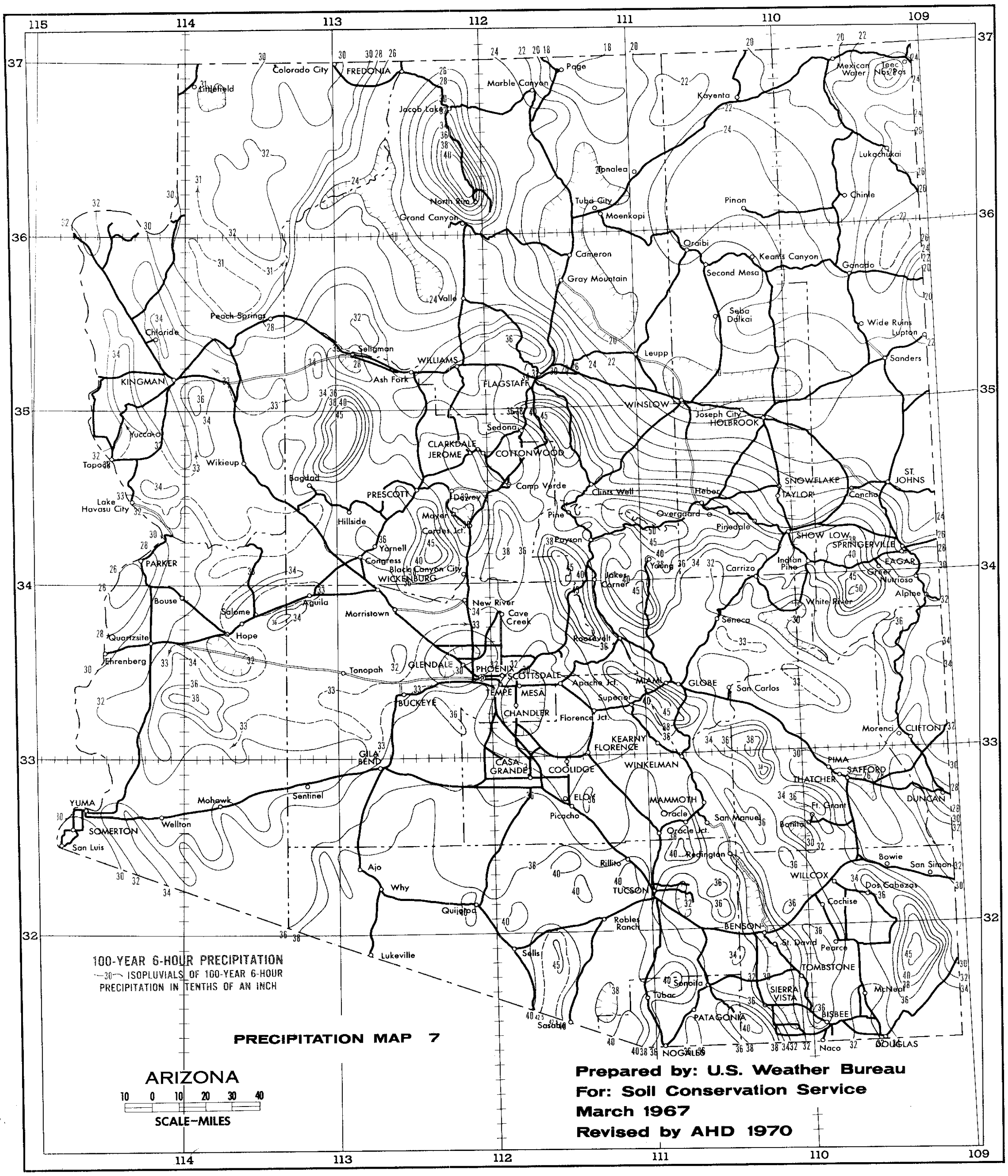
APPENDIX A

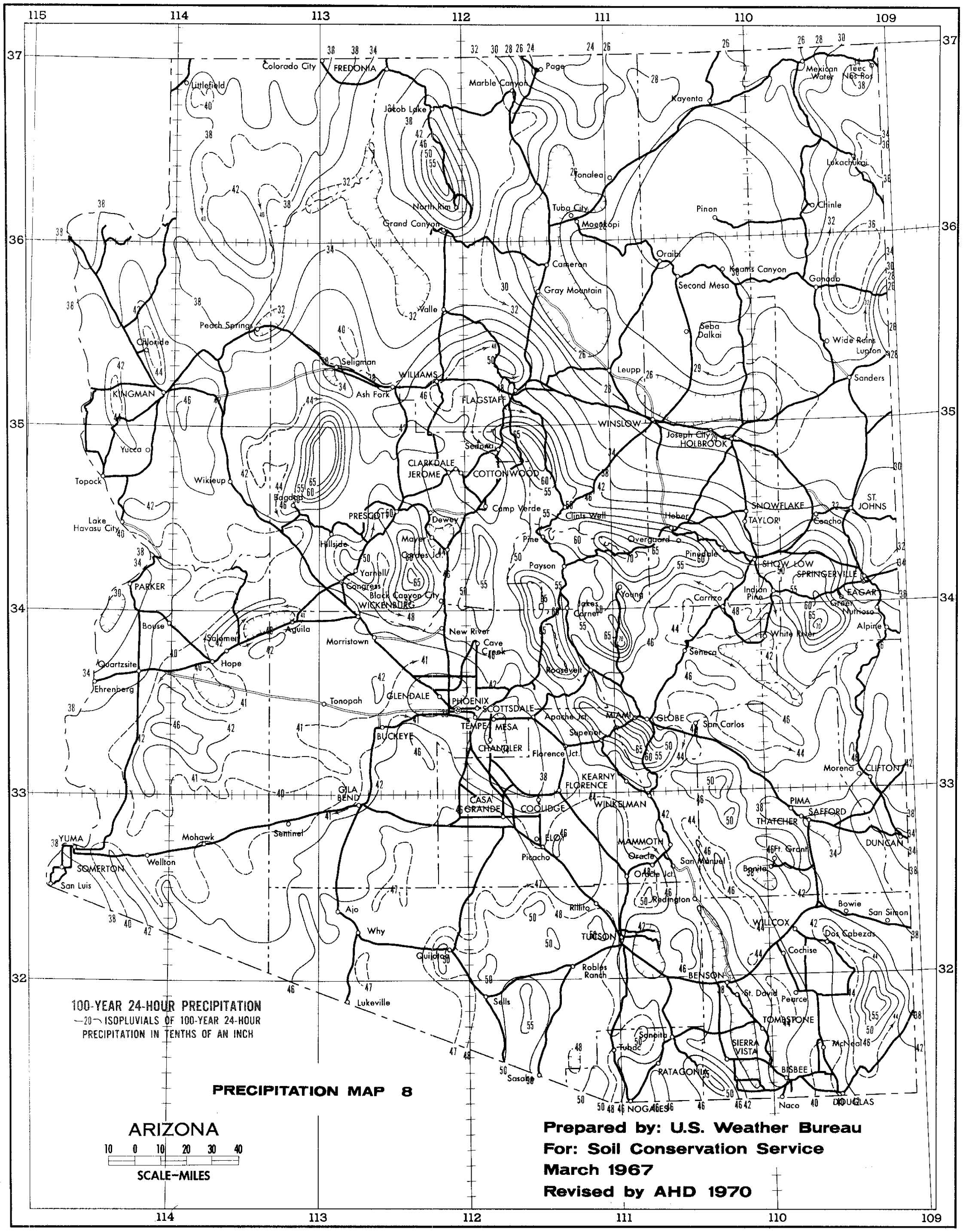
Level 1 Worksheet and Design Charts

LEVEL 1 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET				
Step	**Parameter Description**	**Equation/Method Determined**	**Value**	**Units**

PROJECT DESCRIPTION:					
1	Project Site Area	A	From site data		Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7		Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8		Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From $P_{100,1}$ Chart		Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3		None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$		Acre-ft

NOTES:


100-YEAR 6-HOUR PRECIPITATION
—30— ISOPLUVIALS OF 100-YEAR 6-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 7
ARIZONA
10 0 10 20 30 40
SCALE–MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970


100-YEAR 24-HOUR PRECIPITATION
—20— ISOPLUVIALS OF 100-YEAR 24-HOUR PRECIPITATION IN TENTHS OF AN INCH
PRECIPITATION MAP 8
ARIZONA
10 0 10 20 30 40
SCALE-MILES
Prepared by: U.S. Weather Bureau
For: Soil Conservation Service
March 1967
Revised by AHD 1970

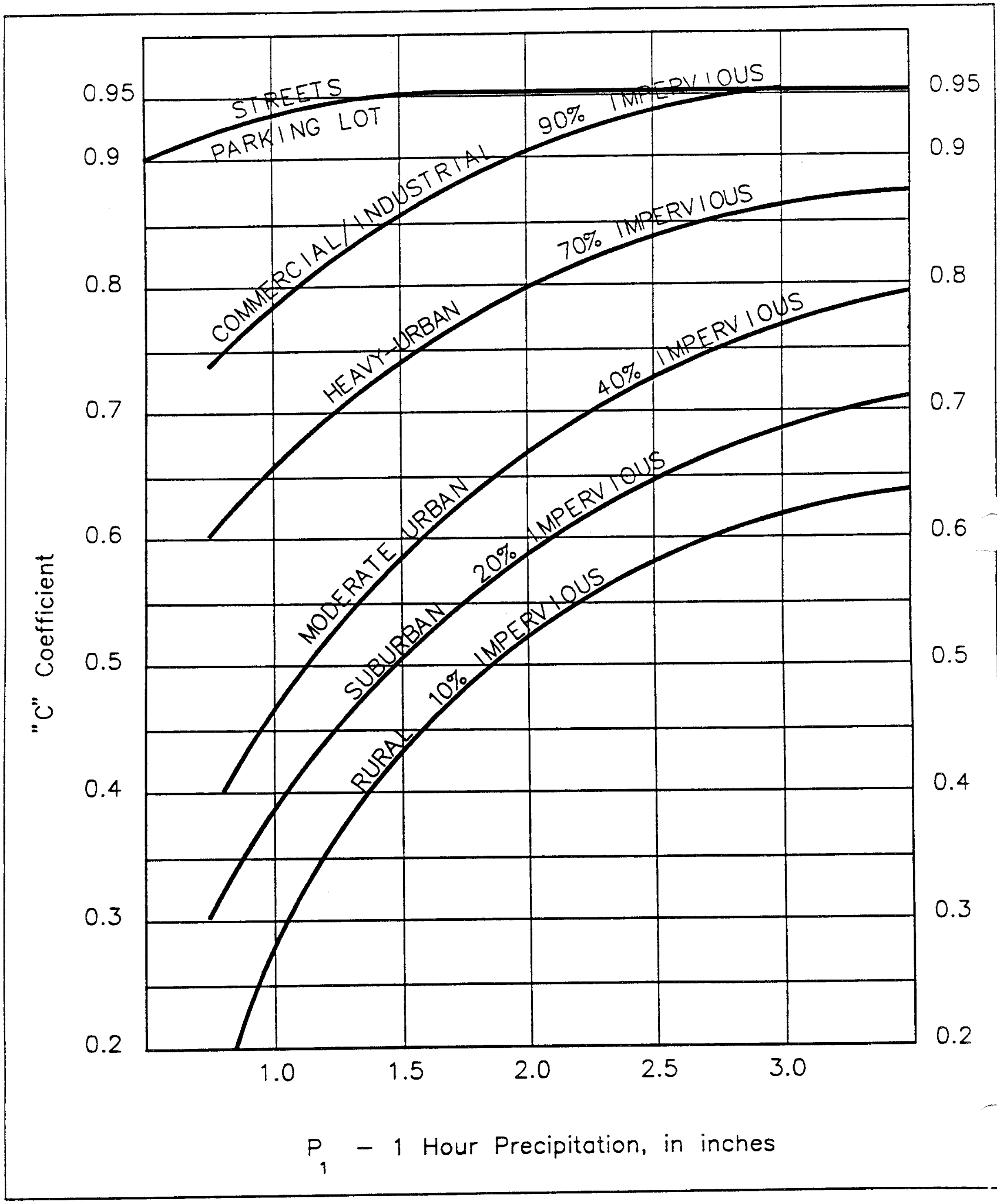
100-YEAR, 1-HOUR RAINFALL TABLE BASED ON ADOT PROCEDURE

Enter left hand column with P100,6 (to the nearest tenth of an inch) and read over to column
with appropriate value of P100,24 (to the nearest half inch) and read value of P100, 1
at intersection of proper row and column

P100,6	Columns correspond to P100,24 value shown in shaded row below:									
Value	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
1.5	1.4	1.2	1.1	1.0	1.0	0.9	0.9	0.9	0.8	0.8	0.8
1.6	1.5	1.3	1.2	1.1	1.0	1.0	0.9	0.9	0.9	0.8	0.8
1.7	1.6	1.4	1.3	1.2	1.1	1.0	1.0	0.9	0.9	0.9	0.9
1.8	1.8	1.5	1.4	1.2	1.2	1.1	1.0	1.0	1.0	0.9	0.9
1.9	1.9	1.6	1.5	1.3	1.2	1.1	1.1	1.0	1.0	1.0	0.9
2	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.1	1.0	1.0	1.0
2.1	2.2	1.9	1.7	1.5	1.4	1.3	1.2	1.1	1.1	1.1	1.0
2.2	2.4	2.0	1.8	1.6	1.5	1.4	1.3	1.2	1.2	1.1	1.1
2.3	2.5	2.1	1.9	1.7	1.5	1.4	1.3	1.3	1.2	1.2	1.1
2.4	2.7	2.3	2.0	1.8	1.6	1.5	1.4	1.3	1.3	1.2	1.2
2.5	2.9	2.4	2.1	1.9	1.7	1.6	1.5	1.4	1.3	1.3	1.2
2.6	3.1	2.6	2.2	2.0	1.8	1.7	1.6	1.5	1.4	1.3	1.3
2.7	3.3	2.7	2.4	2.1	1.9	1.8	1.6	1.5	1.5	1.4	1.3
2.8	3.5	2.9	2.5	2.2	2.0	1.9	1.7	1.6	1.5	1.5	1.4
2.9	3.7	3.1	2.7	2.4	2.1	2.0	1.8	1.7	1.6	1.5	1.5
3	3.9	3.3	2.8	2.5	2.2	2.1	1.9	1.8	1.7	1.6	1.5
3.1	4.2	3.4	3.0	2.6	2.4	2.2	2.0	1.9	1.8	1.7	1.6
3.2	4.4	3.6	3.1	2.8	2.5	2.3	2.1	1.9	1.8	1.7	1.6
3.3	4.7	3.8	3.3	2.9	2.6	2.4	2.2	2.0	1.9	1.8	1.7
3.4	4.9	4.0	3.5	3.0	2.7	2.5	2.3	2.1	2.0	1.9	1.8
3.5	5.2	4.2	3.6	3.2	2.9	2.6	2.4	2.2	2.1	2.0	1.9
3.6	5.4	4.5	3.8	3.3	3.0	2.7	2.5	2.3	2.2	2.0	1.9
3.7	5.7	4.7	4.0	3.5	3.1	2.8	2.6	2.4	2.3	2.1	2.0
3.8	6.0	4.9	4.2	3.7	3.3	3.0	2.7	2.5	2.4	2.2	2.1
3.9	6.3	5.1	4.4	3.8	3.4	3.1	2.8	2.6	2.5	2.3	2.2
4	6.6	5.4	4.6	4.0	3.6	3.2	3.0	2.7	2.6	2.4	2.3
4.1	6.9	5.6	4.8	4.2	3.7	3.4	3.1	2.9	2.7	2.5	2.4
4.2	7.2	5.9	5.0	4.3	3.9	3.5	3.2	3.0	2.8	2.6	2.4
4.3	7.5	6.1	5.2	4.5	4.0	3.6	3.3	3.1	2.9	2.7	2.5
4.4	7.9	6.4	5.4	4.7	4.2	3.8	3.5	3.2	3.0	2.8	2.6
4.5	8.2	6.7	5.6	4.9	4.4	3.9	3.6	3.3	3.1	2.9	2.7
4.6	8.5	6.9	5.9	5.1	4.5	4.1	3.7	3.4	3.2	3.0	2.8
4.7	8.9	7.2	6.1	5.3	4.7	4.3	3.9	3.6	3.3	3.1	2.9
4.8	9.2	7.5	6.3	5.5	4.9	4.4	4.0	3.7	3.4	3.2	3.0
4.9	9.6	7.8	6.6	5.7	5.1	4.6	4.2	3.8	3.6	3.3	3.1
5	10.0	8.1	6.8	5.9	5.3	4.7	4.3	4.0	3.7	3.4	3.2

FIGURE 2-3
RATIONAL "C" COEFFICIENT
DEVELOPED WATERSHEDS

AS A FUNCTION OF RAINFALL DEPTH AND TYPE OF DEVELOPMENT



MARCH 1993

APPENDIX B

Level 2 Worksheets and Design Charts

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET **PAGE 1 OF 2**

Step	Parameter Description	Equation/Method Determined	Value	Units

PROJECT DESCRIPTION:					
1	Project Site Area	A	From site data		Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7		Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8		Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From $P_{100,1}$ Chart		Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3		None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$		Acre-ft
5	Length of longest flow path of site	L	ADOT Hydrology Manual (1993), page 2-4		Miles
	Watershed resistance coefficient for site	K_b	From Kb Chart contained herein		None
	Slope of longest flow path of site	S	ADOT Hydrology Manual (1993), page 2-4		Ft/mile
If Tc (c) ≠ Tc (a), reset Tc (a) to last value of Tc (c) and repeat these steps until Tc (c) = Tc (a) or Tc (c) ≤ 0.17 hrs	Assumed Time of Concentration	Tc (a)	Assumed (assume a value of 0.17 hours as a first guess)		Hours
	Rainfall intensity	i	ADOT Hydrology Manual (1993), Figure 2-1 or 2-2		Inches/ Hour
	Calculated Time of Concentration	Tc (c)	$Tc = 11.4\ L^{0.5}\ K_b^{0.52}\ S^{-0.31}\ i^{-0.38}$ (Eqn. 2-2 from ADOT Manual) **If Tc (c) ≤ 0.17 hrs (10 min.) use Tc (c) = 0.17 hrs.**		Hours
	Developed condition 100-year peak discharge for project site	Q	Q = CiA		Cfs

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET PAGE 2 OF 2					
Step	**Parameter Description**		**Equation/Method Determined**	**Value**	**Units**
6	Area of offsite watershed	A_{off}	Per definition in Level 2 procedure		Mi^2
	Offsite watershed runoff rate	q_{off}	From 100-year Unit Discharge chart for appropriate region		Cfs/ Sq mi
	Contribution of project site to offsite watershed peak discharge	Q_{off}	$Q_{off} = A\ q_{off}/640$		Cfs
7	Ratio of design outflow to design inflow	Q_{off}/Q	Q_{off}/Q		Ratio
	Ratio of required storage volume to runoff volume	V_s/V_r	From Q_{off}/Q vs. V_s/V_r Chart		Ratio
8	Required storage volume	V_s	$V_s = V_r\ (V_s/V_r)$		Acre-ft
9	Outflow structure		Use HEC-10 pipe outflow structure design charts (Appendix B of state standard) or other reference		

NOTES:

Area vs. Roughness Coefficient, Kb for Urban Areas
For Use with ADOT Rational Method for
Level 2 State Standard for Stormwater Detention/Retention

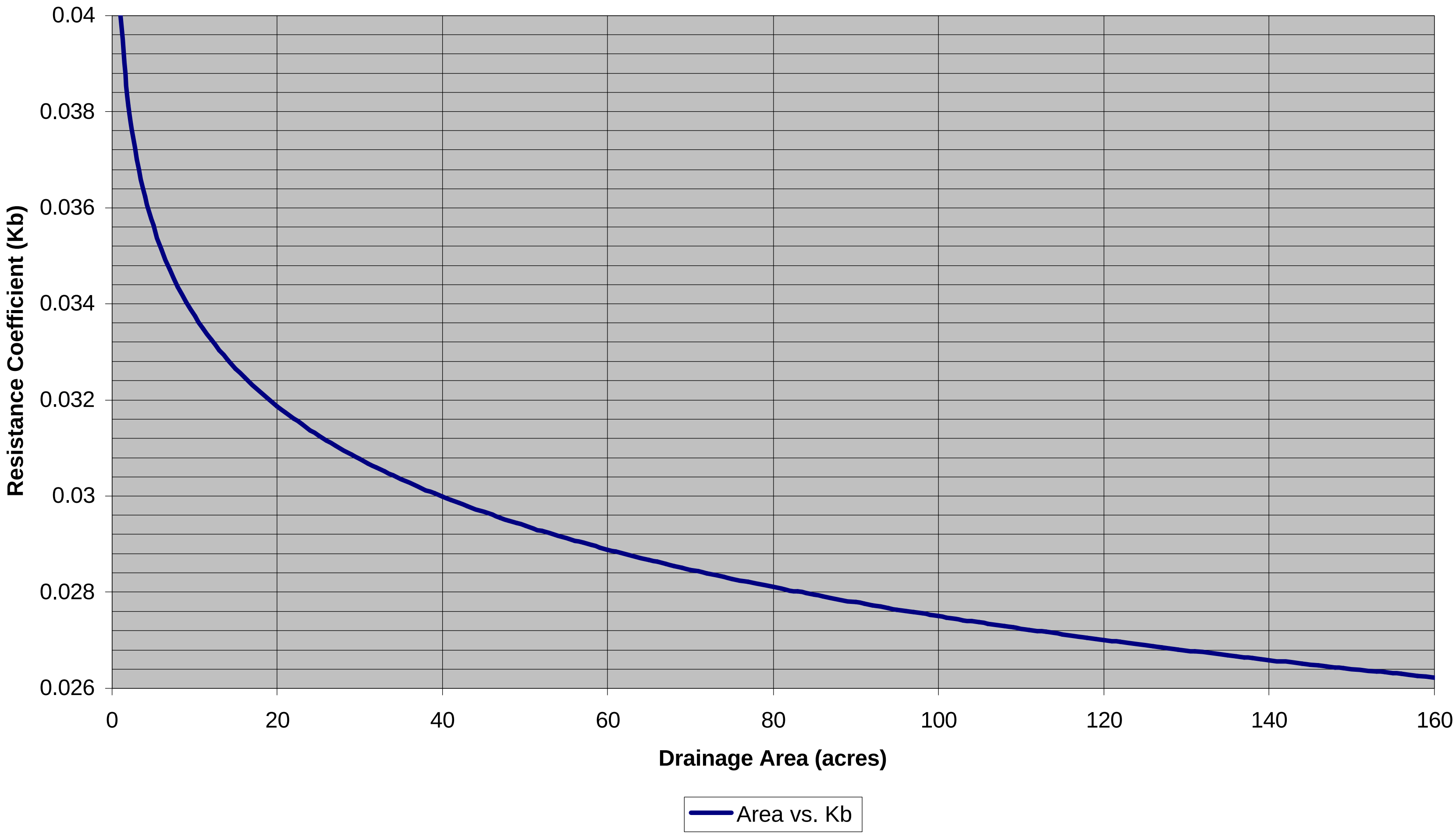


FIGURE 1-1
SHORT-DURATION RAINFALL RATIO ZONES FOR ARIZONA

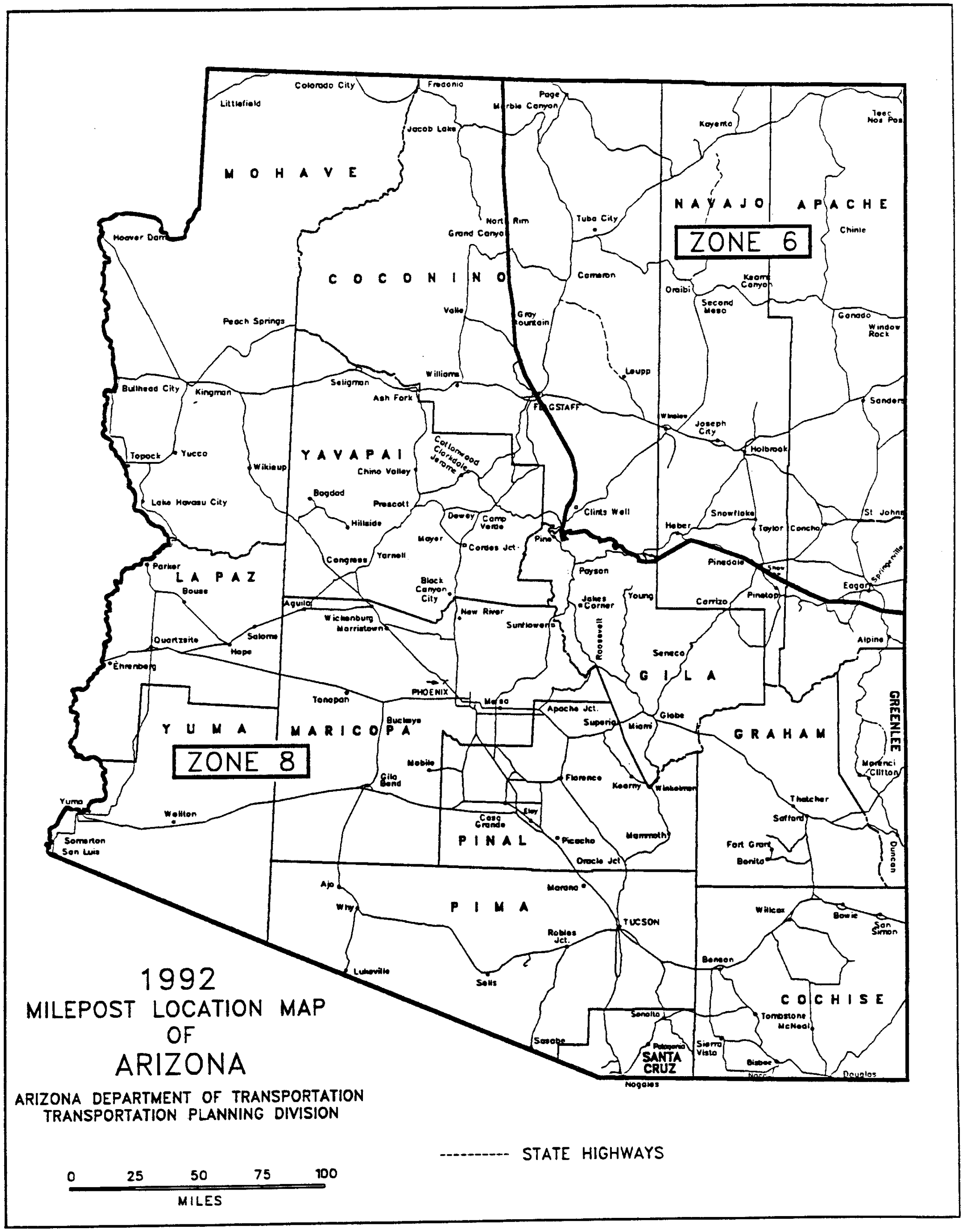


FIGURE 2-1
GENERALIZED I-D-F GRAPH FOR ZONE 6 OF ARIZONA

Example: For a selected 10-year return period, P_1 = 2.0 inches. T_C is calculated as 20 minutes. Therefore, (i) = 4.25 in/hr.

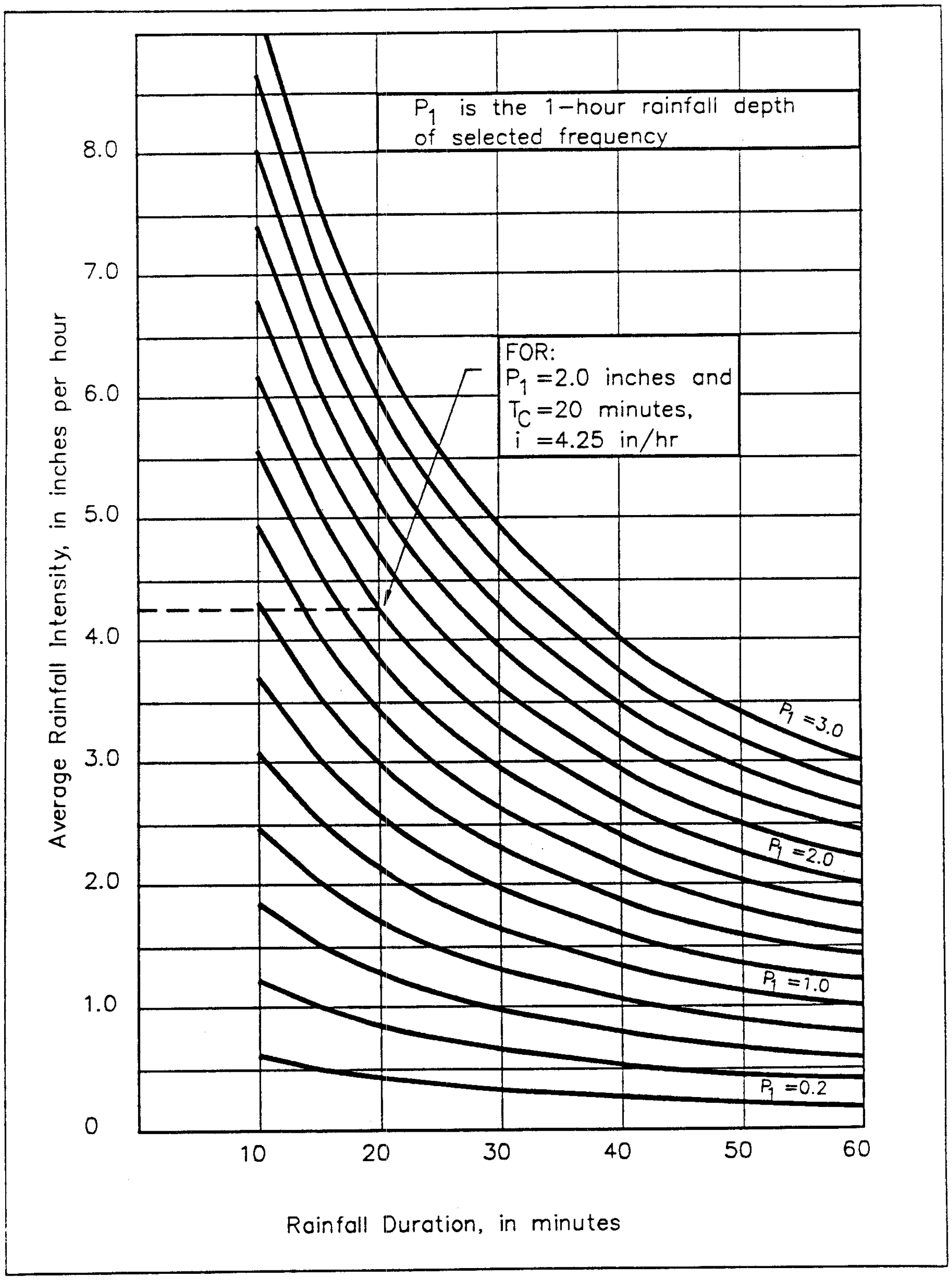
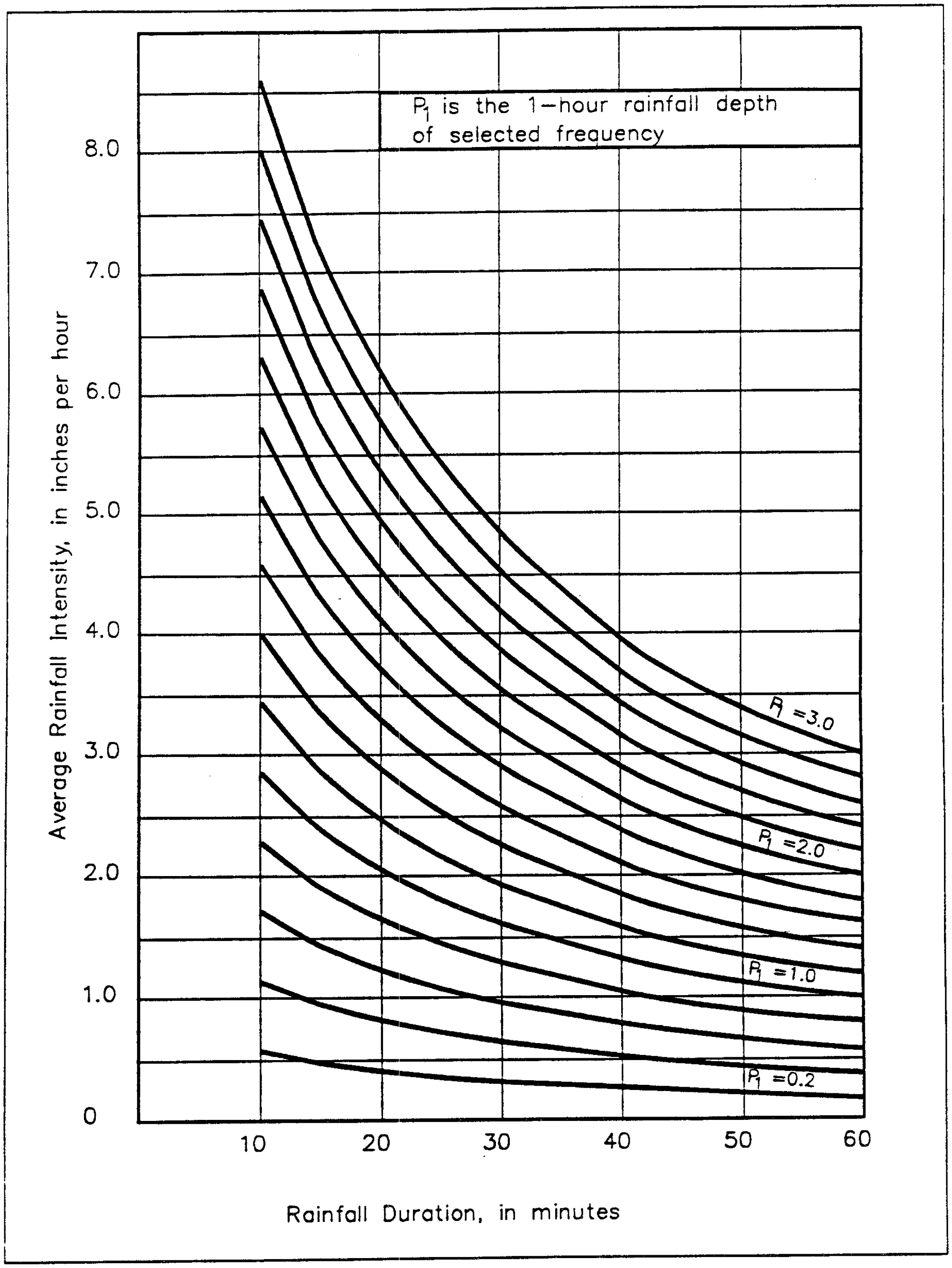


FIGURE 2-2
GENERALIZED I-D-F GRAPH FOR ZONE 8 OF ARIZONA



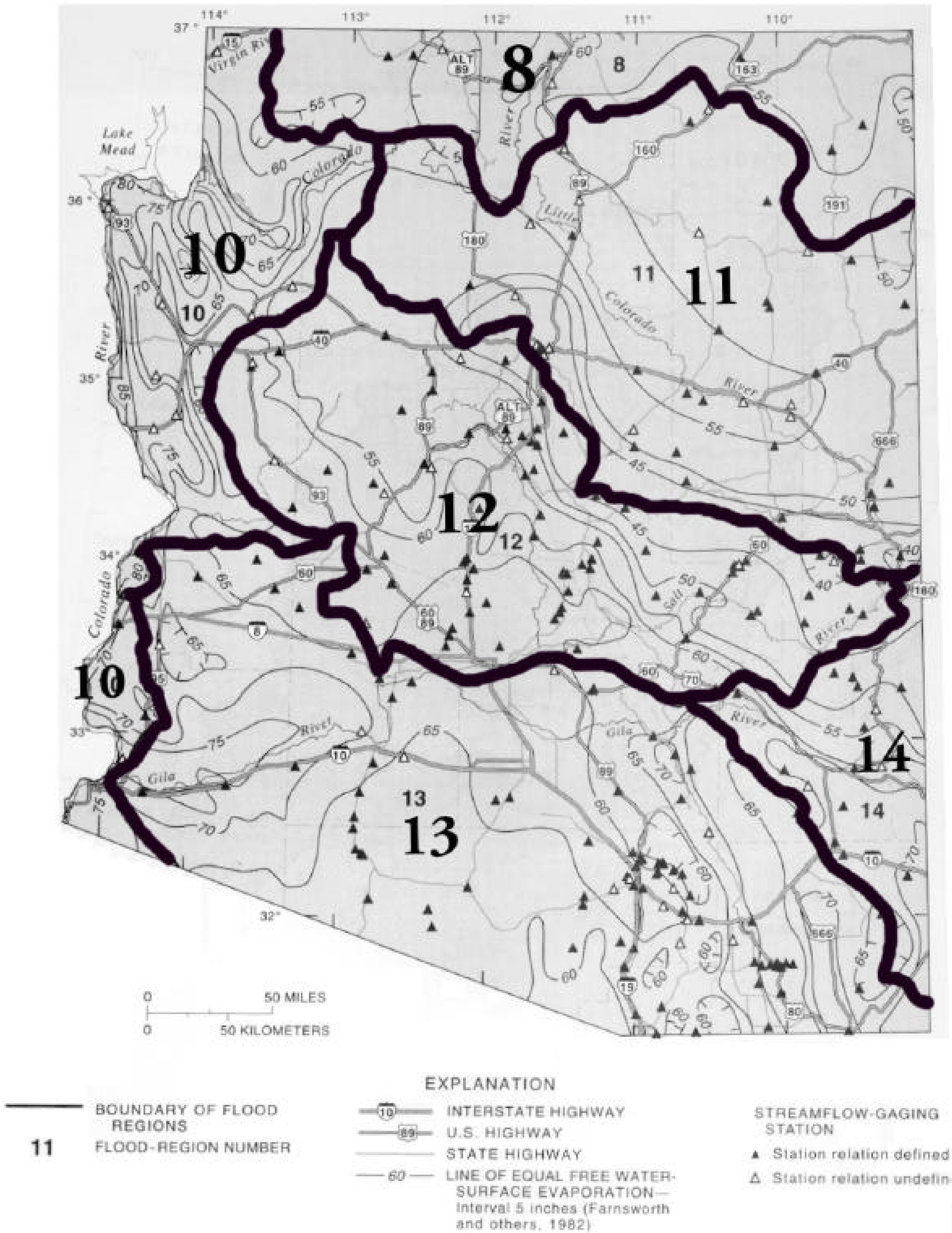
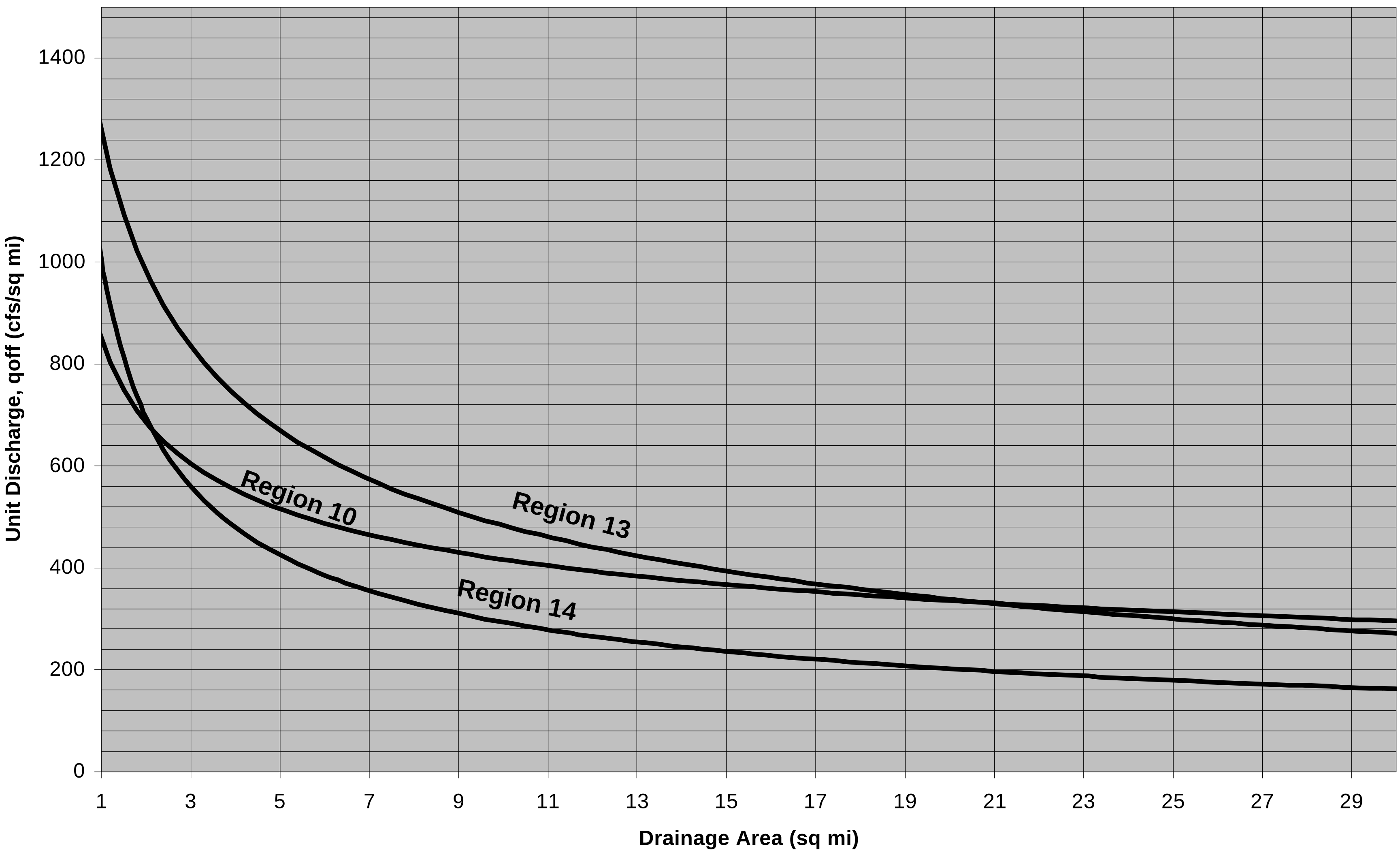
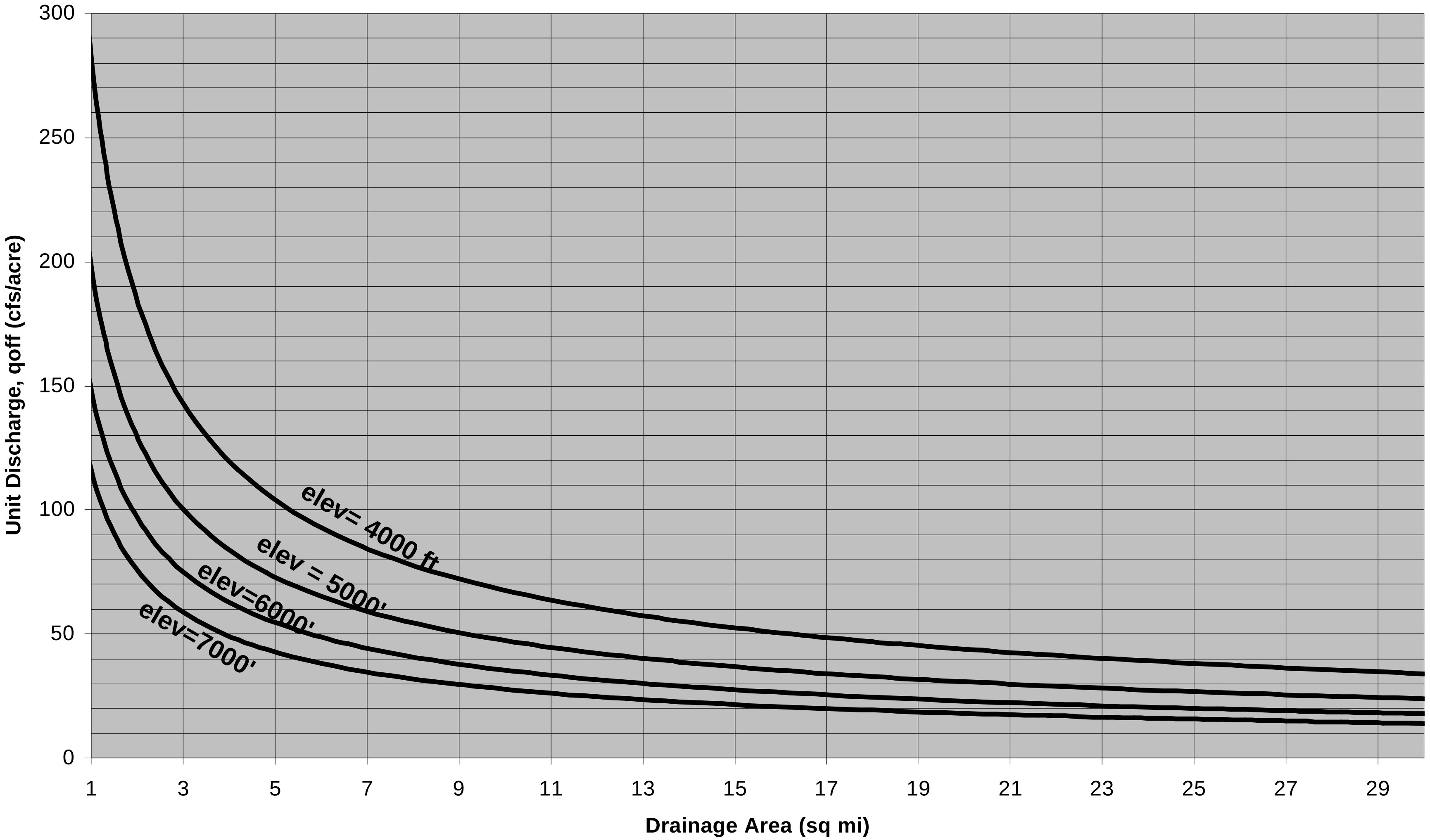


Figure 7. Flood regions in Arizona.

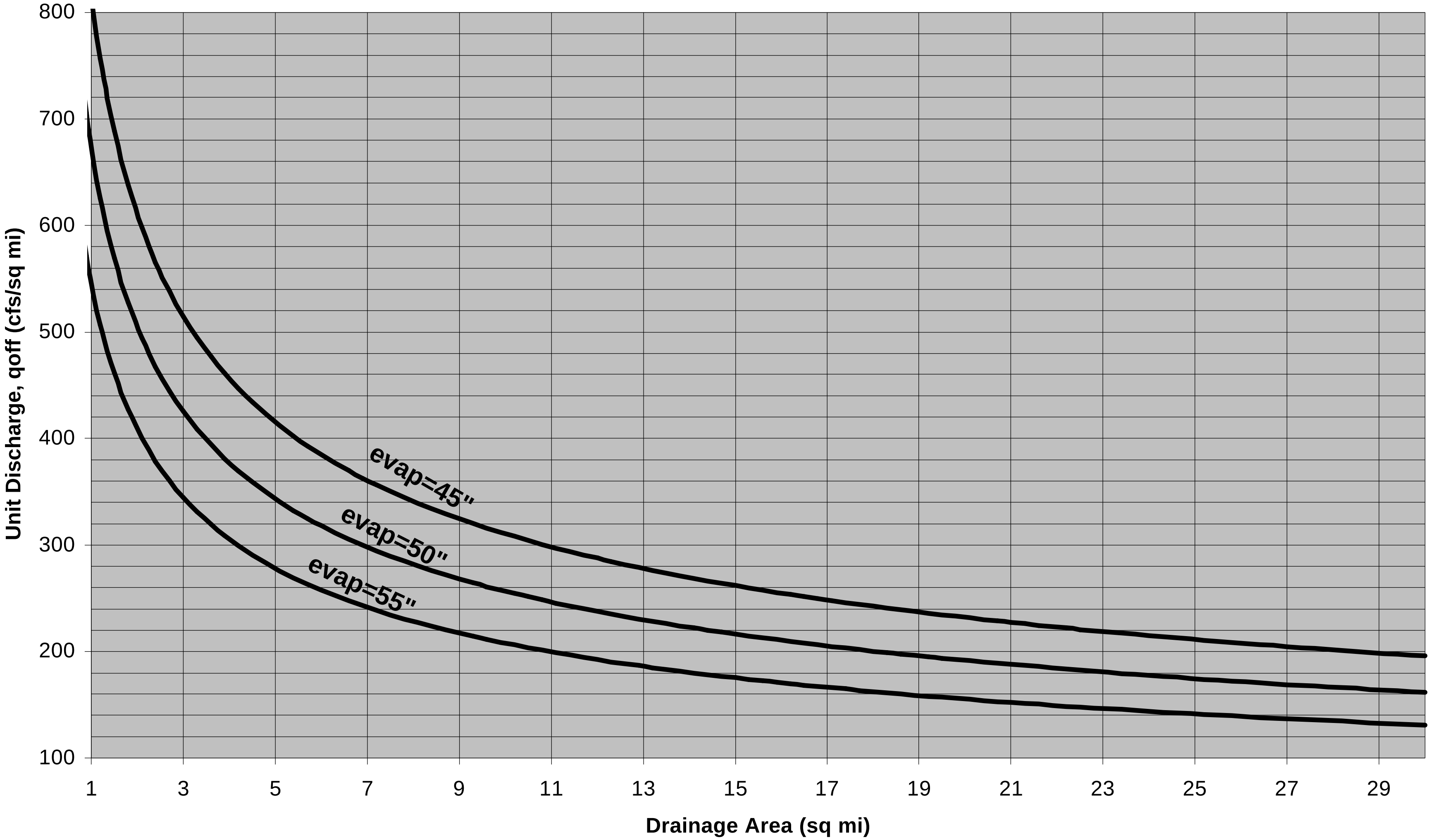
100-yr Unit Dischages for Arizona Regions 10, 13 & 14 from USGS Open File Report 93-419



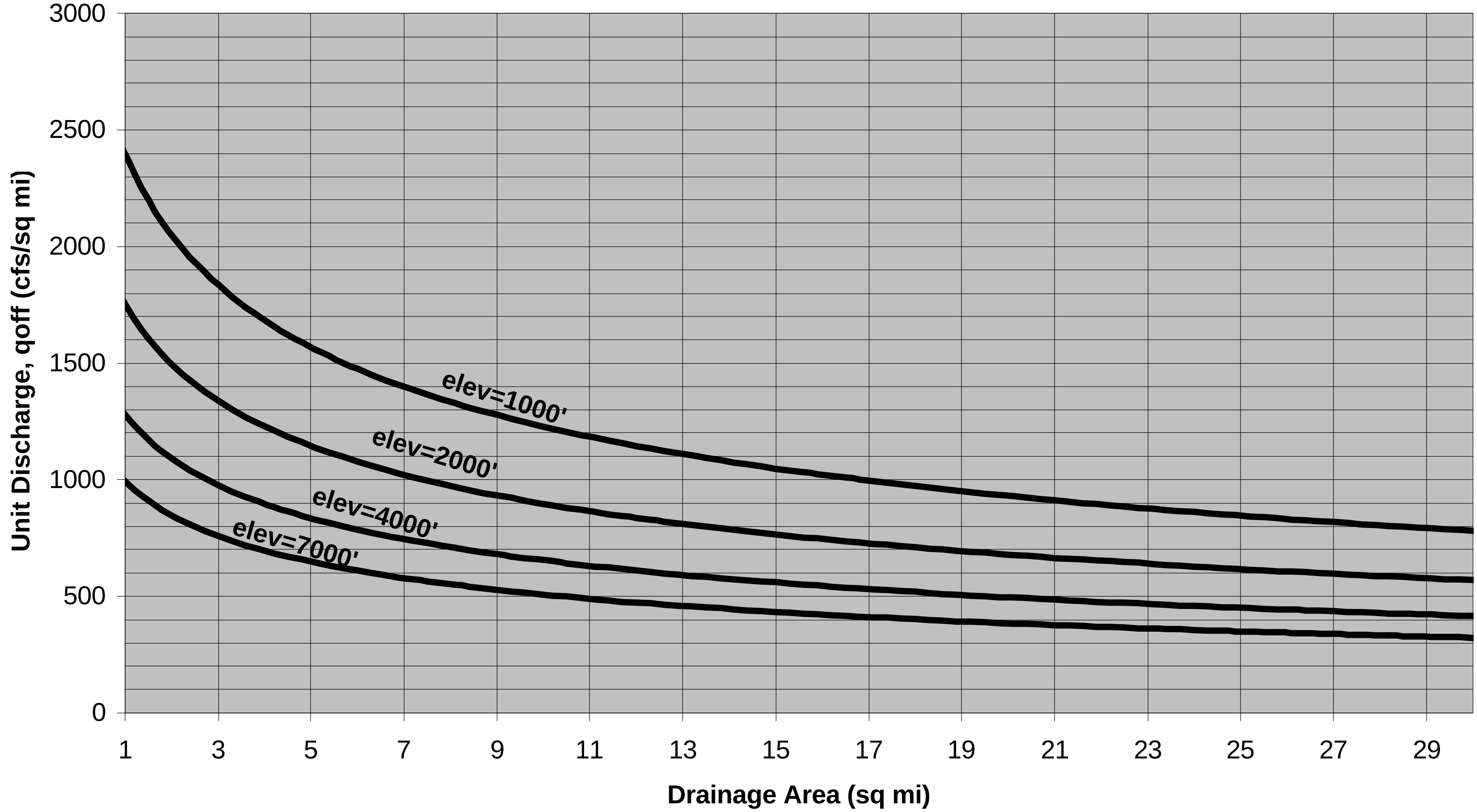
100-yr Unit Discharges for Arizona Region 8 from USGS Open File Report 93-419 for Various Values of Mean Basin Elevations (elev)

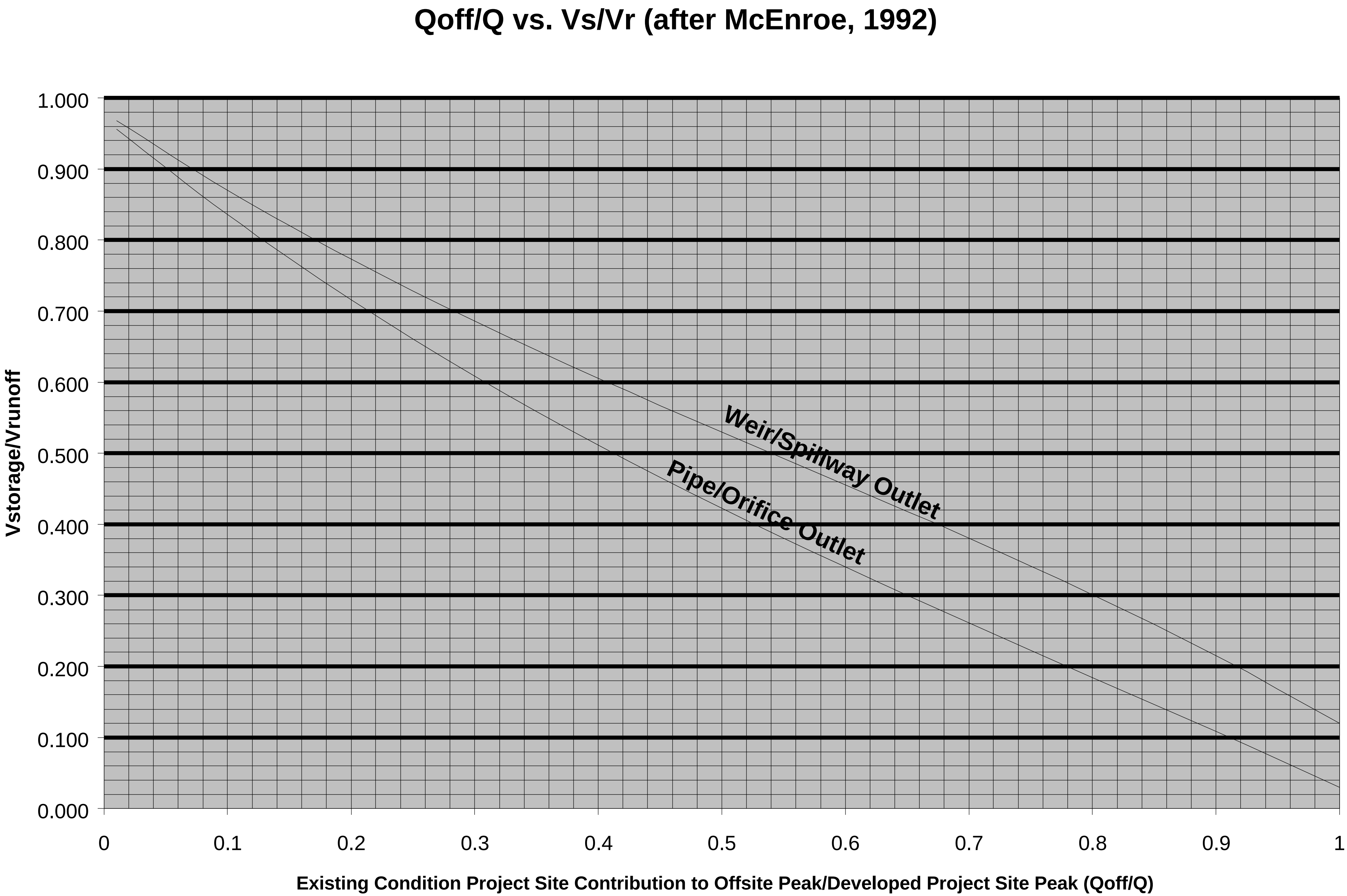


100-yr Unit Discharges for Arizona Region 11 from USGS Open File Report 93-419 for Various Values of Mean Annual Evaporation (evap)



100-yr Unit Discharges for Arizona Region 12 from USGS Open File Report 93-419 for Various Values of Mean Basin Elevation (elev)




Qoff/Q vs. Vs/Vr (after McEnroe, 1992)
Vstorage/Vrunoff
1.000
0.900
0.800
0.700
0.600
0.500
0.400
0.300
0.200
0.100
0.000
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Weir/Spillway Outlet
Pipe/Orifice Outlet
Existing Condition Project Site Contribution to Offsite Peak/Developed Project Site Peak (Qoff/Q)

Capacity Charts for the Hydraulic Design of Highway Culverts

Hydraulic Engineering Circular No. 10

November 1972

U.S. Department of Transportation
Federal Highway Administration

III. REQUIREMENTS AND LIMITATIONS FOR USE OF CHARTS

Because culvert flow problems vary in complexity it is difficult to express headwater-discharge relationships in simple curves or charts without some limitations. The culvert capacity charts are designed to provide an easy method for the direct selection of culvert size for the majority of highway culvert installations, but the following requirements and limitations for the direct use of the charts must be observed for correct solutions.

A. Requirements and Limitations

1. The culvert type under consideration must be represented by the chart as noted in the title. (Other inlet types can be used -- see B-1 next page.)

2. The culvert size must be included on the chart.

3. The culvert invert must be on a continuous straight-line slope from inlet to outlet, and slope downward in the direction of flow (not level).

4. The $L/100S_o$ ratio must not exceed the largest value shown on the chart for the size involved.

5. The headwater depth must be less than 2D for the size considered.

6. The elevation of the tailwater in the outlet channel must not submerge critical depth at the outlet. (Critical depth for various culvert sections may be found from charts in HEC No. 5.)

VII. CULVERT CAPACITY CHARTS

The culvert capacity charts in this section provide a means for selecting a culvert of adequate size to convey the design discharge rate per barrel without exceeding an allowable depth of headwater determined by the site conditions. The allowable headwater depth AHW, and the actual headwater depth HW that results from the culvert size selected, are measured in feet above the culvert invert at the inlet. The 36 culvert capacity charts are divided into 8 groups according to eight basic types of culverts as determined by barrel shape and material. The charts appear in the order of the list shown on the last page of this circular.

Each group of charts is preceded by an explanation of the factors determining the two main inlet types represented by the charts. Information regarding other inlets types classified as equivalent to one of the two types shown in the titles and other design data necessary to the use of each group of charts are also included. Tables of dimensions and cross-sectional areas of the available sizes of each type of culvert are given in some instances.

The procedures for accumulating design data and for selecting a culvert size as previously discussed are summarized in the following steps: (This information should be tabulated on a prepared design data sheet to be used as a work sheet and a record. See tabulation sheet in HEC No. 5).

1. Select the average frequency of the design flood.

2. Determine the estimated peak discharge of the design flood.

3. Obtain all site data. Plot a roadway cross section at the culvert site and a stream channel profile. Make a contoured site plan where necessary.

4. Establish the culvert invert elevations at inlet and outlet and the culvert length. Then determine the invert slope S_o and compute $L/100S_o$.

5. Determine the allowable headwater depth (or depths) AHW, considering the factors discussed in sec. V.

6. Compute the depth of flow in the stream channel (including flood plain) for the design flood, and determine TW depth.

7. Select one or more appropriate culvert types. Compute an approximate barrel area $A_b = Q/10$ to guide selection of type and possible numbers and sizes of multiple barrels. Compute the discharge rate Q per barrel if multiple barrels are used.

8. Determine if the culvert types selected together with the governing headwater, length and slope, meet requirements for the direct use of charts, sec. III.

   a. Select the culvert capacity chart for the culvert and entrance type to be considered.

   b. On the chart, locate the point of intersection of Q and AHW.

   c. Use the culvert $L/10CS_o$ and the $L/100S_o$ of the chart curves to determine the smallest culvert size which will result in an actual headwater depth HW equal to or less than AHW (sec. II C).

   d. Check tailwater as instructed in sec. III.

9. Culvert size may also be selected from the charts for some conditions where the requirements for direct selection of size from charts are not met and therefore step 8 above cannot be followed. These conditions include the following cases as described in sec. IV.

   Case 1 - Paved Invert C.M. Pipe or Pipe-Arch.

   Case 2 - Fully Paved C.M. Pipe.

   Case 3 - Rectangular Concrete Box sizes not in charts.

   Case 4 - Concrete or C.M. Circular Pipe sizes between those of chart curves.

   Case 5 - Oval Concrete Pipe sizes not in chart.

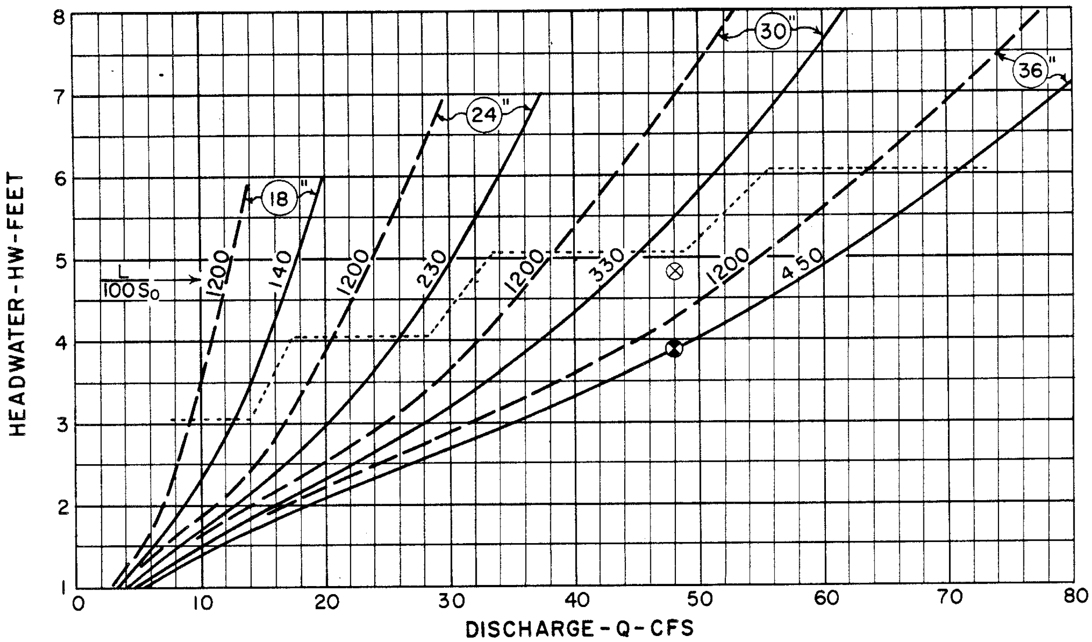
   Case 6 - Corrugated Structural Plate Pipe-Arch sizes not in chart.

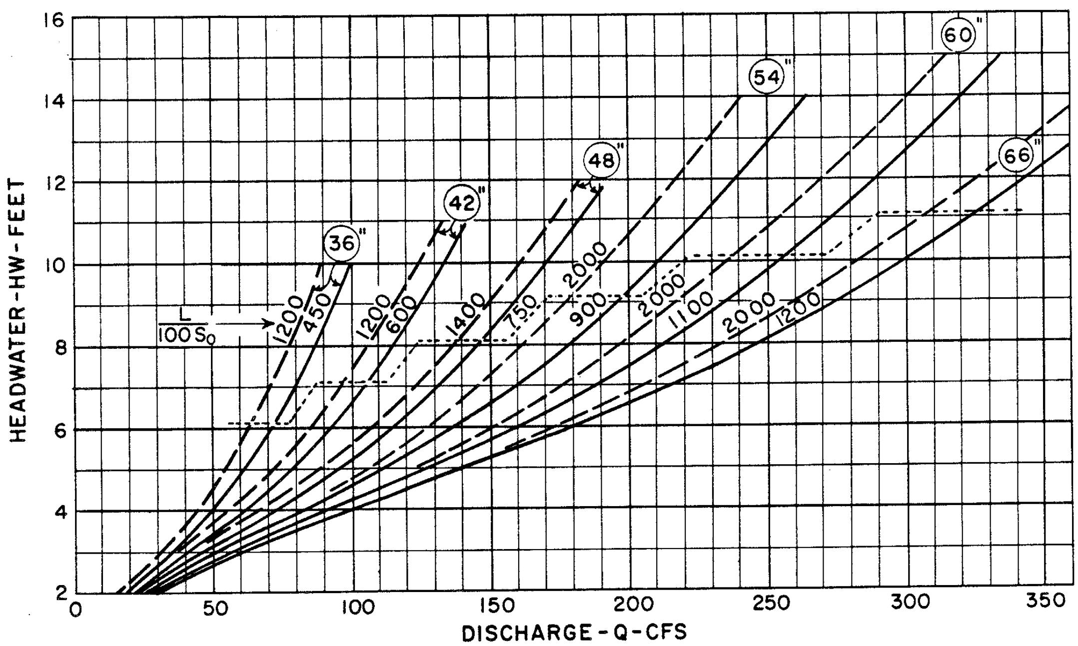
   Case 7 - Culvert slope zero (level invert).

   Case 8 - Broken slope culverts.

   Case 9 - $L/100S_o$ exceeds chart value.

CHART 11





EXAMPLE

⊗ GIVEN:
48 CFS; AHW = 4.8 FT.
L = 60 FT.; S_0 = 0.003

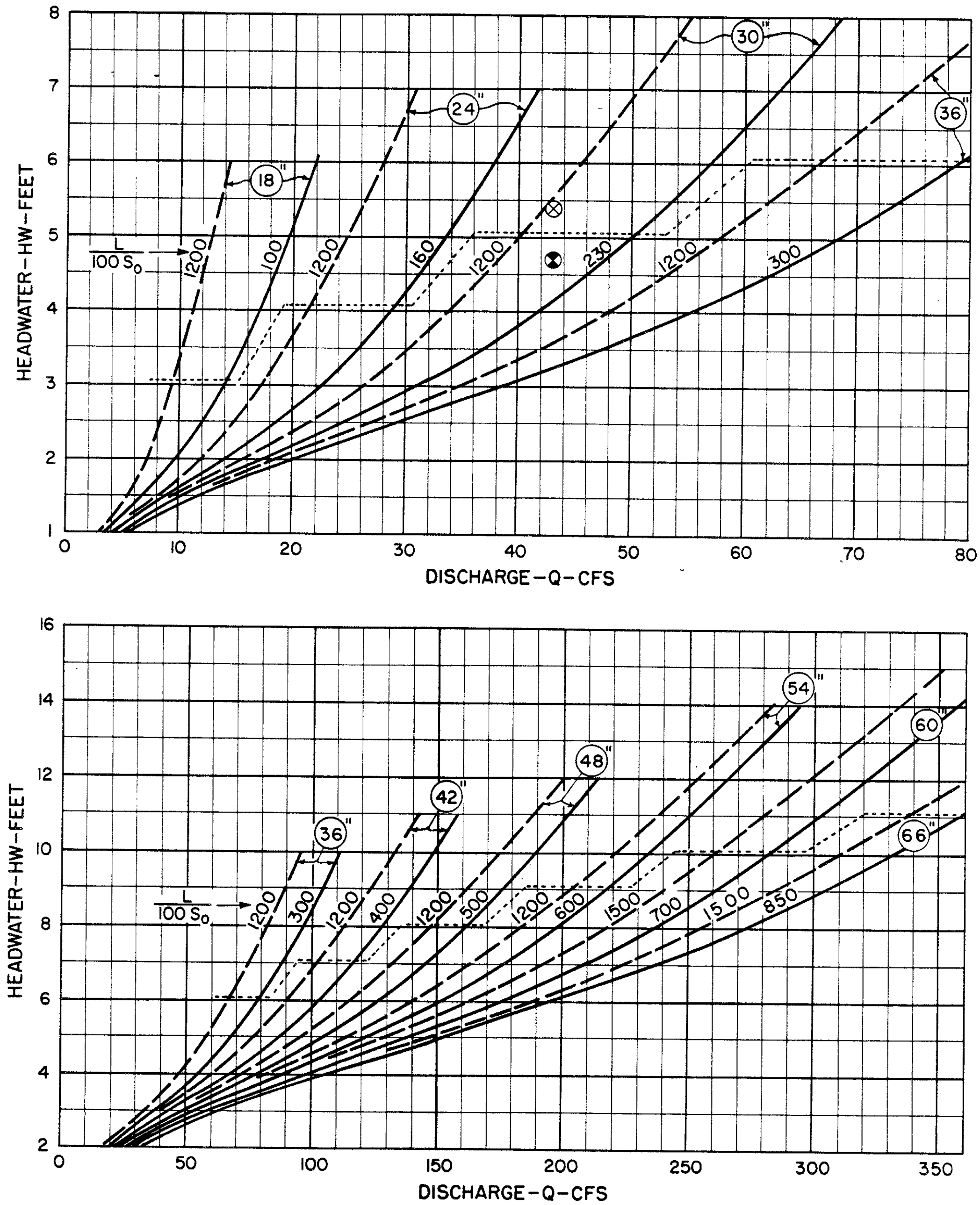
SELECT 36"
HW = 3.9 FT.

BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
SQUARE-EDGED ENTRANCE
18" TO 66"

10-37

CHART 13



EXAMPLE

⊗ GIVEN:
43 CFS ; AHW = 5.4 FT.
L = 120 FT.; S_0 = 0.002

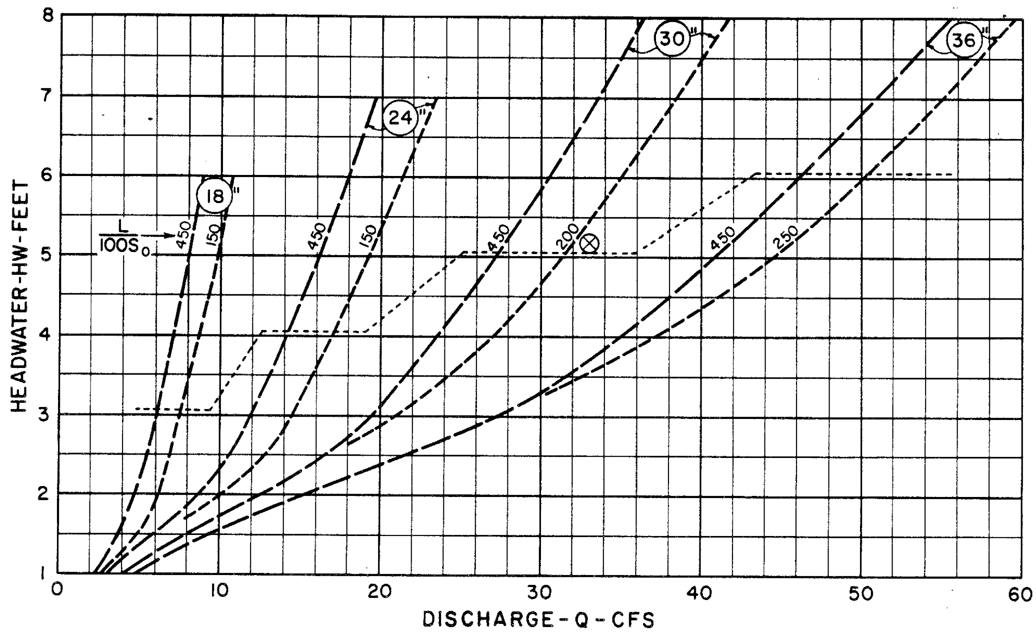
SELECT 30"
HW = 4.7 FT.

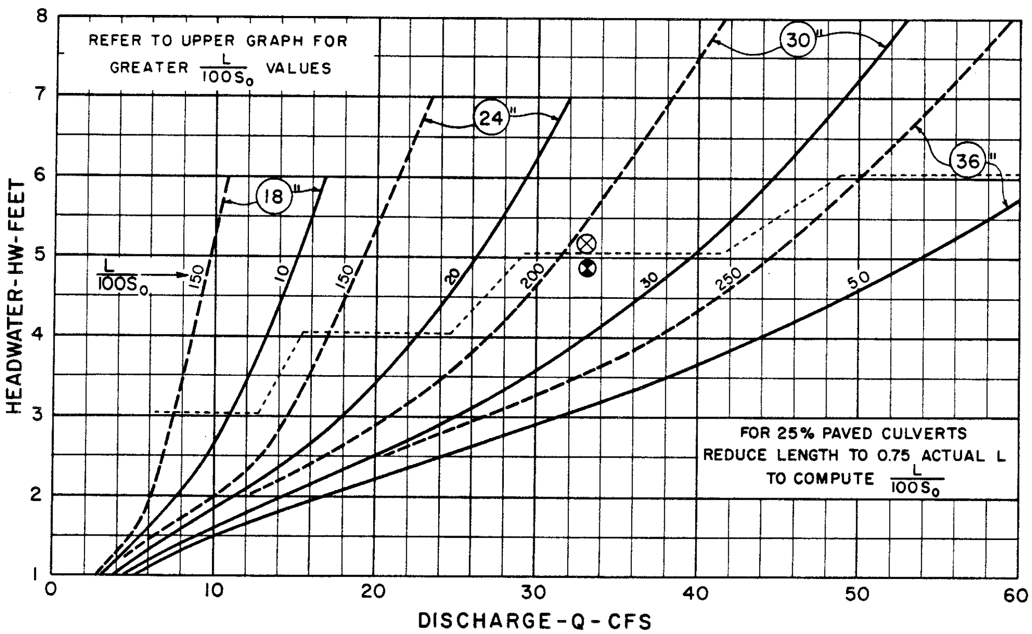
BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
GROOVE-EDGED ENTRANCE
18" TO 66"

10-39

CHART 19





EXAMPLE

⊗ GIVEN:
33 CFS; AHW = 5.2 FT.
L = 70 FT.; S_0 = 0.005

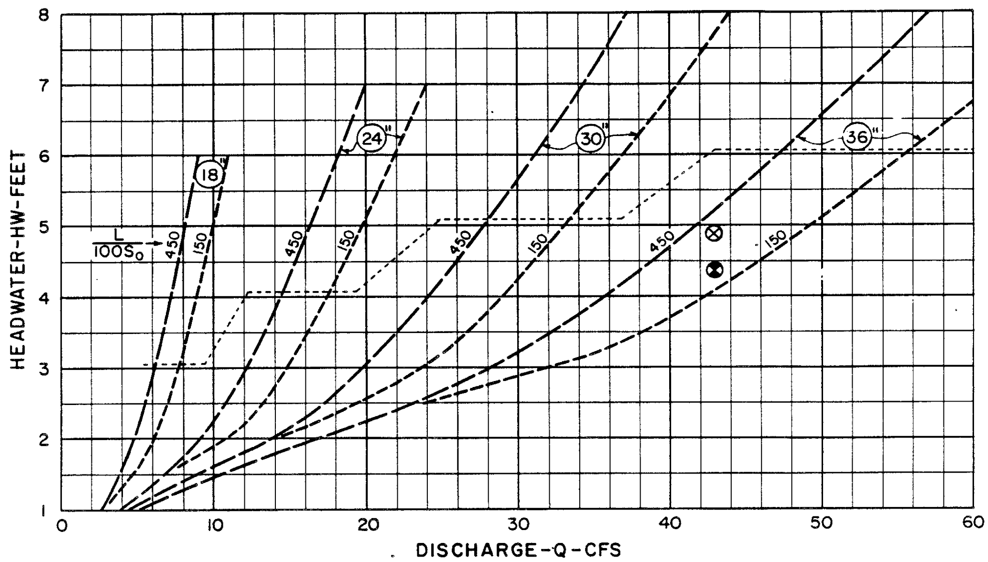
② SELECT 30" UNPAVED
HW = 4.9 FT.

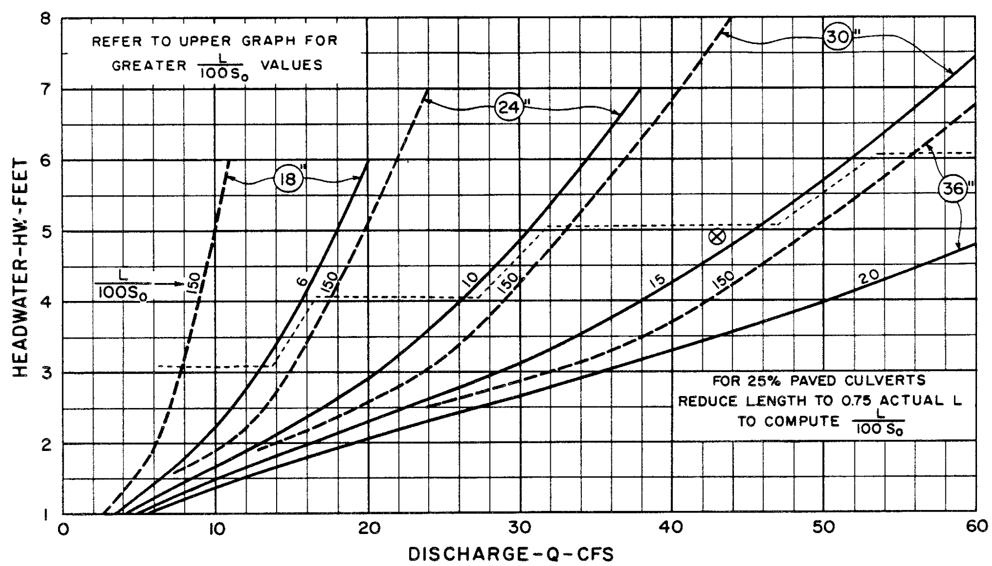
CULVERT CAPACITY
STANDARD
CIRCULAR CORR. METAL PIPE
PROJECTING ENTRANCE
18" TO 36" ○

BUREAU OF PUBLIC ROADS JAN. 1963

10-51

CHART 22





EXAMPLE

⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_o = 0.003

⊗ SELECT 36" UNPAVED
HW = 4.4 FT.

CULVERT CAPACITY
STANDARD
CIRCULAR CORR. METAL PIPE
HEADWALL ENTRANCE
18" TO 36" ○

BUREAU OF PUBLIC ROADS JAN 1963

10-54

APPENDIX C

Example Applications

LEVEL 1 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET				
Step	**Parameter Description**	**Equation/ Method Determined**	**Value**	**Units**

PROJECT DESCRIPTION:		40 acre residential subdivision in Prescott, 4 houses/acre density (moderately urban, see p. 7)			
1	Project Site Area	A	From site data	40	Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7	3.5	Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8	4.9	Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From Figure 1 of State Standard	2.4	Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3	.72	None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$	5.76	Acre-ft

NOTES:

A basin approximately 3 acres in area with a 3 foot maximum depth was designed with 4:1 side slopes. Six, 6" diameter pipes were provided at the low-point draining the pond into the wash which provides natural drainage for the site. Rock riprap is provided at the outlet to prevent erosion. Maintenance access is provided by a 6:1 ramp into the basin bottom.

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET **PAGE 1 OF 2**

Step	Parameter Description		Equation/ Method Determined	Value	Units
PROJECT DESCRIPTION:		40 acre residential subdivision in Prescott, (located in offsite watershed with mean elevation = 5000 ft.) 4 houses/acre density (moderately urban, see p. 7)			
1	Project Site Area	A	From site data	40	Acres
2	100-year, 6-hour rainfall depth	$P_{100,6}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 7	3.5	Inches
	100-year 24-hour rainfall depth	$P_{100,24}$	From ADOT Hydrology Manual (1993), Precipitation Map No. 8	4.9	Inches
	100-year, 1-hour rainfall depth	$P_{100,1}$	From $P_{100,1}$ Chart	2.4	Inches
3	Developed condition runoff coefficient for project site	C	From ADOT Hydrology Manual (1993), Figure 2-3	.72	None
4	Developed condition 100-year, 1-hour runoff volume	V_r	$Vr = (CAP_{100,1})/12$	5.76	Acre-ft
5	Length of longest flow path of site	L	ADOT Hydrology Manual (1993), page 2-4	0.27	Miles
	Watershed resistance coefficient for site	K_b	From Kb Chart contained herein	0.03	None
	Slope of longest flow path of site	S	ADOT Hydrology Manual (1993), page 2-4	26	Ft/mile
	If Tc (c) ≠ Tc (a), reset Tc (a) to last value of Tc (c) and repeat these steps until Tc (c) = Tc (a) or Tc (c) ≤ 0.17 hrs Assumed Time of Concentration	Tc (a)	Assumed (assume a value of 0.17 hours as a first guess)	0.17	Hours
	Rainfall intensity	i	ADOT Hydrology Manual (1993), Figure 2-1 or 2-2	6.75	Inches/ Hour
	Calculated Time of Concentration	Tc (c)	$Tc = 11.4\ L^{0.5}\ K_b^{0.52}\ S^{-0.31}\ i^{-0.38}$ (Eqn. 2-2 from ADOT Manual) **If Tc (c) ≤ 0.17 hrs (10 min.) use Tc (c) = 0.17 hrs.**	0.17	Hours
	Developed condition 100-year peak discharge for project site	Q	Q = CiA	194.5	Cfs

LEVEL 2 STORMWATER DETENTION/RETENTION PROCEDURE WORKSHEET PAGE 2 OF 2					
Step	**Parameter Description**		**Equation/ Method Determined**	**Value**	**Units**
6	Area of offsite watershed	A_{off}	Per definition in Level 2 procedure	5	Mi^2
	Offsite watershed runoff rate	q_{off}	From 100-year Unit Discharge chart for appropriate region	755	Cfs/ Sq mi
	Contribution of project site to offsite watershed peak discharge	Q_{off}	$Q_{off} = A\ q_{off}/640$	47.2	Cfs
7	Ratio of design outflow to design inflow	Q_{off}/Q	Q_{off}/Q	0.243	Ratio
	Ratio of required storage volume to runoff volume	V_s/V_r	From Q_{off}/Q vs. V_s/V_r Chart	0.67 (pipe outlet)	Ratio
8	Required storage volume	V_s	$V_s = V_r\ (V_s/V_r)$	3.82	Acre-ft
9	Outflow structure		Use HEC-10 pipe outflow structure design charts (Appendix B of state standard) or other reference	3 – 50' x 24" CMP outlet at 0.5% slope	

NOTES:

A basin approximately 2 acres in area with a 3 foot maximum depth was designed with 4:1 side slopes. Rock riprap is provided at the outlet to prevent erosion. Maintenance access is provided by a 6:1 ramp into the basin bottom.