DRAINAGE DESIGN MANUAL

for Maricopa County, AZ

Technical Documentation Notebook – Volume 1 of 2

Correspondence & Hydraulics

October 2002

Flood Control District
of Maricopa County
2801 West Durango Street
Phoenix, Arizona 85009
(602) 506-1501

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAINAGE DESIGN MANUAL DOCUMENTATION NOTEBOOK

The purpose of this document is to provide a general tracking and background of the changes to the 1995 edition of the Drainage Design Manual for Maricopa County. This document is organized into two volumes. Volume I contains correspondence and is the key for understanding the changes to the manuals. The Correspondence Volume includes meeting minutes, memorandums and letters documenting discussions and decisions made regarding new methodologies, procedures and techniques for both the *Hydrology* and *Hydraulics Manuals* as well as data tables, figures and examples. Initially, a manual was to be prepared for the City of Phoenix only. Hydrologic and to a certain extent hydraulic methodologies and procedures for this new manual would have deviated somewhat significantly from the methodologies and procedures for the Flood Control District of Maricopa County. Much of the correspondence prior to 1998 dealing primarily with hydrologic issues was specific to new methodologies and procedures proposed for the City of Phoenix. After preparation of the manual began, the City of Phoenix and the Flood Control District of Maricopa County entered into an agreement that resulted in the City of Phoenix adopting the current Flood Control District of Maricopa County manuals with agreed upon revisions. This intergovernmental agreement primarily impacted the *Hydrology Manual*. The City of Phoenix agreed to adopt the current Flood Control District of Maricopa County hydrologic methodologies with improved/simplified procedures with additional focus on methodologies for the development of design discharges for more frequent flooding events. Correspondence prior to this agreement is provided primarily for an overall perspective on the project.

The second section of Volume I is the documentation for the *Hydraulics Manual*. Documentation for the *Hydraulics Manual* consists of copies of references used in the development of new methodologies, procedures, techniques and data. Only references considered not commonly available are provided in this volume.

Volume II is the documentation for the *Hydrology Manual*. Documentation of the *Hydrology Manual* is organized into two sections; technical analyses and testing/ verification. The technical analyses section includes source data and analyses used in the development of the methodologies, procedures and techniques presented in the new chapters. The testing and verification section is further divided into two subsections. The first subsection documents analyses conducted of current City of Phoenix hydrologic procedures in comparison to the current FCDMC hydrologic procedures. The second subsection presents the data and results for the testing of the multiple frequency modeling procedures.

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To: Distribution

From: George Sabol

Date: 8 September 1997

Reference: **PHOENIX STORM DRAINAGE DESIGN MANUAL**
NEW PRECIPITATION ANALYSIS FOR PHOENIX
FILE: 28900040

The source of design rainfall information was, and still is, the NOAA Atlas 2 for Arizona along with a few supplemental publications by other Federal government agencies. However, the need for a revised rainfall analysis of depth-duration-frequency statistics and other rainfall design information for Arizona has been recognized since the mid-1980s. At the time that the Flood Control District of Maricopa County (FCDMC) and the Arizona Department of Transportation (ADOT) were producing its hydrology manuals (from 1986 through about 1992), there was an effort to bring about a reanalysis of rainfall data. That process culminated in an agreement by NOAA to undertake a regional study of rainfall data. Various entities, such as ADOT, FCDMC and other state and county agencies within the region cooperated in financing the NOAA study. That study was initiated in October 1991 and was to have been completed in three years. The document to be produced is NOAA Atlas 14 (semi-arid region precipitation study) and that atlas will cover all or parts of about six states.

I was involved in the initial contacts with NOAA and have had some minor involvement in staying informed about the study since 1991. Over the past few weeks, I have discussed the project with several persons in order to determine the status of that study. The best source of information is the NOAA Project Manager, Dr. Lesley Julian. The status of the study is as follows.

<u>**Isohyetal Maps**</u>

- Draft isohyetal maps for 2- and 100-year frequency, 1-, 6- and 24-hour duration have been prepared. Those drafts are being sent to Mr. Larry Scofield (Arizona Transportation Research Center) on 27 August 1997.

- I contacted Larry Scofield and requested a copy of those maps and any previous study reports that may be useful to us. He will provide those to me.

- Those maps are apparently in English units and there is a question of whether the final product will be English or metric units. The Phoenix manual is to be in English units, but many of the project sponsors (such as ADOT) will require metric unit products. With the Federal initiative for conversion to metric, I anticipate a metric unit product. Therefore, there may be the need for us to perform a conversion or otherwise repackage those maps. This is presently unknown.

Rainfall Area Reduction Factors

- This is a topic of great interest and need. FCDMC adopted a Corps of Engineers criteria for the 6-hour storm based on historic storms in Arizona, and another criteria for 24-hour storms. ADOT uses the criteria in NOAA Atlas 2 which was originally developed by the National Weather Service (NWS) based on midwest storms.

- NOAA is presently working on this topic, but preliminary results probably will be not available until about mid-December.

Temporal Storm Distributions

- Again, this is a topic of great interest and need. FCDMC developed a 6-hour design storm and adopted an SCS 24-hour storm. ADOT uses a hypothetical 24-hour storm.

- NOAA has developed temporal distributions for 12-, 24- and 72-hour storms. They have also looked at seasonal rainfall patterns for "severe" and "garden variety" storms.

- I will obtain and review what has been produced in this regard.

Lesley was very interested in our plan to produce an electronic version of our manual. In that regard, I sent her a copy of the PREFRE program that is used in conjunction with rainfall statistics from the NOAA Atlas to produce tables of rainfall depth-duration-frequency and intensity-duration-frequency. She will evaluate the use or modification of that program with the new NOAA Atlas.

At this point, my work plan is as follows:

1. Obtain all information that is available from NOAA concerning its new study.

2. Perform a preliminary review of that information.

3. Review the draft report that presumably will be available in mid-December.

4. Within a month of obtaining the draft report, provide an assessment of information that will be available with the new NOAA atlas.

5. Finalize a work plan and schedule for the rainfall section of the manual. This will probably result in some rescheduling of some of the work products because of the delays in obtaining information for the NOAA study.

6. I will report on this topic at our next meeting, which is scheduled for 12 September.

George V. Sabol, PhD, PE
Senior Associate

Attachment

Distribution: Robert Gofonia, City of Phoenix
Gary Benton, City of Phoenix
Ralph Goodall, City of Phoenix
Ken Lewis, KVL Consultants

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7 November 1997
File: 28900042

NOAA/NWS W/OH2
1325 East-West Highway
Silver Springs, MD 20910

Attention: Lesley T. Julian, PhD

Dear Lesley:

Reference: PHOENIX STORM DRAINAGE DESIGN MANUAL

Since our phone conversation on 22 August 1997, I have been in communication with and obtained information from Larry Scofield (ATRC), V. Ottozawa-Chatupron (ASLD), Joe Warren (ADOT) and Steve Waters (FCDMC). I have obtained the following concerning the Semi-Arid Precipitation Frequency Study (SA Study):

Draft isopluvial maps dated 27 August 1997 for the following:

A. 2-year, 1-hour

2-year, 6-hour

100-year, 1-hour

100-year, 6-hour

NOTE: Those maps were obtained by plotting files from a diskette provided by Larry Scofield.

B. Minutes for five Semi-Annual Meetings:

5 December 1991

10 June 1992

7 December 1992

9 September 1993

7 November 1994

C. Sixteen Quarterly Progress Reports for the Period February 1992 through November 1995

I am in the processing of reviewing that information for our client, the City of Phoenix, in regard to using the results from the SA Study in a new Phoenix Storm Drainage Design Manual. At this time, I have the following questions:

1. Considering the information that I have indicated herein, do I have all of the relevant and "best" available information for reviewing the status and work product for the SA Study?

2. As I understand, the SA Study is also to provide information concerning the spatial and temporal distribution of storms. Such depth-area-duration and depth-area relations are needed for Phoenix (and Arizona) due to the questionable applicability of some existing relations that are currently being used. Is the SA Study still proceeding along those lines? What is presently available, and/or when will those results be available?

3. Orographic factors in the Phoenix meteorologic/hydrologic area probably significantly influence precipitation. The Phoenix area appears to be very complex in this regard with mountain ranges nearly encircling the City. Observation by myself and others seems to indicated preferred storm paths or storm hot-spots. Those may be influenced by orographic factors and possibly by urbanization in the Valley. Do orographic features play a role in the development of the isopluvial maps? To what extent? Is there an accounting for urban influences or storm tracks, etc.? In this regard, are more "detailed" or larger scale maps of the Phoenix meteorologic/hydrologic area available that may provide better detail of the spatial depth-duration-frequency relations (isopluvial maps) for this area?

4. Have comparisons been made, formally or informally, of the difference between the NOAA Atlas 2 isopluvials and those from the SA Study for the Phoenix area? If so, I would be interested in the results.

Over the next few weeks, I will be assessing the presently available SA Study results in regard to depth-duration-frequency for use in Phoenix. I will send you the comparisons that I compile and will ask you to review my work. I do not want to make an error or draw the wrong inference from the information that I have. Your assistance will be greatly appreciated.

The SA Study has great interest to me. Incidentally, I made the initial contact with John Vogel concerning the need for that study back in 1989 or 1990. Please keep me informed of your results. I would like to receive any future reports and to attend review meetings. I understand that you made presentations on this project recently in both San Diego and Laughlin. Regrettably, I could not attend either meeting. If you had publications or presentation handouts, I would appreciate copies. It has been some time since the last review meeting. For my part, I would find such a project meeting useful. Do you have plans for a review meeting sometime in the near future?

Thank you for your assistance. Please keep me informed and I will do likewise.

Sincerely,

STANTECH CONSULTING INC.

George V. Sabol, PhD, PE
Senior Associate

cc: Mr. Robert Gofonia, City of Phoenix
Mr. Ralph Goodall, City of Phoenix
Mr. Gary Benton, City of Phoenix
Mr. Ray Acuna, City of Phoenix
Mr. Larry Scofield, ATRC
Mr. Joe Warren, ADOT
Mr. V. Ottozawa-Chatupron, ASLD
Mr. Steve Waters, FCDMC

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To: Distribution

From: George V. Sabol

Date: 12 January 1998

Reference: **DESIGN RAINFALL DESIGN MANUAL**
FILE: 28900042

The purpose of this memorandum is the following:

1. Present existing design rainfall criteria that are presently in common use for drainage design in the Phoenix metropolitan area.

2. Provide a discussion of certain items relative to adopting a design rainfall criteria for Phoenix.

3. Provide recommendations for the Phoenix design rainfall criteria.

Existing Design Rainfall Criteria

There are two rainfall criteria that are in common use for stormwater drainage design; that by the Flood Control District of Maricopa County (the Maricopa County method), and that by ADOT. Those two methods are presented in the respective manuals, and copies of those rainfall sections are provided in Attachments 1 and 2, respectively.

Maricopa County Method (1992)

The Maricopa County Manual addresses the following design storms;

1. 100-year, 2-hour storm for determining the volume of runoff for retention/detention facilities, shown in Figure 2.15.

2. 100-year, 6-hour distribution for drainage areas, generally less than 100 square miles, shown in Figure 2.16.

   The 6-hour storm is based on the Corps of Engineers analysis of the 19 August 1954 Queen Creek storm with adjustment so that the Pattern No. 1 mass curve is the equivalent hypothetical distribution.

3. A general storm for drainage areas larger than 100 square miles. The District has been using the SCS Type II distribution for the general storm in recent studies.

The corresponding depth-area reduction factors are:

1. For the 2-hour storm, no depth-area reduction.

2. For the 6-hour storm, the depth-area reduction curve shown in Figure 2.14 is to be used. That curve is based on the historic 19 August 1954 Queen Creek storm.

3. For the general storm, a site-specific curve is to be chosen or developed. In general, the depth-area reduction factors from the NOAA Atlas 2 are used.

The Maricopa County manual provides a general discussion of storm occurrence and types of flood producing storms in Maricopa County. That manual is information as well as directive as to design rainfall criteria. Isopluvial maps are provided for the Maricopa County area, and those maps are based on information in the NOAA Atlas 2.

ADOT Method

The ADOT manual defines two design storms;

1. The 6-hour storm for drainage areas less than or equal to 1.0 square mile.

2. The 24-hour storm for drainage areas larger than 1 square mile.

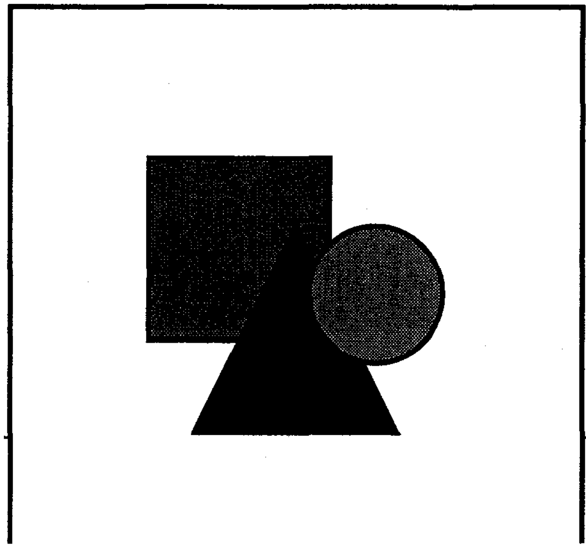
In both cases, the temporal distribution is by the hypothetical distribution. The ratios of rainfall for durations less than 1-hour are from Arkell and Richards (1986) and not from the NOAA Atlas 2.

The ADOT method for depth-area reduction is by curves in NOAA Atlas 2. Those point rainfall area reduction curves are not reproduced in the manual and the depth-area reduction is made automatically by use of the PH record when using the prescribed HEC-1 procedure.

The ADOT manual is more instructive than the Maricopa County manual, but there is little presentation of background information. The isopluvial maps are provided in the appendices and show the entire state. They are based on information in the NOAA Atlas 2, and the isopluvial lines are in color.

Discussion Items Relative to Design Rainfall Criteria Duration and Frequency of Design Storm

Duration & Frequency of Design Storm



Maricopa County uses a 6-hour design storm, ADOT generally uses a 24-hour storm (except for drainage areas smaller than 1 square mile for which the 6-hour storm is used). Mixing storm durations (and frequencies) within a multi-jurisdictional project presents technical problems as well as administrative ones. Within the Phoenix metropolitan area, the 24-hour ADOT criteria is generally more severe than criteria by others within the metropolitan areas. The Phoenix design storm(s) duration and frequency need to be selected with due consideration of:

1. Potential multi-jurisdictional impacts,
2. existing City of Phoenix policy and practices, and
3. construction cost for facilities.

Temporal Distribution of the Design Storm

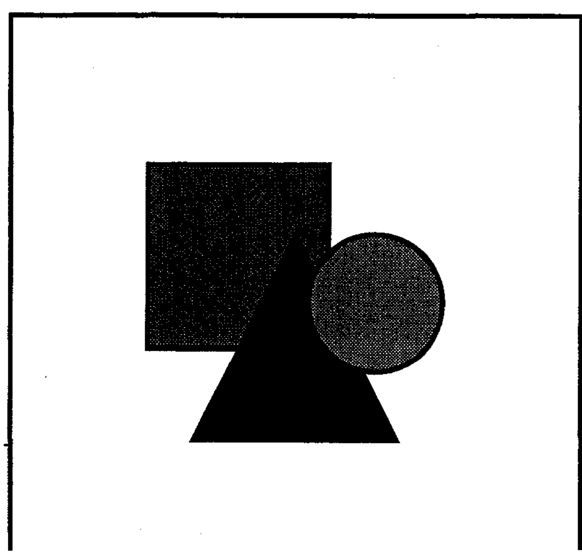
Maricopa County uses a design criteria based on an historical storm. There is a certain hydrologic appeal to this approach which is also used by the Corps of Engineers. The District has need to be consistent with Corps practices, but that is generally not the case for the City of Phoenix. The Maricopa County method may excel in regard to hydrologic "accuracy" but is deficient in regard to "practicality." ADOT uses the hypothetical distribution which is very easy to implement with HEC-1. In general, both the Maricopa County and the ADOT distributions are quite similar for smaller drainage areas, which will generally by the case for drainage studies in the City of Phoenix.

Depth-Area Reduction Factors

Maricopa County uses a point rainfall depth area-reduction curve that is specific to its historic 6-hour storm. It should not be used except for that specific criteria. ADOT adopted the curves in NOAA Atlas 2, and those are considered by many (including NOAA) to be inappropriate for the southwest. The NOAA Atlas 2 curves generally result in larger rainfall depths for larger storm area than actually occurs. Depth-area reduction curves for Arizona are available through NOAA (Arkell and Richards, 1984) that should be considered for use in Phoenix. That curve is shown in Attachment 3 along with a copy of the NOAA Atlas 2 curves. A comparison of point rainfall reduction factors by those two methods is also provided in Attachment 3.

Recommendation

1. Use the hypothetical distribution.
2. Tentatively adopt the depth-area reduction curves in Arkell & Richards.
3. Use the test watersheds to compare results from the recommended method (with duration and frequency to be selected) to results by both the ADOT method and the Maricopa County method.



George V. Sabol, PhD, PE
Senior Associate
Water Resources Division

Attachment

cc: Ralph Goodall, COP
Gary Benton, COP
Ray Acuna, COP
Garry Jaggers, COP
Ken Lewis, KVL Consultants
Ruth Franklin, Bay City Engineers
Scot Schlund, SCI
Pat Ellison, SCI
Chuck Gopperton, SCI
Scott Ogden, SCI
Mike Gerlach, SCI
Carlos Carriaga, SCI

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Stantech
Consulting

Memo

To:	Ralph Goodall, City of Phoenix Gary Benton, City of Phoenix Ray Acuna, City of Phoenix Ed Raleigh, FCDMC Amir Motamedi, FCDMC Ken Lewis, KVL, Inc. Scot Schlund, Stantech
From:	George Sabol
Date:	26 August 1998
Reference:	**City of Phoenix/Maricopa County Stormwater Drainage Design Manual FILE: 28900042**

The following is prepared in preparation for our meeting on 2 September 1998 at 2:00 at the Stantech office.

Contents of Joint Manual

1. It has been generally agreed that the technical content of the joint County/City manual should be identical. Each agency would have a separate, stand-alone volume for drainage policy. It may be desired to also have standards contained in that separate volume. This could occur, for example, if each agency has different standards for storm sewer design frequency, culvert/roadway overtopping criteria, etc. Alternatively, if the standards are the same for both the City and County, they could be contained in the technical manual. This may need to be determined as work on the manual progresses.

2. Stantech is working with the City in regard to the review and drafting of drainage ordinances. That effort will continue independently of the District.

3. The City manual is to address regulations also. That section is to define federal, state and county regulations over and above the City drainage ordinances. Those regulations are the same for both the City and the County; therefore, they could be in the "technical" manual, which is common to both agencies, or they could be included in the "policy" manual and only provided in the City version. Regulations seem to fit better with policy than technical procedures. Regardless of whether the District wants that section in the manual, it is still intended to be produced for the City. The regulations section could also refer to Volume III of the County manual, and any related construction and permitting processes.

4. A section on drainage planning is desired by the City, but is not presently provided in the County manual. That topic probably should be in the technical manual. As with regulations, the District will need to decide if such a section is desired.

5. **Hydrology** is presently provided in the County manual, Volume I. The following are the specific recommendations in regard to that topic:

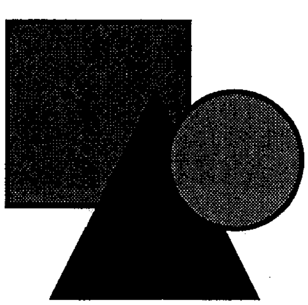
   a. The District wants to maintain the presently defined 6-hour design storm. That procedure is arguably the most scientifically defensible rainfall design storm available and it probably has the highest level of "accuracy." However, the procedure is troublesome for "new" users and its complexity, especially when used with the JD record option of HEC-1, can lead to undetected errors even by experienced users and reviewers. Therefore, if that procedure is to be adopted by the City, certain "improvements" to implementation and review procedures should be undertaken. This is an area that the District may be able to contribute service.

   b. It is generally acknowledged that the County's hydrologic procedure probably consistently overestimates flood magnitudes for frequencies of 2- to 10-years. At the time of manual development, this was recognized, but the need for appropriate estimation of those flood magnitudes was not fully appreciated by the authors. This is a major deterrent to the City's acceptance of that method. It is recommended that an addendum to the present County method be prepared that provides appropriate estimation of 2- to 10-year design peak discharges. That procedure should be prepared by Stantech with review and approval by both the City and the District.

   c. The County manual has little guidance in regard to modeling technique, general guidelines, modeler's/reviewer's checklist, and detailed instructions. The ADOT manual contains that type of information. The City desires more explicit guidance to be added to the County's procedures. It is recommended that such information be added to the County manual, and that this would be done by Stantech.

   d. The County manual does not provide indirect methods for discharge verification, but the ADOT manual does contain some useful information in that regard. It is recommended that Stantech review applicable methods and data from the ADOT (state-wide) manual and include that information that is applicable to Maricopa County.

6. **Hydraulics** is presently presented in the County manual, Volume II. That manual can be adopted with mutually agreed upon additions, overall review and updating. The following are some specific comments in regard to the hydraulics chapters:



   a. Street Drainage is covered in Chapter 3. The City would review that chapter. Modifications, if any, would be provided by Stantech with review and approval by both the City and District.

b. Storm Drains are covered in Chapter 4. That chapter would be reviewed and, if necessary, modified with joint approval.

c. Culverts are covered in Chapter 5. Again, the review and modifications would be as previously described.

d. Bridges are covered, but only very broadly and generally, in Chapter 5. Is this adequate for the City's purposes? Is there a need to expand this section?

e. Open-Channels are covered in depth in Chapter 6. That chapter would be reviewed and modified, if necessary, as previously described.

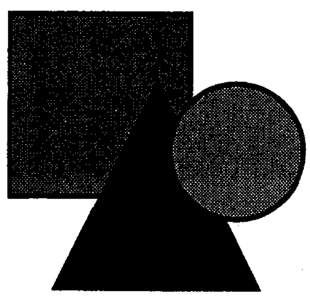
f. Hydraulic Structures are covered in Chapter 7. That section is limited to channel drop structures, conduit outlet structures, and some special channel topics. Additional topics and needs may be identified. That chapter would be reviewed, modified, and possibly expanded, if needed.

g. Detention/Retention Basins are covered in Chapter 8. Some of that material is of a regulatory nature and could be presented elsewhere. Again, review and modifications would be as previously described.

h. Pump Stations are briefly discussed in Chapter 9. Stantech has drafted a more comprehensive section on that topic for the City. Both Chapter 9 and the Stantech draft could be reviewed by both the City and the District to reach agreement on the desired level of presentation.

i. Sedimentation and related topics are discussed in various chapters throughout the County manual. The City wants some level of coverage of this topic. There may be the desire to consolidate information that is presently provided and to enhance that topic in a separate chapter. There are other topics related to non-structural flood management and drainage planning that may need to be added. The District should indicate its desires in this regard.

j. Floodplains are only briefly discussed within the context of open-channels. The City wants a more comprehensive treatment of this topic – particularly in regard to land development, drainage improvements, and the overall regulatory process. The District should indicate its desire in this regard.



7. Stantech's contract with the City also provides for an electronic manual along with computerized computation procedures for certain hydraulics procedures. That service is being provided by Ken Lewis. The District has indicated its interest in that topic. That service could be expanded to include hydrology including the rainfall

isopluvial maps, soil maps (with Green & Ampt parameters), land use maps, etc. Much of that information may be available in the District's HIS database. It is suggested that this scope expansion be discussed. It may be reasonable to have the District provide the databases along with coded calculation procedures and Ken Lewis could integrate the databases and procedures (MCUHP) into a comprehensive HEC-1 loader program. The District may want to consider this suggestion.

Recommendations

The following are specific recommendations for proceeding with a joint County/City manual.

1. The City and District need to agree upon the contents of the hydraulics portion of the manual, and the extent of modifications to the hydrology portion of the manual.

2. Upon completion of item 1 above, Stantech would prepare a scope of work, schedule and fee estimate. That would be submitted to both the City and District for approval.

3. The City and District would each be individually responsible for its "policy" volume with whatever unique sections, such as ordinances, that it wants.

4. The City and District would enter into a joint agreement.

5. The District needs to investigate funding or cost sharing approaches with the City. Depending on the scope of work, this could include "service-in-kind" for certain tasks.

6. Stantech would submit a contract change order to the City of Phoenix.

7. It is anticipated that the agreed upon scope of work will not increase the presently approved fee to the City. Any additional cost, which is contingent upon the scope of work, is anticipated to be a "modest" amount, and presumably could be borne by the District.

Schedule

The following is a recommended time schedule:

1.	Mutual agreement upon hydrology and hydraulics content of manual.	30 Sept 98
2.	Stantech to submit scope of work, schedule, and fee estimate to City and District.	12 Oct 98
3.	City and District to enter into joint agreement.	30 Oct 98
4.	Stantech to submit change order request to City.	30 Oct 98
5.	Executed change order to Stantech.	13 Nov 98
6.	Completion of manual and contract.	13 Nov 99

George V. Sabol, PhD, PE
Senior Associate
Water Resources Division

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CITY OF PHOENIX/MARICOPA COUNTY
STORM DRAINAGE DESIGN MANUAL

Meeting Minutes

8 April 1999 at Stantec Consulting Inc.

Attendance:	***Affiliation:***
Ed Raleigh	FCDMC
Amir Motamedi	FCDMC
Gary Benton	City of Phoenix
Ralph Goodall	City of Phoenix
Ken Lewis	KVL Consultants, Inc.
Dave Burris	Stantec
George Sabol	Stantec

1. The meeting minutes for 4 March were accepted without comment.

2. The Action Items from the last meeting were reviewed. Unresolved Action Items are noted below.

3. The final draft of the Storm Drain Chapter was reviewed. Pat Ellison could not attend the meeting, but he prepared a memo (8 April) indicating changes to the chapter and the status regarding unresolved issues. Sabol reviewed the memo and each item was discussed. The following are relevant comments:

   a. Concerning the COP material that is contained in the chapter appendix:

      - FCDMC should review that material and advise Stantec as to its opinion for inclusion in the Chapter.

      - Its inclusion directly as an appendix is not desired.

      - Some of that material should be considered for the Policy section.

      - Some of that material is not legible.

   b. Regarding various minor loss equations:

      - Certain equations, for example 16.7 and 16.8, are incorrectly shown. They should be absolute value of velocity head differentials.

      - Richard Harris and Pat had coordinated certain subjects. It is unknown if all concerns have been resolved. Ed provided a review copy from Richard that Pat needs to address.

c. Regarding standards (Table 16-5):

- The City and County need to resolve differences and adopt uniform standards, if possible. That should be accomplished in the next 2 weeks and resolution conveyed to Pat.

d. Pages 16-26 and 16-27 are in error and need corrected:

- Design is based on RCP.
- Pipe size needs to be upsized 6 inches if either CMP or CIP pipe are used
- Concrete lined CMP is allowed for mainline pipes.
- Unlined CMP is OK for connector pipes.
- Table in appendix is illegible (alternate materials)
- Material at bottom of Pages 16-26 appears to be based on old City of Phoenix information. AP13 is not applicable. For mainline CMP, only Type F is allowed, etc.

e. The City and FCDMC are to perform a careful review and respond to Pat with comments within 2 weeks. Stantec is to have a corrected final draft by the next meeting.

4. Dave Burris presented the 1st draft of the Open Channel Chapter. That material was reordered and modified. A reconfigured chapter table of contents was provided.

    a. Dave noted that Toe Protection and Grade Control sections are shown as deleted, but that material is to remain in the chapter.

    b. Ed tabled the USGS report on Manning's coefficients for use with natural channels in Maricopa County. The City wants that included in the manual. It can be readily scanned for inclusion on the CD, but would add considerable cost for hardcopy. It needs to be referenced in the Manning's coefficient section.

5. The hydraulics Structures Chapter was discussed relative to scope and content.

    a. The following additional topics are recommended:

    - Flow-Splitter structures (several types)
    - Channel junctions
    - Stair-stepped drop structures (Wood-Patel and maybe ADOT have recent experience.)

- An additional structure for transitions from conduits to channels.

b. Stantec will collect references on these topics to be presented at the next meeting.

c. The chapter, in general, is in good shape and is not expected to require revision other than additions.

6. Ken Lewis presented the electronic manual with a status report and discussion.

    a. The scope and content of each chapter to undergo computerization (computation methods) will be decided as each chapter nears completion.

    b. A work session to agree on computerization for the Street Drainage and Storm Drain Chapters is set for 21 April at 11:30 at Stantec. Lunch will be provided.

    c. Ken will work on a simple standard step procedure to be used for simple applications where HEC-2/HEC-RAS is not needed and where the user can apply the procedure without expertise in those more detailed methods. This is intended for use by consultants with limited computational software and/or expertise.

7. Mike Cusimano will be attending for Ray Acuna (Development Services). He is to receive all submittals. Keep Ray informed of meetings and correspondence. Cindy White is replacing Ray as Floodplain Manager. She should receive all submittals and correspondence.

8. The next meeting was not scheduled.

9. Action Items – The following is a summary of action items:

STANTEC

- Pat Ellison to coordinate with FCDMC and City so as to issue a revised final draft of Storm Drains by 29 April and distribute that final draft.
- Prepare a 2nd draft of Open Channels, and distribute by 29 April.
- Collect reference material for new Hydraulic Structures topics.
- Participate in work session for computation methods on 21 April, 11:30 am at Stantec.

CITY OF PHOENIX

- Coordinate with FCDMC regarding uniform standards for storm drains (Table 16-5).
- Review the Storm Drain chapter and coordinate, as necessary, with Pat Ellison by 23 April.
- Review the 1st draft Open Channel Chapter and be prepared for discussion at the next meeting.

FCDMC

- Participate in work session for computation methods on 21 April, 11:30 am at Stantec.
- Review Storm Drain appendix material and respond to Pat Ellison regarding comments.
- Coordinate with City regarding uniform standards for storm drains (Table 16-5).
- Review the Storm Drain chapter and coordinate, as necessary, with Pat Ellison by 23 April.
- Review the 1st draft Open Channel Chapter and be prepared for discussion at the next meeting.

KVL

- Participate in work session for computation methods on 21 April, 11:30 am at Stantec.

Stantec Consulting Inc.

CITY OF PHOENIX/MARICOPA COUNTY STORM DRAINAGE DESIGN MANUAL

Meeting Minutes

13 May 1999 at Stantec Consulting, Inc.

Attendance:	*Affiliation:*
Ed Raleigh	FCDMC
Bing Zhao	FCDMC
Gary Benton	City of Phoenix
Cindy White	City of Phoenix
Mike Cusimano	City of Phoenix
Dave Burris	Stantec
Pat Ellison	Stantec
George Sabol	Stantec

1. The meeting minutes for 8 April were accepted without comment.

2. The Action Items from the last meeting were reviewed. Unresolved Action Items are noted in item #9.

3. **Storm Drain Chapter**
   Pat Ellison provided an overview of the revised final draft of the chapter. Items to be resolved are:
   a. City and FCDMC to coordinate and reach agreement on storm drain criteria (Table 16-5).

   b. Pat to coordinate with Richard Harris on inclusion of pipe friction in connector pipe analysis.

   c. A workgroup will meet to finalize material provided by the City (previous appendix information). Much of this is of policy nature. The workgroup consists of Pat Ellison, Scot Schlund, Gary Benton and Mike Cusimano. Ed Raleigh will be advised of the meeting and FCDMC can participate as desired.

   Pat sent information to KVL concerning the spreadsheet calculation form and related AutoCAD conversion that will be part of the plan set.

   Gary will send a set of special catch basin mylars to Pat.

4. **Open Channels**
   Dave Burris provided a description of how we arrived at the present draft of the chapter. There was considerable discussion of what is desired in the chapter and how to get there.

The integration of various open channel topics into the Open Channel chapter and the Floodplain chapter was discussed. It was agreed that the Open Channel chapter must be applicable to both "engineered" channels and "natural" channels, and this will be undertaken. The Open Channel chapter will contain three general sections:

1. Open channel hydraulics fundamentals which are applicable to both engineered and natural channels
2. Hydraulic analysis considerations which are generally applicable to both engineered and natural channels, but some will be specific to engineered channels.
3. Design guidelines for various types of engineered channels.

The next draft for the chapter will be prepared to present the proposed chapter. Design guidelines will be generally as shown in the present county manual. In addition, all material that is deleted or rewritten from the county manual will be presented in a reference package. Much of that material will be recaptured in the Floodplain chapter.

The Floodplain chapter will focus on FEMA requirements and ADWR State Standards. It will cross reference the open channel chapter for technical principles and methodology.

The Floodplain chapter will commence production so that both the Open Channel and Floodplain chapters can undergo parallel development. This is to facilitate integration of the two chapters and avoid loss of pertinent information in the process of preparing two chapters.

5. Hydraulic Structures

The recommended reference material for topics to be added was distributed. This is to be reviewed and discussed at the next meeting. The reference material for stair-stepped drop structures was not received from the USBR. That will be provided when received.

6. Interviews with City Maintenance Personnel

Dave Burris met with City Streets and Parks maintenance staff. Dave provided presentation of major maintenance issues. These are:

- 404 permits
- Access into channels
- Trashracks and access barriers
- Fencing and related safety issues
- Use of riprap lined channels

Scot Schlund and Dave will prepare a memo on these topics and will breakout recommended policy and technical topics that should be addressed in Volume 2.

7. Manual Style

George noted that there is no agreement as to manual style, but manual production is advancing to where it is desired to set a style for typesetting. The pros and cons of paragraph numbering as opposed to a more open format were discussed. George will redistribute the previously recommended manual style memo. The Open Channel chapter will be prepared in that fashion to illustrate the recommended format.

8. **Next Meeting – 23 June at 10:00.**
(NOTE: I suggest that the meeting start at 1:00 since the meetings are generally taking 3 hours or longer . I will distribute an e-mail noting this change).

9. **Action Items:**

Stantec
- Pat to coordinate with Richard Harris concerning questions of pipe friction in connector pipe analysis.
- Workgroup meeting to finalize Storm Drain chapter.
- Prepare the Open Channel chapter draft.
- Begin scoping the Floodplain chapter.
- Obtain and distribute reference material for stair-stepped drops.
- Prepare a memo on maintenance interviews with recommendations for policy and technical considerations.
- Resubmit manual style recommendation.

City
- Coordinate with FCDMC to reach agreement on storm drain criteria.
- Workgroup meeting to finalize Storm Drain chapter.
- Review Open Channel draft (to be provided).
- Review Hydraulic Structures reference material.
- Gary to send a set of special catch basin mylars to Pat.

FCDMC
- Coordinate with City to reach agreement on storm drain criteria.
- Workgroup meeting to finalize Storm Drain chapter.
- Review Open Channel draft (to be provided).
- Review Hydraulic Structures reference material.

Stantec Consulting Inc.

CITY OF PHOENIX/MARICOPA COUNTY DRAINAGE DESIGN MANUAL

Meeting Minutes

13 September 1999 at Stantec Consulting, Inc.

Attendance:	*Affiliation:*
Ed Raleigh	FCDMC
Amir Motamedi	FCDMC
Tim Murphy	FCDMC
Ralph Goodall	City of Phoenix
Cindy White	City of Phoenix
Scot Schlund	Stantec
Pat Deschamps	Stantec
Pat Ellison	Stantec
Ruth Franklin	Stantec
George Sabol	Stantec

1. The meeting minutes for 13 May were accepted without comment.

2. The Action items from the last meeting were reviewed. There were no unresolved Action Items.

3. Storm Drain Chapter

   a. The calculation spreadsheet to be provided by KVL needs to be included and instructions for its use prepared.

   b. Ralph Goodall noted that the "Storm Drain Appendix" needs updating. That material is mainly policy and will need to be addressed by the policy "committee" for inclusion in that section of Volume Zero.

   c. Ralph requested that standard n values be specified for design rather than a range for n.

   d. Ralph noted the need for editorial rewrite and reordering of the section.

   e. The City and District provided their red-line review copy (to be copied and returned to each).

4. Open Channel Chapter

   Pat Deschamps provided an overview of the submittal. The following are noted:

   a. The basic content and organization of the chapter is accepted.

   b. Pat will proceed to prepare a complete first draft.

5. Floodplains and Regulations Chapter

Scot Schlund presented a scoping outline of the Floodplain Chapter, and Ruth Franklin provided a similar presentation regarding regulations. It was originally conceived that Floodplains would be a chapter in Volume 2 and Regulations would be a chapter in Volume Zero. It was decided that those topics would be combined and contained in Volume Zero. Stantec will prepare a detailed chapter outline for discussion at the next meeting. That chapter will be specific to the City of Phoenix. A courtesy review by the District will be requested.

6. Hydraulic Structures Chapter

George Sabol reviewed that status of scoping for that chapter. Basically, the existing County chapter will be preserved with the addition of five topics. Background material on those topics was previously submitted.

7. Hydrology (Volume I)

a. Stantec and the District met in June to discuss methodology for the 2-year to 10-year addendum. The District has rainfall-runoff data for "small events" that is available to assist with that task.

b. The NOAA precipitation study is "unresolved." NOAA plans to release the revised maps this year, but not the corresponding supplemental rainfall criteria. Use of that new criteria is uncertain. There is concern in the hydrologic community over its acceptance. Impact of using "old" criteria once "new" criteria are available presents potential liability issues. We have not been successful in getting NOAA to respond to a request (by users in Arizona) to meet to discuss concerns, etc.

c. A report on the electronic manual for hydrology will be scheduled for October.

8. Other Business

a. Manual Style

- Use limited numbering system along with subheading designation.
- Color will be used in the electronic manual but the hardcopy will be black and white. Color copies can be printed from the CD by users.
- Copies will be back-to-back.
- Blank pages will be so noted, as "Blank."
- Stantec will develop a recommended style and use it in future chapter drafts.

b. The interview memos by Dave Burris were compiled by Scot and distributed. Some of that information may need incorporation into policy. Authors of chapters will be provided the interview comments and requested to incorporate technical items into the chapter.

c. Management Presentation – A presentation for use by City and District staff will be discussed at the October electronic manual meeting.

d. Project Schedule – The project completion data was extended to June 2000. The City has received requests as to status and availability of the manual. Stantec is committed to meeting the June schedule. It is imperative that departmental users at the City and District participate in meetings and reviews to achieve a successful product.

9. Next Meetings

a. Electronic Manual – 7 October @ 1:30

b. Review Meeting – 18 October @ 1:30

Note: The 7 October meeting is not acceptable to some and a mutually agreeable time will be determined.

10. Action Items

Stantec

a. Review Storm Drain comments. Prepare documentation necessary regarding revisions/completion requirements.

b. Open Channel Chapter – Prepare first draft.

c. Regulations (Floodplains) Chapter – Prepare detailed scoping outline.

d. Hydraulic Structures Chapter – Begin chapter expansion for new topics.

e. Develop style per agreement.

f. Arrange meeting on electronic manual.

g. Distribute submittals (Open Channel & Regulations Chapters) by 4 October.

KVL

a. Prepare for presentation on electronic manual at a "special" meeting in October (time to be arranged).

b. Coordinate with Stantec and City in regard to a Management Presentation.

City

a. Update desired policy in the Storm Drain Chapter.

b. Review submittals prior to 18 October meeting.

District

a. Review submittals prior to 18 October meeting.

Stantec Consulting Inc.

CITY OF PHOENIX/MARICOPA COUNTY DRAINAGE DESIGN MANUAL

Meeting Minutes

18 October 1999 at Stantec Consulting, Inc.

Attendance:	***Affiliation:***
Ed Raleigh	FCDMC
Amir Motamedi	FCDMC
Tim Murphy	FCDMC
Ralph Goodall	City of Phoenix
Cindy White	City of Phoenix
Gary Benton	City of Phoenix
Hasan Mushtaq	City of Phoenix
Scot Schlund	Stantec
Pat Deschamps	Stantec
Pat Ellison	Stantec
Robin Wade	Stantec

1. The meeting minutes for 13 September meeting were accepted without comment.
2. Action items from 13 September were reviewed. Outstanding action items included "special" meeting for the electronic manual and coordination of the Management Presentation. All other action items completed.
3. Open Channel Chapter

   Pat Deschamps went over the "draft" of the Open Channel Chapter that included:

   a. Enhanced discussion of Momentum and Specific Force applications.

   b. Enhanced discussion on resistance to flow and Manning's "n" values. Basically additional information is provided.

   c. Enhanced discussion of Control Sections that included:

   - How control sections can move depending on depth or flow regime.
   - Classic water surface profiles will be provided in the chapter such as M1, M2, S1, S2, etc.
   - Amir asked if Stantec required comments. Pat D., responded in the affirmative.
   - Amir asked how we will keep track of new changes? Pat D., stated that we will use the chapter outline and annotate it.
   - Amir delivered Kofi Awuma's comments on the chapter.

4. Storm Drain Chapter

Pat Ellison reported on corrections to the Storm Drain Chapter.

a. Pat E. report that he and George Sabol had discussed comments and had divided these into three areas: technical, editorial, and policy. Only technical will be discussed at this time. Editorial will be addressed by George at a later time. Policy will be moved to Volume "0".

b. Pat E. developed three new figures for various storm drain junction types that supplement Figure 15.6 of the old manual. A handout was provided. Ralph stated that this will help clarify that section.

c. Pat E. reported that format and equation numbers will be added.

d. The Manning's "n" value table is revised and a handout was provided. Ralph requested that concrete lined CMP (used as mainlines in City) and HDPE (used as mainline and laterals in City) pipe be added to the table. As a note, unlined CMP is used for laterals only in City. HDPE is used only as connector in County

5. Manual Style

Robin Wade prepared a handout sample of the proposed Manual Style and provided a brief explanation. Input regarding the style is as follows:

a. Highlights column on right margin is well liked by all.

b. Ralph suggested that we put numbering in highlight text along with chapter name. He also suggested that we put highlights closer to right edge of the page and move left margin to the right to provide more space for 3-hole punch.

c. Ralph likes the spacing and font as presented.

6. Electronic Manual Meeting

A teleconference was conducted with Ken Lewis and a meeting for the Electronic Manual was scheduled for 26 October 1:30 p.m. at Stantec.

7. Management Presentation

During the teleconference with Ken Lewis, discussion of a Management Presentation ensued. The following general topics for the presentation were agreed to:

a. It will be a joint meeting and presentation to City of Phoenix and Flood Control District upper level management.

b. The presentation will focus on benefits of the manuals to the City, District and community.

c. It will be a maximum of 20 to 30 minutes.

d. It will stress the cost savings of a "joint" manual approach.

e. It will present where can we go and what can we do in the future.

f. The City and District staff will participate in the presentation along with Stantec and KVL. No more than four speakers.

g. Web deployment needs to be completed before the presentation. Most likely date for presentation will be late January to early February 2000.

h. Ken would like to merge the two software packages together. He will present this further at the special meeting on the electronic manual.

8. Regulations/Floodplain Chapter

Scot S. provided a handout and discussed the "fleshed out" concept of the Regulations Chapter. Ralph stated that this was what he had in mind and liked the concept. Stantec can proceed with this approach. Some general comments regarding the content are as follows:

a. Permit/application form references using dates are to be deleted.

b. Ralph would like all Nationwide 404 permit numbers listed along with form numbers as applicable

c. Ralph stated that we can contact Angela Brooks at City of Phoenix for status of 404 training.

d. Ralph stated that we need to clarify that this chapter is for design consultants and development not intra-City work. The ongoing Logan Simpson Design training program is handling intra-city 404 Permit issues.

9. Next meeting: 18 November 1999 at 1:30 p.m. at Stantec.

10. Action Items

Stantec

a. Continue work to final draft Storm Drain Chapter.
b. Continue work to final draft of Open Channel Chapter.
c. Provide Cindy White and Hasan Mushtaq completed draft chapters.
d. Continue developing draft of Regulations Chapter.

KVL

a. Conduct presentation for Electronic Manual on 26 October at 1:30 p.m.

City

a. Comments on Open Channel Chapter
b. Review submittals prior to 18 November meeting.

District

a. Comments on Open Channel Chapter
b. Review submittals prior to 18 November meeting.

Stantec Consulting Inc.

CITY OF PHOENIX/MARICOPA COUNTY DRAINAGE DESIGN MANUAL

Meeting Minutes

19 January 2000 at Stantec Consulting, Inc.

Attendance:	***Affiliation:***
Ed Raleigh	FCDMC
Tim Murphy	FCDMC
Amir Motamedi	FCDMC
Kofi Awumah	FCDMC
Ralph Goodall	City of Phoenix
Gary Benton	City of Phoenix
Ken Lewis	KVL
Scot Schlund	Stantec
Sandy Steichen	Stantec
Pat Ellison	Stantec
George Sabol	Stantec

1. The meeting minutes for 18 November meeting were accepted without comment.

2. Action items from 18 November were reviewed. All Action Items are satisfactory.

3. Regulations Chapter

   The draft of the Regulations Chapter was discussed by Scot. It is noted that this chapter if for Volume Zero of the City manual. The County may adopt portions of that chapter for its use.

   Scot will edit the chapter in regard to:

   a. Deleting and/or condensing certain Nationwide Permit discussions.

   b. Reorganizing by function, with each function subtitled Federal, State, County and Local.

4. Detention/Retention Basins

   Sandy provided an overview of the changes made to the County manual for that chapter. Several items were discussed with resolutions, as noted:

   a. The multi-use discussion that was added to that chapter will remain there. A similar multi-use discussion is needed for the Open Channel Chapter. Multi-use will also be addressed in the Planning chapter.

   b. The Simplified Method (from the State Standard) had been added, but can be deleted since that method is the procedure that is currently provided in the County chapter.

   c. The chapter will be revised to indicate that routing methods are also acceptable to determine volume, particularly for larger, regional basins.

d. The sediment discussion will be deleted and covered in the new Sediment Chapter.

e. Table 8.2 will be deleted and treated more comprehensively in the Hydraulic Structures Chapter.

f. Section 8.8 will be deleted in its entirety.

g. The example will be deleted and no example is needed for this chapter.

h. The chapter will be titled "Stormwater Storage."

i. George recommended deletion of certain sections that briefly (but inadequately address geotechnical considerations, in his opinion) from the manual. The existing section (8.3.3.3) will be revised to indicate the requirement for appropriate geotechnical considerations and reporting. Section 8.3.3.6 was recommended for deletion. Ed will investigate the "source" of that information. Final decision on the disposition of that topic will be made at the next meeting.

j. KVL does not need to include storage routing in its computation package since the use of HEC-1 is expected.

5. Open Channels

Sandy will be producing a complete first draft for submittal by the next meeting. She received comments from Kofi and George. Others with comments should convey those to Sandy.

6. Storm Drains

That chapter was not discussed due to lack of time. Pat received comments from Ed. KVL has completed its computational spreadsheet and that can be appropriately treated in the chapter.

7. Electronic Manual

Ken distributed an example spreadsheet analysis that is produced via his software. Testing is required. Ken will provide a copy of the software for initial testing by Stantec and District staff.

Subsequently, he will install the software to City computers for its testing.

8. Other Business

a. Volume Zero will contain Planning, Regulations, Standards and Policies.

b. Upon acceptance of the second draft of each chapter, it will be provided to KVL for loading to the electronic manual.

c. The Regulations Chapter was submitted according to the recommended style and there are not suggestions to modify that style.

The following chapters will be converted to that style and delivered to KVL:

Pump Stations
Street Drainage
Storm Drains

d. Stantec has increased its staffing of this project and expects to complete the manual this summer. The next 6 months are expected to be rather busy in this regard to both Stantec and the City and District reviewers.

9. Next Meeting

The next meeting is scheduled for 8 March 2000, 1:30 pm at Stantec

10. Action Items

Stantec

a. Second draft of Regulations Chapter

b. Second draft of Stormwater Storage Chapter

c. First draft of complete Open Channel Chapter

d. Conversion of previously completed chapters to new style

KVL

a. Installation of software on Stantec and District computers. Followed by testing of the software.

District

a. Testing of software.

b. Investigation of source of information on geotechnical considerations in the Stormwater Storage Chapter.

Meeting Notes

Stantec

FILE: 82000042

Date:	8 March 2000	
Place/Time:	Stantec/1:30 pm	
Next Meeting:	12 April 2000 @ Stantec, 1:00 pm	
Attendees:	City of Phoenix	Gary Benton, Ralph Goodall, Jesse Gonzales
	FCDMC	Kofi Awumah, Amir Motamedi, Chris Perry, Ed Raleigh, Tim Murphy
	Stantec	George Sabol, Sandy Steichen, Frank Thomas
	KVL Consultants	Ken Lewis
Distribution:	Attendees/Absentees	

Item:

No comments were received on the 19 January 2000 meeting minutes and they were so approved.

A. Review Previous Action Items

Stantec

1. The second draft of Regulations Chapter is on hold pending release of final 404 nationwide permit rules.
2. The second draft of Stormwater Storage Chapter was distributed for review in early February.
3. The first draft of the complete Open Channel Chapter was provided to the group March 1st.
4. Conversion of previously completed chapters to new format has been on-going. The Pump Station Chapter was distributed in this final format.
5. The street drainage and storm drain chapters are close to completion pending software testing.

KVL

1. Installation of software on Stantec & District computers has been undertaken. This will be followed by software testing.

Reference: Drainage Design Manual

District

1. Testing of software is on-going.
2. The investigation into the source of information on geotechnical considerations in Stormwater Storage Chapter concluded that it stemmed from the original McLaughlin version.

B. Discussion on Open Channel Chapter first draft comments:

1. Chapter status was overviewed, identifying the changes since the chapter was last visited last October.
2. The viability of shotcrete as a channel lining was discussed given the propensity for failure. The concern of using shotcrete in a supercritical channel was also raised. It was concluded that if properly specified and construction, it would be as durable as concrete. However, it was identified that the chapter as written did not have tight enough specifications. The ADOT specification was identified as being appropriate, however, past experience seems to suggest that engineers use the MAG standard which is inadequate. The C.O.P. will look into having the standard changed at MAG. The issue of construction inspection, which is outside the venue of the drainage manual, was also identified as a presumed problem with privately constructed drainage channels. The FCDMC is to coordinate with its maintenance department to ascertain if shotcrete has greater maintenance issues than concrete as a channel lining. If not, the chapter will be amended to strengthen the specifications for shotcrete lined channels.
3. A discussion ensued regarding minimum design velocity in that the minimum needed to be identified and tied to a specific return frequency such as 2ft/s at 25% of Q100. The issue here is one of sedimentation. It was concluded that no minimum would be set due to potential conflicts between the minimum and maximum design velocities and that the sedimentation issue would be addressed in that chapter. A discussion in the minimum velocity section would identify the issue as a sedimentation problem and make reference to that particular chapter. It was also concluded that for concrete (or shotcrete if applicable) lined channels, minimum slope would be set to 0.0015 ft/ft. This would be identified as a subset to the chapter.
4. There was a discussion about safety issues associated with concrete channels and channels in general. Ladders in supercritical channels pose there own problems in terms of impacts to flow regime. This issue was tabled to later as it will be addressed in its own chapter or at the introduction of the Hydraulics Manual.

Stantec

C. Discussion on Stormwater Storage 2nd Draft Comments

1. Chapter status was overviewed.

2. A preliminary discussion on the potential conflict between the use of a 6 hour storm for channel design (off-site) and the 2 hour storm for on-site retention was deferred to another meeting.

3. A concern with the introductory text that promoted regional facilities yet the majority of new development over the last 10+ years has focused on localized retention. Given the nuances of design for detention facilities that are dependent upon hydrograph shape, a regional approach is apparent. However, for land development, it is more cost effective to locate retention basins where street capacity starts to exceed standards for gutter flow. Since retention is the preferred form of stormwater storage in practice here in the Valley, it appears that regional facilities appropriate for watershed basins that have existing flood problems or were primarily developed before retention regulations or to accommodate run-off in excess of the 2 hour 100 year storm for watershed subject to recent development. It was agreed to modify the text accordingly.

4. It was agreed to add low flow outlets for retention basins as a mechanism to drain retention basins with the intent that outlet flow would be significantly less than existing peak discharge and it drains in 36 hours cumulatively. For maintenance purposes, the minimum size of pipe was set to 18" with orifice plates reduce flow as necessary.

5. The discussion on safety fences was to be moved with the other safety issues as discussed above.

6. A discussion on the basis for separation of off-site flows from on-site flows was held. It was concluded that separation is preferred as the rising limb of the off-site hydrograph tends to fill the on-site storage prior to the on-site peak discharge. However, the text would be modified to allow exceptions on a case by case basis since there were valid situations where combination of off-site and on-site is justified.

7. It was agreed that the geotechnical discussion would be modified to identify that geotechnical considerations were necessary for each embankment situation, although it would be up to the design engineer to properly apply geotechnical analysis relative to the significance of the project. References for geotechnical guidance would be given in the chapter. Providing geotechnical guidance was determined to be beyond the intent of the manual.

8. It was agreed to identify that sedimentation would need to be accommodated in the design of stormwater storage facilities with referral to the Sedimentation Chapter

Reference: Drainage Design Manual

D. Scoping For Hydraulic Structures

1. Reference was made to the *12 May 99 memo and attachments.* The following recommendations were made and approved for inclusion into the first draft:

2. Flow Splitters (Bifurcations)
   - Burris write-up with edits included in Special Channel Structures as sub-section 7.5.1.1

3. Channel Junctions
   - For subcritical flow, 12 degree max angle of departure, 3 times topwidth minimum centerline radius, same depth design flow tributary & main channel. For supercritical flow regime, momentum analysis would be required. The text would reference the Corps document.

4. Stair Stepped Drop Structures
   - Augment 7.3.4.5 (pg. 7-48) to identify minimal dissipation of energy at design flows.... channel drop structures, require 2:1 slope of drop, horizontal steps, 30" max per drop and require stilling basin /riprap length based on conventional smooth sloping drop analysis.

5. Transitions from conduits to channels
   - It was agreed to exclude this issue as it was determined to be not necessary.

6. Access Ramps
   - Burris write-up to be addended and placed in Special Channel Structures as sub-section 7.5.3

7. Side channel spillways
   - The FCDMC address this issue frequently. A subsection of the chapter will be established with a note indicating awaiting guidelines. The FCDMC will develop guidelines.

8. Trash Racks
   - It was agreed to move the trash rack discussion from the Culverts Chapter to the Hydraulic Structures Chapter and add a discussion on energy loss computations.

Stantec

E. Scoping For Bridges & Culverts

Discussion Items and Recommendations for Culverts & Bridges Chapters

1. Due to minimal design guidelines offered for bridges, culverts and bridges to be covered in one chapter. Detailed bridge hydraulics are to indicated as beyond the scope of the manual.

2. Access barriers are to be eliminated from this chapter and addressed in the safety section.

3. The storm sewer interaction with culverts portion of 5.3 Entrances and Outlets for Culverts and Storm Drains will be compared with discussions in the Storm Drain Chapter (and included if appropriate) and eliminated from the Culvert chapter.

4. Section 5.3.3, Estimating Erosion at Culvert Outlets, will be shown as a strike out in the first draft as it is questionable if professionals routinely perform this analysis for culvert design in the valley.

5. The riprap apron design procedures are to remain as the FCDMC requires consultants to design to these guidelines.

6. Culvert design with drop inlets (pages 5-15, Section 5.2.2.14, and 5-27) needs clarification…is this referring to drops located immediately upstream of culvert entrance or catch basin tying into the culvert. The group is to review these passages to determine their intent.

7. The storage routing discussion will be shown as strike-out as the procedures for routing are included in Volume 1. Hydrology. A discussion of stage – discharge is to remain.

F. Other Business

1. Standards are to be relocated to "Volume Zero". The new format will be used to reference the standards in the margin (i.e. See Std. 5-1). Global standards should have their own prefix.

2. Safety: Safety issues from each chapter are to be consolidated into the introduction of Volume

3. The revised schedule was overviewed.

4. The timing and planning for presentations to senior management at C.O.P. and FCDMC was discussed. A date certain will be set for the presentation during the next meeting in April.

5. The DDMS-W is ready for release now. It was agreed that there was no reason to hold off on its release.

6. Future agendas will try to separate hydrology issues from hydraulic issues to be more efficient with the group's time.

G. New Action Items

Stantec

Stantec

1. Finish first draft Planning for Volume 0

2. Finish Regulations second draft

3. Prepare first draft Standards for Volume 0

Reference: Drainage Design Manual

4. Start Hydrology Volume
5. Finish figures for Street Drainage, Storm Drains, Pump Station
6. Prepare second draft Open Channels
7. Prepare first draft Hydraulic Structures
8. Prepare First draft Culverts & Bridges
9. Finish Stormwater Storage
10. Prepare recommendations for sedimentation

City of Phoenix

1. Review Planning, Regulations, Standards
2. Initiate internal policy review
3. The C.O.P. will look into having the shotcrete standard changed at MAG.

FCDMC

1. The FCDMC is to coordinate with its maintenance department to ascertain if shotcrete has greater maintenance issues than concrete as a channel lining.
2. The FCDMC will develop guidelines for side channel spillways.

Interagency

1. Review Open Channels, Hydraulic Structures, and Culverts & Bridges

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately.

STANTEC CONSULTING INC.

Frank Thomas, PE
Project Manager

Meeting Notes

Stantec

FILE: 82000042

Date:	16 May 2000
Place/Time:	Stantec/1:00 pm
Next Meeting:	21 June 2000 @ Stantec, 1:00 pm

Attendees:

City of Phoenix	Gary Benton, Ralph Goodall, Ray Acuna (late)*
FCDMC	Kofi Awumah, Tim Murphy, Ed Raleigh,
Stantec	Timberly Marek, Frank Thomas, George Sabol, Scot Schlund*
KVL Consultants	Ken Lewis*

* early departure

Distribution: Attendees/Absentees

Item:

A. Meeting Minutes Approval

No comments were received on the 12 April 2000 meeting minutes and they were so approved.

B. Review Previous Action Items

Stantec

1. A draft Powerpoint presentation has been prepared (as discussed below).
2. Stantec has investigated PageMaker vs. Word (as discussed below).
3. Final comments on the Street Drainage, Storm Drain, and Stormwater Storage Chapters have been completed, figures are still in progress, and format is to be finalized.
4. The Regulations Chapter of Volume 0 was distributed 5/15 with updated 404 regs.
5. Hydraulics Chapter- trash rack headloss write-up was completed.
6. An outline for the Sedimentation was developed and distributed 5/15.

KVL

1. The software testing for the storm sewer software is starting to hold Ken back. He recommended Chris Perry at FCDMC assist in the review of the software in coordination with Pat Ellison. Ed Raleigh approved Chris's involvement, with the

Reference: Drainage Design Manual

task identified as taking the first round of detailed review. The City and Stantec would undertake further review after the initial refinements were completed.

C. Presentation to C.O.P. senior management

The draft C.O.P. Powerpoint presentation was shown to the attendees. George Sabol provided an overview of the direction Stantec had taken for the first portion of the presentation. More work is needed to integrate the presentation slides from Stantec and the City to look more uniform. The manual maintenance slide requires a title change so it will not be confused with maintenance of stormwater facilities. A slide for stormwater facility maintenance needs to be added. A list of C.O.P. internal invitees has been completed and will be amended to include Chuck Williams at MCDOT and Mike Ellegood at FCDMC.

The schedule for the Presentation Sub-committee is as follows:

a. Meeting June 1, 10 AM at Stantec to further prepare presentation
b. Meeting June 8, 10 AM at Stantec to finish presentation if needed.
c. Final graphics to be done by 6/7
d. Dry run presentation during week of 6/12 (2 days prior to presentation).
e. Presentation week of 6/12 (Ralph Goodall is to identify date).

D. Discussion on Volume/Chapter Format

Volumes 1 & 2 are to retain the same titles as the existing documents. It will be up to each entity to title "Volume 0" as they see fit. The first page of Volumes 1&2 will have a list of agencies and municipalities that have formally adopted these manuals.

Frank Thomas provided commentary on the use of PageMaker instead of Word, which included a hand-out of relevant questions and answers prepared by Timberely Marek. The benefit of PageMaker is that it is inherently more stable than Word (i.e. less likely to crash), is less troublesome in terms of formatting for equations and insertion of figures and tables, and it can handle much larger files. The primary concern was the future availability of PageMaker and the availability of trained personnel to use the software. On the other hand, staying with Word would expose future revisions to formatting problems. Since PageMaker is provided by Adobe and presently available to the City of Phoenix staff for use, the City's position was to use PageMaker. The cost for converting from Word to PageMaker was estimated at $2000 (likely to be sub-contracted). The cost for software itself was identified in the range of $300-$400. It was agreed by C.O.P. and FCDMC to use PageMaker.

Stantec

Reference: Drainage Design Manual

E. Community acceptance of manuals

Ray Acuna indicated the need to start identifying the various controversial issues associated with the manual, particularly the changes to policies and standards. Elimination of on-lot retention was identified as an example of an issue that could hinder acceptance of the manual. Frank Thomas is to provide Mr. Goodall and Mr. Acuna with a list of items potentially controversial.

F. Culverts & Bridges

The culvert scour analysis section of the chapter is to be removed. Dr. Awumah provided a simplified approach for checking headwall toe wall depth. It was agreed to incorporate this analysis into the chapter pending receipt of reference information.

G. Open Channel

The freeboard section is to be modified to identify FEMA freeboard requirements for levees. Chemical interaction between soils and gabion basket material is to be investigated by Stantec and FCDMC, with the gabion write-up modified appropriately. Text is to be added recommending inspection of gabions after major flow events. There were no additional comments offered for the Open Channel Chapter. Mr. Goodall, shall finish his review and provide comments prior to the next meeting.

H. Hydraulic Structures

Stantec has completed refinements to the trash rack headloss analysis and incorporated them into the 2nd draft. Review of the first draft was deferred to the next meeting.

I. Sedimentation

George Sabol overviewed the outline for the Sedimentation Chapter. He explained the intent of the chapter as an introduction to the topic. The chapter will provide references for further investigation, but it will not be a tool to dictate or guide sediment transport analysis. The chapter is not going to be guidelines for sediment transport modeling. The chapter is to provide application guidance for toe down (for bank stabilization projects) and pier scour analysis. Pictures of representative problems here in Maricopa County will be incorporated into the chapter. All are to provide pictures for consideration for inclusion into the chapter.

Reference: Drainage Design Manual

I. Introduction to Volume 2

Mr. Thomas identified the changes to the introduction chapter relative to the first edition of the manual. Review of this chapter was deferred to the next meeting.

J. Other Business

No other business was identified.

K. New Action Items

Stantec

Incorporate culvert toe down analysis into Culverts and Bridges 2nd draft.

Incorporate responses/comments to Regulations first draft.

Pat Ellison is to overview storm drain testing with Chris Perry (FCDMC) after June 1.

Prepare PowerPoint presentation.

Prepare list of potentially controversial issues associated with the proposed policies and standards.

Move forward with conversion to PageMaker.

Investigate gabion material interaction with soils.

Finalize Open Channel Chapter pending receipt of comments from City & FCDMC.

Move forward with Hydrology tasks.

Incorporate figures into chapters.

City of Phoenix

Review Planning, Regulations, Policies, and Standards

Provide Stantec with changes to Policies and Standards prior to initiating internal policy review

The C.O.P. will look into having the shotcrete standard changed at MAG.

Prepare PowerPoint presentation

Stantec

Provide Stantec with its comments on Open Channel Chapter prior to next meeting.

Review first draft Hydraulic Structures Chapter

Review first draft of Volume 2 Introduction

Reference: Drainage Design Manual

FCDMC

The FCDMC will develop guidelines for side channel spillways.

The FCDMC is to assist in the first level of software testing.

The FCDMC is to review issues associated with gabion basket chemical interaction with soils and their ensuing longevity.

KVL

Prepare PowerPoint presentation.

Interagency

Review Volume II Introduction, Open Channels, Hydraulic Structures, and Culverts & Bridges

Provide representative pictures of sediment transport issues/problems for inclusion into the Sediment Transport Chapter.

Prepare PowerPoint presentation

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately.

STANTEC CONSULTING INC.

Frank Thomas, PE
Project Manager

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Meeting Notes

Stantec

FILE: 82000042

Date:	»21 June 2000
Place/Time:	Stantec/1:00 pm
Next Meeting:	26 July 2000 @ Stantec, 1:00 pm

Attendees:	City of Phoenix	Gary Benton, Ralph Goodall
	FCDMC	Kofi Awumah, Tim Murphy, Ed Raleigh,
	Stantec	Frank Thomas

Distribution: Attendees/Absentees

Item:

A. Meeting Minutes Approval

No comments were received on the 16 May 2000 meeting minutes and they were so approved.

B. Review Previous Action Items

Stantec

1. A Powerpoint presentation was made to City of Phoenix management 6/15.
2. Ken Lewis, Pat Ellison, and Chris Perry were meeting on 6/22 to go through software testing. Ken Lewis decided not to change the web page format.
3. List of controversial issues provided to Ralph & Ray.
4. The Regulations Chapter of Volume 0 was distributed 5/15 with updated 404 regulations. The first round comments were addressed with a revised edition distributed 5/22.
5. Culverts & Bridges Chapter- Culvert scour was evaluated as discussed further below.
6. First draft of Sedimentation Chapter is 80% done. The completed first draft will be distributed to the team by the end of July.
7. The first draft of the introduction to the City of Phoenix Policies and Standards was completed and submitted (not previously identified as an action item).
8. The foreword preambles for the Hydrology and Hydraulics Manuals were edited to incorporate an adoption page and accommodate third edition items (not previously identified as an action item).

Reference: Drainage Design Manual

9. Digital copies of the Hydraulic Manual chapters were provided to the City of Denver's representative, as the City is in the process of updating its drainage manual.

C. Culverts & Bridges

The issue of scour analysis was not resolved. Frank Thomas presented sensitivity analysis results that showed the Veronese equation yielding inconsistent and questionable estimates of scour depth. Ed Raliegh and Kofi Awumah desire an equation to estimate scour downstream of culverts. Stantec and Tim Murphy of the FCDMC will do further investigation and evaluation to try to find a suitable equation. Limits, such as minimum applicable velocity or minimum culvert size, may be required for the selected equation. This chapter is complete pending resolution of culvert scour.

D. Open Channel Chapter

Second draft comments were received from the City of Phoenix. Stantec will now incorporate these comments and finalize that chapter. City of Phoenix will pursue getting the MAG standard revised for shotcrete.

E. Hydraulic Structures Chapter

The trash rack hydraulic analysis methodology was modified to permit evaluation of ponded conditions expected at detention facilities. The analysis for culverts was reliant upon angle of approach and approach velocity which made the original methodology difficult to apply in ponded situations. The FCDMC is still evaluating side channel spillways. Stantec is to revise the second draft to put a place holder in the chapter for side channel spillways, incorporating references to be supplied by FCDMC. If the FCDMC does not develop or find a methodology to its satisfaction, then the issue will be handled by referring the designer to specific references.

The FCDMC requested additional work on the weir section. Specifically, definitions for sharp and broad crested weirs are to be incorporated. In addition, "C" values for roadways as broad crested weirs are to be identified. The State standards for stormwater basins are to be reviewed by Stantec for applicable discussions on outflow from basins.

The channel access portion of the chapter is to be edited to eliminate reference to 6:1 or flatter side slopes as not needing access ramps.

First round comments are pending from the City of Phoenix.

F. Introduction Chapter to Hydraulics Manual

Stantec corrected reference to drop height as 2.5' maximum instead of 3'. Kofi Awumah provided his first round comments on the chapter. Comments on are needed from the rest of the team. Review of this chapter was deferred to the next meeting.

G. Hydraulics Manual Chapter 2- Hydrology

A marked up hard copy of this chapter was provided to the attendees for review and comment during the meeting. Stantec will incorporate the comments received into the second draft for further review and comment at the next meeting.

H. Forewards/Preambles to Hydrology and Hydraulic Manuals

A marked up hard copy of the foreward portions of the Hydrology and Hydraulic manuals was provided to the attendees with extra copies provided to FCDMC for internal dispersal. Stantec recommended changes to make these pages consistent for both manuals. Specifically, an adoption page was added to both, the acknowledgement sections were streamlined, and the revision pages for the Hydrology Manual were proposed to be relegated to the introduction chapter. Reference to Volumes I and II are to be eliminated to avoid further confusion. Volume I is to be referred/labeled as Hydrology Manual. Volume II is to referred/labeled as Hydraulics Manual.

The issue of dating versions of the manual was discussed as it was not known whether a date should be on the front title page. Frank Thomas identified verbage in the foreword that indicated that the manual(s) were to be posted on the web and continually updated. The issue of updating and archiving the manual is to be discussed at the next meeting with specific input sought from Ken Lewis and Timberly Marek. Framemaker is to be investigated to see what options it may have for tracking changes.

Stantec was given permission to proceed with the edits in strike-out format for review by the team.

I. Sedimentation

Pictures of representative problems here in Maricopa County were provided by Tim Murphy. These are to be returned to FCDMC.

J. Other Business

Stantec is to provide a copy of the most recent versions of all chapters in the Hydraulics Manual to the team along with status of each chapter.

No other business was identified.

K. New Action Items

Stantec

Identify suitable equation with FCDMC and incorporate culvert toe down/scour analysis into Culverts and Bridges 2nd draft.

Stantec

Modify first draft of Chapter 2 Hydrology of the Hydraulics Manual.

Finalize Open Channel Chapter by incorporating City of Phoenix comments.

Incorporate responses/comments to Regulations and Planning first drafts pending receipt from City. Assist City of Phoenix in facilitating review of draft policies.

Reference: Drainage Design Manual

Modify side channel spillway section in Hydraulic Structures Chapter to include FCDMC supplied references. Stantec will modify the discussion on weirs and channel access ramps.

Provide team with first draft of Sedimentation Chapter by end of July.

Modify Preambles to Hydrology & Hydraulic Manuals

Move forward with conversion to PageMaker.

Incorporate figures into chapters.

Stantec is to provide a copy of the most recent versions of all chapters in the Hydraulics Manual to the team along with status of each chapter.

City of Phoenix

Review Planning and Regulations. Provide Stantec with changes to Policies and Standards prior to initiating internal policy review

Facilitate internal review of Policies, and Standards

The C.O.P. will look into having the shotcrete standard changed at MAG.

Provide Stantec with its comments on Hydraulic Structures Chapter prior to next meeting.

FCDMC

The FCDMC will develop guidelines for side channel spillways.

The FCDMC is to assist in the first level of software testing.

The FCDMC will assist Stantec in identifying an appropriate scour equation for use in estimating culvert toe wall/rip-rap requirements.

Interagency

Review & comment on Hydraulics Manual Introduction and Forewords/Preambles to Hydrology and Hydraulic Manuals

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately.

Stantec

STANTEC CONSULTING INC.

Frank Thomas, PE
Project Manager

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Meeting Notes

Stantec

FILE: 82000042

Date:	26 July 2000
Place/Time:	Stantec/1:00 PM
Next Meeting:	06 September 2000 @ Stantec, 1:00 PM

Attendees:	City of Phoenix	Ray Dovalina (part-time) , Ralph Goodall
	FCDMC	Kofi Awumah, Ed Raleigh,
	Stantec	Carlos Carriaga (part time), Frank Thomas

Distribution: Attendees/Absentees

Item:

A. Meeting Minutes Approval

No comments were received on the 21 June 2000 meeting minutes and they were so approved.

B. Review Previous Action Items

Due to the reduced agenda items, previous action items were addressed within the context of each agenda item.

C. Hydrology Manual Update and Overview of Hydraulic Manual

Frank Thomas gave a brief overview of the issues presently being addressed in the Hydrology Manual. The focus is presently on the ADOT (indirect method) regression equations and whether these needed updating for 10 more years of record and the addition of 80 more gages. There has been on-going discussion as to the extent of the data base to be included, i.e. limit the gage data to Maricopa County or the entire State. It was concluded to include gages outside the county as they provide useful information in hydrologically similar basins. George Sabol, Mike Gerlach, and Amir Motemedi have been coordinating on these issues. A more thorough overview of the Hydrology Manual status will be provided at the next meeting.

Frank Thomas gave a chapter by chapter status for the Hydraulics Manual as identified previously in an email to the team. An updated manual with the latest versions of each chapter was provided to the team 7/21/00. It was identified that the adoption of the Hydraulics Manual would be carried out by the City Council and sponsored by Streets.

D. Culverts & Bridges Chapter

A sensitivity analysis of the HEC-14 scour equation for cohessionless material (0.2 mm sand & 2.0 mm sand) was provided for discussion. Here, the assumptions in the analysis were identified. It was agreed to utilize both equations presented. Stantec is to revise that portion of the chapter and re-distribute.

Kofi Awumah suggested that a minimum toe-down depth be identified. Frank Thomas indicated that standards were to be specified in "Volume Zero". A reference to the standards for toe-down is to be incorporated into the Chapter 5 text. Finally, it was determined that a safety factor need not be applied to the results of the scour equation based upon recommendations identified in "Municipal Storm Water Management" by Debo & Reese.

E. Manual Preamble/Foreword

Ed Raleigh suggested that the Foreword include a section on Revisions that discussed in general terms the changes that have occurred on a chapter by chapter basis. The C.O.P. was subsequently satisfied with the Foreword

Frank Thomas related a conversation he had with Ken Lewis regarding the documentation of manual revisions. Here, the two issues seem to be the documentation needed for some future retrieval of archived versions to support legal proceedings and the documentation of changes that would be helpful those designers in the middle of a project at the time of a change. Ken Lewis suggested a separate on-line document to track/list changes in detail. The only change to the manual itself would be a change in the date on the front cover ("Last Updated…").

Further comments on the Preamble/Foreword were deferred, pending review by Tim Murphy at FCDMC.

F. Introduction Chapter to Hydraulics Manual

Discussion of this chapter was deferred until the next meeting.

G. Hydraulics Manual Chapter 2- Hydrology

The first draft changes were accepted. Stantec is to revise and submit the second draft for the team to review.

H. Chapter 7 – Hydraulic Structures

The drop structure section requires review of maximum height and minimum step length to strike dimensions in conflict with those in the Chapter 1, Safety Section. Here, the maximum height desired is 2.5' with a minimum step length of 6'.

Frank Thomas identified that he had updated definitions for sharp and broad crested and identified "C" values for roadways as broad crested weirs.

Reference: Drainage Design Manual

The discussion on low flow check structures is to be re-written as a discussion on grade control structures. In addition, a new section is to be added to discuss groins/dikes. One reference to be reviewed for guidance is the "Highways & Riverine Environments" by FHWA.

Ed Raleigh indicated that he expected their internal evaluation/review of side channel spillways to be complete within the next couple of weeks. This document would identify FCDMC's preferred method(s). He would provide a copy to Stantec.

I. Other Business

No other business was identified.

K. New Action Items

Stantec

Incorporate culvert toe down/scour analysis into Culverts and Bridges 2nd draft.

Distribute second draft of Chapter 2 Hydrology of the Hydraulics Manual.

Incorporate responses/comments to Regulations and Planning first drafts pending receipt from City. Assist City of Phoenix in facilitating review of draft policies.

Review side channel spillway document from FCDMC.

Modify Preamble/Foreword to Hydraulic Manuals pursuant to comments to be received from Tim Murphy. Stantec is to summarize changes to the chapters (briefly)

Modify Hydraulic Structures Chapter to include discussions on groins and guide dikes. Modify low flow check structures to grade control structures. Review drop structure discussion for references to maximum height and minimum step length. Review and incorporate changes provided by FCDMC

Move forward with conversion to PageMaker.

Incorporate figures into chapters.

City of Phoenix

Review Planning and Regulations. Provide Stantec with changes to Policies and Standards prior to initiating internal policy review

Facilitate internal review of Policies, and Standards

Look into having the shotcrete standard changed at MAG.

Stantec

FCDMC

Complete guidelines for side channel spillways.

Assist in the first level of software testing.

Reference: Drainage Design Manual

Interagency

Review & comment on Hydraulics Manual Introduction, 2nd draft of Chapter 2, Toe-down/scour equation portion of Chapter 5, and 2nd draft of Hydraulics Structures Chapter.

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately.

STANTEC CONSULTING INC.

Frank Thomas, PE
Project Manager

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Meeting Notes

Stantec

FILE: 82000042

Date: 06 September 2000

Place/Time: Stantec/1:00 PM

Next Meeting: 11 October 2000 @ Stantec, 1:00 PM

Attendees: City of Phoenix Ray Acuna, Gary Benton, Ralph Goodall, Jason Turnbaugh
FCDMC Kofi Awumah, Amir Motamedi, Ed Raleigh,
Stantec Mike Gerlach (part time), George Sabol, Scot Schlund (part time), Frank Thomas

Distribution: Attendees/Absentees

Item:

A. Meeting Minutes Approval

No comments were received on the 26 July 2000 meeting minutes and they were so approved.

B. Review Previous Action Items

Stantec undertook the following:

Distributed Sedimentation Chapter.

Incorporated culvert toe down/scour analysis into Culverts and Bridges 2nd draft and distributed.

Distributed second draft of Chapter 2 Hydrology of the Hydraulics Manual.

Modified Hydraulic Structures Chapter to include discussions on groins, guide dikes, and grade control structures. Reviewed and modified drop structure discussion for references to maximum height and minimum step length. Reviewed and incorporated changes provided by FCDMC. Distributed to team.

C. Hydrology Manual Update

Mike Gerlach gave an overview of the issues presently being addressed in the Hydrology Manual. The focus is presently on the two indirect verification methods, as presented in the ADOT Manual, that employ regression equations and whether these need updating for 10 more years of record and the addition of more gages. There has been on-going discussion as to the extent of the data base to be included, i.e. limit the gage data to Maricopa County or the entire State. Stantec will not be undertaking statistical analysis of the new gage data as it applies to the USGS regional regression equations (Method 3 of the ADOT Manual). Stantec is still working on the applications for more frequently occurring storm events.

There was an in depth discussion as to eliminate Chapter 7 and incorporate examples into each chapter since many users apparently do not utilize this chapter. The FCDMC desires a consolidated example that steps through the major topics of each chapter. The issue was left unresolved.

Amir Motamedi asked the City of Phoenix if they were satisfied with the evolution of the Hydrology Manual, citing past concerns with the complexity of previous edition. Ralph Goodall indicated his acceptance of the technology and that the new version was indeed becoming more user friendly. He suggested that the manual be peer reviewed. George Sabol asked if the City had a project that could be design via the existing methods and the revised methods as a test. The mechanism for peer review was left for further consideration.

Ed Raleigh brought up the issues pertaining to the time of concentration calculations. George Sabol reiterated the intent of the original edition that was to promote hydrological methods that were accurate, reproducible, and practical. He acknowledged that the method for time of concentration may have leaned more towards accuracy than practicality, but this parameter was of utmost importance. The FCDMC is evaluating those issues and will get back to the team.

Stantec is to assume that the reader has knowledge and basic understanding of HEC-1 as it updates the chapters.

D. Introduction Chapter to Hydraulics Manual

All references to Volume I and Volume II are to be changed as appropriate (this is to be done for all chapters). Stantec will incorporate the FCDMC comments and finalize the chapter.

E. Chapter 2 Hydrology

Chapter 2 was accepted. Stantec is to finalize this chapter.

F. Culverts & Bridges Chapter

Stantec revised the scour analysis portion of the chapter. The changes were accepted.

Stantec

G. Chapter 7 – Hydraulic Structures

The drop structure section was revised for maximum height and minimum step length to strike dimensions in conflict with those in the Chapter 1, Safety Section. Here, the maximum height desired is 2.5' with a minimum step length of 6'. Groins, guide dikes, and grade control structures were added. Low flow check dams were removed. Pending resolution of side channel spillways by the FCDMC and incorporation of comments, Chapter 7 is complete. Stantec is to finalize the chapter and leave a place holder for side channel spillways.

H. Chapter 10 Sedimentation

Two issues were discussed. First was a general discussion pertaining to the overall approach to the chapter. Stantec understood that the chapter was to be an introduction to the topic to enlighten the uninitiated such that the user would have knowledge of the terms and processes associated with sedimentation. Kofi Awumah suggested a format that took the reader from a qualitative approach to a quantitative approach in a three tier process. The FCDMC will evaluate its desires for the chapter and get back to the team.

I. Other Business

There was a discussion pertaining to the beta testing of the Chapter 4 software reflecting the City of Phoenix's appreciation of the FCDMC's efforts. Ed Raliegh is to send Stantec their scope/schedule for this testing.

Stantec is to complete summaries of the changes made to each chapter for inclusion in to the Preamble/Foreword.

K. New Action Items

Stantec

Finish first draft of Chapters 2 through 6 and 8 of the Hydrology Manual and distribute.

Review side channel spillway document from FCDMC.

Finish modifying Preamble/Foreword to Hydraulic Manuals to summarize changes to the chapters (briefly) and distribute.

For all chapters, all references to Volume I and Volume II are to be changed as appropriate.

Stantec will incorporate the FCDMC comments and finalize Chapter 1.

Stantec

Stantec is to finalize Chapter 2.

Stantec is to finalize Chapter 5.

Stantec is to finalize Chapter 7, leaving a place holder for side channel spillways which is to be provided by FCDMC.

Reference: Drainage Design Manual

Move forward with conversion to PageMaker.

Incorporate figures into chapters.

City of Phoenix

Facilitate internal review of Policies, and Standards

FCDMC

Evaluate needs for Sedimentation Chapter relative to the first draft.

Resolve time of concentration issues for the Hydrology Manual.

Look into having the shotcrete standard changed at MAG.

Complete guidelines for side channel spillways.

Assist in the first level of software testing.

Interagency

Review Hydraulics Manual Preamble/Foreword.

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately.

STANTEC CONSULTING INC.

Frank Thomas, PE
Project Manager

Meeting Notes

Stantec

FILE: 82000042

Date:	11 October 2000
Place/Time:	Stantec/1:00 PM
Next Meeting:	08 November 2000 @ Stantec, 1:00 PM

Attendees:		
	City of Phoenix	Gary Benton, Ralph Goodall (late)
	FCDMC	Kofi Awumah, Ed Raleigh, Tim Murphy
	Stantec	George Sabol (late), Frank Thomas

Distribution: Attendees/Absentees

Item:

A. Meeting Minutes Approval

No comments were received on the 06 September 2000 meeting minutes and they were so approved.

B. Review Previous Action Items

Stantec undertook the following:

- Finished first draft of Chapter 2 and distributed at the meeting along with a hand calculation example.
- Reviewed side channel spillway document from FCDMC.
- Distributed modified Preamble/Foreword to Hydraulics Manual that summarized changes to the chapters.
- Removed all references to Volume I and Volume II from Hydraulics Manual.
- Incorporated final comments on second drafts for Chapters 1, 2, 5, & 7 (left a placeholder for side channel spillways).
- Acquired Framemaker software.
- Modified Policy chapter to include water quality policy statement.

C. Schedule Update

Frank Thomas overviewed schedule as follows:

- Framemaker conversion of Hydraulics Manual targeted to be completed in early December for Chapters 1-9, with Chapter 10 completion pending outcome of 10/11/00 meeting.
- To expedite the Hydrology Manual, Stantec will not let meeting schedule dictate submittals, but rather, will send out chapters upon completion and will allow 10 days review prior to starting the next draft. Stantec will target completion of 2nd draft by end of December.

- With 3 of the 7 meetings completed for the C.O.P. Policies & Standards Manual, it appears that first round of internal facilitation will not be completed by mid-December as hoped.

D. Discuss Preamble/Foreword

Minor written comments were received from Kofi Awumah. No other comments were received.

E. Geotechnical Engineering Studies and Other Areas of Expertise
Chapter 8 contained verbage identifying requirements for geotechnical engineering expertise. This section was previously flagged for further discussion. The issue at hand was the need to identify other areas of expertise that may reasonably be expected in some drainage projects. It was decided to utilize much of the verbage in Chapter 8 in a stand alone section of the introduction chapter to highlight some of the other disciplines utilized in drainage design and stormwater management. The application of structural and environmental engineering shall be included in this section.

In addition, the discussion in Chapter 8 was to be clarified for embankments over 2.5' of hydraulic height, with hydraulic height defined.

F. Chapter 4: Storm Sewer Chapter revisited
In lieu of examples based upon the forthcoming storm sewer software, it was accepted that Stantec would provide "hand" calculations that would exemplify the key points of the chapter. Stantec recommended, and it was accepted, that the freeboard depicted in the hydraulic grade line figure would not depict a set distance as this would be relegated to the standards of a particular jurisdictional entity.

G. Chapter 10 Sedimentation
The FCDMC identified that it accepted the chapter content but requested that an outline of the sedimentation analysis process be included analogous to the one provided in the open channel chapter. Labeling levels of analysis was not necessary. The steps in the analysis process would include links to other relevant chapters/sections in the manual. This outline would include the caveat that the checklist was not inclusive. The chapter will provide additional references to be cited.

The erosion setback discussion was satisfactory as is.

The sand & gravel mining section was to provide references only, (3 from FCDMC, and one from George Sabol) as the analysis required for these facilities is beyond the scope of the chapter.

H. Other Business

Stantec is to include the hydrology figures in its distribution of draft chapters.

Reference: Drainage Design Manual

I. New Action Items

Stantec

Complete and distribute remaining 1st drafts of Hydrology Manual Chapters

Finalize Preamble/Foreword, incorporating FCDMC comments.

Modify geotechnical text in Chapter 8, Add Section 1.5 (additional prof. resources)

Finish examples for Chapter 4, Storm Sewers

Prepare & distribute 2nd draft of Chapter 10, Sedimentation

Move forward with conversion to FrameMaker.

Incorporate figures into chapters.

City of Phoenix

Facilitate internal participation & review of Policies and Standards Manual

FCDMC

Resolve time of concentration issues for the Hydrology Manual.

Look into having the shotcrete standard changed at MAG.

Complete guidelines for side channel spillways.

Assist in the first level of software testing.

Interagency

Review Hydrology Manual chapters as they become available.

Review 2nd draft Chapter 10, Sedimentation

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately. **STANTEC CONSULTING INC.**

Frank Thomas, PE
Project Manager

Meeting Notes

Stantec

FILE: 82000042

Date:	13 December 2000
Place/Time:	Stantec/1:00 PM
Next Meeting:	18 January 2001 @ Stantec, 1:00 PM

Attendees: City of Phoenix
FCDMC Amir Motamedi, Tim Murphy. Joe Rumann, Kofi
Awumah
Stantec Frank Thomas, George Sabol, Mike Gerlach

Distribution: Attendees/Absentees

Item:

A. Meeting Minutes Approval

No comments were received on the 08 November 2000 meeting minutes and they were so approved.

B. Identify Modifications to Chapter 10, Sedimentation, discuss 3 tier approach & establish schedule to finalize

George Sabol gave an overview of the changes made to the chapter. Several sections were re-written to eliminate copy right issues. These sections were not done in strike-out format since the substance of the sections remained unchanged. The reference list was expanded to include additional documents for the interested reader.

The section entitled "Approach To Sedimentation Analysis" was discussed in length as to clarifying procedures. Stantec shall make the revisions and distribute to FCDMC for review (completed12/15/00).

Stantec received photos from FCDMC for the Sedimentation chapter that it shall review and insert as appropriate.

The schedule for completion of this chapter was set to the second week in January.

C. Chapter 2 Hydrology Manual

The depth area reduction factors for the 6 and 24 hour storms were discussed with the intent of making them the same for the applicable overlap areas. This chapter is ready to be finalized.

Reference: Drainage Design Manual

D. Chapter 6 (Routing) Hydrology Manual

The Muskingum-Cunge routing method was discussed in terms of its validity of results and applicability to the watercourses in Maricopa County. It was concluded that while its results may be questionable for most watercourses in Maricopa County, it is an acceptable method for large rivers and therefore, should stay in the manual. The FCDMC does not use this method on its projects and its use should be cautioned.

Chapter 6 is ready for finalization.

E. Chapter 4 (Rainfall Losses) Hydrology

The procedure outlined in the chapter needs to be checked with that in DDMS-W.

A footnote should be added to explain that the assumption of soil horizon saturation by irrigation is not valid for large drainage areas since irrigation delivery schedules preclude coverage over large areas at one time. The FCDMC will provide comments within two weeks.

The three soil reports for Maricopa County not previously summarized within the Manual should be included at the FCDMC's discretion.

Upon completion of the first two items, Chapter 4 is ready for finalization.

F. Chapter 5 (Unit Hydrograph Procedures)

This chapter was tabled until the next meeting.

G. Chapter 3

The IDF from PREFRE was accepted to replace the present Phoenix Airport information. The lower limit for the time of concentration was suggested to be 10 minutes instead of 5 minutes with the understanding that little harm would come with this change and that the 5 minute minimum would result in over designing infrastructure. The FCDMC will verify its position on this matter. A note is to be added to the text indicating that the 10 minute minimum should not be used for the design of roof drainage.

This chapter is on hold until Ken Lewis is done with his model development.

H. Chapter 8 – Indirect Methods

Methods 1 & 3 have been reviewed with minimal changes. Stantec is still grappling with determining which database is the most applicable to use for Method 2.

The issue of using the existing methodology for more frequent storms is still under consideration by Stantec. The FDCMC is going to assess more frequent storms as a percentage of the 100 year for certain gauged basins.

Reference: Drainage Design Manual

I. New Business

Stantec reported that it was almost finished incorporating revisions to Chapter 4, Storm Sewers (Hydraulics Manual) as requested by FCDMC. Only one change remained which is to be resolved in the near term.

J. New Action Items

Stantec

Provide KVL and FCDMC with Chapter 4 Storm Sewer example calculations

Finalize Chapters 2, 4, and 6 of the Hydrology Manual

Address questions pertaining to Chapter 4 example.

Finalize Chapter 10, Sedimentation of the Hydraulics Manual, incorporate photos

Finalize Chapter 4 of the Hydraulics Manual

Move forward with conversion to FrameMaker & incorporate figures into chapters.

City of Phoenix

Facilitate internal participation & review of Policies and Standards Manual

FCDMC

Resolve minimum time of concentration issues for the Hydrology Manual.

Look into having the shotcrete standard changed at MAG.

Complete guidelines for side channel spillways.

Assist in the first level of software testing.

Interagency

Review Hydrology Manual chapters as they become available and provide Stantec with comments.

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately. **STANTEC CONSULTING INC.**

Frank Thomas, PE
Project Manager
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Stantec

FLOOD CONTROL DISTRICT
of
Maricopa County

Interoffice Memorandum

Date: December 21, 2000

TO: BIZ

VIA: MAL

FROM: RPH

SUBJECT: Drainage Design Manual for Maricopa County Volume II, Hydraulics, Storm Drains Design Section, Example Problem Review Comments

I have reviewed the subject materials and offer the following comments:

General

1) In order to illustrate the application of flow depth in the junction loss equation, the example problem should include a pipe segment and junction for which partial flow conditions prevail. Please address. *Reply: We acknowledge that the above approach would illustrate calculation of junction losses under partial flow, but feel that typical application/design would not call for partial flow (i.e. we design our system to flow full under design conditions in order to minimize cost)*

2) For junctions where the two in-line pipes are the same size, the transition loss element of the junction loss equation should be replaced with equation 4.10 (currently the manhole loss equation), since the feature is better considered a manhole, than a contraction or expansion. *Reply: We use the manhole equation when there is not a third pipe. When there is a third pipe, we use junction loss equation.*

3) For junctions where the two in-line pipes are not the same size, the transition loss element of the junction loss equation should include absolute value brackets around the velocity heads, and either a contraction or expansion coefficient, K. Please revise accordingly. *Reply: Revised as requested.*

4) The signs (+/-) of the "Z" elements in the junction loss equation should be reversed. Please address. *Reply: These equations have been checked.*

5) Given comment number 2, above, the manhole loss calculated in section 4.4.2 would be redundant. Please address. *Reply: See item 2 response.*

6) In order to illustrate a method to determine the most cost effective designed connector pipes, I suggest that the example problem include the following approach.

Given the catch basin inlet elevation and the current design constraint that there should be at least 1' difference between that elevation and the catch basin HGL elevation, an allowable head loss (AHL) for the connector pipe would be: AHL=(CB inlet elevation-1')-(trunkline HGL elevation). Then the following equation could be solved by iteration:

$$AHL=V^2/2xG(1+K_e+(5.39xN/R^{2/3})^2xL).$$

Where:

V=pipe full flow velocity of selected pipe
G=gravitational acceleration
K_e=entrance loss coefficient
N=Manning's roughness coefficient
R=hydraulic radius of selected pipe
L=pipe length

When the results of the right-hand side are slightly below the left-hand side, the designer should stop iterations and select the next largest sized pipe. For application towards the example problem, I recommend that one of the catch basins be large, such as a P1569 M-2, L=17', which has a total opening width of 37'. Then the equation above could be applied to determine best pipe size. *Reply: Alternatively, don't most designers use ground slope as an approximation of energy slope and design pipe by trial and error? Is the above method more efficient or easier? Please provide further guidance.*

7) By reducing the number of pages and/or inferring some calculations to avoid redundancy, a shorter example problem may promote designer interest. Twenty-six pages seems like too many. Please address. *Reply: Yes, there is an opportunity to reduce the number of pages by using independent/unrelated examples to illustrate the equations/methodologies highlighted in the chapter. However, we thought that the analysis of a complete system would be more illustrative for the uninitiated.*

Page 1

1) The example problem schematic labels should be enlarged to improve legibility. In addition, catch basin inlet elevations should be shown instead of top-of-curb, since they are more important in determining the most efficient design. Please revise accordingly. *Reply: The labels* *will be enlarged. We felt that since the "V" depth is called out in the standards as from the top of curb, that top of curb was more appropriate than gutter elevation.*

Page 2

1) Under "Given, Item 10, the listed feature J010020 could not be found on the SD schematic. Please check. *Reply: This has been corrected.*

2) Since 21" pipe is not commonly available commercially, the problem may be improved by using a 24" pipe instead.

3) An added "Given" item should be a design constraint target for the catch basin HGL elevation to be at least 1' below the catch basin inlet elevation. Please address. *Reply: This has been corrected.*

Page 3

4) As described in item 2.2.2, the location of the invert is not clear. I was given the initial impression that the location was at the retention basin, not the up-pipe invert. A better description is needed. *Reply: This has been corrected.*

Page 5

1) In item 2.6.3, first line, the word "form" should be changed to "form". Please address. *Reply: This has been corrected.*

2) The catch basin ID's listed on this page should begin with a zero to be consistent. Please address. *Reply: This has been corrected.*

Page 6

1) Contrary to what is described in section 2.10.1, the Tc for the system flows at the lower end of the listed pipes (ending at 010030) will be longer due to travel times. Therefore the peak design flows will be less (this approach is applied further on in section 3.2). Please revise accordingly. *Reply: Technically, you are correct, but we designed the system for the peak discharge entering the storm drain. Since there were not any additional inflows from laterals, we used the peak flow into the storm drain. In addition, there is not enough storage in the storm drain to justify routing down the peak, so we would suggest leaving the problem as is.*

Page 9

1) Based upon the description in section 4.2.1, shouldn't the starting HGL listed in section 4.2.4 be equal to 1270.59'. Please check and revise all subsequent results accordingly. *Reply: We used the water surface in the retention basin at the time of peak discharge.*

Page 10

1) The units of S_f should be listed as in ft/ft. Please address. *Reply: This has been corrected.*

Page 14

1) As listed in section 4.6.2, figure 4.8 and equation 4.10 are not numbered the same. Please check/explain. *Reply: Figure 4.8 and equation 4.10 are from Chapter 4, Storm Drains, of the Hydraulics Manual. We assumed that the reader would know that since the example is part of Chapter 4. Perhaps, we need clarification of your concern.*

Page 25

1) Table E-4 should be revised to include a column that shows depth of flow in the pipe. Please add. *Reply: Table E-4 shows crown elevation and HGL. We presumed that flow depth, if partial, could be inferred from that information....again, the premise we were under was that the example reflect typical design situations, including full flow for cost effectiveness.*

Page 26

1) Table E-5 should be revised to include columns for catch basin inlet elevations, catch basin HGL elevations, and the difference between the two, to prove that design criteria are met. *Reply: This has been corrected.*

Meeting Notes

Stantec

FILE: 82000042

Date: 8 February, 2001

Place/Time: Stantec/1:00 PM

Next Meeting: 21 March, 2001 @ Stantec, 1:00 PM

Attendees:		
	City of Phoenix	Ralph Goodall, Gary Benton
	FCDMC	Amir Motamedi, Tim Murphy. Joe Rumann, Kofi Awumah
	Stantec	Frank Thomas, George Sabol, Mike Gerlach
	KVL	Ken Lewis

Distribution: Attendees/Absentees

Item:

1. Meeting Minutes Approval

No comments were received on the previous meeting minutes and they were so approved.

2. DDMSW Procedures

Mike Gerlach discussed differences between computational procedures presented in the Manual and the computerization of those procedures as implemented in DDMSW, specifically in regard to the computation of rainfall losses and the Clark unit hydrograph. The discussion of these differences was focused on the original intent of the procedures in the Manual and the process of following the procedures from a user prospective. Specific items addressed were the input and selection of the Kb parameter, vegetative cover percentage and surface retention parameter as they relate to land use and soils.

Ken Lewis provided input and background as to why Kb, vegetative cover and surface retention are only input options relating to land use and offered several suggestions to resolve the procedural differences. Amir Motamedi, Kofi Awumah and Joe Rumann provided input as to the appropriateness of where those input data should be entered. Amir Motamedi concluded the discussion by pointing out that the procedural differences were not fatal flaws with the program and that specific testing of DDMSW would be appropriate following the procedures as outlined in the Manual.

Ken Lewis suggested that he and Mike Gerlach should meet to work through an example together.

3. Hydrology Manual Status

Mike Gerlach stated that comments regarding the Chapters submitted for review at the previous meeting (Introduction, Chapter 1, Chapter 2, Chapter 4, Chapter 5 and Chapter 6) have been incorporated into a final version with the exception of Chapter 5. Amir Motamedi stated that the FCDMC had no comments other than purely editorial on Chapter 5. Amir Motamedi also stated that there are no current plans to revise the Tc equation at this time. Chapter 5 is ready to be finalized.

Comments on the first draft of Chapter 3 were provided by all attendees from the FCDMC. The comments focused primarily on the limitations of the application of the Rational Method, particularly in regard to the drainage area and routing limitation. It was suggested that the wording of the routing limitation be taken from the Hydraulics Manual. Amir Motamedi provided a table of runoff coefficients (taken from existing publications) that should be considered as a replacement of the current table. The values in the new table would be based on dwelling units per acre and could be extended to all jurisdictional zoning. Frank Thomas suggested that the runoff coefficient table be moved to the policies and standards manual. Stantec was directed to prepare an example using values recommended by the City of Phoenix and values recommended by the FCDMC. Joe Rumann pointed out that a minimum value for Tc of 10-minutes was agreed upon at the previous meeting and that that limitation was not incorporated into the first draft.

Mike Gerlach provided an overview of the contents of Chapter 8. Also discussed was an analysis of gage data as it relates to Indirect Method 2. Mike Gerlach stated that the results of the gage data analysis indicated that adoption of the data as presented in the ADOT Manual and the State Standards is appropriate. Comments on the first draft of Chapter 8 were provided by all attendees from the FCDMC. Kofi Awumah suggested that the word "verification" is inappropriate and should be changed. Kofi Awumah also suggested that the maximum discharge data set and regression equation for Method 2 is not a particularly important "verification" tool and should be eliminated. Amir Modamedi suggested that some/all data points shown on the figures for Method 2 be eliminated as it is difficult to use. Removal of the maximum discharge points would help in this regard. It was also suggested by many to provide the data digitally so that data points of interest could be selected for plotting and that this data could be incorporated into DDMSW. Amir Motamedi also suggested that elimination of curves G, H and possibly C from the data set for Method 1 would provide a maximum discharge data set only, this would then support removal of the maximum discharge data set from Method 2.

Reference: Drainage Design Manual

4. 2- and 5-Year Storm Procedures

Mike Gerlach provided a discussion of a method for modeling of the more frequent storm events. The proposed method would apply a factor to the 100-year runoff hydrograph. The factor would be based on an analysis of gage data. A table presenting a summary of simple statistical analyses of various subsets of the overall data set was distributed. Included on the table were factors from three independent sources. George Sabol added a discussion as to the need and reason for the development of a procedure for modeling more frequent runoff events. A lively discussion ensued with comments, concerns and questions offered by all attendees. Amir Motamedi summarized and concluded the discussion by stating that this issue can be resolved by answering three questions:

1. Is there a need to have a specific procedure other than simply changing the rainfall depth and rerunning DDMSW,

2. If so, what are the sensitive parameters, i.e. rainfall, rainfall losses, routing, etc., and

3. What changes to these parameters would be necessary to achieve reasonable results for the more frequent events.

George Sabol requested that Stantec begin testing of existing studies to answer those questions. Amir Motamedi directed Stantec to begin the testing.

5. New Action Items

Stantec will finalize Chapters 3, 5 and 8 and begin testing on select watersheds.

6. Next Meeting

The next meeting is scheduled for 21 March, 2001.

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately. **STANTEC CONSULTING INC.**

Stantec

Mike Gerlach, PE
Project Engineer
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Meeting Notes

Stantec

FILE: 82000042

Date:	25 September, 2001
Place/Time:	Flood Control District/1:00 PM
Next Meeting:	23 October, 2001
Attendees:	FCDMC Amir Motamedi, Tim Murphy, Tom Loomis Stantec Frank Thomas, George Sabol, Mike Gerlach
Distribution:	Attendees/Absentees

George Sabol began by providing Tom Loomis with a general history and background of the project since its inception. Key items discussed were

- Intent of the original contract under the City of Phoenix and the needs for a manual from that perspective,
- Inclusion of the Flood Control District in the decision making processes and the general adoption of current Flood Control District methodologies with the recognition that certain procedural and methodological updates were necessary, and
- General layout and organization of the manual as a finished product.

Frank Thomas continued by providing Mr. Loomis with an overview, current status and general content of the Policies and Standards Manual as well as the Hydraulics Manual. Key items discussed were:

Mike Gerlach concluded the meeting by providing Mr. Loomis with an overview, current status and general content of the Hydrology Manual. Key items discussed were:

- Status of each chapter as well as the general nature of the changes proposed for each chapter,
- Discussion of the level of effort and general analyses conducted in regard to a new chapter on indirect methods of verification of modeling results, and
- Brief presentation of a proposed approach for modeling of the more frequent storm events. This discussion was concluded with a request for a working meeting with Flood Control District staff to further explain analyses conducted

in this regard and to explore and to provide an opportunity for open discussion on this topic. This meeting is tentatively scheduled for 23 October 2001.

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately. **STANTEC CONSULTING INC.**

Mike Gerlach, PE
Project Engineer
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FLOOD CONTROL DISTRICT
of
Maricopa County

2801 West Durango Street • Phoenix, Arizona 85009-6399
Telephone (602) 506-1501
Fax (602) 506-4601
TT (602) 506-5897

Mr. R. Michael Gerlach, PE
Stantec Consulting Inc.
8211 South 48th Street
Phoenix, AZ 85044

Dear Mike:

The purpose of this letter is to address your request for guidance on several of the tables and figures in the Hydrology Manual. I am also providing example watershed data for use in testing the frequent storm ratio procedures discussed at our last meeting. I have enclosed quite a bit of information, organized as attachments, and offer the following descriptions and guidance for use of this information. Please contact me as soon as possible if you or Dr. Sabol disagree with any of the recommended revisions.

Attachment 1: Depth-Area Reduction Factors for 24-Hour Duration Rainfall. I also found differences between a plot of the tabular data and the original Hydro-40 curve. There are two 11x17 copies in the attachment. One is a Xerox enlargement of the figure from Hydro-40. The other shows the 1995 tabular data and a plot of my recommended data. I have also included copies of historical correspondence regarding the source of the original tabular data. The first two pages show the recommended data table and figure for use in the 2002 manual. The third and fourth pages show comparisons of the original data with the new data, and comparisons of data used in the current version of MCUHP and the recommended replacement data. See Excel spreadsheet "Hydro 40 24hr Reduction.xls" on the CD-ROM.

Attachment 2: Depth-Area Reduction Factors for 6-Hour Duration Rainfall. I also agree that the data for this curve is acceptable. However, I found discrepancies in the way this curve was implemented in MCUPH. Therefore, the first two pages of the attachment are a table and figure showing the data I would like presented in the 2002 manual. I have included additional data points in the table at the break points for the rainfall distribution patterns. I would like to see the data at the pattern break points shaded as shown or identified in another manner. I have also included the reduction curve data recommended for coding in MCUHP (I'll take care of this). This data could also be included in the manual. We decided to limit the watershed area defined by HEC-1 JD records to 100 square miles. If a user needs to model a watershed for the 6-hour storm that is larger than 100 square miles, special coding will be necessary. The recommended table uses the maximum number of JD records allowed, and better simulates the reduction curve. The last two pages show comparisons of the original data with the proposed, so you can see why we are making this change. See Excel spreadsheet "6hr Depth-Area Reduction.xls" on the CD-ROM.

Attachment 3: 2-Hour Storm Distribution for Retention Design. This was the most interesting problem, but probably the most insignificant from a practical standpoint. I used the latitude and longitude for the original Sky Harbor Airport Weather Gage to obtain NOAA Atlas 2 point precipitation values for that location. I then used that data to run PREFRE, and used the PREFRE output to code the PH record in a simple test HEC-1 model. I then used the 2-hour storm rainfall distribution computed by HEC-1 for my comparisons with the original data in the manual, MCUHP, and the data you sent me from the documentation manual. I ran different versions of the HEC-1 model using time intervals of 2-, 5- and 15-minutes. The results are included in Attachment 3. The original tabular data in the manual matches the HEC-1 rainfall distribution computed using a 15-minute time interval. The figure in the manual didn't check against anything I tried, and I can only conclude that it was not created using the available data. Since a 15-minute interval is too large for use with a 2-hour storm, we want to use the 5-minute curve shown on the first two pages of the attachment. We will also have to recode this curve in MCUHP. See Excel spreadsheet "2-hour storm distribution.xls" on the CD-ROM.

Attachment 4: SCS Type II 24-Hour Storm Rainfall Distribution. Wouldn't it be nice if we could agree on a standard naming convention? We use different wording for each storm in the tables and figures. I have included a table and figure for the data used in MCUHP. I would like this information included in the manual for consistency. I have also included a copy of the original SCS table, which matches the recommended data. Interestingly, even this curve has an associated puzzle. The data is coded in MCUHP using a 5-minute time interval, and output using a 15-minute interval. I have no idea where the 5-minute data came from, but it plots a very smooth curve, and includes the original SCS data. The figure included on page two is based on the 5-minute data. See Excel spreadsheet "24-hour storm distribution.xls" on the CD-ROM.

Attachment 5: Slope Adjustment for Steep Watercourses. Bing Zhao has performed a regression analysis on data scaled from the figure in the Hydrology Manual. A table and figure based on the resulting polynomial is included. We want to add a table in the Hydrology Manual that includes the equation, and use the revised figure with background gridlines. The supporting data is included. The equation will be implemented in WMS and DDMSW. See Excel spreadsheet "Slope Adjustment for Tc.xls" on the CD-ROM.

Attachment 6: Excerpt from *Maryvale Area Drainage Master Stud.* For use in evaluating the ratio method for more frequent storms. This is an example of a heavily urbanized watershed with multiple diversions. I suggest using sub basins 2-31, 11-31, 12-31, 14-31, 13-31, and 7-32. Input files, spreadsheets, DDMSW files, and key exhibits in TIF format are on the CD-ROM.

Attachment 7: Excerpt from *Cudia City Wash To 10TH Street Wash Watershed Hydrology Report.* For use in evaluating the ratio method for more frequent storms.

This is an example of a partially urbanized watershed with rain gage and flow gage data available. Stream and precipitation gage data, HEC-1 input files, spreadsheets, DDMSW files, and key exhibits in TIF format are on the CD-ROM.

Attachment 8: Excerpt from *White Tanks/Agua Fria Area Drainage Master Study, Part A: Flood Study Technical Data Notebook.* For use in evaluating the ratio method for more frequent storms. This is an example of an agricultural watershed. No rain gage and flow gage data is available. The HEC-1 input files and the watershed exhibit in TIF format are on the CD-ROM.

Hope you make sense out of all of this. Please give me call with any questions.

Sincerely,

Thomas R Loomis

Thomas R. Loomis, PE, RLS
Special Projects Branch Manager

SUMMARY OF REVIEW OF TABLES AND FIGURES, AND CHANGES TO BE MADE.

Table	Figure	Title	MCUHP1 Array	MCUHP2 Array	Comments
2.1a	2.1a	Depth-Area Reduction Factors for 24-Hour Duration Rainfall	RFC and DAR	RFC and DAR	New table and figure for manual. DDMSW may need to be revised. MCUHP1 and MCUHP2 source code revised 01/02/02 and 1/03/02. MCUHP1 and MCUHP2 do not create JD records using these factors. They should be revised similar to method used for the 6-hour.
2.2	2.14	Depth-Area Curve for Maricopa County 6-Hour Storm	Hard-coded, Line 502	Hard-coded, Line 8912	Currently input to HEC-1 using Pattern breaks only. MCUHP1 and MCUHP2 need to be expanded to include all values in Table 2.2 up to 100 square miles. Don't include areas greater than 100 sm in P1 and P2.
2.3	2.15	2-Hour Storm Distribution for Retention Design	Q4	Q4	New table and figure for manual. MCUHP1 and MCUHP2 source code revised 01/02/02.
2.4	2.16	6-Hour Distribution Pattern 1, 0.0 <= 0.5 sm	Q1	Q1	Agreement between Manual and MCUHP1 and MCUHP2.
2.4	2.16	6-Hour Distribution Pattern 2, 2.8 sm	Q2	Q2	Agreement between Manual and MCUHP1 and MCUHP2.
2.4	2.16	6-Hour Distribution Pattern 3, 16.0 sm	Q3	Q3	Agreement between Manual and MCUHP1 and MCUHP2.
2.4	2.16	6-Hour Distribution Pattern 4, 90.0 sm	Q8	Q8	Agreement between Manual and MCUHP1 and MCUHP2.
2.4	2.16	6-Hour Distribution Pattern 5, 500 sm	Q9	Q9	Agreement between Manual and MCUHP1 and MCUHP2.
None	5.4	Slope Adjustment for Steep Watercourses in Natural Watersheds	Not included	Not included	New table for manual. New equation for DDMSW.
5.2	None	Synthetic Dimensionless Time-Area Relations - Urban	IU	IU	Agreement between Manual and MCUHP1 and MCUHP2.
5.2	None	Synthetic Dimensionless Time-Area Relations - Natural	IN	IN	Agreement between Manual and MCUHP1 and MCUHP2.
5.3	5.9	Phoenix Valley S-Graph	n/a	TTABLE(##,1)	Agreement between Manual and MCUHP2.
5.3	5.10	Phoenix Mountain S-Graph	n/a	TTABLE(##,2)	Agreement between Manual and MCUHP2.
5.3	5.11	Agricultural S-Graph	n/a	TTABLE(##,3)	Agreement between Manual and MCUHP2.
5.3	5.12	Desert/Rangeland S-Graph	n/a	TTABLE(##,4)	Agreement between Manual and MCUHP2.

ATTACHMENT 1

Table 2.1a
Depth-Area Reduction Factors for 24-Hour Duration Rainfall

Area, square miles	Ratio to Point Rainfall
0	1.000
10	0.950
20	0.918
30	0.900
40	0.887
50	0.877
60	0.870
70	0.863
80	0.857
90	0.852
100	0.848
110	0.845
120	0.841
130	0.838
140	0.835
150	0.832
200	0.820
250	0.812
300	0.806
400	0.796
500	0.783

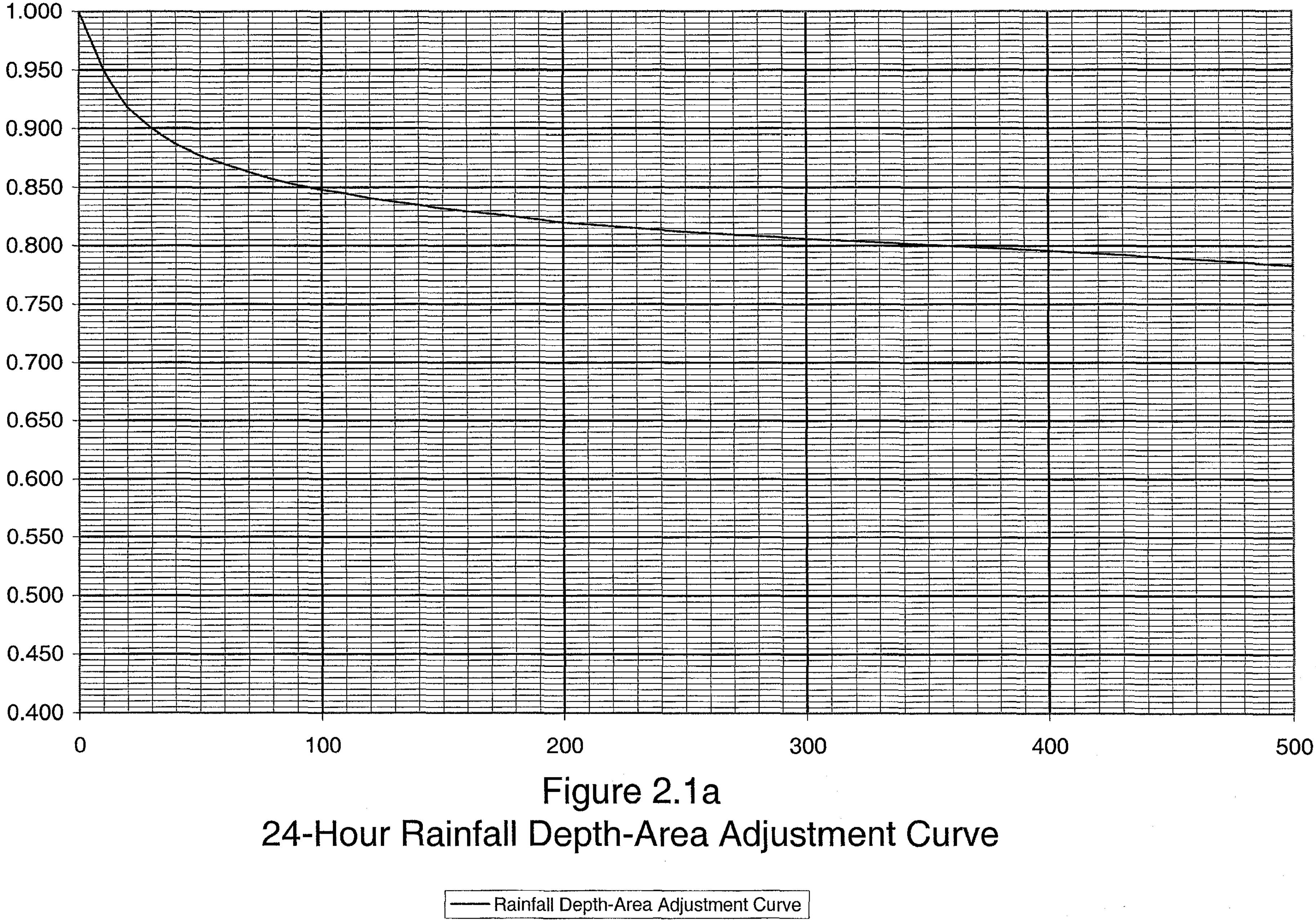
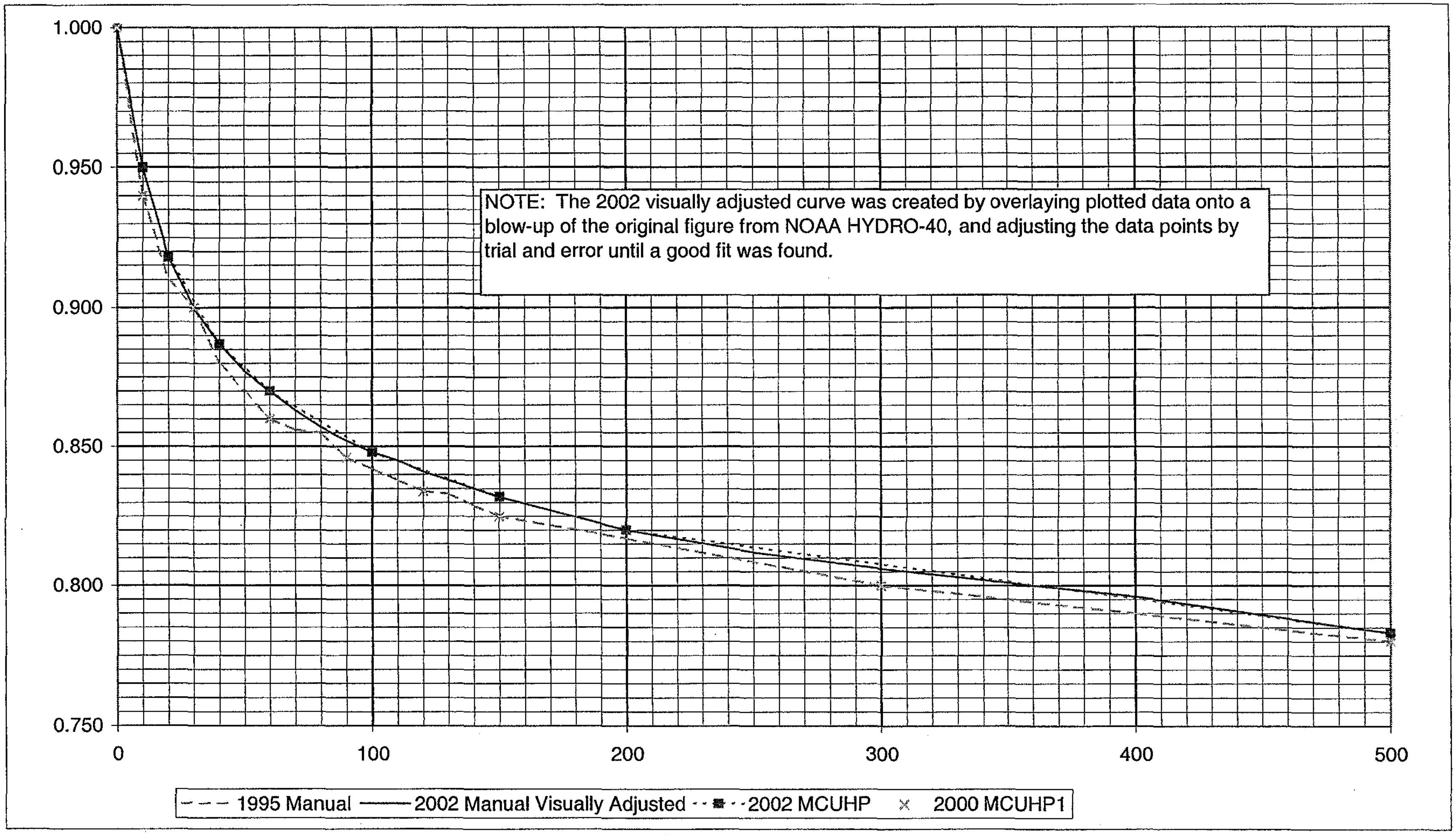


Figure 2.1a
24-Hour Rainfall Depth-Area Adjustment Curve

HYDRO-40 24-hour Depth-Area Ratios for Central Arizona

1995 Manual		2002 Manual		1995 Manual		2002 MCUHP	
Area, sm	Ratio	Area, sm	Ratio	Area, sm	Ratio	Area, sm	Ratio
0	1.000	0	1.000	0	1.000	0	1.000
10	0.940	10	0.950	10	0.940	10	0.950
20	0.910	20	0.918	30	0.900	20	0.918
30	0.900	30	0.900	60	0.860	40	0.887
40	0.880	40	0.887	90	0.846	60	0.870
50	0.870	50	0.877	120	0.834	100	0.848
60	0.860	60	0.870	150	0.825	150	0.832
70	0.856	70	0.863	300	0.800	200	0.820
80	0.855	80	0.857	500	0.780	500	0.783
90	0.846	90	0.852				
100	0.842	100	0.848				
110	0.838	110	0.845				
120	0.834	120	0.841				
130	0.833	130	0.838				
140	0.829	140	0.835				
150	0.825	150	0.832				
200	0.817	200	0.820				
300	0.800	250	0.812				
400	0.790	300	0.806				
500	0.780	400	0.796				
		500	0.783				



1.000
0.950
0.900
0.850
0.800
0.750
0
100
200
300
400
500
NOTE: The 2002 visually adjusted curve was created by overlaying plotted data onto a blow-up of the original figure from NOAA HYDRO-40, and adjusting the data points by trial and error until a good fit was found.
1995 Manual
2002 Manual Visually Adjusted
2002 MCUHP
2000 MCUHP1

MEMORANDUM

Date: March 10, 1993

From: Jorge R. Garré

To: Watershed Management Branch

Subject: Depth-Area Ratios for 24-hour Durantion Rainfall

Since most of you already know, these ratios are found in the *NOAA Technical Memorandum NWS HYDRO-40* and since the copy available in our branch is not the best, I took the initiative to find a better copy and try to generate a table of values. Having a table will diminish the discrepancies among different individuals, making the selection of the point rainfall reduction coefficient more consistent.

In order to be as accurate as posible, I used an engineering scale in centimeters (sorry for those offended). Where the curve between two consecutive points was *almost* linear, I decided not to generate intermediate values because one can just interpolate between the extreme values. The most noticeable change in the curve's slope is encountered for watershed areas ranging from 0-70 square miles, and the least is after 300 square miles.

Please, contact me if you have any questions or revisions.

JRGarré

Jorge R Garré
Hydrologist I

Depth-Area Reduction Factors for 24-Hour Duration Rainfall

Area [Mi^2]	Ratio to Point Rainfall
0	1
10	0.94
20	0.91
30	0.90
40	0.88
50	0.87
60	0.86
70	0.856
80	0.855
90	0.846
100	0.842
110	0.838
120	0.834
130	0.833
140	0.829
150	0.825
200	0.817
300	0.80
400	0.79
500	0.78

MEMORANDUM

Date: April 1, 1993

From: Jorge R. Garré

To: Watershed Management Branch

Subject: Depth-Area Ratios for 24-hour Rainfall Duration

This is in response to concerns raised by Amir Motamedi on March 26, 1993, with regard to my previous memorandum dated March 10th on the above-referenced subject.

The request was classified in two different items as follows:

1. Contact the National Oceanic and Atmospheric Administration (NOAA) to find out if there is any support data on the HYDRO-40.

2. Find out if there has been discrepancies in the *reduction factor's* value, selected by different consultants in previous hydrologic studies.

ANSWER 1:

On March 26th, I spoke with Mr. Marshall Hansen who is Chief of the Water Management Information Division at the National Weather Service Office in Silver Spring, Maryland. Mr. Hansen stated that the authors of the Technical Memorandum NWS HYDRO-40 are not longer working at the agency and that unfortunately there is no support data for this report. He volunteered to develop a tabular set of values from the curves, so that I could compare the results with mine. I also requested a copy from Mr. Hansen of the HYDRO-40 for WSBM's use since the reproduction we have available is a bit distorted.

ANSWER 2:

I reviewed some of the most accessible reports and found that each of them has a different reduction factor value selected for the 100-year, 24-hour event. Although they all are coming from the same data source, each individual's interpretation was different. This fact puts more weight on what I was mentioning before in my previous memorandum, that we need to generate a table of values to diminish the discrepancies among different individuals, therefore, making the selection of the reduction factor more consistent.

CONCLUSION:

After reviewing Mr. Hansen's values I found that they coincide with the values that I have developed. The WSMB can feel comfortable using these numbers, however, talking with Amir I noticed that Mr. Sabol is looking at the accuracy of the HYDRO-40 and evaluating whether it is outdated or not.

Enclosed please find Mr. Hansen's letter and keep it for future reference, so that anybody still using the HYDRO-40 will not question this support data.

Do not hesitate to contact me if you have any questions or revisions

J R Garré

Jorge R. Garré
Hydrologist I

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
NATIONAL WEATHER SERVICE
Silver Spring, Md. 20910

March 29, 1993 W/OH1:EMH

APR 1 1993

JRG

Mr. George Jarre
Maricopa County Flood
Control District
2801 West Durango Street
Phoenix, AZ 85009

Dear Mr. Jarre:

I have enclosed a copy of NOAA Technical Memorandum, NWS HYDRO #40 (1984), as you requested. As I discussed with you over the telephone, the background worksheets for this report are no longer available in this office and therefore I can offer no clarification for the relations presented in Figure 15 of the report.

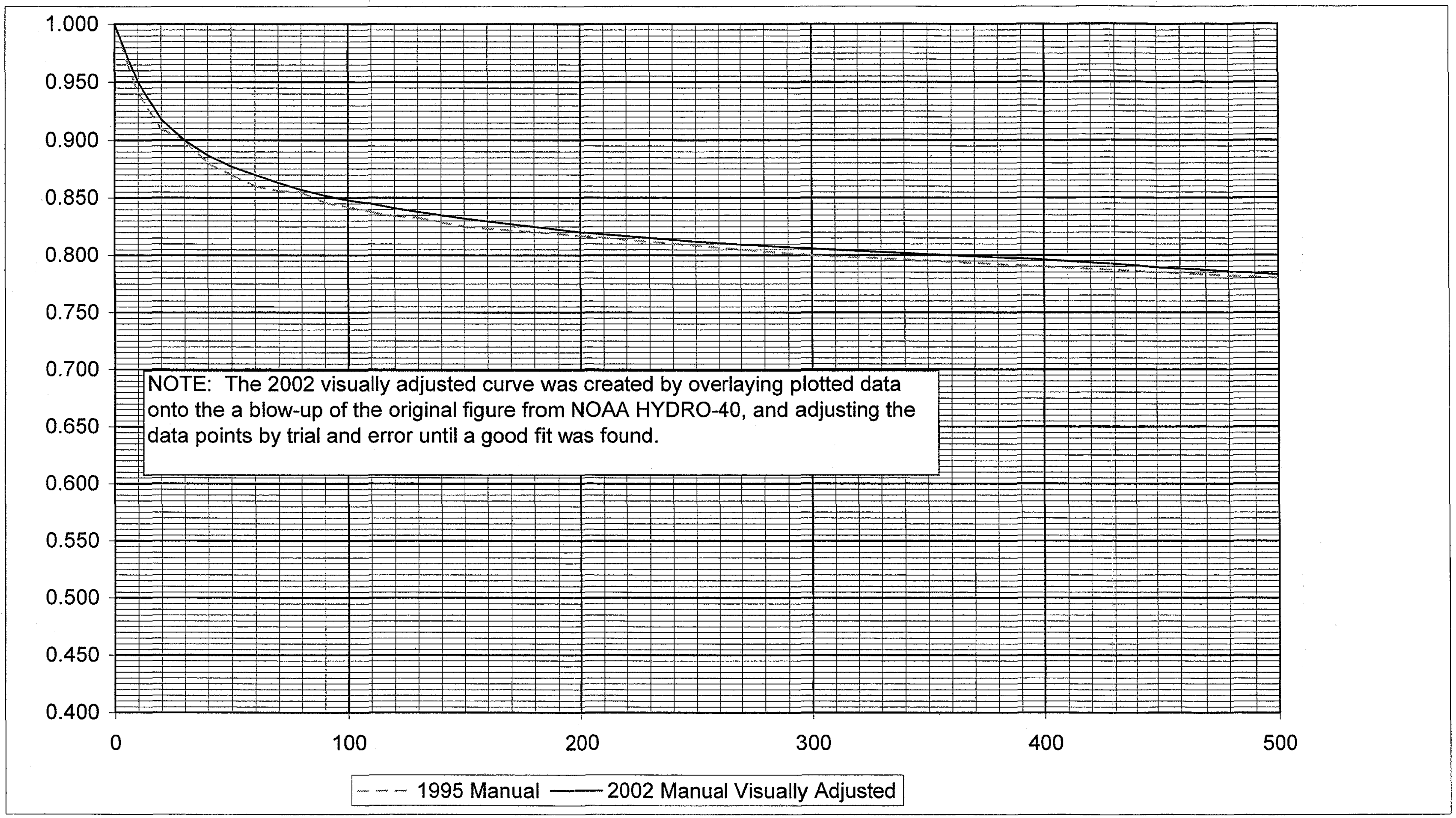
If it is of any help to you, I have gridded the figure in my report and read off information that may be used to reconstruct such a set of curves. As such, these gridded values are also enclosed and may represent an independent interpretation of the figure.

Please call me if you have further questions regarding this information.

Sincerely,

E. Marshall Hansen
Chief, Water Management
Information Division

Enclosures



1.000
0.950
0.900
0.850
0.800
0.750
0.700
0.650
0.600
0.550
0.500
0.450
0.400
0
100
200
300
400
500
NOTE: The 2002 visually adjusted curve was created by overlaying plotted data onto the a blow-up of the original figure from NOAA HYDRO-40, and adjusting the data points by trial and error until a good fit was found.
1995 Manual
2002 Manual Visually Adjusted

AREA (MI^2)

Figure 14.--$\bar{X}'_L$ (2.54-yr depth-area ratio, see sec. 4.3) for 3-, 6-, 12-, and 24-hr in southeast Arizona. Dashed lines are 3-hr and 24-hr Chicago $\bar{X}'_L$ (from TR 24)

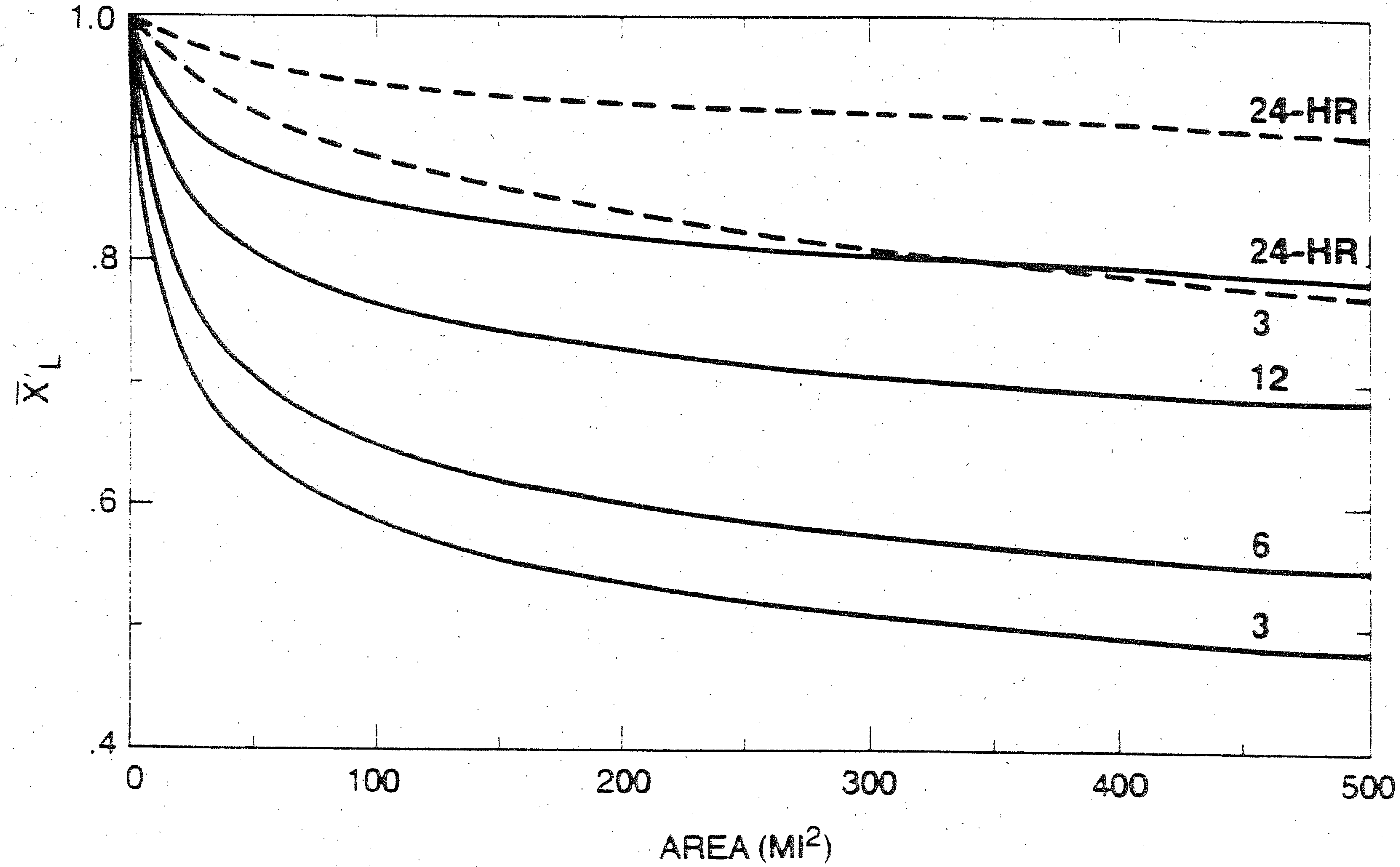


Figure 15.--Same as figure 14, but for central Arizona.

be attributed to a mixture of storm types, but still different from these found in the central Plains.

ATTACHMENT 2

Table 2.2: No Change

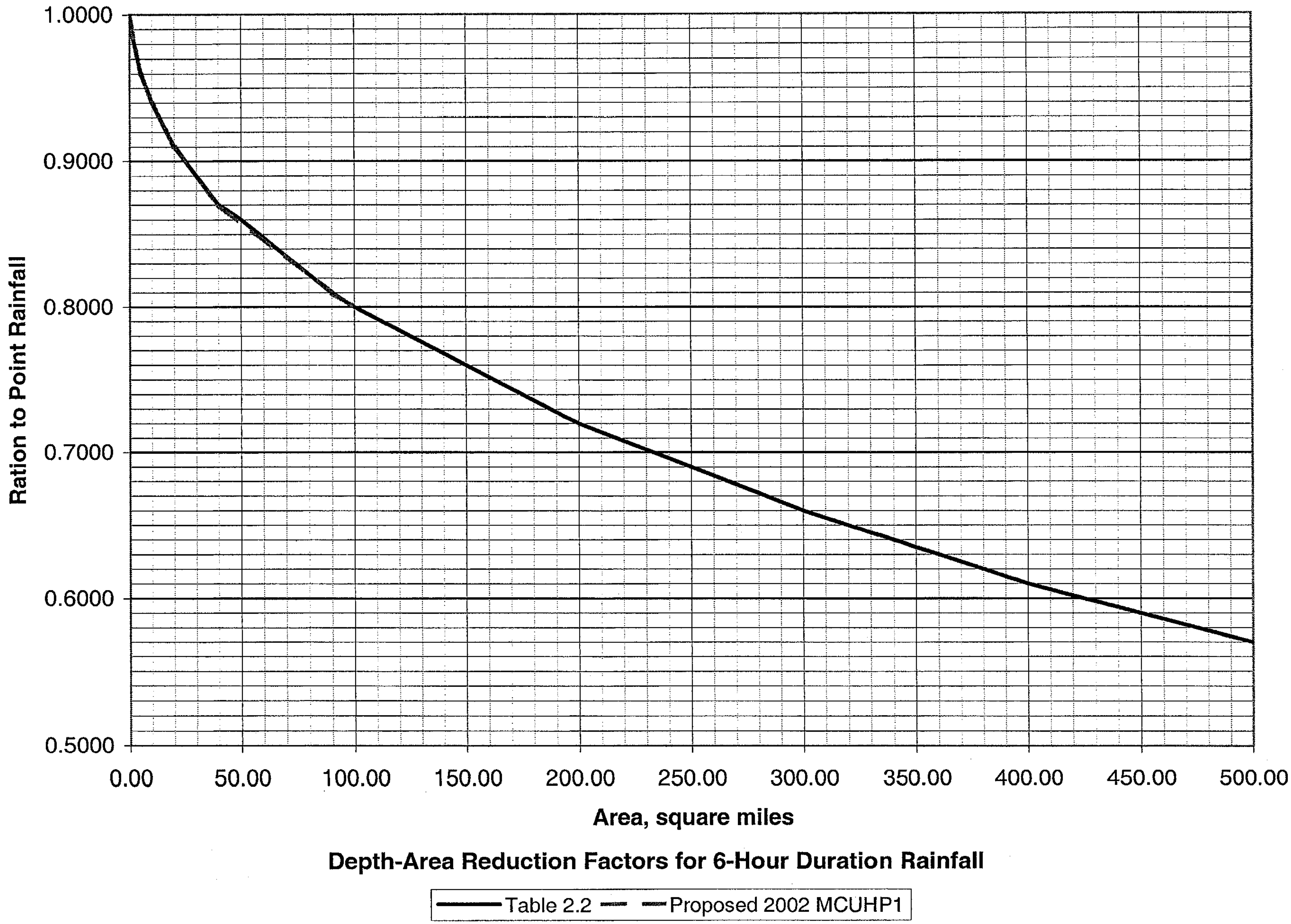
Point Values used in MCUHP1 and MCUHP2:

Table 2.2	
Area	**Factor**
0.00	1.0000
0.50	0.9935
1.00	0.9870
2.80	0.9750
5.00	0.9600
10.00	0.9400
16.00	0.9220
20.00	0.9100
30.00	0.8900
40.00	0.8700
50.00	0.8600
90.00	0.8100
100.00	0.8000
200.00	0.7200
300.00	0.6600
400.00	0.6100
500.00	0.5700

MCUHP1(2)	
Area	**Factor**
0.00	1.0000
2.80	0.9750
5.00	0.9600
10.00	0.9400
16.00	0.9220
20.00	0.9100
40.00	0.8700
90.00	0.8100
100.00	0.8000

MCUHP will only add JD records for up to 100 sm.

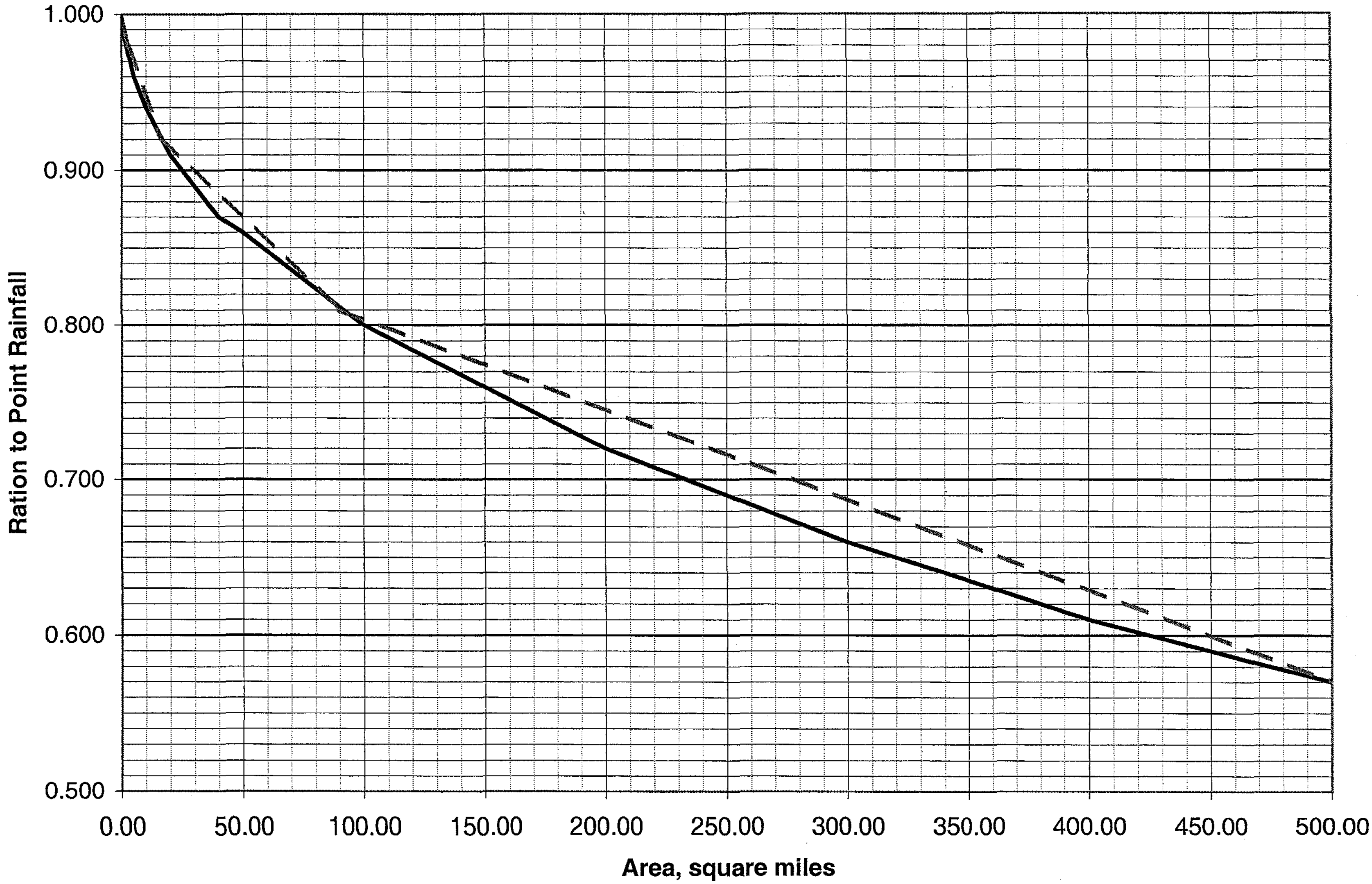
Pattern break point per Figure 2.17



Depth-Area Reduction Factors for 6-Hour Duration Rainfall

Table 2.2	
Area	Factor
0.00	1.000
1.00	0.987
5.00	0.960
10.00	0.940
20.00	0.910
30.00	0.890
40.00	0.870
50.00	0.860
100.00	0.800
200.00	0.720
300.00	0.660
400.00	0.610
500.00	0.570

2000 MCUHP1	
Area	Factor
0.00	1.0000
0.01	1.0000
0.50	0.9935
2.80	0.9800
16.00	0.922
90.00	0.810
500.00	0.570



Depth-Area Reduction Factors for 6-Hour Duration Rainfall

ATTACHMENT 3

Table 2.3
2-Hour Storm Distribution for Retention Design

Time (minutes)	% Rainfall Depth	Time (minutes)	% Rainfall Depth
0	0.00		
5	0.70	65	68.77
10	1.40	70	79.30
15	2.11	75	85.26
20	2.81	80	89.12
25	3.86	85	92.28
30	4.91	90	95.09
35	7.72	95	96.14
40	10.88	100	97.19
45	14.39	105	97.89
50	19.65	110	98.60
55	26.67	115	99.30
60	41.75	120	100.00

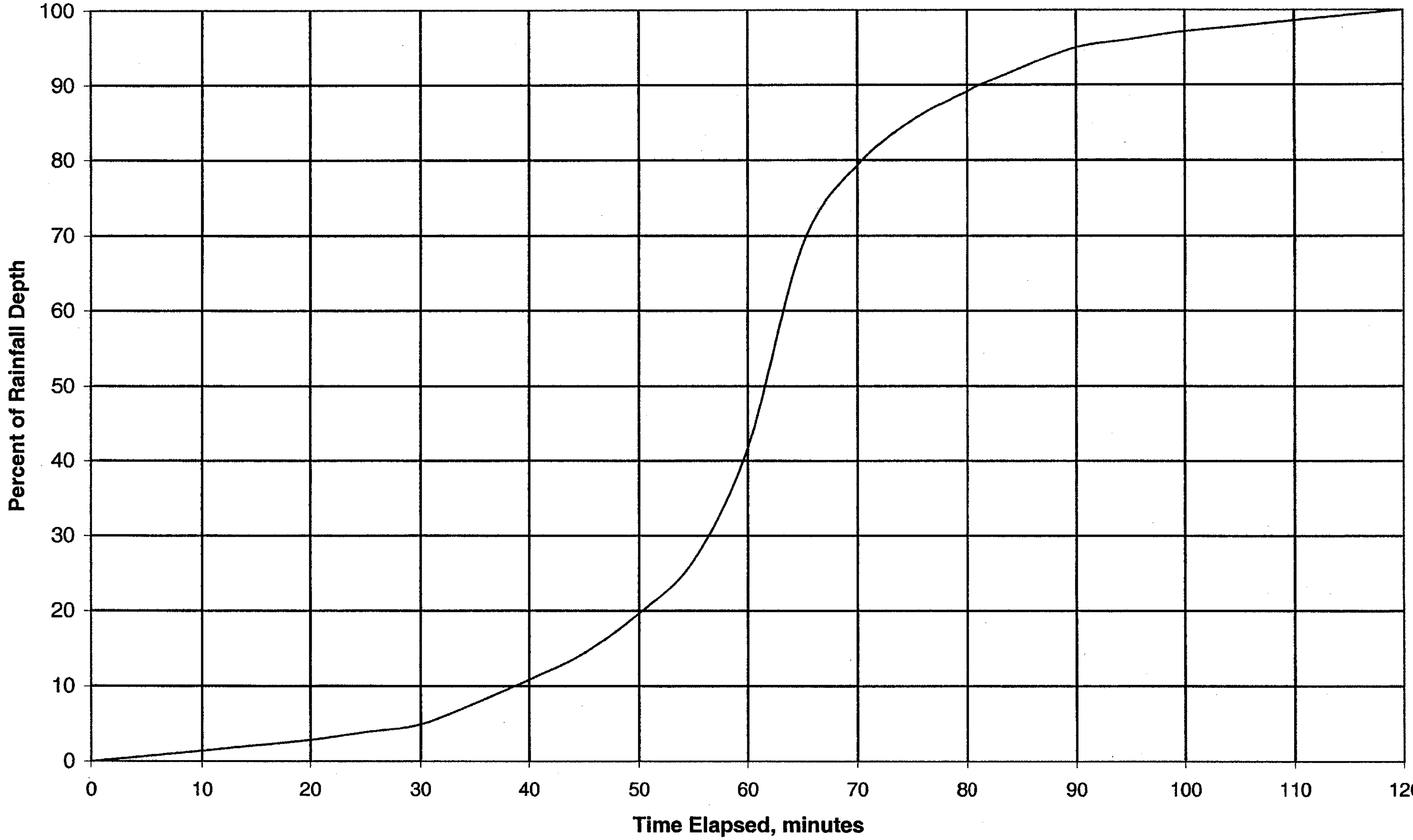
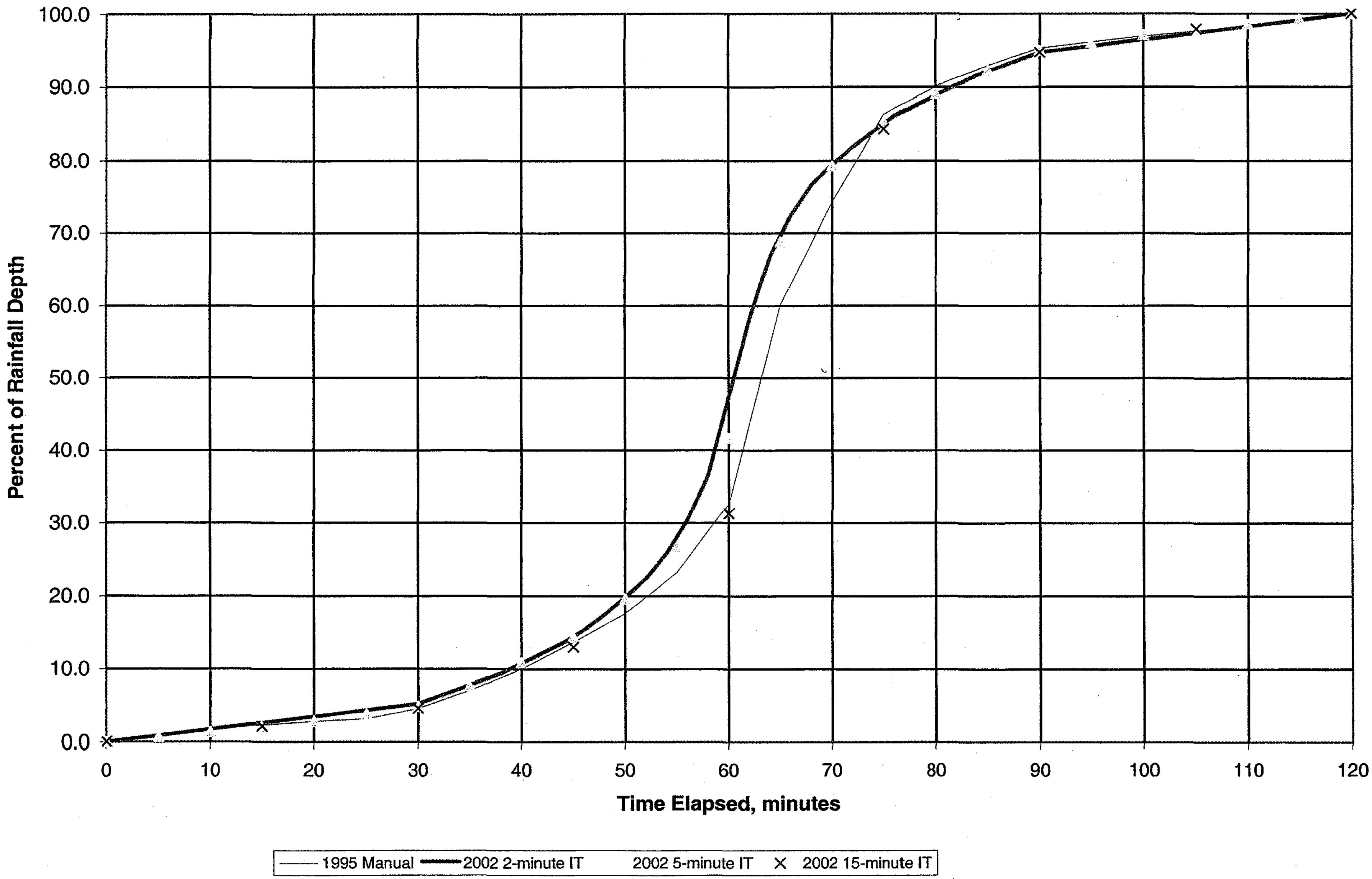


Figure 2.15
2-Hour Mass Curve for Retention Design

Figure 2.15
2-Hour Mass Curve for Retention Design



1995 Manual	
Time, min	Dimensionless
0	0.0
5	1.1
10	1.8
15	2.3
20	2.8
25	3.2
30	4.6
35	7.1
40	10.0
45	13.7
50	17.6
55	23.2
60	32.7
65	60.1
70	74.3
75	86.3
80	90.1
85	93.0
90	95.4
95	96.2
100	97.0
105	97.7
110	98.2
115	99.2
120	100.0

2-minute HEC-1 Main Time Interval			
	Rainfall		
Time, min	Incremental	Cumulative	Dimensionless
0	0	0.00	0.00
2	0.01	0.01	0.35
4	0.01	0.02	0.70
6	0.01	0.03	1.05
8	0.01	0.04	1.39
10	0.01	0.05	1.74
12	0.01	0.06	2.09
14	0.01	0.07	2.44
16	0.01	0.08	2.79
18	0.01	0.09	3.14
20	0.01	0.10	3.48
22	0.01	0.11	3.83
24	0.01	0.12	4.18
26	0.01	0.13	4.53
28	0.01	0.14	4.88
30	0.01	0.15	5.23
32	0.03	0.18	6.27
34	0.03	0.21	7.32
36	0.03	0.24	8.36
38	0.03	0.27	9.41
40	0.04	0.31	10.80
42	0.04	0.35	12.20
44	0.04	0.39	13.59
46	0.05	0.44	15.33
48	0.06	0.50	17.42
50	0.07	0.57	19.86
52	0.07	0.64	22.30
54	0.1	0.74	25.78
56	0.13	0.87	30.31
58	0.18	1.05	36.59
60	0.31	1.36	47.39
62	0.31	1.67	58.19
64	0.25	1.92	66.90
66	0.16	2.08	72.47
68	0.12	2.20	76.66
70	0.08	2.28	79.44
72	0.07	2.35	81.88
74	0.06	2.41	83.97
76	0.06	2.47	86.06
78	0.04	2.51	87.46
80	0.04	2.55	88.85
82	0.04	2.59	90.24
84	0.04	2.63	91.64
86	0.03	2.66	92.68
88	0.03	2.69	93.73
90	0.03	2.72	94.77
92	0.01	2.73	95.12
94	0.01	2.74	95.47
96	0.01	2.75	95.82
98	0.01	2.76	96.17
100	0.01	2.77	96.52
102	0.01	2.78	96.86
104	0.01	2.79	97.21
106	0.01	2.80	97.56
108	0.01	2.81	97.91
110	0.01	2.82	98.26
112	0.01	2.83	98.61
114	0.01	2.84	98.95
116	0.01	2.85	99.30
118	0.01	2.86	99.65
120	0.01	2.87	100.00

Sky Harbor Airport Gage Location:

112	1	38.0832	**112.64530**
33	26	9.46399	**33.16036**

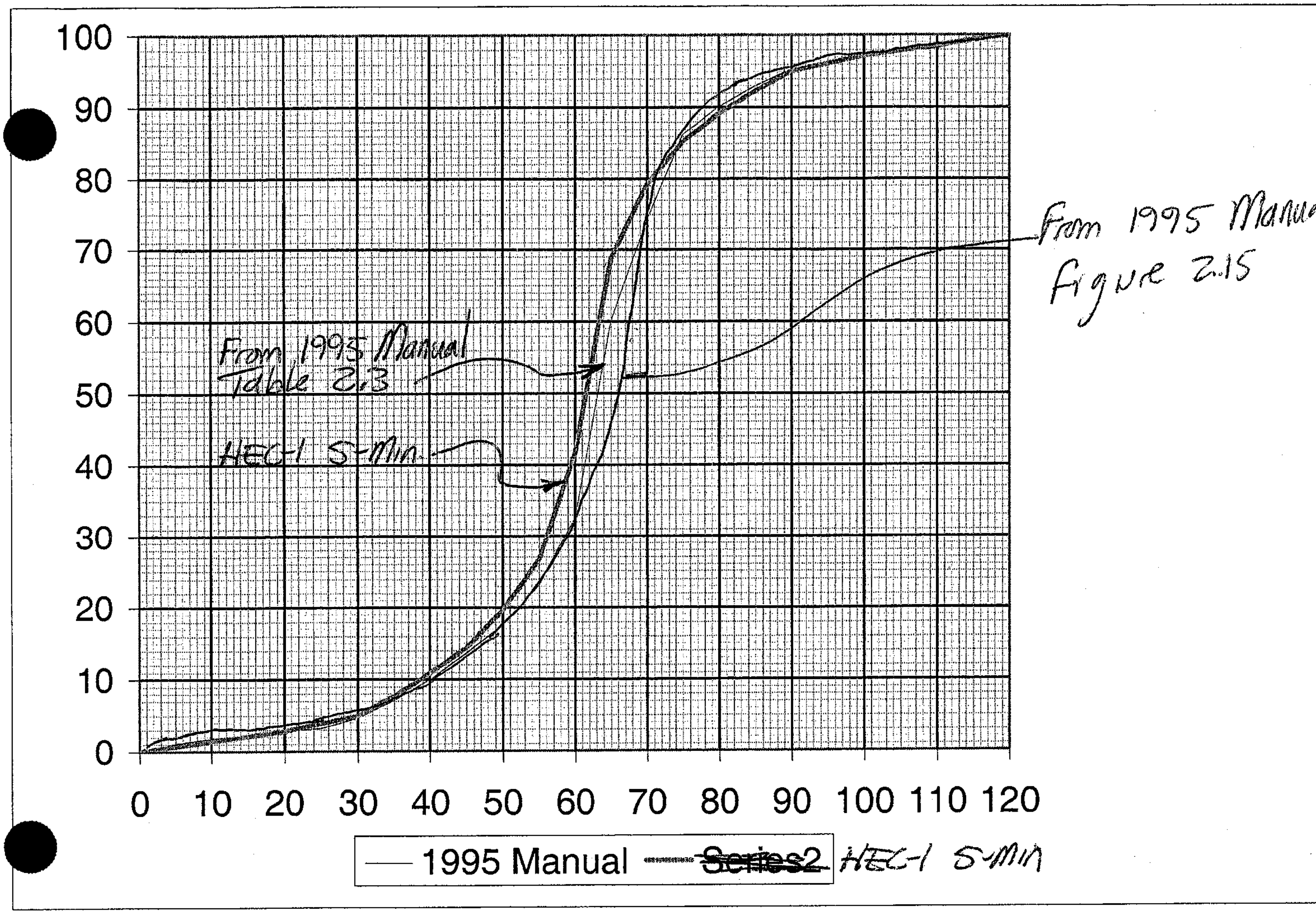
Prefre Input Data from WEB Site:

http://www.nws.noaa.gov/oh/hdsc/noaaatlas2.htm

2-yr 6-hr	1.17
2-yr 24-hr	1.39
100-yr 6-hr	3.39
100-yr 24-hr	4.15
2-yr 1-hr	0.9167
100-yr 1-hr	2.58473
2-yr 2-hr	1.00308
100-yr 2-hr	2.85933

5-minute HEC-1 Main Time Interval			
	Rainfall		
Time, min	Incremental	Cumulative	Dimensionless
0	0	0	0
5	0.02	0.02	0.70
10	0.02	0.04	1.40
15	0.02	0.06	2.11
20	0.02	0.08	2.81
25	0.03	0.11	3.86
30	0.03	0.14	4.91
35	0.08	0.22	7.72
40	0.09	0.31	10.88
45	0.1	0.41	14.39
50	0.15	0.56	19.65
55	0.2	0.76	26.67
60	0.43	1.19	41.75
65	0.77	1.96	68.77
70	0.3	2.26	79.30
75	0.17	2.43	85.26
80	0.11	2.54	89.12
85	0.09	2.63	92.28
90	0.08	2.71	95.09
95	0.03	2.74	96.14
100	0.03	2.77	97.19
105	0.02	2.79	97.89
110	0.02	2.81	98.60
115	0.02	2.83	99.30
120	0.02	2.85	100.00

15-minute HEC-1 Main Time Interval			
	Rainfall		
Time, min	Incremental	Cumulative	Dimensionless
0	0	0	0
15	0.06	0.06	2.11
30	0.07	0.13	4.56
45	0.24	0.37	12.98
60	0.52	0.89	31.23
75	1.51	2.4	84.21
90	0.3	2.7	94.74
105	0.09	2.79	97.89
120	0.06	2.85	100.00



100
90
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
80
90
100
110
120
From 1995 Manual Table 2.3
HEC-1 5-Min
From 1995 Manual Figure 2.15
1995 Manual
~~Series2~~ HEC-1 5-Min

2. Calculate the point rainfall depth, or the areally-averaged point rainfall depth, from Figures 2.2 through 2.7 depending on the desired rainfall frequency.

3. Use either Figure 2.14 or Table 2.2 to determine the depth-area reduction factor.

4. Multiply the point rainfall depth by the appropriate depth-area reduction factor. This is the equivalent uniform depth of rainfall that is to be applied to the entire watershed.

2.4 Design Storm Distributions

According to Table 2.1, three types of design storm distributions are to be used in Maricopa County. This Manual contains information for two of those design storm distributions; the 2-hour storm for the design of retention/detention basins, and the 6-hour local storm. Information for the SCS Type II 24-hour storm has been encoded in the MCUHP programs. Otherwise data regarding the SCS 24-hour storm is generally available elsewhere. Distributions for other general storms for larger watersheds will need to be developed on a case-by-case basis based on appropriate meteorologic and hydrologic factors.

2.4.1 2-hour Storm Distribution

The 2-hour storm distribution is to be used for the design of retention/detention basins (see Table 2.1). The 2-hour distribution shown in Figure 2.15 and Table 2.3 is a dimensionless form of the 2-hour hypothetical distribution for the Phoenix Sky Harbor Airport location. This distribution can be applied throughout Maricopa County for the design of retention/detention facilities.

Table 2.3
2-Hour Storm Distribution for Retention Design

Time (minutes)	% Rainfall Depth	Time (minutes)	% Rainfall Depth
0	0.0		
5	1.1	65	60.1
10	1.8	70	74.3
15	2.3	75	86.3
20	2.8	80	90.1
25	3.2	85	93.0
30	4.6	90	95.4
35	7.1	95	96.2
40	10.0	100	97.0
45	13.7	105	97.7
50	17.6	110	98.2
55	23.2	115	99.2
60	32.7	120	100.0

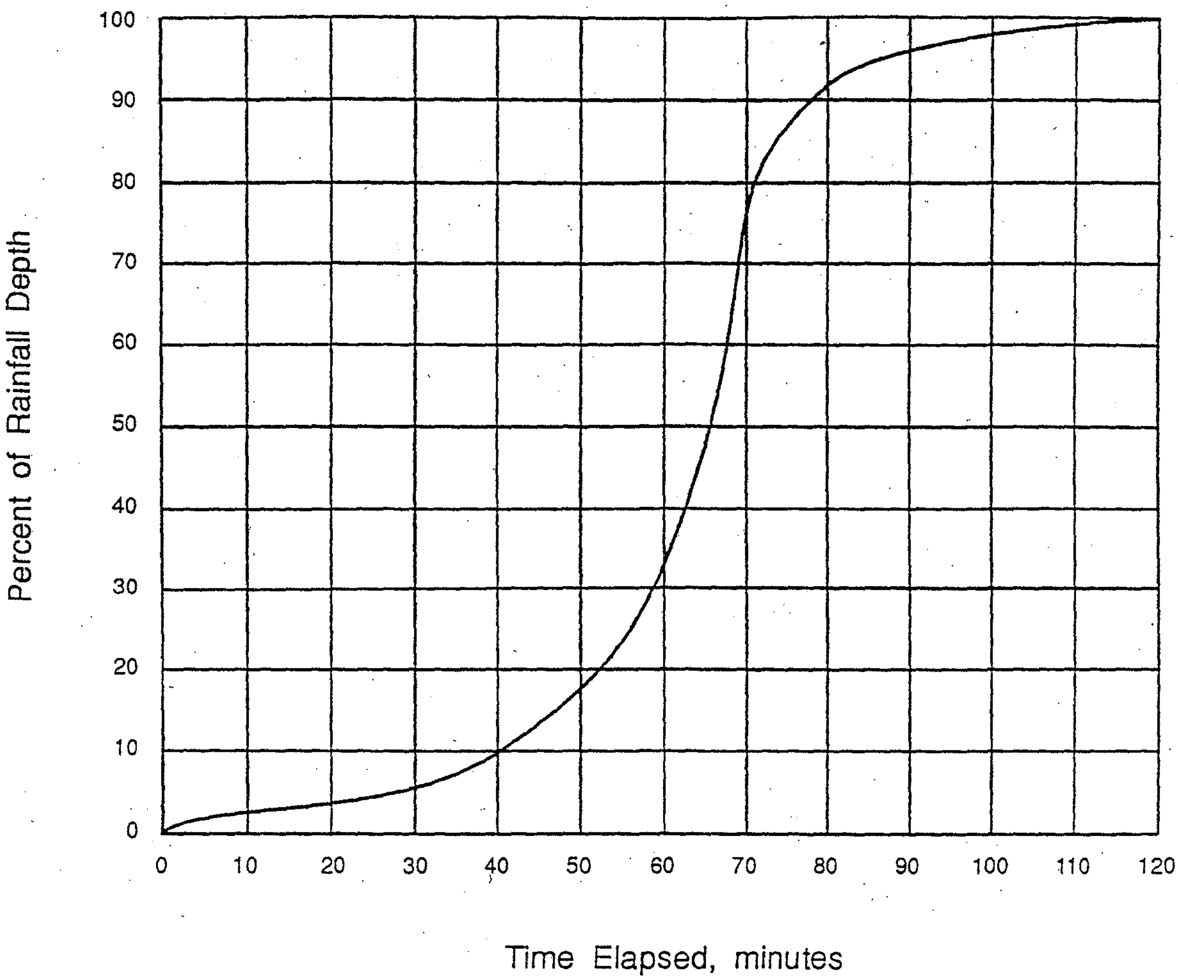


Figure 2.15
2-Hour Mass Curve for Retention Design

2.4.2 6-hour Storm Distribution

The 6-hour storm distributions are used for flood studies in Maricopa County of drainage areas less than 20 square miles, except for on-site retention/detention facilities (see Table 2.1). These distributions would also be used for drainage areas larger than 20 square miles and smaller than 100 square miles by critically centering the storm over all or portions of the drainage area to estimate the peak flood discharges that could be realized on such watersheds due to the occurrence of a local storm over the watershed.

The Maricopa County 6-hour local storm distributions consist of five dimensionless storm patterns. Pattern No. 1 represents the rainfall intensities that can be expected in the "eye" of a local storm. These high, short-duration rainfall intensities would only occur over a relatively small area near the center of the storm cell. Pattern No. 1 is an offset, dimensionless form of the hypothetical distribution derived from rainfall statistics found in *NOAA Atlas for the Western United States, Arizona* (Miller and others, 1973)

*** O U T P U T   D A T A ***

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR Sky Harbor Airport
PRIMARY ZONE NUMBER= 7
SHORT-DURATION ZONE NUMBER= 8

LATITUDE 33.16N LONGITUDE 112.65W

POINT VALUES

	RETURN PERIOD							
DURATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.31	.43	.50	.61	.69	.78	.97	5-MIN
10-MIN	.47	.65	.77	.93	1.06	1.19	1.48	10-MIN
15-MIN	.57	.81	.97	1.19	1.36	1.52	1.91	15-MIN
30-MIN	.75	1.08	1.30	1.60	1.84	2.07	2.60	30-MIN
1-HR	.92	1.34	1.62	2.00	2.29	2.58	3.26	1-HR
2-HR	1.00	1.47	1.78	2.21	2.53	2.86	3.61	2-HR
3-HR	1.06	1.56	1.89	2.35	2.70	3.04	3.84	3-HR
6-HR	1.17	1.73	2.10	2.61	3.00	3.39	4.29	6-HR
12-HR	1.28	1.91	2.33	2.90	3.34	3.77	4.78	12-HR
24-HR	1.39	2.09	2.55	3.18	3.67	4.15	5.26	24-HR

* IF YOUR SITE IS IN ARIZONA OR NEW MEXICO, PLEASE CONSULT THE FOLLOWING PAPER FOR REVISED DEPTH-AREA VALUES:
DEPTH-AREA RATIOS IN THE SEMI-ARID SOUTHWEST UNITED STATES
NOAA TECHNICAL MEMORANDUM NWS HYDRO-40
ZEHR AND MYERS
AUGUST 1984

INPUT DATA

PROJECT NAME=Sky Harbor Airport
ZONE= 7 SHORT-DURATION ZONE= 8
LATITUDE= 33.16 LONGITUDE= 112.65 ELEVATION= 0
2-YR, 6-HR PCPN= 1.17 100-YR, 6-HR PCPN= 3.39
2-YR, 24-HR PCPN= 1.39 100-YR, 24-HR PCPN= 4.15

* * * * E N D   O F   R U N * * * *

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1***************************************                          ***************************************
 *                                     *                          *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1) *                          *   U.S. ARMY CORPS OF ENGINEERS      *
 *           SEPTEMBER 1990            *                          *   HYDROLOGIC ENGINEERING CENTER     *
 *             VERSION 4.0             *                          *        609 SECOND STREET            *
 *                                     *                          *     DAVIS, CALIFORNIA 95616         *
 * RUN DATE  12/28/2001  TIME  09:09:45 *                         *        (916) 756-1104               *
 *                                     *                          *                                     *
***************************************                          ***************************************


                    X     X  XXXXXXX   XXXXX         X
                    X     X  X        X     X       XX
                    X     X  X        X              X
                    XXXXXXX  XXXX     X       XXXXX  X
                    X     X  X        X              X
                    X     X  X        X     X        X
                    X     X  XXXXXXX   XXXXX        XXX


 THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

 THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
 THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
 NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
 DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
 KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


1                                          HEC-1 INPUT                                          PAGE  1

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

             1          ID   Sky Harbor Airport
             2          ID   100-yr 2-hour Storm using dummy sub-basin input data
             3          IT       2                     300
             4          IO       1

             5          KK   SKYH2.IH1
             6          KM   SUB-BASIN  SKY
             7          KM   THIS IS A DUMMY MODEL FOR A 100-YR 2-HR STORM AT SKY HARBOR AIRPORT
             8          KM   2-Minute Main Time Interval
             9          KM
            10          KM   Model used to calculate a 100-yr 2-hr Rainfall Distribution using
            11          KM   the hypothetical distribution.
            12          KM
            13          KM   THIS BASIN USED RAINFALL REDUCTION FACTOR OF 1.000
            14          BA   1.000
            15          PH               0    0.78    1.52    2.58    2.86
            16          LG    .250    .340   5.000    .200    .000
            17          UC   1.067    .397
            18          UA       0       5      16      30      65      77      84      90      94      97
            19          UA     100
            20          ZZ
1***************************************                          ***************************************
 *                                     *                          *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1) *                          *   U.S. ARMY CORPS OF ENGINEERS      *
 *           SEPTEMBER 1990            *                          *   HYDROLOGIC ENGINEERING CENTER     *
 *             VERSION 4.0             *                          *        609 SECOND STREET            *
 *                                     *                          *     DAVIS, CALIFORNIA 95616         *
 * RUN DATE  12/28/2001  TIME  09:09:45 *                         *        (916) 756-1104               *
 *                                     *                          *                                     *
***************************************                          ***************************************


                    Sky Harbor Airport
                    100-yr 2-hour Storm using dummy sub-basin input data

    4 IO         OUTPUT CONTROL VARIABLES
                       IPRNT          1  PRINT CONTROL
                       IPLOT          0  PLOT CONTROL
                       QSCAL         0.  HYDROGRAPH PLOT SCALE

      IT         HYDROGRAPH TIME DATA
                        NMIN          2  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1   0  STARTING DATE
                       ITIME       0000  STARTING TIME
                          NQ        300  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1   0  ENDING DATE
                      NDTIME       0958  ENDING TIME
                       ICENT         19  CENTURY MARK

                 COMPUTATION INTERVAL    .03 HOURS
                      TOTAL TIME BASE   9.97 HOURS

          ENGLISH UNITS
               DRAINAGE AREA         SQUARE MILES
```

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

*** ***

```
           **************
           *            *
 5 KK      *   SKYH2    *     .IH1
           *            *
           **************
                  SUB-BASIN  SKY
                  THIS IS A DUMMY MODEL FOR A 100-YR 2-HR STORM AT SKY HARBOR AIRPORT
                  2-Minute Main Time Interval

                  Model used to calculate a 100-yr 2-hr Rainfall Distribution using
                  the hypothetical distribution.

                  THIS BASIN USED RAINFALL REDUCTION FACTOR OF 1.000

            SUBBASIN RUNOFF DATA

14 BA        SUBBASIN CHARACTERISTICS
                TAREA        1.00  SUBBASIN AREA

             PRECIPITATION DATA

15 PH                          DEPTHS FOR   0-PERCENT HYPOTHETICAL STORM
             ..... HYDRO-35 ......   ............... TP-40 ...............   ........... TP-49 ...........
             5-MIN  15-MIN  60-MIN    2-HR    3-HR    6-HR   12-HR   24-HR   2-DAY   4-DAY   7-DAY  10-DAY
               .78    1.52    2.58    2.86     .00     .00     .00     .00     .00     .00     .00     .00

                                              STORM AREA =     1.00

16 LG        GREEN AND AMPT LOSS RATE
                STRTL        .25  STARTING LOSS
                  DTH        .34  MOISTURE DEFICIT
                 PSIF       5.00  WETTING FRONT SUCTION
                XKSAT        .20  HYDRAULIC CONDUCTIVITY
                RTIMP        .00  PERCENT IMPERVIOUS AREA

17 UC        CLARK UNITGRAPH
                   TC       1.07  TIME OF CONCENTRATION
                    R        .40  STORAGE COEFFICIENT

18 UA        ACCUMULATED-AREA VS. TIME,  11 ORDINATES
                   .0       5.0      16.0      30.0      65.0      77.0      84.0      90.0      94.0      97.0
                100.0

                                                        ***

                                              UNIT HYDROGRAPH PARAMETERS
                                        CLARK   TC=  1.07 HR,       R=   .40 HR
                                       SNYDER   TP=   .48 HR,      CP=   .56

                                                  UNIT HYDROGRAPH
                                             78 END-OF-PERIOD ORDINATES
          12.       36.       57.       89.      132.      175.      219.      267.      313.      377.
         486.      617.      727.      772.      768.      765.      750.      723.      699.      675.
         650.      627.      603.      576.      549.      523.      497.      472.      448.      427.
         407.      389.      365.      336.      309.      284.      261.      240.      221.      203.
         186.      171.      158.      145.      133.      122.      113.      104.       95.       88.
          80.       74.       68.       63.       58.       53.       49.       45.       41.       38.
          35.       32.       29.       27.       25.       23.       21.       19.       18.       16.
          15.       14.       13.       12.       11.       10.        9.        8.
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****

HYDROGRAPH AT STATION   SKYH2

****

DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1		0000	1	.00	.00	.00	0.	*	1		0500	151	.00	.00	.00	0.
1		0002	2	.01	.01	.00	0.	*	1		0502	152	.00	.00	.00	0.
1		0004	3	.01	.01	.00	0.	*	1		0504	153	.00	.00	.00	0.
1		0006	4	.01	.01	.00	0.	*	1		0506	154	.00	.00	.00	0.
1		0008	5	.01	.01	.00	0.	*	1		0508	155	.00	.00	.00	0.
1		0010	6	.01	.01	.00	0.	*	1		0510	156	.00	.00	.00	0.
1		0012	7	.01	.01	.00	0.	*	1		0512	157	.00	.00	.00	0.
1		0014	8	.01	.01	.00	0.	*	1		0514	158	.00	.00	.00	0.
1		0016	9	.01	.01	.00	0.	*	1		0516	159	.00	.00	.00	0.
1		0018	10	.01	.01	.00	0.	*	1		0518	160	.00	.00	.00	0.
1		0020	11	.01	.01	.00	0.	*	1		0520	161	.00	.00	.00	0.
1		0022	12	.01	.01	.00	0.	*	1		0522	162	.00	.00	.00	0.
1		0024	13	.01	.01	.00	0.	*	1		0524	163	.00	.00	.00	0.
1		0026	14	.01	.01	.00	0.	*	1		0526	164	.00	.00	.00	0.
1		0028	15	.01	.01	.00	0.	*	1		0528	165	.00	.00	.00	0.

1	0030	16	.01	.01	.00	0.	*	1	0530	166	.00	.00	.00	0.
1	0032	17	.03	.03	.00	0.	*	1	0532	167	.00	.00	.00	0.
1	0034	18	.03	.03	.00	0.	*	1	0534	168	.00	.00	.00	0.
1	0036	19	.03	.03	.00	0.	*	1	0536	169	.00	.00	.00	0.
1	0038	20	.03	.03	.00	0.	*	1	0538	170	.00	.00	.00	0.
1	0040	21	.04	.04	.00	0.	*	1	0540	171	.00	.00	.00	0.
1	0042	22	.04	.04	.00	0.	*	1	0542	172	.00	.00	.00	0.
1	0044	23	.04	.04	.00	0.	*	1	0544	173	.00	.00	.00	0.
1	0046	24	.05	.05	.00	0.	*	1	0546	174	.00	.00	.00	0.
1	0048	25	.06	.06	.00	0.	*	1	0548	175	.00	.00	.00	0.
1	0050	26	.07	.05	.02	0.	*	1	0550	176	.00	.00	.00	0.
1	0052	27	.07	.04	.03	1.	*	1	0552	177	.00	.00	.00	0.
1	0054	28	.10	.04	.06	3.	*	1	0554	178	.00	.00	.00	0.
1	0056	29	.13	.04	.09	7.	*	1	0556	179	.00	.00	.00	0.
1	0058	30	.18	.03	.14	14.	*	1	0558	180	.00	.00	.00	0.
1	0100	31	.31	.03	.28	26.	*	1	0600	181	.00	.00	.00	0.
1	0102	32	.31	.03	.28	47.	*	1	0602	182	.00	.00	.00	0.
1	0104	33	.25	.03	.22	76.	*	1	0604	183	.00	.00	.00	0.
1	0106	34	.16	.03	.13	112.	*	1	0606	184	.00	.00	.00	0.
1	0108	35	.12	.03	.09	158.	*	1	0608	185	.00	.00	.00	0.
1	0110	36	.08	.03	.05	212.	*	1	0610	186	.00	.00	.00	0.
1	0112	37	.07	.02	.05	275.	*	1	0612	187	.00	.00	.00	0.
1	0114	38	.06	.02	.04	346.	*	1	0614	188	.00	.00	.00	0.
1	0116	39	.06	.02	.03	427.	*	1	0616	189	.00	.00	.00	0.
1	0118	40	.04	.02	.02	519.	*	1	0618	190	.00	.00	.00	0.
1	0120	41	.04	.02	.02	626.	*	1	0620	191	.00	.00	.00	0.
1	0122	42	.04	.02	.02	745.	*	1	0622	192	.00	.00	.00	0.
1	0124	43	.04	.02	.01	864.	*	1	0624	193	.00	.00	.00	0.
1	0126	44	.03	.02	.01	964.	*	1	0626	194	.00	.00	.00	0.
1	0128	45	.03	.02	.01	1035.	*	1	0628	195	.00	.00	.00	0.
1	0130	46	.03	.02	.01	1078.	*	1	0630	196	.00	.00	.00	0.
1	0132	47	.01	.01	.00	1099.	*	1	0632	197	.00	.00	.00	0.
1	0134	48	.01	.01	.00	1105.	*	1	0634	198	.00	.00	.00	0.
1	0136	49	.01	.01	.00	1099.	*	1	0636	199	.00	.00	.00	0.
1	0138	50	.01	.01	.00	1086.	*	1	0638	200	.00	.00	.00	0.
1	0140	51	.01	.01	.00	1067.	*	1	0640	201	.00	.00	.00	0.
1	0142	52	.01	.01	.00	1043.	*	1	0642	202	.00	.00	.00	0.
1	0144	53	.01	.01	.00	1015.	*	1	0644	203	.00	.00	.00	0.
1	0146	54	.01	.01	.00	985.	*	1	0646	204	.00	.00	.00	0.
1	0148	55	.01	.01	.00	952.	*	1	0648	205	.00	.00	.00	0.
1	0150	56	.01	.01	.00	917.	*	1	0650	206	.00	.00	.00	0.
1	0152	57	.01	.01	.00	881.	*	1	0652	207	.00	.00	.00	0.
1	0154	58	.01	.01	.00	843.	*	1	0654	208	.00	.00	.00	0.
1	0156	59	.01	.01	.00	805.	*	1	0656	209	.00	.00	.00	0.
1	0158	60	.01	.01	.00	767.	*	1	0658	210	.00	.00	.00	0.
1	0200	61	.01	.01	.00	730.	*	1	0700	211	.00	.00	.00	0.
1	0202	62	.00	.00	.00	694.	*	1	0702	212	.00	.00	.00	0.
1	0204	63	.00	.00	.00	656.	*	1	0704	213	.00	.00	.00	0.
1	0206	64	.00	.00	.00	617.	*	1	0706	214	.00	.00	.00	0.
1	0208	65	.00	.00	.00	578.	*	1	0708	215	.00	.00	.00	0.
1	0210	66	.00	.00	.00	538.	*	1	0710	216	.00	.00	.00	0.
1	0212	67	.00	.00	.00	500.	*	1	0712	217	.00	.00	.00	0.
1	0214	68	.00	.00	.00	464.	*	1	0714	218	.00	.00	.00	0.
1	0216	69	.00	.00	.00	429.	*	1	0716	219	.00	.00	.00	0.
1	0218	70	.00	.00	.00	397.	*	1	0718	220	.00	.00	.00	0.
1	0220	71	.00	.00	.00	367.	*	1	0720	221	.00	.00	.00	0.
1	0222	72	.00	.00	.00	339.	*	1	0722	222	.00	.00	.00	0.
1	0224	73	.00	.00	.00	312.	*	1	0724	223	.00	.00	.00	0.
1	0226	74	.00	.00	.00	288.	*	1	0726	224	.00	.00	.00	0.
1	0228	75	.00	.00	.00	265.	*	1	0728	225	.00	.00	.00	0.
1	0230	76	.00	.00	.00	244.	*	1	0730	226	.00	.00	.00	0.
1	0232	77	.00	.00	.00	225.	*	1	0732	227	.00	.00	.00	0.
1	0234	78	.00	.00	.00	207.	*	1	0734	228	.00	.00	.00	0.
1	0236	79	.00	.00	.00	190.	*	1	0736	229	.00	.00	.00	0.
1	0238	80	.00	.00	.00	175.	*	1	0738	230	.00	.00	.00	0.
1	0240	81	.00	.00	.00	161.	*	1	0740	231	.00	.00	.00	0.
1	0242	82	.00	.00	.00	148.	*	1	0742	232	.00	.00	.00	0.
1	0244	83	.00	.00	.00	136.	*	1	0744	233	.00	.00	.00	0.
1	0246	84	.00	.00	.00	125.	*	1	0746	234	.00	.00	.00	0.
1	0248	85	.00	.00	.00	115.	*	1	0748	235	.00	.00	.00	0.
1	0250	86	.00	.00	.00	106.	*	1	0750	236	.00	.00	.00	0.
1	0252	87	.00	.00	.00	97.	*	1	0752	237	.00	.00	.00	0.
1	0254	88	.00	.00	.00	89.	*	1	0754	238	.00	.00	.00	0.
1	0256	89	.00	.00	.00	82.	*	1	0756	239	.00	.00	.00	0.
1	0258	90	.00	.00	.00	76.	*	1	0758	240	.00	.00	.00	0.
1	0300	91	.00	.00	.00	69.	*	1	0800	241	.00	.00	.00	0.
1	0302	92	.00	.00	.00	64.	*	1	0802	242	.00	.00	.00	0.
1	0304	93	.00	.00	.00	59.	*	1	0804	243	.00	.00	.00	0.
1	0306	94	.00	.00	.00	54.	*	1	0806	244	.00	.00	.00	0.
1	0308	95	.00	.00	.00	50.	*	1	0808	245	.00	.00	.00	0.
1	0310	96	.00	.00	.00	46.	*	1	0810	246	.00	.00	.00	0.
1	0312	97	.00	.00	.00	42.	*	1	0812	247	.00	.00	.00	0.
1	0314	98	.00	.00	.00	39.	*	1	0814	248	.00	.00	.00	0.
1	0316	99	.00	.00	.00	35.	*	1	0816	249	.00	.00	.00	0.
1	0318	100	.00	.00	.00	33.	*	1	0818	250	.00	.00	.00	0.
1	0320	101	.00	.00	.00	30.	*	1	0820	251	.00	.00	.00	0.
1	0322	102	.00	.00	.00	28.	*	1	0822	252	.00	.00	.00	0.
1	0324	103	.00	.00	.00	25.	*	1	0824	253	.00	.00	.00	0.
1	0326	104	.00	.00	.00	23.	*	1	0826	254	.00	.00	.00	0.
1	0328	105	.00	.00	.00	21.	*	1	0828	255	.00	.00	.00	0.
1	0330	106	.00	.00	.00	19.	*	1	0830	256	.00	.00	.00	0.
1	0332	107	.00	.00	.00	17.	*	1	0832	257	.00	.00	.00	0.
1	0334	108	.00	.00	.00	14.	*	1	0834	258	.00	.00	.00	0.
1	0336	109	.00	.00	.00	11.	*	1	0836	259	.00	.00	.00	0.
1	0338	110	.00	.00	.00	8.	*	1	0838	260	.00	.00	.00	0.

1	0340	111	.00	.00	.00	6.	*	1	0840	261	.00	.00	.00	0.
1	0342	112	.00	.00	.00	4.	*	1	0842	262	.00	.00	.00	0.
1	0344	113	.00	.00	.00	3.	*	1	0844	263	.00	.00	.00	0.
1	0346	114	.00	.00	.00	2.	*	1	0846	264	.00	.00	.00	0.
1	0348	115	.00	.00	.00	2.	*	1	0848	265	.00	.00	.00	0.
1	0350	116	.00	.00	.00	1.	*	1	0850	266	.00	.00	.00	0.
1	0352	117	.00	.00	.00	1.	*	1	0852	267	.00	.00	.00	0.
1	0354	118	.00	.00	.00	1.	*	1	0854	268	.00	.00	.00	0.
1	0356	119	.00	.00	.00	1.	*	1	0856	269	.00	.00	.00	0.
1	0358	120	.00	.00	.00	0.	*	1	0858	270	.00	.00	.00	0.
1	0400	121	.00	.00	.00	0.	*	1	0900	271	.00	.00	.00	0.
1	0402	122	.00	.00	.00	0.	*	1	0902	272	.00	.00	.00	0.
1	0404	123	.00	.00	.00	0.	*	1	0904	273	.00	.00	.00	0.
1	0406	124	.00	.00	.00	0.	*	1	0906	274	.00	.00	.00	0.
1	0408	125	.00	.00	.00	0.	*	1	0908	275	.00	.00	.00	0.
1	0410	126	.00	.00	.00	0.	*	1	0910	276	.00	.00	.00	0.
1	0412	127	.00	.00	.00	0.	*	1	0912	277	.00	.00	.00	0.
1	0414	128	.00	.00	.00	0.	*	1	0914	278	.00	.00	.00	0.
1	0416	129	.00	.00	.00	0.	*	1	0916	279	.00	.00	.00	0.
1	0418	130	.00	.00	.00	0.	*	1	0918	280	.00	.00	.00	0.
1	0420	131	.00	.00	.00	0.	*	1	0920	281	.00	.00	.00	0.
1	0422	132	.00	.00	.00	0.	*	1	0922	282	.00	.00	.00	0.
1	0424	133	.00	.00	.00	0.	*	1	0924	283	.00	.00	.00	0.
1	0426	134	.00	.00	.00	0.	*	1	0926	284	.00	.00	.00	0.
1	0428	135	.00	.00	.00	0.	*	1	0928	285	.00	.00	.00	0.
1	0430	136	.00	.00	.00	0.	*	1	0930	286	.00	.00	.00	0.
1	0432	137	.00	.00	.00	0.	*	1	0932	287	.00	.00	.00	0.
1	0434	138	.00	.00	.00	0.	*	1	0934	288	.00	.00	.00	0.
1	0436	139	.00	.00	.00	0.	*	1	0936	289	.00	.00	.00	0.
1	0438	140	.00	.00	.00	0.	*	1	0938	290	.00	.00	.00	0.
1	0440	141	.00	.00	.00	0.	*	1	0940	291	.00	.00	.00	0.
1	0442	142	.00	.00	.00	0.	*	1	0942	292	.00	.00	.00	0.
1	0444	143	.00	.00	.00	0.	*	1	0944	293	.00	.00	.00	0.
1	0446	144	.00	.00	.00	0.	*	1	0946	294	.00	.00	.00	0.
1	0448	145	.00	.00	.00	0.	*	1	0948	295	.00	.00	.00	0.
1	0450	146	.00	.00	.00	0.	*	1	0950	296	.00	.00	.00	0.
1	0452	147	.00	.00	.00	0.	*	1	0952	297	.00	.00	.00	0.
1	0454	148	.00	.00	.00	0.	*	1	0954	298	.00	.00	.00	0.
1	0456	149	.00	.00	.00	0.	*	1	0956	299	.00	.00	.00	0.
1	0458	150	.00	.00	.00	0.	*	1	0958	300	.00	.00	.00	0.
							*							

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TOTAL RAINFALL = 2.85, TOTAL LOSS = 1.23, TOTAL EXCESS = 1.62

	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	9.97-HR
+	(CFS)	(HR)					
			(CFS)				
+	1105.	1.57		174.	105.	105.	105.
			(INCHES)	1.615	1.615	1.615	1.615
			(AC-FT)	86.	86.	86.	86.

CUMULATIVE AREA = 1.00 SQ MI

1

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+					6-HOUR	24-HOUR	72-HOUR			
+	HYDROGRAPH AT	SKYH2	1105.	1.57	174.	105.	105.	1.00		

*** NORMAL END OF HEC-1 ***

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1***************************************                  ***************************************
*                                       *                 *                                     *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                 *   U.S. ARMY CORPS OF ENGINEERS      *
*            SEPTEMBER 1990             *                 *   HYDROLOGIC ENGINEERING CENTER     *
*              VERSION 4.0              *                 *         609 SECOND STREET           *
*                                       *                 *      DAVIS, CALIFORNIA 95616        *
* RUN DATE  12/28/2001  TIME  09:10:05  *                 *          (916) 756-1104             *
*                                       *                 *                                     *
*****************************************                 ***************************************


                    X     X  XXXXXXX   XXXXX           X
                    X     X  X        X     X         XX
                    X     X  X        X                X
                    XXXXXXX  XXXX     X         XXXXX  X
                    X     X  X        X                X
                    X     X  X        X     X          X
                    X     X  XXXXXXX   XXXXX          XXX


THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
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1                                          HEC-1 INPUT                                         PAGE  1

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

            1         ID   Sky Harbor Airport
            2         ID   100-yr 2-hour Storm using dummy sub-basin input data
            3         IT       5                   300
            4         IO       1

            5         KK   SKYH5.IH1
            6         KM   SUB-BASIN  SKY
            7         KM   THIS IS A DUMMY MODEL FOR A 100-YR 2-HR STORM AT SKY HARBOR AIRPORT
            8         KM   5-Minute Main Time Interval
            9         KM
           10         KM   Model used to calculate a 100-yr 2-hr Rainfall Distribution using
           11         KM   the hypothetical distribution.
           12         KM
           13         KM   THIS BASIN USED RAINFALL REDUCTION FACTOR OF 1.000
           14         BA   1.000
           15         PH              0    0.78    1.52    2.58    2.86
           16         LG    .250    .340   5.000    .200    .000
           17         UC   1.067    .397
           18         UA       0       5      16      30      65      77      84      90      94      97
           19         UA     100
           20         ZZ
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1***************************************                  ***************************************
*                                       *                 *                                     *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                 *   U.S. ARMY CORPS OF ENGINEERS      *
*            SEPTEMBER 1990             *                 *   HYDROLOGIC ENGINEERING CENTER     *
*              VERSION 4.0              *                 *         609 SECOND STREET           *
*                                       *                 *      DAVIS, CALIFORNIA 95616        *
* RUN DATE  12/28/2001  TIME  09:10:05  *                 *          (916) 756-1104             *
*                                       *                 *                                     *
*****************************************                 ***************************************
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                           Sky Harbor Airport
                           100-yr 2-hour Storm using dummy sub-basin input data

   4 IO         OUTPUT CONTROL VARIABLES
                      IPRNT          1  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE

     IT         HYDROGRAPH TIME DATA
                       NMIN          5  MINUTES IN COMPUTATION INTERVAL
                      IDATE      1   0  STARTING DATE
                      ITIME       0000  STARTING TIME
                         NQ        300  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE      2   0  ENDING DATE
                     NDTIME       0055  ENDING TIME
                      ICENT         19  CENTURY MARK

                  COMPUTATION INTERVAL     .08 HOURS
                       TOTAL TIME BASE   24.92 HOURS

          ENGLISH UNITS
              DRAINAGE AREA         SQUARE MILES
```

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PRECIPITATION DEPTH   INCHES
LENGTH, ELEVATION     FEET
FLOW                  CUBIC FEET PER SECOND
STORAGE VOLUME        ACRE-FEET
SURFACE AREA          ACRES
TEMPERATURE           DEGREES FAHRENHEIT
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*** ***

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               **************
               *            *
   5 KK        *   SKYH5    *     .IH1
               *            *
               **************
                      SUB-BASIN  SKY
                      THIS IS A DUMMY MODEL FOR A 100-YR 2-HR STORM AT SKY HARBOR AIRPORT
                      5-Minute Main Time Interval

                      Model used to calculate a 100-yr 2-hr Rainfall Distribution using
                      the hypothetical distribution.

                      THIS BASIN USED RAINFALL REDUCTION FACTOR OF 1.000

                 SUBBASIN RUNOFF DATA

  14 BA            SUBBASIN CHARACTERISTICS
                      TAREA        1.00  SUBBASIN AREA

                   PRECIPITATION DATA

  15 PH                             DEPTHS FOR   0-PERCENT HYPOTHETICAL STORM
                   ..... HYDRO-35 ......   ............... TP-40 ...............   ........... TP-49 ...........
                   5-MIN  15-MIN  60-MIN    2-HR    3-HR    6-HR   12-HR   24-HR   2-DAY   4-DAY   7-DAY  10-DAY
                     .78    1.52    2.58    2.86     .00     .00     .00     .00     .00     .00     .00     .00

                                                  STORM AREA =     1.00

  16 LG            GREEN AND AMPT LOSS RATE
                      STRTL         .25  STARTING LOSS
                        DTH         .34  MOISTURE DEFICIT
                       PSIF        5.00  WETTING FRONT SUCTION
                      XKSAT         .20  HYDRAULIC CONDUCTIVITY
                      RTIMP         .00  PERCENT IMPERVIOUS AREA

  17 UC            CLARK UNITGRAPH
                         TC        1.07  TIME OF CONCENTRATION
                          R         .40  STORAGE COEFFICIENT

  18 UA            ACCUMULATED-AREA VS. TIME,  11 ORDINATES
                         .0       5.0      16.0      30.0      65.0      77.0      84.0      90.0      94.0      97.0
                     100.0

                                                         ***

                                                UNIT HYDROGRAPH PARAMETERS
                                          CLARK   TC=  1.07 HR,        R=   .40 HR
                                         SNYDER   TP=   .50 HR,       CP=   .58

                                                      UNIT HYDROGRAPH
                                               32 END-OF-PERIOD ORDINATES
                 29.       106.      210.      340.      576.      753.      747.      695.      635.      572.
                505.       445.      391.      331.      268.      217.      176.      142.      115.       93.
                 76.        61.       50.       40.       33.       26.       21.       17.       14.       11.
                  9.         7.
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****

HYDROGRAPH AT STATION   SKYH5

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DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1		0000	1	.00	.00	.00	0.	*	1		1230	151	.00	.00	.00	0.
1		0005	2	.02	.02	.00	0.	*	1		1235	152	.00	.00	.00	0.
1		0010	3	.02	.02	.00	0.	*	1		1240	153	.00	.00	.00	0.
1		0015	4	.02	.02	.00	0.	*	1		1245	154	.00	.00	.00	0.
1		0020	5	.02	.02	.00	0.	*	1		1250	155	.00	.00	.00	0.
1		0025	6	.03	.03	.00	0.	*	1		1255	156	.00	.00	.00	0.
1		0030	7	.03	.03	.00	0.	*	1		1300	157	.00	.00	.00	0.
1		0035	8	.08	.08	.00	0.	*	1		1305	158	.00	.00	.00	0.
1		0040	9	.09	.09	.00	0.	*	1		1310	159	.00	.00	.00	0.
1		0045	10	.10	.10	.00	0.	*	1		1315	160	.00	.00	.00	0.
1		0050	11	.15	.14	.01	0.	*	1		1320	161	.00	.00	.00	0.
1		0055	12	.20	.10	.10	4.	*	1		1325	162	.00	.00	.00	0.
1		0100	13	.43	.08	.35	23.	*	1		1330	163	.00	.00	.00	0.
1		0105	14	.77	.07	.70	81.	*	1		1335	164	.00	.00	.00	0.
1		0110	15	.30	.07	.24	193.	*	1		1340	165	.00	.00	.00	0.
1		0115	16	.17	.06	.11	358.	*	1		1345	166	.00	.00	.00	0.
1		0120	17	.11	.06	.05	583.	*	1		1350	167	.00	.00	.00	0.
1		0125	18	.09	.05	.04	857.	*	1		1355	168	.00	.00	.00	0.
1		0130	19	.08	.05	.03	1052.	*	1		1400	169	.00	.00	.00	0.

1	0135	20	.03	.03	.00	1103.	*	1	1405	170	.00	.00	.00	0.
1	0140	21	.03	.03	.00	1077.	*	1	1410	171	.00	.00	.00	0.
1	0145	22	.02	.02	.00	1015.	*	1	1415	172	.00	.00	.00	0.
1	0150	23	.02	.02	.00	933.	*	1	1420	173	.00	.00	.00	0.
1	0155	24	.02	.02	.00	840.	*	1	1425	174	.00	.00	.00	0.
1	0200	25	.02	.02	.00	745.	*	1	1430	175	.00	.00	.00	0.
1	0205	26	.00	.00	.00	651.	*	1	1435	176	.00	.00	.00	0.
1	0210	27	.00	.00	.00	555.	*	1	1440	177	.00	.00	.00	0.
1	0215	28	.00	.00	.00	461.	*	1	1445	178	.00	.00	.00	0.
1	0220	29	.00	.00	.00	379.	*	1	1450	179	.00	.00	.00	0.
1	0225	30	.00	.00	.00	310.	*	1	1455	180	.00	.00	.00	0.
1	0230	31	.00	.00	.00	253.	*	1	1500	181	.00	.00	.00	0.
1	0235	32	.00	.00	.00	205.	*	1	1505	182	.00	.00	.00	0.
1	0240	33	.00	.00	.00	166.	*	1	1510	183	.00	.00	.00	0.
1	0245	34	.00	.00	.00	134.	*	1	1515	184	.00	.00	.00	0.
1	0250	35	.00	.00	.00	109.	*	1	1520	185	.00	.00	.00	0.
1	0255	36	.00	.00	.00	88.	*	1	1525	186	.00	.00	.00	0.
1	0300	37	.00	.00	.00	71.	*	1	1530	187	.00	.00	.00	0.
1	0305	38	.00	.00	.00	58.	*	1	1535	188	.00	.00	.00	0.
1	0310	39	.00	.00	.00	47.	*	1	1540	189	.00	.00	.00	0.
1	0315	40	.00	.00	.00	38.	*	1	1545	190	.00	.00	.00	0.
1	0320	41	.00	.00	.00	31.	*	1	1550	191	.00	.00	.00	0.
1	0325	42	.00	.00	.00	25.	*	1	1555	192	.00	.00	.00	0.
1	0330	43	.00	.00	.00	20.	*	1	1600	193	.00	.00	.00	0.
1	0335	44	.00	.00	.00	16.	*	1	1605	194	.00	.00	.00	0.
1	0340	45	.00	.00	.00	11.	*	1	1610	195	.00	.00	.00	0.
1	0345	46	.00	.00	.00	4.	*	1	1615	196	.00	.00	.00	0.
1	0350	47	.00	.00	.00	2.	*	1	1620	197	.00	.00	.00	0.
1	0355	48	.00	.00	.00	1.	*	1	1625	198	.00	.00	.00	0.
1	0400	49	.00	.00	.00	1.	*	1	1630	199	.00	.00	.00	0.
1	0405	50	.00	.00	.00	0.	*	1	1635	200	.00	.00	.00	0.
1	0410	51	.00	.00	.00	0.	*	1	1640	201	.00	.00	.00	0.
1	0415	52	.00	.00	.00	0.	*	1	1645	202	.00	.00	.00	0.
1	0420	53	.00	.00	.00	0.	*	1	1650	203	.00	.00	.00	0.
1	0425	54	.00	.00	.00	0.	*	1	1655	204	.00	.00	.00	0.
1	0430	55	.00	.00	.00	0.	*	1	1700	205	.00	.00	.00	0.
1	0435	56	.00	.00	.00	0.	*	1	1705	206	.00	.00	.00	0.
1	0440	57	.00	.00	.00	0.	*	1	1710	207	.00	.00	.00	0.
1	0445	58	.00	.00	.00	0.	*	1	1715	208	.00	.00	.00	0.
1	0450	59	.00	.00	.00	0.	*	1	1720	209	.00	.00	.00	0.
1	0455	60	.00	.00	.00	0.	*	1	1725	210	.00	.00	.00	0.
1	0500	61	.00	.00	.00	0.	*	1	1730	211	.00	.00	.00	0.
1	0505	62	.00	.00	.00	0.	*	1	1735	212	.00	.00	.00	0.
1	0510	63	.00	.00	.00	0.	*	1	1740	213	.00	.00	.00	0.
1	0515	64	.00	.00	.00	0.	*	1	1745	214	.00	.00	.00	0.
1	0520	65	.00	.00	.00	0.	*	1	1750	215	.00	.00	.00	0.
1	0525	66	.00	.00	.00	0.	*	1	1755	216	.00	.00	.00	0.
1	0530	67	.00	.00	.00	0.	*	1	1800	217	.00	.00	.00	0.
1	0535	68	.00	.00	.00	0.	*	1	1805	218	.00	.00	.00	0.
1	0540	69	.00	.00	.00	0.	*	1	1810	219	.00	.00	.00	0.
1	0545	70	.00	.00	.00	0.	*	1	1815	220	.00	.00	.00	0.
1	0550	71	.00	.00	.00	0.	*	1	1820	221	.00	.00	.00	0.
1	0555	72	.00	.00	.00	0.	*	1	1825	222	.00	.00	.00	0.
1	0600	73	.00	.00	.00	0.	*	1	1830	223	.00	.00	.00	0.
1	0605	74	.00	.00	.00	0.	*	1	1835	224	.00	.00	.00	0.
1	0610	75	.00	.00	.00	0.	*	1	1840	225	.00	.00	.00	0.
1	0615	76	.00	.00	.00	0.	*	1	1845	226	.00	.00	.00	0.
1	0620	77	.00	.00	.00	0.	*	1	1850	227	.00	.00	.00	0.
1	0625	78	.00	.00	.00	0.	*	1	1855	228	.00	.00	.00	0.
1	0630	79	.00	.00	.00	0.	*	1	1900	229	.00	.00	.00	0.
1	0635	80	.00	.00	.00	0.	*	1	1905	230	.00	.00	.00	0.
1	0640	81	.00	.00	.00	0.	*	1	1910	231	.00	.00	.00	0.
1	0645	82	.00	.00	.00	0.	*	1	1915	232	.00	.00	.00	0.
1	0650	83	.00	.00	.00	0.	*	1	1920	233	.00	.00	.00	0.
1	0655	84	.00	.00	.00	0.	*	1	1925	234	.00	.00	.00	0.
1	0700	85	.00	.00	.00	0.	*	1	1930	235	.00	.00	.00	0.
1	0705	86	.00	.00	.00	0.	*	1	1935	236	.00	.00	.00	0.
1	0710	87	.00	.00	.00	0.	*	1	1940	237	.00	.00	.00	0.
1	0715	88	.00	.00	.00	0.	*	1	1945	238	.00	.00	.00	0.
1	0720	89	.00	.00	.00	0.	*	1	1950	239	.00	.00	.00	0.
1	0725	90	.00	.00	.00	0.	*	1	1955	240	.00	.00	.00	0.
1	0730	91	.00	.00	.00	0.	*	1	2000	241	.00	.00	.00	0.
1	0735	92	.00	.00	.00	0.	*	1	2005	242	.00	.00	.00	0.
1	0740	93	.00	.00	.00	0.	*	1	2010	243	.00	.00	.00	0.
1	0745	94	.00	.00	.00	0.	*	1	2015	244	.00	.00	.00	0.
1	0750	95	.00	.00	.00	0.	*	1	2020	245	.00	.00	.00	0.
1	0755	96	.00	.00	.00	0.	*	1	2025	246	.00	.00	.00	0.
1	0800	97	.00	.00	.00	0.	*	1	2030	247	.00	.00	.00	0.
1	0805	98	.00	.00	.00	0.	*	1	2035	248	.00	.00	.00	0.
1	0810	99	.00	.00	.00	0.	*	1	2040	249	.00	.00	.00	0.
1	0815	100	.00	.00	.00	0.	*	1	2045	250	.00	.00	.00	0.
1	0820	101	.00	.00	.00	0.	*	1	2050	251	.00	.00	.00	0.
1	0825	102	.00	.00	.00	0.	*	1	2055	252	.00	.00	.00	0.
1	0830	103	.00	.00	.00	0.	*	1	2100	253	.00	.00	.00	0.
1	0835	104	.00	.00	.00	0.	*	1	2105	254	.00	.00	.00	0.
1	0840	105	.00	.00	.00	0.	*	1	2110	255	.00	.00	.00	0.
1	0845	106	.00	.00	.00	0.	*	1	2115	256	.00	.00	.00	0.
1	0850	107	.00	.00	.00	0.	*	1	2120	257	.00	.00	.00	0.
1	0855	108	.00	.00	.00	0.	*	1	2125	258	.00	.00	.00	0.
1	0900	109	.00	.00	.00	0.	*	1	2130	259	.00	.00	.00	0.
1	0905	110	.00	.00	.00	0.	*	1	2135	260	.00	.00	.00	0.
1	0910	111	.00	.00	.00	0.	*	1	2140	261	.00	.00	.00	0.
1	0915	112	.00	.00	.00	0.	*	1	2145	262	.00	.00	.00	0.
1	0920	113	.00	.00	.00	0.	*	1	2150	263	.00	.00	.00	0.
1	0925	114	.00	.00	.00	0.	*	1	2155	264	.00	.00	.00	0.

1	0930	115	.00	.00	.00	0.	*	1	2200	265	.00	.00	.00	0.
1	0935	116	.00	.00	.00	0.	*	1	2205	266	.00	.00	.00	0.
1	0940	117	.00	.00	.00	0.	*	1	2210	267	.00	.00	.00	0.
1	0945	118	.00	.00	.00	0.	*	1	2215	268	.00	.00	.00	0.
1	0950	119	.00	.00	.00	0.	*	1	2220	269	.00	.00	.00	0.
1	0955	120	.00	.00	.00	0.	*	1	2225	270	.00	.00	.00	0.
1	1000	121	.00	.00	.00	0.	*	1	2230	271	.00	.00	.00	0.
1	1005	122	.00	.00	.00	0.	*	1	2235	272	.00	.00	.00	0.
1	1010	123	.00	.00	.00	0.	*	1	2240	273	.00	.00	.00	0.
1	1015	124	.00	.00	.00	0.	*	1	2245	274	.00	.00	.00	0.
1	1020	125	.00	.00	.00	0.	*	1	2250	275	.00	.00	.00	0.
1	1025	126	.00	.00	.00	0.	*	1	2255	276	.00	.00	.00	0.
1	1030	127	.00	.00	.00	0.	*	1	2300	277	.00	.00	.00	0.
1	1035	128	.00	.00	.00	0.	*	1	2305	278	.00	.00	.00	0.
1	1040	129	.00	.00	.00	0.	*	1	2310	279	.00	.00	.00	0.
1	1045	130	.00	.00	.00	0.	*	1	2315	280	.00	.00	.00	0.
1	1050	131	.00	.00	.00	0.	*	1	2320	281	.00	.00	.00	0.
1	1055	132	.00	.00	.00	0.	*	1	2325	282	.00	.00	.00	0.
1	1100	133	.00	.00	.00	0.	*	1	2330	283	.00	.00	.00	0.
1	1105	134	.00	.00	.00	0.	*	1	2335	284	.00	.00	.00	0.
1	1110	135	.00	.00	.00	0.	*	1	2340	285	.00	.00	.00	0.
1	1115	136	.00	.00	.00	0.	*	1	2345	286	.00	.00	.00	0.
1	1120	137	.00	.00	.00	0.	*	1	2350	287	.00	.00	.00	0.
1	1125	138	.00	.00	.00	0.	*	1	2355	288	.00	.00	.00	0.
1	1130	139	.00	.00	.00	0.	*	2	0000	289	.00	.00	.00	0.
1	1135	140	.00	.00	.00	0.	*	2	0005	290	.00	.00	.00	0.
1	1140	141	.00	.00	.00	0.	*	2	0010	291	.00	.00	.00	0.
1	1145	142	.00	.00	.00	0.	*	2	0015	292	.00	.00	.00	0.
1	1150	143	.00	.00	.00	0.	*	2	0020	293	.00	.00	.00	0.
1	1155	144	.00	.00	.00	0.	*	2	0025	294	.00	.00	.00	0.
1	1200	145	.00	.00	.00	0.	*	2	0030	295	.00	.00	.00	0.
1	1205	146	.00	.00	.00	0.	*	2	0035	296	.00	.00	.00	0.
1	1210	147	.00	.00	.00	0.	*	2	0040	297	.00	.00	.00	0.
1	1215	148	.00	.00	.00	0.	*	2	0045	298	.00	.00	.00	0.
1	1220	149	.00	.00	.00	0.	*	2	0050	299	.00	.00	.00	0.
1	1225	150	.00	.00	.00	0.	*	2	0055	300	.00	.00	.00	0.
							*							

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TOTAL RAINFALL = 2.85, TOTAL LOSS = 1.23, TOTAL EXCESS = 1.62

	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	24.92-HR
+	(CFS)	(HR)					
			(CFS)				
+	1103.	1.58		174.	43.	42.	42.
			(INCHES)	1.614	1.614	1.614	1.614
			(AC-FT)	86.	86.	86.	86.

CUMULATIVE AREA = 1.00 SQ MI

1

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+					6-HOUR	24-HOUR	72-HOUR			
	HYDROGRAPH AT									
+		SKYH5	1103.	1.58	174.	43.	42.	1.00		

*** NORMAL END OF HEC-1 ***

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1***************************************                    ***************************************
 *                                     *                    *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                    *   U.S. ARMY CORPS OF ENGINEERS      *
 *          SEPTEMBER 1990              *                    *   HYDROLOGIC ENGINEERING CENTER     *
 *           VERSION 4.0                *                    *        609 SECOND STREET            *
 *                                     *                    *     DAVIS, CALIFORNIA 95616         *
 * RUN DATE  12/28/2001  TIME  09:10:05 *                    *        (916) 756-1104               *
 *                                     *                    *                                     *
***************************************                    ***************************************


                    X     X  XXXXXXX   XXXXX          X
                    X     X  X        X     X        XX
                    X     X  X        X               X
                    XXXXXXX  XXXX     X       XXXXX   X
                    X     X  X        X               X
                    X     X  X        X     X         X
                    X     X  XXXXXXX   XXXXX         XXX


      THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

      THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
      THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
      NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
      DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
      KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
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      LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

        1           ID   Sky Harbor Airport
        2           ID   100-yr 2-hour Storm using dummy sub-basin input data
        3           IT      5                 300
        4           IO      1

        5           KK   SKYH5.IH1
        6           KM   SUB-BASIN  SKY
        7           KM   THIS IS A DUMMY MODEL FOR A 100-YR 2-HR STORM AT SKY HARBOR AIRPORT
        8           KM   5-Minute Main Time Interval
        9           KM
       10           KM   Model used to calculate a 100-yr 2-hr Rainfall Distribution using
       11           KM   the hypothetical distribution.
       12           KM
       13           KM   THIS BASIN USED RAINFALL REDUCTION FACTOR OF 1.000
       14           BA   1.000
       15           PH               0    0.78    1.52    2.58    2.86
       16           LG    .250    .340   5.000    .200    .000
       17           UC   1.067    .397
       18           UA       0       5      16      30      65      77      84      90      94      97
       19           UA     100
       20           ZZ
```


```
1***************************************                    ***************************************
 *                                     *                    *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                    *   U.S. ARMY CORPS OF ENGINEERS      *
 *          SEPTEMBER 1990              *                    *   HYDROLOGIC ENGINEERING CENTER     *
 *           VERSION 4.0                *                    *        609 SECOND STREET            *
 *                                     *                    *     DAVIS, CALIFORNIA 95616         *
 * RUN DATE  12/28/2001  TIME  09:10:05 *                    *        (916) 756-1104               *
 *                                     *                    *                                     *
***************************************                    ***************************************
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                    Sky Harbor Airport
                    100-yr 2-hour Storm using dummy sub-basin input data

   4 IO         OUTPUT CONTROL VARIABLES
                      IPRNT          1  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE

     IT         HYDROGRAPH TIME DATA
                      NMIN           5  MINUTES IN COMPUTATION INTERVAL
                      IDATE      1   0  STARTING DATE
                      ITIME       0000  STARTING TIME
                         NQ        300  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE      2   0  ENDING DATE
                     NDTIME       0055  ENDING TIME
                      ICENT         19  CENTURY MARK

                 COMPUTATION INTERVAL    .08 HOURS
                      TOTAL TIME BASE  24.92 HOURS

        ENGLISH UNITS
              DRAINAGE AREA          SQUARE MILES
```

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

*** ***

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           **************
           *            *
   5 KK    *    SKYH5   *     .IH1
           *            *
           **************
                        SUB-BASIN  SKY
                        THIS IS A DUMMY MODEL FOR A 100-YR 2-HR STORM AT SKY HARBOR AIRPORT
                        5-Minute Main Time Interval

                        Model used to calculate a 100-yr 2-hr Rainfall Distribution using
                        the hypothetical distribution.

                        THIS BASIN USED RAINFALL REDUCTION FACTOR OF 1.000

            SUBBASIN RUNOFF DATA

  14 BA       SUBBASIN CHARACTERISTICS
                    TAREA        1.00  SUBBASIN AREA

              PRECIPITATION DATA

  15 PH                              DEPTHS FOR   0-PERCENT HYPOTHETICAL STORM
              ..... HYDRO-35 ......   ............... TP-40 ...............   ........... TP-49 ...........
              5-MIN  15-MIN  60-MIN    2-HR    3-HR    6-HR   12-HR   24-HR   2-DAY   4-DAY   7-DAY  10-DAY
                .78    1.52    2.58    2.86     .00     .00     .00     .00     .00     .00     .00     .00

                                                 STORM AREA =     1.00

  16 LG       GREEN AND AMPT LOSS RATE
                    STRTL         .25  STARTING LOSS
                      DTH         .34  MOISTURE DEFICIT
                     PSIF        5.00  WETTING FRONT SUCTION
                   XKSAT          .20  HYDRAULIC CONDUCTIVITY
                    RTIMP         .00  PERCENT IMPERVIOUS AREA

  17 UC       CLARK UNITGRAPH
                       TC        1.07  TIME OF CONCENTRATION
                        R         .40  STORAGE COEFFICIENT

  18 UA       ACCUMULATED-AREA VS. TIME,  11 ORDINATES
                     .0       5.0      16.0      30.0      65.0      77.0      84.0      90.0      94.0      97.0
                  100.0

                                                       ***

                                                 UNIT HYDROGRAPH PARAMETERS
                                           CLARK   TC=  1.07 HR,       R=   .40 HR
                                          SNYDER   TP=   .50 HR,      CP=   .58

                                                      UNIT HYDROGRAPH
                                                 32 END-OF-PERIOD ORDINATES
              29.       106.      210.      340.      576.      753.      747.      695.      635.      572.
             505.       445.      391.      331.      268.      217.      176.      142.      115.       93.
              76.        61.       50.       40.       33.       26.       21.       17.       14.       11.
               9.         7.
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HYDROGRAPH AT STATION   SKYH5

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DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1		0000	1	.00	.00	.00	0.	*	1		1230	151	.00	.00	.00	0.
1		0005	2	.02	.02	.00	0.	*	1		1235	152	.00	.00	.00	0.
1		0010	3	.02	.02	.00	0.	*	1		1240	153	.00	.00	.00	0.
1		0015	4	.02	.02	.00	0.	*	1		1245	154	.00	.00	.00	0.
1		0020	5	.02	.02	.00	0.	*	1		1250	155	.00	.00	.00	0.
1		0025	6	.03	.03	.00	0.	*	1		1255	156	.00	.00	.00	0.
1		0030	7	.03	.03	.00	0.	*	1		1300	157	.00	.00	.00	0.
1		0035	8	.08	.08	.00	0.	*	1		1305	158	.00	.00	.00	0.
1		0040	9	.09	.09	.00	0.	*	1		1310	159	.00	.00	.00	0.
1		0045	10	.10	.10	.00	0.	*	1		1315	160	.00	.00	.00	0.
1		0050	11	.15	.14	.01	0.	*	1		1320	161	.00	.00	.00	0.
1		0055	12	.20	.10	.10	4.	*	1		1325	162	.00	.00	.00	0.
1		0100	13	.43	.08	.35	23.	*	1		1330	163	.00	.00	.00	0.
1		0105	14	.77	.07	.70	81.	*	1		1335	164	.00	.00	.00	0.
1		0110	15	.30	.07	.24	193.	*	1		1340	165	.00	.00	.00	0.
1		0115	16	.17	.06	.11	358.	*	1		1345	166	.00	.00	.00	0.
1		0120	17	.11	.06	.05	583.	*	1		1350	167	.00	.00	.00	0.
1		0125	18	.09	.05	.04	857.	*	1		1355	168	.00	.00	.00	0.
1		0130	19	.08	.05	.03	1052.	*	1		1400	169	.00	.00	.00	0.

1	0135	20	.03	.03	.00	1103.	*	1	1405	170	.00	.00	.00	0.
1	0140	21	.03	.03	.00	1077.	*	1	1410	171	.00	.00	.00	0.
1	0145	22	.02	.02	.00	1015.	*	1	1415	172	.00	.00	.00	0.
1	0150	23	.02	.02	.00	933.	*	1	1420	173	.00	.00	.00	0.
1	0155	24	.02	.02	.00	840.	*	1	1425	174	.00	.00	.00	0.
1	0200	25	.02	.02	.00	745.	*	1	1430	175	.00	.00	.00	0.
1	0205	26	.00	.00	.00	651.	*	1	1435	176	.00	.00	.00	0.
1	0210	27	.00	.00	.00	555.	*	1	1440	177	.00	.00	.00	0.
1	0215	28	.00	.00	.00	461.	*	1	1445	178	.00	.00	.00	0.
1	0220	29	.00	.00	.00	379.	*	1	1450	179	.00	.00	.00	0.
1	0225	30	.00	.00	.00	310.	*	1	1455	180	.00	.00	.00	0.
1	0230	31	.00	.00	.00	253.	*	1	1500	181	.00	.00	.00	0.
1	0235	32	.00	.00	.00	205.	*	1	1505	182	.00	.00	.00	0.
1	0240	33	.00	.00	.00	166.	*	1	1510	183	.00	.00	.00	0.
1	0245	34	.00	.00	.00	134.	*	1	1515	184	.00	.00	.00	0.
1	0250	35	.00	.00	.00	109.	*	1	1520	185	.00	.00	.00	0.
1	0255	36	.00	.00	.00	88.	*	1	1525	186	.00	.00	.00	0.
1	0300	37	.00	.00	.00	71.	*	1	1530	187	.00	.00	.00	0.
1	0305	38	.00	.00	.00	58.	*	1	1535	188	.00	.00	.00	0.
1	0310	39	.00	.00	.00	47.	*	1	1540	189	.00	.00	.00	0.
1	0315	40	.00	.00	.00	38.	*	1	1545	190	.00	.00	.00	0.
1	0320	41	.00	.00	.00	31.	*	1	1550	191	.00	.00	.00	0.
1	0325	42	.00	.00	.00	25.	*	1	1555	192	.00	.00	.00	0.
1	0330	43	.00	.00	.00	20.	*	1	1600	193	.00	.00	.00	0.
1	0335	44	.00	.00	.00	16.	*	1	1605	194	.00	.00	.00	0.
1	0340	45	.00	.00	.00	11.	*	1	1610	195	.00	.00	.00	0.
1	0345	46	.00	.00	.00	4.	*	1	1615	196	.00	.00	.00	0.
1	0350	47	.00	.00	.00	2.	*	1	1620	197	.00	.00	.00	0.
1	0355	48	.00	.00	.00	1.	*	1	1625	198	.00	.00	.00	0.
1	0400	49	.00	.00	.00	1.	*	1	1630	199	.00	.00	.00	0.
1	0405	50	.00	.00	.00	0.	*	1	1635	200	.00	.00	.00	0.
1	0410	51	.00	.00	.00	0.	*	1	1640	201	.00	.00	.00	0.
1	0415	52	.00	.00	.00	0.	*	1	1645	202	.00	.00	.00	0.
1	0420	53	.00	.00	.00	0.	*	1	1650	203	.00	.00	.00	0.
1	0425	54	.00	.00	.00	0.	*	1	1655	204	.00	.00	.00	0.
1	0430	55	.00	.00	.00	0.	*	1	1700	205	.00	.00	.00	0.
1	0435	56	.00	.00	.00	0.	*	1	1705	206	.00	.00	.00	0.
1	0440	57	.00	.00	.00	0.	*	1	1710	207	.00	.00	.00	0.
1	0445	58	.00	.00	.00	0.	*	1	1715	208	.00	.00	.00	0.
1	0450	59	.00	.00	.00	0.	*	1	1720	209	.00	.00	.00	0.
1	0455	60	.00	.00	.00	0.	*	1	1725	210	.00	.00	.00	0.
1	0500	61	.00	.00	.00	0.	*	1	1730	211	.00	.00	.00	0.
1	0505	62	.00	.00	.00	0.	*	1	1735	212	.00	.00	.00	0.
1	0510	63	.00	.00	.00	0.	*	1	1740	213	.00	.00	.00	0.
1	0515	64	.00	.00	.00	0.	*	1	1745	214	.00	.00	.00	0.
1	0520	65	.00	.00	.00	0.	*	1	1750	215	.00	.00	.00	0.
1	0525	66	.00	.00	.00	0.	*	1	1755	216	.00	.00	.00	0.
1	0530	67	.00	.00	.00	0.	*	1	1800	217	.00	.00	.00	0.
1	0535	68	.00	.00	.00	0.	*	1	1805	218	.00	.00	.00	0.
1	0540	69	.00	.00	.00	0.	*	1	1810	219	.00	.00	.00	0.
1	0545	70	.00	.00	.00	0.	*	1	1815	220	.00	.00	.00	0.
1	0550	71	.00	.00	.00	0.	*	1	1820	221	.00	.00	.00	0.
1	0555	72	.00	.00	.00	0.	*	1	1825	222	.00	.00	.00	0.
1	0600	73	.00	.00	.00	0.	*	1	1830	223	.00	.00	.00	0.
1	0605	74	.00	.00	.00	0.	*	1	1835	224	.00	.00	.00	0.
1	0610	75	.00	.00	.00	0.	*	1	1840	225	.00	.00	.00	0.
1	0615	76	.00	.00	.00	0.	*	1	1845	226	.00	.00	.00	0.
1	0620	77	.00	.00	.00	0.	*	1	1850	227	.00	.00	.00	0.
1	0625	78	.00	.00	.00	0.	*	1	1855	228	.00	.00	.00	0.
1	0630	79	.00	.00	.00	0.	*	1	1900	229	.00	.00	.00	0.
1	0635	80	.00	.00	.00	0.	*	1	1905	230	.00	.00	.00	0.
1	0640	81	.00	.00	.00	0.	*	1	1910	231	.00	.00	.00	0.
1	0645	82	.00	.00	.00	0.	*	1	1915	232	.00	.00	.00	0.
1	0650	83	.00	.00	.00	0.	*	1	1920	233	.00	.00	.00	0.
1	0655	84	.00	.00	.00	0.	*	1	1925	234	.00	.00	.00	0.
1	0700	85	.00	.00	.00	0.	*	1	1930	235	.00	.00	.00	0.
1	0705	86	.00	.00	.00	0.	*	1	1935	236	.00	.00	.00	0.
1	0710	87	.00	.00	.00	0.	*	1	1940	237	.00	.00	.00	0.
1	0715	88	.00	.00	.00	0.	*	1	1945	238	.00	.00	.00	0.
1	0720	89	.00	.00	.00	0.	*	1	1950	239	.00	.00	.00	0.
1	0725	90	.00	.00	.00	0.	*	1	1955	240	.00	.00	.00	0.
1	0730	91	.00	.00	.00	0.	*	1	2000	241	.00	.00	.00	0.
1	0735	92	.00	.00	.00	0.	*	1	2005	242	.00	.00	.00	0.
1	0740	93	.00	.00	.00	0.	*	1	2010	243	.00	.00	.00	0.
1	0745	94	.00	.00	.00	0.	*	1	2015	244	.00	.00	.00	0.
1	0750	95	.00	.00	.00	0.	*	1	2020	245	.00	.00	.00	0.
1	0755	96	.00	.00	.00	0.	*	1	2025	246	.00	.00	.00	0.
1	0800	97	.00	.00	.00	0.	*	1	2030	247	.00	.00	.00	0.
1	0805	98	.00	.00	.00	0.	*	1	2035	248	.00	.00	.00	0.
1	0810	99	.00	.00	.00	0.	*	1	2040	249	.00	.00	.00	0.
1	0815	100	.00	.00	.00	0.	*	1	2045	250	.00	.00	.00	0.
1	0820	101	.00	.00	.00	0.	*	1	2050	251	.00	.00	.00	0.
1	0825	102	.00	.00	.00	0.	*	1	2055	252	.00	.00	.00	0.
1	0830	103	.00	.00	.00	0.	*	1	2100	253	.00	.00	.00	0.
1	0835	104	.00	.00	.00	0.	*	1	2105	254	.00	.00	.00	0.
1	0840	105	.00	.00	.00	0.	*	1	2110	255	.00	.00	.00	0.
1	0845	106	.00	.00	.00	0.	*	1	2115	256	.00	.00	.00	0.
1	0850	107	.00	.00	.00	0.	*	1	2120	257	.00	.00	.00	0.
1	0855	108	.00	.00	.00	0.	*	1	2125	258	.00	.00	.00	0.
1	0900	109	.00	.00	.00	0.	*	1	2130	259	.00	.00	.00	0.
1	0905	110	.00	.00	.00	0.	*	1	2135	260	.00	.00	.00	0.
1	0910	111	.00	.00	.00	0.	*	1	2140	261	.00	.00	.00	0.
1	0915	112	.00	.00	.00	0.	*	1	2145	262	.00	.00	.00	0.
1	0920	113	.00	.00	.00	0.	*	1	2150	263	.00	.00	.00	0.
1	0925	114	.00	.00	.00	0.	*	1	2155	264	.00	.00	.00	0.

1	0930	115	.00	.00	.00	0.	*	1	2200	265	.00	.00	.00	0.
1	0935	116	.00	.00	.00	0.	*	1	2205	266	.00	.00	.00	0.
1	0940	117	.00	.00	.00	0.	*	1	2210	267	.00	.00	.00	0.
1	0945	118	.00	.00	.00	0.	*	1	2215	268	.00	.00	.00	0.
1	0950	119	.00	.00	.00	0.	*	1	2220	269	.00	.00	.00	0.
1	0955	120	.00	.00	.00	0.	*	1	2225	270	.00	.00	.00	0.
1	1000	121	.00	.00	.00	0.	*	1	2230	271	.00	.00	.00	0.
1	1005	122	.00	.00	.00	0.	*	1	2235	272	.00	.00	.00	0.
1	1010	123	.00	.00	.00	0.	*	1	2240	273	.00	.00	.00	0.
1	1015	124	.00	.00	.00	0.	*	1	2245	274	.00	.00	.00	0.
1	1020	125	.00	.00	.00	0.	*	1	2250	275	.00	.00	.00	0.
1	1025	126	.00	.00	.00	0.	*	1	2255	276	.00	.00	.00	0.
1	1030	127	.00	.00	.00	0.	*	1	2300	277	.00	.00	.00	0.
1	1035	128	.00	.00	.00	0.	*	1	2305	278	.00	.00	.00	0.
1	1040	129	.00	.00	.00	0.	*	1	2310	279	.00	.00	.00	0.
1	1045	130	.00	.00	.00	0.	*	1	2315	280	.00	.00	.00	0.
1	1050	131	.00	.00	.00	0.	*	1	2320	281	.00	.00	.00	0.
1	1055	132	.00	.00	.00	0.	*	1	2325	282	.00	.00	.00	0.
1	1100	133	.00	.00	.00	0.	*	1	2330	283	.00	.00	.00	0.
1	1105	134	.00	.00	.00	0.	*	1	2335	284	.00	.00	.00	0.
1	1110	135	.00	.00	.00	0.	*	1	2340	285	.00	.00	.00	0.
1	1115	136	.00	.00	.00	0.	*	1	2345	286	.00	.00	.00	0.
1	1120	137	.00	.00	.00	0.	*	1	2350	287	.00	.00	.00	0.
1	1125	138	.00	.00	.00	0.	*	1	2355	288	.00	.00	.00	0.
1	1130	139	.00	.00	.00	0.	*	2	0000	289	.00	.00	.00	0.
1	1135	140	.00	.00	.00	0.	*	2	0005	290	.00	.00	.00	0.
1	1140	141	.00	.00	.00	0.	*	2	0010	291	.00	.00	.00	0.
1	1145	142	.00	.00	.00	0.	*	2	0015	292	.00	.00	.00	0.
1	1150	143	.00	.00	.00	0.	*	2	0020	293	.00	.00	.00	0.
1	1155	144	.00	.00	.00	0.	*	2	0025	294	.00	.00	.00	0.
1	1200	145	.00	.00	.00	0.	*	2	0030	295	.00	.00	.00	0.
1	1205	146	.00	.00	.00	0.	*	2	0035	296	.00	.00	.00	0.
1	1210	147	.00	.00	.00	0.	*	2	0040	297	.00	.00	.00	0.
1	1215	148	.00	.00	.00	0.	*	2	0045	298	.00	.00	.00	0.
1	1220	149	.00	.00	.00	0.	*	2	0050	299	.00	.00	.00	0.
1	1225	150	.00	.00	.00	0.	*	2	0055	300	.00	.00	.00	0.
							*							

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*******************************************************************************************************

     TOTAL RAINFALL =     2.85, TOTAL LOSS =     1.23, TOTAL EXCESS =     1.62

  PEAK FLOW      TIME                             MAXIMUM AVERAGE FLOW
                                     6-HR          24-HR         72-HR        24.92-HR
+   (CFS)        (HR)
                           (CFS)
+   1103.        1.58                 174.           43.           42.           42.
                        (INCHES)     1.614         1.614         1.614         1.614
                         (AC-FT)       86.           86.           86.           86.

                        CUMULATIVE AREA =      1.00 SQ MI

1
                                                         RUNOFF SUMMARY
                                                 FLOW IN CUBIC FEET PER SECOND
                                               TIME IN HOURS,  AREA IN SQUARE MILES

                                            PEAK    TIME OF     AVERAGE FLOW FOR MAXIMUM PERIOD       BASIN     MAXIMUM     TIME OF
           OPERATION         STATION        FLOW      PEAK                                             AREA      STAGE     MAX STAGE
+                                                                6-HOUR      24-HOUR      72-HOUR

           HYDROGRAPH AT
+                              SKYH5        1103.     1.58        174.          43.          42.       1.00


 *** NORMAL END OF HEC-1 ***
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ATTACHMENT 4

Table 2.5
SCS Type II 24-hour Storm Rainfall Distribution

Time (minutes)	% Rainfall Depth	Time (minutes)	% Rainfall Depth
0.00	0.00	12.25	70.70
0.25	0.20	12.50	73.50
0.50	0.50	12.75	75.80
0.75	0.80	13.00	77.60
1.00	1.10	13.25	79.10
1.25	1.40	13.50	80.40
1.50	1.70	13.75	81.50
1.75	2.00	14.00	82.50
2.00	2.30	14.25	83.40
2.25	2.60	14.50	84.20
2.50	2.90	14.75	84.90
2.75	3.20	15.00	85.60
3.00	3.50	15.25	86.30
3.25	3.80	15.50	86.90
3.50	4.10	15.75	87.50
3.75	4.40	16.00	88.10
4.00	4.80	16.25	88.70
4.25	5.20	16.50	89.30
4.50	5.60	16.75	89.80
4.75	6.00	17.00	90.30
5.00	6.40	17.25	90.80
5.25	6.80	17.50	91.30
5.50	7.20	17.75	91.80
5.75	7.60	18.00	92.20
6.00	8.00	18.25	92.60
6.25	8.50	18.50	93.00
6.50	9.00	18.75	93.40
6.75	9.50	19.00	93.80
7.00	10.00	19.25	94.20
7.25	10.50	19.50	94.60
7.50	11.00	19.75	95.00
7.75	11.50	20.00	95.30
8.00	12.00	20.25	95.60
8.25	12.60	20.50	95.90
8.50	13.30	20.75	96.20
8.75	14.00	21.00	96.50
9.00	14.70	21.25	96.80
9.25	15.50	21.50	97.10
9.50	16.30	21.75	97.40
9.75	17.20	22.00	97.70
10.00	18.10	22.25	98.00
10.25	19.10	22.50	98.30
10.50	20.30	22.75	98.60
10.75	21.80	23.00	98.90
11.00	23.60	23.25	99.20
11.25	25.70	23.50	99.50
11.50	28.30	23.75	99.80
11.75	38.70	24.00	100.00
12.00	66.30		

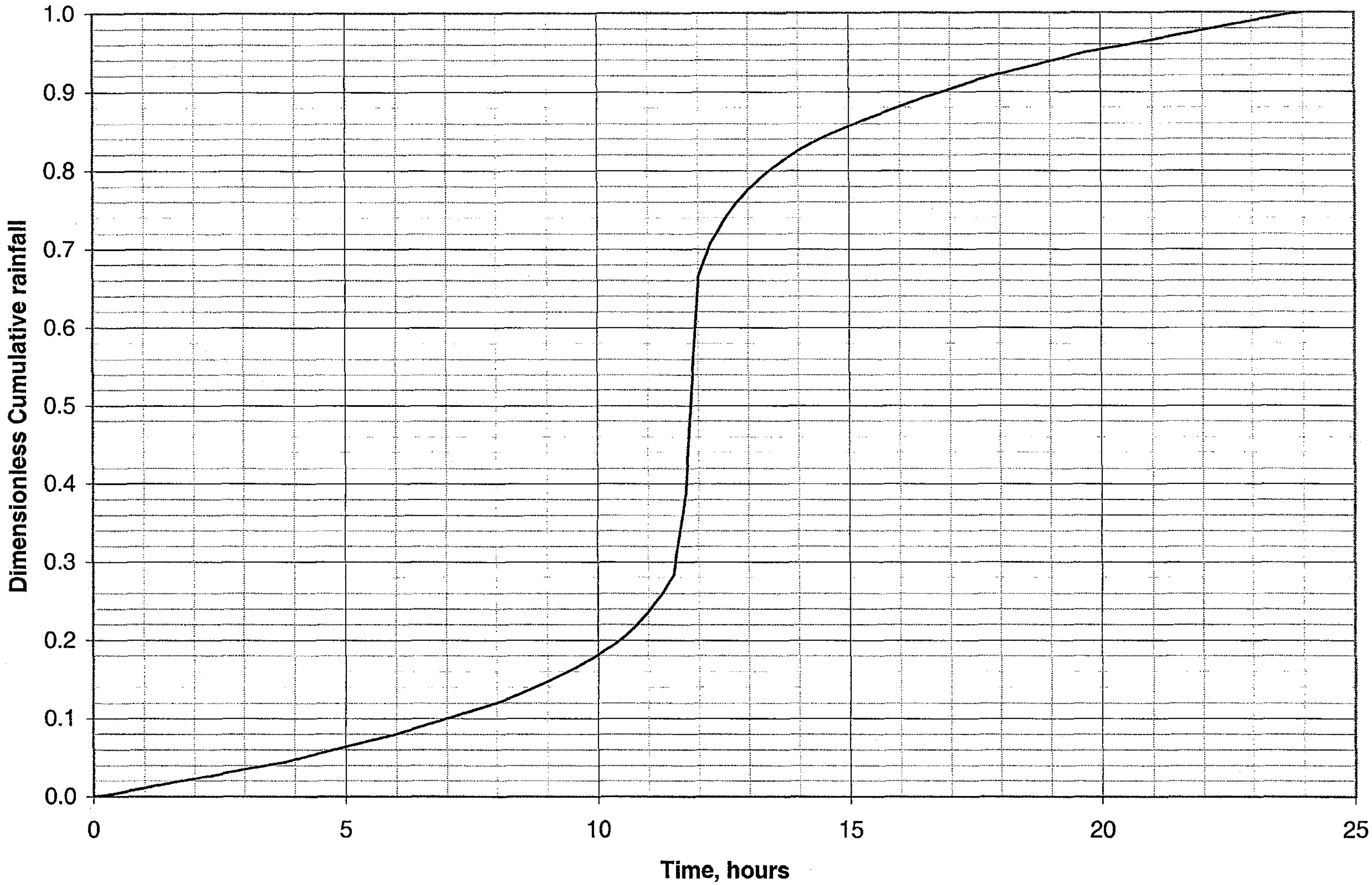


Figure 2.18
SCS Type II 24-hour Storm Rainfall Distribution

RAINFALL TABLE 2

STANDARD SCS 24-HOUR, TYPE II DISTRIBUTION
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

TABLE NO.	TIME INCREMENT
5 RAINFL 2	0.2500

8	0.0	0.0020	0.0050	0.0080	0.0110
8	0.0140	0.0170	0.0200	0.0230	0.0260
8	0.0290	0.0320	0.0350	0.0380	0.0410
8	0.0440	0.0480	0.0520	0.0560	0.0600
8	0.0640	0.0680	0.0720	0.0760	0.0800
8	0.0850	0.0900	0.0950	0.1000	0.1050
8	0.1100	0.1150	0.1200	0.1260	0.1330
8	0.1400	0.1470	0.1550	0.1630	0.1720
8	0.1810	0.1910	0.2030	0.2180	0.2360
8	0.2570	0.2830	0.3870	0.6630	0.7070
8	0.7350	0.7580	0.7760	0.7910	0.8040
8	0.8150	0.8250	0.8340	0.8420	0.8490
8	0.8560	0.8630	0.8690	0.8750	0.8810
8	0.8870	0.8930	0.8980	0.9030	0.9080
8	0.9130	0.9180	0.9220	0.9260	0.9300
8	0.9340	0.9380	0.9420	0.9460	0.9500
8	0.9530	0.9560	0.9590	0.9620	0.9650
8	0.9680	0.9710	0.9740	0.9770	0.9800
8	0.9830	0.9860	0.9890	0.9920	0.9950
8	0.9980	1.0000	1.0000	1.0000	1.0000
9 ENDTBL					

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0. The format for this table is Form #271, Page F-7.

ATTACHMENT 5

Table 5.?
Slope Adjustment For Steep Watercourses

Slope, feet/mile			
Unadjusted, S	Adjusted, S_{adj}	Unadjusted, S	Adjusted, S_{adj}
0	0	400	288
200	200	410	290
210	209	420	292
220	218	430	294
230	226	440	295
240	233	450	296
250	240	460	298
260	246	470	299
270	251	480	300
280	255	490	301
290	260	500	303
300	263	510	304
310	267	520	305
320	270	530	306
330	273	540	307
340	275	550	309
350	278	560	310
360	280	570	311
370	283	580	312
380	285	590	313
390	287	600	313

Equations for Data in Table 5.?

For 0<S<=200:
$S_{adj}=S$
For 200<Slope<=600:
$S_{adj}=a0+a1S+a2S^2+a3S^3+a4S^4+a5S^5+a6S^6+a7S^7$
a0= 6.725897827E+02
a1= -1.634093666E+01
a2= 1.739404649E-01
a3= -8.902683621E-04
a4= 2.552852266E-06
a5= -4.203532411E-09
a6= 3.721179614E-12
a7= -1.374400319E-15
For S>600, $S_{adj}=313$

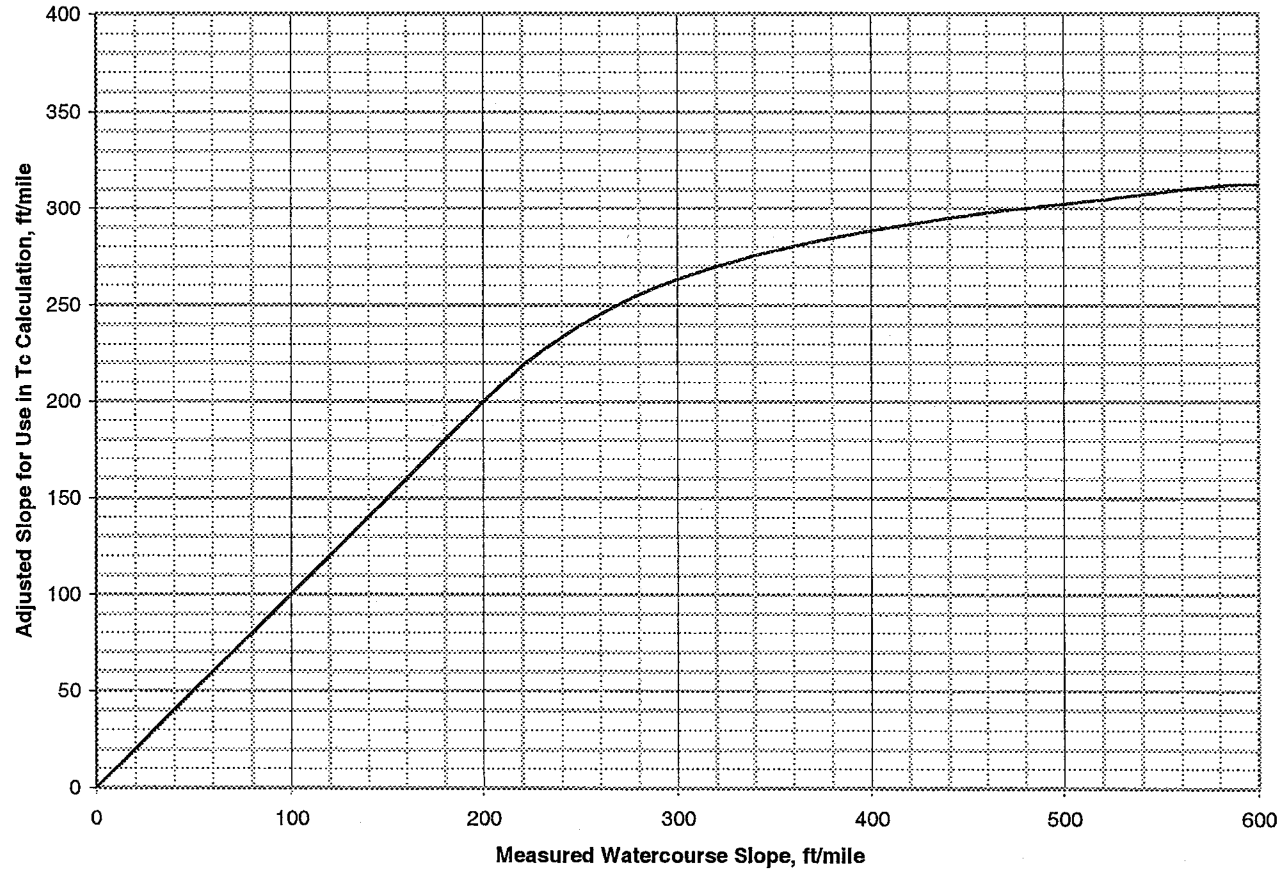


Figure 5.4
Slope Adjustment for Steep Watercourses in Natural Watersheds

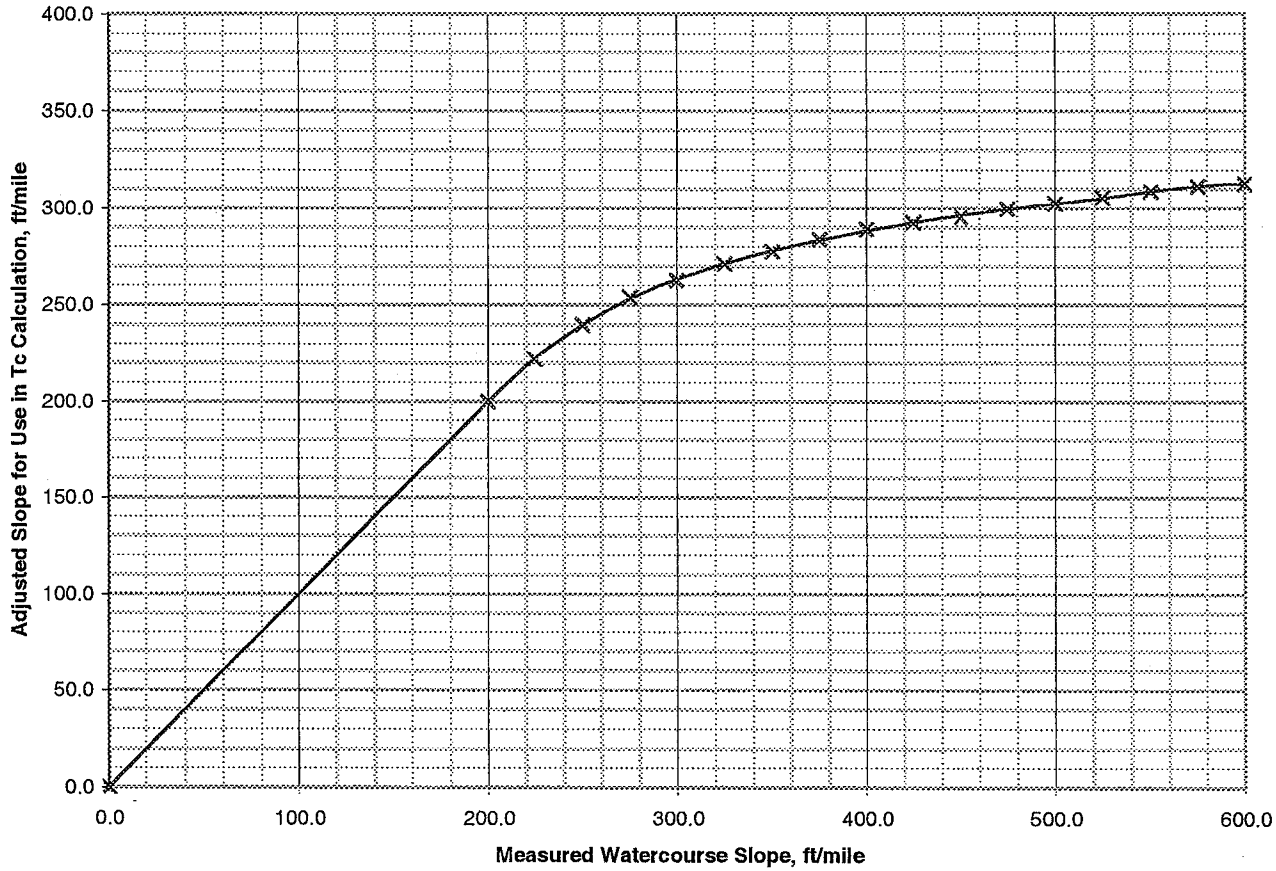


Figure 5.4 Comparison Figure
Slope Adjustment for Steep Watercourses in Natural Watersheds

× Scaled —— 7th Order Polynomial

Slope Adjustment Lookup Table For Steep Watercourses	
S	**S_{adj} (1-7th Order Polynomial)**
0	0
200	200
210	209
220	218
230	226
240	233
250	240
260	246
270	251
280	255
290	260
300	263
310	267
320	270
330	273
340	275
350	278
360	280
370	283
380	285
390	287
400	288
410	290
420	292
430	294
440	295
450	296
460	298
470	299
480	300
490	301
500	303
510	304
520	305
530	306
540	307
550	309
560	310
570	311
580	312
590	313
600	313

Slope Adjustment Values For Steep Watercourses (Scaled from Figure 5.4)	
S	**S_{adj}**
0.0	0.0
200.0	200.0
225.0	221.9
250.0	240.0
275.0	254.0
300.0	262.8
325.0	271.3
350.0	277.7
375.0	283.8
400.0	289.2
425.0	292.8
450.0	295.6
475.0	299.7
500.0	302.5
525.0	305.7
550.0	308.8
575.0	311.3
600.0	312.6

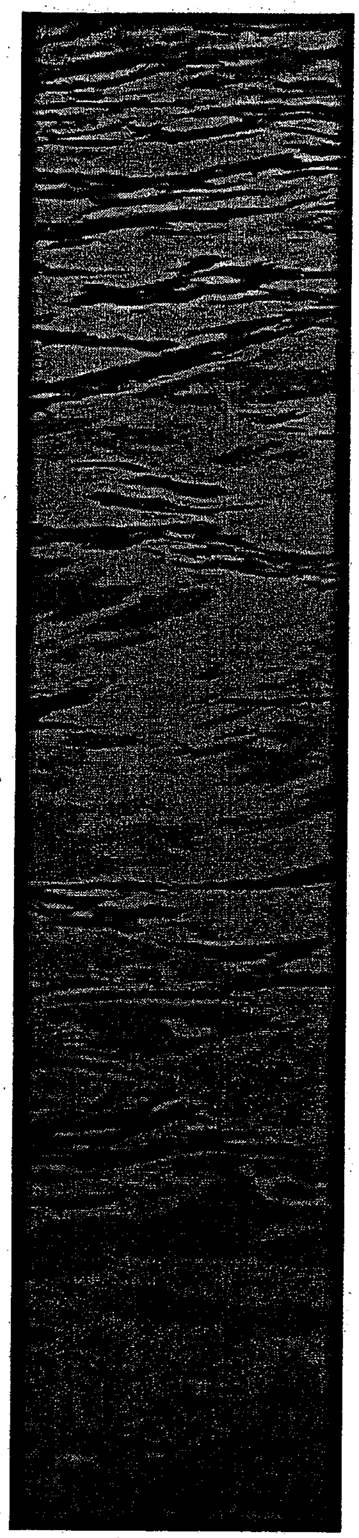
1-7th Order Polynomial Curve Fit Factors

For $0<S<=200$:
$S_{adj}=S$

$$S_{adj}=a0+a1S+a2S^2+a3S^3+a4S^4+a5S^5+a6S^6+a7S^7$$

For 200<Slope<=600:
a0= 6.725897827E+02
a1= -1.634093666E+01
a2= 1.739404649E-01
a3= -8.902683621E-04
a4= 2.552852266E-06
a5= -4.203532411E-09
a6= 3.721179614E-12
a7= -1.374400319E-15
For S>600, S_{adj}=313

ATTACHMENT 6



Maryvale Area Drainage Master Study
Hydrology

FCD #93-29

Prepared for

Flood Control District of Maricopa County

FEBRUARY 1997

Registered Professional Engineer (Civil)
CERTIFICATE NO.
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PATEL
Date Signed 2/7/97
ARIZONA, U.S.A.

Registered Professional Engineer (Civil)
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Prepared by

Wood, Patel & Associates, Inc.

in Association with

CH2M HILL

WOOD/PATEL

CIVIL ENGINEERS • HYDROLOGISTS • LAND SURVEYORS

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February 7, 1997

Mr. Russ Miracle, P.E.
Flood Control District of Maricopa County
2801 West Durango Street
Phoenix, AZ 85009

Re: **Maryvale ADMS**
Hydrology
FCD #93-29
WP #95154

Dear Mr. Miracle:

Enclosed please find four (4) sets of Final Hydrology Reports for the **Maryvale ADMS.**

This submittal, prepared by the Wood/Patel-CH2M Hill team, covers hydrologic analyses for the entire watershed within the ADMS. The HEC-1 model incorporated comments and suggestions from all of our previous submittals including the Pilot HEC-1 Study, Hydrologic Analysis First Phase, Hydrologic Analysis Entire Watershed, Draft Final Report, and Pre-Final Report.

Please note that the report (each set) is prepared in two separate 3-ring binders. The first binder is the main body of the report including drainage parameters, technical backup data, spreadsheets, computer disks, and exhibits. The second binder includes hard printouts of the results of our HEC-1 analysis.

Per our discussion, additional copies of this report are also being submitted separately to the following:

Ray Acuña, P.E. - 2 sets
City of Phoenix

Daniel A. Sherwood, P.E. - 2 sets
City of Glendale

Dan Nissen, P.E. - 2 sets
City of Peoria

This submittal concludes Phase I, Hydrology of our Scope of Work. We are now looking forward to initiating Phase II, Area Drainage Master Plan (ADMP). The ADMP will require site-specific flood mitigation study/solutions. We are ready to meet with the Cities of Phoenix, Glendale and Peoria to coordinate the ADMP. I will be contacting you soon to initiate the coordination meetings.

Wood, Patel & Associates, Inc. • 1550 East Missouri, Suite 203, Phoenix, Arizona 85014 • (602) 234-1344 • Fax (602) 234-1322
E-Mail: woodpatl@netzone.com

Mr. Amir Motamedi and you provided critical technical support and decision-making guidance throughout the study phase. These contributions represent a key role in the successful completion of the hydrology phase. We sincerely enjoyed these relationships and look forward to similar support through the ADMP phase.

Sincerely,

WOOD, PATEL & ASSOCIATES, INC.

Ashok C. Patel

Ashok C. Patel, P.E., R.L.S.
Principal

ACP/djp

Enclosures

cc: Steve Walker, P.E., CH2M Hill
David Dust, P.E., CH2M Hill

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Contents

Contents, Continued

Tables

Figures

Exhibits A Through S: (see Appendix L)

Exhibits 1 Through 6: (see Appendix M)

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1. Introduction

1.1 Scope

Wood, Patel & Associates, Inc. has been contracted by the Flood Control District of Maricopa County (District) to prepare the Maryvale Area Drainage Master Study (ADMS). CH2M HILL assisted Wood, Patel and Associates, Inc. by preparing the "existing conditions" hydrologic analyses for the approximately 100-square-mile Maryvale ADMS study area.

The primary purposes of this report are to:

- Document the methodologies and procedures used to develop the HEC-1 models for the Maryvale ADMS study area, under existing conditions.
- Document the results of the hydrologic analyses.

1.2 Objectives

To allow the identification and quantification of flood hazards within the study area, the primary objectives of the hydrologic modeling component of the Maryvale ADMS project were to develop HEC-1 models that:

- Provide runoff computations for a primarily urban drainage area, where sub-basin drainage boundaries are not well defined and flow paths are very complex.
- Evaluate 10-year, 6-hour; 100-year, 6-hour; and 100-year, 24-hour rainfall events.
- Allow efficient evaluation of both existing and future storm drain systems.
- Provide a cost-effective planning and analysis tool.
- Allow for efficient updating.

The HEC-1 modeling approach documented in this report meets these objectives and the criteria specified in the project's scope of work. It is anticipated that the HEC-1 model created for the Maryvale ADMS will be a useful and cost-effective planning tool.

1.3 Study Area Description

The study area for the Maryvale ADMS is approximately 100 square miles in size and encompasses portions of the City of Peoria, the City of Glendale, the City of Avondale, the City of Tolleson, the City of Phoenix, and un-incorporated Maricopa County. As indicated in Figure 1-1, the study area is bounded on the north by the Arizona Canal Diversion Channel (ACDC) and Skunk Creek, on the east by I-17, on the south by I-10, and on the west by the Agua Fria River and the Agua Fria Freeway.

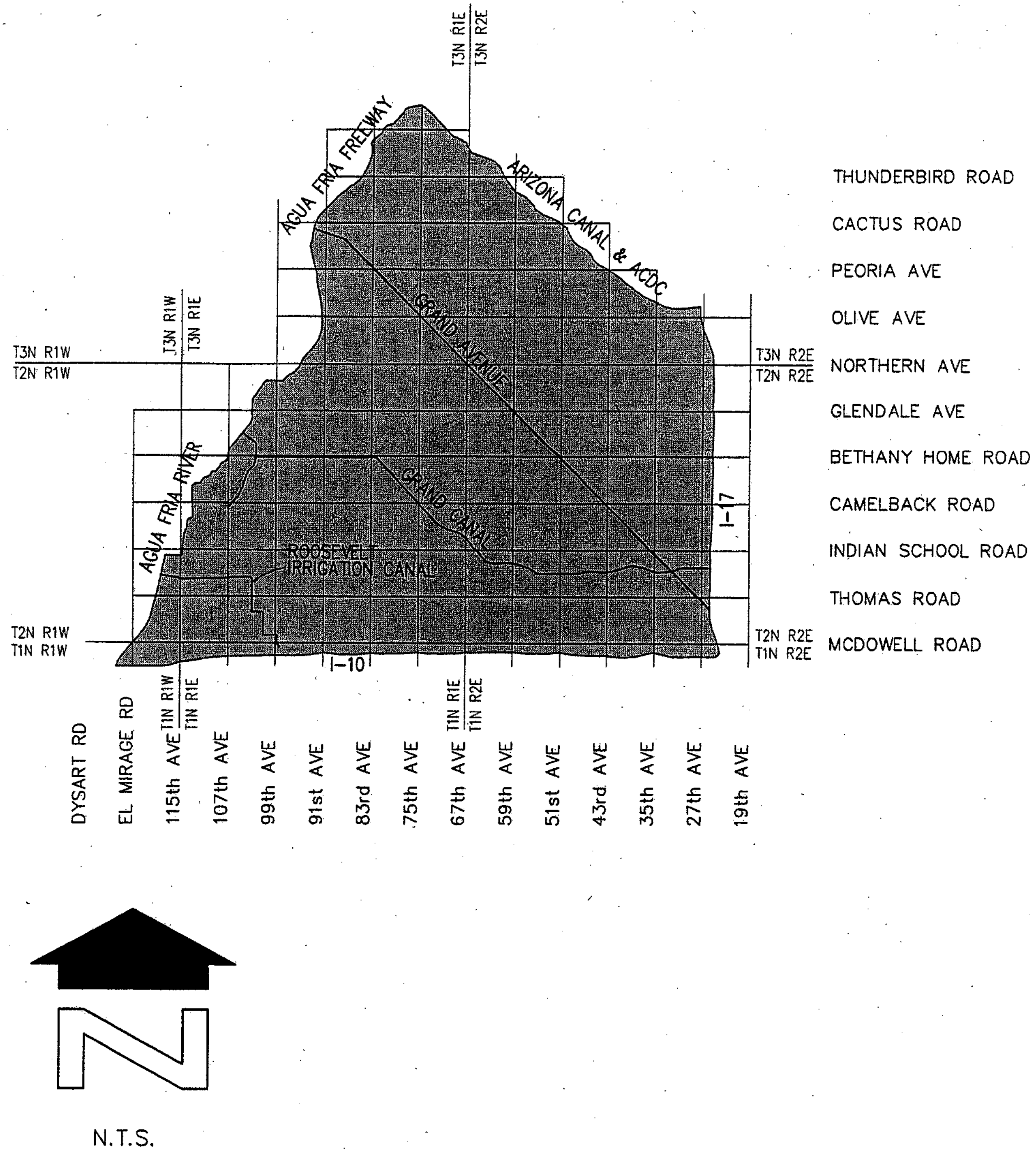

Figure 1-1: STUDY AREA MAP

CH2M HILL

The study area can be characterized as follows.

- **Slope.** The slope of the land is generally from the northeast to the southwest at approximately 0.4 percent.
- **Land Uses.** The primary land use with the study area is single family residential. However, essentially all of the study area had been agricultural land at some point in the past.
- **Primary Flooding Locations.** During significant rainfall events, stormwater ponds at several locations along the upstream side of the Roosevelt Canal, the Grand Canal, and Grand Avenue and/or the adjacent railroad embankment. However, flooding also occurs at numerous intersections and along several major streets.
- **Existing Channel Systems.** The ACDC and Skunk Creek define the northern boundary of the study area. The ACDC and Skunk Creek are regional flood control facilities, with design capacities greater than the 100-year event. Within the study area, two major flood control channel systems were built and are maintained by the Arizona Department of Transportation (ADOT), as illustrated in Figure 1-2. These channel systems extend along the east side of the Agua Fria Freeway and along the north side of I-10. The existing ADOT channel system along the east side of the Agua Fria Freeway extends from approximately Greenway Road to Glendale Avenue. This channel system outfalls at several locations into the New River and collects both surface and storm drain flow. The ADOT channel system, along the north side of I-10, outfalls into the Agua Fria River and collects both surface and storm drain flow.
- **Existing Storm Drain Systems (Peoria and Glendale).** There are three major storm drain systems in the northern portion of the study area, as shown schematically in Figure 1-2. The Cactus Road storm drain system currently extends from approximately 67th Avenue west to 83rd Avenue. The Cactus Road storm drain system will be extended to the ADOT channel system along the east side of the Agua Fria Freeway, with an expected completion date of July 1997. The Olive Avenue storm drain system extends from approximately 51st Avenue west to the ADOT channel system, along the east side of the Agua Fria Freeway. The Olive Avenue storm drain system has two surge basins located at approximately 71st Avenue and 83rd Avenue. The Olive Avenue storm drain system has a 10-year event design capacity. The Peoria Avenue storm drain system extends from approximately Grand Avenue to the Agua Fria Freeway channel system.
- **Existing Storm Drain Systems (Phoenix).** As shown in Figure 1-2, the City of Phoenix has a complex system of storm drains that convey stormwater from as far north as Northern Avenue south to the Salt River. Primary components of this system include storm drains along 27th, 35th, 39th, 43rd, 51st, 67th, 75th, 83rd, and 91st Avenues. The design capacities for these storm drain systems are generally estimated to be between the 2- and 10-year events.

- **Future Storm Drain Systems.** Several future storm drain systems were included in the HEC-1 model for "existing conditions." These future systems include storm drains along Butler Drive, Northern Avenue, and Orangewood/Glendale Avenues, as indicated in Figure 1-2. In addition, the Cactus Road storm drain system was assumed to be complete and, therefore, extend to the Agua Fria Freeway channel.

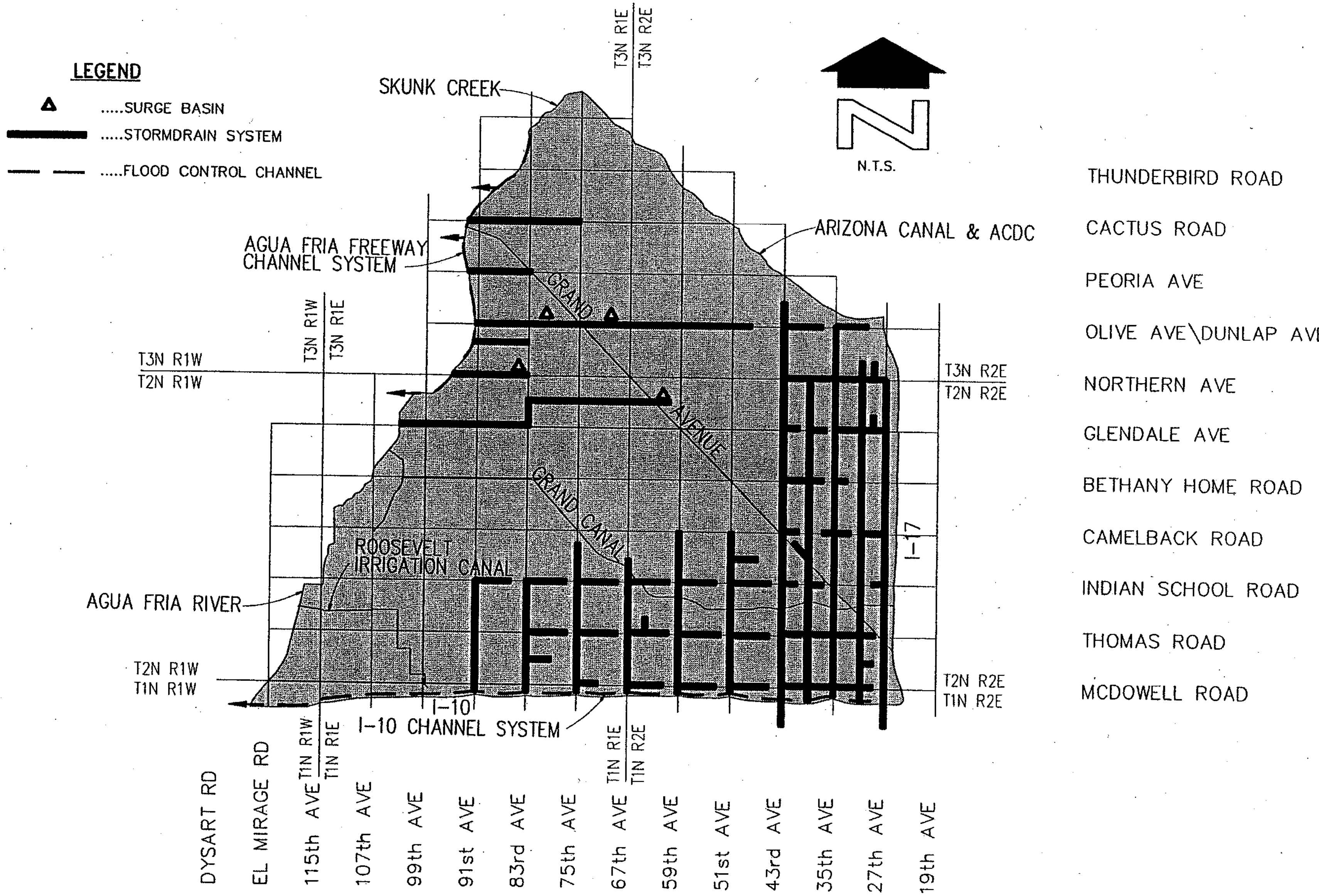

Figure 1-2: SCHEMATIC OF STORMDRAIN SYSTEMS

2. A Brief Chronology of Previous Studies

Reports of the flooding of agricultural fields and of flows overtopping canal banks date back to the 1940's, when agriculture was essentially the only land use in the Maryvale ADMS study area. Even as late as the 1960s, agriculture was still the primary land use within the study area (FCDMC, 1962). In the 1960s, the U.S. Army Corps of Engineers documented, in detail, the storm and flood event of August 16, 1963. During the 1963 storm event, the Corps documented flooding of both residential and commercial properties along primarily the Grand Canal.

Also in the early 1960s, the District prepared the "Flood Control Survey Report"(FCDMC, 1962) and the "Comprehensive Flood Control Program Report"(FCDMC, 1963). These studies identified and documented the flood hazards along Grand Avenue and the canal systems. The latter of these two studies documents plans for several regional flood control facilities, including the ACDC, the New River Dam, and Adobe Dam. Many of the regional flood control facilities identified in the 1963 study were designed and built in the 1970s, 1980s, and the early 1990s.

In the late 1980s, the Arizona Department of Transportation evaluated flood hazards and drainage requirements for the future Outer Loop Highway (WLB, 1987), and future highway improvements along Grand Avenue (Tudor, 1989). Also in the 1980s, the District prepared the "Glendale-Peoria Area Drainage Master Plan"(Camp Dresser and McKee et. al., 1987). This study evaluated flooding hazards and flood control alternatives for an area located within the study area for the Maryvale ADMS. The flood control alternatives evaluated in this study primarily involved networks of storm drain systems.

In the early 1990s, the District sponsored studies that evaluated various storm drain alternatives along Cactus Road, Butler Drive, Northern Avenue, and Orangewood/Glendale Avenue (SFC, 1992) and (WPA, 1995). In 1995, the District had flooding hazards and flood control alternatives evaluated for the flooding area along the Grand Canal, between approximately 35th and 67th Avenues (CVL, 1995). This study evaluated storm drain and retention basin alternatives for relieving flooding along the north side of the canal.

The Maryvale ADMS is intended to address the drainage issues for a study area that essentially encompasses the study areas of the previous studies. Within this study area, there is a complex network of existing drainage facilities. In addition, there are several future flood control systems at various levels of planning and design. Hence, an important aspect of this study was to develop a hydrologic model that is flexible and adaptable.

3. Hydrologic Analyses

3.1 HEC-1 Modeling Approach

To meet the primary objectives of the hydrologic modeling component of the Maryvale ADMS project, it was essential to develop a HEC-1 modeling approach that is adaptable. The HEC-1 modeling approach used in this study was developed by first evaluating the drainage patterns within a 3.8-square-mile pilot study area.

The pilot study area selected appeared to be reasonably representative of the highly urbanized portion of the total 100-square-mile study area. Detailed evaluation of the detailed topographic mapping and site conditions for the pilot study area indicated that:

- Arterial streets are very important conveyors of storm water; however, the capacity of the arterial streets will typically be exceeded during the 10-year event. Stormwater flows will exit typical arterial streets as weir flow down side streets.
- Since the arterial streets are the primary conveyors of stormwater, it is logical to consider the approximately 1-square-mile area bounded by arterial streets as a sub-basin. However, it is important to recognize that stormwater can cross all four sides of the essentially square sub-basin and that the sub-basin does not have a single concentration point. Instead, each sub-basin has a concentration line that extends along the arterial streets along the downstream sides of the essentially square sub-basin.
- The flow patterns within a typical 1-square-mile sub-basin are very complex and can vary significantly during the course of a rainfall event. However, the flow patterns within the sub-basin are primarily controlled by residential and collector street patterns.

Figure 3-1 is a HEC-1 schematic diagram for a typical sub-basin. This figure illustrates the key elements and structure of the HEC-1 modeling approach used in this study. As indicated in Figure 3-1, the HEC-1 model for a typical sub-basin includes the following standard elements:

- Hydrograph computations for the sub-basins per the District's methodologies.
- Normal depth channel routings for the arterial streets.
- Normal depth composite-channel routings for flow passing through a sub-basin along multiple streets.
- In some cases, reservoir routings for ponding areas and regional retention basin systems within a sub-basin.
- Hydrograph combines along streets and within sub-basins.

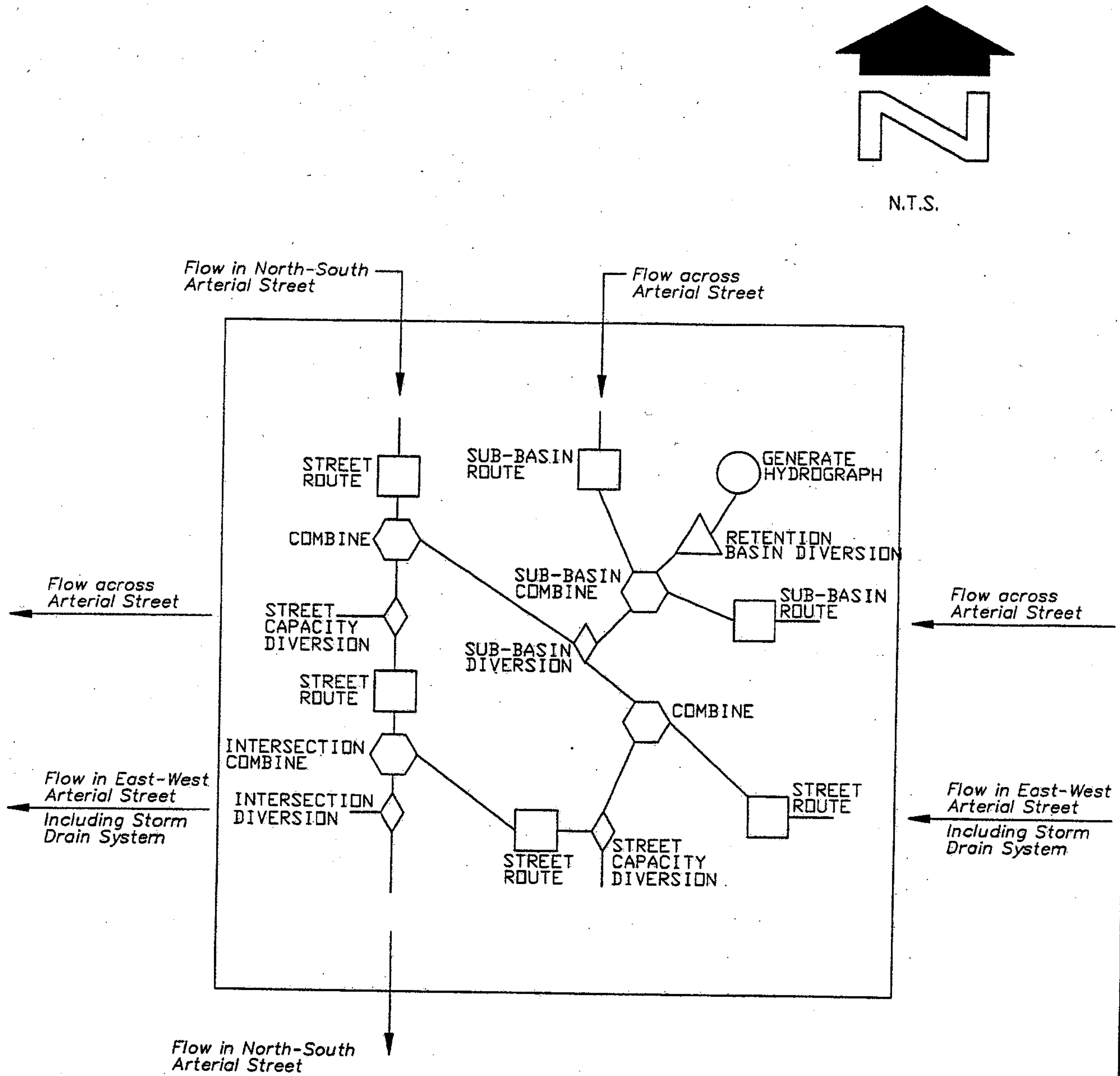

Figure 3-1: HEC-1 SCHEMATIC FOR A ONE SQUARE MILE SUB-BASIN

CH2M HILL

However, the uniqueness of the proposed HEC-1 modeling approach lies in its structure and the use of various categories of flow diversions. The ability to model flow diversions flexibly was the key for developing a modeling approach for the study area that meets the project objectives. As indicated in Figure 3-1, flow diversions are used in five distinctly different ways:

1. **Street Capacity Diversions.** Street capacity diversions are used to compute the flow being conveyed by a major/arterial street and the flow that exits the arterial street and enters the adjacent sub-basin via both residential and collector streets.

2. **Street Intersection Diversions.** Street intersection diversions are used to distribute the flow passing through an intersection. Intersection diversion rating curves for arterial streets are based on relative street capacities and the capacity of existing or future storm drain systems.

3. **Sub-basin Diversions.** Sub-basin diversions are used to identify the southerly and westerly components of runoff generated within a sub-basin and flow entering a sub-basin from adjacent arterial streets. Rating curves for these diversions are primarily based on easily measured areas; however, some judgment is required.

4. **Retention Basin Diversions.** In cases where a portion of a sub-basin drains to a retention basin or system of retention basins, flow diversions are used to divert the volume of water corresponding to the measured capacities of the retention basins. The percentage of the flow rate that can be diverted (i.e., the DQ-record information) corresponds to the percentage of the sub-basin area that drains to the retention basin(s).

5. **Surge Basin Diversions.** The Olive Avenue, Northern Avenue, and Orangewood/Glendale Avenue storm drain systems have surge basins, as indicated in Figure 1-2. Flow diversions are used to simulate the diversion of flow from a storm drain system into a surge basin.

The schematic for the Maryvale ADMS HEC-1 model is shown in Exhibit 1. The computational sequence of the HEC-1 model has been set up to minimize the number of hanging hydrographs and emphasize the east-west streets. Emphasizing or following east-west streets simplifies the task of modifying the model to reflect storm drain alternatives that may extend along the east-west streets. Yet, the structure of the model is such that the computational path for the north-south streets is relatively easy to identify and follow.

In addition, the structure of the HEC-1 modeling approach is such that:

- Flow can be routed into and out of a typical 1-square-mile sub-basin across essentially all four sides of the sub-basin.

- Flow can be routed along all four sides of a typical 1-square-mile sub-basin.

- Street intersection, street capacity, and sub-basin diversions can be modified to reflect future or existing storm drain systems, along arterial and collector streets.

- Retention basin diversions can be added and modified to reflect future retention facilities.

As indicated in Exhibit 1, the HEC-1 element names are based on section numbers and street names. A detailed description of the element name nomenclature is provided in Appendix B and in the ID-records of the HEC-1 data sets.

3.2 HEC-1 Input Data Development

3.2.1 General

The input parameters for the Maryvale ADMS HEC-1 models have been measured from or are primarily based on the following sources of data:

- Detailed topographic mapping (i.e., 1"=200' with a contour interval of 1 or 2 feet) prepared by Kenney Aerial Mapping, based on photography flown on March 28, 1994.
- 1990/1991 land use data provided in GIS format by the District.
- Soil type data, based on the *Soil Survey of Maricopa County, Arizona: Central Part* (SCS, 1977), as provided in GIS format by the District.
- NOAA Atlas II precipitation data as documented in *Drainage Design Manual for Maricopa County: Volume I: Hydrology* (FCDMC, Jan. 1995).

The following sections of this report describe the specific sources of data and the techniques used to develop the HEC-1 input data. However, detailed documentation and computation sheets for the various components of the HEC-1 model are provided in the appendix as follows:

Appendix A:	Precipitation Data
Appendix B:	Sub-basin Parameters
Appendix C:	Sub-basin Diversions
Appendix D:	Arterial and Composite Street Routes
Appendix E:	Retention Basin and Ponding Area Data
Appendix F:	Street Capacity Diversions
Appendix G:	Street Intersection Diversions
Appendix H:	Storm Drain and Surge Basin Data
Appendix I:	Cumulative Area Computations for Hydrograph Combines
Appendix J:	I-10 Freeway Channel Analysis
Appendix K:	HEC-1 Element Nomenclature, HEC-1 Output and Input Files

3.2.2 Rainfall Event Parameters

3.2.2.1 Precipitation Data

Adjusted point rainfall depths for the 10-year, 6-hour; 100-year, 6-hour; and the 100-year, 24-hour events were computed for the study area. The point rainfall values were computed using the District's DDMS/PREFRE software package and the isopluvial maps, as documented in the *Drainage Design Manual for Maricopa County: Volume I Hydrology* (FCDMC, 1995). The point rainfall depth computations are documented in Appendix A.

3.2.2.2 Rainfall Distributions

Rainfall events with 6- and 24-hour rainfall distributions were evaluated for the study area.

- **6-Hour Rainfall Distributions.** The dimensionless storm patterns documented in the *Drainage Design Manual for Maricopa County: Volume I Hydrology* (FCDMC, 1995), were used in this study.

- **24-Hour Rainfall Distribution.** The SCS Type II distribution was used in this study to model the 100-year, 24-hour event.

3.2.2.3 Multiple versus Single Storm Event

The District's DDMS software allows computation of sub-basin parameters (i.e. Clark Unit Hydrograph Parameters "Tc" and "R") for either "multiple" or "single" storm events. When the single storm option is used, the HEC-1 model has a single point rainfall value and a single rainfall distribution, that corresponds to a specified drainage area. Whereas, the "multiple storm" HEC-1 model uses point rainfall values and rainfall distributions, from the multiple point rainfall values and distributions listed in the HEC-1 data set, based on the drainage area specified for each sub-basin.

The multiple storm option has been used to model the general "existing conditions." However, there may be applications of the basic Maryvale ADMS HEC-1 Model, where the single storm option may be more appropriate.

3.2.3 Sub-basin Parameters

3.2.3.1 Sub-basin Boundaries

As indicated in Figure 1-1, the study area encompasses approximately 100 square miles. As shown in Exhibits 1 and 2, the study area has been delineated into 141 sub-basins in a manner consistent with the HEC-1 modeling approach described in Section 3.1 of this report. The sub-basin parameter data collected for the study area is documented in detail in Appendix B.

3.2.3.2 Land Use and Soil Data

The District provided land use and soil map data in GIS format. The land use and soil map data are shown in Exhibits 3 and 4. GIS software was used to compute the sub-basin areas, the area of each soil unit in each sub-basin, and the area of each land use category in each sub-basin. This information was used as input data for the DDMS software. The various parameters assigned to each land use type are as given in Table 3-1. The percent impervious specified for each land use (i.e. RTIMP) was assumed to be 100% effective.

3.2.3.3 Unit Hydrograph

The Clark Unit Hydrograph option in HEC-1 was used for all sub-basins. The HEC-1 input parameters (UA and UC-records) were generated using the District's DDMS software. The DDMS sub-basin parameter data for the study area are given in Appendix B

3.2.3.4 Precipitation Losses

The Green-Ampt precipitation loss option was used for all sub-basins. Green-Ampt parameters for each sub-basin were computed using the DDMS software, based on the land use and soil data.

3.2.3.5 Time of Concentration Flow Paths

Time of concentration flow path data was determined for each sub-basin using the detailed topographic mapping. The time of concentration flow paths for each sub-basin are shown in Exhibit 2.

TABLE 3-1
Land Use Parameters

Land Use Type	DTHETA Condition	% Veg. Cover	RTIMP Percent	IA Inches	Kn	Kb Roughness Type
Desert	Dry	25	0	0.350	0.030	Low
Open	Dry	10	0	0.200	0.020	Min
VLDR	Normal	30	5	0.300	0.050	Hi
LDR	Normal	50	15	0.300	0.050	Hi
MDR	Normal	50	30	0.250	0.050	Hi
MFR	Normal	50	45	0.250	0.050	Hi
Ind	Normal	60	55	0.150	0.030	Min
Comm	Normal	75	50	0.100	0.020	Min
Park	Normal	90	10	0.200	0.100	Hi
Rowcrop	Normal	85	0	0.500	0.100	Hi
School1	Normal	80	45	0.290	0.050	Hi

3.2.4 Sub-basin Diversions

Within the study area, the slope of the land is generally from the northeast to the southwest at approximately 0.4 percent. Hence, stormwater flowing within a typical sub-basin is directed and conveyed to the arterial streets along the west and south sides of the sub-basin, by residential and collector streets. That is, some of the stormwater is conveyed west to the north-south arterial street, by residential and collector streets, while the remainder of the stormwater is conveyed south to the east-west arterial street by residential and collector streets.

Sub-basin diversions are used to separate or divide the hydrograph, for the stormwater within a sub-basin, into a southerly and a westerly component. The hydrograph for the stormwater within a sub-basin includes runoff generated within a sub-basin and typically the flow entering a sub-basin from the adjacent arterial streets.

Sub-basin diversion data have been computed based on the drainage patterns within each of the sub-basins. The drainage patterns within the sub-basins have been evaluated using the topographic mapping for the study area and site observations. The sub-basin diversion data computations are documented in Appendix C.

3.2.5 Arterial and Sub-basin Street Routes

3.2.5.1 Arterial Street Routes

The Normal-Depth Channel Routing option in HEC-1 is used for arterial street routes. The input data for arterial street routes are based on a typical arterial street section and measured street slopes and reach lengths. Detailed documentation for each arterial street route is provided in Appendix D.

3.2.5.2 Sub-basin Street Routes

The Normal-Depth Channel Routing option in HEC-1 is used for sub-basin street routes. The input data for sub-basin street routes was based on a composite section and measured street slopes and reach lengths. Detailed documentation for each sub-basin street route is provided in Appendix D.

3.2.6 Retention Basin and Ponding Area Data

3.2.6.1 Retention Basin Data

In cases where a portion of a sub-basin drains to a retention basin or system of retention basins, flow diversions are used to divert the volume of water corresponding to the measured capacities of the retention basins. The percentage of the flow rate that can be diverted (i.e., the DQ-record information) corresponds to the percentage of the sub-basin area that drains to the retention basin(s). The retention basin computation sheets for each sub-basin are given in Appendix E.

3.2.6.2 Ponding Area Data

During significant rainfall events, stormwater ponds at several locations along the upstream side of the Roosevelt Canal, the Grand Canal, and Grand Avenue and/or the adjacent railroad embankment. Storage volume, ponding elevation, and outfall data were computed based on the detailed topographic mapping and supplemental surveyed spot elevations. The ponding area computation sheets are given in Appendix E.

3.2.6.3 Maryvale Mitigation Area

Within the Maryvale Mitigation Area study limits, the ponding areas have been modeled based primarily on the data documented by CVL (1995) ; however, this data has been revised to reflect preliminary analyses of the conveyance capacity along the north side of the Grand Canal and additional storm drain capacity computations. The data used to develop the HEC-1 input data for the ponding areas and corresponding diversions are provided in Appendix E and Appendix H.

3.2.7 Street Capacity Diversions

Street capacity diversions are used to compute the flow being conveyed by arterial street and the flow that exits the arterial street and enters the adjacent sub-basin via both residential and collector streets. Two general cases of street capacity diversions have been evaluated. The typical case (Case "1") is when flow exits the arterial street as weir flow into the side streets. Along most arterial streets, the side streets have been designed with a high point or grade break near the intersection. This prevents stormwater flowing in the gutter of the arterial from being directly conveyed into the side street. However, a special case, that occurs at several locations, is when stormwater can exit the arterial street directly into the side street. In this case, normal depth calculations are use to develop rating data for both the arterial and side street(s). As indicated in the detailed documentation given in Appendix F, spreadsheets have been used to compute the rating curves for the street capacity diversions based on easily measured physical parameters and future/existing storm drain system capacities.

3.2.8 Street Intersection Diversions

Street intersection diversions are used to distribute the flow passing through an intersection. Intersection diversion rating curves for arterial streets are based on relative street capacities and the capacity of existing or future storm drain systems. Two general types of street intersections have been evaluated. *Type A--Arterial Street Intersections* are the typical type of arterial street intersections within the study area. The characteristics of a Type A intersection are as follows:

- Continuous pavement crowns in both directions; that is, valley gutters are **not** used to direct stormwater through the intersection.
- Street slopes in the vicinity of the intersection are relatively mild (i.e., less than 1%).
- Intersection is subject to low velocity inundation.

The special case or *Type B--Arterial Street Intersections* refers to the condition where the pavement crown is continuous in only one direction through the intersection. In this case, the intersection diversion is based on normal depth and weir flow computations. The intersection diversion data collected for the study area are documented in detail in Appendix G.

3.2.9 Storm Drain Systems

3.2.9.1 Existing Storm Drain Systems within the Cities of Peoria and Glendale

There are three major storm drain systems in the northern portion of the study area, as shown schematically in Figure 1-2. The Cactus Road storm drain system currently extends from approximately 67th Avenue west to 83rd Avenue. The Cactus Road storm drain system will be extended to the ADOT channel system along the east side of the Agua Fria Freeway, with an expected completion date of July 1997. The Olive Avenue storm drain system extends from approximately 51st Avenue west to the ADOT channel system along the east side of the Agua Fria Freeway. The Olive Avenue storm drain system has two surge basins located at approximately 71st Avenue and 83rd Avenue. The Olive Avenue storm drain system has a 10-year event design capacity. The Peoria Avenue storm drain system extends from approximately Grand Avenue to the Agua Fria Freeway channel system.

The Peoria Avenue, Cactus Road, and Olive Avenue storm drain systems have been incorporated into the HEC-1 model, by modification of the appropriate street intersection and street capacity diversion rating curves. The Cactus Road storm drain system has been modeled assuming the system is complete and outfalls to the Agua Fria Freeway Channel. Storm drain capacity data for these systems are based on the capacities given in the design reports for these storm drain systems. The storm drain capacity data and detailed documentation describing how the storm drain systems are reflected in the HEC-1 model are provided in Appendix H.

3.2.9.2 Existing Storm Drain Systems within the City of Phoenix

As shown schematically in Figure 1-2, the City of Phoenix has a complex system of storm drains that convey stormwater from as far north as Northern Avenue south to the Salt River. Primary components of this system include storm drains along 27th, 35th, 39th, 43rd, 51st, 67th, 75th, 83rd, and 91st Avenues. The design capacities for these storm drain systems are generally estimated to be between the 2- and 10-year event.

The capacities of the storm drain systems located within the City of Phoenix have been estimated based on pipe slopes and diameters, assuming pipe full conditions. The capacity computations for these storm drain systems are given in Appendix H.

3.2.9.3 Future Storm Drain Systems Included In "Existing Conditions"

Several future storm drain systems were included in the HEC-1 model for "existing conditions." These future systems include:

- The storm drain systems evaluated and proposed in the "Northern /Orangewood Storm Drain Project: Concept Routing Study FCD #94-12" by Wood, Patel & Associates(1996). The study proposed systems along Butler Drive, Northern Avenue, and Orangewood/Glendale Avenues, as indicated in Figure 1-2. The storm drain capacity data for these storm drain systems are given in Appendix H.

- Extension and completion of the Cactus Road storm drain system. The Cactus Road storm drain system will be extended to the ADOT channel system along the east side of the Agua Fria Freeway, with an expected completion date of July 1997.

3.2.10 Cumulative Area Computations for Hydrograph Combines

When hydrographs generated from diverts are combined, HEC-1 requires a drainage area specified on the HC-record. This area is used to compute an interpolated hydrograph for the "combined hydrograph," based on the data given in the JD-records (HEC,1990).

For this study, areas have been computed for each combine node based on the total area of all the sub-basins located upstream of the combine node. These "Cumulative Area Computations," given in Appendix I, list the areas and names for all of the upstream sub-basins for each combine node.

The drainage area specified for each of the combine nodes represents the maximum drainage area that may contribute flow to the combine node. It is recognized that a combine node may only receive a small fraction or none of the runoff hydrograph for some of the upstream sub-basins.

3.3 Data Management

The HEC-1 model for the Maryvale ADMS study area is composed of approximately 12,320 lines of data. The model includes approximately:

- 140 Sub-Basins.
- 400 Arterial Street Routes.
- 330 Composite Street Routes.
- 200 Street Capacity Diversions.
- 140 Sub-Basin Diversions.
- 90 Intersection Diversions.
- 30 Retention Basin Diversions.
- 24 Ponding Areas (i.e., reservoir routes).
- 80 Miles of Storm Drain Systems.

When possible, Excel spreadsheets have been used to compute and manipulate the data required to generate the input data for the HEC-1 model. As provided in the appendices, listings of these spreadsheets have also been used to document the input data development.

As indicated in Exhibits 3 and 4, developing the sub-basin data for the HEC-1 model involved the manipulation of a large amount of soil and land use data. GIS software, more specifically ArcInfo, was used to compute the total area for each Sub-Basin and the portions thereof within each of the soil groups and land use categories.

4. Evaluation of Study Results

4.1 Results of Hydrologic Modeling

The computed peak discharges for the 10-year, 6-hour and the 100-year, 6-hour events are summarized in Exhibits 5 and 6, respectively. The computed peak discharges indicated in Exhibits 5 and 6 represent surface and/or storm drain flows. The complete HEC-1 input and output files are provided in Appendix K.

As illustrated in Exhibits 5 and 6, it is important to note that the HEC-1 models do not concentrate extremely large flows in the arterial streets. That is, the structure of the HEC-1 model results in the computed runoff being distributed amongst the arterial streets , the residential/collector streets, and storm drain systems. .Based on site observations and the topographic mapping, the distribution of the runoff amongst the HEC-1 flow paths appears to be consistent with the anticipated flooding patterns.

4.1.1 Flood Prone Areas Along 35th and 27th Avenues

It is apparent in Exhibit 1 that the typical sub-basin schematic, as illustrated in Figure 1-3, is appropriate for the vast majority of the study area. However, some of the sub-basins located adjacent to 35th or 27th Avenues do not drain southwest, as implied in the typical sub-basin schematic. North of Grand Avenue and South of Olive Avenue, grade breaks and low lying areas cause 35th and 27th Avenues to collect and convey storm water directly south. Along 35th and 27th Avenues, storm water is conveyed as surface and/or storm drain flows. Review of the detailed topographic mapping indicates that the gutter elevations in 27th Avenue are higher than adjacent areas in several locations. This is specifically the case along the west side of 27th Avenue from Maryland Avenue to Missouri Avenue. The results of this study indicate that several low lying areas adjacent to 35th and 27th Avenues are prone to flooding.

4.1.2 I-10 Freeway Channel

The I-10 Freeway and drainage channel define the southern boundary of the study area. The channel system along the north side of the freeway collects and conveys storm water westerly to the Agua Fria River. The channel system collects surface flows and flows from the City of Phoenix's storm drain systems.

The capacity of the I-10 Freeway channel has been assessed on a simple level. In Table 4-1, estimated water surface elevations are compared with top-of-bank elevations at several locations along the channel. Flow depths in the I-10 Freeway channel have been estimated based on uniform flow computations; even though, backwater flow conditions are anticipated in the channel, during major flood events. The results of the analyses, summarized in Table 4-1, indicate that the I-10 Freeway Channel may not have capacity for the 100-year event in the vicinity of 83rd and 91st Avenues.

Table 4-1: Comparison of Computed Peak Discharges and Capacity of I-10 Freeway Channel

Summary: This table indicates that the I-10 Freeway Channel may not have capacity for the 100-Year event in the vicinity of 83rd and 91st Avenues.

ADOT: I-10 Freeway Channel Configuration Data[2]						10-Year, 6-Hour Event			100-Year, 6-Hour Event			100-Year, 24-Hour Event		
Location[1]	Approx. Invert Elev.	Approx. Channel Slope (ft/ft)	Approx. Bottom Width (ft)	Approx. Side Slope	Approx. Top of Bank Elev.	Q (cfs)	Est. Flow Depth (ft) [3]	Est. Water Surface Elev.	Q (cfs)	Est. Flow Depth (ft) [3]	Est. Water Surface Elev.	Q (cfs)	Est. Flow Depth (ft) [3]	Est. Water Surface Elev.
43rd Ave	1055.4	0.00100	10	2:1	1070	1950	9.1	1065	4140	12.6	1068	3620	11.9	1067
51st Ave	1051.0	0.00056	10	2:1	1070	2620	11.8	1063	5430	16.1	1067	5030	15.6	1067
59th Ave	1038.0	0.00044	10	2:1	1060	3080	13.3	1051	5770	17.4	1055	5520	17.1	1055
67th Ave	1034.0	0.00194	20	2:1	1050	3490	8.5	1043	6560	11.5	1046	6390	11.4	1045
75th Ave	1023.9	0.00177	20	2:1	1044	4040	9.3	1033	7550	12.6	1037	7410	12.5	1036
83rd Ave[4]	1013.5	0.00020	20	2:1	**1032**	4150	15.9	1029	**7700**	**21.0**	**1035**	**7920**	**21.3**	**1035**
91st Ave[4]	1003.4	0.00056	20	2:1	**1022**	4590	16.2	1020	**8680**	**22.3**	**1026**	**9070**	**22.8**	**1026**
99th Ave	999.0	0.00213	20	2:1	1016	4580	9.5	1009	9030	13.1	1012	9540	13.5	1013
107th Ave	990.5	0.00139	20	2:1	1010	4590	10.5	1001	9360	14.8	1005	9920	15.2	1006
115th Ave	981.5	0.00071	60	2:1	996	4610	8.0	990	9530	12.0	994	10120	12.4	994

Notes:

(1) The channel configurations correspond to a location immediately downstream of the indicated road crossing.

(2) The channel configuration data is based on topographic mapping (1"=200' , 2' contour interval) flown in March 1994.

(3) The estimated flow depths are based on uniform flow computations; however, backwater flow conditions are anticipated during major flood events.

(4) The results of this analysis indicate that overtopping of the channel banks may occur at this location.

4.1.3 Ponding Areas Along Grand Avenue And The Canals

A total of 24 ponding areas were assessed with the HEC-1 model in this study. These ponding areas are shown in Exhibits 5, 6 and A through S (Appendix L). The computed peak discharges exiting from the ponding areas and the corresponding high water elevations are summarized in Table 4-2.

The ponding area delineations shown in Exhibits A through S are based strictly on the results of the HEC-1 model. These ponding areas correspond to depressed areas that lack a positive drainage outfall, as identified with the detailed topographic mapping(1"=200', 2' contour interval). There are also areas along the canals and Grand Avenue that are subject to inundation by relatively slow moving stormwater flows. Within the study area, the Flood Insurance Rate Maps (or FIRMS) illustrate flood hazard areas that correspond to a combination of ponding areas and areas subject to relatively slow moving stormwater flows. However, detailed evaluation of the flow depths in the areas subject to innundation by stormwater flows is beyond the scope of the HEC-1 modeling in this study.

4.2 Comparison To Previous Studies

The results of this study have been compared to three drainage previous studies. The study areas for these studies are encompassed by the Maryvale ADMS study area. These three studies are:

- The "Off-site Drainage Concept Study for the Outer Loop Highway" prepared by WLB(1987) for the Arizona Department of Transportation.
- The "Northern/Orangewood Storm Drain Project, Concept Routing Study" prepared by Wood, Patel & Associates(1996) for the District.
- The "Maryvale Area Flooding Mitigation Project, Phase I Pre-Design Report" prepared by CVL(1996).for the District.

4.2.1 Off-site Drainage Concept Study - Outer Loop Highway

The off-site hydrology report for the Outer Loop Highway between Buckeye Road and Northern Avenue was prepared by the WLB Group(1987). The WLB Group was subcontracted by DeLeuw, Cather and Company, the Outer Loop Management Consultant for the Arizona Department of Transportation. The scope of the study was to evaluate the off-site flows that will impact the Outer Loop and to investigate alternative conceptual drainage designs. The WLB Group used HEC-1 to model the 78.3 square mile study area. The hydrologic analysis was based on the SCS Dimensionless Unit Hydrograph and the 24 hour "Hypothetical Storm Distribution" (WLB,1987).

The Outer Loop Highway study has been selected for comparison purposes, since the study area is very similar in extent to the study area for Maryvale ADMS. Computed peak discharges from the Outer Loop Highway study are compared with the results of this study in Table 4-3. As indicated in Table 4-3, the 100-year, 24-hour peak discharges computed as part of the Maryvale ADMS are primarily less than those documented in the Outer Loop

Table 4-2: Summary of Computed Water Surface Elevations

Ponding Area Exhibit	Ponding Area Name	Computed Peak Discharge from the Ponding Area (cfs): Q100-Yr 6-Hr (cfs)	Computed Peak Discharge from the Ponding Area (cfs): Q100-Yr 24-Hr (cfs)	Computed Water Surface Elevation: Q100-Yr 6-Hr Event	Computed Water Surface Elevation: Q100-Yr 24-Hr Event
A	RES8N	401	488	1053.1	1053.1
A	RES8S	1042	1152	1055.5	1055.5
B	PA55	121	124	1102.4	1102.8
B	PA59	615	626	1101.1	1101.1
B	PA63	965	1042	1098.2	1098.3
C	PA43	1032	982	1108.4	1108.3
C	PA47	373	368	1104.9	1104.9
C	PA51	947	910	1105.7	1105.7
D	RES14E	83	86	1087.6	1087.6
E	RES36	2409	2305	1138.2	1138.2
F	RES33	0	0	1123.0	1123.0
G	RES6	2141	1681	1146.0	1145.8
H	RES26T	3104	2893	1113.2	1113.2
I	RES29S	565	620	1021.7	1021.7
J	RES29W	627	674	1018.4	1018.4
K	RES10	994	1070	1068.7	1068.8
L	RES32E	188	197	1021.1	1021.1
M	RES22	473	379	1136.2	1136.2
N	RES9	897	970	1058.5	1058.6
O	RES26S	1100	989	1138.6	1138.6
P	RES8	458	369	1150.6	1150.5
Q	RES26E	3033	2547	1099.4	1099.3
R	RES24N	382	364	1097.5	1097.5
S	RES26N	398	145	1141.8	1141.7

Table 4-3: Comparison To Outer Loop Highway Study

Summary: This table indicates that the 100-year, 24 hour discharges computed as part of the Maryvale ADMS are primarily less than those documented in the Outer Loop Highway study by WLB(1987).

	Outer Loop Highway Study (WLB, 1987)		Maryvale ADMS		Comparison of Study Results
Location	Q100-Yr 24-Hr (cfs)	HEC-1 Node ID	Q100-Yr 24-Hr (cfs)	HEC-1 Node ID	Percent Difference (2)
35th Ave & I-10 Freeway Channel	2730	CP1	2370	DI10B	-15%
43rd Ave & I-10 Freeway Channel	5230	CP2	3620	DI10C	-44%
51st Ave & I-10 Freeway Channel	7320	CP3	5030	DI10D	-46%
59th Ave & I-10 Freeway Channel	9380	CP4	5520	DI10E	-70%
67th Ave & I-10 Freeway Channel	10110	CP5	6390	DI10F	-58%
75th Ave & I-10 Freeway Channel	10530	CP6	7410	DI10G	-42%
83rd Ave & I-10 Freeway Channel	10790	CP7	7920	DI10H	-36%
91st Ave & I-10 Freeway Channel	10880	CP8	9070	DI10I	-20%
Northern Avenue & Grand Avenue (1)	2620	CP10	1020	C2631S	-157%
Olive Avenue & Grand Avenue(1)	1260	CP11	2360	C622EA	47%
Glendale & 99th Avenues	3360	CP 13	1210	CGA99	-178%
Bethany Home Road & 99th Avenue	6790	CP14	910	DBH99	-646%
Camelback Road & 99th Avenue	790	CP15	530	CCB99	-49%
Indian School Road & 99th Avenue	2400	CP16	830	CIS99	-189%
Thomas Road & 99th Avenue	3690	CP17	580	CTR99	-536%

NOTES:

(1) The precise location of the concentration point in the WLB study is not known; hence, this comparison is approximate.

(2) The percent differences were computed as follows:

Percent Dif. = [(Q Maryvale ADMS) - (Q Outer Loop Hwy)]/(Q Maryvale ADMS)

Highway study. In comparison, the Outer Loop Highway study is a "big picture" type of hydrologic study and does not include detailed analysis of the ponding areas along the canal and Grand Avenue. It would be expected that an analysis that does not evaluate ponding storage along the canals and Grand Avenue would have higher computed peak discharges. Hence, the results of the comparison are consistent with the level of detail associated with each of the two studies.

4.2.2 Northern/Orangewood Storm Drain Project

The Northern/Orangewood Storm Drain Project was prepared by Wood, Patel & Associates(1996). This study evaluated various storm drain alternatives along Butler Drive, Northern Avenue, Orangewood Avenue, and Glendale Avenue. The storm drain systems proposed in the study have been included in the HEC-1 model for the Maryvale ADMS.

In Table 4-4, the results from hydrologic analysis for the Northern /Orangewood Storm Drain Project are compared with computed discharges for the Maryvale ADMS study. As indicated in Table 4-4, the results of these two studies are very similar. The levels of detail and the basic hydrologic parameters for these two studies were very similar. Hence, the results of the comparison are consistent with the level of detail associated with each of the two studies; even though, the overall HEC-1 modeling approaches used in the two studies are significantly different.

4.2.3 Maryvale Area Flooding Mitigation Project

The Maryvale Area Flooding Mitigation Project was prepared by Coe & Van Loo Consultants (CVL, 1996). This study evaluated flooding mitigation alternatives for the area along the north side of the Grand Canal, between approximately 35th Avenue and 67th Avenue. The flood control facilities proposed in the study have not been included in the "existing conditions" HEC-1 model for the Maryvale ADMS.

In Table 4-5, the results from hydrologic analysis for the Maryvale Area Flooding Mitigation Project are compared with computed discharges for the Maryvale ADMS study. As indicated in Table 4-5, the results of these two studies are significantly different. The differences are primarily due to the different approaches used to estimate the capacity for flow along the north side of the Grand Canal. In the Maryvale ADMS study, a HEC-2 model was used to evaluate the capacity for flow along the north side of the Grand Canal This analysis appears in a separate document entitled "Maryvale ADMS: Preliminary Grand Canal Floodplain Analysis." The ponding area analyses in Appendix E, subsection "Maryvale Mitigation Area" incorporate the results of this HEC-2 model.

Table 4-4: Comparison To Northern/Orangewood Storm Drain Project

Summary: This table indicates that the 10-year, 6 hour discharges computed as part of the Maryvale ADMS in this study are very similar to those documented in the Northern/Orangewood Storm Drain Project (WPA, 1996).

	Northern/Orangewood Storm Drain Project(WPA, 1996)		Maryvale ADMS		Comparison of Study Results
Location	Q10-Yr 6-Hr (cfs)	HEC-1 Node ID	Q10-Yr 6-Hr (cfs)	HEC-1 Node ID	Percent Difference [1]
Olive Avenue @ Grand Avenue	900	116C	890	C622EA	-1%
75th Avenue @ Glendale Avenue	170	270C	140	CGA75	-21%
83rd Avenue @ Glendale Avenue	390	290C	330	CGA83	-18%
91st Avenue @ Glendale Avenue	455	310C	430	CGA91	-6%
99th Avenue @ Glendale Avenue	490	330C	440	CGA99	-11%

NOTES:

(1) The percent differences were computed as follows:

Percent Dif. = [(Q Maryvale ADMS) - (Q Northern/Orangewood SD Project)]/(Q Maryvale ADMS)

Table 4-5: Comparison To Maryvale Area Flooding Mitigation Project

Summary: This table indicates that the 100-year, 6 hour discharges computed as part of the Maryvale ADMS are significantly different than those documented in the Maryvale Area Flooding Mitigation Project (CVL, 1996). The differences are primarily due to the different approaches used to estimate the capacity for flow along the North side of the Grand Canal. In this study, a HEC-2 model was used to evaluate the capacity for flow along the North side of the Grand Canal.

	Maryvale Area Flooding Mitigation Project (CVL,1996)		Maryvale ADMS		Comparison of Study Results
Location	Q100-Yr 6-Hr (cfs)	HEC-1 Node ID	Q100-Yr 6-Hr (cfs)	HEC-1 Node ID	Percent Difference (1)
Flow Along N. Side of Grand Canal in Vicinity of 39th Ave	440	D39	600	D39	27%
Total Flow to South of Grand Canal in Vicinity of 39th Ave	80	D39S	1080	D39s	93%
Flow Along N. Side of Grand Canal in Vicinity of 43rd Ave	960	D43	200	D43	-380%
Total Flow to South of Grand Canal in Vicinity of 43rd Ave	30	D43S	830	D43S	96%
Flow Along N. Side of Grand Canal in Vicinity of 47th Ave	770	D47	370	D47	-108%
Total Flow to South of Grand Canal in Vicinity of 47th Ave	550	D47S	0	D47S	-
Flow Along N. Side of Grand Canal in Vicinity of 51st Ave	None	-	20	D51	-
Total Flow to South of Grand Canal in Vicinity of 51st Ave	1080	D51S	930	D51S	-16%
Flow Along N. Side of Grand Canal in Vicinity of 59th Ave	500	D59	270	D59	-85%
Total Flow to South of Grand Canal in Vicinity of 59th Ave	460	D59S	350	D59S	-31%
Total Flow to South of Grand Canal in Vicinity of 63rd Ave	2020	S63	970	PA63	-108%

NOTES:

(1) The percent differences were computed as follows:

Percent Dif. = [(Q Maryvale ADMS) - (Q Maryvale Mitigation Project)]/(Q Maryvale ADMS)

5. Conclusions

The hydrologic modeling component of the Maryvale ADMS project has five primary objectives. As described below, it was necessary to develop a very flexible modeling approach and an efficient data management approach, to meet the project's objectives.

1) **Objective:** *Develop HEC-1 models that provide runoff computations for a primarily urban drainage area, where sub-basin drainage boundaries are not well defined and flow paths are very complex. Thereby allowing identification and quantification of flood hazards within the study area for existing conditions.*

   - The ability to model flow diversions flexibly was the key for developing a modeling approach that meets this project's objective. The proposed HEC-1 modeling approach uses flow diversions (i.e. DT/DI/DQ Records) in five distinctly different ways. Using physical parameters obtained from the detailed topographic mapping, flow diversions are used to simulate retention basins, surges basins associated with storm drain systems, and complicated street flow conditions.

2) **Objective:** *Develop HEC-1 models that evaluate the 10-year, 6-hour; 100-year, 6-hour; and 100-year, 24-hour rainfall events.*

   - Since the flow diversions and other aspects of the HEC-1 models are based strictly on physical parameters measured from the detailed topographic mapping, the model can be used to evaluate a wide range of storm events. In other words, the model structure **is not** based on assumptions which may only be valid for certain magnitudes of storm events.

3) **Objective:** *Develop HEC-1 models that allows efficient evaluation of both existing and future storm drain systems.*

   - The HEC-1 models prepared for this study take into account all of the major storm drain systems documented by previous studies and the City of Phoenix quarter section mapping. The HEC-1 models have been specifically setup to allow the incorporation of various types of flood control alternatives.

4) **Objective:** *Develop HEC-1 models that provide a cost-effective planning and analysis tool.*

   - Due to flexibility of the modeling approach, flood control alternatives involving various combinations of storm drain systems and/or retention basins can be easily incorporated into the HEC-1 models. DDMS can be used to update sub-basin parameters, that can be re-incorporated into the models without changing the computational structure. In addition, typical improvements associated with subdivisions can also be simulated without requiring significant modification of the computational structure of the HEC-1 models, where the computational structure of the models are illustrated in Exhibit 1.

5) **Objective:** *Develop HEC-1 models that allow for efficient updating.*

- Updating the HEC-1 models to reflect future street improvements and/or sub-divisions should only involve adjusting the elements of the model directly impacted by the construction activities. The overall structure of the model should not require adjustment.

The results of this study identify the locations and magnitudes of flood hazards, within the study area. In conclusion, the modeling approach and the HEC-1 models developed for this study meet the aforementioned objectives and the criteria specified in the project's scope of work.

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ATTACHMENT 7

CUDIA CITY WASH TO 10TH STREET WASH WATERSHED

Volume 1.8

Arizona Canal Diversion Channel
Area Drainage Master Study
ACDC/ADMS Phase I

HYDROLOGY REPORT

November, 1994

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CUDIA CITY WASH TO
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HYDROLOGY REPORT

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10TH STREET WASH WATERSHED
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1.0 SYNOPSIS

Kaminski-Hubbard Engineering, Inc. (KHE) was retained by the Flood Control District of Maricopa County (FCDMC) to prepare a comprehensive hydrologic analysis of the watershed contributing to the Arizona Canal Diversion Channel (ACDC) from Cudia City Wash to 10th Street Wash for existing and future conditions. This study area, as indicated in Figure 1, is one of several subwatersheds analyzed as a part of the ACDC Area Drainage Master Study (ADMS). This watershed drains the southern portion of the Phoenix Mountains from the 10th Street Wash boundary to the Indian Bend Wash boundary.

Within the watershed, twelve sub-basin area groupings were defined to address the precipitation depth/areal reduction issue for side inflow to the ACDC. The size of area groupings ranged between 0.25 square miles to 4.82 square miles.

There are seven existing detention basins within the watershed that collect runoff from the Phoenix Mountains for flood control purposes. The largest of these detention basins is impounded behind the Dreamy Draw Dam. These detention basins greatly reduce the amount of runoff reaching the ACDC from the Phoenix Mountains. The only significant future condition improvement would be the extension of the Squaw Peak Highway from Northern Avenue to Shea Boulevard.

This report presents the hydrologic analysis of the Cudia City Wash to 10th Street Wash watershed for both existing and future conditions upstream of the ACDC. Table 1 summarizes the controlling peak discharges for existing conditions at specific locations along the ACDC. Table 2 presents the controlling peak discharges for future conditions.

TABLE 1

Controlling Peak Discharge (Existing Conditions)

Location	Drainage Area (Sq. Mi.)	2-Yr. (cfs)	10-Yr. (cfs)	100-Yr. (cfs)
Cudia City Wash	4.82	589	2,512	5,411
Stanford Drive Wash	1.17	131	551	1,329
Flynn Lane Wash	1.04	121	577	1,114
Myrtle Avenue Wash	0.80	137	535	1,115
Dreamy Draw East	0.68	154	664	1,230
Dreamy Draw	1.97	141	416	852
Northern Avenue	0.99	95	459	900

TABLE 2

Controlling Peak Discharge (Future Conditions)

Location	Drainage Area (Sq. Mi.)	2-Yr. (cfs)	10-Yr. (cfs)	100-Yr. (cfs)
Cudia City Wash	4.82	726	2,899	5,750
Stanford Drive Wash	1.17	170	644	1,479
Flynn Lane Wash	1.04	148	608	1,152
Myrtle Avenue Wash	0.80	141	532	1,146
Dreamy Draw East	0.68	183	730	1,358
Dreamy Draw	2.07	140	422	897
Northern Avenue	0.98	123	504	966

2.0 INTRODUCTION

A hydrologic analysis of the Cudia City Wash to 10th Street Wash watershed for both existing and future conditions was developed by Kaminski-Hubbard Engineering, Inc. (KHE) for the Flood Control District of Maricopa County (FCDMC) as part of the Arizona Canal Diversion Channel (ACDC) Area Drainage Master Study (ADMS), Phase I. The majority of flows contributing to the ACDC originate from the Phoenix Mountains. The watershed is bounded by the Indian Bend Wash boundary to the east and the 10th Street Wash boundary to the west.

The watershed contains seven existing detention basins that significantly affect the amount of runoff reaching the ACDC. The largest detention basin is impounded by the Dreamy Draw Dam and was modelled in the U.S. Army Corps of Engineers (COE) 1982 hydrology study (Ref. 16). However, the other detention basins were not included in the COE model which have necessitated a revision to this hydrologic analysis.

Currently, the Squaw Peak Highway from Glendale Avenue to Northern Avenue has an impact on the original flow patterns in the Dreamy Draw area. Associated with the highway improvements was the construction of the Myrtle Wash detention basin. For future considerations, the Squaw Peak Highway will be extended through the Dreamy Draw area from Northern Avenue to Shea Boulevard. These improvements will not greatly affect the flow patterns in the area.

This report presents the existing and future hydrologic analysis for the watershed contributing to the ACDC from Cudia City Wash to 10th Street Wash. The hydrology was developed using the FCDMC's new design criteria and included detention basin modelling excluded from the previous COE report (Ref. 16).

3.0 STUDY PARAMETERS

3.1 Study Area

The watershed contributing storm runoff to the ACDC from Cudia City Wash to 10th Street Wash contains approximately 13.1 square miles. The watershed is characterized by moderate to steep mountains having moderate vegetation. The watershed is bounded by the Phoenix Mountains to the north, the Indian Bend Wash boundary to the east, the ACDC and Camelback Mountain to the south, and the 10th Street Wash boundary to the west.

The watershed was divided into twelve sub-basin area groupings having concentration points at the ACDC. The contributing areas at each concentration point range between 0.25 square miles to 4.82 square miles. The largest sub-basin area grouping is the subwatershed contributing to the Cudia City Wash. The Cudia City Wash outlet is the beginning of the ACDC and drains approximately 4.8 square miles.

The watershed contains seven detention basins which are included in the computer model. The largest detention basin is Dreamy Draw Dam, which has a contributing area of 1.30 square miles. The Myrtle Wash detention basin is located upstream of the Squaw Peak Parkway and provides detention storage for parkway drainage. Two detention basins are located within the Squaw Peak Park boundaries. The remaining three detention basins are situated to collect runoff from the Phoenix Mountains to lessen its effect on the downstream residential areas.

The extension of the Outer Loop Highway (OLH) from Northern Avenue to Shea Boulevard will slightly modify drainage patterns near the Dreamy Draw area for future conditions.

3.2 Mapping

The available mapping utilized in this study are as follows:

1. **FCDMC Mapping:** The watershed was flown as a part of this study for the purpose of obtaining 1 inch = 400 foot contour and aerial mapping. The contour interval is 2 feet. These maps were flown between November 1990 and August 1991. These maps were used to establish the sub-basin drainage delineation, flow patterns, and storage volume calculations for detention facilities. The aerial maps were also utilized to provide land use information for existing conditions.

2. **USGS Quadrangle Maps:** Paradise Valley and Sunnyslope, Arizona, 7.5 minute series. The horizontal scale is 1 inch = 2000 feet. The contour interval is 20 feet. These maps were photo revised in 1982.

3. **City of Phoenix Storm Drain Maps:** These maps are at a scale of 1 inch = 400 feet and provide a schematic location of storm drains and culverts in the area.

4. **City of Phoenix Zoning Maps:** These maps are at a scale of 1 inch = 400 feet and provide zoning designations and boundaries in the area.

5. **Construction Plans:** Construction plans for drainage structures associated with Lincoln Drive, McDonald Drive, Tatum Boulevard, Glendale Avenue, Northern Avenue, 16th Street, 32nd Street, 40th Street, and 44th Street were used for routing and sub-basin delineation purposes. Construction plans for the following detention basins were utilized for reservoir routing purposes: Dreamy Draw Dam; Myrtle Wash detention basin; Squaw Peak Park Detention Basin Nos. 1 and 2; Detention Basin Nos. 4 and 6; and the Biltmore Mountain Estates detention basin. Construction plans for the ACDC were used to determine sub-area grouping concentration points. Construction plans for the Squaw Peak Highway from Glendale Avenue to Shea Boulevard were used for drainage delineation purposes.

6. **General Plan for Phoenix:** This general plan was used to determine the extent of future development. Areas of future parks, open-spaces, and traffic corridors were considered during the future hydrologic analysis.

7. **Field Reconnaissance:** Field investigations were undertaken to verify hydrologic information obtained from aerial and topographic mapping. Areas of new development or developments under construction and existing on-site retention areas were identified. All major drainage structures within the watershed were identified. The flow paths of all major mile and half-mile streets were identified. Some drainage patterns were documented for local streets.

3.3 Study Criteria

The following criteria and guidelines were set forth by the FCDMC prior to and during the drainage study:

1. Hydrology calculations will be completed for the 2-, 10-, and 100-year storms;

2. Storm durations of 6- and 24-hour will be evaluated for all three storms;

3. The U.S. Army Corps of Engineers (COE) HEC-1 computer program will be used for hydrograph computations;

4. Sub-basins will be limited to a maximum of five square miles in area;

5. The Clark Unit Hydrograph method will be utilized;

6. The Green-Ampt Loss Method will be utilized for estimation of precipitation losses;

7. The Maricopa County Unit Hydrograph Procedure 1 (MCUHP1) computer program, as provided by the FCDMC, will be used to compute times of concentration and storage coefficients for the Clark Unit Hydrograph Method.

8. Rainfall distributions and depth-area relations for the 6-hour storm duration will be based on NOAA HYDRO-40 (Ref. 21) and COE (Ref. 15) data, as presented in the FCDMC's <u>Drainage Design Manual</u> (Ref. 6). This data is included in the MCUHP1 program to develop areal reduction for the watershed.

9. The SCS Type II rainfall distribution will be used for the 24-hour storm, with corresponding depth-area ratios based on NOAA HYDRO-40 (Ref. 21). This data is included in the MCUHP1 program.

10. Existing and future flow rates are to be determined.

11. Transmission losses will be estimated based on existing field data or literature. Existing field data or literature was not available to estimate infiltration losses. Due to this study's detailed determination for the watershed roughness coefficient (Kb), the exclusion of transmission losses has little impact on the flow peaks and volumes.

4.0 HYDROLOGY

4.1 General

The existing and future hydrology for the Cudia City Wash to 10th Street Wash watershed was analyzed for the 2-, 10-, and 100-year storms. The 6- and 24-hour storm durations were evaluated for all three storms. The Cudia City Wash to 10th Street Wash watershed was modeled using the COE HEC-1 computer program. The May, 1991, version of HEC-1 was used for this study. The Clark Unit Graph, the Green-Ampt Loss Rate, and the Muskingum-Cunge Routing options were used in the HEC-1 computer model. The HEC-1 modeling also included allowances for routing hydrographs through detention basins using the Modified Puls Method. This section describes the assumptions and methodologies used to develop the HEC-1 computer model for existing and future conditions within the Cudia City Wash to 10th Street Wash watershed.

4.2 Previous Hydrologic Investigations

Previous hydrologic investigations of the watershed were reviewed for historical, as well as, hydrologic information that could be used as part of our analysis for both existing and future conditions. Particular attention was given to hydrologic modeling techniques, sub-basin delineation, storm frequency and duration, reach routing methods, location of concentration points, treatment of detention basin areas, and location of future drainage structures. A brief summary of previous investigations performed for the Cudia City Wash to 10th Street Wash watershed are presented below.

Gila River Basin, New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part 2 (Ref. 16)

In 1982, a hydrologic investigation was performed by the COE for flood control projects in the Phoenix area. The COE procedure of watershed modelling is to determine the Standard Project Flood (SPF) that would result from the most severe combination of meteorologic and hydrologic conditions that are considered reasonable for the area. The lesser storm frequency events are calculated as a percentage of the SPF. As an example, the 100-year peak discharge is 45 percent of the SPF.

The Cudia City Wash to 10th Street Wash watershed was divided into seven sub-basins and evaluated for future fully developed conditions. The COE uses local dimensionless S-graphs to produce hydrographs from the rainfall excess.

Cudia City Wash Runoff Analysis (Ref. 23)

In 1986, a hydrologic investigation was performed by W.S. Gookin and Associates (WSG) for the Cudia City Wash watershed. Cudia City Wash drains a 5.12 square mile watershed bounded by the eastern slopes of the Phoenix Mountains, the southern slope of Mummy Mountain, and the west half of the northern slope of Camelback Mountain.

The watershed was divided into fourteen sub-basins. The SCS Method was used to compute a hydrograph of each sub-basin for the 100-year, 24-hour duration storm. A 100-year, 24-hour storm precipitation depth of 3.8 inches was used in conjunction with a Type IIA rainfall distribution. A Lotus spreadsheet was used to combine and route the sub-basin hydrographs.

Review of W.S. Gookin and Associates Analyses of Cudia City Wash Hydrology (Ref. 7)

The FCDMC perform a review of the WSG report for the Cudia City Wash watershed. The following concerns with the hydrologic model were discovered for the study area: inappropriate design rainfall depth and distribution, too long a computation interval, in appropriate combining and routing of hydrographs from various sub-basins, and suspect times of concentration.

The FCDMC created a HEC-1 computer model for the watershed using the WSG modelling parameters. The kinematic wave method was used for hydrograph routing. The HEC-1 model peak discharge results from each sub-basin ranged between 19% - 51% lower than the WSG estimates. However, the final routed and combined HEC-1 model result at the Arizona Canal was 13% greater than the WSG result.

The FCDMC developed their own independent HEC-1 model for the Cudia City Wash watershed. A combination of the kinematic wave and SCS methods were used to generate hydrographs from the study area. The kinematic wave option was used on urbanized basins having moderately steep slopes. The SCS method was used on two sub-basins having steep slopes. A 100-year, 24-hour storm depth of 4.04 inches and the City of Phoenix storm distribution was used.

Final Drainage Report, SR-51 Squaw Peak Hwy., Glendale Avenue to Northern Avenue (Ref. 8)

The drainage design concept for the Squaw Peak Highway from Glendale Avenue to Northern Avenue was prepared by Howard Needles Tammen and Bergendoff (HNTB) for the Arizona Department of Transportation (ADOT). The project area is located near the western boundary of the Phoenix Mountain Preserve and is characterized by numerous steep washes flowing westerly to the Squaw Peak Highway.

The study area was divided into ten sub-basins, of which, the area contributing to Myrtle Wash is the largest. The two northernmost and southernmost sub-basins contribute directly to the Dreamy Draw East Wash. The middle five areas flow into a detention basin and storm drain system located at the Pointe development. The Dreamy Draw East Wash eventually discharges into Myrtle Wash, which outfalls into the ACDC.

The Myrtle Wash detention basin was designed to provide 100-year detention storage for flows generated from the Squaw Peak Highway and a residential subdivision area east of the highway. This basin is necessary to maintain the peak flow in Myrtle Wash downstream of the highway to pre-existing conditions. The pavement drainage structures and storm drain systems were designed for the 10-year return period. The cross drainage culverts at Myrtle Wash, Pleasant Drive, and Dreamy Draw Wash were design for the 100-year return period. All other cross drainage pipe culverts were designed for the 50-year return period.

<u>Final Drainage Report for Squaw Peak Hwy., Section 2, Northern Avenue to 29th Street (Ref. 11)</u>
The drainage design concept report for the Squaw Peak Highway from Northern Avenue to 29th Street was prepared by Urban Engineering for the ADOT. The project area is located in the Dreamy Draw watershed and is characterized by steep washes flowing southwesterly to the Squaw Peak Highway. Dreamy Draw Dam is a major flood control structure constructed by the COE within the watershed.

The watershed is divided into two components, onsite and offsite areas. The onsite areas consist of pavement and median areas associated with the Squaw Peak Highway. The rational method was used to determine onsite runoff for a 10-year return period. The intensity factor was determined using a time of concentration of 10 minutes. The offsite hydrology impacting the highway was developed by Baker Engineers (Ref. 1). This report identified twenty-two sub-basins within the Dreamy Draw watershed that impact the project. Baker Engineers used the COE HEC-1 program to determine the peak discharges for 50- and 100-year frequency storms using the SCS Method. Based on an evaluation of the watershed by Urban Engineering, additional sub-basins were developed which directly impact the project.

<u>Final Drainage Report for Squaw Peak Hwy. - SR-51, Section 3, 29th Street to Shea Boulevard (Ref. 4)</u>
The drainage plan for the Squaw Peak Highway from 29th Street to Shea Boulevard was prepared by Entranco Engineers for the ADOT. The project is located within the northern foothills of the Phoenix Mountains, which is northeast of the Dreamy Draw Dam. The overland flow in the area is typically to the north-northeast. However, an existing detention basin at 32nd Street and Mountain View Road is drained to the Dreamy Draw Dam.

A General Plan Drainage Report was prepared by Baker Engineers for the Squaw Peak Highway (Ref. 1). The study area was found to have two main drainage basins resulting from a crest in the Squaw Peak Highway at Station 173+30. West of the crest, drainage will flow to the southwest toward Dreamy Draw Dam. East of the crest, drainage will flow north to the Indian Bend Wash. The trunk line that drains the area west of the crest begins at the above mentioned detention basin and outfalls into the Dreamy Draw Dam. Along the trunk line offsite and onsite pavement drainage enters the system.

East of the crest, the basin is drained by two trunk lines. Onsite pavement drainage from the Squaw Peak Highway is collected an conveyed by one trunk line to a temporary detention basin south of Shea Boulevard. In the future, this line will extend northward and discharge into the Indian Bend Wash near Thunderbird Road. Offsite flows are collected and conveyed by the second trunk line to a detention basin south of Shea Boulevard. In the future, this trunk line will connect with the existing 78-inch storm drain in Shea Boulevard which outlets to the Indian Bend Wash.

4.3 Parameter Estimation

4.3.1 Drainage Area Boundaries

Existing Condition

The initial delineation of sub-basins for the Cudia City Wash to 10th Street Wash watershed was developed using information presented in previous drainage reports (Ref. 1, 8, 16 & 23). Next, this initial delineation was evaluated using the new 1-inch to 400 feet topographic and aerial maps flown as a part of this study. Particular attention was given to the areas contributing to detention basins and major roadway cross drainage structures. The initial delineation was also supplemented by construction drawings of major collector streets and the Squaw Peak Highway.

The initial delineation was then verified or revised based on field investigations. This field investigation included driving major mile and half-mile streets to distinguish flow pattern. These flow patterns were recorded and later referred to during time of concentration calculations for each sub-basin. The field investigations also included the determination of onsite retention locations and non-contributing areas within the watershed. The non-contributing areas were evaluated for each storm frequency. A parcel area labeled as non-contributing for a two-year storm may be contributing for a 10- and 100-year storm analysis.

The sub-basins were delineated so that concentration points were provided at major street intersections, impoundment areas and stream confluences. Concentration points were also located such that comparisons could be made with other hydrologic investigations. The major concentration points along the ACDC were chosen at major wash inlets to the channel based on sub-basin area groupings. The sub-basin delineations are presented in Plate 1 for existing conditions.

Future Condition

The drainage delineation for future conditions were predominantly the same as presented for the existing conditions. However, a slight modification was made within the Dreamy Draw sub-basin area grouping as a result of the Squaw Peak Highway extension from Northern Avenue to Shea Boulevard. The modified delineations were taken from drainage plans developed for ADOT by various consultants (Ref. 4, 11, 12, & 13). Based on this information and other discussions with the FCDMC, the sub-basin delineations are as shown in Plate 5.

4.3.2 Rainfall Parameters

Rainfall Distributions

The rainfall distribution used for the 6-hour storm duration are as documented in the FCDMC's Drainage Design Manual (Ref. 6) and contained in the MCUHP1 program. The SCS Type II distribution was used for the 24-hour storm. The rainfall distributions are presented in Tables 8 & 9 in Section I of the Appendix.

Precipitation Data

The point precipitation values were obtained using the NOAA Atlas isopluvial maps for Maricopa County, Arizona. The point precipitation values are presented in Table 6 in Section I of the Appendix.

Areal Reduction Factors

The point precipitation values used for the various sub-basin area groupings were adjusted to account for the reduction in precipitation depth over a spatial area. Reduction factors for the 6-hour duration storms were obtained from the FCDMC's Drainage Design Manual (Ref. 6). This information was also included in the FCDMC's MCUHP1 program. The 24-hour storm reduction factors were obtained from the NOAA Technical Memorandum NWS HYDRO-40 (Ref. 21). These factors are presented in Table 7 in Section I of the Appendix.

4.3.3 Physical Parameters

Loss Rate Estimation

The Green-Ampt loss rate method in HEC-1 was used to estimate rainfall losses for both existing and future conditions. This method involves a two phase process in simulating rainfall losses. The first phase involves no infiltration of rainfall until the accumulated rainfall equals the initial loss (IA). Recommended IA values are presented in Table 4.1 in the Drainage Design Manual (Ref. 6).

The second phase is the infiltration of rainfall into the soil immediately after IA is completely satisfied. The three Green-Ampt infiltration parameters as coded in HEC-1 are: hydraulic conductivity at natural saturation (XKSAT); wetting front capillary suction (PSIF); and volumetric soil moisture deficit at the start of rainfall (DTHETA).

The Green-Ampt parameters were determined using a spreadsheet provided by the FCDMC, Watershed Management Branch. The XKSAT values were determined by the FCDMC for all map units contained in the SCS Soil Survey (Ref. 19) using log averaging of major and minor soil XKSAT values. These map units along with their corresponding XKSAT and percent rock outcrop values are presented in lookup tables within the Green-Ampt Spreadsheet.

The area of each soil unit within each sub-basin was determined and used as input into the Green-Ampt Loss Parameter spreadsheet. The soil units within each sub-basin are shown on Plate 3 for existing conditions and Plate 7 for future conditions. These area calculations were determined using ARC INFO GIS. The spreadsheet subsequently computed average sub-basin XKSAT values using log averaging methods. Next, values for PSIF and each DTHETA condition (i.e. dry, normal, wet) were interpolated using the computed XKSAT. These tables were contained within the spreadsheet and were similar to Table 4.2 in Drainage Design Manual (Ref. 6).

The computed Green-Ampt parameters were based strictly on soil characteristics and adjustments were necessary to account for vegetative cover and land use. These guidelines are presented in the FCDMC's Drainage Design Manual (Ref. 6) and are incorporated in the Green-Ampt Loss Parameter Spreadsheet. The area of each land use within each sub-basin was also determined and used as input into the spreadsheet. The various land uses categories within each sub-basin are shown on Plate 2 for existing conditions and Plate 6 for future conditions. Again, these area calculations were performed using ARC INFO GIS.

The "percent impervious" for each sub-basin was computed as a function of both natural rock outcrop and land use. The percentage of impervious rock outcrop within each sub-basin was estimated from soil unit data provided in the SCS Soil Survey (Ref. 19). A factor of 0.6 was used to convert the "percentage of rock outcrops" to the "percent impervious" for each sub-basin.

Next, the impervious areas associated with various land use categories were determined for each sub-basin. The City of Phoenix zoning designations were classified into land use categories based on aerial mapping and are presented in Table 10 in Section I of the Appendix.

The total "percent impervious" value for each sub-basin was computed as a summation of the above two "percent impervious" values. This computation was also incorporated into the Green-Ampt Loss Parameter spreadsheet. The average Green-Ampt parameters for existing and future conditions are presented in Tables 11 and 12, respectively in Section II of the Appendix.

<u>Time Of Concentration</u>

The Clark Unit Hydrograph method requires the estimation of the time of concentration, T_c. The following empirical equation was used to compute the time of concentration as a function of watershed characteristics (Ref. 6):

$$T_c = 11.4L^{0.5}K_b^{0.52}S^{-0.31}i^{-0.38}$$

where:

T_c	=	time of concentration, in hours.
L	=	length of the flow path for T_c, in miles.
K_b	=	representative watershed resistance coefficient.
S	=	watercourse slope, in feet/mile.
i	=	the average rainfall excess intensity, during the time T_c, in inches/hour.

The length of the flow path for T_c and its corresponding slope within each sub-basin were determined using 1 inch to 400 feet topographic maps. Street flow patterns observed from the field investigations were also used to determine the flow path for T_c considerations. The MCUHP1 program, as provided by the FCDMC, was used to calculate the time of concentration, T_c, and storage coefficient, R, for each sub-basin.

The watershed resistance coefficient, K_b, necessary to determine T_c was estimated using the following equation (Ref. 6):

$$K_b = m \log A + b$$

where:

K_b = watershed resistance coefficient.

A = drainage area, in acres.

m&b = parameters dependent on land use and vegetation cover.

The watershed resistance coefficient, K_b, for each sub-basin was weighted to account for varying roughness conditions associated with mixed land use classifications. The land use classifications within each sub-basin were categorized into roughness types using the descriptions presented in Table 5.1 (Ref. 6). All vacant areas were placed under the category of moderately high roughness (Type C). Low and very low density residential areas were labelled as having moderately low roughness (Type B). Medium density and multi-family residential areas were placed under the category of minimal roughness (Type A).

The time of concentration flow paths for existing and future conditions are presented in Plate 4 and 8, respectively. The hydrologic sub-basin characteristics for existing conditions are presented in Tables 13, 14 and 15 in Section III of the Appendix. The characteristics for future conditions are presented in Tables 17, 18 and 19 in Section III of the Appendix.

4.3.4 Routing Parameters

Channel Routing

For this study, the Muskingum-Cunge method was used to route a hydrograph through a downstream sub-basin. Channel cross-section information, slopes, and Manning's roughness coefficients were estimated using topographic mapping and observations made during the field investigation. Channel routing flow paths for existing and future conditions are presented in Plates 4 and 8, respectively. Channel routing work sheets are presented in Section IV of the Appendix.

Existing field data or literature was not available to estimate infiltration losses. Based on the watershed topography and this study's detail for the watershed resistance coefficient, not including transmission losses has little impact on the flow peaks and volumes.

Reservoir Routing

The Modified Puls method was used for reservoir routing through a detention basin. A total of seven detention basins were located within the Cudia City Wash to 10th Street Wash watershed. The largest detention basin was impounded behind the Dreamy Draw Dam as a part of the COE flood control plan for Phoenix. The reservoir routing parameters were obtained from the COE report for Dreamy Draw Dam (Ref. 14), verified by KHE, and used in this study.

There are two detention basins, Nos. 1 and 2, located within the Squaw Peak Park boundaries that control the amount of runoff from the Phoenix Mountains. Both detention basins are drained by 24-inch diameter concrete pipes with parking lots functioning as overflow spillways. The storage volumes were determined using 1 inch to 400 feet topographic maps. The overflow spillway sections were surveyed for weir flow calculations.

The North Mountain Detention Basin No. 4 is located north of Northern Avenue between 18th Street and the Squaw Peak Highway. This basin limits the amount of runoff from the Phoenix Mountains that ultimately reach the ACDC south of Northern Avenue. This basin is drained by a 27-inch concrete pipe having a 16-inch by 12-inch orifice inlet. The overflow spillway width is approximately 80 feet. The storage volume was determined using 1 inch to 400 feet topographic maps.

Detention Basin No. 6 is located west of the Squaw Peak Highway and north of Orangewood Avenue within the Pointe development. This basin collects offsite runoff and conveys the low level flows under the Dreamy Draw Condominiums through a 12-inch corrugated metal pipe. The storage volumes were determined using 1 inch to 400 feet topographic maps.

The Myrtle Wash detention basin is located east of the Squaw Peak Highway and just north of Myrtle Wash. This basin provides detention storage for flows generated from the Squaw Peak Highway and a portion of the residential subdivision east of the basin. The reservoir routing parameters were obtained from the HNTB report for the Myrtle Wash detention basin (Ref. 8).

The Biltmore Mountain Estates detention basin is located north of Lincoln Drive and east of Arizona Biltmore Circle. Due to the relatively small capacity of the detention basin, the 100-year peak discharge is not significantly impacted. Therefore, the basin is modelled in HEC-1 as a diversion for the 100-year frequency storms. The diversion is such that the runoff volume, up to the storage capacity of the basin, is diverted out of the watershed. However, the reservoir routing parameters for the basin are used for the 2- and 10-year storm analysis.

The detention storage calculations for the above basins are presented in Section IV of the Appendix. Section IV also contains discharge calculations for both low flow and overflow spillways. In some cases, the pipe and weir flow parameters are also presented as input to the model.

4.4 Special Considerations

4.4.1 Storm Drain Pipes

There are very few storm drain pipes within this watershed. These storm drain systems do not significantly affect the drainage patterns within the watershed, i.e., flows diverted out of the watershed area or from one sub-basin to another. Therefore, all storm drain systems were ignored in the HEC-1 model.

4.4.2 Onsite Retention

The City of Phoenix requires that all new developments retain the 100-year 2-hour duration storm volume that falls onsite. Field investigations within the watershed found that a majority of lots had no onsite retention or minimal retention at best. A few commercial and industrial sites constructed in the last few years had complied with the onsite retention requirements. However, there was no detailed mapping available to accurately determine the retention volume for a given site, much less whether they were 10- or 100-year volumes. Therefore, the retention volume for the parcels in question were assumed to retain the 10-year 2-hour storm volume. The total estimated retention volume for each sub-basin was subtracted from the bottom of the hydrograph by diverting the estimated volume. These computations are presented in Section V of the Appendix.

Particular attention was placed on determining the non-contributing areas associated with a 2-year storm. Those areas that required onsite volume computations were automatically labeled as non-contributing. Next, impervious area associated with land use were assumed to contribute 100% of their areas. The remaining pervious areas were assumed to be non-contributing. These computations are presented in Section V of the Appendix.

5.0 RESULTS AND CONCLUSIONS

The HEC-1 computer model was used to compute the 2-, 10-, and 100-year peak discharges for existing and future conditions within the Cudia City Wash to 10th Street Wash watershed. The 6-hour and 24-hour events were evaluated using the Clark Unit Hydrograph method for each storm frequency. The hydrologic analysis for both existing and future conditions was developed through the consolidation of previous hydrologic investigations and verifying or updating that information with new topographic mapping and our own field investigations.

The existing peak discharge results for the Cudia City Wash to 10th Street Wash watershed are summarized in Table 3. The future peak discharge results of this study are presented in Table 4. Evaluation of the results indicate that larger peak discharges occur from a 6-hour duration storm for all three (3) recurrence intervals.

The total watershed area contributing to the ACDC from Cudia City Wash to 10th Street Wash is approximately 13.1 square miles. However, this study was interested only in side inflows to the ACDC and not the combining and routing of flows within the ACDC. Therefore, sub-basin area groupings were developed to determine the contributing areas at twelve inflow locations along the ACDC. These sub-basin area groupings were also used for precipitation depth/areal reduction purposes.

Six of the seven detention basins within the watershed were found to have sufficient capacity to detain the 100-year 24-hour duration storm runoff. These detention basins provide flood protection against storm runoff from the Phoenix Mountains. Low level outflows from these basins were routed downstream and did not significantly contribute to the downstream peak discharges.

The Biltmore Mountain Estates detention basin had insufficient storage to detain the 100-year 24-hour storm runoff. This detention basin was modelled as a 5.4 acre-feet of storage volume diversion from its corresponding sub-basin peak discharge. For the 2- and 10-year storm frequencies, the reservoir storage parameters are included in the HEC-1 model.

A comparison between this study's 100-year 6-hour future peak discharge results and the results of previous investigations are presented in Table 5. The KHE peak discharge results are considerably lower than the COE results at certain side inflow locations along the ACDC. The only detention basin modelled by the COE was the impoundment area behind the Dreamy Draw Dam. The difference in drainage areas was attributed to the COE sub-basin delineation on 7.5 minute quadrangle maps and this study's 1 inch to 400 feet topographic maps.

TABLE 3

Existing Peak Discharges
At ACDC (CFS)

Location	HEC-I.D.	2-Year		10-Year		100-Year		100-Year 24-Hour Time To Peak (Hrs.)
		6 Hr.	24 Hr.	6 Hr.	24 Hr.	6 Hr.	24 Hr.	
Cudia City Wash	107DC	589	571	2,512	2,336	5,085	5,411	12.3
Taward Wash	110DC	73	48	233	140	453	304	12.1
Stanford Drive Wash	112DC	131	72	551	380	1,329	1,083	12.1
Cunningham Wash	113DC	105	64	480	264	853	516	12.1
Biltmore	115DC	68	40	289	158	1,048	612	12.1
Treatment Plant	116DC	205	106	478	241	757	430	12.0
Maryland Avenue	117DC	40	14	324	144	622	337	12.1
Flynn Lane Wash	121DC	121	72	577	352	1,114	778	12.1
Myrtle Avenue Wash	124DC	137	71	535	337	1,115	811	12.3
Dreamy Draw East	126DC	154	93	664	407	1,230	822	12.1
Dreamy Draw	129DC	141	119	416	320	852	719	12.4
Northern Avenue	131DC	95	60	459	306	900	659	12.1

TABLE 4

Future Peak Discharges
At ACDC (CFS)

Location	HEC-I.D.	2-Year		10-Year		100-Year		100-Year 24-Hour Time To Peak (Hrs.)
		6 Hr.	24 Hr.	6 Hr.	24 Hr.	6 Hr.	24 Hr.	
Cudia City Wash	107DC	716	726	2,899	2,557	5,412	5,750	12.3
Taward Wash	110DC	82	53	249	149	483	320	12.1
Stanford Drive Wash	112DC	170	101	644	445	1,479	1,184	12.1
Cunningham Wash	113DC	137	82	517	279	914	551	12.1
Biltmore	115DC	116	69	519	203	1,274	738	12.1
Treatment Plant	116DC	214	110	493	250	781	440	12.0
Maryland Avenue	117DC	56	23	348	162	645	355	12.1
Flynn Lane Wash	121DC	148	88	608	372	1,152	811	12.1
Myrtle Avenue Wash	124DC	141	75	532	338	1,146	825	12.3
Dreamy Draw East	126DC	183	114	730	432	1,358	902	12.1
Dreamy Draw	129DC	140	129	422	330	897	759	12.3
Northern Avenue	131DC	123	79	504	331	966	699	12.1

TABLE 5

Comparison of 100-Year Peak Discharges with Previous Studies

Location	Kaminski-Hubbard (1994)		COE (Ref. 16) (1982)		FCDMC (Ref. 7) (1987)	
	Drainage Area (Sq. Mi.)	Q (CFS)	Drainage Area (Sq. Mi.)	Q (CFS)	Drainage Area (Sq. Mi.)	Q (CFS)
Cudia City Wash	4.82	5,750	4.91	6,800	4.91	6,540
Stanford Drive Wash	1.17	1,479	1.38	2,400	----	----
Flynn Lane Wash	1.04	1,152	1.10	1,900	----	----
Myrtle Avenue Wash	0.80	1,146	1.18	2,300	----	----
Dreamy Draw East	0.68	1,358	----	----	----	----
Dreamy Draw	2.07	897	3.08	1,300	----	----
Northern Avenue	0.98	966	----	----	----	----

6.0 REFERENCES

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APPENDIX

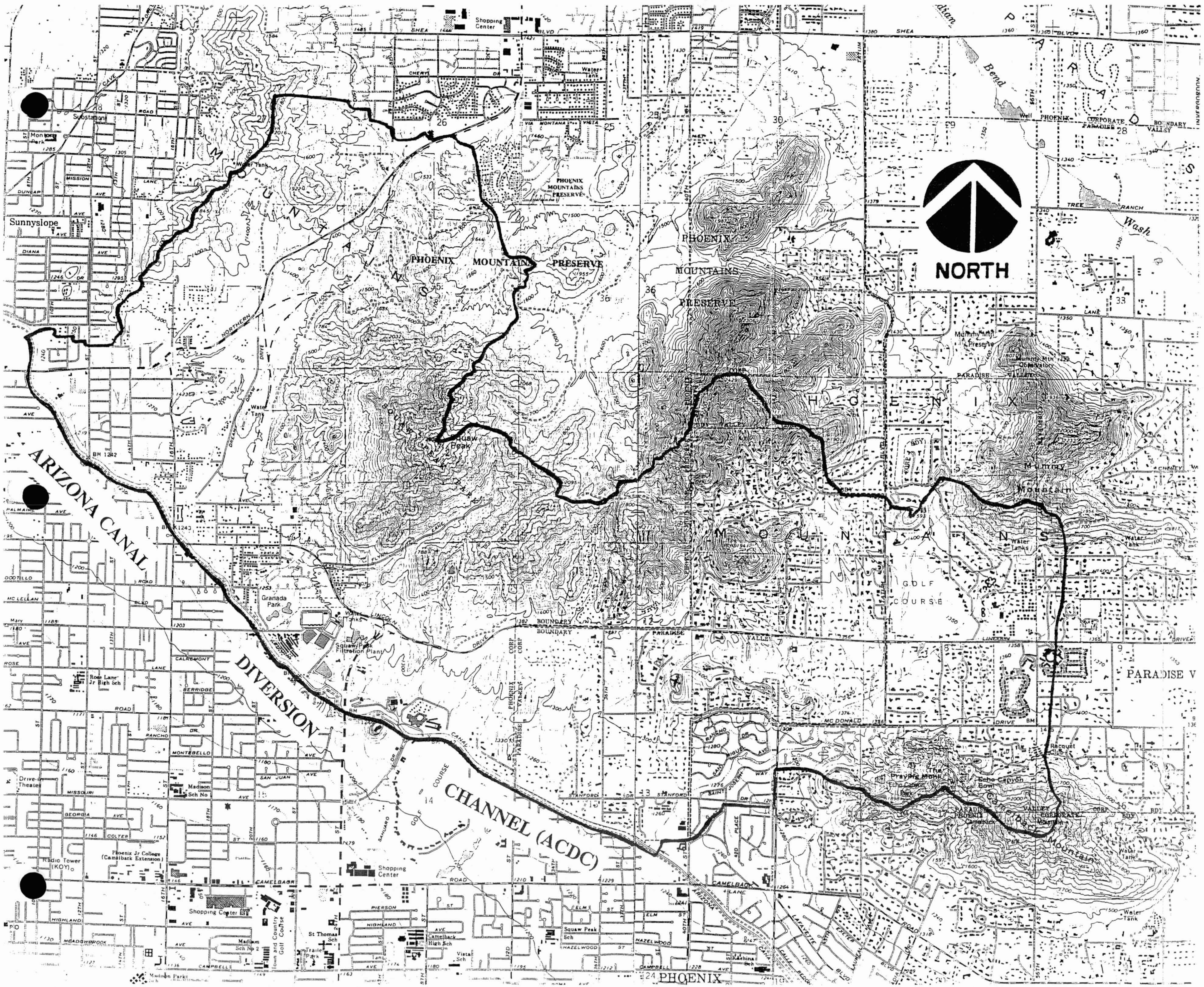
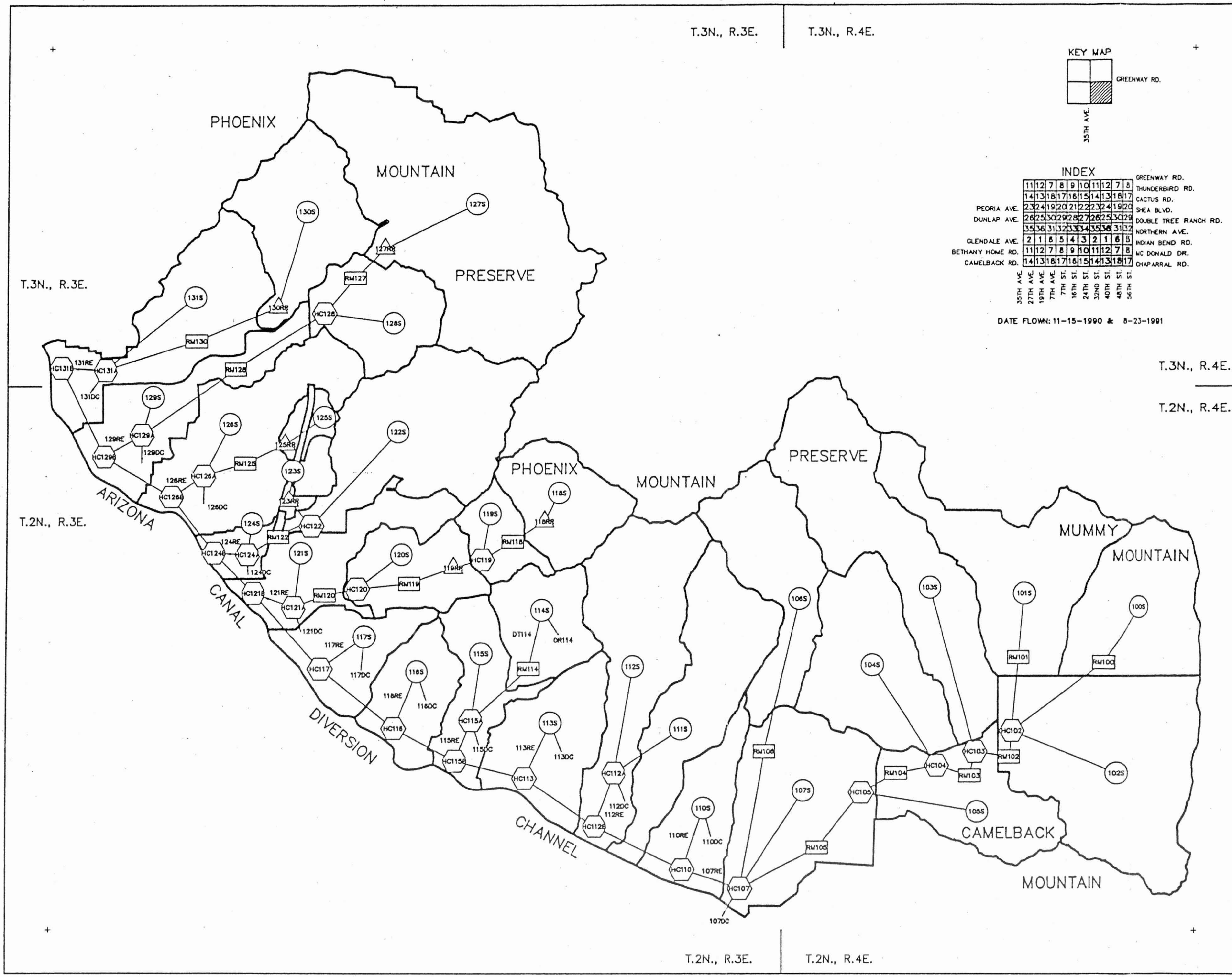


FIGURE 1 - VICINITY MAP



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

ACDC/ADMS PHASE 1 CUDIA CITY WASH TO 10TH STREET WASH HYDROLOGY STUDY

LEGEND

Drainage Basin Boundary

A Compute Runoff from Sub-Basin A

B Compute Runoff from Sub-Basin B

C Combine Hydrographs

D Route Hydrograph

E Route Hydrograph through Retention Basin E

G F Divide Hydrograph into F and G

DRAINAGE AREA MAP & HEC-1 SCHEMATIC

EXISTING CONDITIONS

PLATE 1

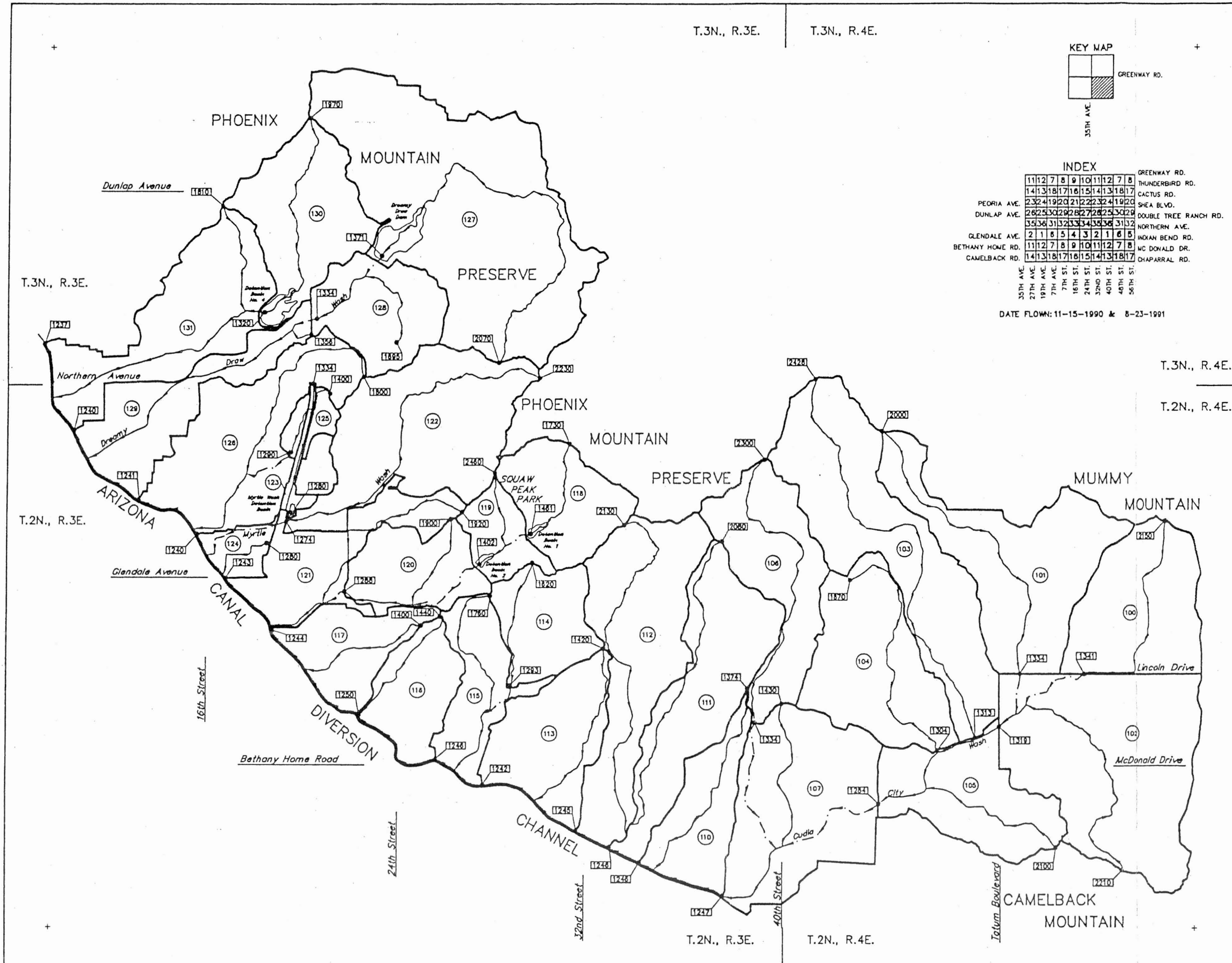
KAMINSKI HUBBARD engineering, inc.
SURVEYING • CIVIL • HYDROLOGY
4550 N. BLACK CANYON HWY., SUITE C
PHOENIX, ARIZONA 85017
(602) 242-5588

N

SCALE

1200 600 0 1200 2400

FEET



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

ACDC/ADMS PHASE 1 CUDIA CITY WASH TO 10TH STREET WASH HYDROLOGY STUDY

LEGEND

- Drainage Basin Boundary
- (140) Drainage Sub-Basin Number
- Major Drainage Basin Concentration Point
- Drainage Sub-Basin Concentration Point
- Flow Diversion Point
- Routing Flow Path
- Length of Longest Watercourse
- [1514] Elevation Along Flow Path

FLOW ROUTING MAP
EXISTING CONDITIONS

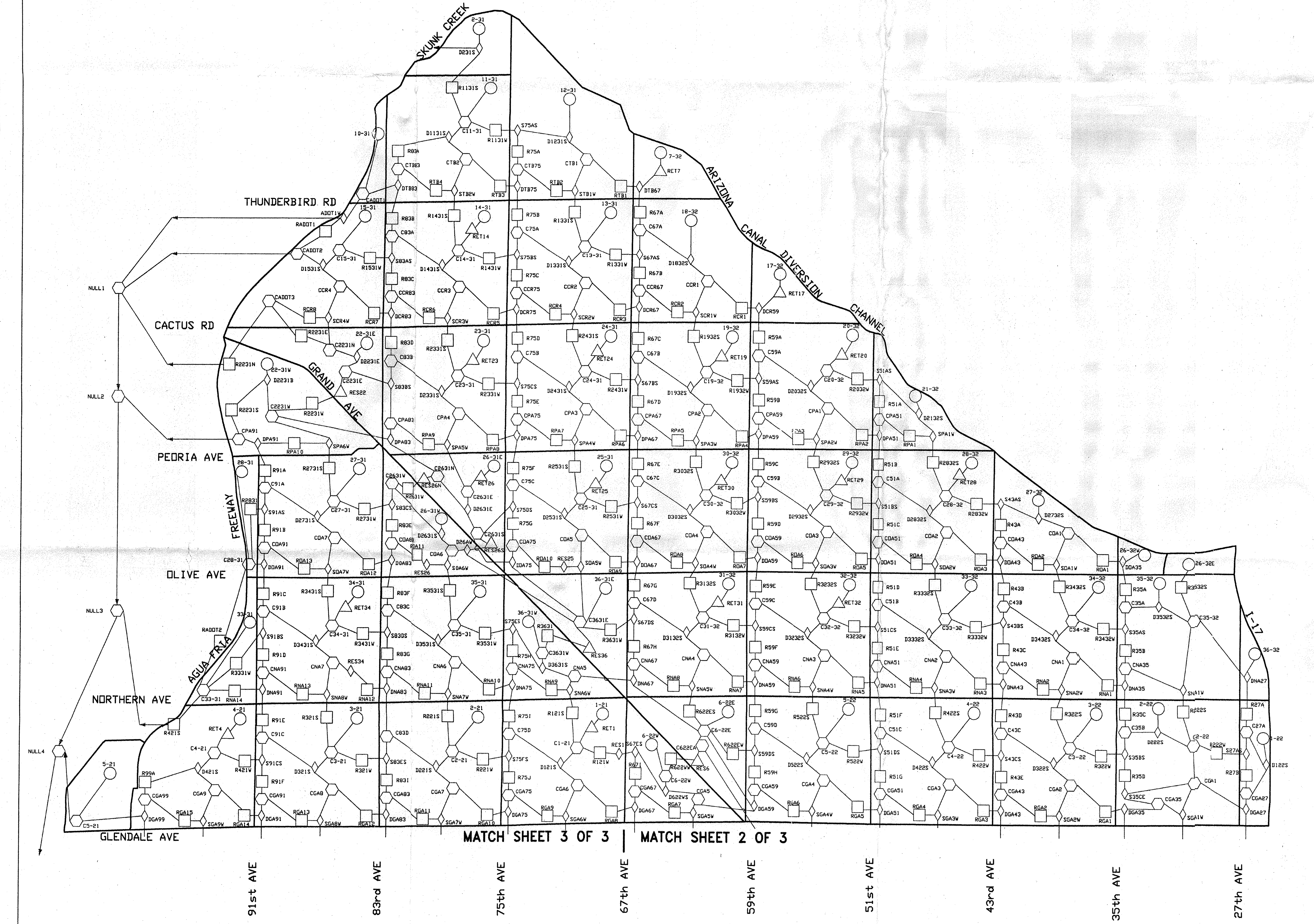
PLATE 4

KAMINSKI HUBBARD engineering, inc.
SURVEYING • CIVIL • HYDROLOGY
4550 N. BLACK CANYON HWY., SUITE C
PHOENIX, ARIZONA 85017
(602) 242-5588

N

SCALE
1200 600 0 1200 2400
FEET

CONTOUR INTERVAL 10 FEET

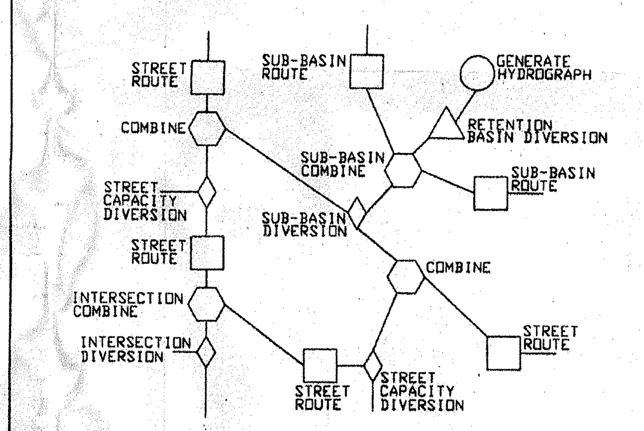


FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

MARYVALE AREA DRAINAGE MASTER STUDY

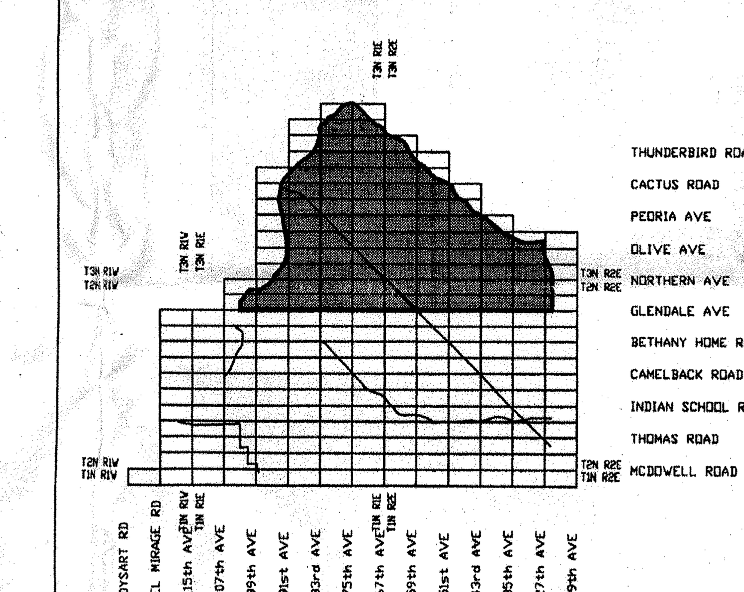
FCD CONTRACT # 93-29

LEGEND



DRAINAGE SUB-BASIN BOUNDARY

INDEX MAP



0 2000' 4000' 6000'

SCALE: 1" = 2000'

EXHIBIT 1
HEC-1 SCHEMATIC

CH2M HILL | WOOD / PATEL

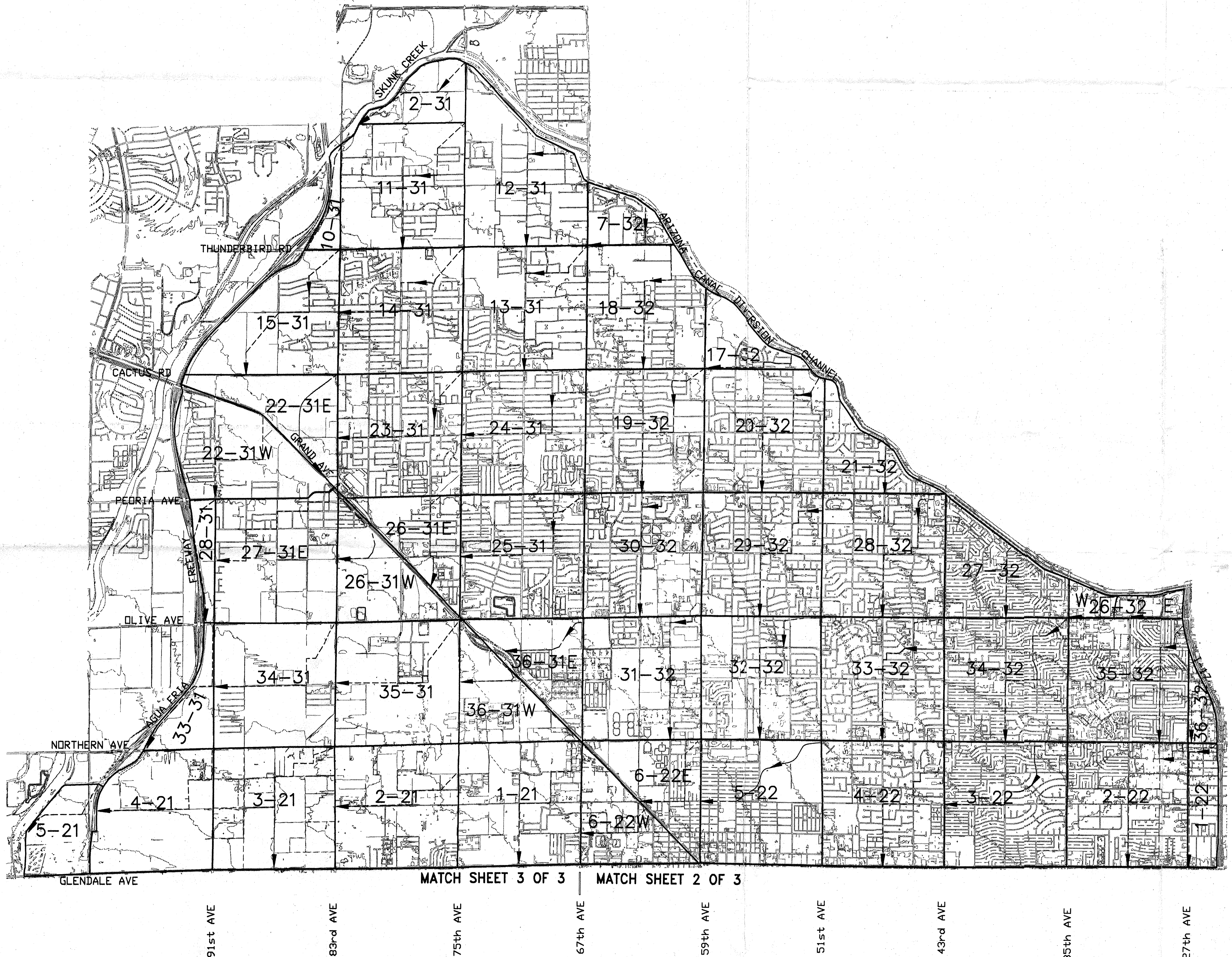
	BY	DATE	
DESIGN	–	–	FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
DESIGN CHK.	–	–	RECOMMENDED BY: DATE
PLANS	–	–	APPROVED BY: DATE
PLANS CHK.	DWD	OCT 96	CHIEF ENGINEER AND GENERAL MANAGER
SUBMITTED BY:		DATE: OCT 96	SHEET 1 of 3

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

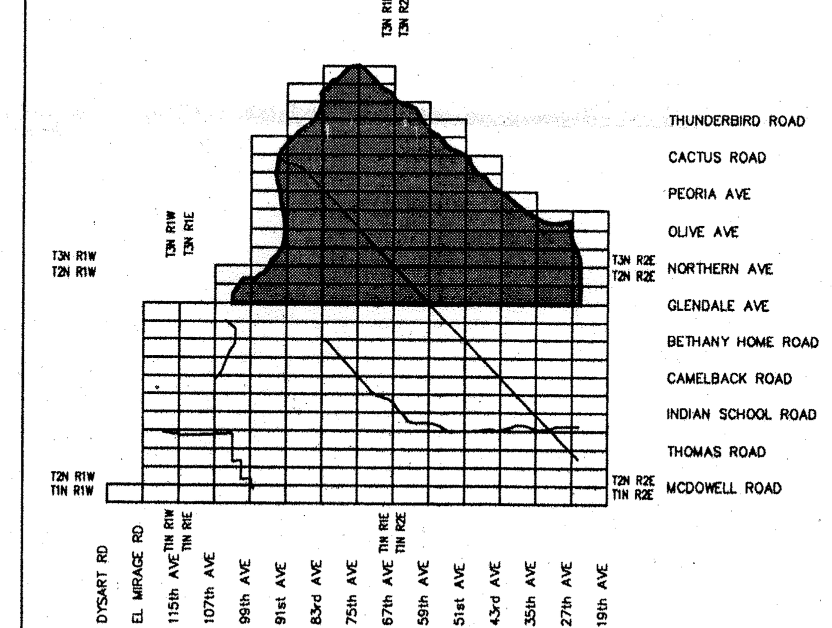
MARYVALE AREA DRAINAGE MASTER STUDY
"FCD CONTRACT # 93-29"

LEGEND

═══	ROAD/PAVEMENT
─┼─┼─	RAILROAD
---1110---	INDEX CONTOUR
16-32 (boxed)	DRAINAGE SUB-BASIN BOUNDARY
16-32	SUB-BASIN I.D. NO.
──◄──	SUB-BASIN FLOW PATH



INDEX MAP

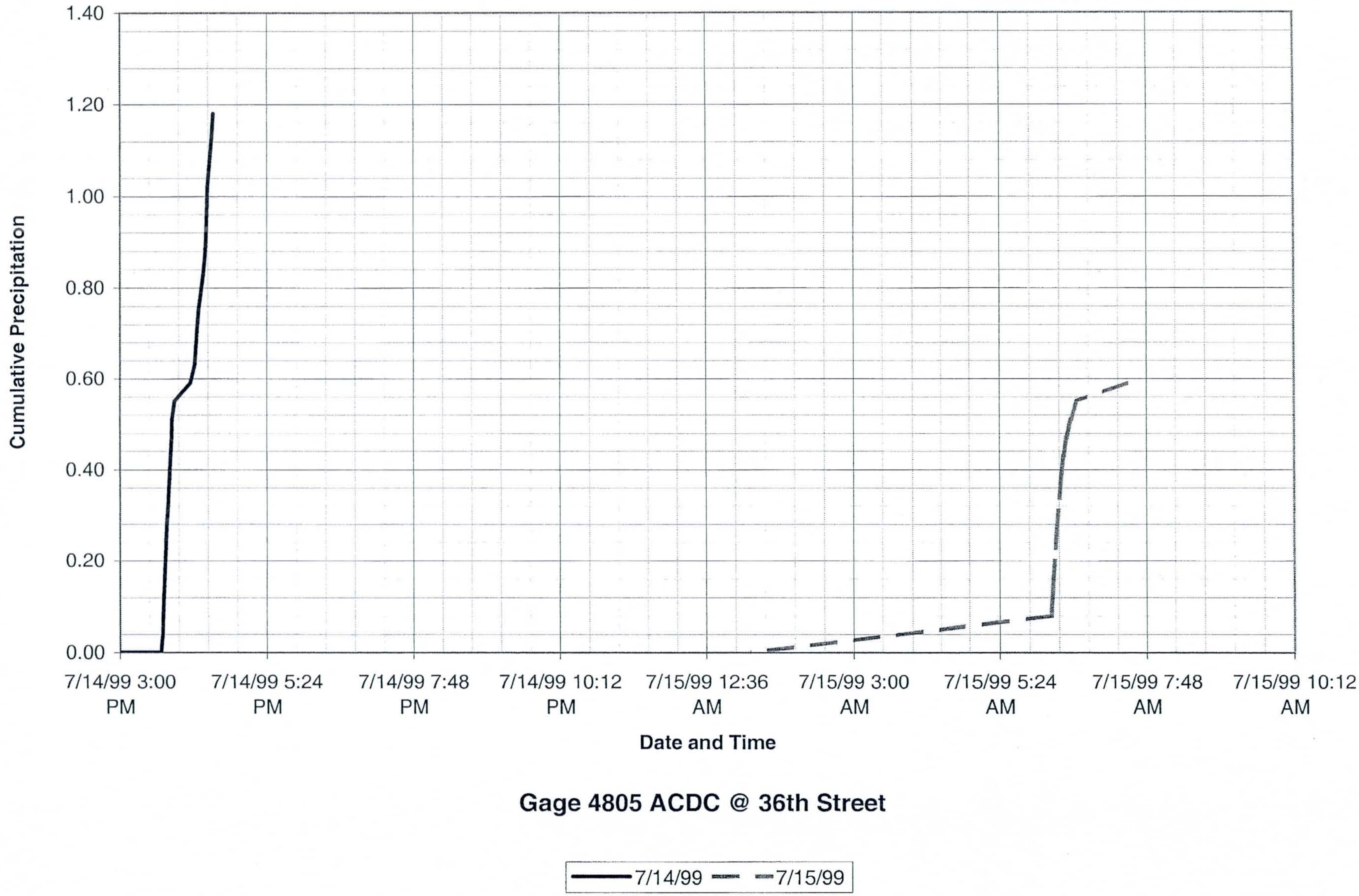


0 2000' 4000' 6000'
SCALE: 1" = 2000'

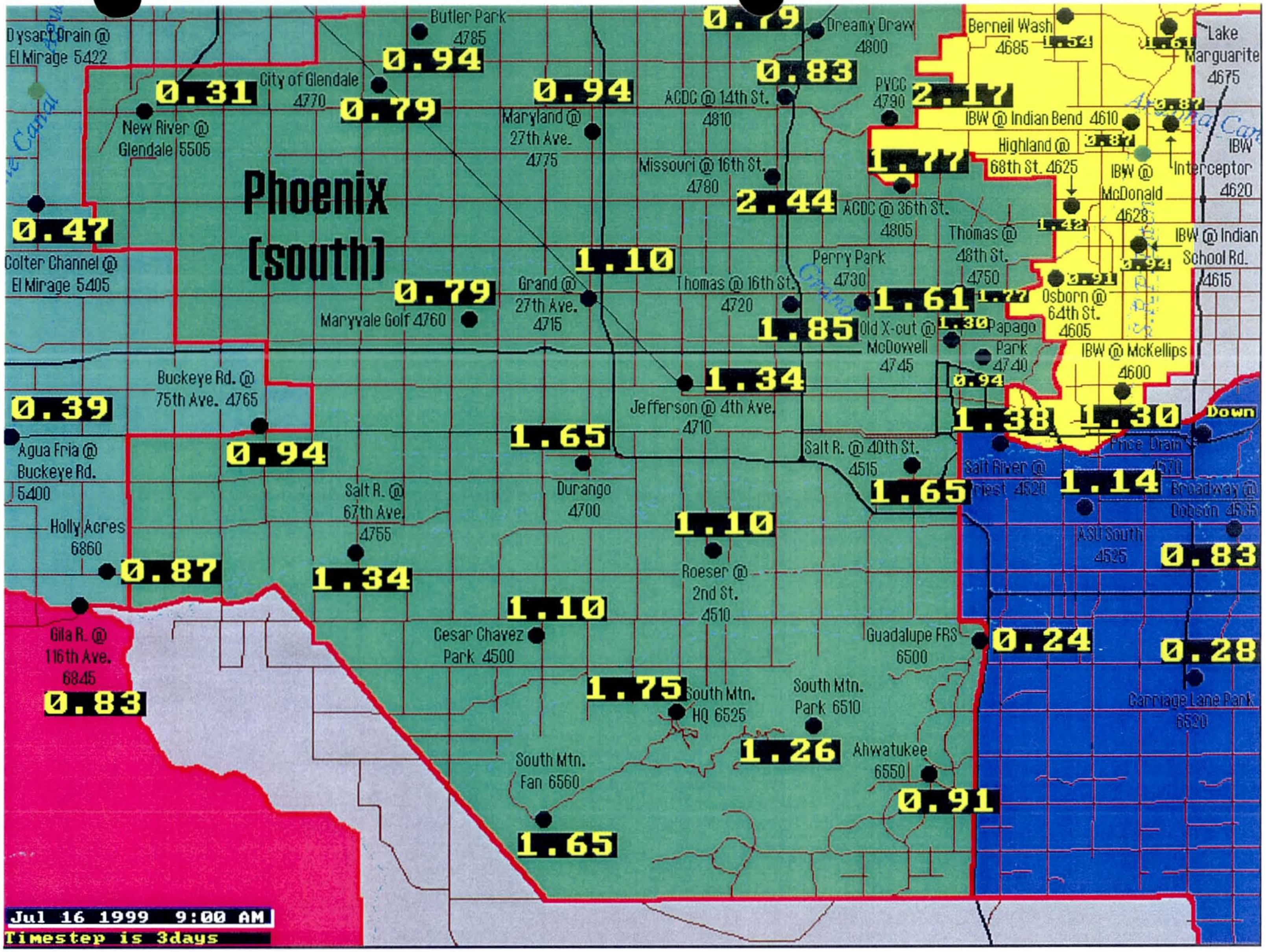
EXHIBIT 2
SUB-BASIN BOUNDARY & FLOW PATH MAP

CH2M HILL | WOOD / PATEL

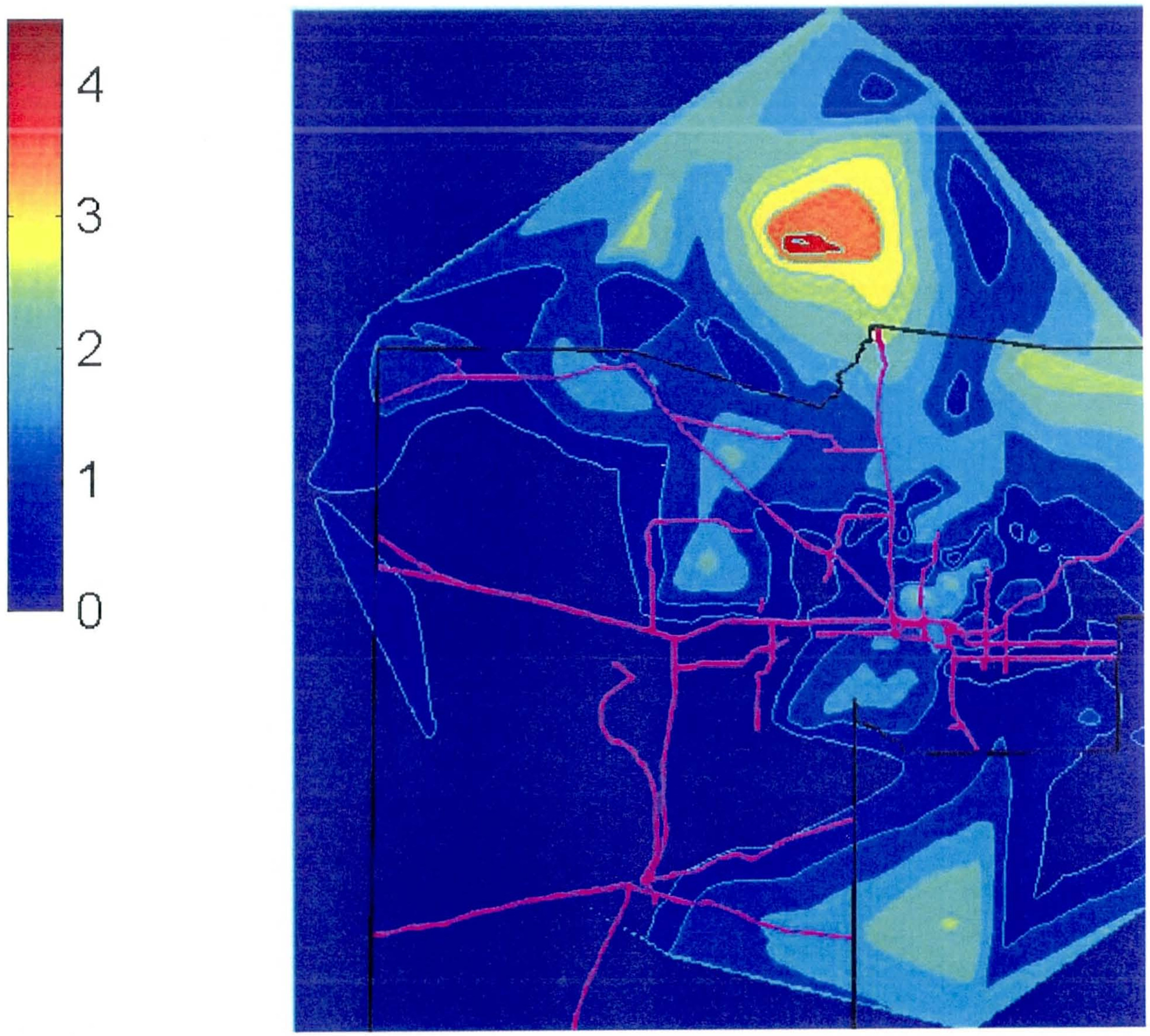
	BY	DATE	FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
DESIGN	–	–	RECOMMENDED BY: DATE
DESIGN CHK.	–	–	
PLANS	–	–	APPROVED BY: DATE
PLANS CHK.	DWD	OCT 96	CHIEF ENGINEER AND GENERAL MANAGER
SUBMITTED BY:		DATE: OCT 96	SHEET 1 OF 3



Gage 4805 ACDC @ 36th Street


Dysart Drain @ El Mirage 5422
0.31
City of Glendale 4770
Butler Park 4785
0.94
0.79
0.94
0.79
Dreamy Draw 4800
0.83
Berneil Wash 4685
Lake Marguarite 4675
New River @ Glendale 5505
Maryland @ 27th Ave. 4775
ACDC @ 14th St. 4810
PVCC 4790
2.17
IBW @ Indian Bend 4610
Highland @ 68th St. 4625
IBW @ McDonald 4628
Interceptor 4620
IBW
Phoenix (south)
Missouri @ 16th St. 4780
2.44
1.77
ACDC @ 36th St. 4805
Thomas @ 48th St. 4750
IBW @ Indian School Rd. 4615
0.47
Colter Channel @ El Mirage 5405
1.10
Perry Park 4730
0.79
Grand @ 27th Ave. 4715
Thomas @ 16th St. 4720
1.61
Osborn @ 64th St. 4605
Maryvale Golf 4760
1.85
Old X-cut @ McDowell 4745
Papago Park 4740
IBW @ McKellips 4600
Buckeye Rd. @ 75th Ave. 4765
1.34
Jefferson @ 4th Ave. 4710
0.94
0.39
1.38
1.30
Down
Agua Fria @ Buckeye Rd. 5400
0.94
1.65
Salt R. @ 40th St. 4515
Salt River @ Priest 4520
1.14
Broadway @ Dobson 4535
Salt R. @ 67th Ave. 4755
Durango 4700
1.65
1.10
ASU South 4525
Holly Acres 6860
0.87
1.34
Roeser @ 2nd St. 4510
0.83
Gila R. @ 116th Ave. 6845
1.10
Cesar Chavez Park 4500
Guadalupe FRS 6500
0.24
0.28
0.83
1.75
South Mtn. HQ 6525
South Mtn. Park 6510
Carriage Lane Park 6520
1.26
Ahwatukee 6550
South Mtn. Fan 6560
0.91
1.65
Jul 16 1999 9:00 AM
Timestep is 3days

3 Days Cumulative Rainfall Depth(ir
09:00:00 7/16/1999



ACDC AT 36TH STREET

CUDIA CITY WASH BASIN

FCD GAGE ID# 4808

STATION DESCRIPTION

LOCATION – This gage is located on the left bank of the sediment pool where Cudia City Wash enters the Arizona Canal Diversion Channel (ACDC) (start of ACDC.) The gage is northwest of the intersection of Camelback Road and 40th Street. Latitude N33 30 09; Longitude W112 00 00. Located in SE1/4 SE1/4 S13 T2N R3E in the Sunnyslope 7.5-minute USGS quad map.

ESTABLISHMENT – The gage was installed on February 24, 1994.

DRAINAGE AREA – 4.82 mi^2

GAGE – The gage is a pressure transducer type instrument with the diaphragm at 0.12 feet gage height relative to the culvert invert of the low flow sediment basin outlet at 0.00 feet gage height, levels of December 26, 1996.

The staff gage reads in gage height.

Two crest gages are on site.

CSG#1 (Lower) has pin elevation 1.98 feet gage height.

CSG#2 (Upper) has pin elevation 6.03 feet gage height.

HISTORY – Pressure transducer installed February 24, 1994. Two crest stage gages and 0-5 foot enamel staff plate installed March 1997.

REFERENCE MARKS

RM1 – brass tablet, COE Cudia #3, on north bank of basin. Elevation 1,251.58 feet MSL or 15.63 feet gage height.

RP1 – Culvert invert has elevation 1,235.95 feet MSL, or gage height 0.00 feet. (Design has invert at 1,236.00 feet.)

CHANNEL AND CONTROL – The control is a 36-inch pipe up to gage height 7.00 feet where flow begins over a 200-foot wide relatively sharp weir.

RATING – The current rating is Rating #1 and is dated February 24, 1994. The rating is a combination of an HY8 culvert analysis for culvert flows and weir equation solution for flow over the weir.

DISCHARGE MEASUREMENTS –

Very difficult and dangerous for most or all flows.

POINT OF ZERO FLOW – The PZF is the invert of the culvert outlet at gage height 0.00 feet.

FLOODS – A peak discharge of 324 cfs occurred on July 14, 1999.

REGULATION – The sediment basin regulates flows of Cudia City Wash.

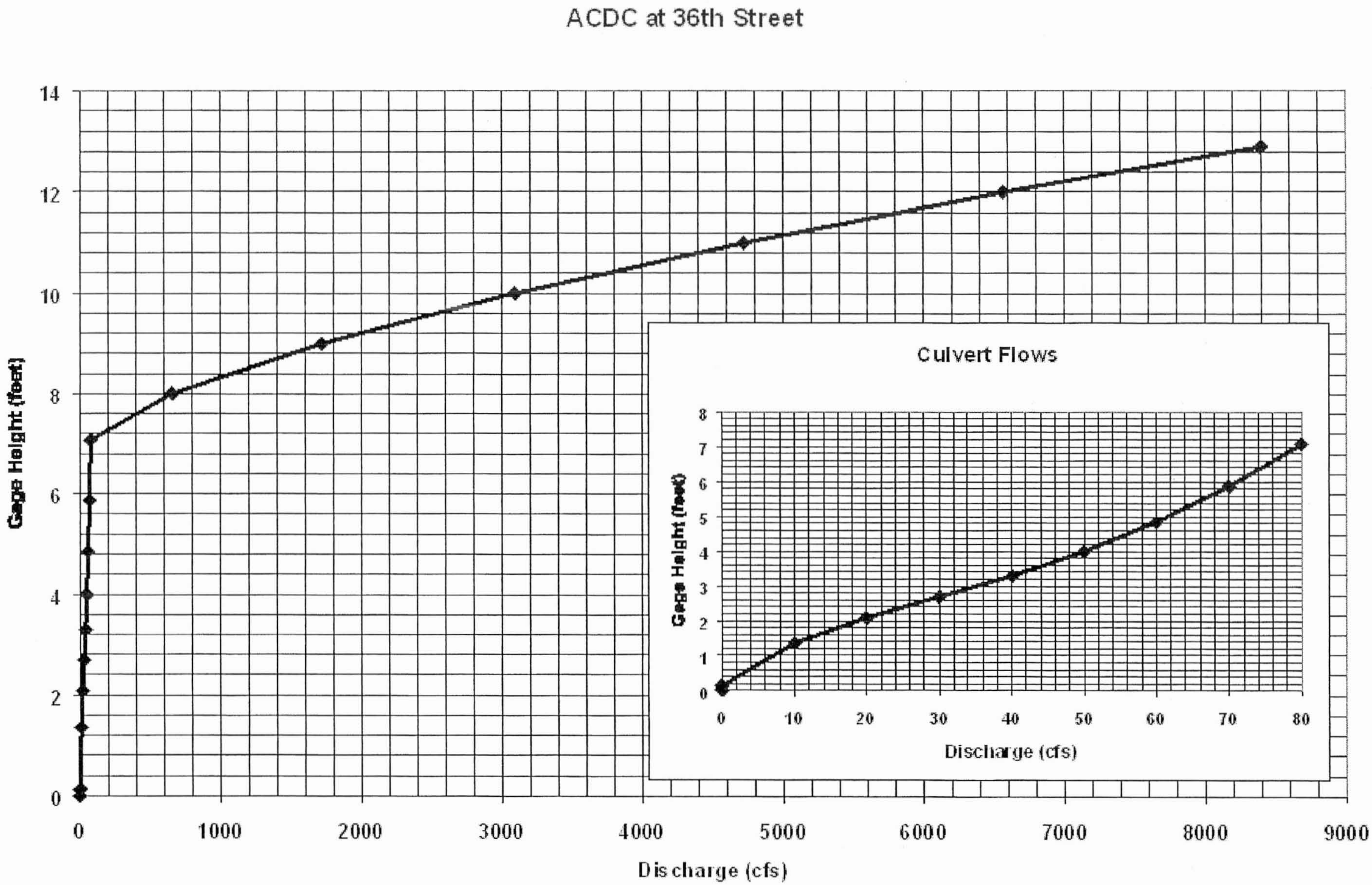
DIVERSIONS – None known

ACCURACY – Fair – weir equation in question. Verification with measurement if possible.

JUSTIFICATION – Monitor inflows in the upper ACDC.

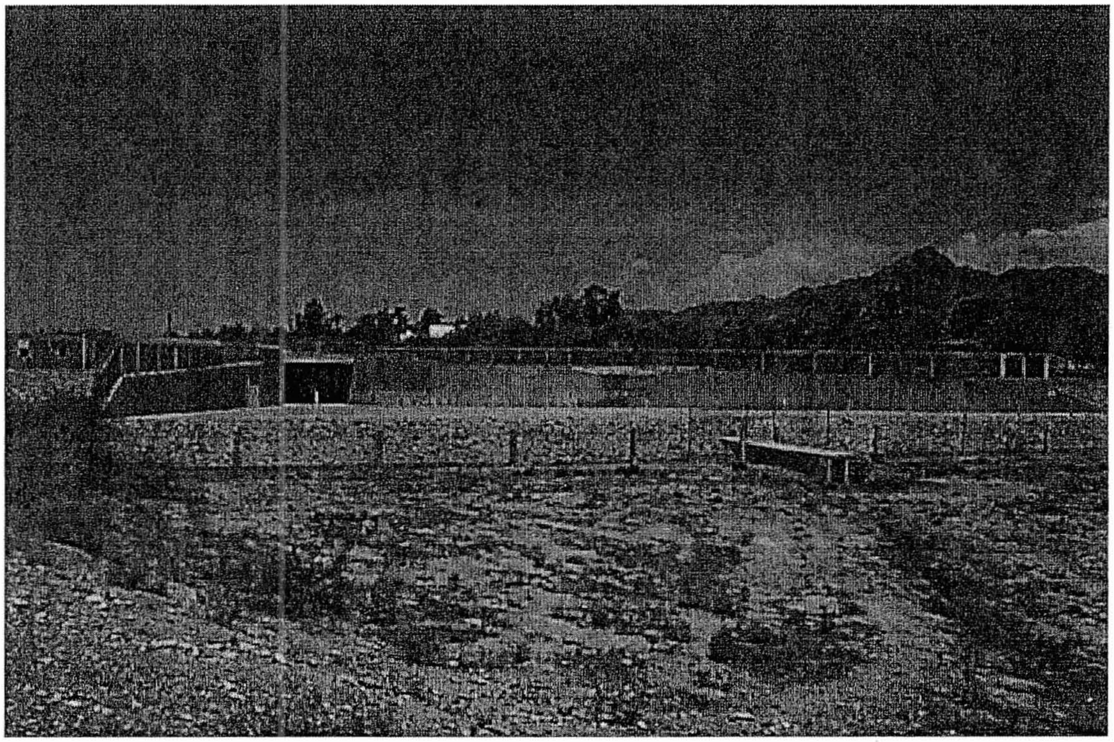
UPDATE February 7, 2000

DE Gardner


ACDC at 36th Street
Gage Height (feet)
Discharge (cfs)
0
2
4
6
8
10
12
14
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
Culvert Flows
Gage Height (feet)
Discharge (cfs)
0
1
2
3
4
5
6
7
8
0
10
20
30
40
50
60
70
80

SITE: ACDC AT 36TH STREET

GAGE ID: 4808



Gage is located on the north bank of the Arizona Canal Diversion Channel just northwest of the intersection of Peoria and 43rd Avenues.

SELECT ANY OF THE FOLLOWING FOR ADDITIONAL INFORMATION:

FOR CURRENT CONDITIONS CLICK HERE

STATION DESCRIPTION

RATING CURVE

RATING TABLE (Adobe PDF format)

BASIN CAPACITY INFORMATION (Adobe PDF format)

GAGE CROSS SECTION PLOT

SITE DATA

DRAINAGE AREA	

	4.82 MI^2
JURISDICTION	**PHOENIX, ARIZONA**
WATERSHED	**ACDC**
SECTION/TOWNSHIP/RANGE	**SE1/4 SE1/4 S13 T2N R3E**
LATITUDE	**N 33 30 09**
LONGITUDE	**W 112 00 00**
USGS QUAD MAP	**SUNNYSLOPE 7.5-MINUTE**
INSTALLATION DATE	**FEBRUARY 24, 1994**
STAGE GAGE TYPE	**PRESSURE TRANSDUCER**
STAFF GAGE	**ONE**
CREST STAGE GAGE	**TWO**
STAGE GAGE ELEVATION	**0.12 FEET GAGE HEIGHT**
POINT OF ZERO FLOW	**0.00 FEET GAGE HEIGHT**

MAXIMUM FLOOD	**324 CFS**	**7.67 FEET G.H.**	**JULY 14, 1999**

RATING INFORMATION

RATING TABLE			
CURRENT RATING NUMBER 1, DATED FEBRUARY 24, 1994			
GAGE HEIGHT (FEET)	**DISCHARGE (CFS)**	**GAGE HEIGHT (FEET)**	**DISCHARGE (CFS)**
0.00	0	10.00	3,094
5.00	62	11.00	4,720
7.00	80	12.00	6,565
8.00	660	12.90	8,392
9.00	1,720		

WATER YEAR PEAKS

Water Year	Peak Gage Height (feet)	Peak Discharge (cfs)	Date of Peak
2002			

2001	1.02	7	10/27/00
2000	0.89	6	3/6/00
1999	7.67	324	7/14/99
1998	2.00	19	3/26/98
1997	1.02	7	9/12/97
1996	1.23	9	11/1/95
1995	1.60	13	9/28/95
1994	1.20	9	3/19/94

FLOOD EVENT PEAKS

Event Date	Peak Discharge (cfs)	Water Year
SEE	ABOVE	TABLE

CLICK HERE to download a *.pdf (Adobe Acrobat) file containing all daily mean totals for this station.

CREST STAGE GAGE INFORMATION

CREST GAGE NUMBER	PIN ELEVATION (FEET, GAGE HEIGHT)	CREST GAGE NUMBER	PIN ELEVATION (FEET, GAGE HEIGHT)
1 (LOWER)	1.98	2 (UPPER)	6.03

STAFF GAGE INFORMATION

STAFF GAGE RANGE	LOW POINT	LOCATION
0 - 5 FEET	0.00	

ATTACHMENT 8

WHITE TANKS / AGUA FRIA
AREA DRAINAGE MASTER STUDY

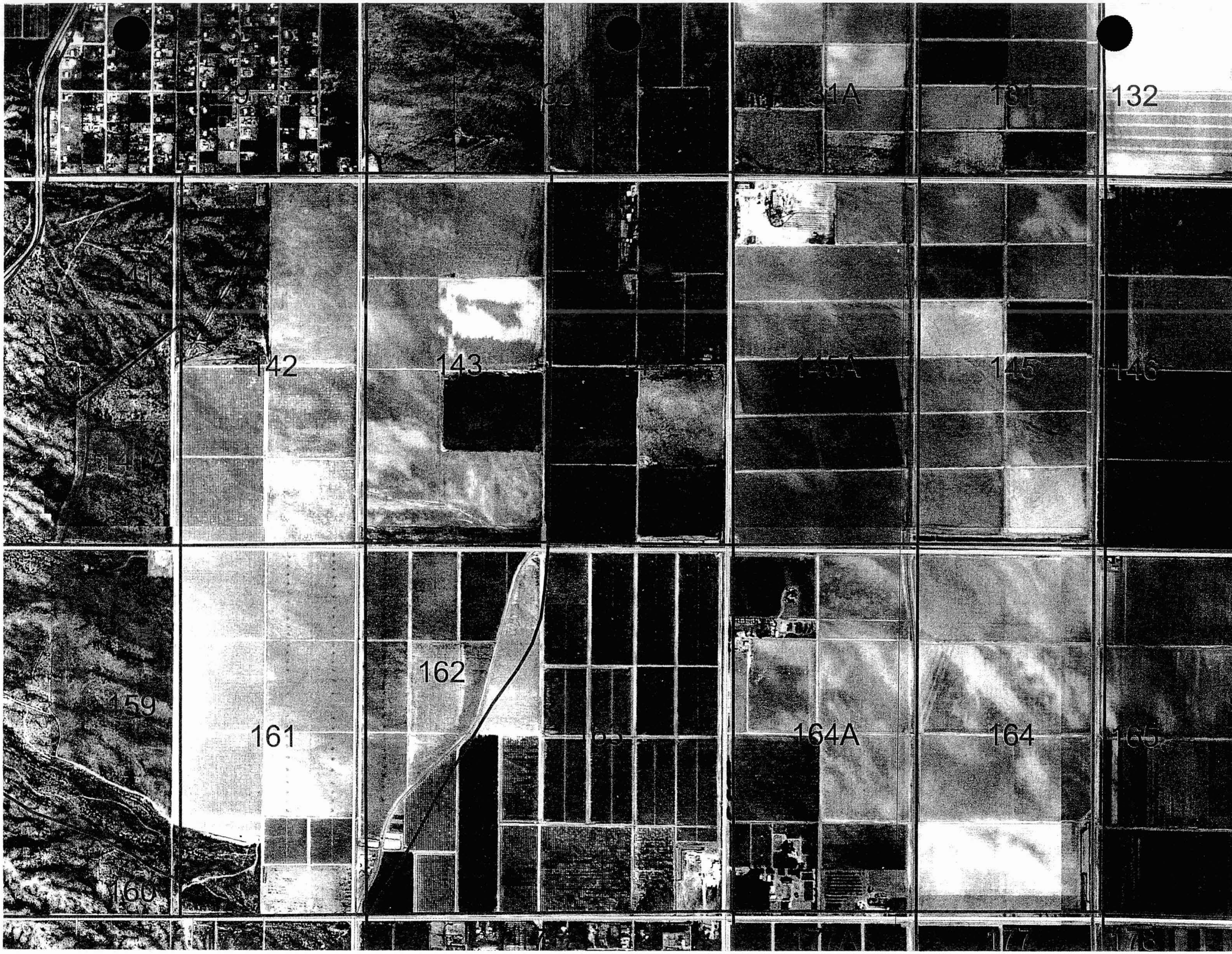
Part A:
Flood Study Technical Data Notebook

Prepared For:
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
2801 West Durango
Phoenix, Arizona 85009

October, 1992

REGISTERED PROFESSIONAL ENGINEER (CIVIL)
CERTIFICATE NO. 15594
MARK T. GAVAN
Date Signed 10-21-92
ARIZONA, U.S.A.

Prepared by:
THE WLB GROUP, INC.
333 East Osborn Road, Suite 380
Phoenix, Arizona 85012
(602) 279-1016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SECTION 3: HYDROLOGIC ANALYSIS

3.1 METHOD DESCRIPTION

The hydrologic methodology incorporated in the White Tanks/Agua Fria Area Drainage Master Study (ADMS) utilizes the new "Hydrologic Design Manual for Maricopa County, Arizona" dated April, 1990. This manual is a comprehensive compilation of technical procedures for the estimation of rainfall-runoff which is used for the purpose of designing and analyzing drainage facilities in Maricopa County.

Hydrologic parameters were calculated for each subbasin within the study area. The WLB Group, Inc. created a worksheet utilizing the Lotus 1-2-3 program in which subbasin parameters; such as flow length, slope, land use, soil type, vegetative cover, and soil moisture condition, were used to calculate average Green-Ampt loss rate parameters and lag time for each subbasin. These values were then input into a computer program supplied by the Flood Control District of Maricopa County (FCDMC) called MCUHP2 (Maricopa County Unit Hydrograph Procedure 2) dated October 2, 1990. This program calculates unit hydrographs based on the U.S. Army Corps of Engineers S-graphs that were developed for the Phoenix Area. The program also creates HEC-1 input files that can be utilized within the HEC-1 Flood Hydrograph Package computer program created by the U.S. Army Corps of Engineer's Hydrologic Engineering Center. The HEC-1 program used for this study was the June 1, 1988 version and was acquired directly from the U.S. Army Corps of Engineers Hydrologic Engineering Center.

3.2 PARAMETER ESTIMATION

Due to the large amount of base data generated by this Area Drainage Master Study, separate notebooks for each physical parameter calculated are supplied as appendices to this report and will be referred to when discussing each parameter calculated.

3.2.1 Drainage Area Boundaries

The drainage area for the White Tanks/Agua Fria ADMS is approximately 220 square miles with approximately 2/3 of the watershed draining to the Gila River and 1/3 of the watershed draining to the Agua Fria River. The drainage area is bounded on the north by McMicken Dam and Grand Avenue; on the east by the Agua Fria River; on the south by the Gila River; and on the west by Dean Road and the White Tank Mountains. Several incorporated communities are located within the study area including the Cities of Avondale, El Mirage, Goodyear, Litchfield Park, and Surprise; the Town of Buckeye; Luke Air Force Base; and strip annexed areas of the Cities of Glendale and Phoenix.

Prominent features located within the drainage area are the White Tank Mountains, White Tanks Flood Retarding Structures #3 and #4, Interstate 10, interim Estrella Freeway, Atchison Topeka and Santa Fe Railroad, Southern Pacific Railroad, Airline Canal, Buckeye Canal, Beardsley Canal, Roosevelt Irrigation District Canal, Litchfield Park Detention Facility, Dysart Drain, Tuthill Dike, Bullard Wash, Caterpillar Proving Grounds, Case Proving Grounds, White Tank Mountain Regional Park, Agua Fria River, and Gila River. (Refer to the attached 11" x 17" Study Area Map.)

Subbasins were delineated using 1" = 400', 2-foot contour interval topographic mapping developed for this study by Cooper Aerial and Western Air Maps. Also, aerial photographs were used and field reconnaissance trips were taken to determine subbasin boundaries that were not readily apparent on the maps. Points of concentration that were of particular interest were also used to define subbasin boundaries. Refer to the following 11" x 17" Drainage Area Map. A 1" = 4000' Drainage Area Map is also provided with the hardcopy of the HEC-1 model located in Appendix C under separate cover.

3.2.2 Physical Parameters

3.2.2.1 *Unit Hydrograph Calculation*: The Phoenix Valley S-graph was incorporated per instructions from the FCDMC to calculate unit hydrographs for use within the HEC-1 model. This, along with the use of Green-Ampt loss rate parameters, forms the basis for calculating runoff hydrographs for each subbasin throughout the watershed. The Phoenix Valley S-graph was selected based on the criteria of being applied to a large, mostly undeveloped watershed. The majority of the watershed is in agricultural uses with a lesser degree of desert and mountainous terrain and even fewer areas of urban development.

The Phoenix Valley S-graph was developed by the U.S. Army Corps of Engineers and can be found in "New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology, Part 1", U.S. Army Corps of Engineers, Los Angeles District, October, 1974.

The MCUHP2 program uses the Phoenix Valley S-graph to calculate unit hydrographs. Input requirements for MCUHP2 include basin area, basin lag, and Green-Ampt loss rates.

A number of variables are involved in calculating loss-rate parameters for the Green-Ampt method. The "Hydrologic Design Manual for Maricopa County" describes the steps involved in calculating these parameters and this manual is available from the Flood Control District of Maricopa County upon request. It would be repetitive and cumbersome to relate all of the details involved in this procedure and it is left up to the individual to acquaint themselves with this methodology and to refer to the manual during the following description of procedures if the reader is not familiar with them.

The WLB Group, Inc. created a Lotus 1-2-3 worksheet to help reduce the amount of hand calculations involved in developing the input parameters for MCUHP2. The FCDMC has recently updated this worksheet and now includes it with the new Hydrologic Design Manual for use by its consultants. The following steps were utilized within the worksheet to calculate basin lag time and average Green-Ampt loss rate parameters within each subbasin.

1. Measure flow path length and calculate elevation difference. This may be broken down into incremental elements representing areas of the same hydrologic properties and basin slopes.

2. The representative slope is then calculated according to the following formulas:

$$I = (Li^3 + Hi)^{.5}, \text{ where } i = 1, 2, 3, \ldots n$$

and

L_1, L_2, L_3, etc. Incremental Lengths Along the Longest Flow Path, Miles

H_1, H_2, H_3, etc. Incremental Elevation Differences for Each Length, Feet

and representative slope is then calculated from:

$$\text{Avg. } S = (L \div I)^2 \text{ ft/mi}$$

where

L = Total Length of the Longest Flow Path
I = Value From Previous Formula

This average slope formula will take into account differences within a watershed due to varying topographic situations and varying slopes. This formula was taken from the "Hydrology Manual for Engineering Design and Floodplain Management Within Pima County, Arizona". It should be noted that "I" and "S" are usually calculated in feet and feet/feet respectively. But for this study Li was computed in miles and, therefore, S is in feet/mile for use in the lag equation that follows.

3. The lag for each subbasin is then calculated based on a formula created by the U.S. Army Corps of Engineers (1974):

$$\text{Lag} = 1.2\,(L * Lca \div S^{1/2})^{0.38}$$

where

L = Length of Longest Watercourse, miles
Lca = Length Along Longest Watercourse, Measured Upstream to a Point Opposite the Center of the Area, miles
S = Overall Average Slope of Longest Watercourse Between Headwater and Collection Point, ft/mile

Note: To obtain the Lag (in hours) for any area, multiply the lag obtained from the formula by $\bar{n}/.050$ or $20\bar{n}$.

$\bar{n}$ = Visually Estimated Mean of the N (Manning's Formula) Values of all the Channels Within an Area

4. The land use classification is then chosen along with an estimated percentage of vegetative cover and percentage of impervious areas. If the impervious areas are noncontiguous and undeveloped, only 50% of that impervious area is used for calculation purposes as directed by the Flood Control District of Maricopa County.

Aerial photographs were used along with zoning maps to help classify areas of differing land uses. (See the attached 11" x 17" Current Land Use and Zoning Map.) The aerial photographs also helped to define the percentage of vegetative cover in an area. Field investigation, along with numerous photographs, also help document this procedure. (See Appendix D for typical photographs of the area.)

The soil moisture condition for the calculation of DTHETA, and the surface retention loss, IA, are based upon the land use type. For instance, irrigated agricultural land is assumed to be in a saturated condition with a corresponding surface retention loss of 0.50 inches, residential land is assumed to be in a normal moisture condition with a corresponding surface retention loss of 0.12 inches, and desert land is assumed to be in a dry condition with a corresponding surface retention loss of 0.35 inches. These parameters were directed by the Flood Control District of Maricopa County. Refer to the "Hydrologic Design Manual for Maricopa County" for a more indepth discussion of DTHETA.

The rate of hydraulic conductivity to bare ground hydraulic conductivity, CK, is also a function of the percent of vegetative cover. This value was calculated as an average value for each subbasin. Refer to Fig. 4.10 in the "Hydrologic Design Manual for Maricopa County" and to Appendix E, Volume 6 of 15 for examples of the parameter averaging.

5. The next step was to planimeter areas of distinct soil classification within each subbasin and input the percentage of area for each soil group into the worksheet. This was accomplished by using Soil Conservation Service soil survey maps created for Maricopa County. Subbasins were transposed on these maps and distinct soil classification areas were then planimetered. Each soil group has distinct values associated with it for calculation of the Green-Ampt loss rate parameters. These parameters are then averaged based upon the percentage of different soil classifications within each subbasin. Refer to Appendix E, Volume 6 of 15, to see how parameter averaging is performed. The following 11" x 17" Hydrologic Soil Group Map shows locations of various types of hydrologic soil groups within the study area.

6. The average loss rate values, along with basin area and lag time, are then used as input into the FCDMC's computer program MCUHP2 to calculate a unit hydrograph for the HEC-1 model. This was done for each subbasin within the watershed; the corresponding S-graph Parameter sheets for each subbasin are included under separate cover in Appendix E. This appendix also includes a copy of the Soil Loss Rate Tables used in this study. A copy of the MCUHP2 input data as backup documentation to verify that the data was input correctly is located in Appendix F under separate cover.

3.2.2.2 *Channel Routing*: Channel routing throughout the watershed was accomplished by using the normal depth (modified Puls) routing procedure as outlined in HEC-1. This method utilizes an eight point typical cross section along with an average channel slope, channel length and typical Manning's n-values. The 1" = 400', 2-foot contour interval topographic mapping was incorporated to determine typical cross sections and channel geometry.

Two iterations of the HEC-1 model were run to calculate velocities in each routing reach. Initially, velocities were assumed for each routing reach within the watershed. After this initial model had been run, normal depth computations were performed to estimate velocity for each routing reach utilizing the computed discharges. The velocity estimates were based on a trapezoidal channel shape with an average Manning's n-value for the cross section. The resulting velocity estimates were then used to compute the number of steps for each channel routing reach. The number of steps was set equal to (reach length ÷ (average velocity x time interval)). The second iteration of the HEC-1 model was then run to produce the final discharges used in this study. Channel routing parameters are located in Appendix G and Velocity Calculations are located in Appendix H. Both of these appendices are under separate cover.

3.2.2.3 *Stage-Storage Discharge Parameters*: Stage-storage-discharge tables were created to model the numerous ponding areas located throughout the watershed. These areas are typically comprised of ponding behind structures such as dams, roadway embankments, railroad embankments or canal banks. Outfalls from these ponding areas include culverts, bridges, and weir flow over the top of the embankment. A list of existing drainage structures is located in Appendix I under separate cover and can also be found in the HEC-1 input documentation.

Ponding areas were identified using the 1" = 400' topographic mapping. The stage-storage data was computed by planimetering areas between adjacent contours and computing average volumes associated with that area and depth.

Bureau of Public Roads culvert charts were incorporated to calculate outflow from ponding areas where appropriate. The weir flow equation was used when flow overtopped an embankment or overtopped a particular impoundment. Stage-Storage Discharge tables can be found in Appendix I under separate cover.

3.2.2.4 *Diversions*: Numerous diversion tables were also incorporated throughout the watershed. This was due to the fact that a majority of the watershed is fairly flat with no well defined channels to contain the runoff. Consequently, flooding in the study area is characterized by wide, shallow flow paths which are easily diverted along man-made obstructions, such as railroads and irrigation canals.

Agriculture is the predominant practice throughout this area and fields are separated by major mile, half-mile, and farm access roadways. These roadways, along with irrigation canals, tend to pond water at the southeastern corner of the fields. From this point, flows break over the intersection of the two roads and will either continue east at the capacity of that particular road, flow overland to the southeast spreading out into another agricultural field, or flow south at the capacity of that road. It is not uncommon to have a three-way split at these locations.

These types of diversions were calculated by taking a cross section upstream along the centerline of each major road and computing weir flow as it applies to each diversion.

A second type of diversion, using the same cross section method along the centerline of the road, was to model the flow with a normal depth calculation. This was used when weir flow was not applicable at an intersection.

The third type of diversion usually involved a culvert analysis. If an embankment was present and the culvert capacity was exceeded, a diversion would take place above a certain limiting elevation. This diversion was calculated using either weir flow or normal depth methodology depending on the situation.

Finally, the fourth type of diversion would take place at a canal bank. Diversions were calculated by weir flow if the flow was to cross over the top of the canal bank and continue downstream or by normal depth methods if the flow was diverted along the upstream bank of the canal. Diversion tables can be found in Appendix I, under separate cover, and the Drainage Area Map identifies where diversions take place in the watershed. Each diversion is distinctly labeled except for the diverts associated with subbasins 43 and 43-1 through 43-8 - where space limitations on the Drainage Area Map required their exclusion. Refer to the exhibit on the following page for an enlargement of this area.

3.2.2.5 *Hydrograph Combinations*: The HEC-1 model for the White Tanks/Agua Fria ADMS was set up so that the area associated with each hydrograph combination was directly input into the model. The criteria to be followed, as directed by the FCDMC, was to hand calculate the total area that would be contributing to any given concentration point. Diversions were assumed to be contributing the whole area to the next concentration point, therefore, the corresponding area assigned to each concentration point would correspond to the total area of all subbasins that drain, either partially or fully to that point. The calculated areas were checked thoroughly by the FCDMC and concurrence was reached for the areas submitted on the HEC-1 model. This procedure was undertaken because the HEC-1 model assigns an area of zero to the diverts and carries that area to the next concentration point. Because rainfall depth decreases with increases in drainage area, the zero area associated with the diverts would, in some instances, result in overestimating peak discharges.

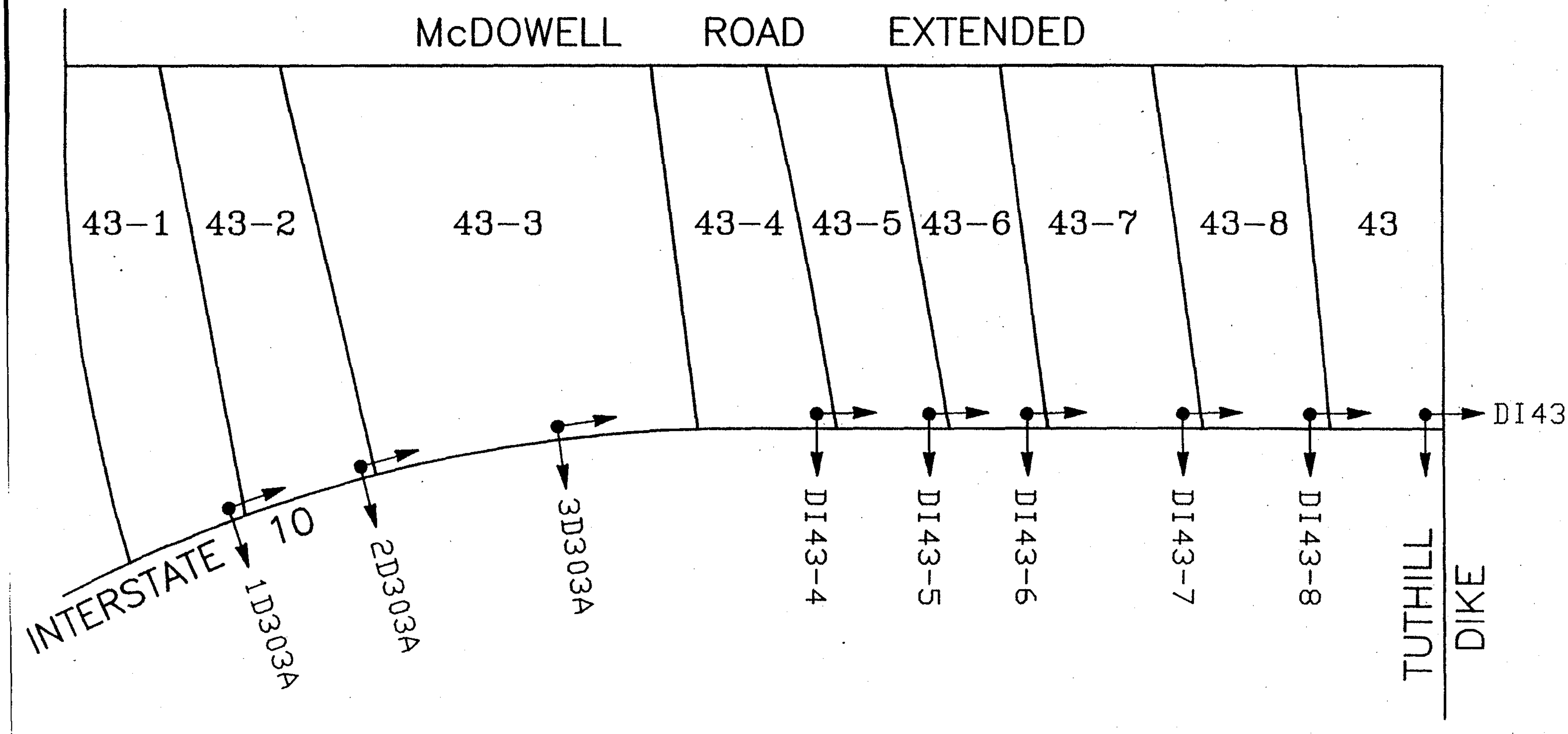
3.2.2.6 *Manning's N-Value Documentation*: Manning's n-value determinations for subbasins and routing reaches within the watershed were made based on field reconnaissance, aerial photographs, picture documentation, and sound engineering judgement. Typical "n" values were designated for agricultural areas, n = .12, and urban areas, n = .03, and these values were mutually agreed upon by The WLB Group and the FCDMC. Desert and mountainous areas have varying "n" values ranging from .03 to .20 and were incorporated based on the hydrologic conditions of that subbasin. Picture documentation of typical basin "n" values and channel and overbank "n" values are presented in Appendix D, under separate cover.

3.2.3 Statistical Parameters

No statistical analysis was performed with the White Tanks/Agua Fria ADMS as stream gage data is not available in this area.

It should be pointed out, however, that the Phoenix Valley S-graph used to compute the unit hydrographs is based on a statistical analysis of streamflow in and around Maricopa County (U.S. Army Corps of Engineer, 1974).

DIVERSIONS FOR SUBBASINS 43 - 43-8



WHITE TANKS / AGUA FRIA AREA DRAINAGE MASTER STUDY

N.T.S.

The WLB Group WLB

3.2.4 Precipitation

Precipitation data for the White Tanks/Agua Fria ADMS was developed from criteria as presented in the "Hydrologic Design Manual for Maricopa County". Initially, The WLB Group was instructed to use the 100-year, 6-hour storm to compute peak discharges. This, along with a new depth-area reduction curve designed for Maricopa County and 6-hour rainfall distribution patterns based upon drainage area, was incorporated into the 100-year model.

Sensitivity analyses were then run and tested against the 100-year, 24-hour storm. The 24-hour storm gave larger peak discharges as the area contributing to a watercourse increased. These discharges also increased uniformly downstream, whereas, the 6-hour storm did not.

The 6-hour storm produced larger peak flows for smaller watersheds (approximately .5 square miles or less), but, as the size of the area increased, the peak flows would, in some cases, decrease in a downstream direction. This was due to the sharp increase in rainfall intensity associated with the rainfall distribution patterns for small drainage areas. This discrepancy was the reason that the 100-year, 24-hour storm was chosen to model the watershed and to ultimately delineate the 100-year floodplains.

Precipitation amounts were developed for different return periods and frequency storms using the procedure stated in the "Precipitation-Frequency Atlas of the Western United States, Volume VIII - Arizona, NOAA Atlas 2," published by the National Weather Service's National Oceanic and Atmospheric Administration. This data is presented in Appendix A in the back of this report. Depth-area reduction of point rainfall was also taken from a graph in NOAA Atlas 2 since the 24-hour storm was used, and the Soil Conservation Service Type II rainfall distribution pattern was used to distribute the rainfall data accordingly.

3.2.5 Gage Data

No stream gages are located in the study area.

3.3 CALIBRATION

Due to the lack of stream gages or precipitation data in the study area, it is difficult to calibrate peak discharges computed in the HEC-1 model. However, a few previous studies have been performed on an isolated basis in different areas of the watershed. The new discharges were compared to the previous values to ascertain whether the results seemed reasonable. The reports and hydraulic analyses that WLB compared its results to are listed as follows:

1. "A Hydrologic Analysis of the White Tanks Flood Retarding Structures #3 and #4", by the Flood Control District of Maricopa County (FCD), October, 1989

INPUT PARAMETER COMPARISONS

Hydrologic Parameters	WLB	FCD
Storm Frequency and Duration	100-Year, 24-Hour	100-Year, 24-Hour
Rainfall Amount	4.03 In.	4.20 In.
Tabulation Interval	5-Minute	15-Minute
Loss Rate	Green-Ampt	SCS Curve Number
Distribution Pattern	SCS Type II	SCS Type II
Areal Distribution	NOAA Atlas II	None
Hydrograph Development	COE Phoenix Valley S-Graph	COE Phoenix Mountain S-Graph SCS Unit Hydrograph
Routing Method	Normal Depth	Normal Depth Kinematic Wave

COMPARISON OF DISCHARGES

Location	Discharges, CFS WLB	FCD
Inflow to White Tanks F.R.S. #3	6649	7640
Inflow to White Tanks F.R.S. #4	6026	5830

These discharges are reasonably close and the differences may be attributed to FCD's rainfall amount of 4.20 inches versus WLB's amount of 4.0 inches. Also, FCD used the SCS Curve Number Loss Rate while WLB incorporated the FCD's new methodology which incorporates Green-Ampt loss rate parameters. Also, a 15 minute time interval was used in the FCD study while a 5 minute time interval was utilized in this study.

2. "Conceptual Drainage Report for Litchfield Park Detention Facility", by Coe and Van Loo, June, 1989.

3. "Flow Estimation to Camelback and Dysart Roads", by Boyle Engineering Corporation, April, 1988.

4. "Hydrologic Evaluation, Litchfield Park Dam, Maricopa County, Arizona", by Dames & Moore, January 1986.

INPUT PARAMETER COMPARISONS

Hydrologic Parameters	WLB	CVL	Boyle	D & M
Storm Frequency & Duration	100/24	100/24	100/24	100/24
Rainfall Amount	4.03 In.	3.75 In.	3.77 In.	3.90 In.
Tabulation Interval	5-Min.	10-Min.	15-Min.	N/A
Loss Rate	Green-Ampt	SCS Curve	SCS Curve	SCS Curve
Distribution Pattern	SCS Type II	SCS Type II	SCS Type II	SCS Type II
Aerial Distribution	NOAA At. II	None	None	N/A
Hydrograph Development	COE Phx. Valley S-Gr	SCS Unit Hydrograph	SCS Unit Hydrograph	N/A
Routing Method	Norm. Depth	Kinematic	Kinematic	N/A

COMPARISON OF PEAK DISCHARGES

Location	WLB	CVL	Boyle	D & M
At Litchfield Park Detention Facility	959	769	525	1031
At Camelback and Dysart Road	1049	953	717	960

Again, these differences can be attributed to modeling techniques and WLB performed a HEC-2 analysis on Dysart Drain to better approximate the actual capacity of this facility and the corresponding breakout flows. Also, WLB had 1" = 400', 2-foot contour interval mapping to better estimate diversions and to delineate the watershed with greater precision.

5. "Conceptual Master Drainage Report for Litchfield Park Development Master Plan", by Coe & Van Loo, September 1989.

6. "Arizona Department of Transportation Interstate 10 Plans, Ehrenberg - Phoenix, Maricopa County I-10-2(34)," September 19, 1985.

INPUT PARAMETER COMPARISONS

Hydrologic Parameters	WLB	CVL	ADOT
Storm Frequency and Duration	100/24	100/6	100/3
Rainfall Amount	4.03 In.	3.15 In.	2.92 In.
Tabulation Interval	5-Minute	10-Minute	N/A
Loss Rate	Green-Ampt	SCS Curve #	SCS Curve #
Distribution Pattern	SCS Type II	SCS Type II	N/A
Areal Distribution	NOAA Atlas II	None	None
Hydrograph Development	COE Phx. Valley S-Graph	SCS Unit Hydrograph	SCS: Part II
Routing Method	Normal Depth	Kinematic Wave	N/A

COMPARISON OF DISCHARGES

Location	CVL	WLB	ADOT
At Reems Road & Northern Ave.	1001	2347	---
Divert E. at Reems Rd. & Northern Ave.	300	812	---
Remainder Flow to the S. at Reems Road and Northern Ave.	701	1536	---
At Camelback Road and Bullard Wash	2941	4243	---
At RID Canal and Bullard Wash	3585	4703	---
At Bullard Wash and I-10	*	5319 Upstream 4450 Downstream	5000 Upstream
At RID Canal and I-10	1347	826	

* Not Computed

The differences here are attributed to different storm durations and associated rainfall amounts, different subbasin divisions, a more intense scrutiny of diversions throughout the watershed, a HEC-2 analysis of Dysart Drain, and use of 1" = 400', 2-foot contour interval mapping over the entire watershed.

A number of sensitivity analyses were also performed to test the assumptions of hydrologic moisture condition and vegetation cover in the agricultural areas. Models were developed assuming fallow field (not planted) with the three different soil moisture conditions - saturated, normal and dry. These three moisture conditions were also used with a fully vegetated condition model. After reviewing these analyses, the FCDMC directed us to use the fully vegetated field in a saturated condition for agricultural areas in the watershed. It was understood that some areas would be fallow in a dry condition, vegetated in a normal or dry condition, etc., but the directed assumption gives an average condition without being too conservative or too under-conservative.

Also, an analysis was performed to determine if the numerous small agricultural reservoirs in the study area should be incorporated in the model. A typical agricultural reservoir was modeled and the results convinced the FCDMC that the storage would be filled during the early part of the storm before the peak arrived, therefore, these reservoirs would not be modeled. Another factor in the decision to not include the reservoirs is that there is no guarantee that they would not be filled in by the farmer or filled with sediment during the storm.

3.4 SPECIAL PROBLEMS/SOLUTIONS

The very nature of the watershed in the White Tanks/Agua Fria ADMS, with vastly differing hydrologic elements, tends to lead to modeling problems.

Initially, the watershed was separated into the following four distinct regions.

1. Watershed draining to White Tanks Flood Retarding Structure #3.
2. Watershed draining to White Tanks Flood Retarding Structure #4.
3. Watershed north of Dysart Drain and Northern Avenue.
4. Watershed south of Dysart Drain and Northern Avenue.

This was done to facilitate the FCDMC's review process and to allow the WLB Group to work on different regions while one was in for review.

This worked reasonably well as volumes of base data were generated in this study. The model was then joined together to create one complete hydrologic model of the entire watershed.

Two future conditions were assumed to be in place for the existing condition model. These assumptions were that the interim Estrella Freeway and Camelback Channel would be in place by the time the study was finished. The interim Estrella Freeway was assumed to collect flows along the west side of the roadway and pass these flows through at either at grade crossings or under the road in culvert crossings. For ease of modeling these were assumed to take place at major mile intersections although some flows may cross over or under at various locations between the intersections. The reason this assumption was made was based on the fact that these flows would eventually collect at the next major mile intersection to the southeast as overland flows naturally collect there now. This assumption was also used along the railroad at Cotton Lane.

The Colter Street Channel will be built by the Maricopa County Department of Transportation along an alignment of Coter Street which is approximately 1/4 mile north of Camelback Road. A Camelback Road alignment was assumed for this HEC-1 analysis which results in slightly larger flows, but does not compromise the integrity of the model. Flows will be collected in the channel from Litchfield Road and along inflow points to the east and are then conveyed to the Agua Fria River.

The Dysart Drain (also known as the Luke Air Force Base Drainage Channel) is located north and east of Luke Air Force Base and was modeled by a HEC-2 split flow analysis. Subsequent breakout flows were then incorporated into the HEC-1 model. Many iterations were required for this analysis to compute final diversion tables for the HEC-1 model.

To make the HEC-1 model a complete unit, it was necessary to route flows around the edge of the watershed in the Agua Fria River and Gila River. Since these are both very wide rivers, the assumption was made to route flows in a 1000 foot wide trapezoidal channel with representative Manning's n-values. The calculated flows are insignificant in comparison to the 100-year flow on the Agua Fria River and the Gila River.

As mentioned previously in this report, numerous diversions and ponding areas were modeled in the White Tank/Agua Fria ADMS. The procedures for modeling these areas are described in section 3.2.2. Of special note are the diversions located at the intersections of Olive Avenue and Beardsley Canal and Northern Avenue and Beardsley Canal. These diversions were modeled previously by the FCDMC in a report entitled "A Hydrologic Analysis of The White Tanks F.R.s #3 & #4". This data was incorporated in the HEC-1 model and into the subsequent HEC-2 analysis.

3.5 FINAL RESULTS/COMPUTER MODEL

The final results of the HEC-1 model are presented in numerical order in the Runoff Summary on the following pages. This is the same Runoff Summary generated by the HEC-1 model but it has been rearranged into numerical order for ease of locating discharges. Final output for the HEC-1 model is located in Appendix C, under separate cover, and another copy of the numerical Runoff Summary is included as well.

Four operations are shown in the Runoff Summary. These are respectively:

A) Runoff hydrographs for each subbasin.
B) Intermediate and final concentration points for combined hydrographs.
C) Diversion hydrographs.
D) Storage routing routines through reservoirs or ponding areas.

Routed flow discharges and returned diversion flows are not shown in this table. The HEC-1 output should be referred to if these discharges are required.

A note about the naming sequence of different operations in the HEC-1 model. Runoff hydrographs are designated as a number, combinations of numbers, or combinations of numbers and letters, ie, 41, 41-1, 41A1.

Final concentration points have the designation CP followed by the watershed number where that particular concentration point is located. Intermediate concentration points are designated as I1CP or 1I, 2I, etc; again, followed by the subbasin number. Concentration points combined in the Agua Fria or Gila River are designated as RCP followed by the subbasin number. It should also be mentioned here that routings in the river reaches are designated as RR standing for river route.

Diversions are designated by D, DI, 1D, 2D, etc. Storage routing through ponding areas or reservoirs is designated by SR with the one exception being the storage routine behind WT#4 which was inadvertently called RS47. Otherwise, these naming schemes stay consistent throughout the model.

Due to the nature and differing hydrologic regions of the watershed, it is difficult to put the model together in a systematic order. The model, therefore, is very complex and difficult to follow. A HEC-1 Key Map was created that breaks out the order in which the model was created. Distinct groups of subbasins make up a hydrologic area that drains to a common concentration point. These areas are numbered and have a corresponding tab in the HEC-1 output hardcopy so that it is easier to identify certain areas within the model that are of particular interest. The key map is located in the front of Appendix C where the HEC-1 hardcopy is located.

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1 HYDROGRAPH AT	1	1342.	12.50	175.	50.	48.	1.94		
2 HYDROGRAPH AT	2	1174.	12.75	206.	58.	56.	1.82		
3 HYDROGRAPH AT	3	828.	12.33	83.	21.	20.	.81		
4 HYDROGRAPH AT	3A	296.	12.33	29.	7.	7.	.29		
5 HYDROGRAPH AT	4	339.	12.25	26.	7.	6.	.30		
6 HYDROGRAPH AT	5	716.	12.25	65.	18.	18.	.72		
7 HYDROGRAPH AT	6	591.	12.08	41.	12.	11.	.45		
8 HYDROGRAPH AT	7	390.	12.08	28.	8.	8.	.31		
9 HYDROGRAPH AT	8	704.	12.33	73.	21.	20.	.81		
10 HYDROGRAPH AT	9	1096.	12.42	127.	36.	35.	1.40		
11 HYDROGRAPH AT	10	1173.	12.75	201.	52.	50.	2.02		
12 HYDROGRAPH AT	11	1313.	12.50	165.	44.	43.	1.56		
13 HYDROGRAPH AT	12	1149.	12.58	156.	40.	39.	1.38		
14 HYDROGRAPH AT	13	1170.	12.42	137.	34.	33.	1.30		
15 HYDROGRAPH AT	14	1163.	12.33	130.	37.	35.	1.47		
16 HYDROGRAPH AT	15	1039.	12.42	117.	32.	30.	1.26		
17 HYDROGRAPH AT	16	1255.	12.42	155.	43.	41.	1.13		
18 HYDROGRAPH AT	17	929.	12.25	110.	27.	26.	1.07		
19 HYDROGRAPH AT	18	923.	12.17	73.	21.	20.	.81		
20 HYDROGRAPH AT	19	622.	12.42	71.	20.	19.	.79		
21 HYDROGRAPH AT	20	861.	12.33	97.	27.	26.	1.07		
22 HYDROGRAPH AT	21	688.	12.42	74.	20.	19.	.79		
23 HYDROGRAPH AT	22	525.	12.25	50.	14.	13.	.57		
24 HYDROGRAPH AT	22A	764.	12.25	69.	18.	17.	.50		
25 HYDROGRAPH AT	23	289.	12.08	18.	5.	5.	.16		
26 HYDROGRAPH AT	24	207.	12.25	18.	5.	4.	.14		
27 HYDROGRAPH AT	25	500.	12.33	50.	12.	12.	.46		
28 HYDROGRAPH AT	26	943.	12.50	115.	30.	29.	1.16		
29 HYDROGRAPH AT	27	999.	12.42	110.	28.	27.	1.00		
30 HYDROGRAPH AT	28	747.	12.50	89.	22.	22.	.86		
31 HYDROGRAPH AT	29	228.	12.25	18.	5.	4.	.22		
32 HYDROGRAPH AT	30	244.	12.33	21.	5.	5.	.28		
33 HYDROGRAPH AT	31	525.	12.50	63.	16.	15.	.71		
34 HYDROGRAPH AT	32	956.	12.42	117.	33.	32.	1.29		
35 HYDROGRAPH AT	33	643.	12.25	59.	17.	16.	.65		
36 HYDROGRAPH AT	34	361.	12.25	33.	9.	9.	.36		
37 HYDROGRAPH AT	35	400.	12.25	35.	10.	10.	.39		
38 HYDROGRAPH AT	36	193.	12.25	16.	4.	4.	.24		
39 HYDROGRAPH AT	37	672.	12.42	85.	24.	23.	.95		
40 HYDROGRAPH AT	38	715.	12.25	64.	17.	17.	.76		
41 HYDROGRAPH AT	39	588.	12.50	74.	18.	18.	.77		
42 HYDROGRAPH AT	40	525.	12.25	48.	13.	13.	.52		
43 HYDROGRAPH AT	41A1	48.	12.00	3.	1.	1.	.02		
44 HYDROGRAPH AT	41A2	60.	12.00	3.	1.	1.	.03		
45 HYDROGRAPH AT	41A3	69.	12.00	4.	1.	1.	.03		
46 HYDROGRAPH AT	41A	91.	12.08	5.	1.	1.	.05		
47 HYDROGRAPH AT	41-1	208.	12.17	14.	3.	3.	.15		
48 HYDROGRAPH AT	41-2	143.	12.17	9.	2.	2.	.10		
49 HYDROGRAPH AT	41	567.	12.42	64.	16.	16.	.58		
50 HYDROGRAPH AT	42	1029.	12.50	131.	33.	32.	1.18		
51 HYDROGRAPH AT	43-1	76.	12.08	5.	1.	1.	.04		
52 HYDROGRAPH AT	43-2	19.	12.00	1.	0.	0.	.01		
53 HYDROGRAPH AT	43-3	107.	12.00	6.	1.	1.	.05		
54 HYDROGRAPH AT	43-4	64.	12.00	3.	1.	1.	.03		
55 HYDROGRAPH AT	43-5	43.	12.00	2.	1.	1.	.02		
56 HYDROGRAPH AT	43-6	45.	12.00	2.	1.	1.	.02		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
57	HYDROGRAPH AT	43-7	45.	12.00	2.	1.	1.	.02		
58	HYDROGRAPH AT	43-8	23.	12.00	1.	0.	0.	.01		
59	HYDROGRAPH AT	43	23.	12.00	1.	0.	0.	.01		
60	HYDROGRAPH AT	44	300.	12.25	25.	6.	6.	.22		
61	HYDROGRAPH AT	45-1	143.	12.08	9.	2.	2.	.08		
62	HYDROGRAPH AT	45	401.	12.42	48.	12.	12.	.38		
63	HYDROGRAPH AT	46-1	184.	12.25	18.	4.	4.	.15		
64	HYDROGRAPH AT	46	651.	12.58	94.	24.	23.	.85		
65	HYDROGRAPH AT	WT3	413.	12.50	54.	14.	13.	.44		
66	HYDROGRAPH AT	WT4	997.	12.25	88.	22.	21.	.77		
67	HYDROGRAPH AT	100	283.	12.58	45.	11.	11.	.26		
68	HYDROGRAPH AT	100A	212.	12.50	31.	8.	7.	.18		
69	HYDROGRAPH AT	101	233.	12.25	22.	5.	5.	.16		
70	HYDROGRAPH AT	102	135.	12.42	17.	4.	4.	.10		
71	HYDROGRAPH AT	102A	525.	12.58	73.	18.	18.	.51		
72	HYDROGRAPH AT	103	286.	12.92	55.	14.	13.	.37		
73	HYDROGRAPH AT	104	236.	12.17	20.	5.	5.	.15		
74	HYDROGRAPH AT	105	354.	12.25	34.	9.	8.	.21		
75	HYDROGRAPH AT	106	871.	12.50	111.	28.	27.	.77		
76	HYDROGRAPH AT	107	398.	13.08	88.	22.	21.	.60		
77	HYDROGRAPH AT	108	478.	13.25	117.	29.	28.	.79		
78	HYDROGRAPH AT	109	536.	13.25	140.	35.	34.	.85		
79	HYDROGRAPH AT	110	270.	12.83	51.	13.	12.	.31		
80	HYDROGRAPH AT	111	443.	12.67	71.	18.	17.	.50		
81	HYDROGRAPH AT	112	534.	13.33	138.	34.	33.	.97		
82	HYDROGRAPH AT	113	431.	13.08	117.	29.	28.	.50		
83	HYDROGRAPH AT	113A	409.	13.08	106.	27.	26.	.50		
84	HYDROGRAPH AT	114	326.	13.00	84.	21.	20.	.38		
85	HYDROGRAPH AT	115	379.	13.08	89.	22.	22.	.49		
86	HYDROGRAPH AT	116	575.	13.50	166.	41.	40.	1.02		
87	HYDROGRAPH AT	117	335.	12.83	63.	17.	16.	.41		
88	HYDROGRAPH AT	117A	195.	12.83	38.	9.	9.	.21		
89	HYDROGRAPH AT	118	126.	12.83	23.	6.	6.	.15		
90	HYDROGRAPH AT	119	600.	13.17	143.	36.	34.	.86		
91	HYDROGRAPH AT	119A	356.	13.17	91.	23.	22.	.47		
92	HYDROGRAPH AT	120	397.	13.25	106.	27.	26.	.54		
93	HYDROGRAPH AT	121A	324.	12.92	60.	15.	14.	.50		
93	HYDROGRAPH AT	121	325.	12.92	60.	15.	14.	.50		
94	HYDROGRAPH AT	122	552.	13.33	146.	37.	35.	.89		
95	HYDROGRAPH AT	123	338.	13.00	74.	18.	18.	.44		
96	HYDROGRAPH AT	124	355.	13.33	97.	24.	23.	.57		
97	HYDROGRAPH AT	125	1044.	12.50	134.	33.	32.	1.00		
98	HYDROGRAPH AT	126	562.	13.33	146.	36.	35.	.95		
99	HYDROGRAPH AT	127	469.	12.08	42.	12.	11.	.22		
100	HYDROGRAPH AT	128	312.	12.92	59.	15.	14.	.41		
101	HYDROGRAPH AT	129	378.	12.67	61.	16.	15.	.43		
102	HYDROGRAPH AT	130	647.	13.25	172.	43.	41.	1.00		
103	HYDROGRAPH AT	131	355.	13.08	79.	20.	19.	.49		
104	HYDROGRAPH AT	131A	354.	13.08	79.	20.	19.	.49		
105	HYDROGRAPH AT	132	271.	13.00	56.	14.	13.	.41		
106	HYDROGRAPH AT	133	328.	13.08	71.	18.	17.	.49		
107	HYDROGRAPH AT	134	334.	13.08	73.	18.	18.	.50		
108	HYDROGRAPH AT	135	315.	13.17	72.	18.	17.	.49		
109	HYDROGRAPH AT	136	315.	13.08	69.	17.	17.	.46		
110	HYDROGRAPH AT	137	307.	13.42	85.	21.	20.	.54		
111	HYDROGRAPH AT	138	587.	13.33	155.	39.	37.	1.00		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
112	HYDROGRAPH AT	139	338.	13.08	77.	19.	19.	.47		
113	HYDROGRAPH AT	140	194.	12.67	32.	8.	8.	.18		
114	HYDROGRAPH AT	141	460.	12.33	47.	12.	11.	.47		
115	HYDROGRAPH AT	141A	202.	12.17	14.	4.	3.	.14		
116	HYDROGRAPH AT	142	351.	13.00	74.	18.	18.	.51		
117	HYDROGRAPH AT	143	354.	13.00	75.	19.	18.	.50		
118	HYDROGRAPH AT	144	351.	13.00	74.	18.	18.	.51		
119	HYDROGRAPH AT	145	328.	13.08	72.	18.	17.	.48		
120	HYDROGRAPH AT	145A	327.	13.08	73.	18.	18.	.49		
121	HYDROGRAPH AT	146	548.	13.17	130.	33.	31.	.90		
122	HYDROGRAPH AT	147	342.	13.00	72.	18.	17.	.50		
123	HYDROGRAPH AT	148	328.	13.00	71.	18.	17.	.48		
124	HYDROGRAPH AT	149	312.	13.08	68.	17.	16.	.48		
125	HYDROGRAPH AT	150	193.	12.75	33.	8.	8.	.23		
126	HYDROGRAPH AT	151	208.	12.75	36.	9.	9.	.25		
127	HYDROGRAPH AT	152	284.	12.92	58.	15.	14.	.35		
128	HYDROGRAPH AT	153	112.	13.33	30.	7.	7.	.20		
129	HYDROGRAPH AT	154	171.	12.58	26.	7.	6.	.17		
130	HYDROGRAPH AT	155	250.	12.75	47.	12.	11.	.26		
131	HYDROGRAPH AT	156	252.	12.75	46.	12.	12.	.30		
132	HYDROGRAPH AT	156A	508.	12.17	46.	13.	13.	.31		
133	HYDROGRAPH AT	157	946.	12.58	150.	37.	36.	.89		
134	HYDROGRAPH AT	158	494.	13.08	108.	29.	28.	.97		
135	HYDROGRAPH AT	158A	114.	13.25	26.	6.	6.	.38		
136	HYDROGRAPH AT	158B	483.	13.00	104.	28.	27.	.67		
137	HYDROGRAPH AT	158C	105.	12.58	17.	4.	4.	.09		
138	HYDROGRAPH AT	158D	560.	12.25	55.	15.	15.	.37		
139	HYDROGRAPH AT	158E	767.	12.17	70.	20.	19.	.45		
140	HYDROGRAPH AT	159	531.	12.33	51.	13.	12.	.58		
141	HYDROGRAPH AT	160	432.	12.42	47.	12.	11.	.39		
142	HYDROGRAPH AT	161	294.	13.00	58.	14.	14.	.50		
143	HYDROGRAPH AT	162	268.	12.50	34.	8.	8.	.25		
144	HYDROGRAPH AT	163	551.	12.92	112.	28.	27.	.75		
145	HYDROGRAPH AT	164	363.	13.08	81.	20.	20.	.49		
146	HYDROGRAPH AT	164A	365.	13.08	82.	20.	20.	.49		
147	HYDROGRAPH AT	165	548.	13.25	133.	33.	32.	.90		
148	HYDROGRAPH AT	166	533.	13.33	142.	36.	34.	.98		
149	HYDROGRAPH AT	167	528.	13.33	141.	35.	34.	.97		
150	HYDROGRAPH AT	168	340.	13.17	79.	20.	19.	.51		
151	HYDROGRAPH AT	169	368.	13.17	89.	22.	21.	.51		
152	HYDROGRAPH AT	170	301.	12.67	53.	14.	13.	.29		
153	HYDROGRAPH AT	171	409.	13.42	115.	29.	28.	.70		
154	HYDROGRAPH AT	172	122.	12.58	18.	5.	4.	.12		
155	HYDROGRAPH AT	173	198.	13.00	39.	10.	9.	.31		
156	HYDROGRAPH AT	173A	191.	12.75	34.	8.	8.	.20		
157	HYDROGRAPH AT	173B	88.	12.83	14.	4.	3.	.20		
158	HYDROGRAPH AT	174	612.	12.25	55.	14.	13.	.45		
159	HYDROGRAPH AT	175	375.	12.25	34.	9.	8.	.28		
160	HYDROGRAPH AT	175A	362.	12.50	42.	10.	10.	.47		
161	HYDROGRAPH AT	176	701.	12.42	79.	20.	19.	.67		
162	HYDROGRAPH AT	176A	378.	13.25	92.	23.	22.	.62		
163	HYDROGRAPH AT	177	336.	13.08	73.	18.	18.	.49		
164	HYDROGRAPH AT	177A	334.	13.08	73.	18.	17.	.49		
165	HYDROGRAPH AT	178	298.	13.08	64.	16.	15.	.44		
166	HYDROGRAPH AT	179	333.	13.08	76.	19.	18.	.46		
167	HYDROGRAPH AT	180	574.	13.33	154.	39.	37.	.99		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
168 HYDROGRAPH AT	181	283.	12.92	54.	14.	13.	.37		
169 HYDROGRAPH AT	181A	249.	13.17	57.	14.	14.	.39		
170 HYDROGRAPH AT	182	212.	12.75	35.	9.	9.	.24		
171 HYDROGRAPH AT	183	258.	12.42	31.	8.	8.	.21		
172 HYDROGRAPH AT	184	531.	13.17	124.	31.	30.	.79		
173 HYDROGRAPH AT	185	465.	13.08	103.	26.	25.	.69		
174 HYDROGRAPH AT	186	187.	13.25	48.	12.	12.	.33		
175 HYDROGRAPH AT	187	81.	12.00	5.	1.	1.	.35		
176 HYDROGRAPH AT	188	240.	12.25	19.	5.	5.	.19		
177 HYDROGRAPH AT	189	521.	12.42	57.	14.	14.	.51		
178 HYDROGRAPH AT	190	657.	12.92	128.	32.	31.	.86		
179 HYDROGRAPH AT	191	610.	13.25	149.	37.	36.	.99		
180 HYDROGRAPH AT	192	346.	13.08	75.	19.	18.	.50		
181 HYDROGRAPH AT	192A	345.	13.08	75.	19.	18.	.50		
182 HYDROGRAPH AT	193	545.	13.25	132.	33.	32.	.91		
183 HYDROGRAPH AT	194	535.	13.42	144.	36.	35.	.99		
184 HYDROGRAPH AT	195	256.	13.50	72.	18.	17.	.49		
185 HYDROGRAPH AT	196	313.	13.00	68.	17.	17.	.47		
186 HYDROGRAPH AT	197	524.	13.50	150.	38.	36.	1.00		
187 HYDROGRAPH AT	198	494.	13.42	140.	35.	34.	.91		
188 HYDROGRAPH AT	199	74.	13.08	16.	4.	4.	.11		
189 HYDROGRAPH AT	200	231.	12.67	37.	9.	9.	.29		
190 HYDROGRAPH AT	201	420.	12.33	44.	11.	11.	.34		
191 HYDROGRAPH AT	202	258.	13.42	73.	18.	18.	.48		
192 HYDROGRAPH AT	203	162.	12.25	14.	4.	3.	.11		
193 HYDROGRAPH AT	204	381.	12.25	32.	8.	8.	.27		
194 HYDROGRAPH AT	205	125.	12.08	8.	2.	2.	.06		
195 HYDROGRAPH AT	206	188.	12.17	14.	3.	3.	.12		
196 HYDROGRAPH AT	207	619.	13.25	153.	38.	37.	1.00		
197 HYDROGRAPH AT	207A	469.	12.42	48.	12.	12.	.50		
198 HYDROGRAPH AT	208	632.	13.17	149.	37.	36.	1.00		
199 HYDROGRAPH AT	209	336.	13.08	73.	18.	17.	.50		
200 HYDROGRAPH AT	209A	335.	13.08	75.	19.	18.	.50		
201 HYDROGRAPH AT	210	296.	13.08	64.	16.	16.	.46		
202 HYDROGRAPH AT	211	309.	13.08	69.	17.	17.	.49		
203 HYDROGRAPH AT	212	279.	13.42	77.	19.	19.	.54		
204 HYDROGRAPH AT	213	373.	12.50	46.	12.	11.	.35		
205 HYDROGRAPH AT	214	208.	12.25	18.	5.	4.	.16		
206 HYDROGRAPH AT	215	419.	12.33	44.	11.	11.	.35		
207 HYDROGRAPH AT	215A	497.	12.42	55.	14.	13.	.45		
208 HYDROGRAPH AT	216	366.	12.92	69.	17.	17.	.51		
209 HYDROGRAPH AT	217	350.	13.00	76.	19.	18.	.49		
210 HYDROGRAPH AT	218	624.	13.25	153.	38.	37.	1.00		
211 HYDROGRAPH AT	219	343.	13.08	76.	19.	18.	.50		
212 HYDROGRAPH AT	220	335.	13.08	75.	19.	18.	.50		
213 HYDROGRAPH AT	221	303.	13.08	69.	17.	17.	.48		
214 HYDROGRAPH AT	221A	175.	13.25	43.	11.	10.	.31		
215 HYDROGRAPH AT	222	541.	13.58	172.	48.	46.	1.10		
216 HYDROGRAPH AT	223	1763.	12.42	327.	110.	106.	1.26		
217 HYDROGRAPH AT	224	1054.	12.33	109.	27.	26.	.80		
218 HYDROGRAPH AT	225	460.	12.42	59.	15.	15.	.43		
219 HYDROGRAPH AT	225A	441.	12.42	50.	12.	12.	.37		
220 HYDROGRAPH AT	226	1573.	12.33	192.	54.	52.	1.18		
221 HYDROGRAPH AT	227	331.	12.25	30.	8.	7.	.23		
222 HYDROGRAPH AT	228	361.	12.33	36.	9.	9.	.28		
223 HYDROGRAPH AT	228A	125.	12.17	8.	2.	2.	.08		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
224 HYDROGRAPH AT	229	724.	12.17	61.	16.	15.	.51		
225 HYDROGRAPH AT	230	56.	12.33	6.	1.	1.	.04		
226 HYDROGRAPH AT	230A	292.	12.17	21.	5.	5.	.18		
227 HYDROGRAPH AT	231	265.	12.42	28.	7.	7.	.35		
228 HYDROGRAPH AT	232	1006.	12.42	109.	27.	26.	.93		
229 HYDROGRAPH AT	233	496.	12.42	56.	14.	13.	.50		
230 HYDROGRAPH AT	234	347.	13.17	81.	20.	19.	.53		
231 HYDROGRAPH AT	235	303.	13.17	71.	18.	17.	.47		
232 HYDROGRAPH AT	236	582.	13.25	146.	37.	35.	1.00		
233 HYDROGRAPH AT	237	300.	13.25	73.	18.	18.	.50		
234 HYDROGRAPH AT	238	314.	13.17	72.	18.	17.	.50		
235 HYDROGRAPH AT	239	313.	13.08	69.	17.	17.	.48		
236 HYDROGRAPH AT	240	282.	13.00	57.	14.	14.	.40		
237 HYDROGRAPH AT	241	1436.	12.50	176.	44.	42.	1.51		
238 HYDROGRAPH AT	242	965.	12.67	143.	36.	34.	1.14		
239 HYDROGRAPH AT	242A	161.	12.08	10.	3.	2.	.09		
240 HYDROGRAPH AT	243	298.	12.33	34.	8.	8.	.24		
241 HYDROGRAPH AT	243A	253.	12.42	31.	8.	8.	.22		
242 HYDROGRAPH AT	243B	64.	12.67	10.	3.	3.	.07		
243 HYDROGRAPH AT	244	200.	12.58	28.	7.	7.	.19		
244 HYDROGRAPH AT	244A	486.	12.17	36.	9.	9.	.31		
245 HYDROGRAPH AT	245	181.	13.17	41.	10.	10.	.40		
246 HYDROGRAPH AT	246	670.	12.50	84.	21.	20.	.75		
247 HYDROGRAPH AT	247	330.	13.08	76.	19.	18.	.50		
248 HYDROGRAPH AT	248	592.	13.25	152.	38.	37.	1.00		
249 HYDROGRAPH AT	249	571.	13.25	143.	36.	35.	1.00		
250 HYDROGRAPH AT	250	316.	13.08	71.	18.	17.	.49		
251 HYDROGRAPH AT	250A	308.	13.17	74.	18.	18.	.51		
252 HYDROGRAPH AT	251	313.	13.17	72.	18.	17.	.50		
253 HYDROGRAPH AT	252	326.	13.17	76.	19.	18.	.50		
254 HYDROGRAPH AT	253	641.	13.17	148.	37.	36.	1.00		
255 HYDROGRAPH AT	253A	291.	12.33	29.	7.	7.	.25		
256 HYDROGRAPH AT	254	556.	12.58	81.	20.	20.	.59		
257 HYDROGRAPH AT	254A	443.	12.08	40.	11.	11.	.22		
258 HYDROGRAPH AT	255	1512.	12.25	152.	41.	40.	.94		
259 HYDROGRAPH AT	256	326.	13.00	72.	18.	17.	.43		
260 HYDROGRAPH AT	257	217.	13.42	62.	16.	15.	.34		
261 HYDROGRAPH AT	258	334.	12.42	37.	9.	9.	.38		
262 HYDROGRAPH AT	258A	150.	12.25	13.	3.	3.	.12		
263 HYDROGRAPH AT	259	242.	12.25	18.	5.	4.	.20		
264 HYDROGRAPH AT	260	536.	12.33	56.	15.	14.	.50		
265 HYDROGRAPH AT	261	264.	13.17	62.	16.	15.	.41		
266 HYDROGRAPH AT	262	616.	13.25	149.	37.	36.	1.03		
267 HYDROGRAPH AT	263	328.	13.08	72.	18.	17.	.50		
268 HYDROGRAPH AT	264	337.	13.00	70.	18.	17.	.50		
269 HYDROGRAPH AT	265	118.	13.33	31.	8.	8.	.22		
270 HYDROGRAPH AT	265A	627.	13.08	137.	34.	33.	.95		
271 HYDROGRAPH AT	266	181.	13.33	48.	12.	12.	.33		
272 HYDROGRAPH AT	267	335.	13.17	79.	20.	19.	.50		
273 HYDROGRAPH AT	268	589.	13.25	147.	37.	35.	.95		
274 HYDROGRAPH AT	269	491.	13.00	101.	25.	24.	.66		
275 HYDROGRAPH AT	270	446.	12.25	49.	14.	13.	.30		
276 HYDROGRAPH AT	271	377.	13.25	96.	24.	23.	.57		
277 HYDROGRAPH AT	271A	95.	13.33	25.	6.	6.	.16		
278 HYDROGRAPH AT	271B	109.	12.83	20.	5.	5.	.15		
279 HYDROGRAPH AT	271C	263.	12.17	22.	5.	5.	.18		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
280 HYDROGRAPH AT	272	231.	12.25	18.	4.	4.	.20		
281 HYDROGRAPH AT	273	671.	12.42	75.	19.	18.	.65		
282 HYDROGRAPH AT	274	329.	13.50	94.	23.	23.	.66		
283 HYDROGRAPH AT	275	136.	12.42	17.	4.	4.	.11		
284 HYDROGRAPH AT	276	187.	13.00	38.	10.	9.	.26		
284 HYDROGRAPH AT	281	487.	13.25	122.	31.	29.	.85		
285 HYDROGRAPH AT	277	570.	13.08	127.	32.	31.	.91		
286 HYDROGRAPH AT	278	537.	13.33	138.	35.	34.	1.00		
287 HYDROGRAPH AT	279	79.	12.25	7.	2.	2.	.05		
288 HYDROGRAPH AT	279A	89.	12.33	10.	3.	2.	.07		
289 HYDROGRAPH AT	279B	46.	12.25	4.	1.	1.	.03		
290 HYDROGRAPH AT	279C	78.	12.25	8.	2.	2.	.05		
291 HYDROGRAPH AT	279D	33.	12.17	3.	1.	1.	.02		
292 HYDROGRAPH AT	280	340.	13.33	89.	22.	21.	.61		
293 HYDROGRAPH AT	280A	53.	12.75	9.	2.	2.	.06		
295 HYDROGRAPH AT	282	161.	12.33	17.	4.	4.	.12		
296 HYDROGRAPH AT	283	102.	13.08	23.	6.	6.	.16		
297 HYDROGRAPH AT	284	326.	13.17	76.	19.	18.	.52		
298 HYDROGRAPH AT	285	56.	12.33	6.	1.	1.	.04		
299 HYDROGRAPH AT	285A	82.	12.33	9.	2.	2.	.06		
300 HYDROGRAPH AT	285B	83.	12.33	9.	2.	2.	.06		
301 HYDROGRAPH AT	286	473.	13.08	109.	27.	26.	.70		
302 HYDROGRAPH AT	287	223.	12.67	36.	9.	9.	.23		
303 HYDROGRAPH AT	287A	331.	12.83	68.	17.	16.	.34		
304 HYDROGRAPH AT	287B	127.	12.50	18.	4.	4.	.10		
305 HYDROGRAPH AT	287C	270.	12.67	57.	14.	14.	.23		
306 HYDROGRAPH AT	287D	239.	12.67	39.	10.	9.	.23		
307 HYDROGRAPH AT	287E	194.	12.58	28.	7.	7.	.20		
308 HYDROGRAPH AT	288	160.	13.25	44.	11.	10.	.22		
309 HYDROGRAPH AT	288A	607.	13.33	188.	47.	46.	.83		
310 HYDROGRAPH AT	288B	179.	13.42	54.	14.	13.	.26		
311 HYDROGRAPH AT	289	519.	13.83	189.	48.	46.	1.00		
312 HYDROGRAPH AT	289A	326.	13.17	77.	19.	18.	.50		
313 HYDROGRAPH AT	290	413.	12.67	64.	16.	15.	.55		
314 HYDROGRAPH AT	291	454.	13.25	112.	28.	27.	.99		
315 HYDROGRAPH AT	292	1004.	12.50	131.	35.	33.	.96		
316 HYDROGRAPH AT	293	679.	12.67	113.	29.	28.	.77		
317 HYDROGRAPH AT	293A	76.	12.50	10.	3.	2.	.07		
318 HYDROGRAPH AT	294	274.	12.17	24.	6.	6.	.18		
319 HYDROGRAPH AT	294A	368.	12.25	34.	8.	8.	.26		
320 HYDROGRAPH AT	295	419.	12.25	41.	10.	10.	.31		
321 HYDROGRAPH AT	295A	114.	12.08	8.	2.	2.	.06		
322 HYDROGRAPH AT	296	565.	13.00	117.	29.	28.	.81		
323 HYDROGRAPH AT	297	240.	13.17	55.	14.	13.	.38		
324 HYDROGRAPH AT	297A	206.	12.92	39.	10.	9.	.27		
325 HYDROGRAPH AT	298	591.	13.00	122.	30.	29.	.84		
326 HYDROGRAPH AT	299	289.	13.00	61.	15.	15.	.40		
327 HYDROGRAPH AT	300	353.	12.83	66.	16.	16.	.39		
328 HYDROGRAPH AT	301	535.	12.17	46.	12.	12.	.30		
329 HYDROGRAPH AT	302	242.	12.17	21.	5.	5.	.15		
330 HYDROGRAPH AT	303	910.	12.67	138.	35.	33.	1.21		
331 HYDROGRAPH AT	303A	531.	12.25	49.	12.	12.	.42		
332 HYDROGRAPH AT	304	1221.	12.33	128.	32.	31.	1.10		
333 HYDROGRAPH AT	305	666.	13.00	141.	35.	34.	.92		
334 HYDROGRAPH AT	306	367.	12.92	71.	18.	17.	.49		
335 HYDROGRAPH AT	307	219.	12.58	30.	8.	7.	.21		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
336 HYDROGRAPH AT	308	270.	12.75	46.	12.	11.	.32		
337 HYDROGRAPH AT	309	666.	13.17	157.	39.	38.	1.07		
338 HYDROGRAPH AT	310	183.	12.58	28.	7.	7.	.19		
339 HYDROGRAPH AT	311	420.	13.33	107.	27.	26.	.73		
340 HYDROGRAPH AT	311A	241.	12.92	46.	12.	11.	.31		
341 HYDROGRAPH AT	312	423.	13.17	97.	24.	23.	.66		
342 HYDROGRAPH AT	313	258.	13.17	62.	16.	15.	.43		
343 HYDROGRAPH AT	314	293.	13.00	61.	15.	15.	.42		
344 HYDROGRAPH AT	315	278.	13.25	68.	17.	16.	.47		
345 HYDROGRAPH AT	316	512.	13.17	122.	31.	29.	.82		
346 HYDROGRAPH AT	317	341.	13.25	82.	21.	20.	.57		
347 HYDROGRAPH AT	318	581.	12.67	97.	27.	26.	.62		
348 HYDROGRAPH AT	319	577.	12.50	84.	23.	23.	.54		
349 HYDROGRAPH AT	320	677.	12.42	88.	25.	24.	.64		
350 HYDROGRAPH AT	321	942.	12.17	74.	18.	18.	.66		
351 HYDROGRAPH AT	322	261.	12.92	51.	13.	12.	.34		
352 HYDROGRAPH AT	323	181.	12.33	18.	5.	4.	.15		
353 HYDROGRAPH AT	324	279.	12.83	53.	13.	13.	.36		
354 HYDROGRAPH AT	325	294.	13.25	74.	18.	18.	.49		
355 HYDROGRAPH AT	325A	404.	13.08	88.	22.	21.	.61		
356 HYDROGRAPH AT	326	373.	13.00	79.	20.	19.	.54		
357 HYDROGRAPH AT	327	373.	13.00	77.	19.	18.	.53		
358 HYDROGRAPH AT	328	521.	12.92	103.	26.	25.	.76		
359 HYDROGRAPH AT	329	496.	12.83	89.	23.	22.	.63		
360 HYDROGRAPH AT	330	467.	12.83	87.	23.	22.	.59		
361 HYDROGRAPH AT	331	489.	13.08	109.	28.	27.	.75		
362 HYDROGRAPH AT	332	353.	13.17	84.	21.	21.	.56		
363 HYDROGRAPH AT	333	388.	13.08	84.	21.	20.	.58		
364 HYDROGRAPH AT	334	452.	13.00	95.	24.	23.	.64		
365 HYDROGRAPH AT	335	277.	12.92	54.	13.	13.	.35		
366 HYDROGRAPH AT	336	552.	13.92	206.	57.	55.	1.28		
367 HYDROGRAPH AT	336A	149.	14.00	54.	14.	13.	.37		
368 HYDROGRAPH AT	336B	168.	12.00	9.	2.	2.	.08		
369 HYDROGRAPH AT	337	744.	12.25	84.	25.	24.	.49		
370 HYDROGRAPH AT	338	260.	12.58	38.	9.	9.	.31		
371 HYDROGRAPH AT	338A	736.	12.42	87.	24.	23.	.77		
372 HYDROGRAPH AT	339	625.	13.17	150.	38.	36.	1.00		
373 HYDROGRAPH AT	340	348.	13.00	72.	18.	17.	.48		
374 HYDROGRAPH AT	341	506.	13.08	113.	28.	27.	.79		
375 HYDROGRAPH AT	342	327.	12.83	60.	15.	14.	.39		
376 HYDROGRAPH AT	342A	240.	13.17	56.	14.	13.	.38		
377 HYDROGRAPH AT	343	323.	13.17	74.	19.	18.	.51		
378 HYDROGRAPH AT	344	343.	13.33	88.	22.	21.	.61		
379 HYDROGRAPH AT	345	296.	13.00	63.	16.	15.	.43		
380 HYDROGRAPH AT	346	403.	12.75	68.	17.	16.	.51		
381 HYDROGRAPH AT	346A	110.	13.00	23.	6.	6.	.16		
382 HYDROGRAPH AT	346B	159.	13.25	39.	10.	9.	.27		
383 HYDROGRAPH AT	346C	135.	12.50	17.	4.	4.	.12		
384 HYDROGRAPH AT	347	671.	13.08	148.	38.	36.	1.02		
385 HYDROGRAPH AT	348	283.	12.75	50.	12.	12.	.34		
386 HYDROGRAPH AT	348A	229.	12.58	32.	8.	8.	.22		
387 HYDROGRAPH AT	348B	355.	13.33	92.	23.	22.	.66		
388 HYDROGRAPH AT	349	654.	13.00	137.	34.	33.	.93		
389 HYDROGRAPH AT	350	106.	13.33	27.	7.	7.	.19		
390 HYDROGRAPH AT	351	607.	12.83	113.	28.	27.	.76		
391 HYDROGRAPH AT	352	150.	12.75	26.	7.	6.	.18		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
392	HYDROGRAPH AT	352A	89.	13.08	20.	5.	5.	.13		
393	HYDROGRAPH AT	353	183.	12.75	32.	8.	8.	.22		
394	HYDROGRAPH AT	354	315.	12.58	45.	11.	11.	.31		
395	HYDROGRAPH AT	355	85.	12.92	18.	4.	4.	.10		
396	HYDROGRAPH AT	355A	46.	12.50	6.	1.	1.	.04		
397	HYDROGRAPH AT	356	346.	12.58	51.	13.	12.	.35		
398	HYDROGRAPH AT	357	132.	12.67	20.	5.	5.	.14		
399	HYDROGRAPH AT	358	119.	13.17	29.	7.	7.	.19		
400	HYDROGRAPH AT	359	104.	12.58	14.	4.	3.	.10		
401	HYDROGRAPH AT	360	168.	13.00	36.	9.	9.	.25		
402	HYDROGRAPH AT	361	138.	13.17	33.	8.	8.	.21		
403	HYDROGRAPH AT	362	339.	12.83	63.	16.	15.	.42		
404	HYDROGRAPH AT	363	360.	13.25	91.	23.	22.	.63		
405	HYDROGRAPH AT	364	559.	13.00	115.	29.	28.	.78		
406	HYDROGRAPH AT	364A	99.	12.25	9.	2.	2.	.07		
407	HYDROGRAPH AT	365	272.	12.83	47.	12.	11.	.38		
408	HYDROGRAPH AT	366	289.	13.00	61.	15.	15.	.44		
409	HYDROGRAPH AT	367	240.	12.08	16.	4.	4.	.15		
410	HYDROGRAPH AT	368	367.	13.58	111.	28.	27.	.83		
411	HYDROGRAPH AT	369	133.	12.25	11.	3.	3.	.15		
412	HYDROGRAPH AT	370	61.	12.33	5.	1.	1.	.07		
413	HYDROGRAPH AT	371	487.	13.67	154.	39.	37.	.93		
414	HYDROGRAPH AT	372	796.	13.75	262.	66.	63.	1.62		
415	HYDROGRAPH AT	373	386.	12.75	68.	17.	16.	.43		
416	HYDROGRAPH AT	374	459.	13.50	137.	34.	33.	.76		
417	HYDROGRAPH AT	375	262.	13.42	74.	19.	18.	.41		
418	HYDROGRAPH AT	376	344.	13.00	75.	19.	18.	.42		
419	HYDROGRAPH AT	377	120.	12.42	15.	4.	4.	.09		
420	HYDROGRAPH AT	377A	188.	13.00	39.	10.	9.	.27		
421	HYDROGRAPH AT	377B	58.	13.58	18.	5.	4.	.12		
422	HYDROGRAPH AT	378	349.	13.42	95.	24.	23.	.78		
422	HYDROGRAPH AT	379	220.	13.58	67.	17.	16.	.46		
423	HYDROGRAPH AT	380	188.	12.83	37.	9.	9.	.20		
424	HYDROGRAPH AT	381	184.	12.92	39.	10.	9.	.21		
425	HYDROGRAPH AT	381A	115.	12.58	17.	4.	4.	.10		
426	HYDROGRAPH AT	381B	67.	12.75	12.	3.	3.	.07		
427	HYDROGRAPH AT	382	539.	13.17	136.	34.	33.	.71		
428	HYDROGRAPH AT	383	154.	12.92	31.	8.	8.	.18		
429	HYDROGRAPH AT	383A	122.	13.00	27.	7.	7.	.16		
430	HYDROGRAPH AT	384	262.	12.83	55.	14.	13.	.29		
431	HYDROGRAPH AT	385	298.	13.17	77.	19.	19.	.39		
432	HYDROGRAPH AT	386	222.	13.25	57.	14.	14.	.35		
433	HYDROGRAPH AT	387	205.	13.00	44.	11.	11.	.25		
434	2 COMBINED AT	CP2	2284.	12.75	370.	100.	96.	3.76		
435	2 COMBINED AT	I1CP3	997.	12.42	110.	28.	27.	1.10		
436	2 COMBINED AT	CP3	2245.	12.92	468.	119.	115.	4.86		
437	2 COMBINED AT	CP5	1053.	12.25	90.	25.	24.	1.02		
438	2 COMBINED AT	I1CP7	1289.	12.17	130.	35.	34.	1.47		
439	2 COMBINED AT	CP7	1668.	12.17	158.	43.	41.	1.78		
440	2 COMBINED AT	I1CP9	2527.	12.33	277.	74.	71.	3.18		
441	2 COMBINED AT	CP9	3227.	12.33	350.	94.	91.	3.99		
442	2 COMBINED AT	I1CP10	3816.	12.75	525.	133.	129.	6.01		
443	2 COMBINED AT	CP10	5141.	12.75	911.	230.	222.	10.87		
444	2 COMBINED AT	CP12	4125.	12.83	861.	217.	209.	12.25		
445	2 COMBINED AT	CP13	1743.	12.58	301.	78.	76.	2.86		
446	2 COMBINED AT	I1CP15	1920.	12.50	246.	68.	66.	2.73		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
447 2 COMBINED AT	CP15	1920.	12.50	246.	68.	66.	3.86		
448 2 COMBINED AT	I1CP17	1738.	12.75	321.	80.	77.	4.93		
449 2 COMBINED AT	CP17	3428.	12.75	621.	158.	153.	7.79		
450 2 COMBINED AT	I1CWT3	3605.	12.83	668.	167.	161.	8.23		
451 2 COMBINED AT	CPWT3	6649.	12.92	1450.	362.	349.	20.48		
452 2 COMBINED AT	CP19	1353.	12.33	144.	40.	39.	1.60		
453 2 COMBINED AT	I1CP21	1933.	12.42	217.	60.	58.	2.39		
454 2 COMBINED AT	CP21	1933.	12.42	217.	60.	58.	3.46		
455 2 COMBINED AT	I1C22A	1111.	12.33	120.	31.	30.	1.07		
456 2 COMBINED AT	CP22A	1108.	12.33	119.	31.	30.	4.69		
457 2 COMBINED AT	CP23	288.	12.08	55.	17.	16.	3.62		
458 2 COMBINED AT	I1CP25	1414.	12.42	169.	44.	42.	5.15		
459 2 COMBINED AT	CP25	1414.	12.42	169.	44.	42.	5.29		
460 2 COMBINED AT	I1CP27	1648.	12.50	225.	57.	55.	2.16		
461 2 COMBINED AT	CP27	3011.	12.50	350.	88.	85.	7.45		
462 2 COMBINED AT	I1CP30	879.	12.50	110.	28.	27.	1.14		
463 2 COMBINED AT	CP30	879.	12.50	110.	28.	27.	1.36		
464 2 COMBINED AT	CP31	1258.	12.58	173.	43.	42.	2.07		
465 2 COMBINED AT	CP33	1003.	12.25	91.	26.	25.	1.01		
466 2 COMBINED AT	I1CP35	1297.	12.25	127.	36.	35.	1.40		
467 2 COMBINED AT	CP35	2155.	12.33	243.	69.	67.	2.69		
468 2 COMBINED AT	I1CP36	2217.	12.42	258.	73.	70.	2.93		
469 2 COMBINED AT	CP36	2886.	12.42	343.	97.	93.	3.88		
470 2 COMBINED AT	CP38	3253.	12.42	407.	113.	109.	4.64		
471 2 COMBINED AT	I1CP39	3708.	12.58	427.	112.	108.	5.41		
472 2 COMBINED AT	CP39	6110.	12.67	765.	197.	190.	12.86		
473 2 COMBINED AT	CP41A2	60.	12.00	3.	1.	1.	.05		
474 2 COMBINED AT	CP41A3	86.	12.08	5.	1.	1.	.08		
475 2 COMBINED AT	CP41A	91.	12.08	5.	1.	1.	.13		
476 2 COMBINED AT	CP41-1	208.	12.17	14.	3.	3.	.28		
477 2 COMBINED AT	CP41-2	143.	12.17	12.	3.	3.	.38		
478 2 COMBINED AT	CP41	567.	12.42	64.	16.	16.	.68		
479 2 COMBINED AT	I1CP42	1454.	12.50	179.	46.	44.	1.70		
480 2 COMBINED AT	CP42	7140.	12.67	934.	241.	232.	14.56		
481 2 COMBINED AT	CP43-1	465.	12.58	42.	11.	10.	1.00		
482 2 COMBINED AT	CP43-2	19.	12.00	1.	0.	0.	1.01		
483 2 COMBINED AT	CP43-3	107.	12.00	6.	1.	1.	1.06		
484 2 COMBINED AT	CP43-4	64.	12.00	3.	1.	1.	1.09		
485 2 COMBINED AT	CP43-5	43.	12.00	2.	1.	1.	1.11		
486 2 COMBINED AT	CP43-6	45.	12.00	2.	1.	1.	1.13		
487 2 COMBINED AT	CP43-7	45.	12.00	2.	1.	1.	1.15		
488 2 COMBINED AT	CP43-8	23.	12.00	1.	0.	0.	1.16		
489 2 COMBINED AT	I1CP43	22.	12.00	1.	0.	0.	1.17		
490 2 COMBINED AT	CP43	6786.	12.83	799.	206.	198.	13.90		
491 2 COMBINED AT	CP45-1	1440.	12.92	90.	22.	22.	13.98		
492 2 COMBINED AT	CP45	1030.	13.08	101.	25.	24.	14.36		
493 2 COMBINED AT	CP46-1	316.	13.25	33.	8.	8.	14.51		
494 2 COMBINED AT	I1CP46	637.	12.58	92.	23.	22.	15.36		
495 2 COMBINED AT	CP46	1737.	12.67	259.	65.	63.	17.43		
496 2 COMBINED AT	I1CWT4	27.	12.67	5.	1.	1.	.05		
497 2 COMBINED AT	I2CWT4	29.	12.67	8.	2.	2.	.07		
498 2 COMBINED AT	I3CWT4	41.	12.58	10.	3.	2.	.09		
499 2 COMBINED AT	I4CWT4	46.	12.58	11.	3.	3.	.10		
500 2 COMBINED AT	I5CWT4	323.	12.25	36.	9.	9.	.32		
501 2 COMBINED AT	I6CWT4	4057.	13.00	747.	195.	187.	14.12		
502 2 COMBINED AT	I7CWT4	552.	13.25	120.	30.	29.	14.36		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
503 2 COMBINED AT	I8CWT4	618.	13.25	153.	38.	37.	14.51		
504 2 COMBINED AT	I9CWT4	1048.	12.25	237.	60.	57.	15.28		
505 2 COMBINED AT	I10WT4	2009.	13.00	491.	123.	119.	18.20		
506 2 COMBINED AT	CPWT4	6026.	13.00	1228.	316.	304.	18.57		
507 2 COMBINED AT	CP100	283.	12.58	70.	19.	18.	.44		
508 2 COMBINED AT	CP101	635.	12.50	95.	24.	23.	.67		
509 2 COMBINED AT	CP102	753.	12.50	112.	28.	27.	.77		
510 2 COMBINED AT	CP107	1044.	12.75	199.	50.	48.	1.37		
511 2 COMBINED AT	1I108	999.	12.58	228.	57.	55.	1.56		
512 2 COMBINED AT	CP108	1158.	12.75	283.	71.	68.	1.93		
513 2 COMBINED AT	CP109	535.	13.25	184.	53.	51.	1.29		
514 2 COMBINED AT	1I112	534.	13.33	157.	40.	38.	1.12		
515 2 COMBINED AT	2I112	765.	13.33	202.	51.	49.	1.62		
516 2 COMBINED AT	CP112	790.	13.33	236.	59.	57.	1.83		
517 2 COMBINED AT	CP113A	560.	13.08	183.	46.	45.	2.33		
518 2 COMBINED AT	CP113	556.	13.08	284.	73.	71.	2.83		
519 2 COMBINED AT	CP114	376.	13.08	140.	36.	35.	3.21		
520 2 COMBINED AT	1I115	1313.	13.17	288.	72.	69.	1.86		
521 2 COMBINED AT	2I115	2429.	13.17	570.	143.	137.	3.79		
522 2 COMBINED AT	CP115	2705.	13.17	688.	173.	167.	7.00		
523 2 COMBINED AT	CP116	892.	13.75	331.	93.	90.	2.31		
524 2 COMBINED AT	1I117	359.	13.33	114.	30.	29.	.72		
525 2 COMBINED AT	2I117	1104.	13.83	439.	122.	118.	3.03		
526 2 COMBINED AT	CP117	1172.	13.75	476.	131.	127.	3.24		
527 2 COMBINED AT	CP119	706.	13.17	169.	42.	41.	1.36		
528 2 COMBINED AT	CP119A	533.	13.25	161.	40.	39.	1.83		
529 2 COMBINED AT	1I120	895.	13.33	266.	67.	64.	2.37		
530 2 COMBINED AT	CP120	1056.	13.33	342.	86.	83.	4.20		
531 2 COMBINED AT	1I121A	465.	12.92	137.	35.	33.	2.33		
532 2 COMBINED AT	CP121A	465.	12.92	146.	37.	36.	2.83		
533 2 COMBINED AT	CP121	708.	13.00	205.	52.	50.	3.33		
534 3 COMBINED AT	1I122	571.	14.33	357.	99.	96.	4.10		
535 2 COMBINED AT	CP122	2976.	13.58	1041.	273.	263.	7.89		
536 2 COMBINED AT	CP124	665.	13.33	171.	43.	41.	1.01		
537 2 COMBINED AT	1I125	1148.	14.08	598.	164.	158.	4.24		
538 2 COMBINED AT	CP125	1318.	14.08	654.	178.	171.	5.25		
539 2 COMBINED AT	1I126	563.	13.33	169.	42.	41.	1.10		
540 2 COMBINED AT	CP126	856.	13.33	381.	101.	98.	6.35		
541 2 COMBINED AT	CP128	537.	13.25	158.	39.	38.	1.77		
542 3 COMBINED AT	1I130	715.	14.42	323.	82.	79.	3.24		
543 3 COMBINED AT	CP130	1703.	13.42	724.	184.	178.	5.54		
544 2 COMBINED AT	CP131A	1102.	13.58	553.	143.	138.	6.03		
545 2 COMBINED AT	1I131	1274.	13.58	621.	163.	157.	6.52		
546 2 COMBINED AT	CP131	1909.	13.50	822.	215.	207.	8.52		
547 2 COMBINED AT	CP132	460.	13.58	208.	56.	54.	8.93		
548 2 COMBINED AT	1I133	579.	13.42	270.	74.	71.	9.42		
549 2 COMBINED AT	CP133	725.	14.33	354.	101.	97.	14.48		
550 2 COMBINED AT	1I134	2655.	14.00	980.	262.	252.	8.39		
551 2 COMBINED AT	2I134	2655.	14.00	980.	262.	252.	8.83		
552 2 COMBINED AT	CP134	2847.	14.00	1106.	298.	287.	15.42		
553 2 COMBINED AT	1I135	885.	14.17	391.	112.	107.	15.63		
554 2 COMBINED AT	CP135	970.	14.08	449.	129.	124.	16.48		
555 2 COMBINED AT	1I136	315.	13.08	139.	39.	37.	1.47		
556 2 COMBINED AT	CP136	931.	14.92	523.	154.	148.	16.94		
557 2 COMBINED AT	1I137	907.	13.50	590.	174.	167.	17.12		
558 2 COMBINED AT	CP137	1306.	14.50	795.	230.	221.	21.72		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
559 2 COMBINED AT	1I138	586.	13.33	312.	93.	90.	6.25		
560 2 COMBINED AT	2I138	613.	13.42	385.	113.	109.	7.35		
561 2 COMBINED AT	CP138	934.	15.58	596.	184.	177.	23.82		
562 2 COMBINED AT	CP138	1329.	15.50	812.	261.	252.	24.02		
563 2 COMBINED AT	1I139	521.	13.33	119.	31.	30.	.69		
564 2 COMBINED AT	2I139	832.	13.83	399.	111.	107.	7.04		
565 2 COMBINED AT	CP139	843.	13.83	429.	119.	115.	7.22		
566 2 COMBINED AT	CP141A	540.	12.33	61.	15.	15.	.62		
567 2 COMBINED AT	CP142	692.	12.50	134.	34.	32.	1.13		
568 2 COMBINED AT	CP143	992.	13.00	209.	52.	50.	1.63		
569 2 COMBINED AT	1I144	858.	13.83	316.	79.	76.	6.05		
570 2 COMBINED AT	CP144	1398.	13.67	524.	131.	126.	7.68		
571 2 COMBINED AT	CP145A	1467.	13.25	563.	141.	136.	8.17		
572 2 COMBINED AT	1I145	1590.	14.00	708.	190.	183.	9.00		
573 2 COMBINED AT	CP145	2946.	14.00	1263.	330.	318.	11.63		
574 2 COMBINED AT	1I146	629.	13.25	259.	72.	69.	15.02		
575 2 COMBINED AT	CP146	1441.	14.08	813.	229.	221.	18.49		
576 2 COMBINED AT	1I147	337.	13.00	118.	36.	35.	14.62		
577 2 COMBINED AT	CP147	2010.	14.42	818.	229.	220.	15.92		
578 2 COMBINED AT	CP148	323.	13.00	134.	35.	34.	17.46		
578 2 COMBINED AT	1I148	323.	13.00	85.	22.	21.	16.40		
579 2 COMBINED AT	CP149	533.	13.58	316.	93.	89.	22.20		
580 2 COMBINED AT	CP150	196.	12.75	51.	13.	13.	17.69		
581 2 COMBINED AT	1I151	312.	12.92	85.	22.	21.	17.94		
582 2 COMBINED AT	CP151	638.	13.75	388.	114.	110.	23.66		
583 2 COMBINED AT	CP152	320.	13.25	170.	48.	46.	24.01		
584 2 COMBINED AT	CP153	423.	16.17	246.	78.	75.	21.92		
585 2 COMBINED AT	CP154	845.	16.33	529.	175.	168.	24.19		
586 2 COMBINED AT	CP155	243.	12.75	45.	11.	11.	24.27		
587 2 COMBINED AT	CP156	829.	14.25	462.	130.	125.	7.52		
588 2 COMBINED AT	1I157	919.	12.58	526.	204.	196.	25.08		
589 2 COMBINED AT	2I157	1258.	17.17	798.	289.	279.	25.08		
590 2 COMBINED AT	CP157	1771.	17.00	1228.	411.	396.	26.25		
591 2 COMBINED AT	1I158	497.	13.08	148.	42.	40.	1.28		
592 2 COMBINED AT	CP158	557.	14.17	219.	62.	60.	2.42		
593 2 COMBINED AT	RC158A	578.	13.25	125.	35.	34.	1.96		
593 2 COMBINED AT	CP158A	577.	13.17	145.	39.	37.	1.14		
593 2 COMBINED AT	CP158B	483.	13.00	119.	33.	31.	.76		
593 2 COMBINED AT	CP160	955.	12.42	98.	25.	24.	.97		
593 2 COMBINED AT	RC158D	1149.	12.33	125.	35.	34.	.82		
594 2 COMBINED AT	CP161	1032.	12.58	156.	39.	38.	1.47		
595 2 COMBINED AT	1I163	726.	13.00	146.	36.	35.	1.00		
596 2 COMBINED AT	CP163	1575.	13.00	301.	75.	72.	2.47		
597 2 COMBINED AT	1I164A	364.	13.08	104.	28.	27.	8.17		
598 2 COMBINED AT	CP164A	1865.	13.17	403.	103.	99.	10.64		
599 2 COMBINED AT	1I164	2141.	13.25	480.	123.	119.	11.13		
600 2 COMBINED AT	CP164	2141.	13.25	790.	202.	194.	15.08		
601 2 COMBINED AT	1I165	858.	15.17	434.	124.	119.	12.53		
602 2 COMBINED AT	2I165	2031.	13.67	1106.	298.	287.	15.98		
603 2 COMBINED AT	CP165	3187.	15.08	1882.	520.	501.	22.84		
604 2 COMBINED AT	1I166	1837.	15.58	799.	253.	243.	16.90		
605 2 COMBINED AT	CP166	1975.	15.58	902.	281.	271.	18.44		
606 2 COMBINED AT	1I167	589.	13.42	263.	78.	76.	24.63		
607 2 COMBINED AT	CP167	1870.	15.92	1044.	336.	324.	25.61		
608 2 COMBINED AT	1I168	330.	13.17	144.	50.	49.	24.17		
609 2 COMBINED AT	2I168	330.	13.17	188.	64.	62.	26.12		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
610 2 COMBINED AT	CP168	515.	13.50	351.	112.	108.	26.47		
611 2 COMBINED AT	1I169	357.	13.17	86.	22.	21.	24.52		
612 2 COMBINED AT	2I169	359.	13.83	131.	33.	32.	25.13		
613 2 COMBINED AT	CP169	491.	13.83	201.	58.	56.	27.24		
614 2 COMBINED AT	CP172	1768.	17.08	1227.	413.	398.	26.37		
615 2 COMBINED AT	CP173	1753.	17.42	1224.	419.	404.	26.68		
616 2 COMBINED AT	RCP173	1826.	17.33	1460.	512.	493.	30.12		
617 2 COMBINED AT	CP173B	496.	14.58	229.	65.	63.	2.62		
618 2 COMBINED AT	RC173B	880.	14.58	354.	99.	96.	3.44		
619 2 COMBINED AT	CP173A	421.	13.58	148.	37.	36.	.90		
620 2 COMBINED AT	RC173A	1822.	17.42	1563.	545.	525.	31.02		
621 2 COMBINED AT	CP175A	687.	12.42	76.	19.	18.	.75		
622 2 COMBINED AT	1I176	700.	12.42	79.	20.	19.	.95		
623 2 COMBINED AT	CP176	792.	12.50	134.	34.	32.	1.40		
624 2 COMBINED AT	CP176A	378.	13.25	92.	23.	22.	3.09		
625 2 COMBINED AT	CP177A	581.	13.17	165.	41.	40.	3.58		
626 2 COMBINED AT	1I177	814.	13.33	237.	59.	57.	4.07		
627 2 COMBINED AT	CP177	902.	13.75	321.	82.	79.	16.68		
628 2 COMBINED AT	CP178	993.	13.42	382.	97.	94.	17.12		
629 2 COMBINED AT	1I179	1156.	13.58	448.	116.	111.	17.58		
630 2 COMBINED AT	CP179	3923.	14.17	2252.	630.	606.	25.34		
631 2 COMBINED AT	1I180	1796.	14.33	1228.	384.	370.	26.33		
632 2 COMBINED AT	CP180	1797.	14.33	1286.	401.	386.	30.29		
633 2 COMBINED AT	CP181	292.	14.75	221.	73.	70.	30.66		
633 2 COMBINED AT	CP181A	1200.	16.25	766.	265.	255.	26.00		
634 2 COMBINED AT	1I182	204.	12.75	34.	8.	8.	30.90		
635 2 COMBINED AT	CP182	903.	16.58	611.	221.	213.	38.46		
636 2 COMBINED AT	1I183	527.	16.42	253.	77.	74.	25.82		
637 2 COMBINED AT	2I183	797.	16.42	395.	124.	120.	26.21		
638 2 COMBINED AT	CP183	1078.	16.42	654.	209.	201.	27.07		
639 2 COMBINED AT	1I184	1051.	17.50	656.	231.	222.	27.86		
640 2 COMBINED AT	2I184	1467.	17.50	944.	332.	319.	40.32		
641 2 COMBINED AT	CP184	1574.	17.50	1104.	388.	373.	41.09		
642 2 COMBINED AT	1I185	487.	13.58	154.	39.	38.	.98		
643 2 COMBINED AT	CP185	552.	13.67	312.	113.	109.	42.07		
644 2 COMBINED AT	CP187	173.	13.67	52.	13.	13.	.68		
645 2 COMBINED AT	RCP187	1801.	17.75	1570.	549.	529.	31.70		
646 2 COMBINED AT	CP188	1457.	12.83	212.	53.	51.	11.08		
647 2 COMBINED AT	1I189	650.	12.67	135.	34.	33.	5.61		
648 2 COMBINED AT	CP189	940.	12.58	192.	48.	46.	6.12		
649 2 COMBINED AT	CP190	1336.	13.08	319.	80.	77.	6.98		
650 3 COMBINED AT	1I191	1774.	13.25	451.	113.	109.	10.07		
651 2 COMBINED AT	CP191	2358.	13.25	593.	149.	144.	12.18		
657 2 COMBINED AT	CP192A	1140.	13.50	177.	44.	43.	12.68		
658 2 COMBINED AT	CP192	1241.	13.75	251.	63.	60.	13.18		
659 2 COMBINED AT	1I193	596.	13.25	156.	39.	38.	14.09		
660 2 COMBINED AT	CP193	2347.	14.50	1229.	312.	301.	36.34		
661 2 COMBINED AT	1I194	514.	13.42	138.	35.	33.	31.19		
662 2 COMBINED AT	CP194	875.	13.50	740.	197.	189.	37.33		
663 2 COMBINED AT	1I195	1482.	15.58	1056.	336.	324.	35.64		
664 3 COMBINED AT	CP195	2559.	15.50	1800.	602.	580.	43.14		
665 2 COMBINED AT	1I196	439.	17.42	313.	124.	119.	38.93		
666 2 COMBINED AT	CP196	945.	17.42	806.	349.	336.	51.75		
667 2 COMBINED AT	CP197	611.	18.42	457.	182.	175.	42.09		
668 2 COMBINED AT	1I198	609.	19.17	440.	169.	163.	42.00		
669 2 COMBINED AT	2I198	846.	19.25	693.	271.	261.	42.98		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
670 2 COMBINED AT	CP198	1418.	19.17	1137.	450.	433.	44.09		
671 2 COMBINED AT	CP200	231.	12.67	53.	13.	13.	.40		
672 2 COMBINED AT	RCP200	1786.	18.08	1550.	552.	532.	32.10		
672 2 COMBINED AT	CP201	2025.	15.67	1298.	385.	371.	43.48		
673 2 COMBINED AT	CP202	565.	13.42	388.	181.	174.	52.23		
674 2 COMBINED AT	CP203	598.	18.92	456.	179.	172.	42.20		
675 2 COMBINED AT	1I204	1413.	19.25	1136.	453.	436.	44.36		
676 2 COMBINED AT	CP204	1721.	19.25	1453.	626.	603.	58.13		
677 2 COMBINED AT	CP205	1721.	19.33	1453.	627.	604.	58.19		
678 2 COMBINED AT	CP206	1722.	19.33	1453.	629.	606.	58.31		
679 2 COMBINED AT	RCP206	3249.	16.67	2901.	1156.	1114.	68.69		
680 2 COMBINED AT	CP207A	1450.	13.00	259.	65.	62.	11.58		
681 2 COMBINED AT	1I208	1921.	13.25	631.	160.	154.	13.18		
682 2 COMBINED AT	CP208	2175.	13.33	746.	188.	182.	14.18		
683 2 COMBINED AT	CP209A	1706.	13.58	519.	131.	126.	14.68		
684 2 COMBINED AT	1I209	1026.	14.08	295.	74.	71.	13.68		
685 2 COMBINED AT	CP209	1285.	14.08	373.	93.	90.	16.18		
686 2 COMBINED AT	1I210	292.	13.08	64.	16.	15.	13.64		
687 2 COMBINED AT	CP210	304.	13.08	123.	31.	30.	16.58		
688 2 COMBINED AT	1I212	1344.	15.50	636.	168.	162.	36.88		
689 2 COMBINED AT	CP212	1350.	15.58	658.	172.	166.	37.23		
690 2 COMBINED AT	CP214	1392.	13.08	276.	69.	67.	11.74		
691 2 COMBINED AT	1I215	874.	12.42	99.	25.	24.	.80		
692 2 COMBINED AT	CP215	1484.	12.42	373.	93.	90.	12.54		
693 2 COMBINED AT	CP217	372.	13.08	111.	28.	27.	1.49		
694 2 COMBINED AT	1I218	614.	13.25	152.	38.	37.	15.18		
695 2 COMBINED AT	CP218	669.	13.25	189.	47.	46.	15.67		
696 2 COMBINED AT	1I219	629.	14.75	321.	92.	89.	14.68		
697 2 COMBINED AT	2I219	1239.	14.67	553.	151.	145.	15.18		
698 2 COMBINED AT	CP219	1270.	14.58	565.	154.	148.	16.67		
699 2 COMBINED AT	1I220	650.	14.42	276.	70.	68.	15.18		
700 2 COMBINED AT	2I220	1115.	14.50	415.	106.	102.	16.68		
701 2 COMBINED AT	CP220	1241.	14.58	476.	121.	117.	18.17		
702 2 COMBINED AT	1I221	490.	15.33	202.	59.	56.	16.66		
703 2 COMBINED AT	2I221	707.	15.25	323.	89.	86.	17.06		
704 2 COMBINED AT	1I221A	276.	14.17	109.	28.	27.	.80		
704 2 COMBINED AT	CP221	1018.	15.17	450.	122.	118.	19.11		
705 2 COMBINED AT	CP221A	1377.	16.17	713.	194.	187.	38.03		
706 2 COMBINED AT	CP222	1380.	16.50	769.	238.	229.	39.13		
707 2 COMBINED AT	1I223	2054.	16.33	1344.	488.	471.	44.74		
708 2 COMBINED AT	CP223	2523.	16.33	1806.	690.	665.	54.67		
709 2 COMBINED AT	CP224	2453.	16.50	1745.	655.	631.	55.47		
710 2 COMBINED AT	1I225	520.	12.42	119.	66.	64.	.43		
711 2 COMBINED AT	CP225	959.	12.42	168.	79.	76.	.80		
712 2 COMBINED AT	1I226	1483.	12.33	180.	51.	49.	53.41		
713 2 COMBINED AT	CP226	1483.	12.33	572.	233.	224.	53.41		
714 2 COMBINED AT	RC228A	3236.	16.83	2897.	1142.	1100.	68.77		
715 3 COMBINED AT	CP229	863.	12.33	127.	32.	31.	1.02		
716 2 COMBINED AT	CP230A	530.	12.25	95.	24.	23.	1.20		
717 2 COMBINED AT	1I230	227.	12.42	33.	8.	8.	1.06		
718 2 COMBINED AT	CP230	672.	12.42	114.	29.	28.	1.24		
719 2 COMBINED AT	CP231	279.	12.50	53.	13.	13.	1.37		
720 2 COMBINED AT	CP233	1685.	12.58	427.	107.	103.	13.04		
721 2 COMBINED AT	1I234	689.	13.08	150.	37.	36.	1.04		
722 2 COMBINED AT	CP234	2084.	12.83	572.	143.	138.	14.08		
723 2 COMBINED AT	1I235	483.	13.42	144.	36.	35.	1.96		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
724 2 COMBINED AT	CP235	1467.	13.42	486.	122.	117.	16.04		
725 2 COMBINED AT	1I236	710.	13.33	187.	47.	45.	16.67		
726 2 COMBINED AT	CP236	1222.	13.58	425.	108.	104.	26.34		
727 2 COMBINED AT	1I237	513.	13.92	195.	51.	49.	16.17		
728 2 COMBINED AT	2I237	519.	13.92	275.	74.	72.	17.17		
729 2 COMBINED AT	CP237	993.	13.83	500.	132.	128.	27.84		
730 2 COMBINED AT	1I238	843.	15.17	453.	130.	126.	17.17		
731 2 COMBINED AT	2I238	918.	15.25	477.	137.	132.	18.67		
732 2 COMBINED AT	CP238	1226.	15.25	751.	213.	205.	30.34		
733 2 COMBINED AT	1I239	701.	15.75	319.	96.	93.	18.65		
734 2 COMBINED AT	2I239	1547.	16.25	757.	217.	209.	19.59		
735 2 COMBINED AT	CP239	2367.	16.17	1410.	406.	391.	31.76		
736 2 COMBINED AT	CP240	634.	16.50	552.	191.	184.	32.16		
737 2 COMBINED AT	1I241	2429.	16.92	1743.	680.	655.	56.98		
738 2 COMBINED AT	2I241	3693.	17.08	2453.	907.	874.	59.77		
739 2 COMBINED AT	CP241	4243.	17.08	2889.	1083.	1044.	78.75		
740 2 COMBINED AT	1I242	1000.	12.67	213.	80.	77.	1.94		
741 2 COMBINED AT	CP242	1049.	12.58	223.	83.	80.	2.03		
742 2 COMBINED AT	CP243B	1111.	12.67	233.	85.	82.	2.10		
743 2 COMBINED AT	CP243A	1261.	12.67	263.	93.	89.	2.32		
744 2 COMBINED AT	CP244A	485.	12.17	49.	13.	12.	1.51		
745 2 COMBINED AT	1I243	338.	12.42	63.	16.	16.	1.98		
746 2 COMBINED AT	CP243	1460.	12.67	324.	108.	104.	4.30		
747 2 COMBINED AT	1I244	582.	12.42	78.	20.	19.	1.70		
748 2 COMBINED AT	CP244	1195.	12.50	192.	48.	47.	1.74		
749 3 COMBINED AT	1I245	1056.	13.25	252.	64.	62.	2.14		
750 2 COMBINED AT	CP245	1860.	13.17	574.	171.	164.	5.05		
751 2 COMBINED AT	RCP245	3300.	17.58	2987.	1219.	1174.	73.82		
752 2 COMBINED AT	CP246	1239.	12.58	193.	48.	46.	1.68		
754 2 COMBINED AT	1I248	1313.	13.42	372.	93.	90.	15.08		
755 2 COMBINED AT	2I248	1602.	13.50	446.	112.	108.	15.58		
756 2 COMBINED AT	CP248	2174.	13.67	679.	170.	164.	17.54		
757 2 COMBINED AT	1I249	559.	13.25	188.	47.	45.	18.54		
758 2 COMBINED AT	CP249	694.	14.00	262.	66.	63.	28.84		
759 2 COMBINED AT	1I250	308.	16.08	145.	45.	43.	26.83		
760 2 COMBINED AT	2I250	411.	16.08	231.	67.	64.	28.33		
761 2 COMBINED AT	CP250	909.	14.42	466.	126.	121.	30.83		
762 2 COMBINED AT	1I250A	314.	14.50	180.	46.	44.	328.35		
763 2 COMBINED AT	2I250A	366.	14.58	214.	55.	53.	30.85		
764 2 COMBINED AT	CP250A	1254.	14.58	675.	180.	173.	33.84		
765 2 COMBINED AT	1I251	301.	13.17	87.	30.	29.	30.84		
766 2 COMBINED AT	2I251	1711.	16.50	937.	258.	249.	32.26		
767 2 COMBINED AT	CP251	2394.	16.50	1543.	424.	408.	35.76		
768 2 COMBINED AT	CP252	439.	16.83	325.	96.	92.	36.26		
769 2 COMBINED AT	1I253	4123.	17.67	2883.	1105.	1064.	79.75		
770 2 COMBINED AT	2I253	4122.	17.67	2883.	1111.	1070.	80.00		
771 2 COMBINED AT	CP253	4121.	17.67	2888.	1114.	1073.	84.50		
772 2 COMBINED AT	CP255	1512.	12.25	183.	50.	48.	1.84		
773 2 COMBINED AT	CP256	325.	13.00	95.	24.	23.	.90		
774 2 COMBINED AT	RCP258	3295.	17.75	2986.	1193.	1149.	74.20		
775 2 COMBINED AT	CP259	243.	12.25	20.	5.	5.	.32		
776 2 COMBINED AT	CP260	1336.	12.75	249.	63.	61.	2.18		
777 2 COMBINED AT	CP261	264.	13.17	62.	16.	15.	.91		
778 2 COMBINED AT	CP262	2219.	13.83	770.	194.	187.	18.57		
779 2 COMBINED AT	CP263	1071.	14.08	423.	108.	104.	19.07		
780 2 COMBINED AT	1I264	324.	13.00	91.	23.	22.	29.34		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
781 2 COMBINED AT	CP264	412.	14.33	212.	54.	52.	30.87		
782 2 COMBINED AT	1I265A	601.	13.08	132.	33.	32.	34.79		
783 2 COMBINED AT	CP265A	604.	13.08	161.	40.	39.	35.01		
784 2 COMBINED AT	1I266	256.	15.42	93.	23.	23.	34.17		
785 2 COMBINED AT	CP266	1964.	16.75	1341.	369.	355.	36.09		
786 2 COMBINED AT	1I267	485.	13.67	353.	109.	105.	36.76		
787 2 COMBINED AT	CP267	2106.	16.92	1534.	434.	418.	37.09		
788 2 COMBINED AT	1I268	4088.	18.00	2880.	1128.	1086.	85.45		
789 2 COMBINED AT	CP268	4703.	17.92	3270.	1269.	1222.	86.28		
790 2 COMBINED AT	CP269	491.	13.00	172.	45.	44.	1.25		
791 2 COMBINED AT	1I270	521.	12.33	89.	25.	24.	.52		
792 2 COMBINED AT	CP270	520.	12.33	92.	26.	25.	3.09		
793 2 COMBINED AT	1I271	1104.	13.50	265.	72.	69.	2.41		
794 2 COMBINED AT	CP271A	284.	13.42	86.	22.	21.	.59		
794 2 COMBINED AT	CP271	1104.	13.50	330.	93.	90.	2.57		
795 2 COMBINED AT	CP271B	109.	12.83	54.	15.	14.	.49		
796 2 COMBINED AT	CP271C	263.	12.17	68.	20.	19.	.79		
796 2 COMBINED AT	1I271C	263.	12.17	67.	20.	19.	.67		
797 2 COMBINED AT	1I272	116.	12.42	12.	3.	3.	.32		
798 2 COMBINED AT	2I272	304.	12.25	29.	7.	7.	.52		
799 2 COMBINED AT	CP272	304.	12.25	39.	14.	13.	1.19		
800 2 COMBINED AT	RCP272	3287.	18.08	2984.	1149.	1107.	75.39		
801 2 COMBINED AT	CP273	1407.	12.83	323.	82.	79.	2.83		
802 2 COMBINED AT	1I274	465.	13.58	134.	33.	32.	1.57		
803 2 COMBINED AT	CP274	687.	13.17	178.	45.	43.	4.40		
804 2 COMBINED AT	CP275	135.	12.42	17.	4.	4.	4.51		
805 2 COMBINED AT	CP276	186.	13.00	38.	9.	9.	4.77		
806 2 COMBINED AT	1I277	569.	13.08	127.	38.	36.	1.82		
807 2 COMBINED AT	2I277	556.	13.08	274.	76.	74.	19.48		
808 2 COMBINED AT	CP277	606.	13.17	279.	78.	75.	24.25		
809 2 COMBINED AT	1I278	1291.	15.50	537.	139.	134.	19.07		
810 2 COMBINED AT	2I278	1645.	15.50	723.	191.	184.	30.87		
811 2 COMBINED AT	CP278	1660.	15.42	793.	224.	216.	31.87		
812 2 COMBINED AT	CP279D	32.	12.17	3.	1.	1.	24.27		
813 2 COMBINED AT	CP279C	76.	12.25	8.	2.	2.	24.32		
814 2 COMBINED AT	CP279B	45.	12.25	4.	1.	1.	24.32		
815 2 COMBINED AT	CP279A	88.	12.42	12.	3.	3.	24.42		
816 2 COMBINED AT	1I279	76.	12.25	7.	2.	2.	24.47		
817 2 COMBINED AT	CP279	1632.	15.58	788.	224.	216.	36.14		
818 2 COMBINED AT	1I280	358.	13.33	104.	26.	25.	36.47		
819 2 COMBINED AT	CP280	826.	16.17	419.	117.	112.	43.77		
820 2 COMBINED AT	CP280A	585.	17.58	389.	119.	114.	43.83		
821 2 COMBINED AT	CP281	742.	13.92	270.	70.	67.	35.86		
822 2 COMBINED AT	CP282	669.	14.17	252.	65.	63.	35.98		
823 2 COMBINED AT	CP283	231.	17.33	154.	48.	47.	36.25		
824 2 COMBINED AT	CP284	1266.	17.58	990.	300.	289.	37.61		
825 2 COMBINED AT	CP285B	229.	17.58	154.	50.	48.	36.31		
826 2 COMBINED AT	1I285A	1257.	17.83	958.	283.	273.	37.67		
827 2 COMBINED AT	CP285A	1259.	17.83	958.	283.	273.	37.89		
828 2 COMBINED AT	CP285	1007.	18.00	719.	190.	183.	37.93		
829 2 COMBINED AT	1I286	4662.	18.50	3264.	1225.	1180.	86.98		
830 2 COMBINED AT	CP286	4662.	18.50	3264.	1235.	1190.	87.20		
831 2 COMBINED AT	1I287	4644.	18.67	3261.	1231.	1186.	87.43		
832 2 COMBINED AT	CP287	5319.	18.58	3724.	1362.	1312.	88.27		
833 2 COMBINED AT	CP287A	618.	13.83	249.	64.	62.	2.42		
834 2 COMBINED AT	1I287B	163.	14.17	68.	18.	17.	.36		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
835 2 COMBINED AT	CP287B	212.	14.25	107.	44.	42.	2.78		
836 2 COMBINED AT	1I287C	599.	16.83	391.	144.	139.	4.32		
837 2 COMBINED AT	CP287C	649.	16.83	430.	167.	161.	7.10		
838 2 COMBINED AT	1I287D	293.	14.00	111.	29.	28.	.73		
839 2 COMBINED AT	CP287D	382.	19.58	345.	149.	143.	7.83		
840 2 COMBINED AT	CP287E	199.	12.58	129.	39.	37.	1.74		
841 2 COMBINED AT	RC287E	3256.	19.17	2973.	1033.	995.	77.13		
842 2 COMBINED AT	CP288A	606.	13.33	188.	48.	46.	2.08		
843 2 COMBINED AT	1I289	598.	16.17	296.	88.	84.	3.09		
844 2 COMBINED AT	CP289	660.	16.17	387.	135.	130.	4.09		
845 2 COMBINED AT	RCP290	3278.	18.33	2980.	1127.	1086.	75.94		
846 2 COMBINED AT	CP291	453.	13.25	120.	32.	31.	1.54		
847 2 COMBINED AT	1I292	1237.	12.75	408.	105.	101.	3.79		
848 2 COMBINED AT	CP292	1704.	13.25	586.	150.	144.	5.36		
849 2 COMBINED AT	1I293	150.	14.25	45.	11.	11.	4.77		
850 2 COMBINED AT	3I293	751.	12.75	419.	116.	112.	25.02		
850 2 COMBINED AT	2I293	677.	12.67	156.	40.	38.	5.54		
851 2 COMBINED AT	CP293	1408.	13.50	693.	185.	178.	25.98		
852 2 COMBINED AT	CP293A	1367.	14.08	668.	181.	174.	26.05		
853 2 COMBINED AT	1I294	56.	13.08	10.	3.	2.	24.32		
854 2 COMBINED AT	CP294	264.	12.17	33.	8.	8.	24.50		
855 2 COMBINED AT	1I294A	730.	15.92	432.	141.	135.	36.40		
856 2 COMBINED AT	2I294A	730.	15.92	432.	141.	136.	36.40		
857 2 COMBINED AT	CP294A	730.	15.92	439.	144.	139.	36.40		
858 2 COMBINED AT	CP295	722.	16.08	447.	151.	145.	36.71		
859 3 COMBINED AT	CP296	674.	18.83	485.	191.	184.	44.76		
860 2 COMBINED AT	CP297	230.	13.17	152.	60.	57.	36.69		
861 2 COMBINED AT	1I297A	246.	18.25	238.	102.	98.	38.16		
862 2 COMBINED AT	CP297A	519.	18.33	432.	153.	148.	38.20		
863 2 COMBINED AT	CP298	4446.	19.83	3602.	1348.	1299.	89.11		
864 2 COMBINED AT	CP302	241.	12.17	22.	10.	9.	1.89		
865 2 COMBINED AT	1RC302	3282.	19.33	3009.	1018.	982.	85.71		
866 2 COMBINED AT	RCP302	3291.	19.42	3019.	1026.	989.	85.71		
867 2 COMBINED AT	1I303	25.	13.58	5.	1.	1.	.05		
868 2 COMBINED AT	2I303	27.	13.67	9.	2.	2.	.08		
869 2 COMBINED AT	3I303	32.	14.42	14.	4.	3.	.13		
870 2 COMBINED AT	4I303	65.	14.25	25.	6.	6.	.28		
871 2 COMBINED AT	5I303	90.	14.50	35.	9.	9.	.38		
872 2 COMBINED AT	6I303	132.	14.50	60.	16.	15.	.96		
873 2 COMBINED AT	CP303	911.	12.67	192.	50.	49.	2.17		
874 4 COMBINED AT	1C303A	530.	12.25	97.	24.	24.	1.48		
875 2 COMBINED AT	CP303A	516.	12.25	94.	24.	23.	18.99		
876 2 COMBINED AT	CP304	1646.	12.42	225.	56.	54.	2.58		
877 2 COMBINED AT	1I306	873.	13.83	360.	95.	92.	26.05		
878 2 COMBINED AT	CP306	974.	13.00	414.	109.	105.	26.54		
879 2 COMBINED AT	CP308	1063.	13.25	448.	118.	114.	26.86		
880 2 COMBINED AT	1I309	645.	13.17	152.	38.	37.	25.57		
881 2 COMBINED AT	CP309	1190.	15.25	590.	185.	178.	27.37		
882 2 COMBINED AT	CP311	721.	13.00	533.	175.	169.	37.44		
883 2 COMBINED AT	CP311A	270.	12.92	54.	14.	13.	.37		
884 2 COMBINED AT	CP312	645.	20.00	476.	200.	193.	45.42		
885 2 COMBINED AT	CP313	247.	13.17	59.	15.	14.	37.12		
886 2 COMBINED AT	CP315	510.	19.92	427.	162.	156.	38.67		
887 2 COMBINED AT	CP316	4438.	20.17	3599.	1333.	1284.	89.93		
888 2 COMBINED AT	CP317	341.	13.25	141.	36.	34.	.97		
889 2 COMBINED AT	CP318	580.	12.67	156.	43.	41.	1.01		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
890 2 COMBINED AT	CP319	578.	12.50	127.	36.	34.	.84		
891 2 COMBINED AT	CP320	675.	12.42	88.	25.	24.	2.53		
892 2 COMBINED AT	RCP320	3313.	19.75	3044.	990.	956.	86.35		
893 2 COMBINED AT	1I321	1309.	12.25	264.	69.	66.	2.83		
894 2 COMBINED AT	CP321	1309.	12.25	306.	80.	77.	4.60		
895 2 COMBINED AT	CP322	1104.	13.00	265.	71.	68.	4.94		
896 2 COMBINED AT	CP323	1600.	12.58	242.	61.	59.	2.73		
897 2 COMBINED AT	CP324	1180.	13.25	221.	57.	55.	3.09		
898 2 COMBINED AT	CP325	880.	13.33	205.	51.	50.	1.41		
899 2 COMBINED AT	CP326	533.	13.08	109.	27.	26.	.75		
900 2 COMBINED AT	CP327	1232.	13.50	513.	136.	131.	27.39		
901 2 COMBINED AT	CP328	1086.	16.50	583.	202.	195.	28.13		
902 2 COMBINED AT	CP329	663.	12.83	117.	30.	29.	.82		
903 2 COMBINED AT	1I330	983.	13.08	582.	195.	188.	38.03		
904 2 COMBINED AT	CP330	1446.	13.08	931.	407.	392.	48.59		
905 2 COMBINED AT	1I331	632.	21.25	473.	201.	194.	46.17		
906 2 COMBINED AT	CP331	632.	21.25	473.	214.	206.	46.54		
907 2 COMBINED AT	CP332	388.	13.75	139.	35.	34.	37.68		
908 2 COMBINED AT	CP333	388.	13.08	137.	36.	35.	1.00		
909 2 COMBINED AT	1I334	4432.	20.33	3596.	1330.	1281.	90.57		
910 2 COMBINED AT	CP334	4915.	20.33	4007.	1473.	1419.	91.31		
911 2 COMBINED AT	CP335	4906.	20.58	4005.	1457.	1403.	91.66		
912 2 COMBINED AT	1I336	311.	13.83	138.	36.	34.	33.31		
913 2 COMBINED AT	2I336	4899.	20.75	4010.	1468.	1414.	94.97		
914 2 COMBINED AT	CP336	4899.	20.75	4013.	1519.	1463.	96.25		
915 3 COMBINED AT	CP337	1120.	12.83	357.	103.	99.	2.34		
916 2 COMBINED AT	CP336A	4897.	20.83	4012.	1527.	1471.	96.62		
917 2 COMBINED AT	CP337B	58.	13.58	17.	5.	4.	.42		
918 2 COMBINED AT	CP338A	734.	12.42	86.	24.	23.	3.30		
919 2 COMBINED AT	CP338	852.	12.67	124.	33.	32.	3.61		
920 2 COMBINED AT	RCP338	3302.	20.42	3039.	918.	887.	87.43		
921 2 COMBINED AT	CP339	1396.	13.33	402.	108.	104.	5.94		
922 2 COMBINED AT	CP340	1090.	13.67	284.	75.	72.	3.57		
923 2 COMBINED AT	CP341	518.	13.92	199.	50.	48.	1.40		
924 2 COMBINED AT	CP342	858.	13.75	263.	66.	64.	1.80		
925 2 COMBINED AT	CP342A	625.	13.58	164.	41.	40.	1.13		
925 2 COMBINED AT	1IC343	1315.	13.75	581.	154.	148.	27.90		
926 2 COMBINED AT	CP343	1821.	13.83	1093.	368.	355.	30.59		
927 2 COMBINED AT	CP344	974.	17.33	580.	216.	208.	28.74		
928 2 COMBINED AT	CP345	756.	13.17	179.	45.	44.	1.25		
929 2 COMBINED AT	CP346A	1404.	13.50	935.	402.	387.	48.59		
930 2 COMBINED AT	CP346B	1268.	14.25	879.	369.	355.	38.58		
931 2 COMBINED AT	CP346C	1244.	14.08	878.	373.	359.	38.31		
932 2 COMBINED AT	1I347	641.	13.08	268.	71.	68.	38.70		
933 2 COMBINED AT	2I347	635.	13.08	300.	88.	84.	52.64		
934 2 COMBINED AT	CP347	635.	13.08	300.	88.	84.	52.98		
935 2 COMBINED AT	CP348A	379.	13.42	165.	44.	42.	1.22		
936 2 COMBINED AT	CP348B	667.	13.42	253.	67.	65.	1.88		
937 2 COMBINED AT	CP349	1430.	13.83	514.	141.	136.	6.87		
938 2 COMBINED AT	1I350	1417.	14.25	532.	148.	143.	7.06		
939 2 COMBINED AT	CP350	1417.	14.25	532.	155.	150.	8.79		
940 2 COMBINED AT	CP351	1053.	14.00	386.	103.	99.	4.33		
941 2 COMBINED AT	CP352	1238.	14.33	486.	130.	126.	3.60		
942 2 COMBINED AT	CP352A	925.	14.75	391.	107.	103.	4.46		
943 2 COMBINED AT	1I353	488.	14.58	216.	58.	56.	1.62		
944 2 COMBINED AT	CP353	1291.	13.92	478.	124.	120.	3.42		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
945 2 COMBINED AT	CP354	604.	13.92	206.	52.	51.	1.44		
946 2 COMBINED AT	1I355	1740.	14.50	1053.	350.	338.	31.04		
947 2 COMBINED AT	2I355	2031.	14.75	1238.	399.	385.	32.48		
948 2 COMBINED AT	CP355	2031.	14.75	1448.	614.	591.	48.68		
950 2 COMBINED AT	CP356	1792.	14.08	1095.	377.	363.	30.94		
951 2 COMBINED AT	CP357	750.	13.42	199.	50.	49.	1.39		
952 2 COMBINED AT	CP358	896.	18.00	829.	263.	253.	41.02		
953 2 COMBINED AT	1I359	404.	13.00	82.	21.	20.	.61		
954 2 COMBINED AT	2I359	1036.	13.33	281.	71.	68.	2.00		
955 2 COMBINED AT	CP359	1281.	14.42	776.	341.	328.	40.58		
956 2 COMBINED AT	1I360	789.	15.83	652.	263.	254.	40.83		
957 2 COMBINED AT	CP360	1026.	15.83	851.	342.	329.	40.83		
958 2 COMBINED AT	1I362	4387.	22.33	3542.	1222.	1177.	101.20		
959 2 COMBINED AT	CP362	4372.	22.42	3568.	1296.	1248.	119.10		
960 2 COMBINED AT	1I363	1777.	21.67	1455.	505.	486.	97.40		
961 2 COMBINED AT	2I363	4372.	22.00	3587.	1233.	1188.	98.56		
961 2 COMBINED AT	3I363	4392.	22.00	3610.	1294.	1246.	100.44		
962 2 COMBINED AT	CP363	4392.	22.00	3610.	1305.	1257.	100.78		
963 2 COMBINED AT	1I364	558.	13.00	157.	41.	39.	1.16		
964 2 COMBINED AT	CP364	3110.	21.17	2554.	970.	934.	97.93		
965 2 COMBINED AT	RCP364	3727.	21.92	2791.	905.	875.	133.69		
966 2 COMBINED AT	1I364A	4895.	20.92	4011.	1514.	1459.	96.69		
967 2 COMBINED AT	CP364A	4895.	20.92	4011.	1516.	1461.	96.77		
968 2 COMBINED AT	1RC367	3300.	20.58	3034.	893.	862.	89.88		
969 2 COMBINED AT	RCP367	3325.	20.58	3058.	986.	953.	89.88		
969 2 COMBINED AT	CP368	367.	13.58	150.	43.	41.	1.27		
970 2 COMBINED AT	RCP368	3332.	21.58	2698.	833.	805.	91.41		
971 2 COMBINED AT	RCP369	3313.	21.42	2769.	828.	800.	90.14		
972 2 COMBINED AT	RCP370	3320.	20.92	3008.	913.	882.	89.99		
973 2 COMBINED AT	1I371	1499.	14.83	595.	179.	173.	9.72		
975 2 COMBINED AT	CP371	1698.	20.67	1315.	556.	536.	65.40		
976 3 COMBINED AT	1I372	1929.	19.42	1515.	502.	483.	55.96		
977 2 COMBINED AT	2I372	2007.	19.42	1554.	530.	510.	56.61		
978 2 COMBINED AT	CP372	2124.	19.42	1616.	602.	580.	61.07		
979 2 COMBINED AT	1I373	1770.	15.92	1354.	520.	501.	49.11		
980 2 COMBINED AT	CP373	1770.	15.92	1354.	520.	501.	49.15		
981 2 COMBINED AT	1I374	1658.	16.92	1333.	500.	482.	49.91		
982 2 COMBINED AT	2I374	1783.	16.83	1376.	516.	497.	50.32		
983 2 COMBINED AT	CP374	1870.	16.83	1405.	533.	513.	50.74		
986 2 COMBINED AT	CP377	120.	12.42	15.	4.	4.	.30		
987 2 COMBINED AT	RC377A	7914.	23.42	5218.	1661.	1603.	156.34		
988 2 COMBINED AT	RC377B	7907.	23.58	4986.	1541.	1487.	156.76		
989 2 COMBINED AT	CP378	4349.	23.25	3184.	1038.	1000.	119.88		
990 2 COMBINED AT	RCP378	7917.	23.25	5339.	1716.	1656.	156.07		
991 2 COMBINED AT	1I379	332.	16.75	189.	70.	67.	61.78		
992 2 COMBINED AT	2I379	332.	16.75	189.	70.	67.	61.88		
993 2 COMBINED AT	3I379	337.	16.75	189.	86.	82.	62.24		
994 2 COMBINED AT	CP379	1797.	21.67	1425.	551.	531.	65.96		
995 2 COMBINED AT	CP380	188.	12.83	44.	13.	13.	.41		
996 2 COMBINED AT	RCP380	7874.	24.75	3565.	1000.	966.	158.18		
997 2 COMBINED AT	RC381B	7878.	24.67	3817.	1058.	1022.	157.77		
998 2 COMBINED AT	CP382	556.	13.17	275.	128.	123.	61.78		
999 2 COMBINED AT	RCP383	7882.	24.50	4063.	1127.	1088.	157.70		
1000 2 COMBINED AT	RC383A	7887.	24.33	4308.	1218.	1175.	157.52		
1001 2 COMBINED AT	CP384	269.	14.33	122.	33.	32.	.65		
1002 2 COMBINED AT	RCP386	7890.	24.25	4401.	1256.	1213.	157.36		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1003 2 COMBINED AT	RCP387	7894.	24.08	4537.	1312.	1266.	157.01		
1004 2 COMBINED AT	CPWTAF	9330.	24.92	4819.	1487.	1435.	183.74		
1010 DIVERSION TO	DCP3	490.	12.92	60.	15.	14.	4.86		
1011 DIVERSION TO	DCP10	1486.	12.75	193.	48.	46.	10.87		
1012 DIVERSION TO	DI43	1441.	12.92	80.	20.	19.	13.90		
1013 DIVERSION TO	DI43-4	21.	12.17	3.	1.	1.	1.09		
1014 DIVERSION TO	DI43-5	19.	12.17	2.	1.	1.	1.11		
1015 DIVERSION TO	DI43-6	6.	12.17	2.	1.	1.	1.13		
1016 DIVERSION TO	DI43-7	20.	12.08	2.	1.	1.	1.15		
1017 DIVERSION TO	DI43-8	12.	12.08	1.	0.	0.	1.16		
1018 DIVERSION TO	DI45-1	325.	13.00	35.	9.	9.	13.98		
1019 DIVERSION TO	DI45	330.	13.17	85.	21.	21.	14.36		
1020 DIVERSION TO	DI46-1	118.	13.42	33.	8.	8.	14.51		
1021 DIVERSION TO	DI119	166.	12.67	26.	7.	6.	.50		
1022 DIVERSION TO	DI120	263.	13.33	79.	20.	19.	1.83		
1023 DIVERSION TO	1D121A	263.	13.33	79.	20.	19.	1.83		
1024 DIVERSION TO	2D121A	58.	13.08	9.	2.	2.	2.33		
1025 DIVERSION TO	1D122	442.	13.08	226.	58.	56.	2.83		
1026 DIVERSION TO	2D122	96.	13.08	20.	5.	5.	3.21		
1027 DIVERSION TO	DI124	338.	13.00	74.	18.	18.	.44		
1028 DIVERSION TO	DI125	200.	13.33	56.	14.	14.	1.01		
1029 DIVERSION TO	DI126	439.	14.08	218.	59.	57.	5.25		
1030 DIVERSION TO	DI128	410.	13.17	98.	25.	24.	1.36		
1031 DIVERSION TO	DI130	0.	.08	0.	0.	0.	1.83		
1032 DIVERSION TO	DI134	2643.	13.58	949.	243.	234.	7.89		
1033 DIVERSION TO	DI136	308.	13.33	87.	22.	21.	1.01		
1034 DIVERSION TO	DI138	439.	14.08	218.	59.	57.	5.25		
1035 DIVERSION TO	DI139	685.	13.33	305.	81.	78.	6.35		
1036 DIVERSION TO	DI144	795.	13.42	242.	61.	58.	5.54		
1037 DIVERSION TO	DI145	1550.	13.50	665.	172.	166.	8.52		
1038 DIVERSION TO	DI146	290.	14.33	141.	40.	39.	14.48		
1039 DIVERSION TO	1D147	145.	14.33	71.	20.	19.	14.48		
1040 DIVERSION TO	2D147	2060.	14.00	741.	193.	186.	15.42		
1041 DIVERSION TO	DI148	172.	14.08	52.	13.	13.	16.48		
1042 DIVERSION TO	DI149	435.	14.50	265.	76.	74.	21.72		
1043 DIVERSION TO	DI150	49.	13.00	20.	5.	5.	17.46		
1044 DIVERSION TO	DI152	205.	13.75	122.	34.	33.	23.66		
1045 DIVERSION TO	DI153	435.	14.50	265.	76.	74.	21.72		
1046 DIVERSION TO	DI154	849.	15.75	529.	172.	166.	24.02		
1047 DIVERSION TO	DI155	0.	.08	0.	0.	0.	24.01		
1048 DIVERSION TO	DI158A	0.	.08	0.	0.	0.	1.14		
1050 DIVERSION TO	DI164	991.	14.00	315.	79.	76.	11.63		
1051 DIVERSION TO	DI165	868.	14.00	365.	91.	88.	11.63		
1052 DIVERSION TO	DI166	1943.	14.42	801.	224.	216.	15.92		
1053 DIVERSION TO	DI167	1753.	15.58	831.	263.	253.	18.44		
1054 DIVERSION TO	1D168	205.	13.75	122.	34.	33.	23.66		
1055 DIVERSION TO	2D168	117.	15.92	51.	14.	13.	25.61		
1056 DIVERSION TO	1D169	0.	.08	0.	0.	0.	24.01		
1057 DIVERSION TO	2D169	140.	13.50	84.	26.	25.	26.47		
1058 DIVERSION TO	DI175A	375.	12.25	34.	9.	8.	.28		
1059 DIVERSION TO	DI176A	0.	.08	0.	0.	0.	2.47		
1060 DIVERSION TO	DI177	294.	13.25	93.	24.	23.	15.08		
1061 DIVERSION TO	1D183	532.	15.92	256.	69.	67.	25.61		
1062 DIVERSION TO	2D183	274.	16.25	150.	47.	46.	26.00		
1063 DIVERSION TO	DI191	0.	.08	0.	0.	0.	3.09		
1064 DIVERSION TO	DI193	2291.	14.17	1121.	280.	270.	25.34		

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1065 DIVERSION TO	DI194	0.	.08	0.	0.	0.	25.34		
1066 DIVERSION TO	1D195	1507.	14.33	1086.	340.	328.	30.29		
1067 DIVERSION TO	2D195	292.	14.75	221.	73.	70.	30.66		
1068 DIVERSION TO	DI196	450.	16.58	309.	112.	108.	38.46		
1069 DIVERSION TO	DI197	629.	17.50	441.	155.	149.	41.09		
1070 DIVERSION TO	DI198	629.	17.50	441.	155.	149.	41.09		
1071 DIVERSION TO	1D201	2050.	15.50	1302.	375.	361.	43.14		
1072 DIVERSION TO	DI208	1339.	12.83	490.	123.	119.	12.18		
1073 DIVERSION TO	2D208	476.	13.25	118.	29.	28.	1.00		
1074 DIVERSION TO	DI209A	1541.	13.33	447.	112.	108.	14.18		
1075 DIVERSION TO	DI209	1110.	13.75	225.	56.	54.	13.18		
1076 DIVERSION TO	2D209	288.	13.58	82.	20.	20.	14.68		
1077 DIVERSION TO	DI210	0.	.08	0.	0.	0.	13.18		
1078 DIVERSION TO	2D210	222.	14.08	60.	15.	14.	16.18		
1079 DIVERSION TO	DI212	1536.	14.50	600.	150.	145.	36.34		
1080 DIVERSION TO	DI218	91.	13.08	38.	9.	9.	1.49		
1081 DIVERSION TO	1D219	626.	13.33	297.	76.	73.	14.18		
1082 DIVERSION TO	2D219	58.	13.25	13.	3.	3.	15.67		
1083 DIVERSION TO	1D220	636.	13.58	204.	52.	50.	14.68		
1084 DIVERSION TO	2D220	156.	14.58	64.	16.	16.	16.67		
1085 DIVERSION TO	1D221	550.	14.08	170.	43.	41.	16.18		
1086 DIVERSION TO	2D221	407.	14.58	131.	33.	32.	18.17		
1087 DIVERSION TO	DI225	60.	8.75	60.	53.	51.	54.67		
1088 DIVERSION TO	1D226	600.	17.42	475.	182.	175.	51.75		
1089 DIVERSION TO	2D226	0.	.08	0.	0.	0.	52.23		
1090 DIVERSION TO	DR228A	3212.	17.58	2894.	1064.	1025.	68.77		
1091 DIVERSION TO	1D230	187.	12.33	28.	7.	7.	1.02		
1092 DIVERSION TO	2D230	464.	12.25	81.	20.	20.	1.20		
1093 DIVERSION TO	DI231	172.	12.33	25.	6.	6.	1.02		
1094 DIVERSION TO	DI237	420.	13.25	131.	33.	32.	15.67		
1095 DIVERSION TO	DI238	893.	14.58	408.	114.	110.	16.67		
1096 DIVERSION TO	DI239	733.	14.58	319.	82.	79.	18.17		
1097 DIVERSION TO	DI245	1116.	12.50	162.	40.	39.	1.74		
1098 DIVERSION TO	1D248	990.	12.83	225.	56.	54.	14.08		
1099 DIVERSION TO	2D248	770.	13.42	237.	59.	57.	16.04		
1100 DIVERSION TO	DI249	325.	13.58	78.	20.	19.	26.34		
1101 DIVERSION TO	1D250	403.	13.58	116.	29.	28.	26.34		
1102 DIVERSION TO	2D250	301.	13.83	88.	22.	21.	27.84		
1103 DIVERSION TO	1D250A	306.	13.83	129.	32.	31.	27.84		
1104 DIVERSION TO	2D250A	57.	15.25	20.	5.	5.	30.34		
1105 DIVERSION TO	1D251	160.	15.25	56.	14.	13.	30.34		
1106 DIVERSION TO	2D251	1731.	16.17	888.	228.	220.	31.76		
1107 DIVERSION TO	DI256	119.	13.42	23.	6.	6.	.34		
1108 DIVERSION TO	DI259	19.	12.25	1.	0.	0.	.12		
1109 DIVERSION TO	DI261	0.	.08	0.	0.	0.	.50		
1110 DIVERSION TO	DI262	1955.	13.67	630.	158.	152.	17.54		
1111 DIVERSION TO	DI263	1024.	13.83	356.	90.	87.	18.57		
1112 DIVERSION TO	DI264	77.	14.00	24.	6.	6.	28.84		
1113 DIVERSION TO	DI265A	3.	14.58	1.	0.	0.	33.84		
1114 DIVERSION TO	1D266	276.	14.58	48.	12.	12.	33.84		
1115 DIVERSION TO	2D266	1948.	16.50	1264.	346.	333.	35.76		
1116 DIVERSION TO	DI267	415.	16.83	309.	91.	88.	36.26		
1117 DIVERSION TO	DI271A	211.	13.00	62.	16.	15.	.90		
1118 DIVERSION TO	DI272	122.	12.25	12.	3.	3.	.12		
1119 DIVERSION TO	DI277	98.	13.17	22.	5.	5.	.91		
1120 DIVERSION TO	1D278	676.	13.83	256.	64.	62.	18.57		

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

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1121	DIVERSION TO	2D278	731.	14.08	297.	76.	73.	19.07		
1122	DIVERSION TO	DI280	55.	14.00	20.	5.	5.	35.86		
1123	DIVERSION TO	DI283	236.	16.75	153.	43.	42.	36.09		
1124	DIVERSION TO	DI284	1285.	16.92	973.	284.	273.	37.09		
1125	DIVERSION TO	DI289	887.	13.75	287.	78.	75.	2.57		
1126	DIVERSION TO	DI291	186.	12.92	16.	4.	4.	.55		
1127	DIVERSION TO	DI292	979.	12.92	278.	71.	68.	2.83		
1128	DIVERSION TO	2D292	682.	13.17	178.	45.	43.	4.40		
1129	DIVERSION TO	1D293	90.	12.67	17.	4.	4.	4.51		
1130	DIVERSION TO	2D293	89.	13.00	28.	7.	7.	4.77		
1131	DIVERSION TO	3D293	583.	13.25	279.	78.	75.	24.25		
1132	DIVERSION TO	1D294	31.	12.25	3.	1.	1.	24.27		
1133	DIVERSION TO	2D294	45.	12.50	8.	2.	2.	24.32		
1134	DIVERSION TO	1D294A	10.	12.25	2.	0.	0.	24.32		
1135	DIVERSION TO	2D294A	67.	12.58	12.	3.	3.	24.42		
1136	DIVERSION TO	3D294A	738.	15.67	433.	133.	128.	36.14		
1137	DIVERSION TO	DI297	220.	17.83	153.	50.	48.	36.31		
1138	DIVERSION TO	1D297A	246.	17.83	238.	94.	91.	37.89		
1139	DIVERSION TO	2D297A	274.	18.08	195.	52.	50.	37.93		
1140	DIVERSION TO	DI302	14.	20.58	13.	4.	4.	1.74		
1141	DIVERSION TO	1D303	23.	12.17	3.	1.	1.	.02		
1142	DIVERSION TO	2D303	24.	12.00	2.	0.	0.	.05		
1143	DIVERSION TO	3D303	24.	12.25	5.	1.	1.	.08		
1144	DIVERSION TO	4D303	23.	12.25	5.	1.	1.	.13		
1145	DIVERSION TO	5D303	45.	12.33	11.	3.	3.	.28		
1146	DIVERSION TO	6D303	33.	12.50	12.	3.	3.	.38		
1147	DIVERSION TO	7D303	47.	12.50	26.	7.	6.	.68		
1148	DIVERSION TO	1D303A	282.	12.83	42.	11.	10.	1.00		
1149	DIVERSION TO	D164A	117.	13.67	33.	8.	8.	7.68		
1149	DIVERSION TO	2D303A	10.	12.17	1.	0.	0.	1.01		
1150	DIVERSION TO	3D303A	23.	12.17	5.	1.	1.	1.06		
1151	DIVERSION TO	1D306	894.	13.25	300.	78.	75.	5.36		
1152	DIVERSION TO	2D306	140.	14.17	65.	17.	17.	26.05		
1153	DIVERSION TO	DI320	0.	.08	0.	0.	0.	1.89		
1154	DIVERSION TO	DI321	122.	12.58	45.	11.	11.	2.73		
1155	DIVERSION TO	DI338A	0.	.08	0.	0.	0.	2.53		
1156	DIVERSION TO	DI346C	1273.	13.50	880.	384.	370.	48.59		
1157	DIVERSION TO	DI347	0.	.08	0.	0.	0.	.34		
1158	DIVERSION TO	DI350	100.	14.92	39.	11.	10.	4.46		
1159	DIVERSION TO	DI360	315.	14.25	210.	86.	83.	38.58		
1160	DIVERSION TO	DI363	1782.	20.92	1458.	547.	527.	96.77		
1161	DIVERSION TO	DIGILA	470.	21.17	405.	167.	161.	97.93		
1162	DIVERSION TO	DI367	373.	13.50	335.	103.	99.	2.34		
1163	DIVERSION TO	DI371	1681.	19.42	1302.	492.	474.	61.07		
1164	DIVERSION TO	DI379	63.	19.42	36.	10.	10.	61.07		
1165	ROUTED TO	SRWT3	0.	.08	0.	0.	0.	20.48	1197.48	17.58
1166	ROUTED TO	SR16	0.	.08	0.	0.	0.	1.13	1215.39	24.92
1167	ROUTED TO	SR20	0.	.08	0.	0.	0.	1.07	1456.94	24.92
1168	ROUTED TO	SR21	191.	13.25	40.	12.	11.	3.46	1347.09	14.67
1169	ROUTED TO	SR23	0.	.08	0.	0.	0.	3.62	1279.06	24.92
1170	ROUTED TO	SR24	0.	.08	0.	0.	0.	.14	1217.43	13.25
1171	ROUTED TO	SR25	1374.	12.50	146.	37.	36.	5.29	1214.98	12.50
1172	ROUTED TO	SR27	1642.	12.50	205.	52.	50.	2.16	1214.44	12.50
1173	ROUTED TO	SR29	0.	.08	0.	0.	0.	.22	1168.31	13.17
1174	ROUTED TO	SR38	3220.	12.50	357.	93.	90.	4.64	1300.13	12.50
1176	ROUTED TO	SR41A1	23.	12.17	3.	1.	1.	.02	1135.75	12.17

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1177 ROUTED TO	SR41A2	60.	12.00	3.	1.	1.	.05	1135.66	12.08
1178 ROUTED TO	SR41A3	24.	12.25	5.	1.	1.	.08	1131.18	12.25
1179 ROUTED TO	SR41A	23.	12.25	5.	1.	1.	.13	1126.26	12.25
1180 ROUTED TO	SR41-1	148.	12.33	14.	3.	3.	.28	1116.03	12.33
1181 ROUTED TO	SR41-2	33.	12.50	12.	3.	3.	.38	1101.42	12.42
1182 ROUTED TO	SR41	531.	12.50	64.	16.	16.	.68	1100.02	12.58
1183 ROUTED TO	SR42	6601.	12.75	797.	205.	198.	14.56	1102.23	12.83
1184 ROUTED TO	SR43-1	282.	12.83	42.	11.	10.	1.00	1094.94	12.83
1185 ROUTED TO	SR43-2	10.	12.17	1.	0.	0.	1.01	1093.36	12.17
1186 ROUTED TO	SR43-3	23.	12.17	5.	1.	1.	1.06	1093.79	12.17
1187 ROUTED TO	SR43-4	21.	12.17	3.	1.	1.	1.09	1094.17	12.17
1188 ROUTED TO	SR43-5	19.	12.17	2.	1.	1.	1.11	1091.68	12.17
1189 ROUTED TO	SR43-6	6.	12.17	2.	1.	1.	1.13	1089.99	12.17
1190 ROUTED TO	SR43-7	20.	12.08	2.	1.	1.	1.15	1089.50	12.08
1191 ROUTED TO	SR43-8	12.	12.08	1.	0.	0.	1.16	1086.51	12.08
1192 ROUTED TO	SR43	5503.	12.92	799.	206.	198.	13.90	1092.87	13.00
1193 ROUTED TO	SR45-1	1428.	13.00	90.	22.	22.	13.98	1086.77	13.08
1194 ROUTED TO	SR45	815.	13.17	101.	25.	24.	14.36	1084.85	13.25
1195 ROUTED TO	SR46-1	118.	13.42	33.	8.	8.	14.51	1080.94	12.50
1196 ROUTED TO	SR46	1705.	12.75	259.	65.	63.	17.43	1070.76	12.75
1197 ROUTED TO	RS47	0.	.08	0.	0.	0.	18.57	1040.07	24.92
1198 ROUTED TO	SR138	1327.	15.75	812.	261.	252.	24.02	1048.81	15.75
1199 ROUTED TO	SR139	835.	13.92	428.	118.	114.	7.22	1047.83	14.00
1200 ROUTED TO	SR158A	304.	13.75	78.	20.	20.	1.14	1115.42	13.83
1201 ROUTED TO	SR212	1337.	15.58	626.	161.	155.	36.88	1080.12	15.67
1202 ROUTED TO	SR221A	271.	14.25	95.	24.	23.	.80	1079.02	14.33
1203 ROUTED TO	SR225	84.	13.42	81.	47.	45.	.80	1063.81	13.50
1204 ROUTED TO	SR226	534.	13.08	493.	225.	217.	53.41	1077.10	18.50
1205 ROUTED TO	SR241	4245.	17.08	2888.	1083.	1043.	78.75	1048.40	17.08
1206 ROUTED TO	SR253	4102.	17.75	2881.	1101.	1061.	84.50	1026.53	17.75
1207 ROUTED TO	SR258	0.	.08	0.	0.	0.	.38	969.35	13.67
1208 ROUTED TO	SR259	0.	.08	0.	0.	0.	.32	1010.21	14.08
1209 ROUTED TO	SR268	4695.	18.00	3267.	1242.	1196.	86.28	1012.11	18.08
1210 ROUTED TO	SR269	36.	16.75	17.	5.	5.	1.25	1011.90	24.58
1211 ROUTED TO	SR270	468.	12.42	85.	22.	21.	3.09	1012.77	12.42
1212 ROUTED TO	SR271	904.	13.75	290.	79.	76.	2.57	1014.73	13.83
1213 ROUTED TO	SR271A	284.	13.42	86.	22.	21.	.59	1018.30	13.42
1214 ROUTED TO	SR271C	69.	17.42	26.	6.	6.	.79	1011.57	17.75
1215 ROUTED TO	SR272	0.	.08	0.	0.	0.	1.19	977.27	24.00
1216 ROUTED TO	SR273	1398.	12.92	323.	82.	79.	2.83	1057.40	13.00
1217 ROUTED TO	SR274	682.	13.17	178.	45.	43.	4.40	1045.57	13.25
1218 ROUTED TO	SR275	90.	12.67	17.	4.	4.	4.51	1039.57	12.67
1219 ROUTED TO	SR276	186.	13.00	38.	9.	9.	4.77	1036.06	13.00
1220 ROUTED TO	SR277	583.	13.25	279.	78.	75.	24.25	1025.94	13.25
1221 ROUTED TO	SR279A	67.	12.58	12.	3.	3.	24.42	1015.68	12.58
1222 ROUTED TO	SR279B	47.	12.25	4.	1.	1.	24.32	1020.09	12.25
1223 ROUTED TO	SR279C	45.	12.50	8.	2.	2.	24.32	1021.39	12.50
1224 ROUTED TO	SR279D	32.	12.25	3.	1.	1.	24.27	1023.59	12.25
1225 ROUTED TO	SR279	1634.	15.67	788.	224.	216.	36.14	1015.15	15.75
1226 ROUTED TO	SR280A	583.	17.67	385.	108.	104.	43.83	1008.92	17.75
1227 ROUTED TO	SR280	594.	16.83	390.	117.	112.	43.77	1009.52	16.92
1228 ROUTED TO	SR281	732.	14.00	259.	66.	64.	35.86	1008.53	14.00
1229 ROUTED TO	SR282	562.	14.58	252.	65.	63.	35.98	1002.91	14.58
1230 ROUTED TO	SR284	1259.	17.67	959.	282.	271.	37.61	1011.20	17.75
1231 ROUTED TO	SR285A	1259.	17.83	958.	283.	273.	37.89	1006.58	17.83
1232 ROUTED TO	SR285B	225.	17.75	154.	50.	48.	36.31	1009.07	17.92

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

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
1233 ROUTED TO	SR285	1007.	18.08	719.	190.	183.	37.93	1003.61	18.08
1234 ROUTED TO	SR287	4450.	19.58	3604.	1346.	1296.	88.27	994.06	19.67
1235 ROUTED TO	SR287A	55.	14.00	55.	27.	26.	2.42	980.87	16.58
1236 ROUTED TO	SR287B	65.	18.75	61.	25.	24.	2.78	979.95	19.67
1237 ROUTED TO	SR287C	376.	19.17	340.	125.	120.	7.10	978.78	19.58
1238 ROUTED TO	SR287D	67.	24.92	62.	25.	24.	7.83	981.50	24.92
1239 ROUTED TO	SR287E	14.	20.58	13.	4.	4.	1.74	985.23	20.25
1240 ROUTED TO	SR290	319.	12.92	64.	16.	15.	.55	996.51	12.92
1241 ROUTED TO	SR291	128.	14.33	115.	32.	31.	1.54	982.80	14.33
1242 ROUTED TO	SR293	1394.	13.58	683.	181.	174.	25.98	1007.92	13.67
1243 ROUTED TO	SR293A	1360.	14.17	641.	171.	165.	26.05	1006.10	14.25
1244 ROUTED TO	SR294	0.	.08	0.	0.	0.	24.50	977.26	18.17
1245 ROUTED TO	SR294A	729.	15.92	439.	142.	137.	36.40	1008.08	16.00
1246 ROUTED TO	SR297	0.	.08	0.	0.	0.	36.69	956.46	24.92
1247 ROUTED TO	SR298	4443.	19.92	3600.	1338.	1289.	89.11	980.27	19.92
1248 ROUTED TO	SR302	39.	12.58	21.	9.	9.	1.89	975.84	12.58
1249 ROUTED TO	SR305	615.	13.17	132.	33.	32.	.92	1003.66	13.17
1250 ROUTED TO	SR306	973.	13.50	414.	107.	103.	26.54	1005.48	13.00
1251 ROUTED TO	SR320	51.	13.25	45.	21.	20.	2.53	959.87	13.17
1252 ROUTED TO	SR321	931.	12.75	224.	58.	56.	4.60	1001.17	12.75
1253 ROUTED TO	SR323	1578.	12.58	219.	55.	53.	2.73	1001.91	12.58
1254 ROUTED TO	SR336	4897.	20.83	4012.	1515.	1459.	96.25	934.34	20.83
1255 ROUTED TO	SR336B	101.	12.17	9.	2.	2.	.08	923.48	12.17
1256 ROUTED TO	SR337	373.	13.50	335.	103.	99.	2.34	958.33	13.50
1257 ROUTED TO	SR338A	646.	12.50	86.	24.	23.	3.30	943.54	12.50
1258 ROUTED TO	SR346B	1271.	14.25	879.	367.	353.	38.58	908.81	14.33
1259 ROUTED TO	SR346C	1220.	14.25	871.	360.	347.	38.31	910.03	14.25
1260 ROUTED TO	SR347	611.	13.17	300.	88.	84.	52.98	908.16	13.17
1261 ROUTED TO	SR348	280.	12.83	49.	12.	12.	.34	908.40	12.83
1262 ROUTED TO	SR348B	645.	13.58	253.	67.	65.	1.88	914.58	13.67
1263 ROUTED TO	SR349	1416.	13.92	514.	141.	136.	6.87	895.06	14.00
1264 ROUTED TO	SR350	1374.	14.42	503.	143.	138.	8.79	889.29	14.42
1265 ROUTED TO	SR351	1003.	14.08	386.	103.	99.	4.33	894.17	14.25
1266 ROUTED TO	SR352	1144.	14.67	461.	121.	117.	3.60	892.88	14.67
1267 ROUTED TO	SR352A	897.	14.92	354.	95.	91.	4.46	891.48	15.00
1268 ROUTED TO	SR353	1288.	13.92	478.	124.	120.	3.42	898.37	16.67
1269 ROUTED TO	SR354	539.	14.17	206.	52.	51.	1.44	895.18	14.25
1270 ROUTED TO	SR355	1824.	15.25	1365.	544.	524.	48.68	895.16	15.33
1271 ROUTED TO	SR355A	0.	.08	0.	0.	0.	.04	892.56	13.92
1272 ROUTED TO	SR356	1775.	14.17	1069.	355.	342.	30.94	899.68	14.17
1273 ROUTED TO	SR358	860.	20.92	791.	219.	211.	41.02	895.47	21.33
1274 ROUTED TO	SR359	803.	15.33	655.	271.	261.	40.58	901.91	15.50
1275 ROUTED TO	SR360	940.	16.25	841.	323.	311.	40.83	895.21	16.42
1276 ROUTED TO	SR361	0.	.08	0.	0.	0.	.21	896.66	16.58
1277 ROUTED TO	SR362	4363.	22.50	3557.	1250.	1204.	119.10	902.86	22.67
1278 ROUTED TO	SR364	3110.	21.17	2554.	961.	926.	97.93	912.34	21.25
1279 ROUTED TO	SR368	221.	14.25	133.	39.	37.	1.27	917.77	14.25
1280 ROUTED TO	SR377	0.	.08	0.	0.	0.	.30	892.51	13.83
1281 ROUTED TO	SR379	1769.	21.83	1408.	538.	518.	65.96	861.15	22.17
1282 ROUTED TO	SR381	78.	13.58	16.	4.	4.	.21	866.93	13.67
1283 ROUTED TO	SR381A	0.	.08	0.	0.	0.	.10	865.43	14.50

The following is a listing of the various versions of aerial recution curves.

Case 1: Depth-area reduction curves from Table 2.2 and 2.1a of the 1995 Edition of the Hydrology Manual for the 6- and 24-hour storms, respectively.

Case 2: The 6-hour depth-area reduction curve was recreated from Figure 2.14 of the 1995 Edition of the Hydrology Manual. Data shown was taken from the Figure 2.14 and then smoothed visually to match the curve. The 24-hour depth-area curve was recreated from NWS Hydro-40 plot. The data shown for the 24-hour duration was taken from that publication and then smoothed visually to match the curve.

Case 3: Data taken directly from NWS Hydro-40 figures.

Case 4: Data suggested by Tom Loomis PE, RLS at the FCDMC. The 6-hour curve was unchanged from the 1995 Edition of the Hydrology Manual excepth for the addition of data points at the rainfall distribution pattern points. The 24-hour curve is based on a similar plotting and smoothing effort as was done for Case 2 (see correspondence Attachments 1 and 2).

Case 5: 6- and 24-hour depth-area reductions curves combined into a single curve from the data of Case 2. A set of regression equations is fit to the smoothed and combined data (actually just the 24 hour data).

Those equations are:

1. $7.29167E\text{-}8 * Area^4 - 7.70833E\text{-}6 * Area^3 + 3.30208E\text{-}4 * Area^2 - 8.60417E\text{-}3 * Area + 1.0$ (for $0.0 < Area <= 40.0$ with $R^2 = 1.000$)
2. $6.36681E\text{-}12 * Area^4 - 8.91371E\text{-}9 * Area^3 + 4.56903E\text{-}6 * Area^2 - 1.1213E\text{-}3 * Area + 0.914728$ (for $40.0 < Area <= 500.0$ with $R^2 = 0.9975$)

The recommended data set is Case 4

Case 1

6-Hour Duration Drainage Area sq. miles	6-Hour Duration Ratio to Point Rainfall	24-Hour Duration Drainage Area sq. miles	24-Hour Duration Ratio to Point Rainfall
0	1	0	1
1	0.987	10	0.94
5	0.96	20	0.91
10	0.94	30	0.9
20	0.91	40	0.88
30	0.89	50	0.87
40	0.87	60	0.86
50	0.86	70	0.856
100	0.8	80	0.855
200	0.72	90	0.846
300	0.66	100	0.842
400	0.61	110	0.838
500	0.57	120	0.834
		130	0.833
		140	0.829
		150	0.825
		200	0.817
		300	0.8
		400	0.79
		500	0.78

Case 2

6-Hour Duration Drainage Area sq. miles	6-Hour Duration Ratio to Point Rainfall	24-Hour Duration Drainage Area sq. miles	24-Hour Duration Ratio to Point Rainfall
0	1	0	1
0.5	0.992	10	0.94
1	0.987	20	0.91
2.8	0.9725	30	0.89
5	0.96	40	0.8775
10	0.94	50	0.868
16	0.92	100	0.84
20	0.909	150	0.8225
30	0.889	200	0.8125
40	0.8725	300	0.8
90	0.81	400	0.79
100	0.8	500	0.78
200	0.72		
300	0.66		
400	0.61		
500	0.57		

Case 3

6-Hour Duration Drainage Area sq. miles	6-Hour Duration Ratio to Point Rainfall	24-Hour Duration Drainage Area sq. miles	24-Hour Duration Ratio to Point Rainfall
		0	1
		12.5	0.94
		25	0.91
		50	0.88
		100	0.85
		150	0.83
		200	0.82
		250	0.8125
		500	0.7875

Case 4

6-Hour Duration Drainage Area sq. miles	6-Hour Duration Ratio to Point Rainfall	24-Hour Duration Drainage Area sq. miles	24-Hour Duration Ratio to Point Rainfall
0	1.000	0	1.000
0.5	0.994	10	0.950
1	0.987	20	0.918
2.8	0.975	30	0.900
5	0.960	40	0.887
10	0.940	50	0.877
16	0.922	60	0.870
20	0.910	70	0.863
30	0.890	80	0.857
40	0.870	90	0.852
90	0.810	100	0.848
100	0.800	110	0.845
		120	0.841
		130	0.838
		140	0.835
		150	0.832
		200	0.820
		250	0.812
		300	0.806
		400	0.796
		500	0.783

Case 5

Drainage Area sq. miles	Ratio to Point Rainfall
0.0	1.000
0.5	0.996
1.0	0.992
2.8	**0.978**
5.0	0.964
10.0	0.940
16.0	**0.920**
20.0	0.910
30.0	0.890
40.0	0.877
50.0	0.869
90.0	**0.845**
100.0	0.840
110.0	0.836
120.0	0.832
130.0	0.828
140.0	0.825
150.0	0.822
200.0	0.812
250.0	0.806
300.0	0.800
400.0	0.790
500.0	**0.780**

The following is a listing of the various versions of the 2-hour precipitation mass curve.

Column 1:	Time increment in minutes
Column 2:	Mass curve from Table 2.3 of the 1995 Edition of the Hydrology Manual
Column 3:	5-min. mass curve developed from HEC-1 output for a hypothetical distribution. The input the the hypothetical distribution was derived from point precipitation data for Phoenix Sky Harbor International Airport as shown in the original George V. Sabol Consulting Engineers Hydrology Manual documentation notebooks.
Column 4:	Mass curve recreated from Figure 2.15 of the 1995 Edition of the Hydrology Manual. Data shown was taken from the Figure 2.15 and then smoothed visually to match the curve.
Column 5:	Same as (3) except HEC-1 input determined independantly by Tom Loomis, PE, RLS at the FCDMC (see correspondance, Attachment 3).
Column 6:	Time increment in minutes
Column 7:	Same as (5) except time increment of 2 minutes (see correspondence, Attachment 3).

Recommended data is is shown in green.

Time minutes (1)	Manual 1995 Table 2.3 (2)	Hypothetical Distribution Output (3)	Manual 1995 Figure 2.15 (4)	Hypothetical Distribution Output (5)	Time minutes (6)	Hypothetical Distribution Output (5)
0	0.0	0.0	0.0	0.0	0.0	0.0
5	1.1	0.7	1.1	0.7	2.0	0.3
10	1.8	1.5	2.0	1.4	4.0	0.7
15	2.3	2.2	2.5	2.1	6.0	1.0
20	2.8	2.9	3.1	2.8	8.0	1.4
25	3.2	3.6	3.8	3.9	10.0	1.7
30	4.6	4.7	5.1	4.9	12.0	2.1
35	7.1	7.3	7.2	7.7	14.0	2.4
40	10.0	10.2	10.0	10.9	16.0	2.8
45	13.7	13.8	13.7	14.4	18.0	3.1
50	17.6	19.3	17.9	19.6	20.0	3.5
55	23.2	26.2	23.2	26.7	22.0	3.8
60	32.7	41.5	32.7	41.8	24.0	4.2
65	60.1	68.7	48.0	68.8	26.0	4.5
70	74.3	79.3	76.0	79.3	28.0	4.9
75	86.3	85.1	86.5	85.3	30.0	5.2
80	90.1	89.1	91.5	89.1	32.0	6.3
85	93.0	92.4	94.5	92.3	34.0	7.3
90	95.4	95.3	96.3	95.1	36.0	8.4
95	96.2	96.4	97.5	96.1	38.0	9.4
100	97.0	97.1	98.2	97.2	40.0	10.8
105	97.7	97.8	98.8	97.9	42.0	12.2
110	98.2	98.5	99.3	98.6	44.0	13.6
115	99.2	99.3	99.7	99.3	46.0	15.3
120	100.0	100.0	100.0	100.0	48.0	17.4
					50.0	19.9
					52.0	22.3
					54.0	25.8
					56.0	30.3
					58.0	36.6
					60.0	47.4
					62.0	58.2
					64.0	66.9

The following is a listing of the various versions of the 2-hour precipitation mass curve.

Column 1: Time increment in minutes

Column 2: Mass curve from Table 2.3 of the 1995 Edition of the Hydrology Manual

Column 3: 5-min. mass curve developed from HEC-1 output for a hypothetical distribution. The input the the hypothetical distribution was derived from point precipitation data for Phoenix Sky Harbor International Airport as shown in the original George V. Sabol Consulting Engineers Hydrology Manual documentation notebooks.

Column 4: Mass curve recreated from Figure 2.15 of the 1995 Edition of the Hydrology Manual. Data shown was taken from the Figure 2.15 and then smoothed visually to match the curve.

Column 5: Same as (3) except HEC-1 input determined independantly by Tom Loomis, PE, RLS at the FCDMC (see correspondance, Attachment 3).

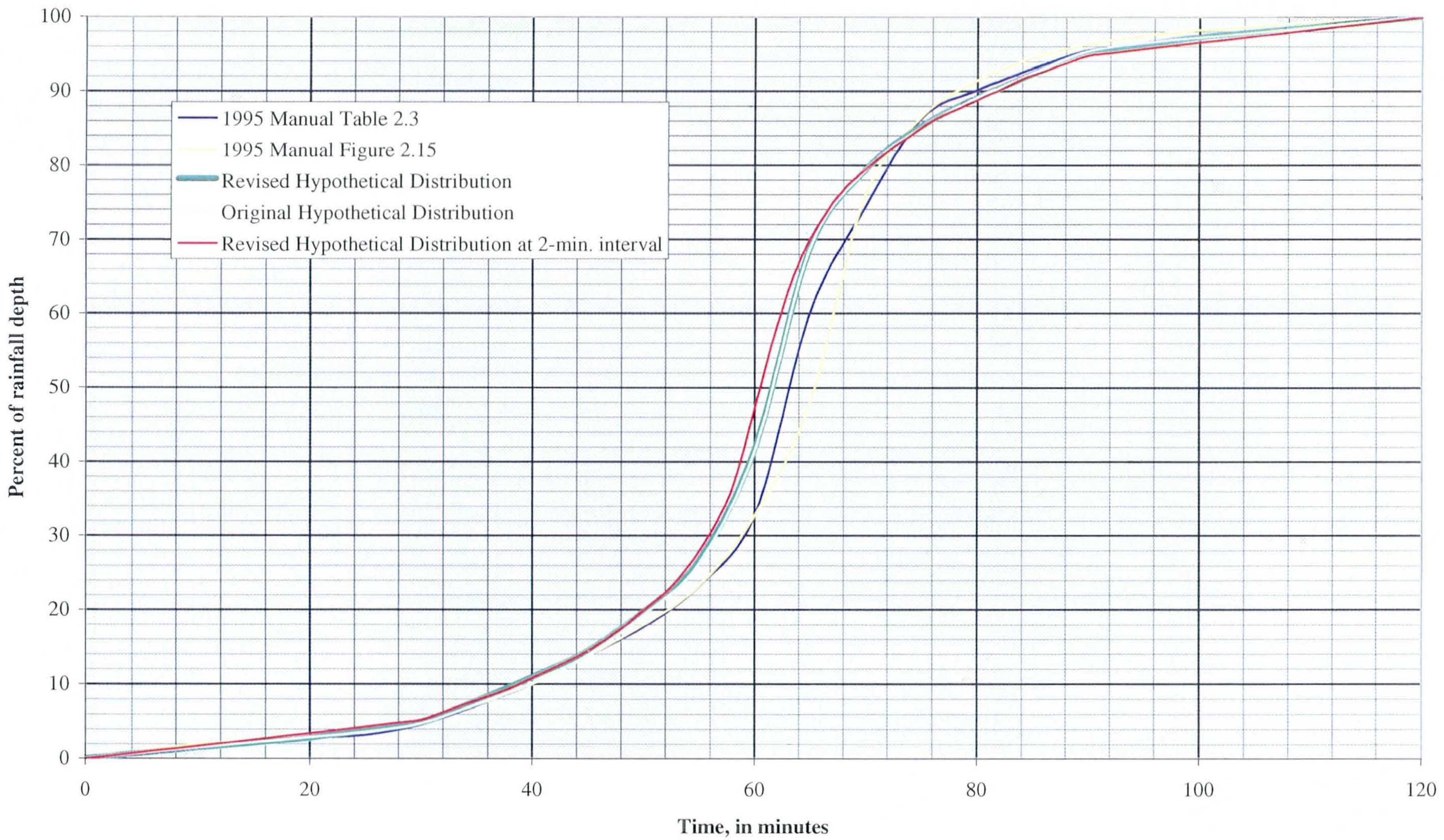
Column 6: Time increment in minutes

Column 7: Same as (5) except time increment of 2 minutes (see correspondence, Attachment 3).

Recommended data is is shown in green.

Time minutes (1)	Manual 1995 Table 2.3 (2)	Hypothetical Distribution Output (3)	Manual 1995 Figure 2.15 (4)	Hypothetical Distribution Output (5)	Time minutes (6)	Hypothetical Distribution Output (5)
					66.0	72.5
					68.0	76.7
					70.0	79.4
					72.0	81.9
					74.0	84.0
					76.0	86.1
					78.0	87.5
					80.0	88.9
					82.0	90.2
					84.0	91.6
					86.0	92.7
					88.0	93.7
					90.0	94.8
					92.0	95.1
					94.0	95.5
					96.0	95.8
					98.0	96.2
					100.0	96.5
					102.0	96.9
					104.0	97.2
					106.0	97.6
					108.0	97.9
					110.0	98.3
					112.0	98.6
					114.0	99.0
					116.0	99.3
					118.0	99.7
					120.0	100.0

Comparison of 2-hour precipitation mass curve data



21 January 2002
File: 82000042

Tel: 602-506-4767
Fax: 602-506-4601
trl@mail.maricopa.gov

Flood Control District of Maricopa County
2801 W. Durango St.
Phoenix AZ 85009

Attention: **Mr. Thomas Loomis, PE, RLS**

Dear Tom:

Reference: **Hydrology Manual**

Thank you for your input regarding the correlation of data between several of the tables and figures in the Hydrology Manual. I have reviewed the data that you provided and have adopted all the recommended data sets with the exception of the aerial reduction curves.

Prior to your involvement in Manual update, Mr. Joe Rumann suggested that the aerial reduction curves for the 6- and 24-hour duration storms could be combined into a single curve. Mr. Rumann's suggestion is based on the fact that there is little difference in the reduction factors between the two durations in the modeling range for the 6-hour duration storm (up to 100 sq. miles. In the hope that a single aerial reduction curve might eliminate confusion, it was agreed that the 24-hour duration storm aerial reduction curve be used for both storms and that a footnote be added regarding the upper limit of the 6-hour storm.

From a plot of the 24-hour duration aerial reduction data shown in Table 2.1a of the 1995 Edition of the Hydrology Manual, it was clear that the data did not plot as a smooth curve as shown in NWS Hydro-40. At that time, I felt that an equation representing the aerial reduction curve could be useful for consideration in future DDMSW updates. By breaking the 24-hour data into two sets I was able to closely fit regression equations to the data. From this a single table and figure for aerial reduction factors was prepared for the "2002" Edition of the Hydrology Manual. These equations and the resulting data are listed in the attached table as Case 5 and plotted in the attached figure. Also shown in the attached figure and table is the data that you developed. As can be seen from both the figure and table, these two data sets compare very favorably. Based on this comparison, I simply updated the aerial reduction table in the "2002" Edition of the Hydrology Manual to include values at each of the 6-hour pattern numbers as you requested. Please let me know if this is acceptable.

Reference: Hydrology Manual

With these data discrepancies resolved there are just a few more outstanding issues that need attention. First is the Rational Method C Coefficient table. During the project status meeting of 7 February 2000 Mr. Amir Motamedi provided a table of C Coefficients based on a different break down of land use categories to be considered as a replacement to the current table. In addition to the use of these new values is the issue of how to present the data, either as a range or as a single value. Furthermore, can the land use categories for Rational Method C Coefficients be related to the land use categories provided in the Rainfall Losses Chapter.

Second, I understand that there is a desire to model golf courses as having a saturated antecedent moisture condition (DTHETA = 0.0). While this does not have any procedural implications it does imply a standard of application that should be stated in the Hydrology Manual or the "Policies and Standards Manual".

Finally, there is the issue of general format and consistency. Currently in the Hydraulics Manual all equations and variables are shown in italics in a different font than the body text (Times New Roman for equations and Arial for body text). In the Hydrology Manual there is a bad mix of italicization and font variations.

Please let me know how you wish us to proceed on these last issues.

Sincerely,

STANTEC CONSULTING INC.

Mike Gerlach, PE
Water Resources
mgerlach@stantec.com

Mr. R. Michael Gerlach, PE
Stantec Consulting Inc.
8211 South 48th Street
Phoenix, AZ 85044

Dear Mike:

The purpose of this letter is to address the questions in your letter to me dated 21 January 2002. I understand your questions to be as follows:

1. Is it acceptable to use the 24-hour aerial reduction curve for both the 6- and 24-hour storms?

2. Is it acceptable to use the two regression equations you developed to reproduce the 24-hour aerial reduction curve?

3. Should the C-Coefficient table provided by Amir Motamedi, which is based on a different breakdown of land use categories, be used?

4. Should the C-Coefficients be presented as a range or as single values?

5. Can the land use categories for the Rational Method C-Coefficients be related to the land use categories provided in the Rainfall Losses Chapter?

6. Is the issue of modeling golf courses using a saturated antecedent moisture condition better addressed in the Policies and Standards Manual?

7. Should the font style used in the Hydraulics Manual (Times New Roman for equations, and Arial for body text) be used in the Hydrology Manual, which currently has a mix of italicization and font variations?

My responses are as follows:

Question 1. We have decided to keep using both the 6-hour and the 24-hour aerial reduction curves.

Question 2. We would rather use a single equation. There is no rush so we will do this task in-house.

Questions 3, 4 and 5. I have included an attachment that contains a new table of C-Coefficients for Chapter 3 (Table 3.2) based on a range of values, and a new table of IA, RTIMP and Vegetation Cover Density values for Chapter 4 (Table 4.2a). These two tables are based on a common set of land use categories. Please incorporate these two tables. Existing Table 4.1 can be eliminated. I will be placing a table of single C-Coefficient values for each land use category in the Policies and Standards Manual.

Question 6. Yes. I will address this in the Policies and Standards Manual; so do not revise this in the Hydrology Manual.

Question 7. I have agreed with Frank Thomas not to change the font selections being used in the Hydraulics Manual. Based on a telephone conversation with you last month, you said you could make font changes to the Hydrology Manual fairly readily. Therefore, change the Hydrology Manual to use the same font selection as is used in the Hydraulics Manual.

Please give me call with any questions.

Sincerely,

Thomas R. Loomis, PE, RLS
Special Projects Branch Manager

Table 3.2
C Coefficients for Use with the Rational Method

Land Use	Return Period			
	2-10 Year	25 Year	50 Year	100 Year
Streets and Roads				
Paved Roads	0.75 – 0.85	0.83 – 0.94	0.90 – 0.95	0.94 – 0.95
Gravel Roadways & Shoulders	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Industrial Areas				
Heavy	0.70 – 0.80	0.77 – 0.88	0.84 – 0.95	0.88 – 0.95
Light	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Business Areas				
Downtown	0.75 – 0.85	0.83 – 0.94	0.90 – 0.95	0.94 – 0.95
Neighborhood	0.55 – 0.65	0.61 – 0.72	0.66 – 0.78	0.69 – 0.81
Residential Areas				
Lawns – Flat	0.10 – 0.25	0.11 – 0.28	0.12 – 0.30	0.13 – 0.31
– Steep	0.25 – 0.40	0.28 – 0.44	0.30 – 0.48	0.31 – 0.50
Suburban	0.30 – 0.40	0.33 – 0.44	0.36 – 0.48	0.38 – 0.50
Single Family	0.45 – 0.55	0.50 – 0.61	0.54 – 0.66	0.56 – 0.69
Multi-Unit	0.50 – 0.60	0.55 – 0.66	0.60 – 0.72	0.63 – 0.75
Apartments	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Parks/Cemetaries	0.10 – 0.25	0.11 – 0.28	0.12 – 0.30	0.13 – 0.31
Playgrounds	0.40 – 0.50	0.44 – 0.55	0.48 – 0.60	0.50 – 0.63
Agricultural Areas	0.10 – 0.20	0.11 – 0.22	0.12 – 0.24	0.13 – 0.25
Bare Ground	0.20 – 0.30	0.22 – 0.33	0.24 – 0.36	0.25 – 0.38
Undeveloped Desert	0.30 – 0.40	0.33 – 0.44	0.36 – 0.48	0.38 – 0.50
Mountain Terrain (Slopes > 10%)	0.60 – 0.80	0.66 – 0.88	0.72 – 0.95	0.75 – 0.95

Note: Values of C for 25, 50 and 100 Year were derived using frequency adjustment factors of 1.10, 1.20, and 1.25, respectively, with an upper limit of 0.95 for C for the ~~2-10~~ Year values.
100

Typical Lot
1 acre
$P = 2.7''/12 = 0.225'$
$C_{Res} = 0.6$
$V = CPA$
$= 0.6 * 0.225' * 1$
$= 0.135$ ac ft
@ 3' depth avg, top width = 0.045 acre
≈ 5% land
add side slopes, say 7%

Table 3-2

C Coefficients for Use with the Rational Method

Land Use	Return Period 2 - 10 Year	25 Year	50 Year	100 Year
Residential Areas				
Lawns - Flat	0.10 - 0.25	0.11 - 0.28	0.12 - 0.30	0.13 - 0.31
Lawns - Steep	0.25 - 0.40	0.28 - 0.44	0.30 - 0.48	0.38 - 0.50
Roff Tops and Driveways	0.75 - 0.85	0.83 - 0.94	0.90 - 0.95	0.94 - 0.95
Desert Landscaping	0.30 - 0.40	0.33 - 0.44	0.36 - 0.48	0.38 - 0.50
1 Dwelling Unit per Acre (20% Impervious Area)[1]	0.39 - 0.49	0.43 - 0.54	0.47 - 0.57	0.49 - 0.59
2 Dwelling Unit per Acre (25% Impervious Area)[1]	0.41 - 0.51	0.46 - 0.57	0.50 - 0.60	0.52 - 0.61
3 Dwelling Unit per Acre (30% Impervious Area)[1]	0.44 - 0.54	0.48 - 0.59	0.52 - 0.62	0.55 - 0.64
4 Dwelling Unit per Acre (38% Impervious Area)[1]	0.47 - 0.57	0.52 - 0.63	0.57 - 0.66	0.59 - 0.67
8 Dwelling Unit per Acre (65% Impervious Area)[1]	0.59 - 0.69	0.66 - 0.77	0.71 - 0.79	0.74 - 0.79
Multifamily Residential Zoning	0.62 - 0.72	0.68 - 0.79	0.74 - 0.81	0.77 - 0.82
Apartments	0.66 - 0.76	0.73 - 0.84	0.79 - 0.86	0.83 - 0.86

Notes

[1]Reference: Applied Hydrology, Chow, Maidement and Mays, 1988

- For 1 to 8 dwelling units per acre, the remaining pervious area is considered to be desert landscaping

Table 3.2
C Coefficients for Use with the Rational Method

Land Use	Standard Abreviation	Description	Range of C Coefficient by Storm Frequency[1,2]							
			2-10 Year		25 Year		50 Year		100 Year	
			min	max	min	max	min	max	min	max
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Very Low Density Residential	VLDR	40,000 sf and greater lot size[3]	0.33	0.42	0.36	0.46	0.40	0.50	0.41	0.53
Low Density Residential	LDR	12,000-40,000 sf lot size[3]	0.42	0.48	0.46	0.53	0.50	0.58	0.53	0.60
Medium Density Residential	MDR	6,000-12,000 sf lot size[3]	0.48	0.65	0.53	0.72	0.58	0.78	0.60	0.82
Multiple Family Residential	MFR	1,000-6000 sf lot size[3]	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
Industrial 1	I1	Light and Garden[3]	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
Industrial 2	I2	General and Heavy[3]	0.70	0.80	0.77	0.88	0.84	0.95	0.88	0.95
Commercial 1	C1	Light, Neighborhood, Residential[3]	0.55	0.65	0.61	0.72	0.66	0.78	0.69	0.81
Commercial 2	C2	Central, General, Office, Intermediate[3]	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
Pavement and Rooftops	P	Asphalt and concrete, sloped rooftops	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
Gravel Roadways & Shoulders	GR	Graded and compacted, treated and untreated	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
Agricultural	AG	Tilled fields, irrigated pastures, slopes<1%	0.10	0.20	0.11	0.22	0.12	0.24	0.13	0.25
Lawns/Parks/Cemetaries 1	LPC1	Over 80% maintained lawn, slope<5%	0.10	0.25	0.11	0.28	0.12	0.30	0.13	0.31
Lawns/Parks/Cemetaries 2	LPC2	Over 80% maintained lawn, slope>5%	0.25	0.40	0.28	0.44	0.30	0.48	0.31	0.50
Desert Landscape 1	DL1	Landscaping with impervious under treatment	0.55	0.85	0.61	0.94	0.66	0.95	0.69	0.95
Desert Landscape 2	DL2	Landscaping w/o impervious under treatment	0.30	0.40	0.33	0.44	0.36	0.48	0.38	0.50
Undeveloped Desert and Rangeland	NDR	Little topographic relief, slopes<5%	0.25	0.40	0.28	0.44	0.30	0.48	0.31	0.50
Hillslopes, Sonoran Desert	NHS	Moderate topographic relief, slopes>5%	0.40	0.55	0.44	0.61	0.48	0.66	0.50	0.69
Mountain Terrain	NMT	High topographic relief, slopes>10%	0.55 .60	0.80	0.61	0.88	0.66	0.95	0.69	0.95

[1] Values of C for 25, 50 and 100 Year frequencies were derived using adjustment factors of 1.10, 1.20 and 1.25, respectively, applied to the ~~2-10~~ 100 Year values, with an upper limit of 0.95.

[2] The ranges of C Coefficient shown for urban land uses are derived from the maximum lot coverage requirements from zoning densities for most communities in Maricopa County. Refer to the adopted policies and standards for specific requirements by each community. The ranges of C Coefficient for other land uses are derived using ??????.

[3] C Coefficients shown for urban land uses are for lot coverage only, and do not include the adjacent street and right-of-way, or alleys.

Table ?? For Policies and Standards Manual
C Coefficients for Use with the Rational Method

Land Use	Standard Abreviation	Description	2-10 Year	25 Year	50 Year	100 Year
(1)	(2)	(3)	(4)	(6)	(8)	(10)
Very Low Density Residential	VLDR	40,000 sf and greater lot size	0.35	0.39	0.42	0.44
Low Density Residential	LDR	12,000-40,000 sf lot size	0.45	0.50	0.54	0.56
Medium Density Residential	MDR	6,000-12,000 sf lot size	0.54	0.59	0.65	0.68
Multiple Family Residential	MFR	1,000-6000 sf lot size	0.70	0.77	0.84	0.88
Industrial 1	I1	Light and Garden	0.65	0.72	0.78	0.81
Industrial 2	I2	General and Heavy	0.75	0.83	0.90	0.94
Commercial 1	C1	Light, Neighborhood, Residential	0.60	0.66	0.72	0.75
Commercial 2	C2	Central, General, Office, Intermediate	0.80	0.88	0.95	0.95
Pavement and Rooftops	P	Asphalt and concrete, sloped rooftops	0.80	0.88	0.95	0.95
Gravel Roadways & Shoulders	GR	Graded and compacted, treated and untreated	0.65	0.72	0.78	0.81
Agricultural	AG	Tilled fields, irrigated pastures, slopes<1%	0.15	0.17	0.18	0.19
Lawns/Parks/Cemetaries 1	LPC1	Over 80% maintained lawn, slope<5%	0.18	0.19	0.21	0.22
Lawns/Parks/Cemetaries 2	LPC2	Over 80% maintained lawn, slope>5%	0.33	0.36	0.39	0.41
Desert Landscape 1	DL1	Landscaping with impervious under treatment	0.70	0.77	0.84	0.88
Desert Landscape 2	DL2	Landscaping w/o impervious under treatment	0.35	0.39	0.42	0.44
Undeveloped Desert and Rangeland	NDR	Little topographic relief, slopes<5%	0.33	0.36	0.39	0.41
Hillslopes, Sonoran Desert	NHS	Moderate topographic relief, slopes>5%	0.48	0.52	0.57	0.59
Mountain Terrain	NMT	High topographic relief, slopes>10%	0.68	0.74	0.81	0.84

Water

does not match 3.2

Not sure what this table is for

Table 4.2a
IA, RTIMP, and Percent Vegetation Cover for Representative Land Uses in Maricopa County

Land Use[1]	Standard Abreviation	Description	IA[2] inches	RTIMP[2,3] %	Vegetation Cover[2,4] %
(1)	(2)	(3)	(4)	(5)	(6)
Very Low Density Residential	VLDR	40,000 sf and greater lot size	0.30	5	30
Low Density Residential	LDR	12,000-40,000 sf lot size	0.30	15	50
Medium Density Residential	MDR	6,000-12,000 sf lot size	0.25	30	50
Multiple Family Residential	MFR	1,000-6000 sf lot size	0.25	45	50
Industrial 1	I1	Light and Garden	0.15	55	60
Industrial 2	I2	General and Heavy	0.15	55	60
Commercial 1	C1	Light, Neighborhood, Residential	0.10	80	75
Commercial 2	C2	Central, General, Office, Intermediate	0.10	80	75
Pavement and Rooftops	P	Asphalt and concrete, sloped rooftops	0.05	95	0
Gravel Roadways & Shoulders	GR	Graded and compacted, treated and untreated	0.10	50	0
Agricultural	AG	Tilled fields, irrigated pastures, slopes<1%	0.50	0	85
Lawns/Parks/Cemetaries 1	LPC1	Over 80% maintained lawn, slope<5%	0.20	Varies[5]	80
Lawns/Parks/Cemetaries 2	LPC2	Over 80% maintained lawn, slope>5%	0.10	Varies[5]	80
Desert Landscape 1	DL1	Landscaping with impervious under treatment	0.10	95	30
Desert Landscape 2	DL2	Landscaping w/o impervious under treatment	0.20	0	30
Undeveloped Desert and Rangeland	NDR	Little topographic relief, slopes<5%	0.35	Varies[5]	Varies[6]
Hillslopes, Sonoran Desert	NHS	Moderate topographic relief, slopes>5%	0.15	Varies[5]	Varies[6]
Mountain	NMT	High topographic relief, slopes>10%	0.25	Varies[5]	Varies[6]

[1] Other land use or zoning classifications, such as Planned Area Developments and schools should be evaluated on a case by case basis.

[2] These values have been selected to fit many typical settings in Maricopa County; however, the engineer/hydrologist should always evaluate the specific circumstances in any particular watershed for hydrologic variations from these typical values.

[3] RTIMP = Percent Effective Impervious Area, including right-of-way. Effective means that all impervious areas are assumed hydraulically connecte(The RTIMP values may need to be adjusted based on an evaluation of hydraulic connectivity.

[4] Vegetation Cover = Percent vegetation cover for pervious areas only.

[5] RTIMP values should estimated on a case by case basis.

[6] Vegetation Cover values should estimated on a case by case basis.

1.0 INTRODUCTION

This is an internal memorandum created to document research done in regards to the Rational Method runoff coefficient, C, and proposed revisions to the Drainage Design Manual for Maricopa County, AZ, Volume I, Hydrology (Hydrology Manual). The Hydrology Manual is currently in the process of revision, with the majority of the proposed revisions being done by a consultant, Stantec Consulting, Inc., with a focus on rainfall-runoff methods for the more frequent 2-, 5-, and 10-year storms. In the process of review of the existing manual, one of the changes proposed involved creation of a consistent set of land use classifications between the application of the unit hydrograph method and the Rational Method (see Table 1). As a result of creating this consistent set of land uses, and conversations with the District Regulatory Division, it became apparent that the existing table of C values would need to be revised as well. C is the variable of the Rational Method with the greatest degree of uncertainty, and there is little empirical data available for assigning C values to the desired land use categories. Therefore, this research was undertaken to provide the best-available basis for assigning C values. The goal was to develop a range of C values for each land use category that provide consistency in results with modeling done using the unit hydrograph method.

TABLE 1: Standard land use categories for the Rational and Unit Hydrograph methods

Land Use Category	Standard Abbreviation	Description
Very Low Density Residential	VLDR	40,000 sf and greater lot size
Low Density Residential	LDR	12,000-40,000 sf lot size
Medium Density Residential	MDR	6,000-12,000 sf lot size
Multiple Family Residential	MFR	1,000-6000 sf lot size
Industrial 1	I1	Light and Garden
Industrial 2	I2	General and Heavy
Commercial 1	C1	Light, Neighborhood, Residential
Commercial 2	C2	Central, General, Office, Intermediate
Pavement and Rooftops	P	Asphalt and concrete, sloped rooftops
Gravel Roadways & Shoulders	GR	Graded and compacted, treated and untreated
Agricultural	AG	Tilled fields, irrigated pastures, slopes<1%
Lawns/Parks/Cemetaries 1	LPC1	Over 80% maintained lawn, slope<5%
Lawns/Parks/Cemetaries 2	LPC2	Over 80% maintained lawn, slope>5%
Desert Landscape 1	DL1	Landscaping with impervious under treatment
Desert Landscape 2	DL2	Landscaping w/o impervious under treatment
Undeveloped Desert/Rangeland	NDR	Little topographic relief, slopes<5%
Hillslopes, Sonoran Desert	NHS	Moderate topographic relief, slopes>5%
Mountain Terrain	NMT	High topographic relief, slopes>10%

2.0 BASIS FOR C COEFFICIENTS

The current set of C values is based on a table of general land use categories modified from the American Society of Civil Engineers (ASCE, 1969), and included in the Hydrology Manual as Table 3.2. The range of C values from ASCE (1969) are described as being applicable for storms of 5- to 10-year frequency. The values in Table 3.2 for the 25-, 50- and 100-year frequencies are determined based on frequency adjustment factors of 1.10, 1.20 and 1.25, respectively, applied to the values shown for storms of 2- through 10-year frequency. Table 3.2 is included herein as Figure 1.

C is defined by Rossmiller (1981) to be *"that fraction of rainfall (expressed as a dimensionless decimal) which appears as surface runoff from the contributing drainage area"*. According to Rossmiller, *"Precipitation is included in the Rational formula by using the average rainfall intensity over a period of time. By default, all other portions of the hydrologic cycle must be contained in the runoff coefficient, C. Therefore, C includes interception, depression storage, infiltration, evaporation, and groundwater flow. The variables needed to estimate C should include soil type, land use, degree of imperviousness, watershed slope, surface roughness, antecedent moisture conditions, duration of rainfall, and the intensity of rainfall as reflected by the recurrence interval."* McCuen (1998) states that *"the value of the runoff coefficient (C) is a function of the land use, cover condition, soil group, and watershed slope."* Chow, Maidment and Mays (1988) substantiate that C is a function of percent imperviousness, slope, ponding character of the surface, rainfall intensity, proximity of the water table, porosity of the soil, and adds other variables including degree of soil compaction and vegetation. According to Maidment (1993), who is also in concurrence, further states ... *"there is considerable evidence that the one set of C values, even with detailed variations for different conditions, will not apply to regions with different hydrologic regimes or even uniformly within large regions. Considerable judgment and experience are required in selecting satisfactory values of C for design, and there is a need to check values against observed flood data in a given region..."*. This is interpreted to mean that C values derived from standard tables in the literature may not necessarily be appropriate in any given region. C values should be based on flood frequency data for the area, and initial abstraction, soil characteristics, slope, percent impervious and vegetation characteristics specific to the watershed in question. It is clear that these C values are valid for estimation of peak discharge, but are not valid for estimation of runoff volumes. The current Hydrology Manual allows the use of C values for estimation of runoff volumes.

FIGURE 1: Table 3.2 from Hydrology Manual

Table 3.2
C Coefficients for Use with the Rational Method

Land Use	Return Period			
	2-10 Year	25 Year	50 Year	100 Year
Streets and Roads				
Paved Roads	0.75 – 0.85	0.83 – 0.94	0.90 – 0.95	0.94 – 0.95
Gravel Roadways & Shoulders	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Industrial Areas				
Heavy	0.70 – 0.80	0.77 – 0.88	0.84 – 0.95	0.88 – 0.95
Light	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Business Areas				
Downtown	0.75 – 0.85	0.83 – 0.94	0.90 – 0.95	0.94 – 0.95
Neighborhood	0.55 – 0.65	0.61 – 0.72	0.66 – 0.78	0.69 – 0.81
Residential Areas				
Lawns – Flat	0.10 – 0.25	0.11 – 0.28	0.12 – 0.30	0.13 – 0.31
– Steep	0.25 – 0.40	0.28 – 0.44	0.30 – 0.48	0.31 – 0.50
Suburban	0.30 – 0.40	0.33 – 0.44	0.36 – 0.48	0.38 – 0.50
Single Family	0.45 – 0.55	0.50 – 0.61	0.54 – 0.66	0.56 – 0.69
Multi-Unit	0.50 – 0.60	0.55 – 0.66	0.60 – 0.72	0.63 – 0.75
Apartments	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Parks/Cemetaries	0.10 – 0.25	0.11 – 0.28	0.12 – 0.30	0.13 – 0.31
Playgrounds	0.40 – 0.50	0.44 – 0.55	0.48 – 0.60	0.50 – 0.63
Agricultural Areas	0.10 – 0.20	0.11 – 0.22	0.12 – 0.24	0.13 – 0.25
Bare Ground	0.20 – 0.30	0.22 – 0.33	0.24 – 0.36	0.25 – 0.38
Undeveloped Desert	0.30 – 0.40	0.33 – 0.44	0.36 – 0.48	0.38 – 0.50
Mountain Terrain (Slopes > 10%)	0.60 – 0.80	0.66 – 0.88	0.72 – 0.95	0.75 – 0.95

Note: Values of C for 25, 50 and 100 Year were derived using frequency adjustment factors of 1.10, 1.20, and 1.25, respectively, with an upper limit of 0.95 for C for the 2-10 Year values.

3.0 APPROACH

There is very little historical gage data available for the urbanized areas of Maricopa County for use in estimating C values for runoff events of record. Because of this, the best available method for estimating C values that are a function of the variables listed in Section 2 is the unit hydrograph method. The Clark unit hydrograph method, in combination with the Green & Ampt rainfall loss equation, can be used to estimate peak discharges from urban watersheds that can in turn be used to estimate C values specific to Maricopa County. The basic approach used is as follows:

1. Establish physical characteristics for a typical urban watershed sub-basin. For the purposes of this study, the following were used (refer to Chapter 3 of the Hydrology Manual for definitions):

A: 0.0156 sm (10 acres); 0.125 sm (80 acres), 0.25 sm (160 acres)

L: 0.35 miles, 1860 feet (a relatively short length was used to provide conservative T_c estimates)

K_b: Type A = 0.028, Type B = 0.054, Type C = 0.102, Type D = 0.143

S_{adj}: 0.5%, 26.4 ft/mi; 1%, 52.8 ft/mi; 2%, 105.6 ft/mi; 5%, 247 ft/mi; 7%, 283 ft/mi; 10%, 306 ft/mi
S_{adj} are the study slopes adjusted per Figure 5.4 of the Hydrology Manual.

Soils:	Clay Loam	Silt	Loam	Sandy Loam
	XKSAT = 0.04	XKSAT = 0.10	XKSAT = 0.25	XKSAT = 0.40

TABLE 2: Land use parameters used for HEC-1 modeling

Land Use Category	Standard Abbreviation	IA inches	RTIMP %	Vegetation Cover %	Surface Roughness Type
Very Low Density Residential	VLDR	0.30	5	30	A
Low Density Residential	LDR	0.30	15	50	A
Medium Density Residential	MDR	0.25	30	50	A
Multiple Family Residential	MFR	0.25	45	50	A
Industrial 1	I1	0.15	55	60	A
Industrial 2	I2	0.15	55	60	A
Commercial 1	C1	0.10	80	75	A
Commercial 2	C2	0.10	80	75	A
Pavement and Rooftops	P	0.05	95	0	A
Gravel Roadways & Shoulders	GR	0.10	50	0	A
Agricultural	AG	0.50	0	85	B
Lawns/Parks/Cemetaries 1	LPC1	0.20	20	80	A
Lawns/Parks/Cemetaries 2	LPC2	0.10	20	80	A
Desert Landscape 1	DL1	0.10	95	30	A
Desert Landscape 2	DL2	0.20	0	30	A
Undeveloped Desert/Rangeland	NDR	0.35	0	10	B
Hillslopes, Sonoran Desert	NHS	0.15	0	15	C
Mountain	NMT	0.25	0	20	D

2. Establish rainfall parameters.

Storm Duration:2-hours. This storm most closely simulates the basis for the Rational Method.

Storm Location:Phoenix metropolitan area. Precipitation data for Sky Harbor Airport was used, as follows:

TABLE 3: Precipitation frequencies for Sky Harbor Airport

duration	p2yr	p5yr	p10yr	p25yr	p50yr	p100yr	p500yr
5 MIN	0.31	0.43	0.50	0.61	0.69	0.78	0.97
10 MIN	0.47	0.65	0.77	0.93	1.06	1.19	1.48
15 MIN	0.57	0.81	0.97	1.19	1.36	1.52	1.91
30 MIN	0.75	1.08	1.30	1.60	1.84	2.07	2.60
1 HOUR	0.92	1.34	1.62	2.00	2.29	2.58	3.26
2 HOUR	1.00	1.47	1.78	2.21	2.53	2.86	3.61
3 HOUR	1.06	1.56	1.89	2.35	2.70	3.04	3.84
6 HOUR	1.17	1.73	2.10	2.61	3.00	3.39	4.29
12 HOUR	1.28	1.91	2.33	2.90	3.34	3.77	4.78
24 HOUR	1.39	2.09	2.55	3.18	3.67	4.15	5.26

Rainfall Distribution: Maricopa County 2-Hour Retention Curve

Storm Frequencies: 2-, 5-, 10-, 25-, 50- and 100-year

Aerial Reduction: None

3. HEC-1 Model preparation. A modified version of MCUHP1, written by Tom Loomis, was used to create eighteen HEC-1 models, 1 for each of the 3 sub-basin areas, and 1 for each of the 6 selected storm frequencies. Each HEC-1 model contains 432 sub-basins including 4 soil textures, 6 slopes, and 18 land uses. The HEC-1 input and output files are on the CD-ROM in Appendix A. The nomenclature for naming each sub-basin is shown in Table 4.

4. Rational Method base condition peak discharges. The base condition peak discharges were calculated using the FCDMC Rational.exe computer program. A C coefficient of 1.0 was used, and peak discharges were calculated for the study slopes and for each surface roughness type from Tables 3.1 and 5.1 of the Hydrology Manual (A, B, C, and D). The results are documented in Appendix C and summarized in Table 3. The computer program applies a frequency factor to the results for the 25-, 50-, and 100-year storms. The values shown in Table 5 reflect removal of the frequency factor.

TABLE 4: Nomenclature used for sub-basin names in HEC-1 models

Name	Nomenclature	Descriptions		
aabcdd i.e.: CLS118	aa = Soil Type as follows:	CL	Clay Loam	
		S	Silt	
		L	Loam	
		SL	Sandy Loam	
	b = Subbasin			
	c = Slope Type:	1	0.5%	
		2	1.0%	
		3	2.0%	
		4	5.0%	
		5	7.0%	
		6	10.0%	
	dd = Land Use as follows:	1	VLDR	Very Low Density Residential
		2	LDR	Low Density Residential
		3	MDR	Medium Density Residential
		4	MFR	Multiple Family Residential
		5	I1	Industrial 1
		6	I2	Industrial 2
		7	C1	Commercial 1
		8	C2	Commercial 2
		9	P	Pavement and Rooftops
		10	GR	Gravel Roadways & Shoulders
		11	AG	Agricultural
		12	LPC1	Lawns/Parks/Cemetaries 1
		13	LPC2	Lawns/Parks/Cemetaries 2
		14	DL1	Desert Landscape 1
		15	DL2	Desert Landscape 2
		16	NDR	Undeveloped Desert and Range-land
		17	NHS	Hillslopes, Sonoran Desert
		18	NMT	Mountain

TABLE 5: Peak discharges using the Rational Method with C=1.00

Type A Surface, 80 acre parcel, Tc Length = 1,848 ft						
Recurrence Interval	Slope in %					
	0.5	1	2	5	7	10
2-yr	190.0	215.0	242.0	274.0	288.0	301.0
5-yr	265.0	299.0	334.0	377.0	394.0	409.0
10-yr	317.0	355.0	396.0	452.0	473.0	479.0
25-yr	408.2	456.4	500.0	581.8	566.4	566.4
50-yr	470.0	521.7	574.2	641.7	646.7	646.7
100-yr	543.2	602.4	661.6	731.2	731.2	731.2
Type B Surface, 80 acre parcel, Tc Length = 1,848 ft						
Recurrence Interval	Slope in %					
	0.5	1	2	5	7	10
2-yr	147.0	177.0	201.0	235.0	248.0	261.0
5-yr	208.0	246.0	279.0	325.0	342.0	360.0
10-yr	264.0	295.0	334.0	386.0	406.0	428.0
25-yr	330.9	380.0	429.1	488.2	510.0	531.8
50-yr	390.8	441.7	491.7	560.8	586.7	612.5
100-yr	454.4	512.0	568.8	647.2	675.2	704.8
Type C Surface, 80 acre parcel, Tc Length = 1,848 ft						
Recurrence Interval	Slope in %					
	0.5	1	2	5	7	10
2-yr	98.0	129.0	161.0	195.0	207.0	220.0
5-yr	151.0	187.0	229.0	271.0	288.0	306.0
10-yr	202.0	242.0	275.0	325.0	344.0	363.0
25-yr	267.3	310.9	352.7	417.3	440.9	465.5
50-yr	313.3	363.3	413.3	480.0	505.0	532.5
100-yr	362.4	422.4	479.2	555.2	584.0	614.4
Type D Surface, 80 acre parcel, Tc Length = 1,848 ft						
Recurrence Interval	Slope in %					
	0.5	1	2	5	7	10
2-yr	75.0	104.0	135.0	176.0	187.0	200.0
5-yr	124.0	157.0	194.0	245.0	261.0	278.0
10-yr	172.0	210.0	250.0	294.0	312.0	332.0
25-yr	229.1	276.4	318.2	378.2	401.8	427.3
50-yr	268.3	324.2	372.5	439.2	463.3	490.0
100-yr	311.2	374.4	433.6	509.6	536.0	566.4

5. Calculation of C Coefficients. The peak discharge, Q, computed through use of the Rational equation, is linearly proportional to the runoff coefficient, C. Therefore, the peak discharges shown in Table 5 were used as follows:

$$C \equiv \frac{Q_{HEC-1}}{Q_{Rat}} \qquad \textit{(Eqn 1)}$$

where: $Q_{HEC\text{-}1}$ is the discharge calculated using HEC-1; and

Q_{Rat} is the discharge calculated using the Rational.exe computer program, corresponding to the slope and T_c characteristics used for computation of $Q_{HEC\text{-}1}$.

A value of C was computed for each corresponding $Q_{HEC\text{-}1}$.

4.0 RESULTS FOR PEAK DISCHARGE ESTIMATION

The results for the 10-, 80- and 160-acre sub-basin models were compared. The results for the 80-acre sub-basin models were selected for use because they are a reasonably conservative average for the three sub-basin sizes, although the differences between the results for different sub-basin areas is not extreme. An example using the results for the VLDR land use is shown in Figure 2. The 80-acre sub-basin results, including HEC-1 peak discharges, computed C values, and the ratio of HEC-1 discharge to the HEC-1 100-year discharge for the 2-, 5-, 10-, 25-, and 50-year storms, are shown in detail in Appendix D. The C value results for a watershed slope of 0.5% are summarized in Tables 6 through 9, and shown graphically in Figures 3 through 8. Figures 3 through 8 are difficult to read, but are shown to provide visual a sense for the relationships of C values between the various land uses. and soil types These figures are not recommended for inclusion in the Hydrology Manual. The LPC1 and LPC2 land uses were originally intended to provide guidance for slopes less than and greater than 5%. The LPC2 land use will probably be changed to a grass-only land use. The other slope-specific land uses (NDR, NHS and NMT) will require special consideration in assigning final C values.

There were several items of concern noted and addressed when preparing the following tables and figures. Most notable is the problem of the limitations of the Clark unit hydrograph T_c equation. The equation is not valid when the computed Tc is close to or greater than the duration of rainfall excess. A check was made in this regard, and the results summarized in Appendix B. The sub-basins where Tc is greater than 90% of the duration of rainfall excess are identified in Appendix B. Those sub-basins are also identified with yellow shading in Tables 6 through 9, and light grey shading in the tables of Appendix D. Many of the peak discharges for the natural land uses (NDR, NHS and NMT) were suspect for the 2-year storm for silt, loam and sandy loam soils. Those peak discharges were adjusted using ratios derived from the DL2 land use. The adjusted peak discharges are highlighted using a darker grey shading in the tables of Appendix D.

FIGURE 2: Comparison of results for VLDR land use by sub-basin size

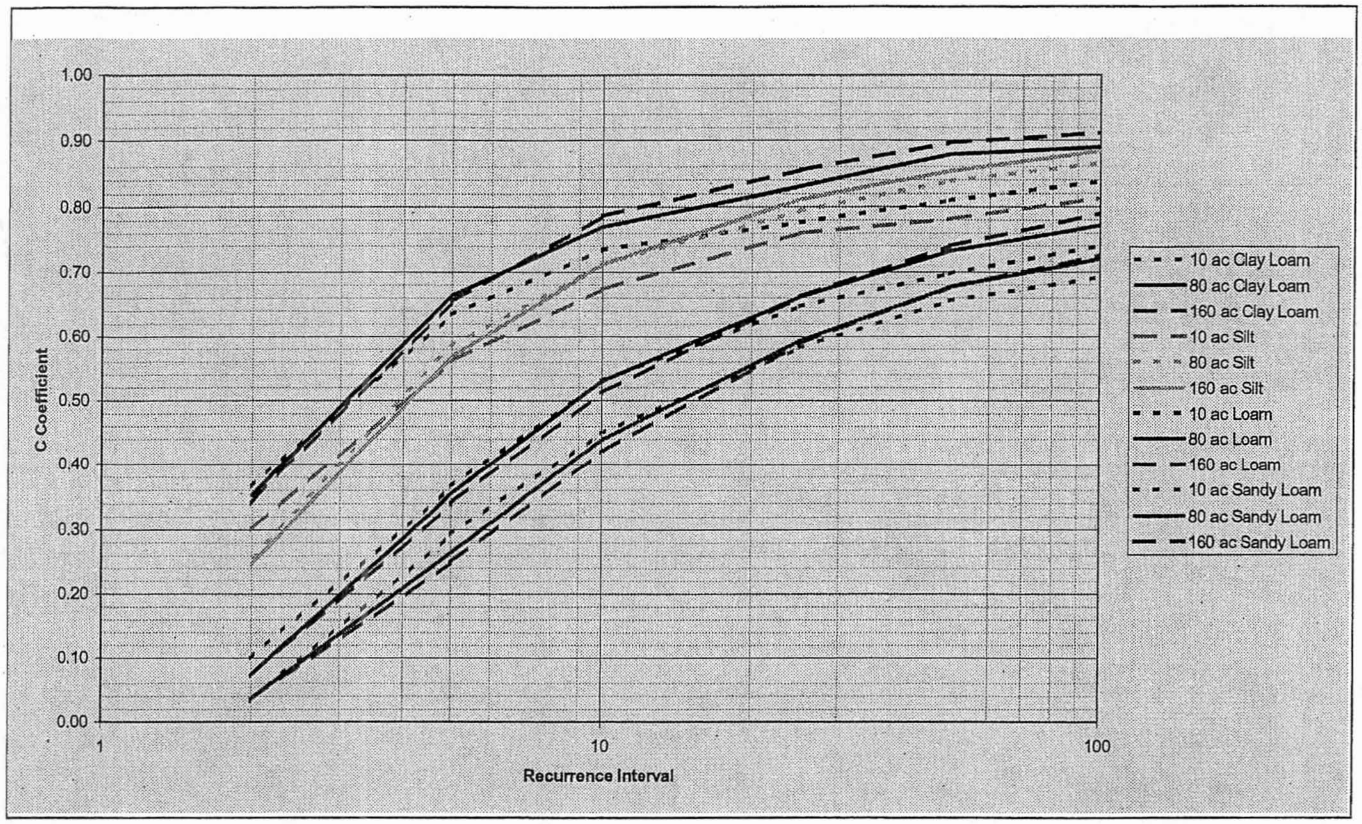


TABLE 6: Rational Method C Coefficients for Clay Soils on a 0.5% Slope

Rational Method C Coefficients, 0.5% T_c Slope Clay, Silty Clay, Sandy Clay, Silty Clay Loam, Clay Loam, and Sandy Clay Loam Soils							
Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.35	0.66	0.77	0.83	0.88	0.89
Low Density Residential	LDR	0.37	0.67	0.77	0.83	0.88	0.89
Medium Density Residential	MDR	0.48	0.72	0.81	0.86	0.90	0.91
Multiple Family Residential	MFR	0.54	0.76	0.84	0.87	0.91	0.92
Industrial 1	I1	0.62	0.80	0.87	0.89	0.92	0.93
Industrial 2	I2	0.62	0.80	0.87	0.89	0.92	0.93
Commercial 1	C1	0.70	0.84	0.91	0.92	0.94	0.94
Commercial 2	C2	0.70	0.84	0.91	0.92	0.94	0.94
Pavement and Rooftops	P	0.75	0.87	0.92	0.93	0.96	0.95
Gravel Roadways & Shoulders	GR	0.66	0.82	0.88	0.90	0.92	0.94
Agricultural	AG	0.04	0.28	0.42	0.54	0.62	0.66
Lawns/Parks/Cemetaries 1	LPC1	0.46	0.70	0.80	0.85	0.89	0.89
Lawns/Parks/Cemetaries 2	LPC2	0.53	0.74	0.82	0.86	0.89	0.90
Desert Landscape 1	DL1	0.75	0.87	0.92	0.93	0.96	0.95
Desert Landscape 2	DL2	0.44	0.70	0.80	0.85	0.89	0.89
Undeveloped Desert and Rangeland	NDR	0.20	0.54	0.66	0.80	0.84	0.86
Hillslopes, Sonoran Desert	NHS	0.36	0.64	0.70	0.78	0.83	0.87
Mountain	NMT	0.27	0.53	0.60	0.72	0.78	0.83
		HEC-1 T_c is greater than 90% of the duration of rainfall excess.					

TABLE 7: Rational Method C Coefficients for Silty Soils on a 0.5% Slope

Rational Method C Coefficients, 0.5% T_c Slope Silt and Silty Loam Soils							
Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.26	0.58	0.71	0.79	0.84	0.87
Low Density Residential	LDR	0.29	0.59	0.72	0.80	0.84	0.87
Medium Density Residential	MDR	0.41	0.66	0.77	0.83	0.86	0.88
Multiple Family Residential	MFR	0.48	0.71	0.80	0.85	0.89	0.90
Industrial 1	I1	0.56	0.75	0.83	0.87	0.90	0.92
Industrial 2	I2	0.56	0.75	0.83	0.87	0.90	0.92
Commercial 1	C1	0.68	0.83	0.89	0.91	0.93	0.94
Commercial 2	C2	0.68	0.83	0.89	0.91	0.93	0.94
Pavement and Rooftops	P	0.75	0.87	0.92	0.93	0.95	0.95
Gravel Roadways & Shoulders	GR	0.61	0.78	0.86	0.88	0.91	0.92
Agricultural	AG	0.02	0.19	0.33	0.47	0.55	0.61
Lawns/Parks/Cemetaries 1	LPC1	0.35	0.62	0.73	0.80	0.84	0.87
Lawns/Parks/Cemetaries 2	LPC2	0.41	0.66	0.75	0.81	0.85	0.87
Desert Landscape 1	DL1	0.74	0.87	0.92	0.93	0.95	0.95
Desert Landscape 2	DL2	0.33	0.62	0.74	0.81	0.85	0.87
Undeveloped Desert and Rangeland	NDR	0.10	0.36	0.51	0.68	0.75	0.80
Hillslopes, Sonoran Desert	NHS	0.16	0.40	0.51	0.64	0.71	0.77
Mountain	NMT	0.15	0.32	0.42	0.55	0.64	0.71
		HEC-1 T_c is greater than 90% of the duration of rainfall excess.					

TABLE 8: Rational Method C Coefficients for Loam Soils on a 0.5% Slope

Rational Method C Coefficients, 0.5% T_c Slope							
Loam Soils							
Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.07	0.35	0.53	0.66	0.73	0.77
Low Density Residential	LDR	0.12	0.38	0.54	0.67	0.73	0.77
Medium Density Residential	MDR	0.24	0.50	0.63	0.73	0.78	0.82
Multiple Family Residential	MFR	0.35	0.58	0.69	0.77	0.82	0.84
Industrial 1	I1	0.44	0.64	0.74	0.80	0.84	0.87
Industrial 2	I2	0.44	0.64	0.74	0.80	0.84	0.87
Commercial 1	C1	0.62	0.78	0.84	0.87	0.90	0.92
Commercial 2	C2	0.62	0.78	0.84	0.87	0.90	0.92
Pavement and Rooftops	P	0.74	0.86	0.92	0.93	0.95	0.95
Gravel Roadways & Shoulders	GR	0.48	0.70	0.78	0.83	0.87	0.89
Agricultural	AG	0.00	0.05	0.15	0.29	0.39	0.47
Lawns/Parks/Cemetaries 1	LPC1	0.15	0.40	0.54	0.66	0.73	0.77
Lawns/Parks/Cemetaries 2	LPC2	0.19	0.44	0.58	0.68	0.74	0.78
Desert Landscape 1	DL1	0.73	0.86	0.91	0.92	0.95	0.95
Desert Landscape 2	DL2	0.09	0.39	0.56	0.67	0.74	0.78
Undeveloped Desert and Rangeland	NDR	0.09	0.22	0.37	0.56	0.65	0.72
Hillslopes, Sonoran Desert	NHS	0.10	0.25	0.36	0.49	0.59	0.66
Mountain	NMT	0.10	0.19	0.28	0.41	0.50	0.59
		HEC-1 T_c is greater than 90% of the duration of rainfall excess.					

TABLE 9: Rational Method C Coefficients for Sandy Loam Soils on a 0.5% Slope

Rational Method C Coefficients, 0.5% T_c Slope							
Sandy Loam Soils							
Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.04	0.26	0.44	0.59	0.68	0.72
Low Density Residential	LDR	0.08	0.29	0.45	0.60	0.68	0.72
Medium Density Residential	MDR	0.20	0.42	0.56	0.66	0.73	0.77
Multiple Family Residential	MFR	0.31	0.52	0.64	0.73	0.78	0.81
Industrial 1	I1	0.40	0.58	0.69	0.77	0.80	0.83
Industrial 2	I2	0.40	0.58	0.69	0.77	0.80	0.83
Commercial 1	C1	0.60	0.75	0.83	0.86	0.89	0.90
Commercial 2	C2	0.60	0.75	0.83	0.86	0.89	0.90
Pavement and Rooftops	P	0.73	0.86	0.91	0.92	0.95	0.95
Gravel Roadways & Shoulders	GR	0.44	0.65	0.74	0.81	0.84	0.87
Agricultural	AG	0.00	0.02	0.09	0.22	0.31	0.40
Lawns/Parks/Cemetaries 1	LPC1	0.12	0.30	0.44	0.58	0.66	0.71
Lawns/Parks/Cemetaries 2	LPC2	0.14	0.33	0.48	0.60	0.67	0.72
Desert Landscape 1	DL1	0.73	0.86	0.91	0.92	0.95	0.95
Desert Landscape 2	DL2	0.04	0.29	0.46	0.60	0.68	0.73
Undeveloped Desert and Rangeland	NDR	0.04	0.15	0.28	0.47	0.58	0.65
Hillslopes, Sonoran Desert	NHS	0.04	0.17	0.28	0.41	0.50	0.58
Mountain	NMT	0.04	0.14	0.21	0.34	0.42	0.51
		HEC-1 T_c is greater than 90% of the duration of rainfall excess.					

FIGURE 3: Rational Method C coefficients for the 2-year storm, T_C Slope = 0.5%

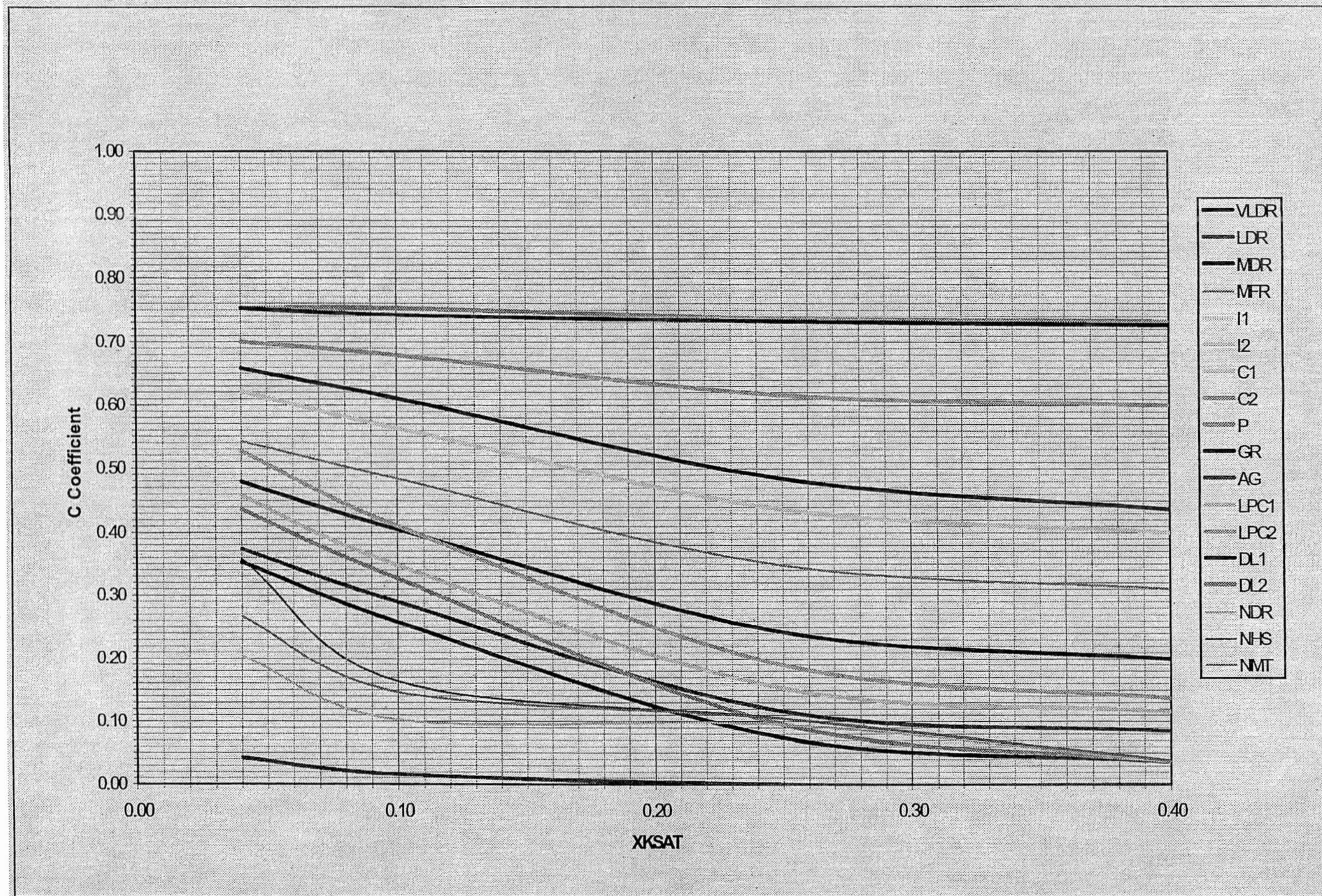


FIGURE 4: Rational Method C coefficients for the 5-year storm, T_C Slope = 0.5%

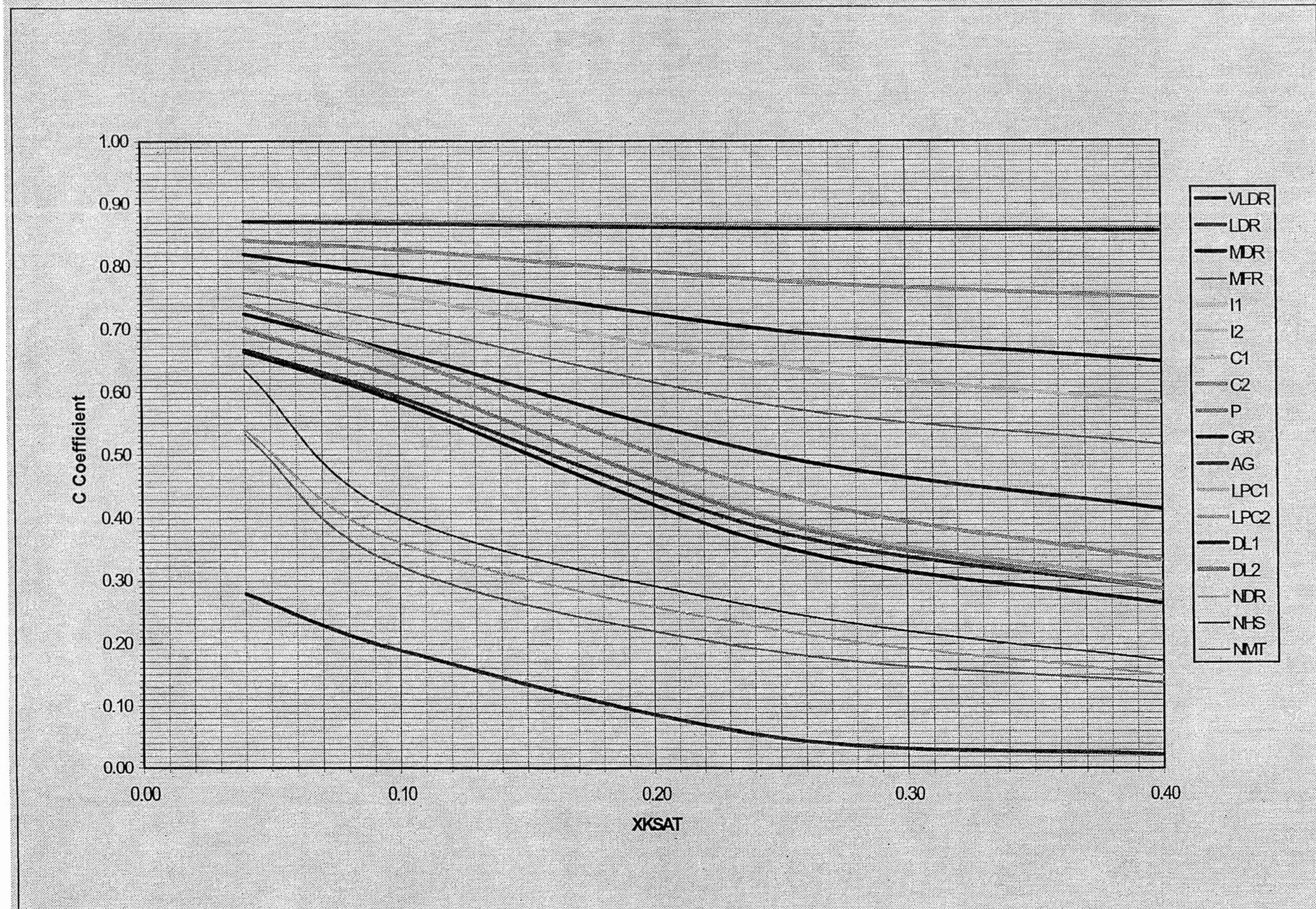


FIGURE 5: Rational Method C coefficients for the 10-year storm, T_C Slope = 0.5%

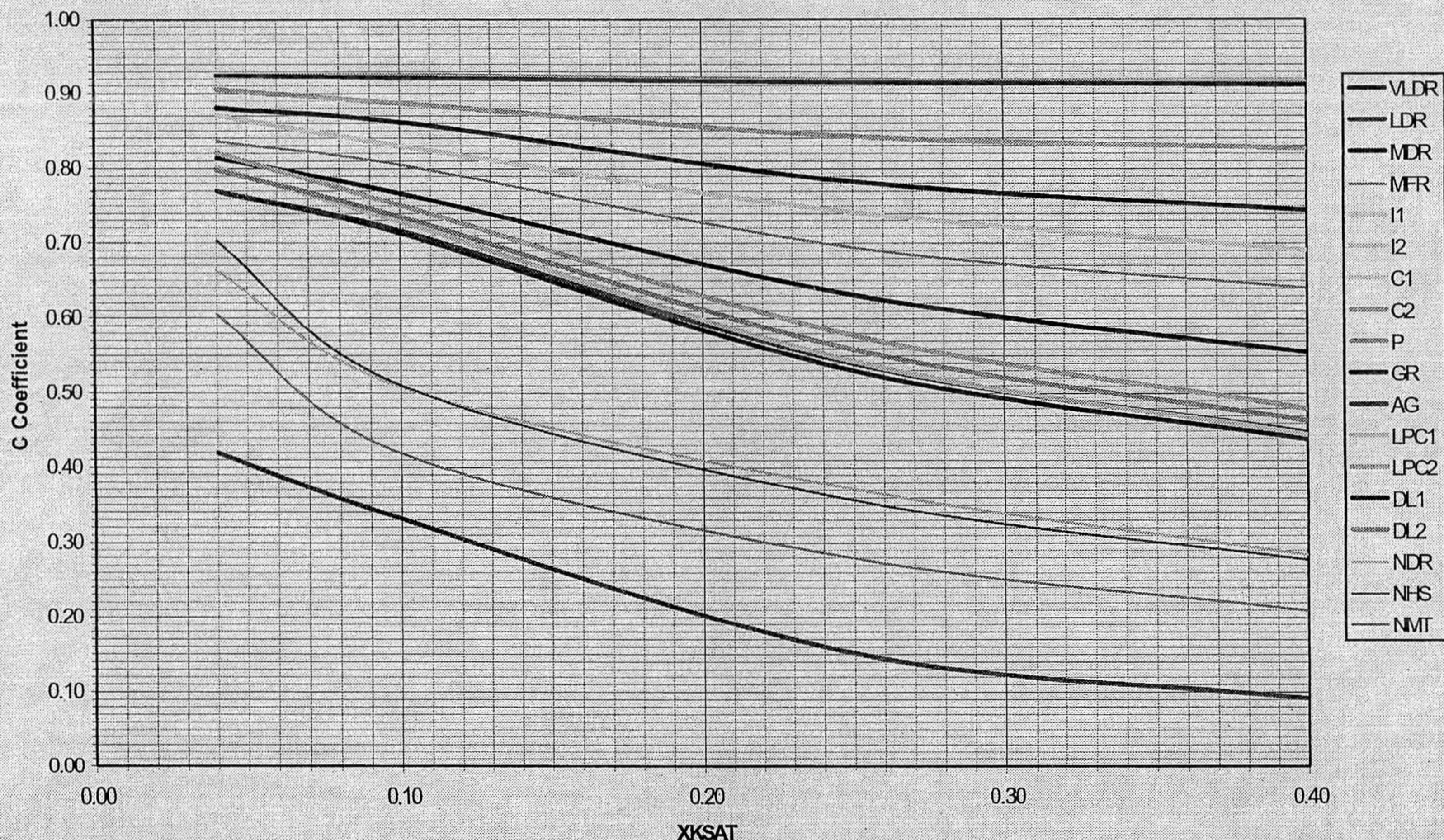


FIGURE 6: Rational Method C coefficients for the 25-year storm, T_C Slope = 0.5%

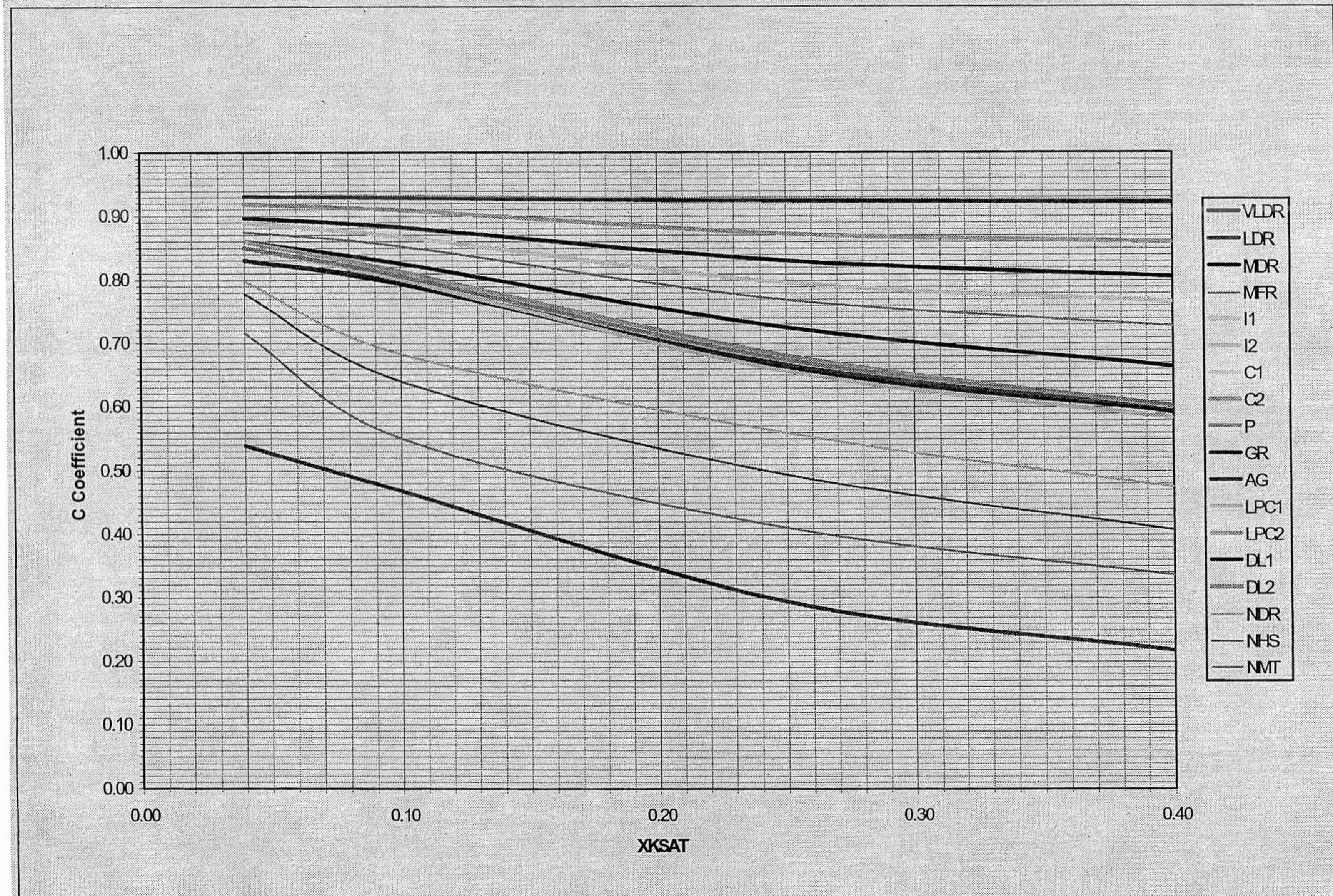


FIGURE 7: Rational Method C coefficients for the 50-year storm, T_C Slope = 0.5%

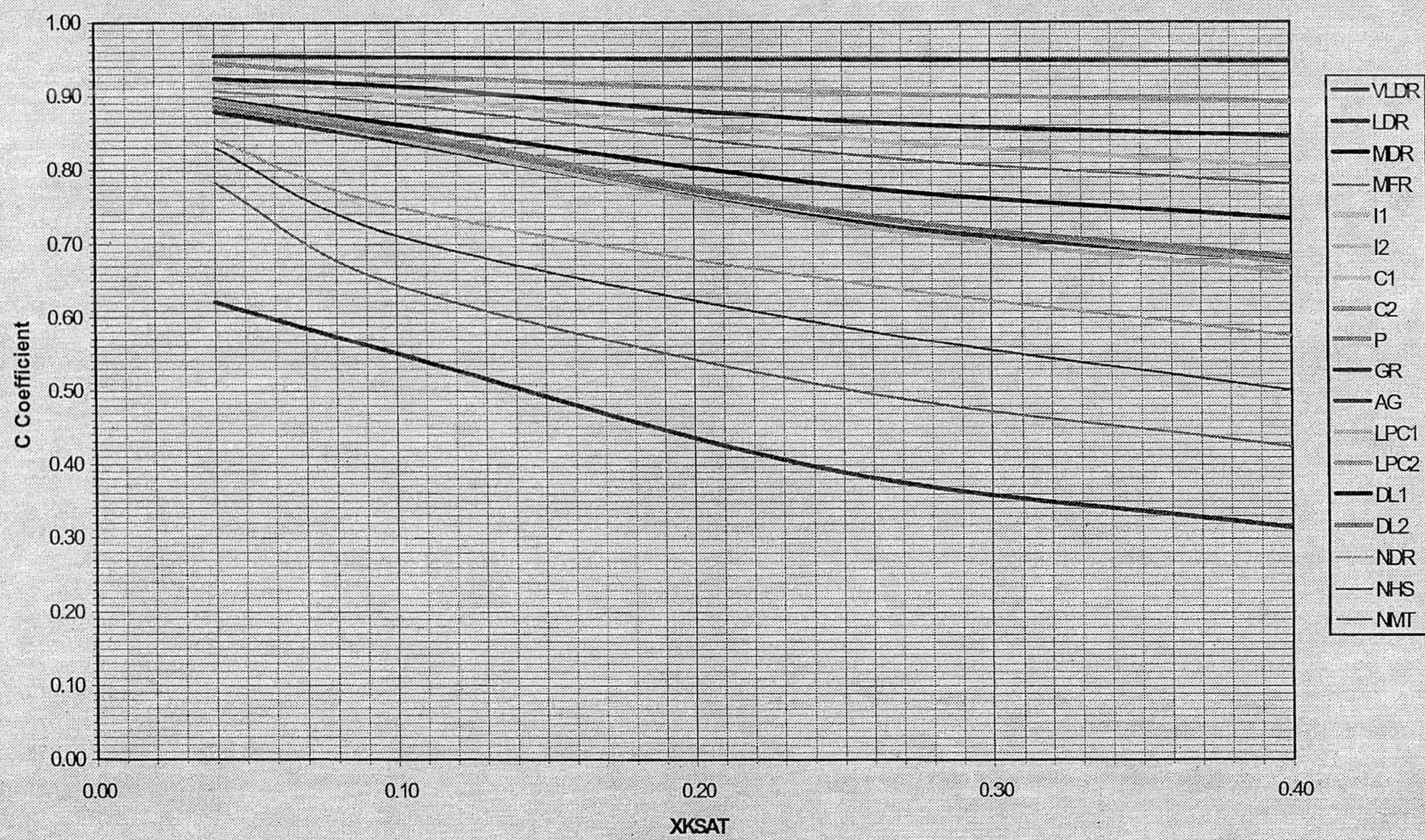
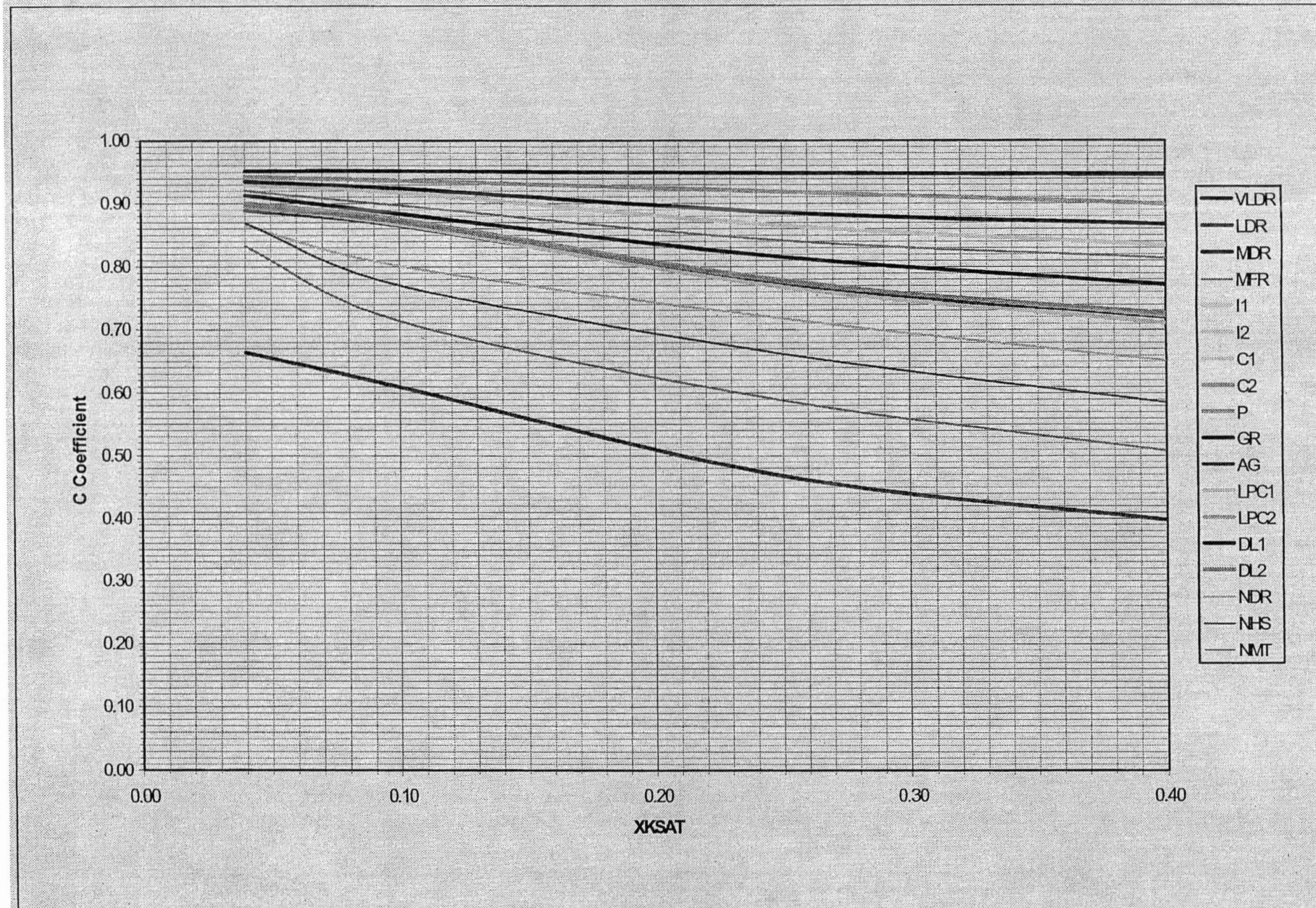


FIGURE 8: Rational Method C coefficients for the 100-year storm, T_C Slope = 0.5%



5.0 SLOPE ADJUSTMENT FOR C COEFFICIENTS

The C values shown in Tables 6 through 9 and Figures 3 through 8 are for use on slopes up to 0.5%. Rather than provide additional tables for each slope considered, it was decided to provide curves of slope adjustment factors that can be applied to the values for 0.5% slopes. The slope adjustment factors were calculated by dividing the C values for slopes greater than 0.5% by the C value for a slope of 0.5%. Refer to Tables E1 through E4 in Appendix E for the calculated slope adjustment factors. The peak discharges estimated using HEC-1 do not increase as much with increasing slope as do those estimated using the Rational equation with a C of 1.0. For this reason, the C values mostly decrease with increasing slope for slopes at or above 5%, which resulted in decreasing slope adjustment factors for most slopes greater than 5%. It was decided to not allow the slope adjustment factors to decrease with increasing slope. Therefore, the slope adjustment factors were adjusted as shown in the bottom four rows of Tables E1 through E4.

After examination of the resulting slope adjustment factors, it was decided to perform exponential regression analyses on the data in order to create eaquations that can be used in the Hydrology Manual and in the Rational.exe computer program. The data was divided into two categories; urban and natural. One equation was created for each recurrence interval storm and for each of the four soil types; clay loam, silt, loam and sandy loam. The values used for the urban regression analyses were the average of the VLDR, LDR, MDR, and MFR land uses as shown in Tables E1 through E4 in Appendix E. The values used for the natural regression analyses were all the data for the NDR, NHS and NMT land uses (surface roughness types B, C and D) as shown in Tables E1 through E4 in Appendix E. The substantiating calculations and plots of the data used in the regression analyses are contained in Appendix E, section E5 for urban and section E6 for natural watersheds. The equation form used in the regression analyses is:

$$SF = a + \frac{b}{S} \qquad \textit{(Eqn 2)}$$

where: SF = calculated slope adjustment factor.

a, b = computed constants for each equation.

S = adjusted watershed slope for use in the T_C equation.

Variables a and b are tabulated in Table 10.

Figures 9 through 12 depict the recommended slope adjustment factors for inclusion in the Hydrology Manual. Figures are used in lieu of tables to account for the full range of possible slopes. The Rational equation is therefore rewritten for use in the Hydrology Manual as follows:

$$Q = SF \times C \times i \times A \qquad \textit{(Eqn 3)}$$

where: Q = discharge in cfs

SF = slope adjustment factor from Figures 9 through 12.

C = C coefficient from Tables 6 through 9 relating the runoff to rainfall.

i = rainfall intensity (inches/hour), lasting for a time, T_C.

A = drainage area (acres)

TABLE 10: Variables for use in the C coefficient slope adjustment equation

Soil	2-year		5-year		10-year		25-year		50-year		100-year	
Type	a	b	a	b	a	b	a	b	a	b	a	b
Urban Land Uses												
Clay Loam	1.25	-0.1340	1.09	-0.0469	1.21	-0.0207	1.03	-0.0172	1.02	-0.0082	1.02	-0.0079
Silt	1.32	-0.1680	1.13	-0.0708	1.21	-0.0269	1.06	-0.0257	1.04	-0.0192	1.02	-0.0109
Loam	1.46	-0.2500	1.27	-0.1420	1.21	-0.0770	1.10	-0.0530	1.07	-0.0376	1.06	-0.0284
Sandy Loam	1.25	-0.1340	1.34	-0.1850	1.21	-0.1130	1.13	-0.0693	1.10	-0.0507	1.08	-0.0418
Natural Land Uses												
Clay Loam	1.39	-0.2220	1.23	-0.1230	1.21	-0.1080	1.13	-0.0660	1.09	-0.0479	1.06	-0.0279
Silt	1.77	-0.4230	1.42	-0.2310	1.21	-0.2130	1.24	-0.1260	1.18	-0.0918	1.11	-0.0553
Loam	1.95	-0.5330	1.61	-0.3370	1.21	-0.3220	1.39	-0.2080	1.30	-0.1560	1.20	-0.1040
Sandy Loam	2.31	-0.7410	1.75	-0.4240	1.21	-0.3590	1.49	-0.2640	1.38	-0.2040	1.28	-0.1450

FIGURE 9: Slope adjustment factors for urban land uses and clay soils

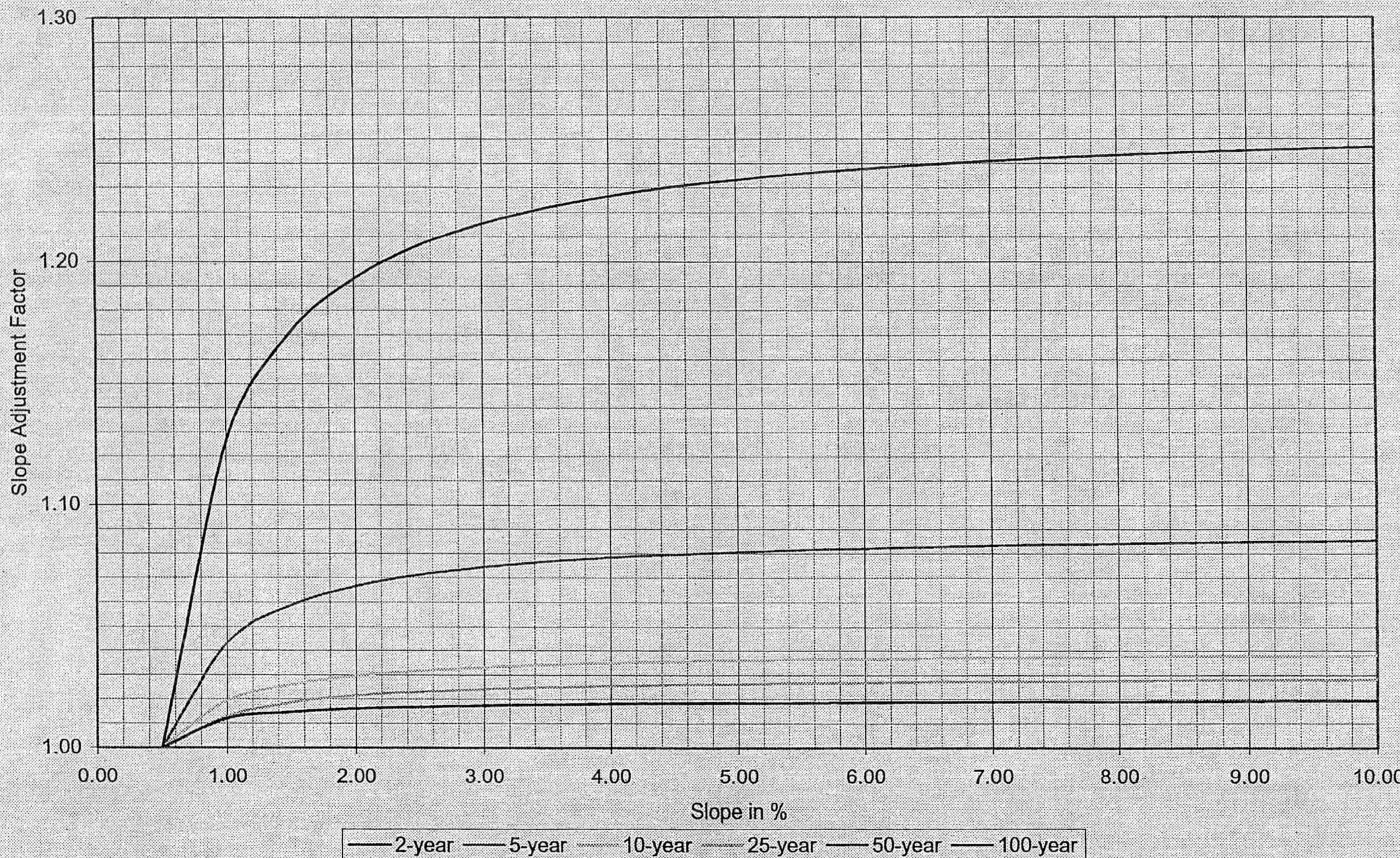


FIGURE 10: Slope adjustment factors for urban land uses and silt soils

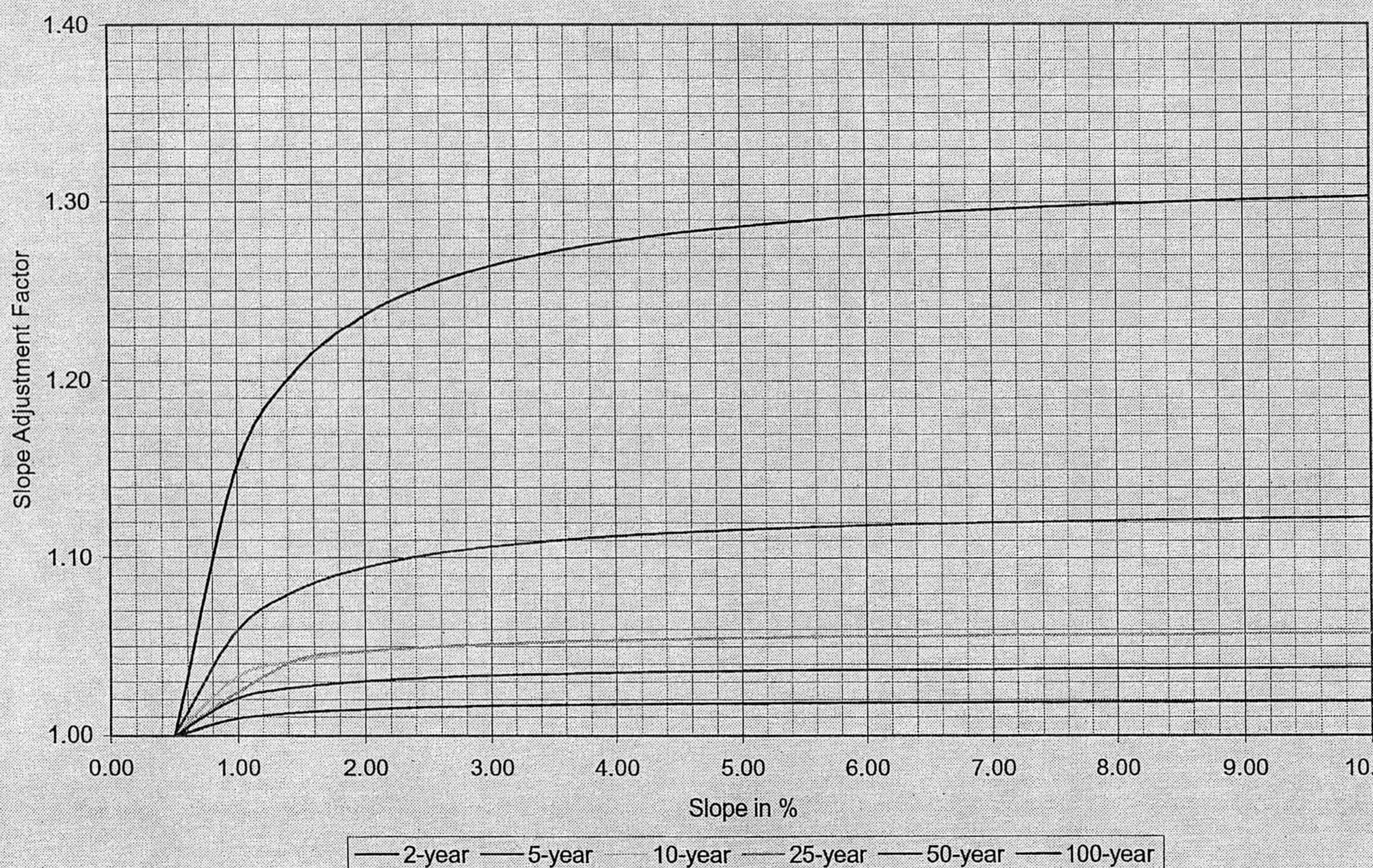


FIGURE 11: Slope adjustment factors for urban land uses and loam soils

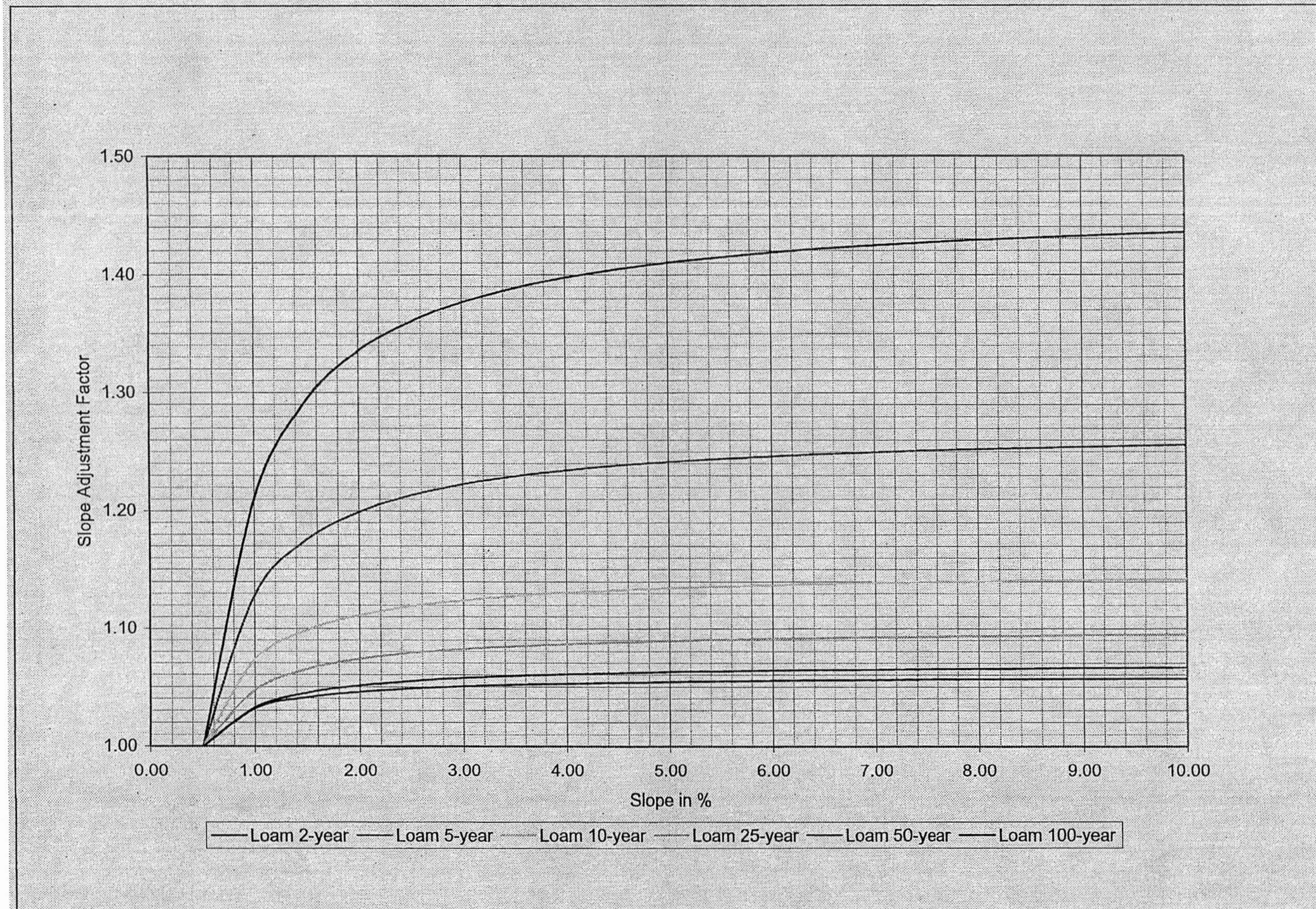


FIGURE 12: Slope adjustment factors for urban land uses and sandy loam soils

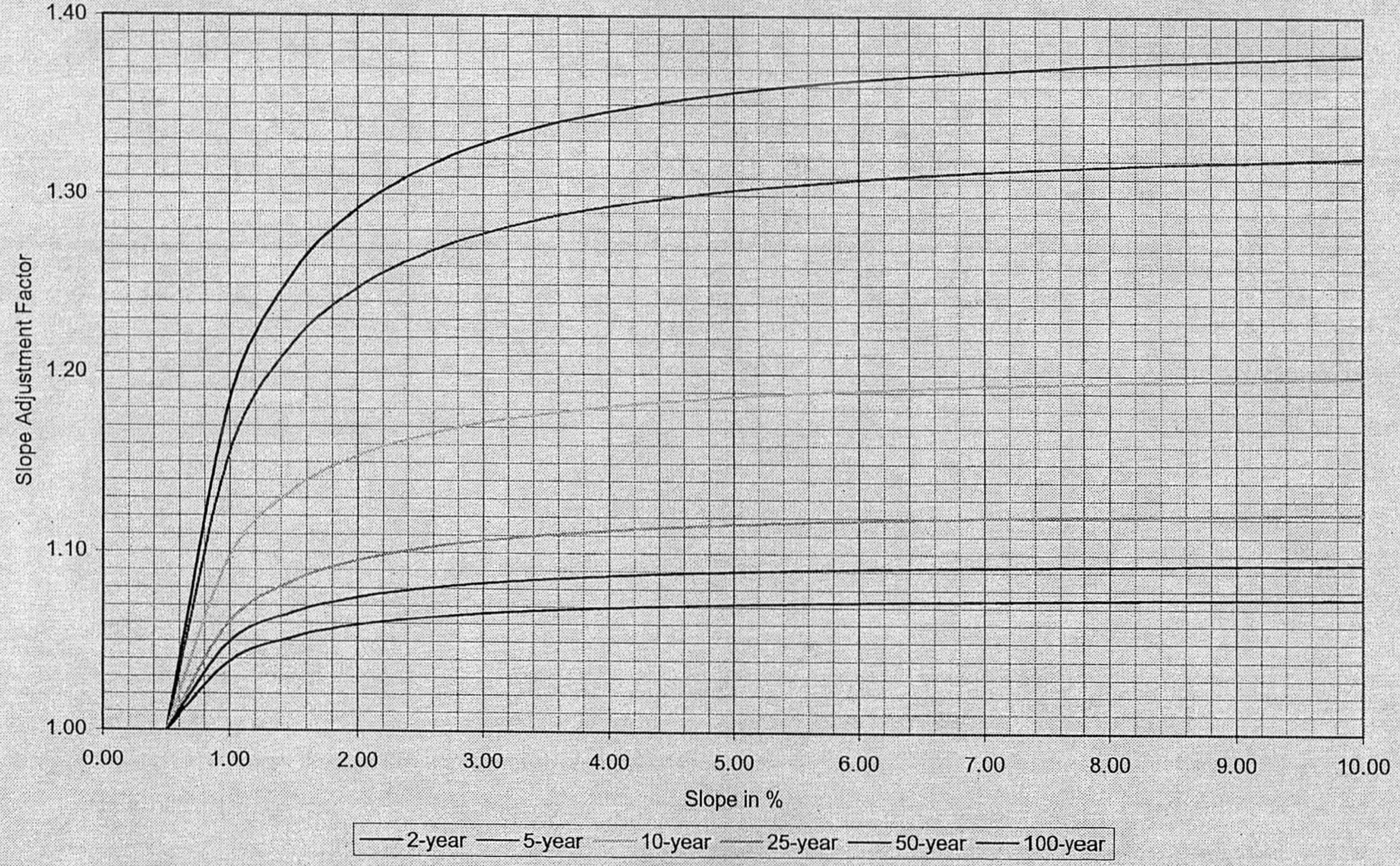


FIGURE 13: Slope adjustment factors for naturaln land uses and clay loam soils

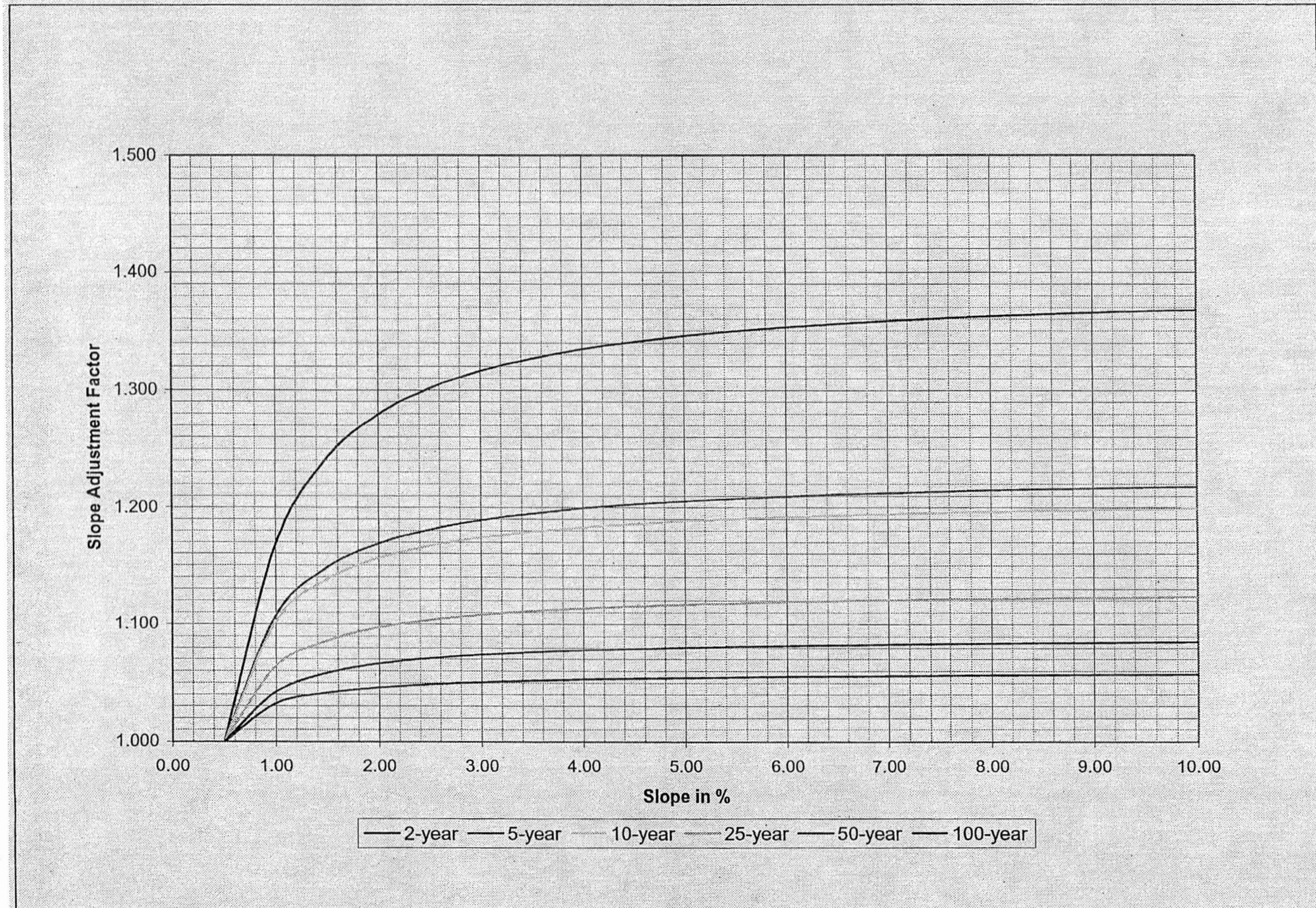


FIGURE 14: Slope adjustment factors for natural land uses and silt soils

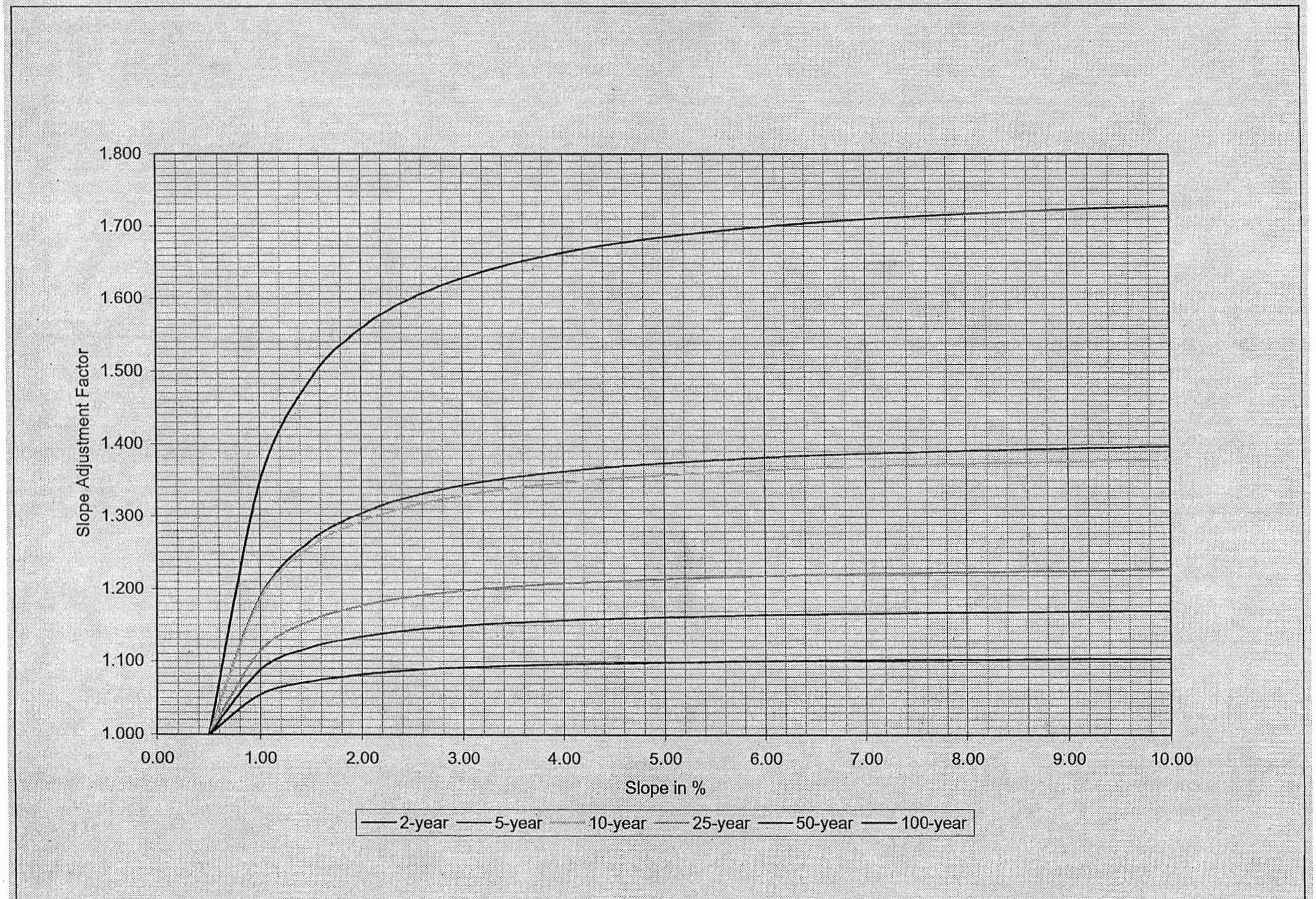


FIGURE 15: Slope adjustment factors for natural land uses and loam soils

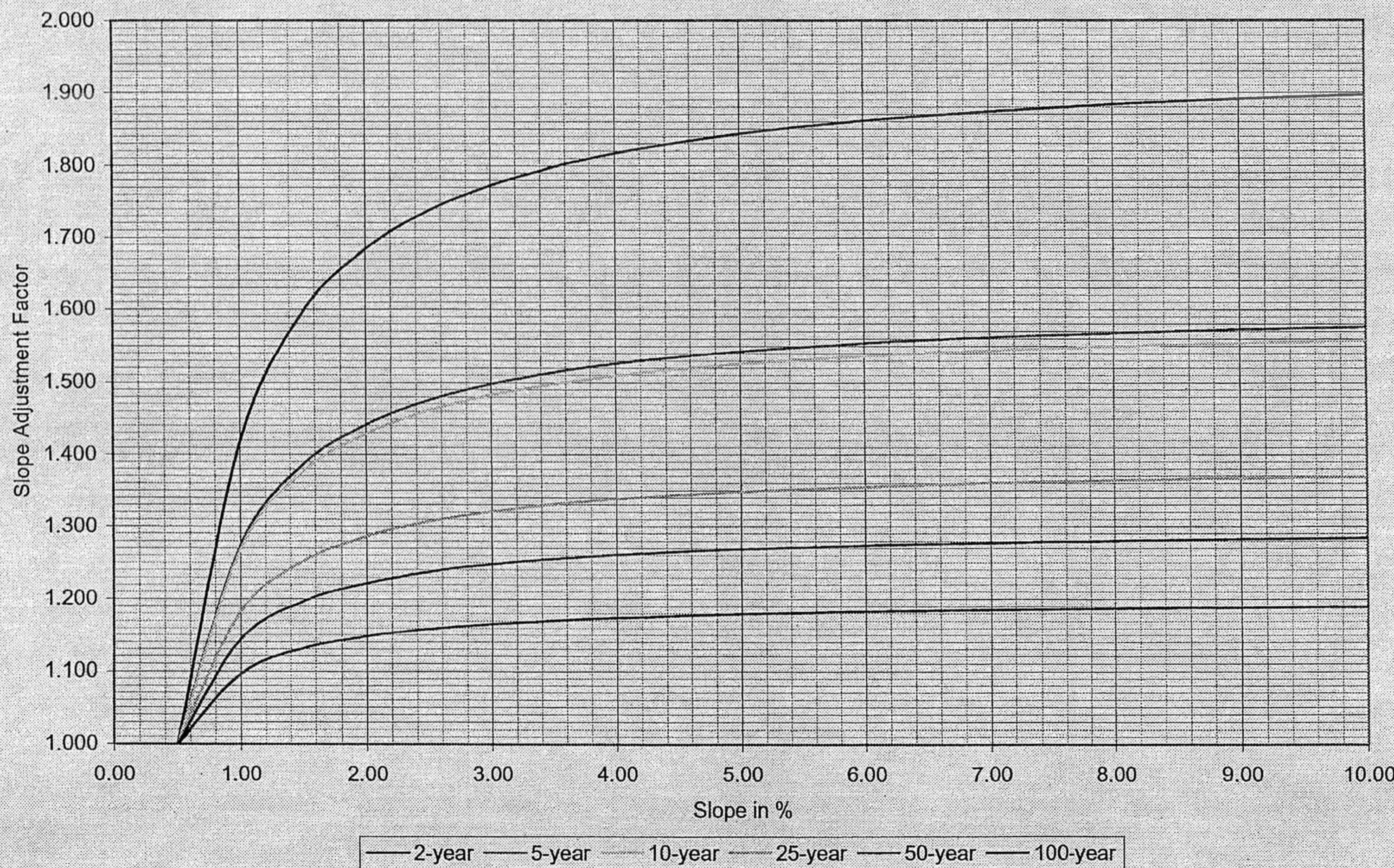
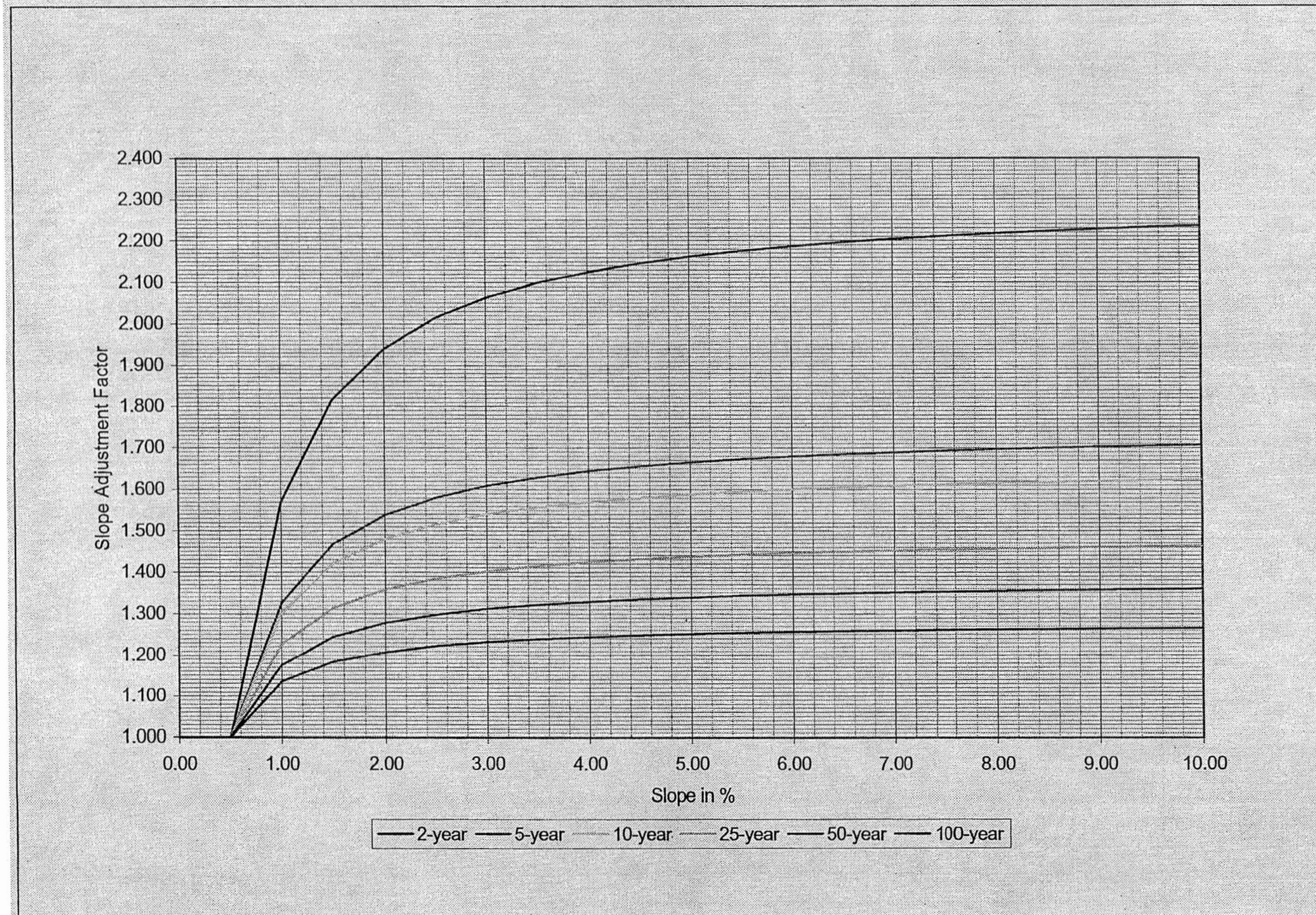


FIGURE 16: Slope adjustment factors for natural land uses and sandy loam soils



6.0 C COEFFICIENT RESULTS FOR RUNOFF VOLUMES

The current Hydrology Manual recommends the use of a modification of the Rational equation for estimation of runoff volumes for retention design purposes. The equation is:

$$V = C \times \frac{P}{12} \times A \qquad \text{(Eqn 4)}$$

where: V = calculated runoff volume in acre-feet.

C = runoff coefficient from Table 3.2 of the Hydrology Manual.

P = rainfall depth in inches.

A = drainage area in acres.

The results of this investigation indicate that the Rational C coefficients is not appropriate to use for this purpose. The value of C includes the affects of hydrologic factors other than rainfall losses. For this reason, the study results were used to estimate appropriate runoff coefficients for the purpose of estimating runoff volumes from 2-hour duration storms. This was accomplished by extracting the rainfall excess values for each sub-basin operation from the HEC-1 output files and dividing by the total storm rainfall. The resulting value is a valid runoff coefficient, designated C_V. The values of C_V recommended for use in the Hydrology Manual are included in Tables 11 through 14. The new equation is recommended to be the same as Eqn 4, with C changed to C_V.

TABLE 11: Rational Method runoff volume coefficients, C_V for clay loam soils

Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.38	0.55	0.61	0.67	0.71	0.74
Low Density Residential	LDR	0.42	0.58	0.63	0.69	0.72	0.75
Medium Density Residential	MDR	0.55	0.67	0.71	0.76	0.78	0.80
Multiple Family Residential	MFR	0.65	0.74	0.77	0.81	0.83	0.85
Industrial 1	I1	0.74	0.81	0.83	0.86	0.87	0.88
Industrial 2	I2	0.74	0.81	0.83	0.86	0.87	0.88
Commercial 1	C1	0.88	0.91	0.92	0.93	0.94	0.95
Commercial 2	C2	0.88	0.91	0.92	0.93	0.94	0.95
Pavement and Rooftops	P	0.97	0.98	0.98	0.98	0.98	0.99
Gravel Roadways & Shoulders	GR	0.77	0.82	0.85	0.87	0.89	0.90
Agricultural	AG	0.13	0.37	0.46	0.55	0.59	0.63
Lawns/Parks/Cemetaries 1	LPC1	0.49	0.63	0.68	0.72	0.75	0.77
Lawns/Parks/Cemetaries 2	LPC2	0.55	0.66	0.71	0.75	0.77	0.80
Desert Landscape 1	DL1	0.97	0.98	0.98	0.98	0.98	0.99
Desert Landscape 2	DL2	0.43	0.58	0.63	0.69	0.72	0.75
Undeveloped Desert and Rangeland	NDR	0.27	0.47	0.54	0.61	0.65	0.69
Hillslopes, Sonoran Desert	NHS	0.43	0.57	0.63	0.68	0.72	0.75
Mountain	NMT	0.35	0.52	0.59	0.65	0.69	0.72

TABLE 12: Rational Method runoff volume coefficients, C_V for silt soils

Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.29	0.47	0.54	0.61	0.65	0.68
Low Density Residential	LDR	0.34	0.50	0.57	0.64	0.67	0.70
Medium Density Residential	MDR	0.48	0.61	0.66	0.71	0.74	0.76
Multiple Family Residential	MFR	0.59	0.69	0.73	0.77	0.79	0.81
Industrial 1	I1	0.68	0.75	0.79	0.82	0.83	0.85
Industrial 2	I2	0.68	0.75	0.79	0.82	0.83	0.85
Commercial 1	C1	0.86	0.89	0.90	0.92	0.93	0.93
Commercial 2	C2	0.86	0.89	0.90	0.92	0.93	0.93
Pavement and Rooftops	P	0.97	0.97	0.98	0.98	0.98	0.98
Gravel Roadways & Shoulders	GR	0.71	0.78	0.81	0.84	0.85	0.87
Agricultural	AG	0.05	0.27	0.37	0.47	0.52	0.57
Lawns/Parks/Cemetaries 1	LPC1	0.40	0.53	0.59	0.65	0.69	0.71
Lawns/Parks/Cemetaries 2	LPC2	0.45	0.56	0.62	0.68	0.71	0.73
Desert Landscape 1	DL1	0.97	0.97	0.98	0.98	0.98	0.98
Desert Landscape 2	DL2	0.32	0.49	0.56	0.62	0.66	0.69
Undeveloped Desert and Rangeland	NDR	0.14	0.33	0.42	0.51	0.56	0.60
Hillslopes, Sonoran Desert	NHS	0.26	0.42	0.50	0.57	0.61	0.65
Mountain	NMT	0.20	0.37	0.46	0.54	0.59	0.62

TABLE 13: Rational Method runoff volume coefficients, C_V for loam soils

Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.14	0.30	0.38	0.46	0.51	0.56
Low Density Residential	LDR	0.21	0.35	0.42	0.49	0.54	0.58
Medium Density Residential	MDR	0.37	0.48	0.53	0.59	0.63	0.66
Multiple Family Residential	MFR	0.50	0.59	0.63	0.68	0.71	0.73
Industrial 1	I1	0.60	0.67	0.70	0.73	0.76	0.78
Industrial 2	I2	0.60	0.67	0.70	0.73	0.76	0.78
Commercial 1	C1	0.82	0.85	0.86	0.88	0.89	0.90
Commercial 2	C2	0.82	0.85	0.86	0.88	0.89	0.90
Pavement and Rooftops	P	0.96	0.97	0.97	0.97	0.97	0.98
Gravel Roadways & Shoulders	GR	0.61	0.68	0.72	0.76	0.78	0.80
Agricultural	AG	0.00	0.12	0.20	0.30	0.36	0.41
Lawns/Parks/Cemetaries 1	LPC1	0.26	0.38	0.44	0.50	0.54	0.58
Lawns/Parks/Cemetaries 2	LPC2	0.29	0.40	0.46	0.52	0.56	0.59
Desert Landscape 1	DL1	0.96	0.96	0.97	0.97	0.97	0.98
Desert Landscape 2	DL2	0.14	0.30	0.38	0.46	0.51	0.55
Undeveloped Desert and Rangeland	NDR	0.05	0.23	0.31	0.41	0.46	0.51
Hillslopes, Sonoran Desert	NHS	0.15	0.30	0.38	0.46	0.51	0.55
Mountain	NMT	0.10	0.27	0.35	0.43	0.49	0.53

TABLE 14: Rational Method runoff volume coefficients, C_V for sandy loam soils

Land Use		Recurrence Interval					
Description	Abbrev.	2-year	5-year	10-year	25-year	50-year	100-year
Very Low Density Residential	VLDR	0.10	0.24	0.32	0.40	0.45	0.49
Low Density Residential	LDR	0.17	0.30	0.37	0.44	0.48	0.52
Medium Density Residential	MDR	0.33	0.43	0.49	0.55	0.58	0.61
Multiple Family Residential	MFR	0.47	0.55	0.60	0.64	0.67	0.69
Industrial 1	I1	0.58	0.63	0.67	0.70	0.72	0.74
Industrial 2	I2	0.58	0.63	0.67	0.70	0.72	0.74
Commercial 1	C1	0.81	0.83	0.85	0.87	0.87	0.88
Commercial 2	C2	0.81	0.83	0.85	0.87	0.87	0.88
Pavement and Rooftops	P	0.96	0.96	0.96	0.97	0.97	0.97
Gravel Roadways & Shoulders	GR	0.58	0.65	0.69	0.72	0.75	0.77
Agricultural	AG	0.00	0.07	0.15	0.24	0.30	0.35
Lawns/Parks/Cemetaries 1	LPC1	0.22	0.32	0.38	0.45	0.49	0.52
Lawns/Parks/Cemetaries 2	LPC2	0.24	0.34	0.40	0.46	0.50	0.53
Desert Landscape 1	DL1	0.95	0.96	0.96	0.97	0.97	0.97
Desert Landscape 2	DL2	0.09	0.24	0.31	0.39	0.44	0.49
Undeveloped Desert and Rangeland	NDR	0.01	0.18	0.26	0.35	0.40	0.45
Hillslopes, Sonoran Desert	NHS	0.10	0.24	0.32	0.40	0.44	0.49
Mountain	NMT	0.06	0.21	0.29	0.37	0.42	0.47

7.0 SUMMARY AND CONCLUSIONS

7.1 General

The proposed C coefficients for peak discharge estimation are compared with the existing values from the current Hydrology Manual in Table 15. The proposed C_V coefficients for runoff volume estimation are compared with the existing C values from the current Hydrology Manual in Table 16. The following description applies to both Tables 15 and 16. The existing land use categories from the current Hydrology Manual are in column 1, and the corresponding existing range of C values are in columns 3, 5, 7 and 9 for the 2-10, 25-, 50- and 100-year storms. The nearest corresponding land use category abbreviation is shown in column 2. Note that no proposed values are shown for the Light Industrial, Neighborhood Business, Lawn, and Playground categories. No corresponding values were calculated for these land uses in this study. The mountain natural land use is not shown because the intent of the table is to compare values that are appropriate for relatively flat watersheds. The proposed range of C values for the various disign storms are shown in columns 4, 6, 8 and 10. The proposed ranges of C values are the lowest and highest values for the four soil types for a 0.5% slope. The current manual provides no guidance for selection of a value in the listed range. The proposed revision would provide a specific value for each situation based on surface and land use characteristics, soil type, recurrence interval, and watershed slope (except for C_V, Table 16). The only other major parameter that affects C is the length, L, used in the Tc equation. A short estimate of L was used for estimation of the base Rational equation peak discharges (C=1) and in the HEC-1 models. The C values in Table 15 calculated using this assumption should be conservative for most small watersheds.

7.2 Effects of the Proposed Method on Peak Discharge Estimation

The proposed C values that are significantly higher than existing, particularly at the low end of the range (loam and sandy loam soils) are highlighted in yellow. The proposed values will produce higher design peak discharges for most residential urban landuses for the less frequent storms, assuming flat slopes. They will produce comparable, or lower, design peak discharges for the more frequent storms. In the steeper foothill areas, application of the slope adjustment factor will produce even higher design peak discharges for the less frequent storms, with a corresponding effect on the more frequent storms. Use of the existing C value table has typically been producing lower peak discharges than would be estimated using the Maricopa County unit hydrograph method, particularly for the less frequent design storms and for areas where silts and clays are the dominate soil type. The existing method appears to produce adequate results for industrial and business land uses for all design storm frequencies considered.

7.3 Effects of the Proposed Method on Volume Estimation

The proposed C_V values that are significantly higher than existing, particularly at the low end of the range (loam and sandy loam soils) are highlighted in yellow. The proposed values will produce higher design retention volumes for most urban landuses, except for Industrial, Business and Multi-unit land uses, for the less frequent storms, assuming flat slopes. They will produce comparable, or lower, design peak discharges for the more frequent storms. Use of the existing C value tablefor retention volume estimation has typically been producing lower design volumes than would be estimated using the Maricopa County Green & Ampt method, particularly for the 100-year design storm and for areas where silts and clays are the dominate soil type. The existing method appears to produce adequate results for industrial, business and apartment land uses for all design storm frequencies considered.

TABLE 15: Comparison of existing C values with proposed C values

Hydrology Manual Land Use Description	Corresponding Proposed LU	2-10 Year		25 Year		50 Year		100 Year	
		Exist	Proposed	Exist	Proposed	Exist	Proposed	Exist	Proposed
Streets and Roads									
Paved Roads	P	0.75-0.85	0.73-0.92	0.83-0.94	0.92-0.93	0.83-0.94	0.95-0.96	0.94-0.95	0.95-0.95
Gravel Roadways & Shoulders	GR	0.60-0.70	0.44-0.88	0.66-0.77	0.81-0.90	0.66-0.77	0.84-0.87	0.75-0.88	0.87-0.94
Industrial Areas									
Heavy	I2	0.70-0.80	0.40-0.87	0.77-0.88	0.77-0.92	0.77-0.88	0.80-0.92	0.88-0.95	0.83-0.93
Light	I1	0.60-0.70	---	0.66-0.77	---	0.66-0.77	---	0.75-0.88	---
Business Areas									
Downtown	C2	0.75-0.85	0.60-0.91	0.83-0.94	0.86-0.92	0.83-0.94	0.89-0.94	0.94-0.95	0.90-0.94
Neighborhood	C1	0.55-0.65	---	0.61-0.72	---	0.61-0.72	---	0.69-0.81	---
Residential Areas									
Lawn - Flat	n/a	0.10-0.25	---	0.11-0.28	---	0.11-0.28	---	0.13-0.31	---
Lawns - Steep	n/a	0.25-0.40	---	0.28-0.44	---	0.28-0.44	---	0.31-0.50	---
Suburban	LDR	0.30-0.40	0.08-0.77	0.33-0.44	0.60-0.83	0.33-0.44	0.68-0.88	0.38-0.50	0.72-0.89
Single Family	MDR	0.45-0.55	0.20-0.81	0.50-0.61	0.66-0.86	0.50-0.61	0.73-0.90	0.56-0.69	0.77-0.91
Multi-Unit	MFR	0.50-0.60	0.31-0.84	0.55-0.66	0.73-0.87	0.55-0.66	0.78-0.91	0.63-0.75	0.81-0.92
Apartments	MFR	0.60-0.70	0.31-0.84	0.66-0.77	0.73-0.87	0.66-0.77	0.78-0.91	0.75-0.88	0.81-0.92
Parks/Cemetaries	LPC1	0.10-0.25	0.12-0.80	0.11-0.28	0.58-0.85	0.11-0.28	0.66-0.89	0.13-0.31	0.71-0.89
Playgrounds	n/a	0.40-0.50	---	0.44-0.55	---	0.44-0.55	---	0.50-0.63	---
Agricultural Areas	AG	0.10-0.20	0.00-0.42	0.11-0.22	0.22-0.54	0.11-0.22	0.31-0.62	0.13-0.25	0.40-0.66
Bare Ground	NDR	0.20-0.30	0.04-0.66	0.22-0.33	0.47-0.80	0.22-0.33	0.58-0.84	0.25-0.38	0.65-0.86
Undeveloped Desert	NHS	0.30-0.40	0.04-0.70	0.33-0.44	0.41-0.78	0.33-0.44	0.50-0.83	0.38-0.50	0.58-0.87

TABLE 16: Comparison of existing C values with proposed C_V values

Hydrology Manual Land Use Description	Corresponding Proposed LU	2-10 Year		25 Year		50 Year		100 Year	
		Exist	Proposed	Exist	Proposed	Exist	Proposed	Exist	Proposed
Streets and Roads									
Paved Roads	P	0.75-0.85	0.96-0.98	0.83-0.94	0.97-0.98	0.83-0.94	0.97-0.97	0.94-0.95	0.97-0.99
Gravel Roadways & Shoulders	GR	0.60-0.70	0.58-0.85	0.66-0.77	0.72-0.87	0.66-0.77	0.75-0.89	0.75-0.88	0.77-0.90
Industrial Areas									
Heavy	I2	0.70-0.80	0.58-0.83	0.77-0.88	0.70-0.86	0.77-0.88	0.72-0.87	0.88-0.95	0.74-0.88
Light	I1	0.60-0.70	---	0.66-0.77	---	0.66-0.77	---	0.75-0.88	---
Business Areas									
Downtown	C2	0.75-0.85	0.81-0.92	0.83-0.94	0.87-0.93	0.83-0.94	0.87-0.94	0.94-0.95	0.88-0.95
Neighborhood	C1	0.55-0.65	---	0.61-0.72	---	0.61-0.72	---	0.69-0.81	---
Residential Areas									
Lawn - Flat	n/a	0.10-0.25	---	0.11-0.28	---	0.11-0.28	---	0.13-0.31	---
Lawns - Steep	n/a	0.25-0.40	---	0.28-0.44	---	0.28-0.44	---	0.31-0.50	---
Suburban	LDR	0.30-0.40	0.17-0.63	0.33-0.44	0.44-0.69	0.33-0.44	0.48-0.72	0.38-0.50	0.52-0.75
Single Family	MDR	0.45-0.55	0.33-0.71	0.50-0.61	0.55-0.76	0.50-0.61	0.58-0.78	0.56-0.69	0.61-0.80
Multi-Unit	MFR	0.50-0.60	0.47-0.77	0.55-0.66	0.64-0.81	0.55-0.66	0.67-0.83	0.63-0.75	0.69-0.85
Apartments	MFR	0.60-0.70	0.47-0.77	0.66-0.77	0.64-0.81	0.66-0.77	0.67-0.83	0.75-0.88	0.69-0.85
Parks/Cemetaries	LPC1	0.10-0.25	0.22-0.68	0.11-0.28	0.45-0.72	0.11-0.28	0.49-0.75	0.13-0.31	0.52-0.77
Playgrounds	n/a	0.40-0.50	---	0.44-0.55	---	0.44-0.55	---	0.50-0.63	---
Agricultural Areas	AG	0.10-0.20	0.00-0.46	0.11-0.22	0.24-0.55	0.11-0.22	0.30-0.59	0.13-0.25	0.35-0.63
Bare Ground	NDR	0.20-0.30	0.01-0.54	0.22-0.33	0.35-0.61	0.22-0.33	0.40-0.65	0.25-0.38	0.45-0.69
Undeveloped Desert	NHS	0.30-0.40	0.10-0.63	0.33-0.44	0.40-0.68	0.33-0.44	0.44-0.72	0.38-0.50	0.49-0.75

7.4 Recommended Refinements Before Revising the Hydrology Manual

This study is based on use of the current RTIMP values specified in the Hydrology Manual for urban land uses and the unit hydrograph method. The land use categories are simplified to include similar zoning lassifications, which is good, but zoning requirements for many of the various municipalities have changed in the last 10-years. The next step recommended is to perform a study of the current zoning requirements for all the municipalities in Maricopa County. This study should compile all the various zoning classifications and the maximum buildable area under each classification. The study should further identify the typical hydraulically-connected impervious area added under each classification for driveways and porches. Any maximum clearing limitsfor each classification should be documented. The land use table for the unit hydrograph method should then be revised as necessary. It is suggested that the following categories be considered for addition to the table:

1. Grass, well-maintained and irrigated (the LPC2 slot could be replaced with this, RTIMP=0)

2. Industrial, heavy (the I1 category in this study)

3. Industrial, light (the I2 category in this study)

4. Commercial, neighborhood (the C2 category in this study)

5. Commercial, downtown (the C1 category in this study)

No distinction was made between I1 and I2 and C1 and C2 for this study in terms of RTIMP, because no data was immediately available. The zoning research will dictate the need for two categories for industrial and commercial. Julie Cox is currently preparing the necessary modeling to develop C values for grass, as an off-shoot of this study.

The tables need to be revised for the natural land uses to address slope issues. The NDR land use is normally associated with slopes less than 5%. The NHS land use is normally associated with slopes between 5% and 10%. The NMT land use is normally associated with slopes greater than 10%. The tables and figures need to be adjusted accordingly. No slope adjustement is necessary for the agricultural land use. It should be made clear in the Hydrology Manual that the values for the agricultural land use are slopes of 0.5% or less.

8.0 REFERENCES

American Society of Civil Engineers, *Design and Construction of Sanitary and Storm Sewers*, prepared by a Joint Committee of the American Society of Civil Engineers and the Water Pollution Control Federation, ASCE Manual on Engineering Practice No. 37, page 51, 1969.

Chow, Maidment and Mays, *Applied Hydrology*, 1988.

Flood Control District of Maricopa County, *Drainage design Manual for Maricopa County, Arizona, Volume I*, Hydrology, 1995.

Maidment, David R., *Handbook of Hydrology*, 1993.

McCuen, Richard H., *Hydrologic Analysis and Design*, 1989.

Ponce, Victor Miguel, *Engineering Hydrology, Principles and Practices*, 1989.

Rossmiller, Ronald L., *The Rational Formula Revisited*, International Symposium on Urban Storm Runoff ,University of Kentucky, July 28-31, 1980.

APPENDIX A HEC-1 Input/Output Files

HEC-1 input and output files on CD-ROM for: 2-, 5-, 10-, 25-, 50-, and 100-year 2-hour storms, and 10-, 80- and 160-acre sub-basin areas

APPENDIX B HEC-1 T_c Check Tables

B1 2-year 2-hour Storm

B2 5-year 2-hour Storm

B3 10-year 2-hour Storm

B4 25-year 2-hour Storm

B5 50-year 2-hour Storm

B6 100-year 2-hour Storm

APPENDIX C Rational Method Calculation Sheets

C1 10-acre Subbasins, L = 660 ft

C2 80-acre Subbasin, L = 1848 ft

C3 160-acre Subbasin, L = 2640 ft

C4 10-acre Subbasin, L = 934.6 ft

APPENDIX D HEC-1 and C Coefficient Summary Tables

D1 Clay Loam Soils

D2 Silt Soils

D3 Loam Soils

D4 Sandy Loam Soils

APPENDIX E C Coefficient Slope Adjustment Calculations

E1 C Coefficient Slope Ratios for Clay Loam Soils

E2 C Coefficient Slope Ratios for Silt Soils

E3 C Coefficient Slope Ratios for Loam Soils

E4 C Coefficient Slope Ratios for Sandy Loam Soils

E5 Slope Adjustment Regression Calculations for Urban Watersheds

E6 Slope Adjustment Regression Calculations for Natural Watersheds

March 8, 2002

Mr. Thomas Loomis
Flood Control District of Maricopa County
2801 W Durango St
Phoenix, AZ 85009-6399

File: 28900042 01/02

Dear Tom:

Re: Drainage Design Manual – Hydraulics

We have reviewed Flood Control's comments pertaining to the Hydraulics Manual and have incorporated the majority of them into the latest version (see attached document). We appreciate the commitment you have made to better this document. The following comments have not been addressed explicitly for the reasons cited below. Comments are in italic font with our response immediately below the comment in regular font. We stand ready to discuss these further with you at your convenience if needed.

	Chapter and Section	**Page**	**Comment**
1.	5.3.2	5-3	*others suggest 2 feet per second* The Debo and Reese citation of 2.5 feet per second minimum velocity for partial flows through a culvert is identified as a guideline. Design criteria for each jurisdictional entity is to be addressed in their own Policies and Standards Manual.
2.	5.3.2	5-10	*Clogging & ineffective area needs to be considered.* Design criteria for each jurisdictional entity is to be addressed in their own Policies and Standards Manual.
3.	5.4.3	5-65	*The minimum d_{50} size shall be 8-inches.* Design criteria for each jurisdictional entity is to be addressed in their own Policies and Standards Manual.

4.	6.4.4	6-26	*grouted riprap,* Lengthy discussions with FCD and COP concluded that rock embedded concrete lined channels were acceptable but grouted rip-rap lined channels were not due to the typically poor construction methods employed…..grouted rip-rap channels simply do not stand up over time and therefore do not perform as originally designed. The concensus at that time was to shy away from endorsing grouted rip-rap.
5.	6.5.3	6-29	*a) for preliminary design purposes only,* *b) The tractive shear stress approach shall be used to confirm the stability of the unlined channel for design purposes (USDOT, FHWA, HEC-11).* Further discussion between FCD and Stantec eliminated the tractive shear stress approach from inclusion into the manual
6.	6.5.3	6-30	*expected during the 100-year event, or a series of annualized events over a 60-year period.* Design criteria for each jurisdictional entity is to be addressed in their own Policies and Standards Manual.
7.	6.6.6	6-57	*I would like to see an example using tractive force to check on unlined channel.* Further discussion between FCD and Stantec eliminated the tractive shear stress approach from inclusion into the manual
8.	7.6.1	7-73	*Has this been discussed with Bing? He is suppose to be developing a program.* Yes, it was agreed that the discussion on side channel spillways would be used as a place holder for revision upon conclusion of Bing's work.

Further questions should be directed to me (602) 438-2200.

Sincerely,

Frank W. Thomas, P.H., P.E.
Project Manager

cc: Ralph Goodall, P.E. (w/out attachment)
City of Phoenix

FWT:cjm

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Meeting Notes

Stantec

FILE: 82000042

Date:	9 April 2002	
Place/Time:	Stantec/11:30 AM	
Next Meeting:		
Attendees:	FCDMC Stantec	Tom Loomis, Amir Motamedi, Joe Rumann George Sabol, Mike Gerlach
Distribution:	Attendees/Absentees	

Item: Multiple Frequency Modeling Procedures

The results of testing that was conducted in regard to the use of ratios as a means of estimating runoff hydrographs for the 10-, 5- and 2-year return periods were presented. The watersheds selected for testing are:

- Agua Fria River Tributary at Youngtown – a gaged, urban watershed with a drainage area of approximately 0.13 sq. miles, relatively long time of concentration flow path and very mild slope.
- Salt River Tributary at South Mountain Park – a gaged, natural watershed with a drainage area of approximately 1.75 sq. miles with steep mountainous slopes.
- Tucson Arroyo at Vine Avenue (Tucson) – a gaged, urban watershed with a drainage area of approximately 8.6 sq. miles.
- Cave Creek above Carefree Highway – a gaged watershed that is predominately undeveloped with a drainage area of approximately 124 sq. miles.
- Hartman Wash at US 60, near Wickenburg – a gaged, natural watershed with a drainage area of approximately 5.5 sq. miles.

HEC-1 models for each test watershed for the 100-, 50-, 25-, 10-, 5- and 2-year return periods were developed in accordance with the current procedures in the Hydrology Manual. Ratios of the 10-, 5-and 2-year peak discharges to the 100-year peak discharge were computed for each watershed. Those ratios were compared to ratios based on USGS LP3 peak discharge estimates for each test watersheds as well as to the average ratios developed from USGS LP3 peak discharge estimates from a sampling of gages throughout the State of Arizona. A summary of the HEC-1 model results and the USGS LP3 peak discharge estimates are provided in the following table. The average ratios developed from the USGS data range from 9 to 12, 21 to 25 and 34 to 37 percent for the 2-, 5- and 10-year return periods, respectively, for natural watersheds and 15, 28 and 40 for the 2-, 5- and 10-year return periods, respectively, for urbanized watersheds.

Reference: Drainage Design Manual

Summary of HEC-1 results compared to USGS data

	Return	USGS		HEC-1	
	Period	Discharge cfs	Ratio %	Discharge cfs	Ratio %
Agua Fria River Tributary at Youngtown[1]	100-Year	190	---	103	---
	50-Year	126	---	81	---
	25-Year	83	---	60	---
	10-Year	48	25.2	39	37.6
	5-Year	31	16.3	26	25.2
	2-Year	16	8.4	13	12.6
Salt River Tributary at South Mountain Park	100-Year	2,260	---	2,868	---
	50-Year	1,500	---	2,349	---
	25-Year	925	---	1,827	---
	10-Year	420	18.6	1,192	41.6
	5-Year	191	8.5	779	27.2
	2-Year	37	1.6	319	11.1
Tucson Arroyo at Vine Avenue in Tucson[2]	100-Year	4,890	---	7,520	---
	50-Year	3,920	---	6,260	---
	25-Year	3,090	---	4,790	---
	10-Year	2,150	44.0	3,180	42.3
	5-Year	1,540	31.5	2,240	29.8
	2-Year	842	17.2	1,130	15.0
Cave Creek above Carefree Highway[3]	100-Year	21,500	---	33,771	---
	50-Year	16,400	---	28,229	---
	25-Year	12,000	---	23,242	---
	10-Year	7,360	34.2	17,132	50.7
	5-Year	4,580	21.3	12.847	38.0
	2-Year	1,780	8.3	6,615	19.6
Hartman Wash at US 60 near Wickenburg[4]	100-Year	7,100	---	4,712	---
	50-Year	4,910	---	3,872	---
	25-Year	3,230	---	2,593	---
	10-Year	1,660	23.4	1,601	34.0
	5-Year	869	12.2	790	16.8
	2-Year	239	3.4	70	1.5

Reference: Drainage Design Manual

Notes:

1. The time of concentration for the 10-, 5- and 2-year return periods is longer than the duration of rainfall excess. The 2-year time of concentration defaulted to 90 minutes.
2. The 2-year time of concentration defaulted to 90 minutes.
3. This is the only multiple basin model of the test cases. Coding the average ratios on the BA record of each subbasin results in discharges of 10,444, 7,063 and 2,165 cfs for the 10-, 5- and 2-year return periods, respectively.
4. The desert/rangeland s-graph was used for this subbasin because the time of concentration for the Clark unit hydrograph defaulted to 90 minutes starting at the 10-year return period.

These results as well as the need for a different approach for modeling of the more frequent events were discussed in consideration of the following points:

- The purpose/intended use of watershed models for the more frequent storm events.
- The limitations of the current modeling techniques that are often encountered for the more frequent events, particularly the assumptions of the design rainfall procedures and the limitations of the Papadakis and Kazan T_c equation.
- What the sensitive parameters were and how those parameters were related to the results.
- The accuracy of the ratio method results in comparison to the current modeling methodology.
- The practicality/reproducibility of the ratio method.

The results of these discussions were that the ratio method should be presented as an alternative to the current modeling methodology. Ratios of 35, 25 and 10 percent were selected for the 10-, 5- and 2-year return periods, respectively.

A discussion will be provided in the manual on the use, suggested applications and potential limitations of using ratios of the 100-year model discharges for the 10-, 5- and 2-year return periods. That will include a general graphic relation of watershed area versus ratio for selected return periods.

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Tom Loomis requested that additional analysis be conducted in regard to potential differences in runoff volume between the current modeling method and ratios as well as potential problems with diversion operations.

The foregoing is considered to be a true and accurate record of all items discussed. If any discrepancies or inconsistencies are noted, please contact the writer immediately. **STANTEC CONSULTING INC.**

Reference: Drainage Design Manual

Mike Gerlach, PE
Project Engineer
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Stantec

30 April 2002
File: 82000042

Mr. Thomas R. Loomis, P.E., R.L.S.
Flood Control District of Maricopa County
2901 W Durango St
Phoenix, AZ 85009

Reference: Hydrology Manual Status

Dear Tom:

The following is a detailed listing, by chapter, of the changes to the Hydrology Manual.

Chapter 1 - The Executive Summary has been inserted into this chapter as Section 1.1., Overview. A brief discussion of the contents of each chapter has been added. At this time, only minor formatting issues need to be addressed before final conversion to FrameMaker.

Chapter 2 - The table identifying the design rainfall criteria has been removed for inclusion in the future Policies and Standards Manual for Maricopa County. The isopluvial maps have been updated to include corporate boundaries and major roadway alignments. The isopluvial lines were extended slightly beyond the County boundary. These figures have been moved to an appendix and are plotted in color. The aerial reduction curve data (both the 6- and 24-hour duration) as well as the 100-year, 2-hour precipitation mass curve were modified so that the tabular data and figure match. A table and figure representing the SCS Type II rainfall distribution was added. All other figures were scanned and edited or recreated. All procedures were moved to a single section and expanded where appropriate. A new section was added providing information regarding common issues, problems and limitations on the implementation of the procedures. A new detailed, hand written, example has also been provided. At this time, only formatting issues need to be addressed before final conversion to FrameMaker.

Chapter 3 - The Runoff Coefficient (C) table was completely revised. The new table is more explicitly tied to land use classifications for Maricopa County. The I-D-F graph for the City of Phoenix has been moved to an appendix. Use of this method for estimating rainfall intensity is optional. A new discussion has been added for the use of the PREFRE Statistics in developing rainfall intensities. A minimum reasonable time of concentration of 10-minutes is discussed in the application section and may need to be provided as a standard in the Policies and Standards Manual for Maricopa County. A new section for a multiple basin study area has been added along with a section providing information regarding common issues, problems and limitations on the implementation of the procedures. Also, a new detailed, hand written, example was added. At this time, some editorial issues must be resolved before conversion to FrameMaker.

Chapter 4 - The rainfall loss characteristics table for land uses has been completely revised. The new table is more explicitly tied to land use classifications in Maricopa County. Table 4.1, Surface Retention Loss for Various Land Surfaces in Maricopa County was incorporated into the new rainfall loss characteristics table. All figures were scanned and edited or recreated. The procedures were expanded where appropriate. A new section was added providing information regarding common issues, problems and limitations on the implementation of the procedures. A new detailed, hand written, example was added that builds on the example from Chapter 2. At this time, only minor formatting issues need to be addressed before conversion to FrameMaker.

Chapter 5 - The example demonstrating the translation of rainfall excess to a runoff hydrograph was expanded. A new table for adjusting the slope for steep watercourse was added and the corresponding figure was revised. All figures were scanned and edited or recreated. The procedures were expanded where appropriate. A new section was added providing information regarding common issues, problems and limitations on the implementation of the procedures. A new detailed, hand written, example was added that builds on the examples from Chapters 2 and 4. At this time, only minor formatting issues need to be addressed prior to conversion to FrameMaker.

Chapter 6 - This is an entirely new chapter that has been added to provide an alternative method for estimating runoff magnitudes for the 10-, 5- and 2-year events. The first draft of this chapter is in process now.

Chapter 7 - The order of the routing procedures were revised, as well as the text, to reflect the preferred order of the various procedures. A new section was added providing information regarding common issues, problems and limitations on the implementation of the procedures. At this time, only minor formatting issues need to be addressed before conversion to FrameMaker.

Chapter 8 - This is an entirely new chapter that has been added for the verification of peak discharges by indirect methods. Information in the chapter is adapted from a similar chapter in the Arizona Department of Transportation Hydrology Manual. All figures were recreated for specific use in Maricopa County.

Additional Changes:

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- The existing Chapter 7, Application has been eliminated. The contents of this chapter have been moved to the User Notes section of the appropriate chapter.
- References - new references are added.
- Appendix A - This appendix corresponds to Chapter 2 and contains three sections. Section 1 is the isopluvial maps. Section 2 is the precipitation depth duration diagram, formerly Appendix F. Section 3 is the PREFRE Users Manual.

- Appendix B - This appendix corresponds to Chapter 3 and contains the City of Phoenix I-D-F graph and a blank I-D-F graph.
- Appendix C - This appendix corresponds to Chapter 4 and contains five sections. Section 1 is the assumptions and criteria used in developing the XKSAT tables in Sections 2 through 4. Sections 2 through 4 are the XKSAT tables for the three Soil Survey reports currently covered in Appendices A through C. Section 5 is the texture classification diagram.
- Appendix D - This appendix corresponds to Chapter 5 and contains two Sections. Section 1 is the T_c and R worksheet. Section 2 is the K_n values for various rainfall-runoff events.
- Appendix E is reserved for the DDMSW Users Manual.

The majority of the effort remaining is word processing and editing. It is anticipated that a final draft version of all the chapters can be completed by 31 May 2002. Assuming three weeks for review by Flood Control District of Maricopa County Staff, the final copy could be delivered by 30 June 2002.

Sincerely,

STANTEC CONSULTING INC.

Michael Gerlach

Michael Gerlach, PE
Engineer, Water Resources
mgerlach@stantec.com

MCG/cjm

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08 May 2002
File: 82000042

Mr. Thomas R. Loomis, P.E., R.L.S.
Flood Control District of Maricopa County
2901 W Durango St
Phoenix, AZ 85009

Dear Tom:

During the 9 April 2002 Hydrology Manual meeting, you requested a comparison of the runoff volumes computed for 2-, 5- and 10-year recurrence intervals using the current procedures (changing rainfall depths and recomputing T_c) and the proposed ratio procedure. This comparison is provided for the five test watersheds used in the evaluation of the ratio procedure. The results of this comparison are summarized in the following table.

Runoff Volume, in acre-feet

	Current Procedure				Ratio Procedure		
Watershed	**100-Yr**	**10-Yr**	**5-Yr**	**2-Yr**	**10-Yr**	**5-Yr**	**2-Yr**
Agua Fria River Trib.	14	7	6	3	5	3	1
Salt River Trib.	204	104	77	44	71	51	21
Tucson Arroyo	944	533	416	269	329	234	95
Hartman Wash	394	142	80	6	139	98	38
Cave Creek							
HEC-1 ID S310	227	126	100	61	80	57	23
HEC-1 ID S350	22	13	11	8	8	6	2
HEC-1 ID C390L	377	228	189	135	131	93	36
HEC-1 ID C410	9,825	5,293	3,985	2,120	3,286	2,295	836

As can be seen from this table, there is a significant difference in the computed runoff volume between the two procedures. Attached are some figures that illustrate the differences in volume. Intuitively, I expected that the ratio of the 100-year runoff (ratio procedure) would produce a greater runoff volume than computing rainfall excess using the same rainfall loss function as the 100-year and the lower rainfall intensity of the 2-, 5- and 10-year events (current procedure). Instead, these results are exactly opposite of what was expected. In fact, the ratio of the computed runoff volume for the 2-, 5- and 10-year to the 100-year is greater than the discharge ratio except for Hartman Wash. I believe that this is due to the estimation of percent impervious or perhaps the drainage area that actually contributes runoff for more frequent events. All the watersheds shown (including the individual subareas shown for Cave Creek) have some percentage of impervious area except for Hartman Wash. Percent impervious is difficult to estimate, particularly in regard to hydraulic conductivity to the basin outlet. For less frequent storms it is likely that this parameter is being overestimated, even for urban areas. To use my house as an example, for more frequent events I am certain that little or no runoff ever leaves my backyard. Based on this, the ratio procedure seems to make more sense both in terms of peak discharge and runoff volume.

Also requested was a check that the ratio procedure was compatible with a model containing diversions. A portion of the Maryvale ADMS model was used for this purpose. This is a complex model incorporating numerous hydrograph diversion and retrieval operations. The results of the 10-year ratio model were compared to the 10-year model prepared by the original study contractors for a small portion of the overall model. The 10-year ratio model ran without error producing results similar to those of the original 10-year model.

Sincerely,

STANTEC CONSULTING INC.

Michael Gerlach

Michael Gerlach, PE
Engineer, Water Resources
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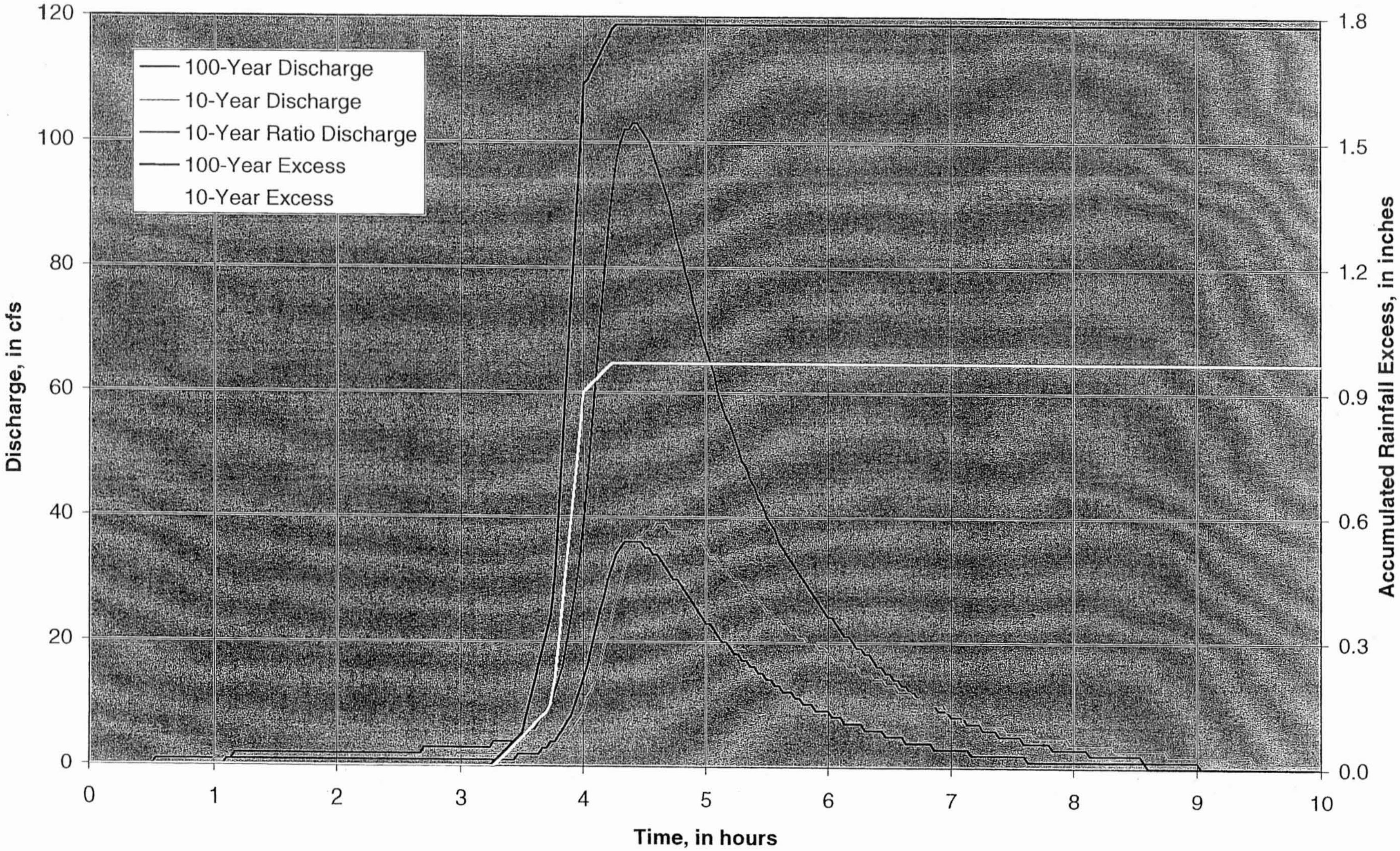
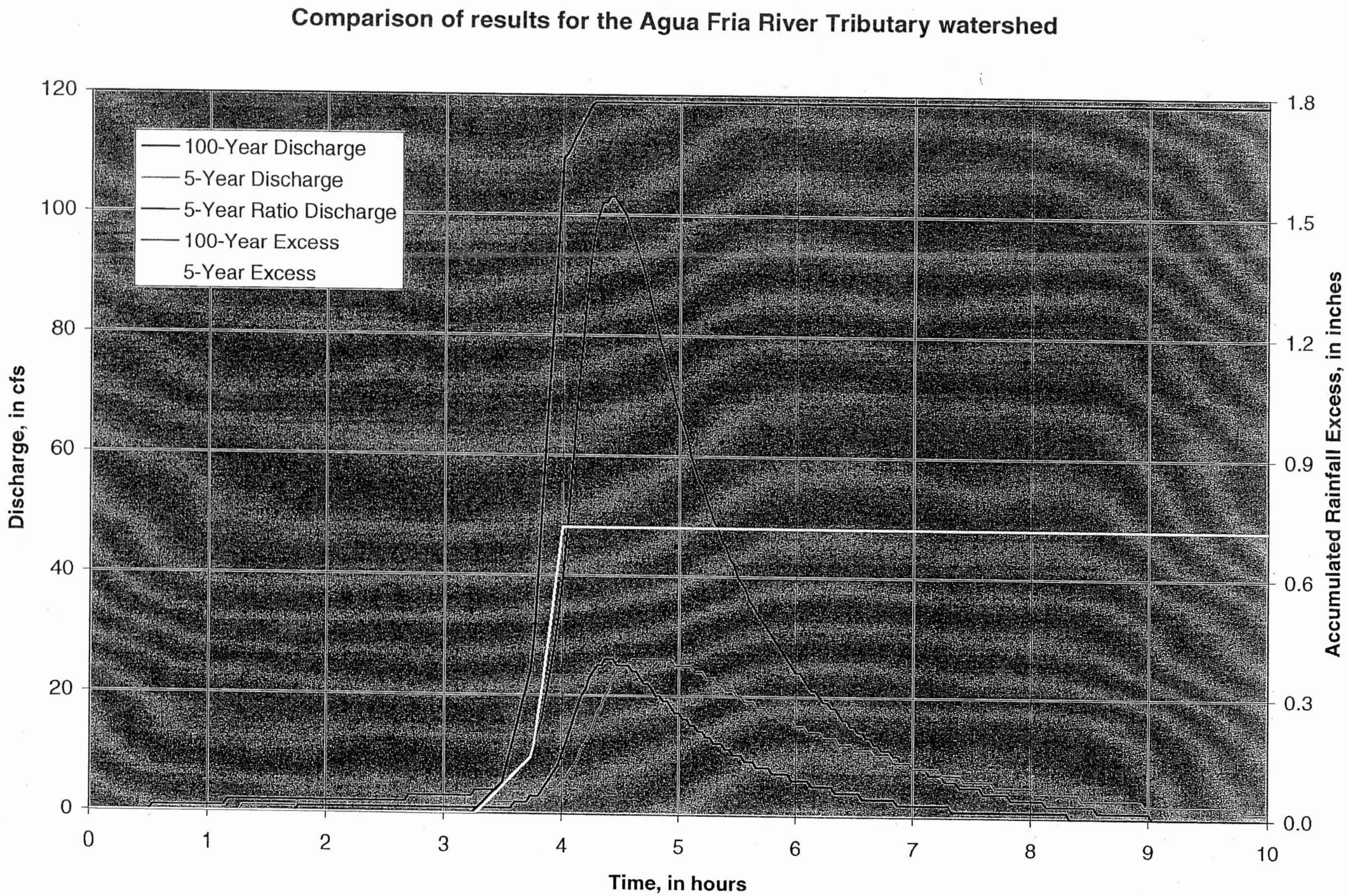
Comparison of results for the Agua Fria River Tributary watershed





Comparison of results for the Agua Fria River Tributary watershed
100-Year Discharge
5-Year Discharge
5-Year Ratio Discharge
100-Year Excess
5-Year Excess
Discharge, in cfs
Accumulated Rainfall Excess, in inches
Time, in hours
0
20
40
60
80
100
120
0.0
0.3
0.6
0.9
1.2
1.5
1.8
0
1
2
3
4
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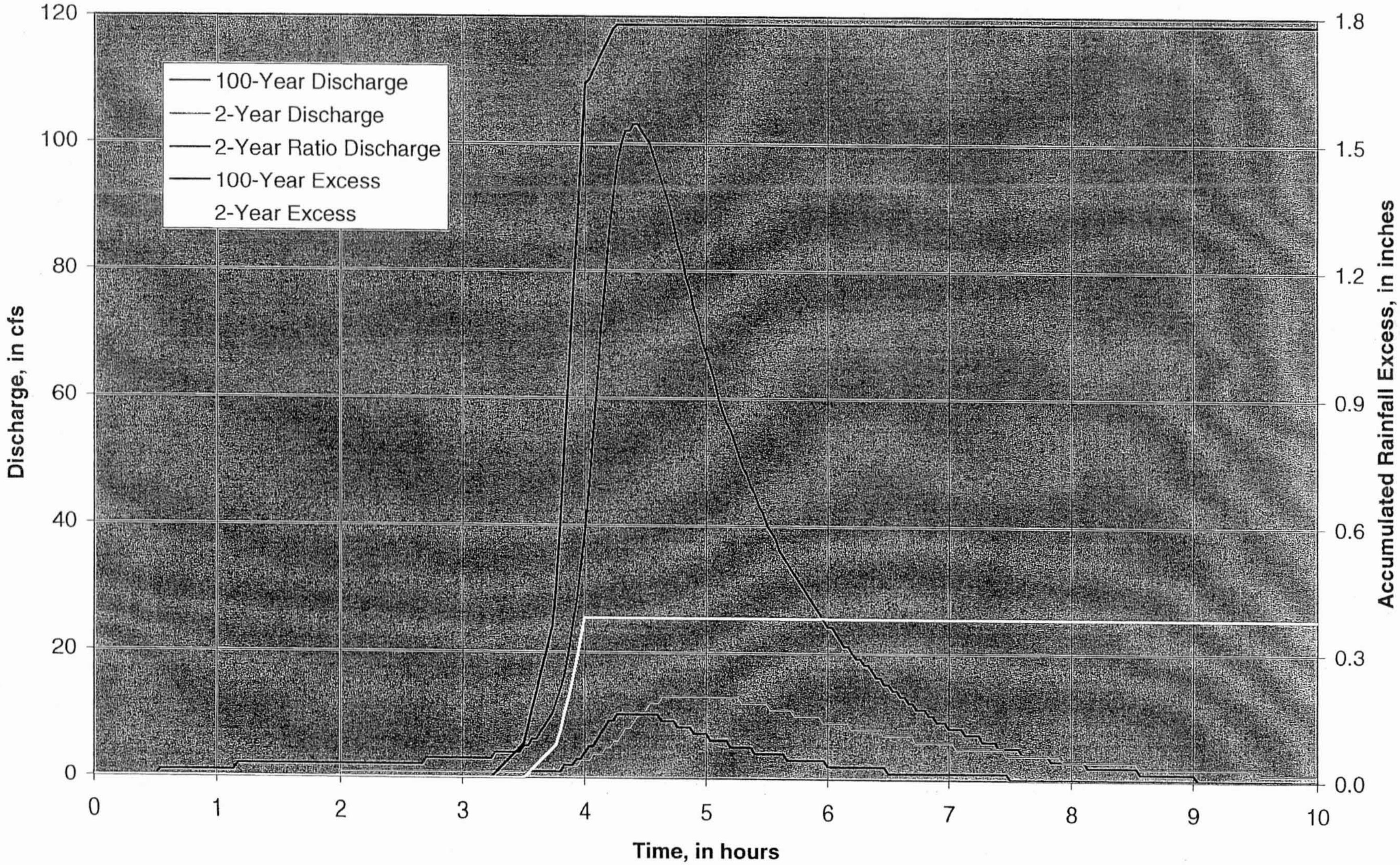
6
7
8
9
10

Comparison of results for the Agua Fria River Tributary watershed



Broad Crested Weirs = "C"

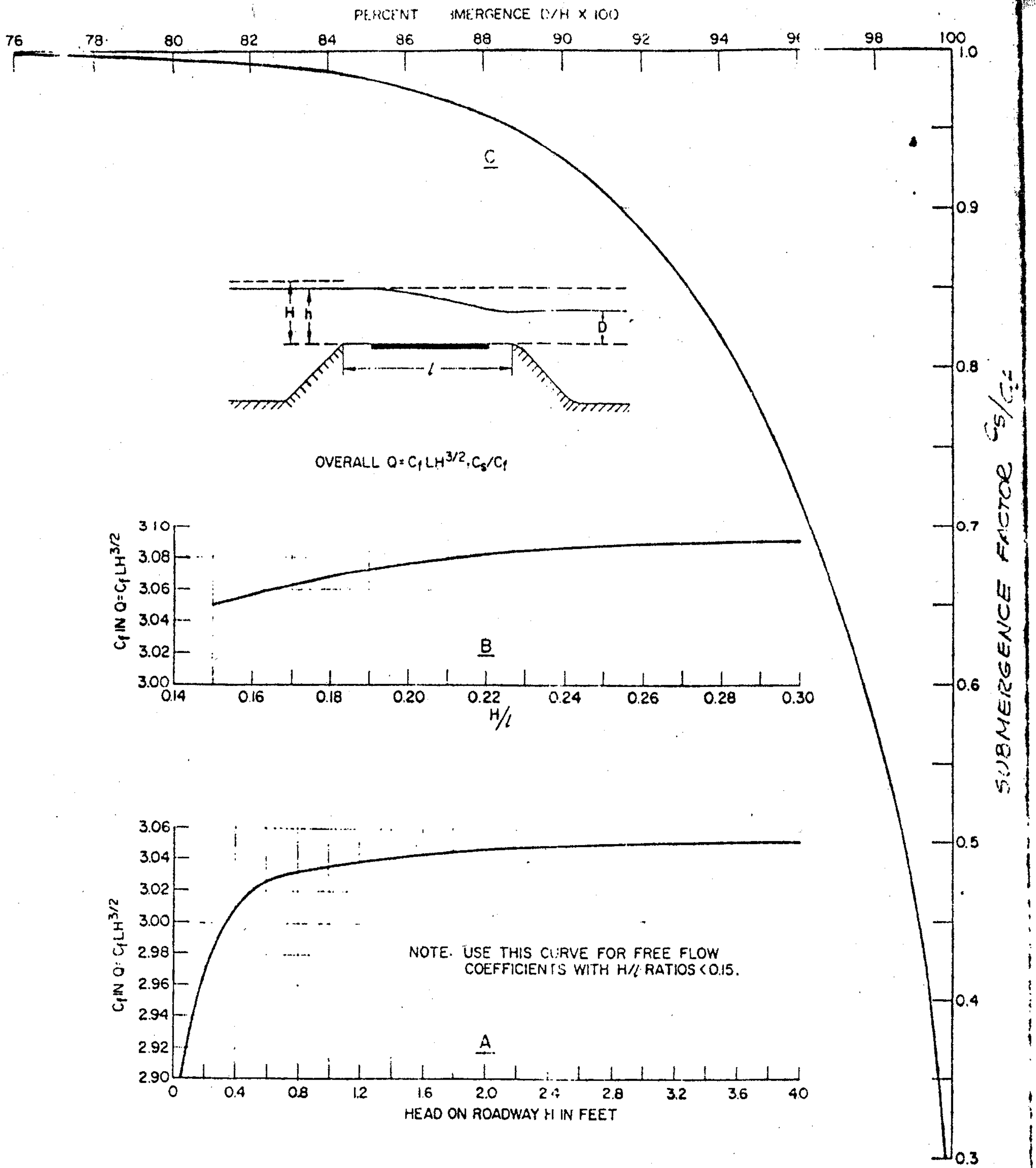


Figure 24.—Discharge coefficients for flow over roadway embankments.

port, Mo., on Interstate 70. A profile across the valley looking upstream is shown on figure 25. The bridge is located well above high water, the approach embankment on the left is set at about the 75-year flood level, yet there is adequate sight distance throughout. This is the ideal valley cross section and the bridge and embankment have been tailor made to fit the site. The arrangement will accommodate any flood that is likely to occur with a minimum of damage. Computation of flow across this

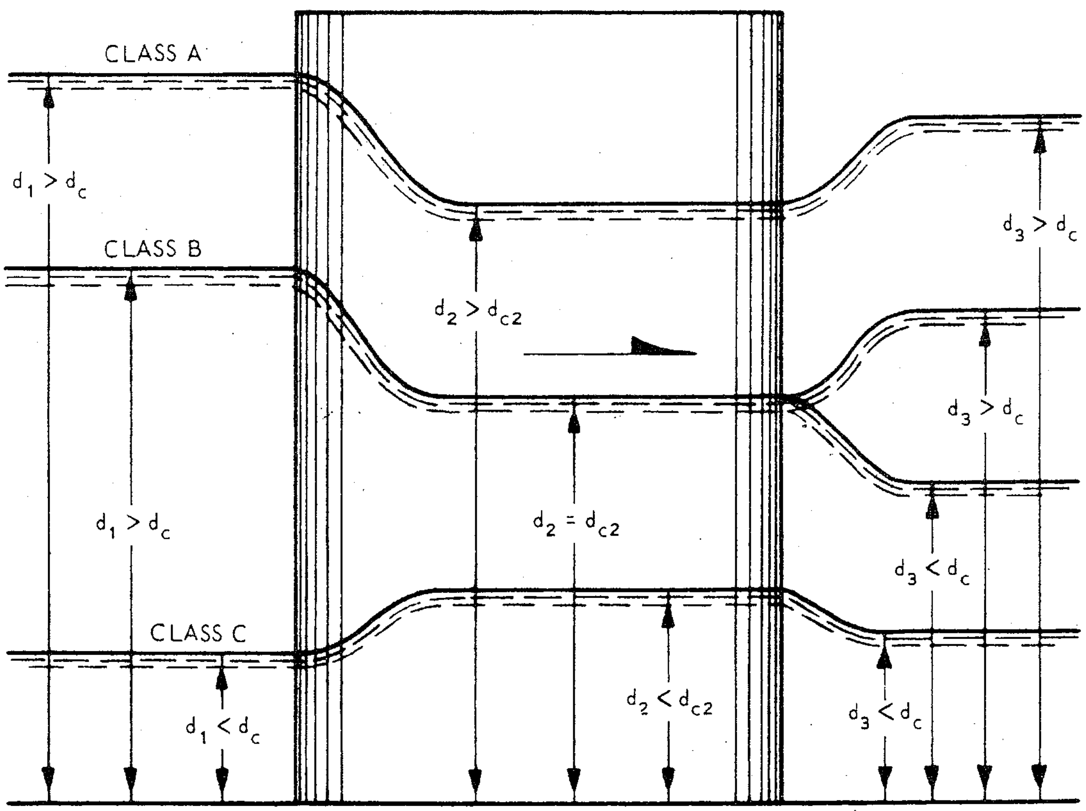
Bureau of Public Roads
Hydraulic of Bridge Waterways
Hyd. Design Series No. 1 1973

Reference used for rip-rap

G. BRADY

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 10

HYDRAULIC DESIGN OF



FLOOD CONTROL CHANNELS

AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE PRESS

TECHNICAL ENGINEERING AND DESIGN GUIDES
AS ADAPTED FROM THE
US ARMY CORPS OF ENGINEERS, NO. 10

HYDRAULIC DESIGN OF FLOOD CONTROL CHANNELS

Published by
ASCE Press
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345 East 47th Street
New York, New York 10017-2398

ABSTRACT

This U. S. Army Corps of Engineers engineer manual EM 1110-2-1601, *Hydraulic Design of Flood Control Channels*, presents procedures for the design analysis and criteria of design for improved channels that carry rapid and/or tranquil flows. This book presents theories and procedures in the hydraulic design of flood control channels, levees, and floodwalls. Typical calculations are presented to illustrate the principles of design for channels under various conditions of flow. The appendices provide additional information of certain specific topics such as the standardization of riprap gradations, calculation of stone size, and combination of flow at open channel junctions.

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CHAPTER 3

RIPRAP PROTECTION

SECTION I
INTRODUCTION

3-1. GENERAL.

The guidance presented herein applies to riprap design for the following conditions: open channels not immediately downstream of stilling basins or other highly turbulent areas (for stilling basin riprap, use HDC 712-1), and channel slopes less than 2 percent. The ability of riprap slope protection to resist the erosive forces of channel flow depends on the interrelation of the following factors: stone shape, size, weight, and durability; riprap gradation and layer thickness; and channel alignment, cross-section, gradient, and velocity distribution. The bed material and local scour characteristics determine the design of toe protection which is essential for riprap revetment stability. The bank material and groundwater conditions affect the need for filters between the riprap and underlying material. Construction quality control of both stone production and riprap placement is essential for successful bank protection. Riprap protection for flood control channels and appurtenant structures should be designed so that any flood that could reasonably be expected to occur during the service life of the channel or structure would not cause damage exceeding nominal maintenance or replacement (see ER 1110-2-1150). While the procedures presented herein yield definite stone sizes, results should be used for guidance purposes and revised as deemed necessary to provide a practical protection design for the specific project conditions.

3-2. RIPRAP CHARACTERISTICS.

The following provides guidance on stone shape, size/weight relationship, unit weight, gradation, and layer thickness. Reference EM 1110-2-2302 for additional guidance on riprap material characteristics and construction.

A. Stone Shape. Riprap should be blocky in shape rather than elongated, as more nearly cubical stones "nest" together best and are more resistant to movement. The stone should have sharp, angular, clean edges at the intersections of relatively flat faces. Stream rounded stone is less resistant to movement, although the drag force on a rounded stone is less than on angular, cubical stones. As rounded stone interlock is less than that of equal-sized angular stones, the rounded stone mass is more likely to be eroded by channel flow. If used, the rounded stone should be placed on flatter side slopes than angular stone and should be about 25 percent larger in diameter. The following shape limitations should be specified for riprap obtained from quarry operations:

1. The stone shall be predominantly angular in shape.

2. Not more than 30 percent of the stones distributed throughout the gradation should have a ratio of a/c greater than 2.5.
max length
Dim of stone parallel to short axis

3. Not more than 15 percent of the stones distributed throughout the gradation should have a ratio of a/c greater than 3.0.

4. No stone should have a ratio of a/c greater than 3.5.

To determine stone dimensions a and c, consider that the stone has a long axis, an intermediate axis, and a short axis, each being perpendicular to the other. Dimension a is the maximum length of the stone, which defines the long axis of the stone. The intermediate axis is defined by the maximum width of the stone. The remaining axis is the short axis. Dimension c is the maximum dimension parallel to the short axis. These limitations apply only to the stone within the required riprap gradation and not to quarry spalls and waste that may be allowed.

B. Relation between Stone Size and Weight. The ability of riprap revetment to resist erosion is related to the size and weight of stones. Design guidance is often expressed in terms of the stone size $D_{\%}$, where % denotes the percentage of the total weight of the graded material (total weight including quarry wastes and spalls) that contains stones of less weight. The relation between size and weight of stone is described herein using a spherical shape by the equation:

$$D_{\%} = \left(\frac{6W_{\%}}{\pi\gamma_s}\right)^{1/3} \qquad (3\text{-}1)$$

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Highways in the River Environment

Participant Notebook

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5.4 FLOW CONTROL STRUCTURES

A flow control structure is defined here as a structure, either within or outside a channel that acts as a countermeasure by controlling the direction, velocity, or depth of flowing water. Structures within this category are sometimes called "river training works". Among the most important properties of a flow control structure is its degree of permeability. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce water velocity. As used here, the term "permeable" means that a structure has definite openings through which water is intended to pass, such as openings between adjacent boards or pilings, or the meshes of wire. Structures made of riprap, or filled with riprap, have some degree of permeability, but these are classed as impermeable because they act essentially as impermeable barriers to a rapidly moving current of water.

Types of flow control structures are distinguished on Fig. 5.4.1.

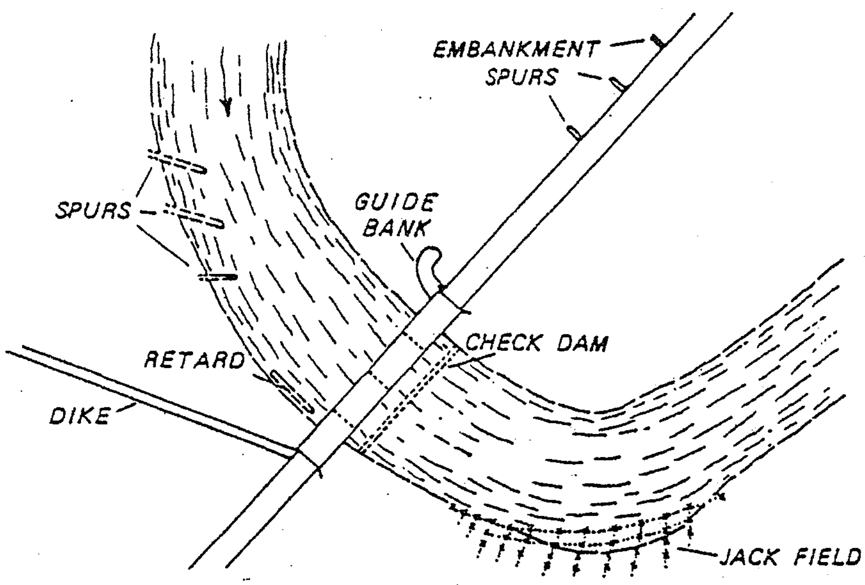


Fig. 5.4.1 Placement of flow control structures relative to channel banks, crossing, and flood plain. Spurs, retards, dikes, and jack fields may be either upstream or downstream from the bridge. (From Brice et al., 1978)

5.4.1 Spurs

A spur is a structure or embankment projected into a stream from the bank at some angle and for a short distance to deflect flowing water away from critical zones, to prevent erosion of the bank, and to establish a more desirable channel alignment or width. By deflecting the current from the bank and causing sediment deposits behind them, a spur or a series of spurs may protect the stream bank more effectively and at less cost than riprapping the bank. Also, by moving the location of any scour away from the bank, failure of the riprap on the spur can often be repaired before damage is done to structures along and across the rivers. Conversely, failure of riprap on the bank may immediately endanger structures.

Spurs are used to protect highway embankments that form the approaches to a bridge crossing. Often these highway embankments cut off the overbank flood flows causing these flows to run parallel to the embankment enroute to the bridge opening. Spurs constructed perpendicular to the highway embankment keep the potentially erosive current away from the embankment, thus protecting it. Spurs as used in this report encompass the terms dikes, jetties, groins, and spur dikes which are also used to describe these structures.

Spurs are also used to channelize a wide, poorly defined stream into a well-defined channel that neither aggrades nor degrades, thus maintaining its location from year to year. Spurs on streams with suspended sediment discharge can cause deposition to establish and maintain the new alignment. The use of spurs in this instance may decrease the length necessary for the bridge opening and may make a more suitable, stable channel approach to the bridge. This decreases the cost of the bridge structure.

The following major recommendations from Brown (1985) are organized by design component for easy reference.

- Extent of Channelbank Protection
  - A common mistake in streambank protection is to provide protection too far upstream and not far enough downstream.
  - The extent of bank protection should be evaluated using a variety of techniques, including: empirical methods, field reconnaissance, evaluation of flow traces for various flow stage conditions, and review of flow and erosion forces for various flow stage conditions. Information from these approaches should then be combined with personal judgment and a knowledge of the flow processes occurring at the local site to establish the appropriate limits of protection.

- Spur Length
  - As the spur length is increased:

    the scour depth at the spur tip increases,

    the magnitude of flow concentration at the spur tip increases,

    the severity of flow deflection increases, and

    the length of channel bank protection increases.
  - The projected length of impermeable spurs should be held to less than 15 percent of the channel width at bank-full stage.
  - The projected length of permeable spurs should be held to less than 25 percent of the channel width. However, this criterion depends on the magnitude of the spur's permeability. Spurs having permeabilities less than 35 percent should be limited to projected lengths not to exceed 15 percent of the channel's flow width. Spurs having permeabilities of 80 percent can have projected lengths up to 25 percent of the channel's bank-full flow width. Between these two limits, a linear relationship between the spur permeability and spur length should be used.

- Spur Spacing
  - The spacing of spurs in a bank-protection scheme is a function of the spur's length, angle, and permeability, as well as the channel bend's degree of curvature.
  - The direction and orientation of the channel's flow thalweg plays a major role in determining an acceptable spacing between individual spurs in a bank-stabilization scheme.
  - Reducing the spacing between individual spurs below the minimum required to prevent bank erosion between the spurs results in a reduction of the magnitude of flow concentration and local scour at the spur tip.
  - Reducing the spacing between spurs in a bank-stabilization scheme causes the flow thalweg to stabilize further away from the concave bank towards the center of the channel.
  - A spacing criteria based on the projection of a tangent to the flow thalweg, projected off the spur tip, as presented in the above discussions, should be used.

- Spur Angle/Orientation
  - The primary criterion for establishing an appropriate spur orientation for the spurs within a given spur scheme is to provide a scheme that efficiently and economically guides the flow through the channel bend, while protecting the channel bank and minimizing the adverse impacts to the channel system.
  - Spurs angled downstream produce a less severe constriction of flows than those angled upstream or normal to flow.
  - The greater an individual spur's angle in the downstream direction, the smaller the magnitude of flow concentration and local scour at the spur tip. Also, the greater the angle, the less severe the magnitude of flow deflection towards the opposite channel bank.
  - Impermeable spurs create a greater change in local scour depth and flow concentration over a given range of spur angles than do permeable spurs. This indicates that impermeable spurs are much more sensitive to these parameters than are permeable spurs.
  - Spur orientation does not in itself result in a change in the length of channel bank protected for a spur of given projected length. It is the greater spur length parallel to the channel bank associated with spurs oriented at steeper angles that results in the greater length of channel bank protected.
  - Retardance spurs should be designed perpendicular to the primary flow direction.
  - Retardance/diverter and diverter spurs should be designed to provide a gradual flow training around the bend. This is accomplished by maximizing the flow efficiency within the bend while minimizing any negative impacts on the channel geometry.
  - The smaller the spur angle, the greater the magnitude of flow control as represented by a greater shift of the flow thalweg away from the concave (outside) channel bank.
  - It is recommended that spurs within a retardance/diverter or diverter spur scheme be set with the upstream-most spur at approximately 150 degrees to the main flow current at the spur tip, and with subsequent spurs having incrementally smaller angles approaching a minimum angle of 90 degrees at the downstream end of the scheme.

- Spur Height
  - The spur height should be sufficient to protect the regions of the channel bank impacted by the erosion processes active at the particular site.
  - If the design flow stage is lower than the channel bank height, spurs should be designed to a height no more than three feet lower than the design flow stage.
  - If the design flow stage is higher than the channel bank height, spurs should be designed to bank height.
  - Permeable spurs should be designed to a height that will permit the passage of heavy debris over the spur crest and not cause structural damage.
  - When possible, impermeable spurs should be designed to be submerged by approximately three feet under their worst design flow condition, thus minimizing the impacts of local scour and flow concentration at the spur tip and the magnitude of flow deflection.

- Spur Crest Profile
  - Permeable spurs should be designed with level crests unless bank height or other special conditions dictate the use of a sloping crest design.
  - Impermeable spurs should be designed with a slight fall towards the spur head, thus allowing different amounts of flow constriction with stage (particularly important in narrow-width channels), and the accomodation of changes in meander trace with stage.

- Channel bed and Channel bank Contact
  - Careful consideration must be given to designing a spur that will maintain contact with the channel bed and channel bank so that it will not be undermined or outflanked.

- Spur Head Form
  - A simple straight spur head form is recommended.
  - The spur head or tip should be as smooth and rounded as possible. Smooth, well-rounded spur tips help minimize local scour, flow concentration, and flow deflection.

5.4.2 Hardpoints

Hardpoints are an erosion control technique consisting of stone fills spaced along an eroding bank line (Fig. 5.4.2). The structures protrude only short distances into the river channel and are supplemented with a root section extending landward into the bank to preclude flanking, should excessive erosion persist. The majority of the structure cannot be seen as the lower part consists of rock placed underwater, and the upper part is covered with topsoil and seeded with native vegetation. The structures are especially adaptable in long, straight reaches not subject to direct attack.

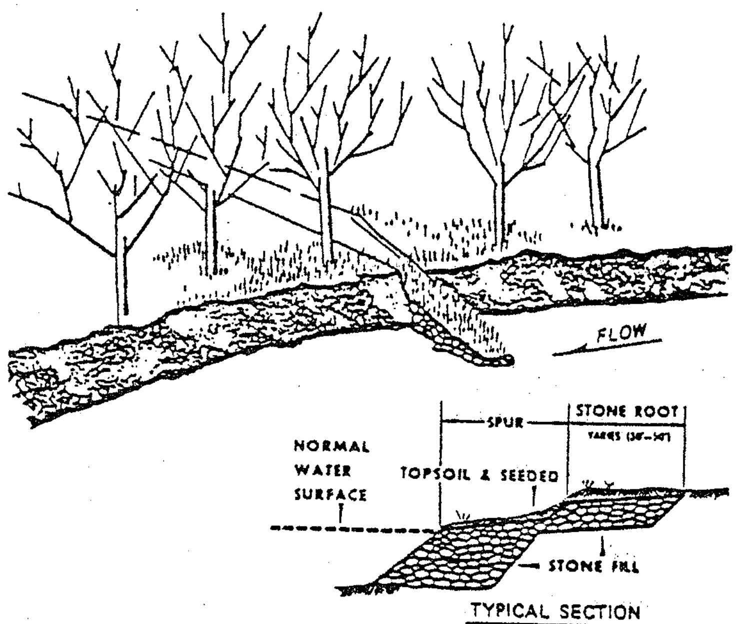


Fig. 5.4.2 Perspective of hard point with section detail. (After Brown, 1985).

5.4.3 Retards

Retards are devices placed parallel to embankments and river banks to decrease the stream velocities and prevent erosion.

- Pile retards can be made of concrete, steel or timber. The design of timber pile retards is essentially the same as timber pile dikes shown in Fig. 5.4.5. They may be used in combination with bank protection works such as riprap. The retard then serves to reduce the velocities sufficiently so that either smaller riprap can be used, or riprap can be eliminated.

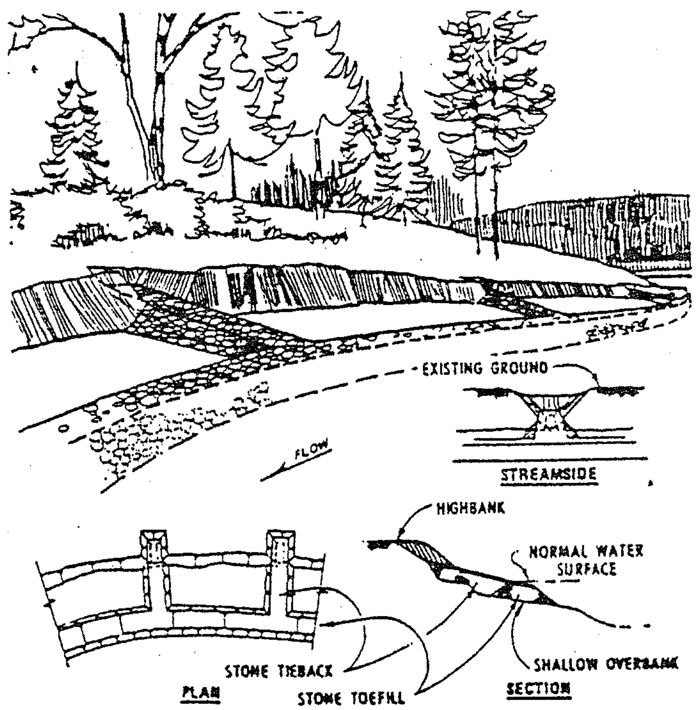


Fig. 5.4.3 Retard.

- Timber or concrete cribs - Timber and concrete cribs are sometimes used for bulkheads and retaining walls to hold highway embankments, particularly where lateral encroachment into the river must be limited. Cribs are made up by interlocking pieces together in the manner shown in Fig. 5.4.4. The crib may be slanted or vertical depending on height and the crib is filled with rock or earth. Reinforced concrete retaining walls are alternatives to timber cribs which can be considered. However, concrete retaining walls are expensive and are generally only used in special confined locations where space precludes other methods of bank protection. In constructing concrete retaining walls drainage holes (weep holes) must be provided. The foundation of these walls should be placed below expected scour depths.

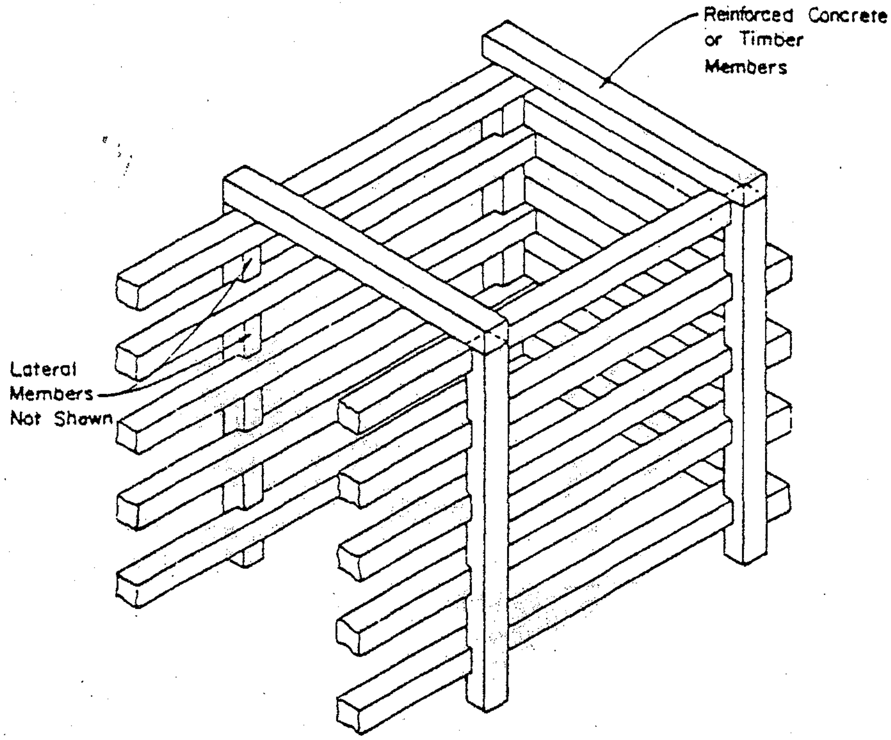
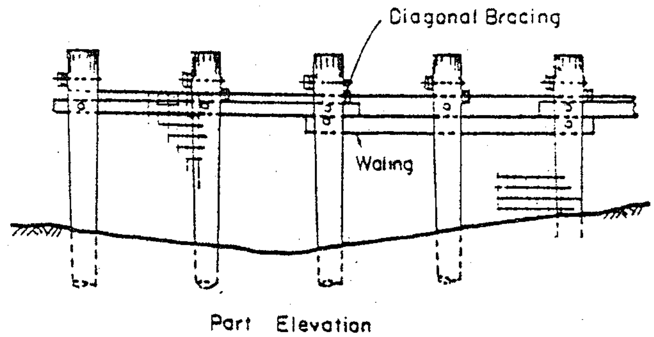


Fig. 5.4.4 Concrete or timber cribs.

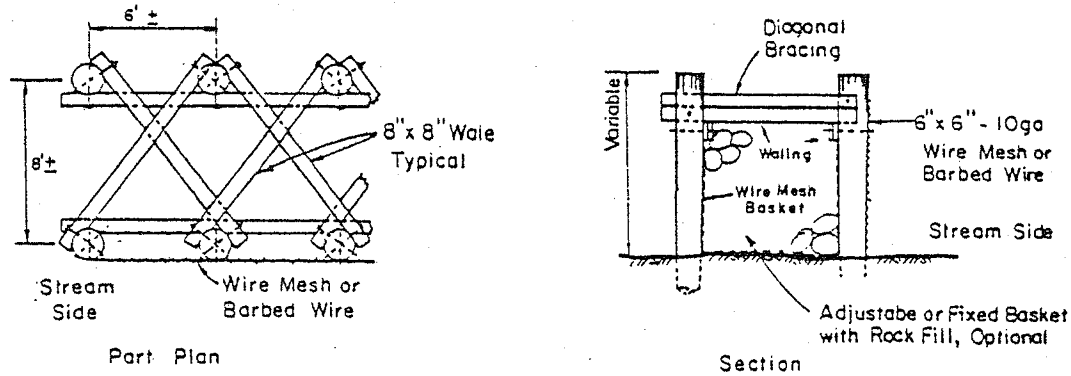
5.4.4 Dikes

There are two principal types of dikes, permeable and impermeable. Permeable dikes are those which permit flow through the dike but at reduced velocities, thereby preventing further erosion of the banks and causing deposition of suspended sediment from the flow.

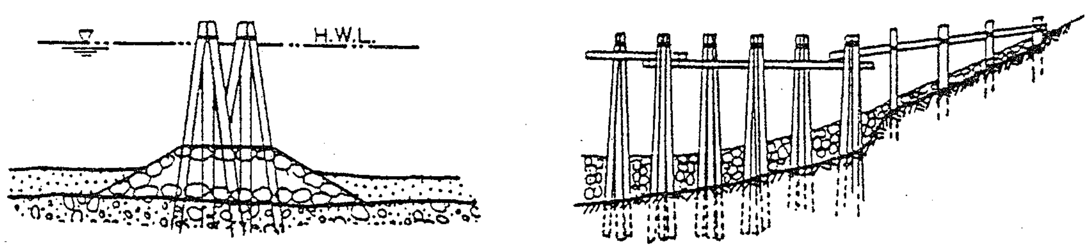
- Timber or steel pile dikes - Pile dikes (also retards) may consist of closely-spaced single, double, or multiple rows. There are a number of variations to this scheme. For example, wire fence may be used in conjunction with pile dikes to collect debris and thereby cause effective reduction of velocity. Double rows of piles can be placed together to form cribs, and rocks may be used to fill the space between the piles. Pile dikes are vulnerable to failure through scour. This can be overcome if the piles can be driven to a large depth to achieve safety from scour, or the base of the piles can be protected from scour with dumped rock in sufficient quantities. The various forms of pile dikes are illustrated in Fig. 5.4.5.



(a) Single row timber pile with wire fence



(b) Double row timber piles with rocks and wire fence



(c) Pile clusters

Fig. 5.4.5. Pile dikes (retards would be similar).

The arrangement of piles depends upon the velocity of flow, quantity of suspended sediment transport, and depth and width of the river. If the velocity of flow is large, pile dikes are not likely to be very effective. Stabilization of the bank by other methods should be considered. On the other hand, in moderate flow velocities with high concentrations of suspended sediments, these dikes can be quite effective. Deposition of suspended sediments in the pile dike field is a necessary consequence of reduced velocities. If there is not sufficient concentration of suspended sediment in the flow, or the velocities in the dike fields are too large for deposition, the permeable pile dikes will only partially be effective in training the river and protecting the bends.

The length of each dike depends on channel width, position relative to other dikes, flow depth and available pile lengths. Generally, pile dikes are not used in large rivers where depths are great, although timber pile dikes have been used in the Columbia River. On the other hand, banks of wide shallow rivers can be successfully protected with dikes. The spacing between dikes varies from 3 to 20 times the length of the upstream dike, with closer spacing favored for best results.

- Stone-fill dikes - Stone-fill dikes are classed as impermeable dikes and do not depend on deposition of sediment between dikes nearly as much as permeable dikes. The principal function is to deflect the flow away from the bank and the dikes must be long enough to accomplish this purpose. The dikes may be angled downstream, angled upstream, or constructed normal to the bank. Variations such as a sloping dike, with declining top elevation away from the bank, L or T head dikes, and curved dikes have been used. Stone-fill dikes are illustrated in Fig. 5.4.6.

The spacing between dikes may vary from three or four dike lengths to 10 or 12 dike lengths depending upon velocity and depth. Short dikes with long spacing are generally not useful for bank protection unless jacks or riprap are used to protect the bank between them.

The ends of the dikes are subjected to local scour and appropriate allowance should be made for loss of dike material into the scour hole. The size of rock to be used for the dike depends on availability of material. Large rocks are generally used to cover the surface, while the internal section may be constructed with smaller rocks or earthfill. Side slopes of 1.5:1 and 2:1 are common.

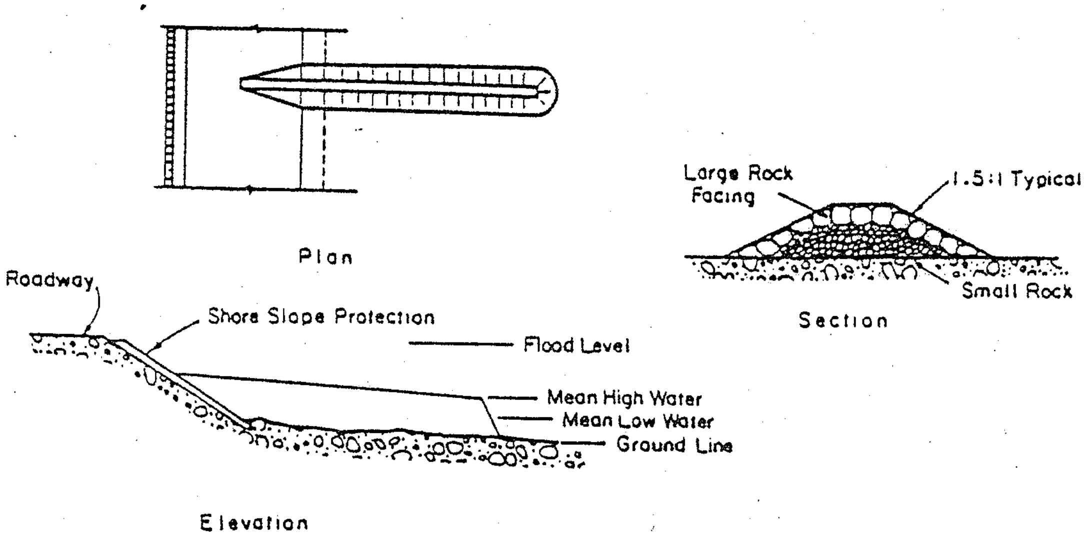


Fig. 5.4.6 Typical stone fill dike.

- Vane Dikes are low-elevation structures designed to guide the flow away from an eroding bank line (Fig. 5.4.7). The structures can be constructed of rock or other erosion-resistant material, the tops of which are constructed below the design water surface elevation and would not connect to the high bank. Water would be free to pass over or around the structure with the main thread of flow directed away from the eroding bank. The structures will discourage high erosive velocities next to an unprotected bank line, encourage diversity of various channel depths, and protect existing natural bottomland characteristics. The findings from a model investigation of these structures include the effects of various vane dike orientation, vane dike length, and gap length (U. S. Army Corps of Engineers, 1981).

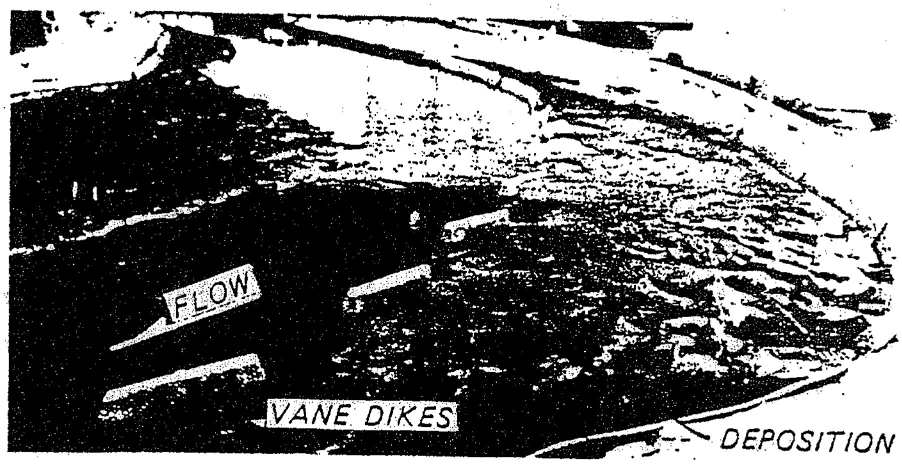


Fig. 5.4.7 Vane dike model, ground walnut shell bed, during low stage portion of test run, elevation 1.482 ft (U. S. Army Corps of Engineers, 1981).

5.4.5 Jetties

The purpose of a jetty field is to add roughness to a channel or overbank area to train the main stream along a selected path. The added roughness along the bank reduces the velocity and protects the bank from erosion. Jetty fields are usually made up of steel jacks tied together with cables. Both lateral and longitudinal rows of jacks are used to make up the jetty field as shown in Fig. 5.4.8.

The lateral rows are usually angled about 45 to 70 degrees downstream from the bank. The spacing varies, depending upon the debris and sediment content in the stream, and may be 50 to 250 feet apart. Jetty fields are effective only if there is a significant amount of debris carried by the stream and the suspended sediment concentration is high.

When jetty fields are used to stabilize meandering rivers, it may be necessary to use jetty fields on both sides of the river channel because in flood stage the river may otherwise develop a chute channel across the point bar. A typical layout is shown in Fig. 5.4.8.

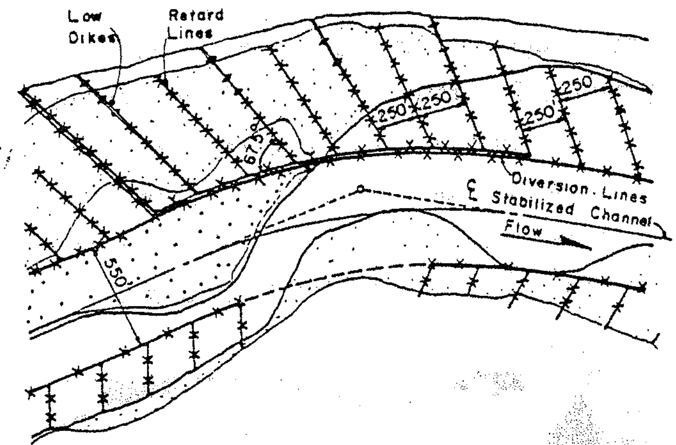
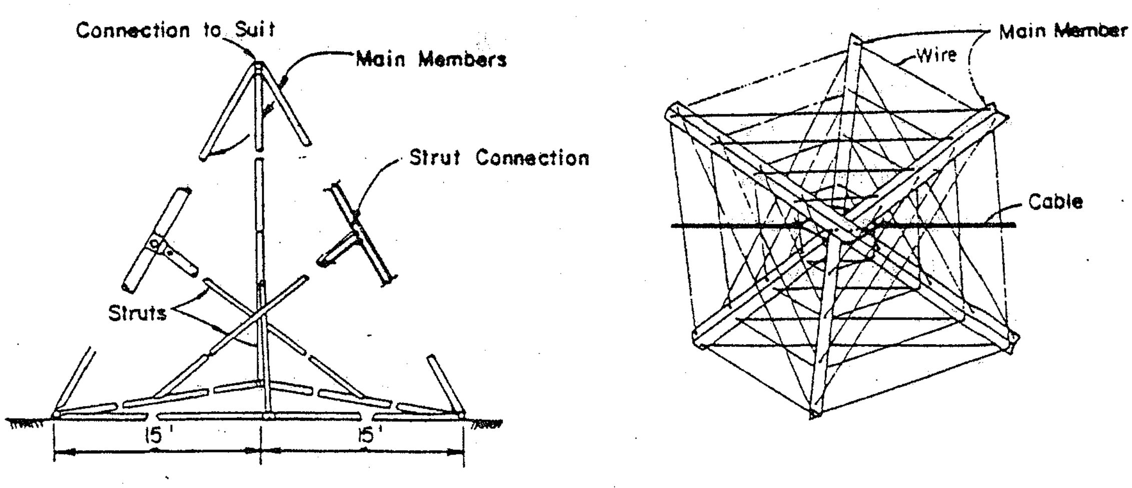


Fig. 5.4.8 Typical jetty-field layout.

<u>Steel jacks</u> are devices with basic triangular frames tied together to form a stable unit. The resulting framework is called a tetrahedron. The tetrahedrons are placed parallel to the embankment and cabled together with the ends of the cables anchored to the bank. Wire fencing may be placed along the row of tetrahedrons. In order to function well, there must be considerable debris in the stream to collect on the fence and the suspended sediment concentration must be large so that there will be deposition behind the retard. Various forms of steel jacks may be assembled. Two types are shown in Fig. 5.4.9. Tiebacks should be spaced every 100 feet and space between jacks should not be greater than their width.



(a) Typical Tetrahedron (b) Kellner Jack

Fig. 5.4.9 Steel jacks.

5.4.6 Fencing

Fencing can be used as a low-cost bank protection technique on small to medium size streams. Special structural design considerations are required in areas subject to ice and floating debris. Both longitudinal (parallel to stream) fence retards and transverse (perpendicular to stream) fences have been used in the prototype with varying degrees of success. A model investigation and literature review of longitudinal fence retards with tiebacks were conducted to identify the following important design considerations:

(1) Channel gradient must be stable and not be steep (tranquil flow);

(2) Toe scour protection can be provided by extending the support posts well below the maximum scour expected or by placing loose rock at the base of the fence to launch downward if scour occurs at the toe;

(3) Tiebacks to the bank are important to prevent flanking of the fence and to promote deposition behind the fence;

(4) Fence retards generally reduce attack on the bank so that vegetation can establish; and

(5) Metal or concrete fences are preferred due to ice damage and fire loss of wooden fences.

5.4.7 Guidebanks

Guide banks are placed at or near the ends of approach embankments to guide the stream through the bridge opening. Constructed properly, flow disturbances, such as eddies and cross-flow, will be minimized to make a more efficient waterway under the bridge. They are also used to protect the highway embankment and reduce or eliminate local scour at the embankment and adjacent piers. The effectiveness of guidebanks is a function of river geometry, quantity of flow on the floodplain, and size of bridge opening. A typical guidebank at the end of an embankment is shown in Fig. 5.4.10.

The recommended shape of a guidebank is a quarter ellipse with a major to minor axis ratio of 2.5. The major axis should be approximately parallel to the main flow direction. For bridge crossings normal to the river, the major axis would be normal to the highway embankment. However, for skewed crossings, the guidebank should be placed at an angle with respect to the embankment with the view of streamlining the flow through the bridge opening. An illustration of guidebanks for a skewed crossing is shown in Fig. 5.4.11 and design dimensions recommended by Karaki are shown in Fig. 5.4.12.

The length of the spur dike, L_s, required depends upon quantity of flow on the floodplain, width of bridge opening and skewness of the highway crossing. Shorter spur dikes may be used where floodplain flow is small or scour potential at piers and embankment ends are small.

The upstream and downstream lengths for straight guidebanks are as follows: the upstream length = 0.75 to 1.5 times the width of the opening; and the downstream length = 0.1 to 0.25 times the width of the opening. It is not necessary that both guidebanks on the upstream side be the same length. For some flow conditions a short curved guidebank on one side and a long straight bank on the other may be the best solution.

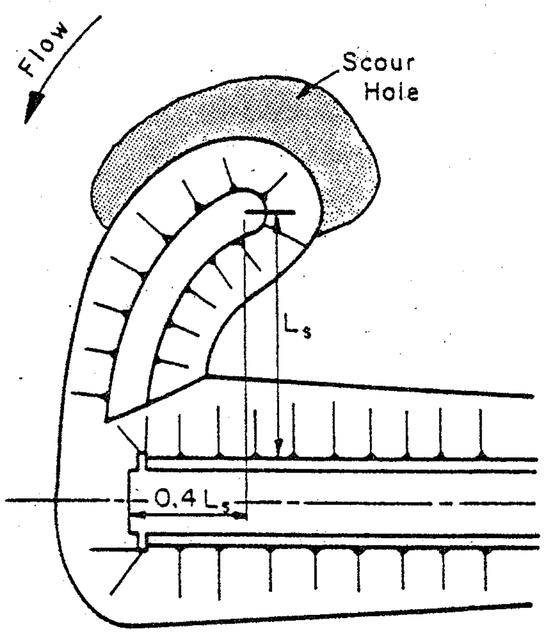


Fig. 5.4.10 Guidebank.

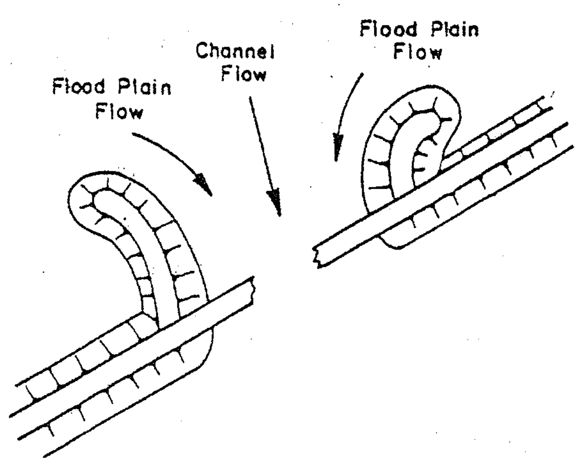


Fig. 5.4.11 Guidebank at skewed highway crossing.

The crest elevation should be 1 ft higher than the elevation of the design flood taking into consideration the effect of the contraction of the flow; this is because the design flow should not overtop the guidebank.

Beside erosion protection, guidebanks provide a more efficient (less head loss) flow of water through a bridge opening. They also decrease scour depth and move the scour energy away from the abutments.

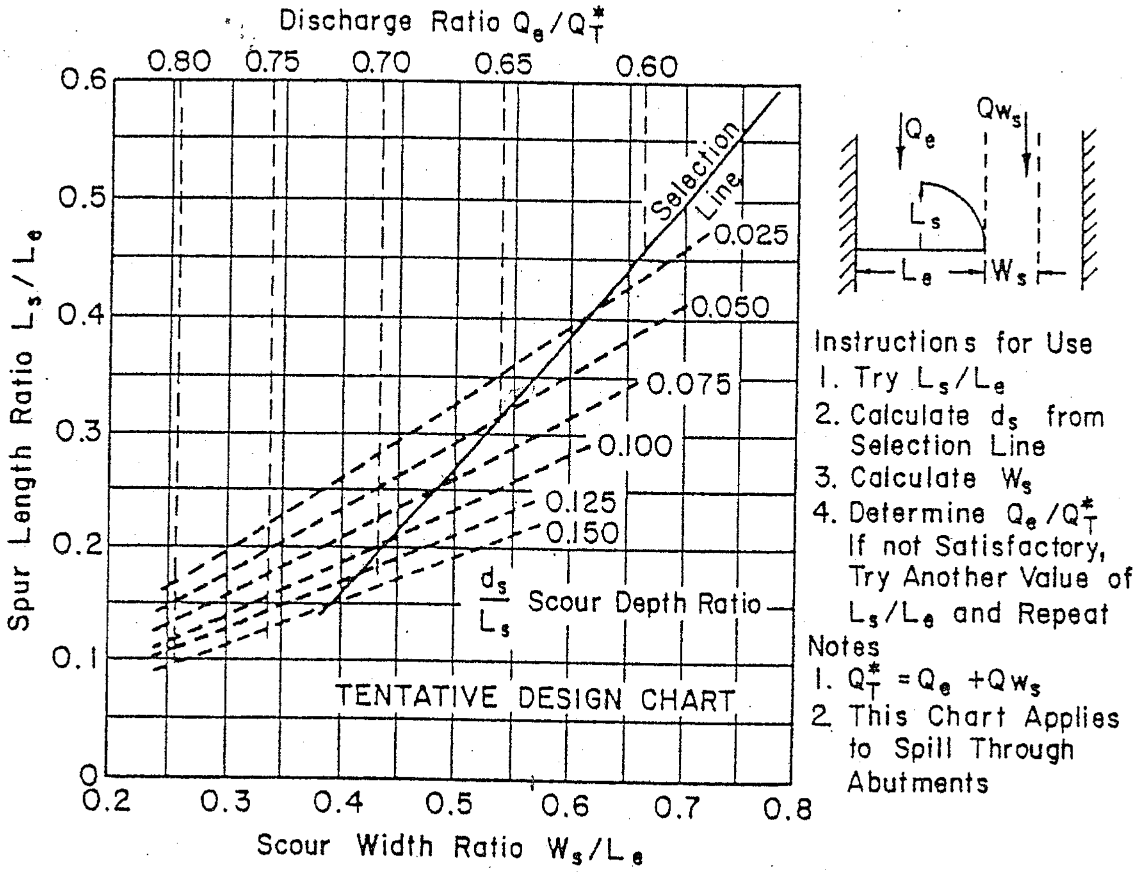


Fig. 5.4.12 Cuidebank design procedure (from Karaki, 1959).

5.4.8 Drop Structures

Drop structures are useful to reduce the slope of a channel. Concrete, soil-cement, gabion sheet pile or timber crib drop structures can be designed considering the stability of the structure and the depth of the scour hole at the toe of the structure. A riprap blanket upstream of the structure is also quite effective as shown in Fig. 5.4.13 for a soil cement drop structure. Definition sketches for a vertical wall and sloping silldrop structure are shown in Figs. 5.4.14 and 5.4.15. The design of vertical wall or sloping sill structures are given in texts by Peterson (1986), Simons, Li and Associates (1982).

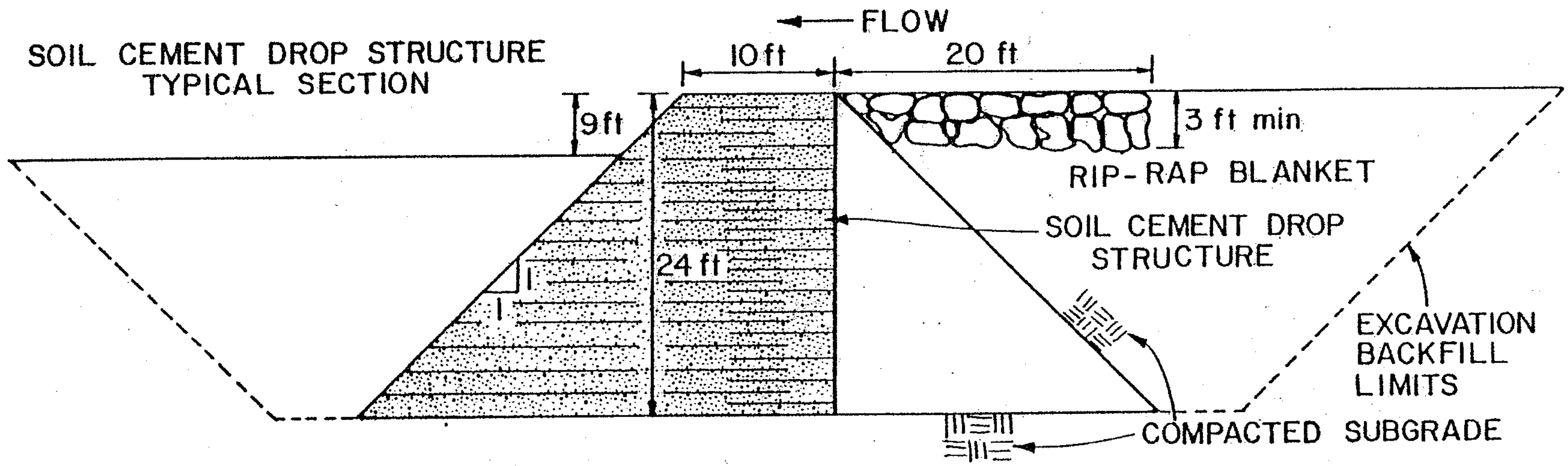


Fig. 5.4.13 Design example of a soil cement drop structure.

Highway Design Manual

CHAPTER 870
CHANNEL AND SHORE PROTECTION - EROSION CONTROL

Topic 873 - Design Concepts

873.1 Introduction

No attempt will be made here to describe in detail all of the various devices that have been used to protect embankments against scour. Methods and devices not described may be used when justified by economical analysis. Not all publicized treatments are necessarily suited to existing conditions for a specific project.

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. These plans should show controlling factors and an end product in such detail that there will be no dispute between the construction engineer and contractor. To serve the dual objectives of adequacy and economy, plans and specifications should be precise in defining materials to be incorporated in the work, and flexible in describing methods of construction or conformance of the end product to working lines and grades.

Recommendations on channel lining, slope protection, and erosion control materials can be requested from the District Hydraulics Engineer, the District Materials Branch and the Erosion Control and Geosynthetics Branch of the Engineering Service Center. The Office of Landscape Architecture can be of assistance in selecting the best practices for temporary and permanent erosion and sediment control measures. The Caltrans Joint Bank Protection Committee is available on request to provide expert advise on extraordinary situations or problems. See Index 802.3 for further information on the organization and functions of the Committee

Combinations of armor-type protection can be used, the slope revetment being of one type and the foundation treatment of another. The use of rigid, non-flexible slope revetment may require a flexible, self-adjusting foundation for example: grouted rock on the slope with heavy rock foundation below, or PCC slope paving with a steel sheet-pile cutoff wall for foundation.

Bank protection may be damaged while serving its primary purpose. Cheap replaceable facilities may be more economical than expensive permanent structures. However, an expensive structure may be economically warranted for highways carrying large volumes of traffic or for which no detour is available.

Cost of stone is extremely sensitive to location. Variables are length of haul, efficiency of the quarry in producing acceptable sizes, royalty to quarry and, necessity for stockpiling and rehandling. On some projects the stone is available in roadway excavation.

Cost of stone is not very sensitive to size. Quarrying produces a wide range of sizes. If only a light riprap is specified, the large stones have to be broken by spot blasting. If heavy riprap is required, the run of the quarry may be usable without reblasting. With Method A placement, one 8 tonne stone can be set quicker than two 4 tonne stones.

873.2 Design High Water and Hydraulics

The most important, and often the most perplexing obligation, in the design of bank and shore protection features is the determination of the appropriate design high water elevation to be used. The design flood stage elevation should be chosen that best satisfies site conditions and level of risk associated with the encroachment. The basis for determining the design frequency, velocity, backwater, and other limiting factors should include an evaluation of the consequences of failure on the highway facility and adjacent property. Stream stability and sediment transport of a watercourse are critical factors in the evaluation process that should be carefully weighted and documented. Designs should not be based on an arbitrary storm or flood frequency. Such designs imply that limiting factors and related risks have been adequately evaluated which is seldom, if ever, the case.

A suggested starting point of reference for the determination of the design high water level is that the protection withstand high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. For example, a modern highway embankment can reasonably be expected to have a service life of 100 years or more. It would therefore be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted, either up or down, to conform with a subsequent analysis which considers the importance of the encroachment and level of related risks.

There is always some risk associated with the design of protection features. Special attention must be given to life threatening risks such as those associated with floodplain encroachments. Significant floodplain risks are classified as those having probability of:

- Catastrophic failure with loss of life.
- Disruption of fire and ambulance services or closing of the only evacuation route available to a community.

Refer to Topic 804, Floodplain Encroachments, for further discussion on evaluation of risks and impacts.

(1) Streambank Locations. The velocity along the banks of watercourses with smooth or uniformly rough tangent reaches may only be a small percentage of the average stream velocity. However, local irregularities of the bank and streambed may cause turbulence that can result in the bank velocity being greater than that of the central thread of the stream. The location of these irregularities is not always permanent as they may be caused by local scour, deposition of rock and sand, or stranding of drift during high water changes. It is rarely economical to protect against all possibilities and therefore some damage should always be anticipated during high water stages.

Essential to the design of streambank protection is sufficient information on the characteristics of the watercourse under consideration. For proper analysis, information on the following types of watercourse characteristics must be developed or obtained:

- Design Discharge
- Design High Water Level
- Flow Types
- Channel Geometry
- Flow Resistance
- Sediment Transport

Refer to Chapter 810, Hydrology, for a general discussion on hydrologic analysis and specifically to Topic 817, Flood Magnitudes; Topic 818, Flood Probability and Frequency; and Topic 819, Estimating Design Discharge. For a detailed discussion on the fundamentals of alluvial channel flow, refer to Chapter III, "Highways in the River Environment", and to HEC-20, Stream Stability at Highway Structures, for further information on sediment transport.

(2) Ocean & Lake Shore Locations. Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(a) Design High Water Level. The flood stage elevation on a lake or reservoir is usually the result of inflow from upland runoff. If the water stored in a reservoir is used for power generation, flood control, or irrigation, the design high water elevation should be based on the owners schedule of operation.

Except for inland tidal basins affected by wind tides, floods and seiches, the static or still-water level used for design of shore protection is the highest tide. In tide tables, this is the stage of the highest tide above "tide-table datum" at MLLW. To convert this to MSL datum there must be subtracted a datum equation (0.8 to 1.2 m) factor. If datum differs from MSL datum, a further correction is necessary. These steps should be undertaken with care and independently checked. Common errors are:

- Ignoring the datum equation.
- Adding the factor instead of subtracting it.
- Using half the diurnal range as the stage of high water.

To clarify the determination of design high-water, Fig. 873.2A shows the *Highest Tide* in its relation to an extreme-tide cycle and to a hypothetical average-tide cycle, together with nomenclature pertinent to three definitions of tidal range. Note that the cycles have two highs and two lows. The average of all the higher highs for a long period (preferably in multiples of the 19-yr. metonic cycle) is MHHW, and of all the *lower* lows, MLLW. The vertical difference between them is the *diurnal range*.

Particularly on the Pacific coast where MLLW is datum for tide tables, the stage of MHHW is numerically equal to diurnal range.

The average of all highs (indicated graphically as the mean of higher high and lower high) is the MHW, and of all the lows, MLW. Vertical difference between these two stages is the *mean range*.

See Index 814.5, Tides and Waves, for information on where tide and wave data may be obtained.

(b) Design Wave Heights.

(1) General. Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. Elaborate numerical models requiring the use of a computer are available, but simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of embayments, inland lakes, and reservoirs. It is recommended that for ocean shore protection designs the assistance of the U.S. Army Corp of Engineers be requested.

Shore protection structures are generally designed to withstand the wave that induces the highest forces on the structure over its economic service life. The design wave is analogous to the design storm considerations for determining return frequency. A starting point of reference for shore protection design is the maximum significant wave height that can occur once in about 20-years.

Economic and risk considerations involved in selecting the design wave for a specific project are basically the same as those used in the analysis of other highway drainage structures.

(2) Wave Distribution Predictions. Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same procedures are used for hindcasting and forecasting. The only difference is the source of the meteorological data. Reference is made to the Army Corps of Engineers, Shore Protection Manual, Volume 1, Chapter 3, for more complete information on the theory of wave generation and predicting techniques.

The prediction of wave heights from boat generated waves must be estimated from observations.

The surface of any large body of water will contain many waves differing in height, period, and direction of propagation. A representative wave height used in the design of bank and shore protection is the significant wave height, H_s. The significant wave height is the average height of the highest one-third of all the waves in a wave train for the time interval (return frequency) under consideration. Thus, the design wave height generally used is the significant wave height, H_s, for a 20-year return period.

Other design wave heights can also be designated, such as H_{10} and H_1. The H_{10} design wave is the average of the highest 10 percent of all waves, and the H_1 design wave is the average of the highest 1 percent of all waves. The relationship of H_{10} and H_1 to H_s can be approximated as follows:

$H_{10} = 1.27\ Hs$ and $H_1 = 1.67\ Hs$

Economics and risk of catastrophic failure are the primary considerations in designating the design wave average height.

(3) Wave Characteristics. Wave height estimates are based on wave characteristics that may be derived from an analysis of the following data:

- Wave gage records
- Visual observations
- Published wave hindcasts
- Wave forecasts
- Maximum breaking wave at the site

(4) Predicting Wind Generated Waves. The height of wind generated waves is a function of fetch length, windspeed, wind duration, and the depth of the water.

(a) Hindcasting -- The U.S. Army Corp of Engineers has historical records of onshore and offshore weather and wave observations for most of the California coastline. Design wave height predictions for coastal shore protection facilities should be made using this information and hindcasting methods. Deep-water ocean wave characteristics derived from offshore data analysis may need to be transformed to the project site by refraction and diffraction techniques. As mentioned previously, it is strongly advised that the Corps technical expertise be obtained so that the data are properly interpreted and used.

(b) Forecasting -- Simplified wind wave prediction techniques may be used to establish probable wave conditions for the design of highway protection on bays, lakes and other inland bodies of water. Wind data for use in determining design wind velocities and durations is usually available from weather stations, airports, and major dams and reservoirs.

The following assumptions pertain to these simplified methods:

- The fetch is short, 120 km or less
- The wind is uniform and constant over the fetch.

It should be recognized that these conditions are rarely met and wind fields are not usually estimated accurately. The designer should therefore not assume that the results are more accurate than warranted by the accuracy of the input and simplicity of the method. Good, unbiased estimates of all wind generated wave parameters should be sought and the cumulative results conservatively interpreted. The individual input parameters should not each be estimated conservatively, since this may bias the result.

The applicability of a wave forecasting method depends on the available wind data, water depth, and overland topography. Water depth affects wave generation and for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation takes place in transitional or shallow water rather than in deep water.

The height of wind generated waves may also be fetch-limited or duration-limited. Selection of an appropriate design wave may require a maximization procedure considering depth of water, wind direction, wind duration, windspeed, and fetch length.

Procedures for predicting wind generated waves are complex and our understanding and ability to describe wave phenomena, especially in the region of the coastal zone, is limited. Many aspects of physics and fluid mechanics of wave energy have only minor influence on the design of shore protection for highway purposes. Designers interested in a more complete discussion on the rudiments of wave mechanics should consult the U.S. Army Corps of Engineers' Shore Protection Manual (SPM), Volume I, 1984.

There is no single theory for the forecasting of wind generated waves for relatively shallow water. Until further research results are available the interim SPM method for wave forecasting in shallow-water represented in Figures 3-27 through 3-36 in the SPM is recommended. This method uses deepwater forecasting relationships and is based on successive approximations in which wave energy is added due to wind stress and subtracted due to bottom friction and percolation.

An initial estimate of wind generated significant wave heights can be made by using Figure 873.2B. If the estimated wave height from the nomogram is greater than 0.6 m, the procedure may need to be refined. It is recommended that advice from the Army Corps of Engineers be obtained to refine significant wave heights, H_s, greater than 0.6 m.

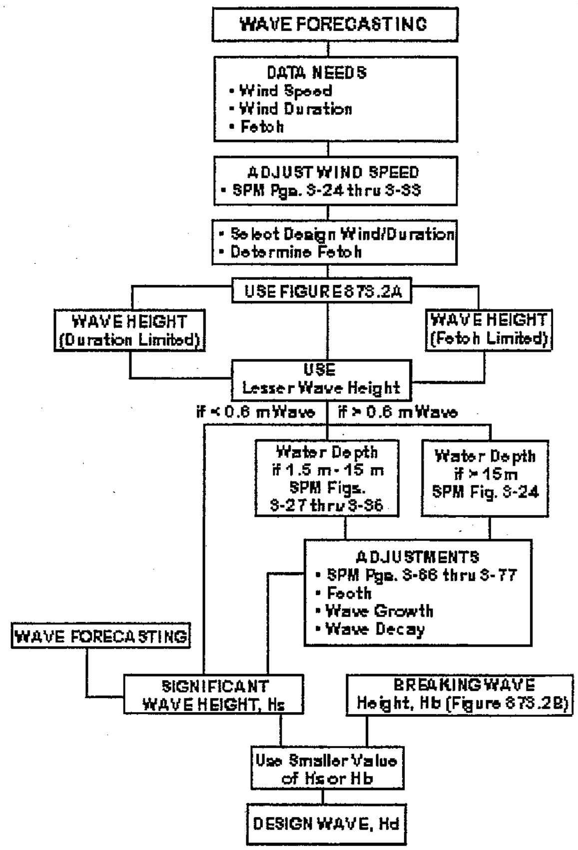
(5) Breaking Waves. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design

stillwater level depth and nearshore bottom slope can support. The design wave height will be the smaller of either the maximum breaker height or the forecasted or hindcasted wave height.

The relationship of the maximum height of breaker which will expend its energy upon the protection, H_b, and the depth of water at the slope protection, d_s, which the wave must pass over are illustrated in Figure 873.2C.

The following diagram, with some specific references to the SPM, summarizes an overly simplified procedure that may be used for highway purposes to estimate wind generated waves and establish a design wave height for shore protection.

Determining Design Wave



(6) Wave Run-up. An estimate of wave run-up, in addition to design wave height, may also be necessary to establish the top elevation of highway slope protection.

Wave run-up is a function of the design wave height, the wave period, bank angle, and the roughness of the embankment protection material. For wave heights of 0.6 m or less wave run-up can be estimated by using Figure 873.2D and appropriate correction factor. The wave run-up height given on the chart is for smooth concrete pavement. Correction factors for reducing the height of run-up for other armor revetment materials are provided in the table. This simple method of estimating wave runup is adequate for most highway projects. The application of more detailed procedures is rarely justified, but if needed they are provided in the

U.S. Army Corps of Engineers manual, Design of Coastal Revetments, Seawalls, and Bulkheads.

(c) Littoral Processes. Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion or erosion that can effect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed.

Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of Engineers' Shore Protection Manual (SPM), Volume I, Chapter 4.

873.3 Armor Protection

(1) General. Armor is the artificial surfacing of bed, banks, shore or embankment to resist erosion or scour. Armor devices can be flexible (self adjusting) or rigid.

(a) Flexible Types.

- Rock slope protection (Standard Plan B13-2).
- Broken concrete slope protection (Standard Plan B13-2).
- Broken concrete, uncoursed.
- Gabions.
- Precast concrete articulated blocks.
- Various reticulated revetment systems.

(b) Rigid Types.

- PCC grouted rock slope protection (Standard Plan B13-2).
- Sacked concrete slope protection (Standard Plan B13-1).
- Concrete slope protection (Standard Plan B13-1).
- Fabric-formed slope protection.
- Air-blown mortar (Standard Plan B13-1).
- Soil cement slope protection.
- Precast concrete cells -- filled.

(c) Other Armor types:

(1) Channel Liners and Vegetation. Temporary channel lining can be used to promote vegetative growth in a drainage way or as protection prior to the placement of permanent armoring. This type of lining is used where an ordinary seeding and mulch application would not be expected to withstand the force of the channel flow. In addition to the following, other suitable products of natural or synthetic materials are available that may be used as temporary or permanent channel liners.

- Excelsior
- Jute
- Paper mats
- Fiberglass roving
- Geosynthetic mats or cells
- Pre-cast concrete blocks or cells

- Brush layering
- Rock riprap in smaller stone sizes

(2) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Crib walls
- Sheet piling
- Sea Walls

(d) General Design Criteria. In selecting the type of flexible or rigid armor protection to use the following characteristics are important design considerations.

(1) The lower limit of armor should be below anticipated scour or on bedrock. If for any reason this is not economically feasible, a reasonable degree of security can be obtained by placement of additional quantities of heavy rock at the toe which can settle vertically as scour occurs.

(2) In the case of slope paving or any expensive revetment which might be seriously damaged by overtopping and subsequent erosion of underlying embankment, extension above design high water may be warranted. The usual limit of extension for streambank protection above design high water is 0.3 to 0.6 m in unconstricted reaches and 0.6 to 1.0 m in constricted reaches.

(3) The upstream terminal can be determined best by observation of existing conditions and/or by measuring velocities along the bank.

The terminal should be located to conform to outcroppings of erosion-resistant materials, trees, shrubs or other indications of stability.

In general, the upstream terminal on bends in the stream will be some distance upstream from the point of impingement or the beginning of curve where the effect of erosion is no longer damaging.

(4) When possible the downstream terminal should be made downstream from the end of the curve and against outcroppings, erosion-resistant materials, or returned securely into the bank so as to prevent erosion by eddy currents and velocity changes occurring in the transition length.

(5) The encroachment of embankment into the stream channel must be considered with respect to its effect on the conveyance of the stream and possible damaging effect on properties upstream due to backwater and downstream due to increased stream velocity or redirected stream flow.

(6) A smooth surface will accelerate velocity along the bank, requiring additional protection at the downstream terminal. Rougher surfaces tend to keep the thread of the stream toward the center of the channel.

(7) Heavy-duty armor used in exposures along the ocean shore may be influenced or dictated by economics, or the feasibility of handling heavy individual units.

(2) Flexible Revetments.

(a) Streambank Rock Slope Protection.

(1) General Features. This kind of protection, commonly called riprap, consists of rock courses placed upon the embankment or the natural slope along a stream. Rock, as a slope protection material, has a number of desireable features which have led to its widespread application.

It is usually the most economical type of revetment where stones of sufficient size and quality are available, it also has the following advantages:

- It is flexible and is not impaired nor weakened by slight movement of the embankment resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the addition of rock where required.
- Construction is not complicated and no special equipment or construction practices are necessary.
- Appearance is natural, and usually acceptable in recreational areas.
- If exposed to fresh water, vegetation may be induced to grow through the rocks adding structural value to the embankment material and restoring natural roughness.
- Additional thickness can be provided at the toe to offset possible scour when it is not feasible to found it upon bedrock or below anticipated scour.
- Wave run-up is less than with smooth types (See Figure 873.2D).
- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a given embankment the following determinations are to be made for the typical section.

- Size of stone (may vary between top and bottom).
- Depth at which the stones are founded (bottom of toe trench).
- Elevation at the top of protection.
- Thickness of protection.
- Need for geotextile and backing material.
- Face slope.

(a) Placement -- Two different methods of placement for rock slope protection are allowed under Section 72 of the Standard Specifications: Placement under Method A requires considerable care, judgment, and precision and is consequently more expensive than Method B. Method A should be specified for heavy duty installations.

Under some circumstances the costs of placing rock slope protection with refinement are not justified and Method B placement can be specified. To compensate for a partial loss and assure stability and a reasonably secure protection, the thickness is increased over the more precise Method A.

(b) Foundation Treatment -- The foundation excavation must afford a stable base on bedrock or extend below anticipated scour.

Terminals of revetments are often destroyed by eddy currents and other turbulence because of nonconformance with natural banks. Terminals should be secured by transitions to stable bank formations, or the end of the revetment should be reinforced by returns of thickened edges.

(c) Embankment Considerations -- Embankment material is not normally carried out over the rock slope protection so that the rock becomes part of the fill. With this type of construction fill material can filter down through the voids of the large stones and that portion of the fill above the rocks could be lost. If it is necessary to carry embankment material out over the rock slope protection a geotextile is required to prevent the loses of fill material.

The embankment fill slope is usually determined from other considerations such as the angle of repose for embankment material, or the normal 1:2 specified for high-standard roads. If the necessary size of rock for the given exposure is not locally available, consideration should be given to flattening of the embankment slope to allow a smaller size stone, or substitution of other types of protection. On high embankments, alternate sections on several slopes should be compared, practically and economically; flatter slopes require smaller stones in thinner sections, but at the expense of longer slopes, a lower toe elevation, increased embankment, and perhaps additional right of way.

(d) Rock Slope Protection Fabric and Rock Backing -- Rock Slope protection fabric and/or rock backing can be used directly on the slope to prevent the erosion of the underlying embankment material or native material through the voids of the rock slope protection. They may be warranted where embankment material is not cohesive, or where the slope protection is subject to wave action. They may not be necessary if the slope protection is graded from fine to coarse from embankment to water exposure as is generally the case with Method B placement. With Method B placement, most of the finer material will naturally settle to the bottom and coarser stones will work to the outside. Consult the District Hydraulics Engineer and/or the District Materials Branch on the need for fabric or rock backing.

When fabric is used with rock slope protection classes 1/2 T or larger, a layer of rock backing is needed to anchor the fabric. Backing material must be sized so that it will not work out through the voids of the large stones overlaying it. For very large classes of protection with severe exposure it may be appropriate to use a smaller class of rock slope protection to perform the backing and bedding function. Determining the need for fabric, rock backing, or multiple layers of rock slope protection requires sound engineering judgment in evaluating the character of the embankment or native material being protected, the slope rate of the embankment, the relative importance and risk of loss of the protected facility as well as the cost of the protective works relative to the protected facility.

Rock slope protection and rock backing material stone sizes, gradings and quality requirements are contained in Section 72-2.02 of the Standard Specifications.

(2) Streambank Protection Design. In the lower reaches of larger rivers wave action resulting from navigation or wind blowing over long reaches may be much more serious than velocity. A 0.6 m wave, for example, is more damaging than direct impingement of a current flowing at 3 m/s.

Well designed streambank rock slope protection should:

- Assure stability and compatibility of the protected bank as an integral part of the channel as a whole. The ideal for stability is a gently curved channel with its outer bank rougher and tougher than the inner bank.
- Connect to natural bank, bridge abutments or adjoining improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.
- Eliminate or ease local embayments and capes so as to streamline the protected bank.
- Consider the effects of backwater above constrictions, superelevations on bends, as well as tolerance of occasional overtopping.
- Not be placed on a slope steeper than 1:1.5. Flatter slopes (see Figure 873.3A) use lighter stones in a thinner section and encourage overgrowth of vegetation, but may not be permissible in narrow channels.
- Use stone of adequate mass to resist erosion, derived from Figure 873.3A or Table 873.3B.
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric and multiple layers of backing should be used where appropriate.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes.
- Reinforce critical zones on outer bends subject to impingement attack, using heavier stones, thicker section, and deeper toe.

(a) Stone Size -- Where current velocity governs, rock size may be estimated by using the nomogragh, Figure 873.3A, and Table 873.3B.

The nomograph is derived from the following formula:

$$W = \frac{0.00002\, V^6\, sg_r \csc^3 (\beta - \alpha)}{(sg_r - 1)^3}$$

Where:

sg_r =	specific gravity of stones.
α =	angle of face slope from the horizontal, see Figure 873.3B.
β =	70° for broken rock.
W =	Weight of minimum stone in lbs.; 2/3 of stones should be heavier.
V =	Velocity of water in ft/sec.

NOTE:
The formula provided above, and the nomograph in Figure 873.3A have not been converted to the Metric System.

Where wave action is dominant, design of rock slope protection should proceed as described for shore protection.

(b) Design Height -- The top of rock slope protection along a stream bank should be carried to the elevation of the design high water. The flood stage elevation adopted for design may be based on an empirically derived frequency of recurrence (probability of exceedance) or historic high water marks. This stage may be exceeded during infrequent floods, but overtopping seldom damages a well-designed pervious revetment.

Design high water should not be based on an arbitrary storm frequency alone, but should consider the cost of carrying the protection to this height, the probable duration and damage if overtopped, and the importance of the facility.

The practice of using an arbitrary height of freeboard as a factor of safety is not logical. For example, an arbitrary 0.6 m freeboard may decrease the probability of overtopping from one that would be caused by a 50-yr flood to one that would be caused by a 60-yr flood in one case, but from one that would be caused by a 50-yr flood to one that would be caused by a 1000-yr flood in another case. Freeboard may be more generous along freeways, on bottleneck routes, on the outside bends of channels, or around critical bridges.

Design high water should be adjusted to the site based on sound engineering judgement.

(b) Rock Slope Shore Protection.

(1) General Features. Rock slope protection when used for shore protection, in addition to the general advantages listed previously for streambank rock slope protection, reduces wave runup as compared to smooth types of protection.

(a) Method A placement is normally specified for shore protection.

(b) Foundation treatment in shore protection may be controlled by tidal action as well as excavation difficulties and production will be limited to only two or three toe or foundation rocks per tide cycle. If toe rocks are not properly bedded, the subsequent vertical adjustment may be detrimental to the protection above. Even though rock is self-adjusting, the bearing of one rock to another may be lost. It is often necessary to construct the toe or foundation to an elevation approximating high tide in advance of embankment construction to prevent erosion of the embankment.

(2) Shore Protection Design.

(a) Stone Size -- For deep-water waves that are shoaling as they approach the protection the required stone size may be determined by Using Chart B, Figure 873.3D.

The nomograph is derived from the following formula:

$$W = \frac{0.003\ d_s^3\ sg_r \csc^3 (\beta - \alpha)}{[(sg_r/sg_w) - 1]^3}$$

Where:

d_s =	maximum depth in feet of water at toe of the rock slope protection, see Figure 873.3C.
sg_r =	specific gravity of stones
sg_w =	specific gravity of water (sea water = 1.0265)
α =	angle of face slope from the horizontal, see Figure 873.3C.
β =	70^o for broken rock
W =	minimum weight in tons of outside stones

NOTE:

The formula provided above, and the Nomograph in figure 873.3D have not been converted to the metric system.

In general, d_s will be the difference between the elevation of the scour line at the toe and the maximum stillwater level. For ocean shore, d_s may be taken as the distance from the scour line to mean sea level plus one-half the maximum tidal range.

If the deep-water waves reach the protection, the stone size may be determined by using Chart A, Figure 873.3D. The nomograph is derived from the following formula:

$$W = \frac{0.00231\ H_d^3\ sg_r \csc^3 (\beta - \alpha)}{[(sgr/sgw) - 1]^3}$$

Where:

H_d =	design wave in feet, (See Index 873.2).

NOTE:

The formula provided above, and the Nomograph in figure 873.3D have not been converted to the metric system.

If in doubt whether waves generated by fetch and wind velocity will be of sufficient size to be affected by shoaling, use both charts and adopt the smaller value.

(b) Dimensions -- Rock should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the estimated depth of probable scour. If the scour depth is questionable, additional thickness of rock may be placed at the toe which will adjust and provide deeper support. In determining the elevation of the scoured beach line the designer should observe conditions during the winter season, consult records, or ask persons who have a knowledge of past conditions.

Wave run-up is reduced by the rough surface of rock slope protection. In order that the wash will not top the rock, it should be carried up to an elevation of twice the maximum depth of water ($2d_s$) or to an elevation equal to the maximum depth of water plus the deep-water wave height ($d_s + H_d$), whichever is the *lower*. See Figure 873.3C.

Consideration should also be given to protecting the bank above the rock slope protection from splash and spray.

Thickness of the protection must be sufficient to accommodate the largest stones. For typical conditions the thickness required for the various sizes are shown on Table 873.3B. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside, and if the wave forces displace these, additional thickness will only add slightly to the time of failure. As the lower portion of the slope protection is subjected to the greater forces, it will usually be economical to specify larger stones in this portion and somewhat smaller stones in the upper portion. The important factor in this economy is that a thinner section may be used for the smaller stones. If the section is tapered from bottom to top, the larger stones can be selected from a single graded supply.

(c) Broken Concrete Slope Protection. Broken concrete salvaged from demolished structure or pavement is a suitable material for slope protection. Method B placement under Section 72 of the Standard Specifications can be used if the size and shape of the broken concrete pieces available approximate those of natural rock. Fairly uniform slabs or other regular shapes can be hand placed on the slope in horizontal courses to form a substantial revetment. This method of placement is expensive and requires judgement in selecting sizes and shapes to avoid filling of open spaces between pieces. Filling of voids with small pieces is of no value to the soundness of the protection. Broken concrete placed in horizontal courses is not self-adjusting but is considered to be flexible since the individual pieces are not bonded together. Details of coursed broken concrete slope protection are shown on Standard Plan B13-2.

A good foundation on bedrock, or that extends below the depth of probable scour, is essential to stability regardless of the placement method.

(d) Gabions. Gabion revetments consist of rectangular wire mesh baskets filled with stone. Size and grade of stone shall be as designated by the district materials department or hydraulics department.

Gabions are formed by filling commercially fabricated and preassembled wire baskets with rock. There are two types of gabions, wall type and mattress type. In wall type the empty cells are positioned and filled in place to form walls in a stepped fashion. Mattress type baskets are positioned on the slope and filled. Wall type revetment is not fully self adjusting but has some flexibility. The mattress type is very flexible. For some locations, gabions may be more aesthetically acceptable than rock riprap. Where larger stone sizes are not readily available and the flow does not abrade the wire baskets, they may also be more cost effective. The range of maximum velocities recommended for use of gabions is 3.0 m/s for sustained flows and 4.5 m/s for intermittent flows.

Refer to HEC-11, Design of Riprap Revetment, Section 6.1.2, for further discussion on the use of gabions for slope protection.

(e) Articulated Precast Concrete. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of articulation, but share common features of flexibility and rapid installation. Most provide for establishment of vegetation within the revetment.

The permeable nature of these revetments permits free draining of the embankment and their flexibility allows the mat to adjust to minor changes in bank geometry. Pre-cast concrete block revetments may be economically justified where suitable rock for slope protection is not readily available. They are generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with cable or synthetic fiber rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 1:1.5 when anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 1:3. An engineering fabric is normally used on the slope to prevent the erosion of the underlying embankment through the voids in the concrete blocks.

Refer to HEC-11, Design of Riprap Revetment, Section 6.2, for further discussion on the use of articulated concrete mattresses.

(3) Rigid Revetments.

(a) PCC Grouted Rock Slope Protection.

(1) General Features. This type of revetment consists of rock slope protection with outer voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3E. It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available, and in other areas to reduce the quantity of rock. Grouting not only protects the stones from the full force of high-velocity water but integrates a greater mass to resist its pressure.

(2) Design Concepts. Grouting will appreciably increase the cost per unit volume

of stone, but the use of smaller stones in PCC grouted rock slope protection than in an equivalent protection using ungrouted stones permits a lesser thickness of protection, which offsets to some extent the cost of PCC.

As this type of protection is rigid without high strength, support by the embankment must be maintained. Slopes steeper than the angle of repose of the embankment are risky, but with rocks grouted in place, little is to be gained with slopes flatter than 1:1.5. Precautions to prevent undermining of embankment are particularly important. The PCC grouted rock must be founded on solid rock or below the depth of possible scour. Ends should be protected by tying into solid rock or forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided as are used with sacked PCC slope protection. If the embankment material is exposed at the top, freeboard is warranted to prevent overtopping.

The volume of concrete required will be that necessary to fill voids. This usually amounts to from 0.25 to 0.33 times the volume of the stone to be grouted.

(3) Specifications. Quality specifications for rock used in PCC grouted rock slope protection are usually the same as for rock used in ordinary rock slope protection. However, as the rocks are protected by the concrete which surrounds them, specifications for specific gravity and hardness may be lowered if necessary. The concrete used to fill the voids is normally 25 mm maximum size aggregate, class B or minor concrete. Except for freeze-thaw testing of aggregates, which may be waived in the contract special provisions, the concrete should conform to the provisions of Standard Specifications section 90, "Portland Cement Concrete."

Size and grading of stone and PCC penetration depth are provided in Standard Specification 72-5.

(b) Sacked-Concrete Slope Protection. This method of protection consists of facing the embankment with sacks filled with concrete. It is an expensive but much used type of revetment. Much hand labor is required but it is simple to construct and adaptable to almost any embankment contour. Economic justification for this type of revetment often depends upon the use of local pit-run material for aggregate, which need not conform to that ordinarily associated with structural concrete. Details of sacked concrete slope protection are shown on Standard Plan B13-1.

Tensile strength is low and as there is no flexibility, the installation must depend almost entirely upon the stability of the embankment for support and therefore should not be placed on face slopes much steeper than the angle of repose of the embankment material. Slopes steeper than 1:1 are rare; 1:1.5 is common. The flatter the slope, the less is the area of bond between sacks. From a construction standpoint it is not practical to increase the area of bond between sacks; therefore for slopes as flat as 1:2 all sacks should be laid as headers rather than stretchers.

Integrity of the revetment can be increased by embedding dowels in adjoining sacks to reinforce intersack bond. A No. 10 deformed bar driven through a top sack into the underlying sack while the concrete is still fresh is effective. At cold joints, the first course of sacks should be impaled on projecting bars that were driven into the last previously placed course. The extra strength may only be needed at the perimeter of the revetment.

Almost all failures of sacked concrete are a result of stream water eroding the embankment material from the bottom, the ends, or the top.

The bottom should be founded on bedrock or below the depth of possible scour. In the case where streambed sands have normal specific gravity a depth of 1.5 m below the flow line of the stream is common practice.

If the ends are not tied into rock or other nonerosive material, cutoff returns are to be provided and if the protection is long, cutoff stubs are built at 10 m intervals, in order to prevent or retard a progressive failure.

Protection should be high enough to preclude overtopping. If the roadway grade is subject to flooding and the shoulder material does not contain sufficient rock to prevent erosion from the top, then pavement should be carried over the top of the slope protection in order to prevent water entering from this direction

For good appearance, it is essential that the sacks be placed in horizontal courses. If the foundation is irregular, corrective work such as placement of entrenched concrete or sacked concrete is necessary to level up the foundation. Refer to "Highways in the River Environment", Section 5.3.4, for further discussion on the use of sacked concrete slope protection.

(c) Concrete Slope Paving.

(1) General Features. This method of protection consists of paving the embankment with portland cement concrete. Details of concrete slope protection are shown on Standard Plan B13-1. Slope paving is used only where flow is controlled and will not over-top the protection.

It is particularly adaptable to locations where high-velocity flow is not detrimental but desirable and the hydraulic efficiency of smooth surfaces is important. It has been used very little in shore protection. On a cubic meter basis the cost is high but as the thickness is generally only 75 to 150 mm, the cost on a basis of area covered will usually be less than for sacked-concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment is such as to warrant the use of mass production equipment such as slip-form pavers.

Due to the rigidity of PCC slope paving, its foundation must be good and the embankment stable. Although reinforcement will enable it to bridge small settlements of the embankment face, even moderate movements could be disastrous. The toe must be on bedrock or extend below possible scour. When this is not feasible without costly underwater construction, rock or PCC grouted RSP have been used as a foundation. A better but much more expensive solution is to place the toe on a PCC wall or piles.

Every precaution must be taken to exclude stream water from pervious zones behind the slope paving. The light slabs will be lifted by comparatively small hydrostatic pressures, opening joints or cracks at other points in a series of progressive failures leading to extensive or complete failure.

Considering the severity of failure from bank erosion or hydrostatic pressure after overtopping, 0.3 to 0.6 m of freeboard above design high water is recommended for this type of revetment. Refer to HEC-11, Design of Riprap Revetment, Section 6.4, for further discussion on the use of concrete slope paving.

(d) Fabric Formed Protection. This method of protection uses sectionalized fabric mattresses filled with a fine aggregate concrete as facing for embankment, river bank, and lake shore. Fabric formed slope paving is a relatively new and cost effective alternative to conventional slope paving methods.

A double-layered envelope of nylon, polypropylene or other suitable synthetic fabric is laid on the area to be protected then filled. Filling consists of pumping a fine aggregate concrete into the inplace fabric mat. Fabric mattresses are made in 50 to 300 mm thickness and in a variety of block sizes and configurations.

Hydrostatic uplift pressure is relieved through filter points or plastic weep tubes inserted in the mats. A filter fabric is used under the mat when relief of hydrostatic pressure is necessary.

A major advantage of this type revetment is the ease of placement. It may be placed in the dry or underwater. The fabric weave is such that it will restrain cement loss while permitting the release of excess mixing water which improves the quality of the concrete.

A secondary advantage is that sufficient silt and soil is often deposited in the mattress indentations to support vegetation. As a result, the root systems that develop help anchor the mattress.

Three most common types of fabric formed mattress configurations are shown in Figure 873.3G.

(e) Soil Cement Slope Protection. This kind of slope protection consists of constructing the outer limit of highway embankments with compacted cement treated material. Standard highway construction equipment may be used to place and compact soil cement slope protection on 1:1.5 to 1:4 slopes. Where rock riprap material is not readily available, soil cement slope protection may be the most economical alternative type revetment. Soil cement is also well suited for use in median ditches or other wide drainage areas that cannot be vegetated.

A wide variety of selected on site soils or local borrow can be used to make durable soil cement slope protection. Any good sandy soil is generally acceptable and depending on the quality of the soil, the percent cement will vary from 7% to 14%. The actual percentage must be determined by laboratory tests. If requested, the District Materials Engineer can provide information on the quality of soil available and recommended cement content.

Either plant mixed or mixed in-place methods may be used. Placed and compacted in horizontal layers, each layer 150 to 200 mm thick and wide enough to be placed with standard highway construction equipment, will result in a stair-step outer face.

Thickness of soil cement slope protection is measured normal to the slope. A 0.3 m thickness is considered adequate for flow velocities up to 3.5 m/s and is a practical minimum thickness where standard methods of constructing highway embankments are used. With variations in design or construction procedures, any desired thickness can be obtained. One such variation is to simultaneously place and compact the horizontal layers of soil cement facing with the embankment. The relationship of facing thickness, t, layer width, w, layer thickness and embankment slope is shown in Figure 873.3H.

Soil cement slope protection is to be founded on nonerodible material or below the depth of possible scour to ensure against undermining of the toe. Consideration should be made to providing cutoff stubs at the ends of the installation to prevent undercutting by waves or current.

In addition to economy, the following are some of the other advantages to using soil cement revetments:

- Slight settlement or other minor movement of the highway embankment does not impair its stability.
- It presents a pleasing appearance, usually acceptable in recreational and environmentally sensitive areas.
- No unusual design considerations are required.
- No unusual construction practices or special equipment are required.
- Properly designed and constructed it is virtually maintenance free.

Refer to "Highways in the River Environment", Section 5.3.10, for further discussion on the use of soil cement slope protection.

(4) Bulkheads. A bulkhead is a steep or vertical structure supporting a natural slope or constructed embankment. As bank and shore protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a slope protection structure, revetment design principles are used, the only essential difference being the steepness of the face slope. As a retaining structure, conventional design methods for retaining walls, cribs and laterally loaded piles are used.

Bulkheads are usually expensive, but may be economically justified in special cases where valuable riparian property or improvements are involved and foundation conditions are not satisfactory for less expensive types of slope protection. They may be used for toe protection in combination with other revetment types of slope protection. Some other considerations that may justify the use of bulkheads include:

- Encroachment on a channel cannot be tolerated.
- Retreat of highway alignment is not viable.
- Right of Way is restricted.
- The force and direction of the stream can best be redirected by a vertical structure.

The foundation for bulkheads must be positive and all terminals secure against erosive forces. The length of the structure should be the minimum necessary, with transitions to other less expensive types of slope protection when possible. Eddy currents can be extremely damaging at the terminals and transitions. If overtopping of the bulkheads is anticipated, suitable protection should be provided.

Along a stream bank, using a bulkhead presumes a channel section so constricted as to prohibit use of a cheaper device on a natural slope. Velocity will be unnaturally high along the face of the bulkhead, which must have a fairly smooth surface to avoid compounding the restriction. The high velocity will increase the threat of scour at the toe and erosion at the downstream end. Allowance must be made for these threats in selecting the type of foundation, grade of footing, penetration of piling, transition, and anchorage at downstream end. Transitions at both ends may appropriately taper the width of channel and slope of the bank. Transition in roughness is desirable if attainable. Refer to "Highways in the River Environment", Section 5.3.11, for further discussion on the use of bulkheads to prevent streambank erosion or failure.

Along a shore, use of a bulkhead presumes a steep lake or sea bed profile, such that revetment on a 1:1.5 or flatter slope would project into prohibitively deep water or permit intolerable wave runup. Such shores are generally rocky, offering good foundation on residual reefs, but historic destruction of the overlying formation attests to the hydraulic power of the sea to be resisted by an artificial replacement. The face of such a bulkhead must be designed to absorb or dissipate as much as practical the shock of these forces. Designers should consult Volume II, U.S. Army Corps of Engineers' Shore Protection Manual, Chapter 6, for more complete information and details on the use of bulkheads, seawalls, and revetments along a shore.

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

(b) Crib walls. Timber and concrete cribs can be used for bulkheads in locations where some flexibility is desirable or permissible. Metal cribs are limited to support of embankment and are not recommended for use as protection because of vulnerability to corrosion and abrasion.

The design of crib walls is essentially a determination of line, foundation grade, and height with special attention given to potential scour and possible loss of backfill at the base and along the toe. Design details for concrete crib walls are shown on Standard Plans C7A through C7G. Concrete crib walls used as bulkheads and exposed to salt water require special provisions specifying the use of coated rebars and special high density concrete. Recommendations from METS should be requested.

Design details for timber crib walls of dimensioned lumber are shown on Standard Plans C9A and C9B. Timber cribs of logs, notched to interlock at the contacts, may also be used. All dimensioned lumber should be treated to resist decay.

(c) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 4.5 m below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(5) Vegetation. Vegetation is the most natural method for stabilization of embankments and channel bank protection. It is relatively easy to maintain, visually attractive and environmentally more desirable. The root system forms a binding network that helps hold the soil. Grass and woody plants above ground provide resistance to the near bank water flow causing it to lose much of its erosive energy.

Erosion control and revegetation mats are flexible three-dimensional mats or nets of natural or synthetic material that protect soil and seeds against water erosion. They permit vegetation growth through the web of the mat material and are used as channel linings where ordinary seeding and mulching techniques will not withstand erosive flow velocities. The designer should recognize that flow velocity estimates and a particular soils resistance to erosion are parameters that must be based on specific site conditions. Using arbitrarily selected values for design of vegetative slope protection without consultation and verification from the Office of Landscape Architecture is not recommended. However, a suggested starting point of reference is Table 862.2 in which the resistance of various unprotected soil classifications to flow velocities are given. Under near ideal conditions, ordinary seeding and mulching methods cannot reasonably be expected to withstand sustained flow velocities above 1.2 m/s. If velocities are in excess of 1.2 m/s, a lining maybe needed (See Table 873.3I).

Temporary channel liners are used to establish vegetative growth in a drainage way or as slope protection prior to the placement of a permanent armoring. Some typical temporary channel liners are:

- Straw
- Excelsior
- Jute
- Woven paper

Vegetative and temporary channel liners are suitable for conditions of uniform flow and moderate shear stresses.

Permanent soil reinforcing mats and rock riprap may serve the dual purpose of temporary and permanent channel liner. Some typical permanent channel liners are:

- Gravel or cobble size riprap
- Fiberglass roving
- Geosynthetic mats
- Polyethelene cells or grids
- Gabion Mattresses

Composite designs are often used where there are sustained low flows of high to moderate velocities and intermediate high water flows of low to moderate velocities. Brush layering is a permanent type of erosion control technique that may also have application for channel protection, particularly as a composite design. Further information on brush layering and fiberglass roving methods and techniques are available from METS.

Design procedures for determining suitable maximum conditions for vegetation, temporary and permanent channel liners are given in Chapter IV, HEC-15, Design of Roadside Channels and Flexible Linings.

873.4 Training Systems

(1) General. Training systems are structures, usually within a channel, that act as countermeasures to control the direction, velocity, or depth of flowing water. As shore protection, they control shoaling and scour by deflecting the strength of currents and waves.

The degree of permeability is among the most important properties of control structures. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce the strength of water velocity, currents or waves.

Training systems of the retard and permeable jetty types are similar in that they are usually extensive or multi-unit open structures like; piling, fencing, and unit frames. They are dissimilar in function and alignment, retards being parallel and groins oblique to the banks. The retard is a milder remedy than jetty construction.

(a) Retard Types. A retard is a bank protection structure designed to check riparian velocity and induce silting and accretion. They are usually placed parallel to the highway embankment or erodible banks of channels on stable gradients. Retards typically take the following forms of construction:

- Fencing - single or double lines
- Palisades - piles and netting
- Timber piling or pile bents
- Steel or timber jacks

Retards are applicable primarily on streams which meander to some extent within a mature valley. Typical uses include the following:

- Protection at the toe of highway embankments that encroach on a stream channel.
- Training and control to inhibit erosion upstream and downstream from stream crossings.
- Control of erosion redeposition of material where progressive embayments are creating a problem.

(1) Fence Type. Fence-type structures are used as retards, permeable or impermeable jetties, and as baffles. These structures can be constructed of various materials.

Fence type retards may be effective on smaller streams and areas subject to infrequent attack, such as overflow areas. Single and double rows of various types of fencing have been used. The principal difference between fence retards and ordinary wire fences is that the posts of retards must be driven sufficiently deep to avoid loss by scour.

Permeability can be varied in the design to fit the requirements of the location for single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height. Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

(2) Piles and Palisades. Retards and jetties may be of single, double, or triple rows of piles with the outside or upstream row faced with wire mesh fencing material, boards or polymeric straps interwoven into a high-strength net. The facing adds to the retarding effect and may trap light brush or debris to supplement its purpose. This type retard is particularly adapted to larger streams where the piles will remain in the water. The number of pile rows and amount of facing may be varied to control the deposition of material. In leveed rivers it is often desirable to discourage accretion so as to not constrict the channel but provide sufficient retarding effect to prevent loss of a light bank protection such as vegetation or light rock facing.

Typical design considerations include:

- If the stream carries heavy debris, the elevation of the top of the pile should be well below the high-water level in order that heavy objects such as logs will pass over the top during normal floods.
- Piles must have sufficient penetration to prevent loss from scour or impact by floating debris or both. This is especially important for the piles at the outer end of jetties. If scour is a problem, the pile may be protected by a layer of rock placed on the streambed. Piles should be long enough to penetrate below probable scour, with penetration of a least 4.5 m in streams with sandy beds and velocities of 3.0 to 4.5 m/s.
- Ends of the system should be joined to the bank in order to prevent parallel

high-velocity flow between the retard and the bank. If the installation is long, additional bank connections may be placed at intervals.

- Facing material should be fastened to the upstream or channel side of the piling in order that the force of the water and impact of debris will not be entirely on the fasteners.

(3) Jacks and Tetrahedrons. Jacks and tetrahedrons are skeletal frames that can be used as retards or permeable jetties. Cables can be used to tie a number of similar units together in longitudinal alignment and for anchorage of key units to deadmen. Struts and wires are added to the basic frames to increase impedance to flow of water directly by their own resistance and indirectly by the debris they collect.

Both devices serve best in meandering streams which carry considerable bed load during flood stages. Impedance of the stream along the string of units will cause deposit of alluvium, especially at the crest and during the falling stage. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, often accompanied by rotation when one leg or side is undercut more than the other. Deposition of the falling stage usually restores the former bed, partially or completely burying the units. In that lowered and rotated poition, they may still be completely effective in future floods.

Retards may be used alone or in combination with other types of slope protection. In combination with a lighter type of armor they may be more economical than a heavier type of protection. They can be used as toe protection for other types of slope protection where a good foundation is impractical because of high water or extreme depth of poor material.

Where new embankment is placed behind the retard consideration should be given to protecting the slope to inhibit erosion until the retard has had an opportunity to function. The slope protection used should promote the establishment of a natural cover, such as discussed under Index 873.3(5), Vegetation.

Retards on tangent reaches of narrow channels may, by slowing the velocity on one side, cause an increase in velocity, on the other. On wider reaches of a meandering stream they may, by slowing a rebounding high velocity thread, have a beneficial effect on the opposite bank. Where the prime purpose of the retard system is to reduce stream bank velocity to encourage deposition of material intended to alter the channel alignment the effect on adjacent property must be assessed. Where deposition of material is the primary function, the service life of the installation is dependent on the deposition rate and the ultimate establishment of a natural retard.

The length of a retard system should extend from a secure anchorage on the upstream end to anchorage on the downstream end beyond the area under direct attack. Since erosion often progresses downstream, this possibility should be considered in determining the planned length.

The top of a retard need not extend to the elevation of design high water. In major rivers and streams where drift is large and heavy it is essential that the retard be low enough to pass debris over the top during stages of high flow.

For further information on retards refer to Section 5.4.3, "Highways in the River Environment".

(b) Jetty Types. A jetty is an elongated artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection or redirection of currents and waves.

This classification may be subdivided with respect to permeability. Impermeable jetties being used to deflect the stream and permeable jetties being used not only to deflect the stream but to permit some flow through the structure to minimize the formation of eddies immediately downstream. Most jetty installations are permeable structures.

Permeable jetties typically take the following forms of construction:

- Palisades -- piles and netting.
- Single and double rows of timber-braced piling.
- Steel or timber jacks.
- Precast concrete, interlocking shapes or hollow blocks.

Impermeable jetties typically take the following forms of construction:

- Guide and spur dikes, earth or rock.
- PCC grouted riprap dikes.
- Single and double lines of sheeting or sheet piling (steel, timber or concrete, framed and braced or on piling).
- Double fence, filled.
- Log or timber cribs, filled.

Impermeable jetties in the form of filled fences and cribs have been used with only limited success. Characteristic performance of these is the development of an eddy current immediately downstream which attacks the bank and often requires secondary protective measures.

Basic principles for permeable jetties are much the same as for retards, the important difference being that they deflect the flow in addition to encouraging deposition. The preceding comment on retards should be considered as related and applicable to jetties when qualified by this basic difference.

Permeable jetties are placed at an angle with the embankment and are more applicable in meandering streams for the purpose of directing or forcing the current away from the embankment. When the purpose is to deposit material and promote growth, the jetties are considered to have fulfilled their function and are expendable when this occurs.

They also encourage deposition of bed material and growth of vegetation. Retards build a narrow strip in front of the embankment, where as permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

The relation between length and spacing of jetties should approximate unity as a general rule to assure complete entrapment and retention of material. The spacing can be increased if the resulting scalloped effect is not detrimental to the desired result.

(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments (Figure 873.4A). They are smooth extensions of the fill slope on the upstream side. Approach embankments are frequently planned to project into wide floodplains, to attain an economic length of bridge. At these locations high water flows can cause damaging eddy currents that scour away abutment foundations and erode approach embankments. The purose of guide dikes is twofold. The first is to align flow from a wide floodplain toward the bridge opening. The second is to move the damaging eddy currents from the approach roadway embankment to the upstream end of the dike.

Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependant on the ratio of flow diverted from the flood plain to flow in the first 30 m of waterway under the bridge. If the use of another shape dike, such as a straight dike, is required for practical reasons more scour should be expected at the upstream end of the dike. The bridge end will generally not be immediately threatened should a failure occur at the upstream end of a guide dike.

Toe dikes are sometimes needed downstream of the bridge end to guide flow away from the structure so that redistribution in the flood plain will not cause erosion damage to the embankment due to eddy currents. The shape of toe dikes is of less importance than it is with upstream guide banks.

For further information on spur dike and guide bank design procedures refer to Section 5.4, "Highways in the River Environment". General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC-18 and HEC-20.

(d) Groins. A groin is a relatively slender barrier structure usually aligned to the primary motion of water designed to trap littoral drift, retard bank or shore erosion, or control movement of bed load.

These devices are usually solid; however, upon occasion to control the elevation of sediments they may be constructed with openings. Groins typically take the following forms of construction:

- Rock mound.
- PCC grouted rock dike.
- Sand filled plastic coated nylon bags.
- Single or double lines of sheet piling.

The primary use of groins is for ocean shore protection. When used as stream channel protection to retard bank erosion and to control the movement of streambed material they are normally of lighter construction than that required for shore installation.

In its simplest or basic form, a groin is a spur structure extending outward from the shore over beach and shoal. A typical layout of a shore protection groin installation is shown in Figure 873.4B.

Assistance from the U.S. Army Corp of Engineers is necessary to adequately design a slope protection groin installation. Designers should consult Volume II, Chapter 6, Section VI, of the Corps' Shore Protection Manual for a more complete discussion on groins. Preliminary studies can be made by using basic information and data available from USGS quadrangle sheets, USC & GS navigation charts, hydrographic charts on currents for the Northeast Pacific Ocean and aerial photos of the area.

For a groin to function satisfactorily, there must be littoral drift to supply and replenish the beach between groins. The groins detain rather than retain the drift and soon will be ineffective unless there is a steady source of replenishment. A new groin installation will starve the downcoast beach, temporarily at least, and permanently if the supply of drift is meager. Reference is made to the Army Corps of Engineers' Shore Protection Manual, Volume 1, Chapter 4, for more detailed information on the littoral process.

Factors pertinent to design include:

(1) Alignment. Factors which influence alignment are effectiveness in detaining littoral drift, and self-protection of the groin against damage by wave action.

A field of groins acts as a series of headlands, with beaches between each pair aligned in echelon, that is, extending from outer end of the downdrift groin to an intermediate point on the updrift groin (Figure 873.4C) The offset in beach line at each groin is a function of spacing of groins, volume of littoral drift, slope of sea bed and strength of the sea, varying measurably with the season. Length and spacing must be complementary to assure continuity of beach in front of a highway embankment.

A series of parallel spurs normal to the beach extending seaward would be correct for a littoral drift alternating upcoast and downcoast in equal measure. However, if drift is predominantly in one direction the median attack by waves contributes materially to the longshore current because of oblique approach. In that case the groin should be more effective if built oblique to the same degree. Such an alignment will warrant shortening of the groin in proportion to the cosine of the obliquity (Fig. 873.4C).

Conformity of groin to direction of approach of the median sea provides an optimum ratio of groin length to spacing, and the groin is least vulnerable to storm damage. Attack on the groin will be longitudinal during a median sea and oblique on either side in other seas.

(2) Grade. The top of groins should be parallel to the existing beach grade. Sand may pass over a low barrier. The top of the groin should be established higher than the existing beach, say 0.6 m as a minimum for moderate exposure combined with an abundance of littoral drift, to 1.5 m for severe exposure and deficiency of littoral drift.

The shore end should be tapered upward to prevent attack of highway embankment by rip currents, and the seaward end should be tapered downward to match the side slope of the groin in order to diffuse the direct attack of the sea on the end of the groin.

(3) Length and Spacing. The length of groin should equal or exceed the sum of the offset in shoreline at each groin plus the width of the beach from low water (LW) to high water (HW) line (Figure 873.4C). The offset is approximately the product of the groin spacing and the obliquity (in radians) of the entrapped beach. The width of beach is the product of the slope factor and the range in stage. The relation can be formulated:

$L = ab + rh$, where

L = Length of groin, (m)

a = obliquity of entrapped beach in radians,

b = beach width between groins, (m)

r = reciprocal of beach slope,

h = range in stage, (m)

For example, with groins 120 m apart, obliquity up to 20 degrees, on a beach sloping 1:10 with a tidal rage of 3 m,

L = .35 x 120 + 10 x 3 = 72 m

The same formula would have required L = 118 m for 250 m spacing, reducing the aggregate length of groins but increasing the depth of water at the outer ends and the average cost per meter. For some combination of length and spacing the total cost will be a minimum, which should be sought for economical design.

If groins are too short, the attack of the sea will still reach the highway embankment with only some reduction of energy. Some sites may justify a combination of short groins with light revetment to accommodate this remaining energy.

(4) Section. The typical section of a groin is shown in Figure 873.4D. The stone may be specified as a single class, or by designating classes to be used as bed, core, face and cap stones.

Face stone may be chosen one class below the requirement for revetment by Chart A or B (Figure 873.3D). Full mass stone should be specified for bed stones, for the front face at the outer end of the groin, and for cap stones exposed to overrun. Core stones in wide groins may be smaller.

Width of groin at top should be at least 1.5 times the diameter of cap stones, or wider if necessary for operation of equipment. Side slopes should be 1:1.5 for optimum economy and ordinary stability. If this slope demands heavier stone than is available, side slope can be flattened or the cap and face stones bound together with grout as shown in Figure 873.3E.

(e) Baffle. A baffle is a pier, vane, sill, fence, wall or mound built on the bed of a stream to control, deflect, check or disturb the flow or to float on the surface to dampen wave action.

Baffles typically take the following forms of construction:

- Single or multiple lines of fence.
- Drop Structures (gabions, rock, concrete, etc.).
- Dikes of earth or rock.
- Floating boom.

These devices may vary in magnitude from a check dam on a small stream to a system of training dikes or permeable jetties for deflecting or directing flow. When using fences, palisades, or dikes as deflectors along the more mature valleys or meandering streams, the potential erosion to previously unexposed areas, threat to adjacent property, eddy currents and possibility of scour should all be assessed. When used as a collecting system to control and direct the flow to new or existing drainage facilities or to bridge openings, the alignment of the installation should be developed as a series of curves and intervening tangents guiding the stream through transitions to maintain smooth and steady flow. The surface and curvature of the training device should be governed by the natural or modified velocity.

Drop structures or check dams are an effective means of gradient control. They may be constructed of rock, gabions, concrete, timber, sacked concrete, filled fences, sheet piling or combinations of any of the above. They are most suited to locations where bed materials are relatively impervious otherwise underflow must be prevented by cutoffs. Refer to "Highways in the River Environment", Section 5.4.8, for further discussion on the use of drop structures.

Floating booms are effective protection against the smaller wave actions common to lakes and tidal basins. Anchorage is the prime structural consideration.

873.5 Design Check List

The designer should anticipate the more significant problems that are likely to occur during the construction and maintenance of channel and shore protection facilities. So far as possible, the design should be adjusted to eliminate or minimize those potential problems.

The logistics of the construction activity such as access to the site, on-site storage of construction materials, time of year restrictions, environmental concerns, and sequence of construction should be carefully considered during the project design. The stream and shoreline morphology and their response to construction activities are an integral part of the planning process. Communication between the designer and those responsible for construction administration as well as maintenance are important.

Channel and shore protection facilities require periodic maintenance inspection and repair. Where practicable, provisions should be made in the facility design to provide access for inspection and maintenance.

The following check list has been prepared for both the designer and reviewer. It will help assure that all necessary information is included in the plans and specifications. It is a comprehensive list for all types of protection. Items pertinent to any particular type can be selected readily and the rest ignored.

1. Location of the planned work with respect to:

   - The highway.
   - The stream or shore.
   - Right of way.

2. Datum control of the work, and relation of that datum to gage datum on streams, and both MSL and MLLW on the shore.

3. A typical cross section indicating dimensions, slopes, arrangement and connections.

4. Quantity of materials (per meter, per protection unit, or per job).

5. Relation of the foundation treatment with respect to the existing ground.

6. Relation of the top of the proposed protection to design high water (historic, with date; or predicted, with frequency).

7. The limits of excavation and backfill as they may affect measurement and payment.

8. Construction details such as weep holes, rock slope protection fabrics, geocomposite drains and associated materials.

9. Location and details of construction joints, cut-off stubs and end returns.

10. Restrictions to the placement of reinforcement.

11. Connections and bracing for framing of timber or steel.

12. Splicing details for timber, pipe, rails and structural shapes.

13. Anchorage details, particularly size, type, location, and method of connection.

14. Size, shape, and special requirements of units such as precast concrete shapes and other manufactured items.

15. Number and arrangement of cables and details of fastening devices.

16. Size, mass per unit area, mesh spacing and fastening details for wire-fabric or geosynthetic materials.

17. On timber pile construction the number of piles per bent, number of bents, length of piling, driving requirements, cut-off elevations, and framing details.

18. On fence-type construction the number of lines or rows of fence, spacing of lines, dimensions of posts, details of bracing and anchorage ties, details of ties at end.

19. The details of gabions and the filling material.

20. The size of articulated blocks, the placement of steel, and construction details relating to fabrication.

21. The corrosion considerations that may dictate specialty concretes, coated reinforcing, or other special requirements.

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URBAN DRAINAGE AND FLOOD CONTROL DISTRICT

TECHNICAL REVIEW GUIDELINES
for
GRAVEL MINING ACTIVITIES

Within or Adjacent to 100-year Floodplains

Prepared In Cooperation With:

Adams County
Colorado Rock Products Association

December 1987
Project Consultant: Wright Water Engineers, Inc.

TECHNICAL REVIEW GUIDELINES
FOR
GRAVEL MINING ACTIVITIES
Within or Adjacent to 100-Year Floodplains

December 1987

Urban Drainage & Flood Control District
2480 West 26th Avenue, Suite 156B
Denver, Colorado 80211
(303) 455-6277

Prepared in Cooperation with:

ADAMS COUNTY
COLORADO ROCK PRODUCTS ASSOCIATION
Project Consultant:
WRIGHT WATER ENGINEERS, INC.

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CHAPTER 1.0

INTRODUCTION

1.1 TITLE

These technical review guidelines together with all future amendments shall be known as "Technical Review Guidelines for Gravel Mining Activities Within or Adjacent to 100-year Floodplains" (hereafter called "Guidelines").

1.2 JURISDICTION

These Guidelines shall apply to all proposed gravel mining operations, or any other excavation below the normal water level of the 100-year floodplain, or within 500 feet of the low flow channel if outside the 100-year floodplain of the South Platte River and its tributaries within the Urban Drainage & Flood Control District (hereinafter called the "District").

1.3 PURPOSE

Presented in these Guidelines are the minimum requirements proposed for gravel mining operations located in or adjacent to 100-year floodplain within the District. It is the intent of these Guidelines to help protect property owners adjacent to rivers and streams from unreasonable hazard resulting from river or stream instabilities. These Guidelines were originally designed to protect South Platte River and related properties and infrastructure from adverse impacts resulting from the extraction of gravel in or near the river by providing guidelines for maintaining its stability and protecting overbank areas from catastrophic failure. As a result these Guidelines are directly applicable to operations along the South Platte River. However, they should also provide initial guidance for operations along its tributaries, but may require modifications on a case-by-case basis to recognize differences in hydrology and site conditions. These Guidelines are also intended to provide a consistent bases for District review and action in matters dealing with gravel mining within or adjacent to 100-year floodplains.

1.4 ENACTMENT AUTHORITY

These Guidelines are adopted by the Urban Drainage & Flood Control District's Board of Directors as District's review guidelines. These Guidelines are not intended to supersede the Rules and Regulations for Mining Operations in the State of Colorado as adopted by the Colorado Mined Land Reclamation Board (34-32-108, CRS) but rather to supplement them. All referrals by all general purpose governments and special districts within the District will be reviewed using these Guidelines as the basis for comment and, if necessary, action before the Colorado Mined Lane Reclamation Board where existing or proposed mining activities threaten the long-term stability of the South Platte River.

1.5 DEFINITION OF TERMS AND ABBREVIATION

1.5.1 Definitions. As used in these Guidelines, the following definitions shall apply:

Adjacent	Within 400 feet when used in reference to the South Platte River. The 400 foot distance shall be measured from the top of the river bank to the top of the gravel pit bank or from the Master Plan alignment boundary to the top of the gravel pit bank, whichever is greater.

Berms	Areas of native material or fill material separating a river or stream from the overbank gravel pits or one overbank gravel pit from another overbank gravel pit.
County	A county totally or partially within District's boundaries.
District	Urban Drainage & Flood Control District
FHAD	A Flood Hazard Area Delineation report prepared by District.
Floodplain	Land adjacent to a watercourse which is subject to flooding as a result of the occurrence of the 100-year or 1 percent frequency flood of a watercourse.
Flood Profile	A graph or a longitudinal profile showing the relationship of the water surface elevation of a flood event to location along a stream or river.
Floodway	That area of the floodplain required for a reasonable passage or conveyance of the 100-year flood and which will convey the flood flows with not more than 0.5 foot rise in the water surface elevation based on the assumption that there will be an equal degree of encroachment onto the overbank conveyances on both sides of the floodplain.

Guidelines	Technical Review Guidelines for gravel mining activities within or adjacent to 100-year floodplains.
Guidelines for Vegetation	Godi, Donald H., and Associates, "Guidelines for Development and Maintenance of Natural Vegetation" for Urban Drainage & Flood Control District, Denver, Colorado, July, 1984.
High Water Line	The elevation used to determine the volume of overbank gravel pits. The high water line elevation shall be the spillway elevation when only one spillway is used or the average of both spillways when two are used.
Hydraulic Radius	The area of flow divided by the length of the section exposed to water.
Instream Mining	Mines located in the channel of the South Platte River or any of its tributaries, or gravel mines which were originally out of the channel if it is proposed to relocate the river or stream through those gravel mines in the future.
Interior Banks	Banks which face the interior portion of gravel pits.
Invert	The lowest point in the river or stream channel.

Jetties (Groins)	Bank stabilization technique involving the placement of strips of stabilized fill projecting into the river channel from the banks.
Lateral Berms	Berms constructed or left in place between pits which are perpendicular to the general direction of flow of a river or a stream.
Local Government	Any general purpose government or a special district totally or partially within District boundaries.
Low Flow Channel	That portion of the channel which is subjected to continuous flow or frequent flows and where the flows are concentrated.
Master Plan	The South Platte River Major Drainageway Planning from Chatfield Dam to Baseline Road Phase B report prepared in June of 1985 or any other Major Drainageway Plan prepared by the District.
Pitside Banks	The interior bank of a gravel pit located adjacent to a river or a stream.
Reach	A hydraulic engineering term to describe longitudinal segments of a stream or river.

Riprap	Broken stone or boulders placed compactly or irregularly on earth or gravel surfaces to protect against the erosive action of water.
River Banks	The banks of a river or a stream.
Riverside Berms	Berms immediately adjacent to a river or a stream (see berms).
Rubble	Inert materials such as concrete blocks, broken concrete, or concrete pavement with a specific gravity of 2.3 or greater which can be used in lieu of rock riprap for erosion protection.
Safety Factor	The ratio of forces resisting movement to those attempting to initiate movement. (abbreviated SF).
Setback	The distance between any property line and the wall or support of structure. Setbacks are not applicable to fences except where specifically indicated.
Thalweg	The lowest portion of a river or a stream channel.
Urban Storm Drainage Criteria Manual	(Abbreviated USDCM) The latest version of the Criteria of the Urban Drainage and Flood Control District.

1.5.2 Abbreviations

As used in these Guidelines, the following abbreviations shall apply:

A_p	Area of gravel pit at the high water line, in square feet.
B	Bottom width of channel, in feet.
D_{50}	Median riprap particle size, in feet.
INV	River or stream invert (i.e., thalweg).
L	Length of the channel crossing between two curves, in feet.
L_H	Horizontal spacing between jetties, in feet.
L_m	Meander length of two consecutive bends and crossings, in feet.
L_s	Length of side channel spillway crest, in feet.
L_{sl}	Length of lateral berm spillway, in feet.
Min	Minimum
n	Stability Factor
NWL	Normal water level

R	Hydraulic radius at normal depth of flow down pitside slope, in feet.
R_H	Horizontal radius of the channel centerline, in feet.
S	Face slope of pitside bank, in feet per foot.
SF	Safety factor
SPR	South Platte River
S_s	Specific gravity of riprap particles.
USDCM	Urban Storm Drainage Criteria Manual.
W.S.	Water surface
	Specific weight of water (equal to 62.4 lbs/ft^3).
	Deflection angle of the horizontal curve, in degrees.
	Face slope of pitside bank, in degrees.
$_s$	Tractive force, in lbs/ft^2.
Ø	Angle of repose of pitside bank construction materials, in degrees.

CHAPTER 2.0

OFF RIVER GRAVEL MINING

2.1 TYPES OF RIVER ALIGNMENT

The centerline alignment of the South Platte River, and possibly its tributaries affects stability. This section describes three classifications of river alignment and how each affects the stability of the South Platte River. This description may or may not apply to tributary streams.

2.1.1 Existing Unstable Alignment

The alignment of the South Platte River through Adams County in 1985 is considered unstable. Studies of aerial photos taken since 1937 clearly show the river continuously shifting its main channel horizontally, sometimes as much as 1,700 feet. In more recent years, the river also has exhibited a tendency to lower its thalweg through degradation. As it exists today, the river channel is considered unstable and capable of significant lateral and vertical movement.

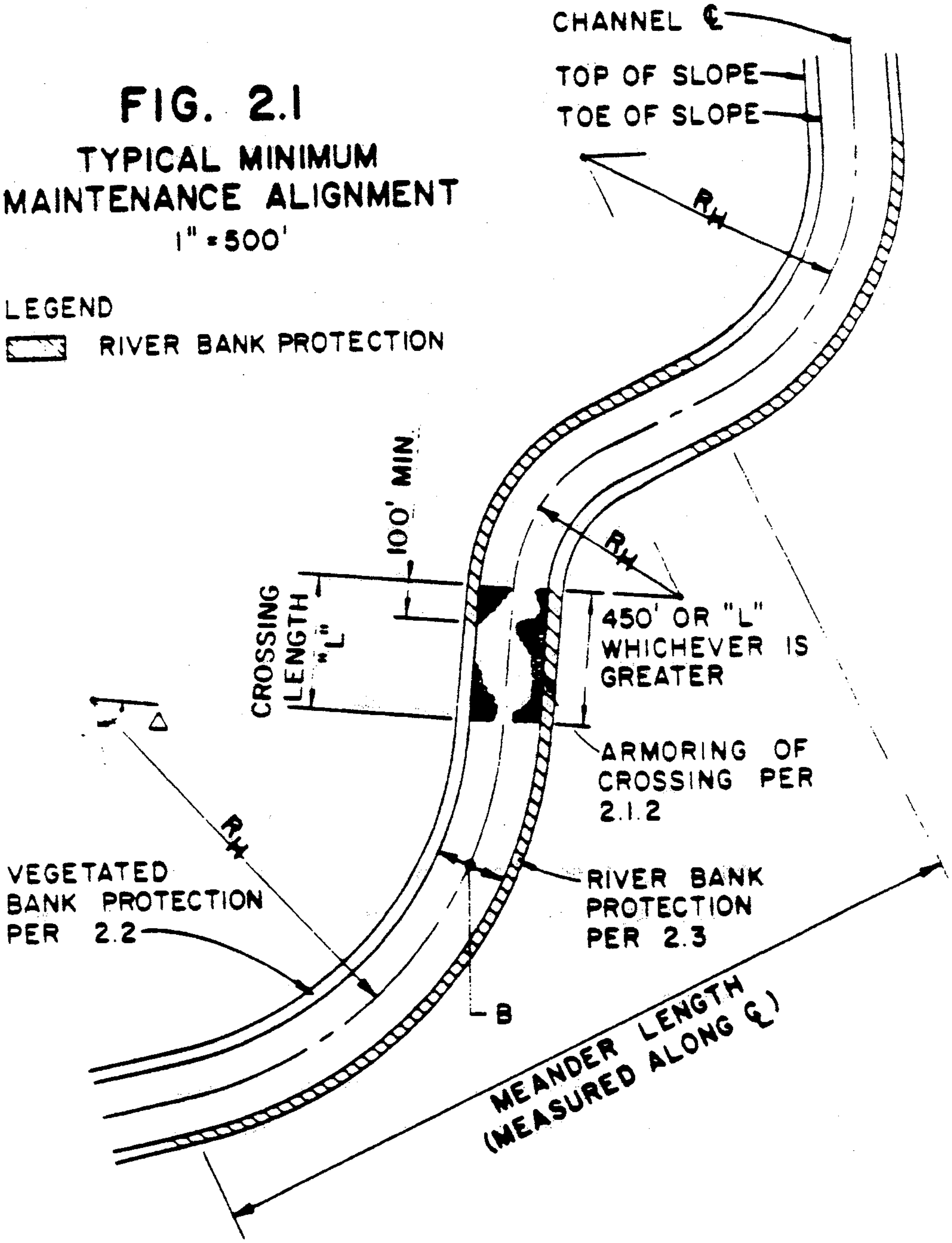
As a minimum, if the gravel mining operator elects to preserve the river in its current unstable alignment, the river banks shall be regraded to have a 3H:1V side slope and then revegetated per section 2.2 of these Criteria. Sufficient river channel width must be provided to convey the 100-year flood to 88th Avenue and the 10-year flood downstream of 88th Avenue as described in the Master Plan. It is the intent to provide this capacity by natural erosion phenomena or excavation, if necessary.

2.1.2 Master Plan Alignment

The alignment shown in the South Platte River Phase B Master Plan, prepared by Wright Water Engineers for the Urban Drainage

FIG. 2.1

TYPICAL MINIMUM
MAINTENANCE ALIGNMENT
1" = 500'



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thick to protect the river profile from degrading and shall consist of Type L or larger riprap as defined in the USDCM, or a gradation of cobble equivalent to Type L riprap.

The minimum maintenance alignment channel is designed to control the 10-year flood. However, all berms will be designed to withstand the 100-year flood without failure during overtopping. If a 100-year berm is requested by the Adams County Engineer, then the higher standard shall supersede the recommended standard, and all design shall conform accordingly. If the operator chooses to elevate the unprotected riverside berm above the 10-year level, the berm shall be protected in a manner consistent with the protection requirements for a minimum maintenance channel alignment for the entire height of the berm.

2.2 REVEGETATION OF BERMS

All berms shall be revegetated with native type grasses and vegetation to control soil erosion, to improve aesthetics and to provide habitats for birds and small animals. Revegetation shall be required on all surfaces of the berms that are not protected using the techniques described in Sections 2.3.1 - Riprap, 2.3.2 - Riprap with Vegetation, 2.4.1 - Riprap, 2.4.2 - Riprap with Vegetation and 2.4.3 - Soil Cement.

Revegetation shall occur as soon after the construction of the berms as possible. The revegetation shall take place in accordance with the requirements of the District as specified in the Guidelines for Vegetation.

2.2.1 Revegetation of River Banks

Revegetation of all river banks not otherwise protected using techniques described in Sections 2.3.1 - Riprap and 2.3.2 - Riprap with Vegetation shall be performed in accordance with the guidelines contained in "Guidelines for Development and Maintenance of Natural Vegetation" hereafter called Guidelines for Vegetation published by the District in 1984. This document contains guidelines for site preparation, selection of grass species, landscape plantings, seed bed preparation, seeding, mulching, fertilizing, irrigation for establishment of vegetation and follow-up maintenance. All vegetated slopes not incorporating riprap should be constructed no steeper than 4H:1V.

2.2.2 Revegetation of Pitside Banks and Top of Berm

Revegetation of pitside banks and tops of berms not otherwise protected with riprap or soil cement shall be done in accordance with the Guidelines for Vegetation (see Section 2.2.1 - Revegetation of Riverbank Berms) and the minimum requirements of the Colorado Mined Land Reclamation Division for reclamation of gravel mines. The allowable slope should be no steeper than 4H:1V.

2.2.3 Revegetation of Riprap With Vegetation Areas

In areas to be protected using the riprap with vegetation technique as described in 2.3.2 - Riprap with Vegetation, it is important that soil fill the voids between the rocks completely. The placement of the rock, soil, and topsoil mixture shall be in accordance with the requirements of Section 2.3.2 - Riprap with Vegetation. Live willow slash shall be mixed into the upper layers of the soil-rock mixture to encourage growth of pioneer shrub-type vegetation. The seed bed preparation and all other steps needed to establish a healthy native grass cover shall be performed in accordance with the Guidelines for Vegetation unless specifically modified in this document. The steepest allowable slope shall be 2.5H:1V.

The placement of vegetation on channel banks, with or without riprap, shall be limited to the area above the 2-year water surface elevation only. Below this elevation riprap will be required.

2.3 RIVERBANK PROTECTION

In areas where the water currents in the river will have a tendency to erode the river banks, it is necessary to provide additional bank protection over what is provided simply by bank vegetation. The next three subsections provide the criteria for the design and installation of three types of allowable bank protection for the South Platte River in Adams County. This type of protection is required for the protected areas of the existing channel alignment conditions and for the areas defined under Sections 2.1.2 - Master Plan Alignment and 2.1.3 - Minimum Maintenance Alignment for the minimum maintenance alignment river channel configurations.

Riverbank protection must be provided at the time that mining occurs adjacent (within 400 feet) to the river channel. It will not be necessary for gravel mining operators to complete improvements on the entire reach along their permit boundary prior to the commencement of mining activities unless the areas are experiencing ongoing bank erosion as determined by the Adams County Engineer or District. However, the portions to be stabilized shall precede the commencement of mining activities along any portion adjacent (within 400 feet) of the river.

2.3.1 Riprap

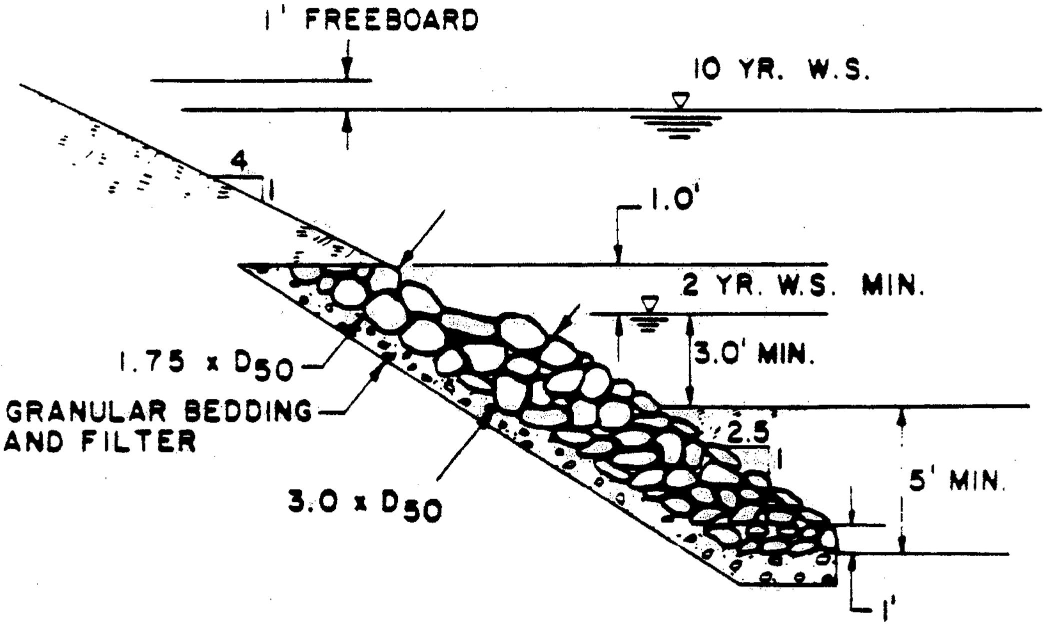
Riprap along riverbanks shall be designed in accordance with the provision of the USDCM except that the banks along the rivers and streams protected with riprap or riprap with vegetation are to be sloped at no steeper than 2.5H:1V up to the 2-year flood level and 4H:1V above the 2-year flood level. All riprap sizing is to be based on the velocities associated with the 100-year event. The rock will be extended vertically downward beyond the toe of the slope at least 5 feet below the channel thalweg (Phase B Master Plan low flow channel bottom). All type VL and L riprap shall be buried and revegetated as required by the USDCM. Larger rock may be buried at the discretion of the owner. If rock is to be buried, the provisions of Sections 2.3.2 - Riprap with Vegetation shall be followed. Typical rock and bedding placement details are illustrated on Figure 2.2.

2.3.2 Riprap with Vegetation

Rock used in conjunction with vegetation shall be sized in accordance with the requirements of the USDCM. The velocities associated with the 100-year flood are to be used for determining rock sizes. The sizing of riprap shall be in accordance with Section 2.3.1 - Riprap.

Soil, seed and rock shall be mixed into one homogeneous mixture with all voids filled. Seed mixtures used in this mass are to conform with the Guidelines for Vegetation; however, they should also include dormant willow slash to encourage the growth of willow shrubbery in addition to native grasses as required. Willow slash shall be included at a rate of 20 pounds of dormant willow slash per 100 pounds of live seed of grass mixture. The combination of seed, willow slash, soil,

FIG. 2.2
TYPICAL RIPRAP SLOPE PROTECTION
(MINIMUM MAINTENANCE ALIGNMENT)



and riprap shall be mixed as needed to provide a homogeneous mass and in a separate operation, transported and placed on the slope to be revegetated. Mixing shall be accomplished using mechanical equipment such as dozers, loaders or backhoes, or by batch mixing using equipment designed specifically for such an operation.

Placement of the mass shall be accomplished in a manner to assure that no segregation of the various constituents occurs. No bedding will be allowed under riprap with vegetation. Subsequent to the placement of the mass, a 6-inch to 12-inch layer of topsoil will be placed above the rock. A typical detail of this installation is shown in Figure 2.3.

2.3.3 Jetties (Groins)

Jetties may be used to protect river banks when the river is in its existing, unstable alignment. The jetties must be installed in such a manner that they not only control river flows during normal and minor flood events, but also protect against embankment erosion during larger events.

The geometric characteristics will conform to those shown on Figure 2.4. The jetties will project from the toe of the channel bank a minimum of 15 feet and a maximum of 25 feet. This distance will be measured from the toe of the bank normal slope (after regrading to 4H:1V) where it intersects the channel invert. The spacing of jetties shall be such that the ratio of centerline spacing to projection is less than or equal to 3. All jetties will extend into the bank a minimum of 15 feet as measured from the toe of the bank slope. Jetties will be constructed perpendicular to the centerline of the channel or with an orientation in the downstream direction of up to 45 degrees.

FIG. 2.3

TYPICAL RIPRAP AND VEGETATION SLOPE PROTECTION
(MINIMUM MAINTENACE ALIGNMENT)

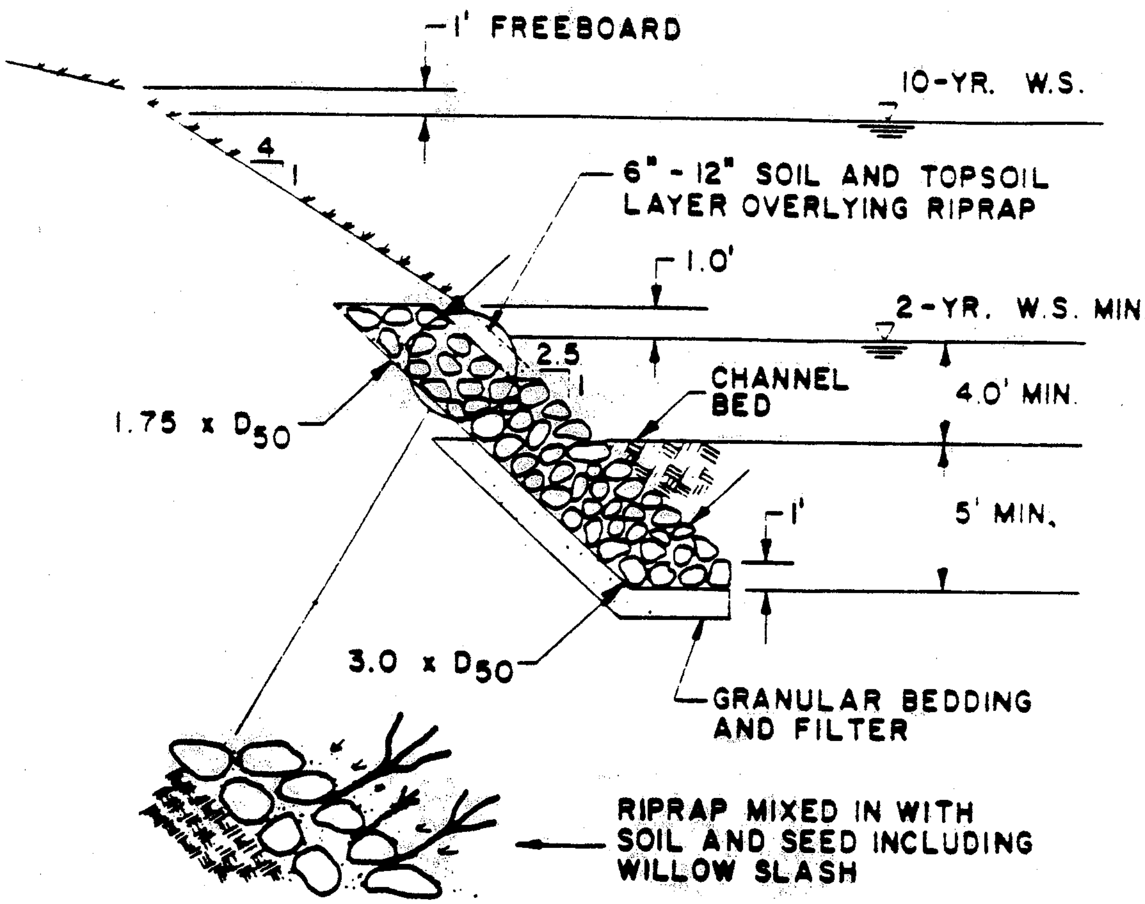


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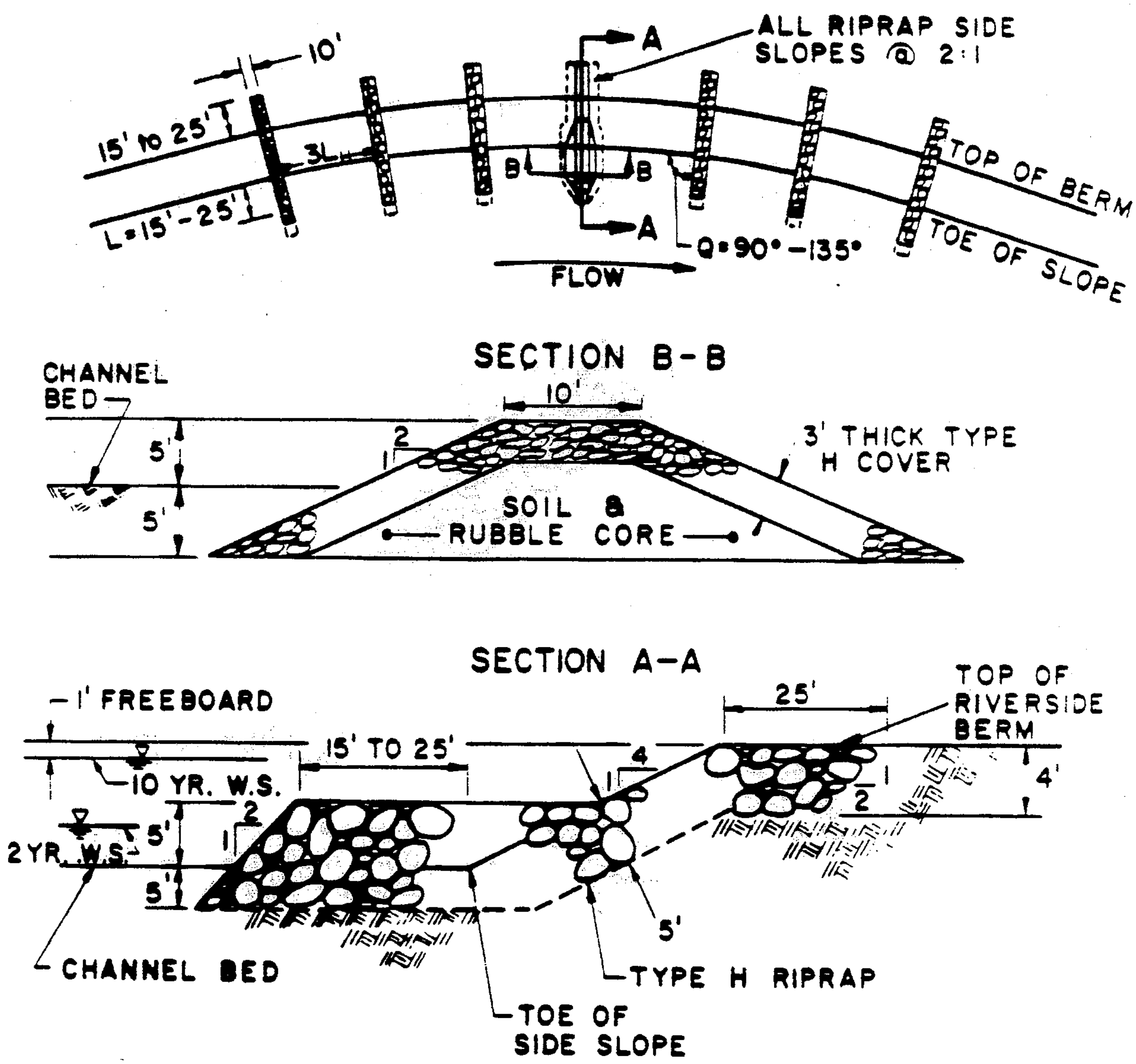
FIG. 2.4

JETTY SLOPE PROTECTION



All jetties will be constructed of Type H riprap (USDCM classification) and toed into the existing riverbed a minimum of 5 feet. The core of the jetties may be constructed from a mixture of concrete rubble with no exposed rebar and soils. However, the surface of the jetty must be Type H rock. Banks between jetties are to be revegetated in accordance with the requirements of Section 2.2.1 - Revegetation of River Banks.

2.4 PITSIDE BANK PROTECTION

The interior banks of reclaimed gravel pits require additional erosion protection beyond revegetation if the berm is to be protected from loss during large flood events. Protection of the pitside banks permits the reduction of top width of the berm provided it is accomplished using the requirements specified in the next five subsections.

Construction of the pitside bank protection will occur coincidentally with the construction of river bank erosion protection measures. That is, when mining activities abut against a river or stream, erosion protection of both the river bank and the pitside bank must be undertaken. In areas experiencing ongoing bank erosion as determined by District, it may be necessary to provide river bank protection prior to the commencement of mining operations, however, it will not be necessary to construct pitside bank protection measures until such time as gravel extraction is completed.

2.4.1 Riprap

When riprap is used to stabilize the pitside slope, the rock shall be placed at a slope no steeper than 2.5H:1V. Riprap sizing will be in accordance with the safety factor method. The minimum safety factor shall be 1.25.

$$SF = \frac{\cos \theta \tan \phi}{n \tan \phi + \sin \theta}$$

$$n = \frac{21\ T_s}{(S_s - 1)\ D_{50}}$$

$$T_s = \gamma RS$$

In which,

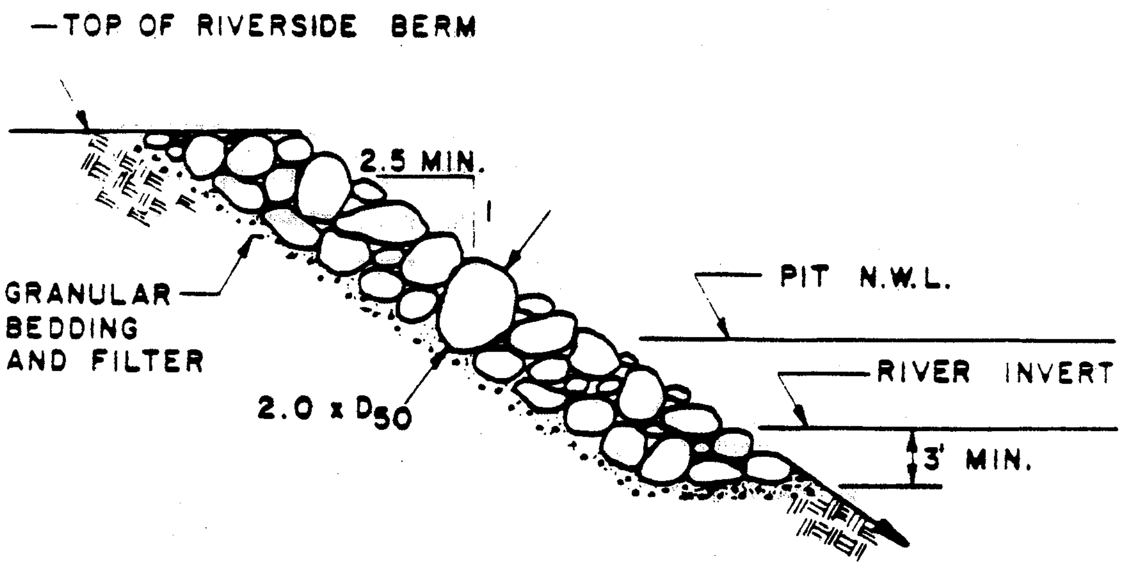
- θ = face slope of pitside bank, in degrees
- ϕ = angle of repose of pitside bank construction materials in degrees
- S_s = specific gravity of riprap particles
- γ = specific weight of water = 62.4 lbs/ft^3
- D_{50} = median riprap particle size, in feet
- R = hydraulic radius at normal depth of flow down pitside slope, in feet
- S = face slope of pitside bank, in feet per foot
- SF = Safety factor
- T_s = Tractive force

Once the D_{50} is established using the above described procedure, a gradation is to be selected. The gradation shall follow the general provisions specified in the USDCM. Material shall be placed in accordance with the criteria specified in the USDCM and as shown on Figure 2.5.

2.4.2 Riprap with Vegetation

The methodology presented in Section 2.4.1 - Riprap shall be used for sizing riprap when used in conjunction with vegetation on pitside banks. When riprap with vegetation is to be used to stabilize the pitside bank of riverside berms, the placement shall be in accordance with the requirements presented in Section 2.3.2 - Riprap with Vegetation.

FIG. 2.5
TYPICAL RIPRAP SLOPE PROTECTION
(PITSIDE SLOPE)



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2.4.3 Soil Cement

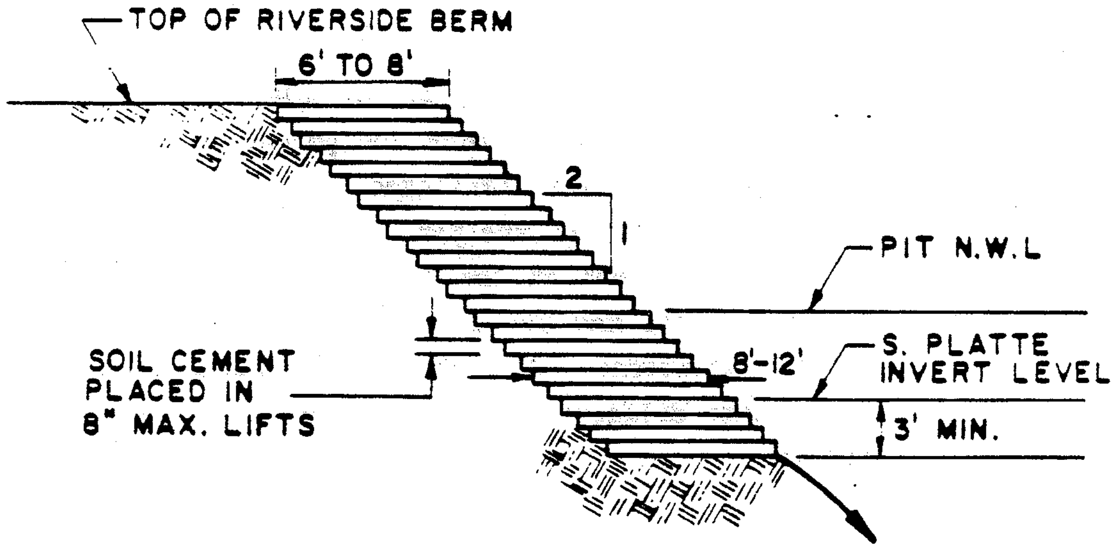
Soil cement may be used to stabilize the pitside slopes of riverside berms against erosion during overtopping. The soil cement must be placed on the pitside slope face in maximum 8-inch lifts at a width of 6 to 8 feet. The general geometric configuration of the soil cement stabilization will conform to the requirements as shown on Figure 2.6.

Soil cement will consist of a mixture of Portland cement, native soils (if conforming to gradation limitations) and water. Native soils may be used if they conform to the following three requirements: (1) the soil contains no material retained on a 2-inch sieve; (2) at least 55 percent of the material passes the No. 4 sieve; and (3) between 5 percent and 35 percent of the material pass the No. 200 sieve. The soil cement mixture should have a minimum of 10 percent Type 2 Portland cement as measured by weight. The moisture content of the soil should range between 10 and 15 percent. In order to determine the proper cement content, optimum moisture content and maximum density of the soil cement mixture, standard laboratory tests must be performed. These test are: ASTM D558, D559 and D560.

Mixing of the soil and cement may be accomplished at the site with mechanical means such as dozers, loaders, etc. or with mechanical batch mixing equipment specifically designed for the preparation of soil cement mixtures. Placement of the mixed and moistened soil cement mixture may be accomplished by standard construction means. Compaction will be accomplished with normal embankment compaction equipment which shall include sheepsfoot roller compaction in order to minimize weaknesses at the interface between successive lifts of soil cement.

FIG. 2.6

TYPICAL SOIL CEMENT SLOPE PROTECTION



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2.4.4 Reinforced Core

A reinforced core may be used to stabilize the pitside slope of riverside berms against erosion from overtopping. The geometric configuration of the reinforced core should conform to the requirements indicated on Figure 2.7. The reinforced core will be made out of large riprap (Type VH or larger as defined in the USDCM or large rubble) with a 5 foot thick layer of material on the pitside face grouted. Soil cement or some other material approved by the District may be used as a core reinforcement in lieu of riprap. The core will be placed within the berm no more than 25 feet from the top of the pitside slope.

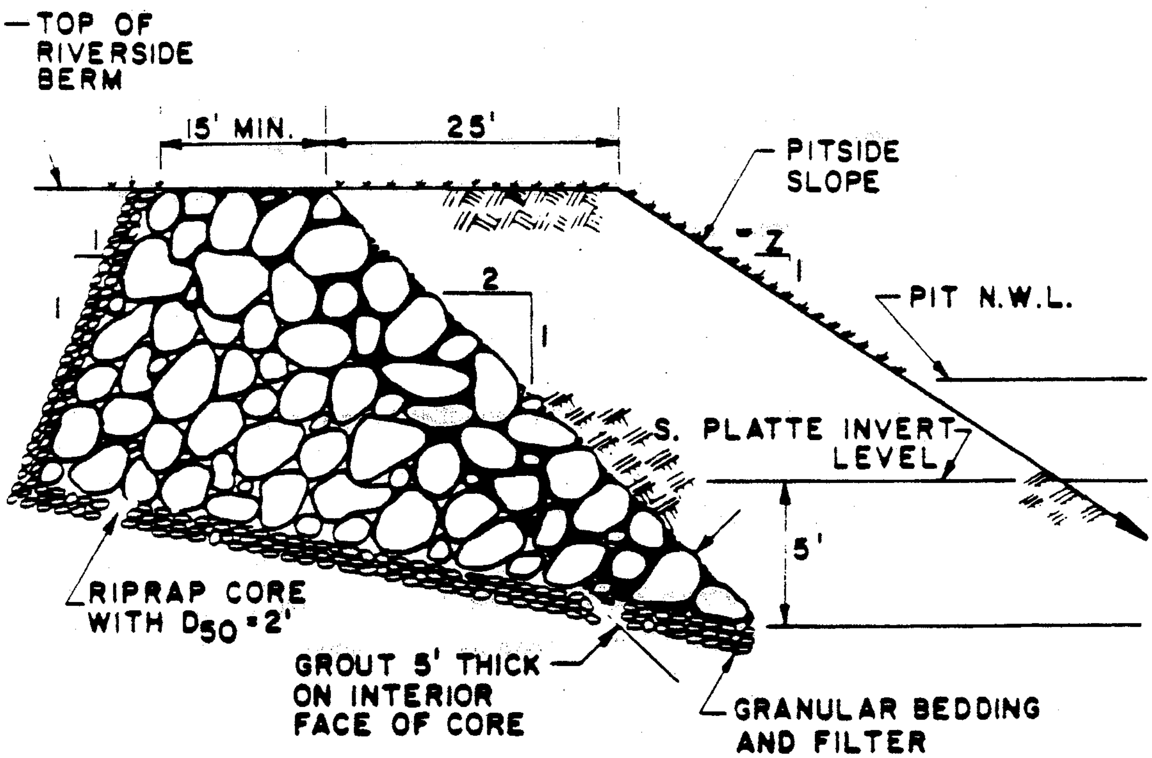
2.4.5 Side Channel Spillway

Side channel spillways may be provided between the river and the pit in order to minimize the potential for failure of the riverside berms. The stabilized spillway allows water to pass between the river and the pit. A spillway on the upstream end of the pit and one at the downstream end of the pit will be required unless the pit is small, in which case one spillway will be sufficient. Side channel spillways prevent the build-up of large differential heads between the river and water in the pit.

Because of the nature of the hydraulic response of the spillways, the differential head cannot be completely eliminated. The objective is to minimize, to the extent possible, the differential head between water in the river and water in the pit when the riverside berms overtop.

FIG. 2.7

TYPICAL REINFORCED RIPRAP CORE



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The elevation of the spillway crest shall be approximately 1 foot above the 2-year flood elevation. This is approximately 5 feet above the channel invert for the Master Plan cross section for South Platte River.

When the riverside berm length of the gravel pit, is less than 1,300 feet, only one side channel spillway will be required, and it shall be located approximately in the midpoint of the berm. If the length of the berm is greater than 1,300 feet, two spillways will be required and will be located along the berm approximately one-fourth of the berm length from the upstream and downstream ends of the berm.

When the length of the berm is less than 1,300 feet the spillway bottom length shall conform to the following geometric relationship:

$$L_s = A_p/12,000$$

or

$L_s = 100$ feet whichever is greater.

In which

L_s = length of the side channel spillway

A_p = area of pit measured in square feet at the high water line

When the length of the riverside berm is greater than 1,300 feet, two spillways shall be constructed and the bottom length of each spillway shall conform to the following relationship:

$$L_s = 0.6\ A_p/12,000$$

or

$L_s = 100$ feet whichever is greater

In which

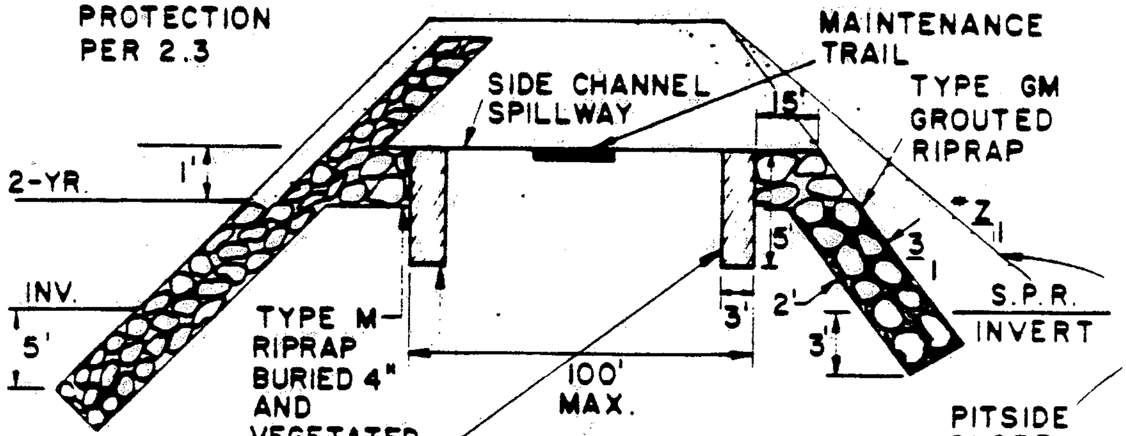
L_s = length of side channel spillway crest

A_p = area of pit measured in square feet at the high water line

Riprap with the grouted riprap rundown on the pit sideslope may be used to stabilize the spillways. Construction of this type of spillway stabilization must conform to the requirements indicated in Figure 2.8. The river bank should be stabilized in accordance with the requirements of Section 2.3 - Riverbank Protection. The crest of the spillway will be vegetated and shall have a 10-foot wide stabilized maintenance access trail along the entire length of the spillway. The objective is to provide a smooth flat driving surface with the ramps between top of berm and spillway bottom being no steeper than 10 percent. Access ramps, if sloped different than the side slopes of the spillway shall not protrude into the bottom width of the spillway. The pitside rundown slope will be protected using Type GM grouted riprap. Grout used for the grouted riprap portion of this structure shall have a minimum compressive strength of 2,000 psi. A concrete mix having 3/4-inch maximum aggregate, 4-inch slump and 2,000 psi - 28 day compressive strength may be substituted for grout. The grout shall be placed in a manner which ensures that all voids within the entire riprap mass are filled with grout.

Concrete may be used to protect spillways through riverside banks. The design of the concrete shall include consideration of all forces which may be encountered including, but not limited to, uplift forces and pore pressures generated by underlying materials. The river banks shall be protected in accordance with the requirements of Section 2.3 - Riverbank

FIG. 2.8
RIPRAP SPILLWAY STABILIZATION



* Z VARIES WITH RECLAIMED USE

Protection. The crest of the spillway and the downstream rundown will be protected using concrete. The geometric configuration of the concrete used to protect the spillway should conform to the dimensions shown on Figure 2.9.

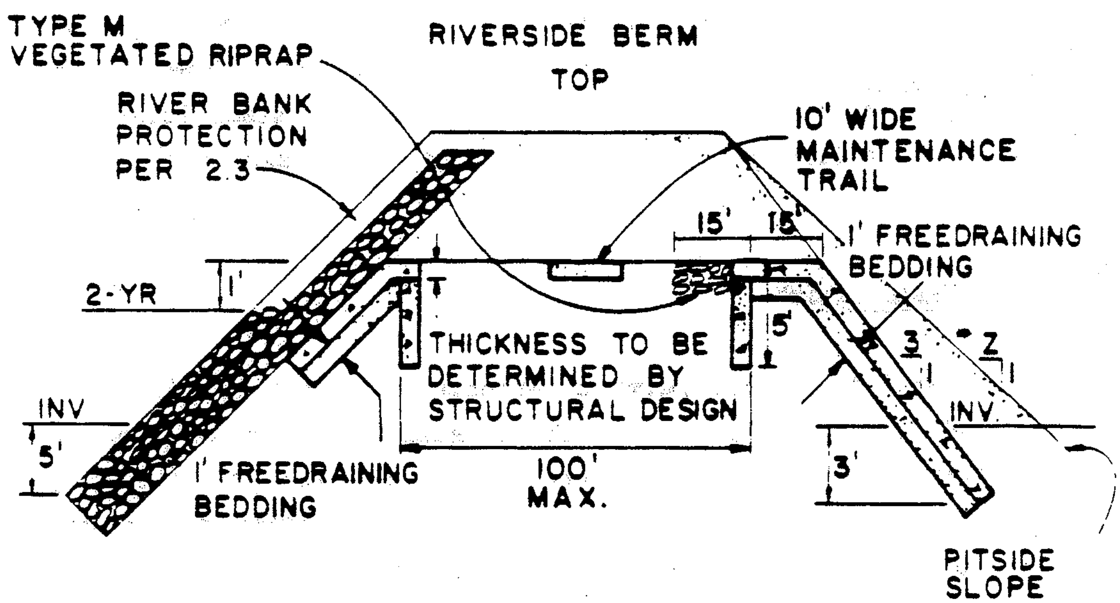
Soil cement is also an acceptable form of protection for side channel spillways. The river banks should be protected in accordance with the requirements of Section 2.3 - Riverbank Protection. The crest of the riverside berm will be protected with a layer of riprap and vegetation. The layer of soil cement on the spillway crest must be a minimum of 2 feet thick. The pitside slope is to be protected with soil cement placed in horizontal layers not exceeding 8 inches in thickness and between 8 and 12 feet wide. The soil cement should conform to the requirements of Section 2.4.3 - Soil Cement. The geometric configuration of soil cement for stabilizing side channel spillways should conform to the dimensions shown on Figure 2.10.

2.5 <u>LATERAL BERMS</u>

Lateral berms are berms constructed, or left in place, between pits and are perpendicular to the general direction of flow of the South Platte River. These lateral berms separate gravel mining pits from one another. Lateral berms may be overtopped during major floods. When overtopped, the berms are subject to erosion due to the relatively high velocity flow and, in time, may fail resulting in a rapid release of water. This phenomenon can propagate in the downstream direction potentially increasing with each successive failure. By protecting lateral berms from catastrophic failure the likelihood of such downstream propagation can be significantly decreased.

FIG. 2.9

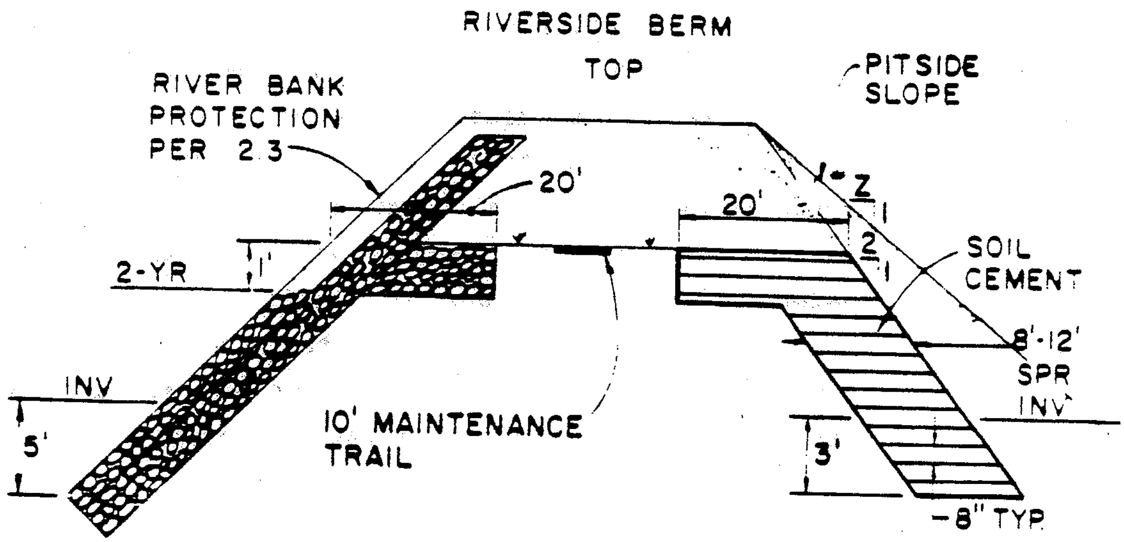
CONCRETE SPILLWAY STABILIZATION



* Z VARIES WITH RECLAIMED USE

FIG. 2.10

SOIL CEMENT SPILLWAY STABILIZATION



*Z VARIES WITH RECLAIMED USE

The stabilization of lateral berms should be accomplished when mining reaches the berm location. When the lateral berm is proposed immediately upstream or downstream of an existing gravel pit, the stabilization proposed in this section must be implemented prior to the initiation of mining operations adjacent to the berm. When no mining has occurred downstream or upstream, the berm may be stabilized when mining reaches the berm location.

2.5.1 Type A Lateral Berms

Type A lateral berms are berms between adjacent gravel pits. Spacing between Type A lateral berms shall not exceed 1/2 mile as measured longitudinally along a river or a stream channel. These berms must be protected against headcutting to withstand overtopping during the 100-year flood. Erosive forces on the upstream face of lateral berms will be minimal with the exception of wave action caused by winds across the gravel pit's water surface. The means for stabilizing the downstream face of the lateral berms are similar to those required to stabilize the pitside face of riverside berms and are as described in Section 2.5.3 - Protection of Lateral Berms.

2.5.2 Type B Lateral Berms

Type B lateral berms also are berms between adjacent gravel pits. These berms differ from Type A lateral berms in their spacing. Type B lateral berms are located at major arterial road crossings as opposed to every half mile. These berms are intended to provide a greater level of protection than the Type A lateral berms. This will be accomplished in part by the arterial road itself. The means for stabilizing the downstream face of Type B lateral berms are similar to those for Type A lateral berms and are described in Section 2.5.3 - Protection of Lateral Berms.

2.5.3 Protection of Lateral Berms

Lateral berms are to be protected in a manner similar to the requirements for the pitside slopes of riverbank berms. In general, the crest of lateral berms are to be revegetated and the downstream face of lateral berms are to be stabilized and protected using the methods proposed in Section 2.4 - Pitside Bank Protection.

Riprap, riprap with vegetation, soil cement, and a reinforced core are all viable options for reinforcing both Type A and Type B lateral berms. These methodologies are to comply with the requirements of Section 2.4.1 - Riprap, 2.4.2 - Riprap with Vegetation, 2.4.3 - Soil Cement, and 2.4.4 - Reinforced Core, respectively. The general geometric characteristics of lateral berm protection will conform to the individual details for the selected method of protection except that the protection need extend only 3 feet below the pit normal water level.

2.5.4. Spillway for Type A Lateral Berms

A spillway will only be allowed for Type A lateral berm protection. The existing or future arterial road crossing associated with Type B lateral berms renders the spillway approach infeasible. The required length of the spillway for Type A lateral berms along the South Platte River will be determined using the following equation:

$$L_{s1} = 2,500/H^{1.5}$$

In which,

L_{sl} = length of lateral berm spillway
H = height between top of berm and spillway crest

2.6 BERM TOP WIDTHS - RIVERBANK BERMS

This section specifies the minimum top width for gravel mine berms located adjacent to the South Platte River. The top width requirements are specified to protect the berms from rapid failure during floods. Such failure may result in the South Platte River flowing through the adjacent gravel pits and subsequently damaging property along the river. The berm top width requirements consider long term stability and safety along the South Platte River. Localized damages to the berms may result during large floods and may require periodic repair and maintenance as determined by the Adams County Engineer.

The top widths are broken into two classifications, one for an unprotected river bank and one for protected river banks. Unless otherwise indicated, the requirements of Sections 2.3 - Riverbank Protection and 2.4 - Pitside Bank Protection constitute adequate bank protection.

The top widths are expressed as the distance between the top of the river bank slope to the top of the pitside slope. The area between these two tops of slopes should be no steeper than 3 percent and as a minimum have a stand of vegetation which reasonably closely resembles the native vegetation along the site.

2.6.1 Existing Unstable Alignment

When the river is allowed to maintain its existing unstable alignment, and no bank protection is provided beyond the necessary revegetation, the minimum allowable top width of riverside berms will be 400 feet. This is the extreme case in terms of setback requirements imposed on gravel mining operators. It is, however, also the least costly measure in terms of capital expenditures.

When the river is maintained in its existing unstable alignment and the riverbank is protected in accordance with the requirements of Section 2.3 - Riverbank Protection the minimum allowable top width for the riverside berm will be 250 feet.

When the river is left in its existing unstable alignment and no river bank protection is provided, but pitside slope protection is provided in accordance with the requirements of Section 2.4 - Pitside Bank Protection, the allowable minimum top width will be 300 feet.

When the river is maintained in its existing unstable alignment and protection is provided for both the river bank and the pitside slope in accordance with Sections 2.3 - Riverbank Protection and 2.4 - Pitside Bank Protection, the allowable minimum top width of the riverside berm will be 150 feet.

2.6.2 Master Plan Alignment and Minimum Maintenance Alignment

The Master Plan alignment and the minimum maintenance alignment are two cases in which the alignment of the river is predetermined and riverside bank stabilization is provided. The Master Plan alignment is a modification of the minimum maintenance alignment and attempts to reconcile not only the geometric requirements but also property line constraints and constraints imposed by the existing river alignment.

Bank stabilization for the Master Plan alignment and the minimum maintenance alignment will be in accordance with those methodologies presented in Section 2.3 - Riverbank Protection. For the Master Plan alignment, the various acceptable types of bank stabilization are presented in the Master Plan drawings and differ slightly from the requirements specified in Section 2.1 - Types of River Alignment and 2.2 - Revegetation of Berms. The principal differentiation is in the required stabilization scheme. In most cases, where the Master Plan alignment deviates from the requirements specified, the areal extent of the recommended slope stabilization scheme will extend beyond those required if the minimum maintenance alignment were adhered to rigorously.

When the alignment of the South Platte River conforms to either the recommended Master Plan alignment or the minimum maintenance alignment, the minimum allowable top width without pitside bank protection will be 200 feet. This top width presumes that the river banks will be stabilized by the owner in accordance with the requirements specified in the Master Plan or as part of the minimum maintenance alignment.

When the requirements of the Master Plan alignment or the minimum maintenance alignment are adhered to and, in addition to complying with the requirements for river bank stabilization, the pitside slope of riverside berms are stabilized in accordance with Section 2.4 - Pitside Bank Protection, the minimum allowable top width for riverside berms will be 100 feet.

2.7 BERM TOP WIDTH-LATERAL BERMS

This section specifies the minimum top width for lateral berms located adjacent to the South Platte River and oriented perpendicular to the river. The top width requirement is specified to protect the berms from failure during floods which result in an overtopping of lateral berms and cascading of water in the downstream direction. Instances of major flooding may result in some localized damage of the lateral berms in spite of the protection measures previously suggested. Routine repair and maintenance of all facilities associated with lateral berms must be accomplished.

2.7.1 Type A Lateral Berms

2.7.1.1 Unprotected Type A Lateral Berms. When Type A lateral berms are left in their unprotected natural state, the minimum allowable top width will be 250 feet. This assumes that the surface of the lateral berm is left in its native condition. If the lateral berm is to be reconstructed subsequent to excavation, the top of the lateral berm and the downstream slope must be revegetated in accordance with the requirements of Section 2.2 - Revegetation of Berms.

2.7.1.2 Protected Type A Lateral Berms. When Type A lateral berms are protected in accordance with the requirements of Section 2.5.3 - Protection of Lateral Berms, the minimum allowable top width will be 100 feet.

2.7.2 Type B Lateral Berms

The top width required for Type B lateral berms will exceed that required for Type A lateral berms because of the more critical nature of the Type B lateral berms. Type A lateral berms are designed to withstand flooding events up to the 100-year event, however, because of the possibility of more severe flood events, Type B lateral berms should withstand floods of higher magnitude than the 100-year event.

2.7.2.1 Unprotected Type B Lateral Berms. When Type B lateral berms are left unprotected except for vegetation as specified in Section 2.2 - Revegetation of berms the minimum allowable top width shall be 350 feet.

2.7.2.2 Protected Type B Lateral Berms. When Type B lateral berms are protected in accordance with the requirements of Section 2.5.3 - Protection of Lateral Berms, the minimum allowable top width shall be 200 feet.

2.8 BERM TOP WIDTHS - SUMMARY TABLE

This section presents a summary table of allowable berm top widths based on the methodologies employed for berm stabilization.

TABLE 2.1

RIVERBANK BERM TOP WIDTH

Area Stabilized	Alignment	Type of Stabilization	Minimum Top Width (ft)
None	2.1.1 -Existing Unstable	None	400
Riverbank only	2.1.1 - Existing unstable	2.3 - Riverbank protection	200
		2.3.1 - Riprap 2.3.2 - Riprap with vegetation 2.3.3 - Jetties	
	2.1.2 - Master Plan	2.3 - Riverbank protection	200
		2.3.1 - Riprap 2.3.2 - Riprap with vegetation 2.3.3 - Jetties	
	2.1.3 - Minimum Maintenance	2.3 - Riverbank protection	200
		2.3.1 - Riprap 2.3.2 - Riprap with vegetation 2.3.3 - Jetties	
Pitside bank only	2.1.1. - Existing unstable	2.4 - Pitside bank protection	300
		2.4.1 - Riprap 2.4.2 - Riprap with vegetation 2.4.3 - Soil cement 2.4.4 - Reinforced core 2.4.5 - Side channel spillway	
Riverbank and Pitside Bank	2.1.1 - Existing unstable	2.3 - Riverbank protection and 2.4 - Pitside bank protection	150
	2.1.2 - Master plan	2.3 - Riverbank protection and 2.4 - Pitside bank protection	100
	2.1.3 - Minimum maintenance	2.3 - Riverbank protection and 2.4 - Pitside bank protection	100

TABLE 2.2
LATERAL BERM TOP WIDTH

Berm Type	Type of Stabilization	Minimum Top Width (ft)
Type A	None	250
	2.5.3 - Protection of lateral berms	100
	2.4.1 - Riprap 2.4.2 - Riprap with vegetation 2.4.3 - Soil cement 2.4.4 - Reinforced core 2.5.4 - Spillway for Type A lateral berms	
Type B	None	350
	2.5.3 - Protection of lateral berms	200
	2.4.1 - Riprap 2.4.2 - Riprap with vegetation 2.4.3 - Soil cement 2.4.4 - Reinforced core 2.5.4 - Spillway for Type A lateral berms	

CHAPTER 3.0

IN-RIVER GRAVEL MINING

Mining in the mainstream of the South Platte River or any of its tributaries is strongly discouraged. Without extensive measures to protect the river, the impact of an instream sand gravel extraction can be widespread and severe. All operators proposing to mine the mainstream of the river shall demonstrate conclusively that they will not adversely impact the river and the properties, roads, bridges, diversion structures and utilities on the river for a distance of four miles in the upstream and downstream directions, in addition to taking the mitigating measures required in Chapter 3.0 and obtaining all other permits required by Federal and State law.

Instream mining of the river includes not only mines which are ongoing in the channel of the South Platte River or its tributaries, but also includes offstream gravel mines when it is proposed to relocate the river through such offstream gravel mines. The impacts of relocating a river or a stream through an off channel pit is equivalent to those associated with the actual mining of the South Platte River bed or its tributaries. As such, these requirements will apply to mines which actively mine within a river or stream channel as well as mining operations which propose to relocate a river or a stream through previously mined pits.

One form of instream mining will be allowed. Lowering of the channel invert to the elevation specified in the Master Plan will be allowed if the operation is stopped as soon as District determines the desired degradation has occurred and the associated bank and channel improvements are also constructed.

3.1 GROUNDWATER IMPACTS

The extraction of gravel and other products from the bed of the South Platte River or its tributaries shall be done in such a manner as to prevent damage to adjoining property. This requires that the adjacent groundwater table not be altered or if alteration takes place the owner of any groundwater rights shall be compensated commensurate with any damage which may occur. It also requires the compliance with applicable Colorado water laws and regulations governing injury to existing water rights as well as the compliance with applicable federal and Colorado water quality laws and regulations.

3.2 SEDIMENT TRANSPORT

The instream gravel pits may result in the deposition of significant portions of the sediment load carried by the South Platte River or its tributaries. The gravel operator must show that his mining will not adversely affect the stability of the river or a stream downstream from his site.

3.3 WATER QUALITY

Long range water quality concerns can be significant in an instream gravel pit lake because of pollution constituents in the river at the location of the excavation. Any operations in the river shall be required to comply with applicable federal and Colorado water quality laws and regulations.

3.4 SIDESLOPE PROTECTION

Excavation of materials from the bed of the South Platte River must be done in a manner which does not jeopardize the stability of the channel banks. To accomplish this, the side slopes of the river bank above the pre-mining channel invert should be extended downward. This slope will generally be between 2.5 and 4 feet horizontal for every one foot vertical. The actual slope will depend on the method of bank stabilization proposed at the site.

3.5 HEADCUTTING CONTROL

When mining in the river bed is permitted, gravel operators must prevent long term headcutting from propagating in the upstream direction. The removal of material from the riverbed results in excessive headcutting which can and does endanger structures upstream crossing the river or a stream or located in the adjacent banks of same. It is imperative that the operators install positive controls as specified below to prevent headcutting of any type from occurring upstream as a result of gravel extraction from the river bottom.

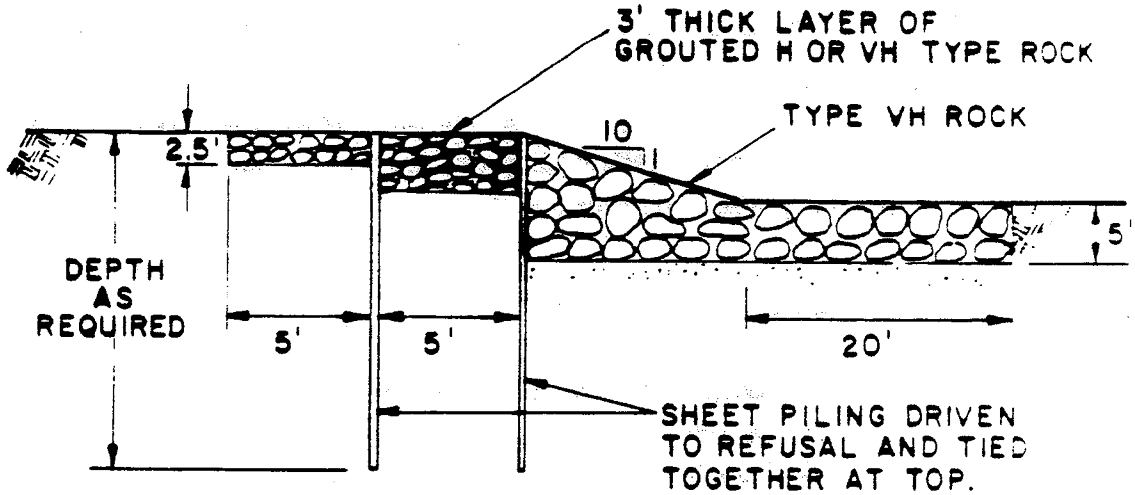
3.5.1 Control Structures

The impacts of the instream gravel extraction must be mitigated with a positive structural control at the upstream end of the instream pit. The structure must be protected from failure through a wide variety of flow rates, gravel pit depths and inconsistent soil properties.

Instream control structures will generally require the construction of a structural control incorporating a double row of sheet piling such as that shown on Figure 3.1. Site specific designs incorporating specific soils and geometric information shall be prepared and submitted to District for approval.

FIG. 3.1

TYPICAL INSTREAM HEADCUTTING CONTROL STRUCTURE



DEPTH OF SHEET PILING, GUAGE OF PILING, SIZE OF ROCK, AND ROCK AND FILTER GRADATIONS ARE TO BE DETERMINED BY A REGISTERED ENGINEER ON A SITE SPECIFIC BASIS AND ARE TO BE SUBMITTED TO THE COUNTY ENGINEER FOR APPROVAL.

PROVIDE AN OPENING FOR LOW FLOWS, SAFE PASSAGE OF BOATERS AND FISH MIGRATION.

THE LOW FLOW OPENING SHALL BE DESIGNED IN STEPS OF POOLS AND CHUTES WITH THE LAST CREST OF THE DOWNSTREAM CHUTE BEING AT THE PROJECTED TAILWATER WHEN ALL GRAVEL IS REMOVED FROM THE PIT. THE VERTICAL DISTANCE BETWEEN SUCESSIVE CRESTS SHALL NOT EXCEED 18-INCHES.

3.5.2 Location of Control Structures

The location of the positive control will be at the upstream end of the gravel pit. In no case should the top of the cutoff be any lower than the Master Plan invert of the South Platte River or the invert or its tributaries identified in Major Drainageway Planning Reports or FHAD reports. The positive control will be extended perpendicular to the main channel into the adjacent overbank a minimum of 250 feet. Where gravel pits exist in the overbank, a positive control should extend across the lateral berms as well.

3.5.3 Overbank Protection

In areas where there is no overbank gravel mining operation in existence and none is proposed, the overbank cutoff may be less substantial. In these cases, the overbank cutoff may consist of Type M (USDCM) riprap placed in a trench 6 feet deep and 3 feet wide and extending across the entire floodplain. This protection will begin at the end of the embedment of the structural positive control as discussed in Section 3.5.2 - Location of Control Structures.

3.6 UPSTREAM PROTECTION

Because of the increased energy grade line slope immediately upstream of the gravel pit cutoff structure, it will be necessary to protect the river upstream from general degradation and against potential damages to bridges, utility crossings and property.

3.6.1 Extent of Upstream Protection

The protection required upstream of the crest of the pit cutoff structure shall extend upstream a minimum of 175 feet.

3.6.2 Type of Protection

The type of protection used to protect the river bottom shall be riprap or cobble. The required size of protection shall be determined according to the requirements of the USDCM Major Drainage Chapter, Section 5-Riprap. Rock or cobble satisfying the proper gradation will be allowed.

CHAPTER 4.0
RECLAMATION

The mineral rules and regulations, published by the Colorado Mined Land Reclamation Board, establish the requirements for reclamation of mined lands in the State of Colorado. These requirements must be satisfied and adhered to for the life of the mine in order to obtain a mining permit for the extraction of gravel. The requirements presented in this document supplement those of the Mined Land Reclamation Board and recognize the special riverine corridor stability needs. In no case do the requirements of this document imply that the requirements of the Mined Land Reclamation Board, as a minimum, should not be adhered to.

4.1 RECREATIONAL CRITERIA

When gravel pit lakes are to be reclaimed as recreational amenities, the side slopes of the lakes should be sloped at a minimum of 5 horizontal to 1 vertical (5H:1V) for a distance from the top of the berm to a point 8 feet under the normal water surface. There the slope can be at 2H:1V to the lake bottom. Where concentrated swimming is to be actively encouraged, side slopes of 20H:1V, or flatter, are recommended in the beach areas.

Irregularities in the lake shorelines are required for variety in the environment and to enhance wildlife habitat. These irregularities also help protect the banks from wave action. Other recreational or wildlife features, such as islands, may be required by local government.

Corridors must be provided for access to recreational areas and maintenance trails. These corridors should be incorporated into riverside berms, or through other portions of the lease properties if developed as part of a overall recreational master plan by the operators or by the local governments. A corridor of 100 feet minimum is recommended to accommodate multiple recreational uses. This river corridor minimum width should be maintained on both sides of the river for its entire length. The recreational corridor may be contained within the required top width of riverside berms.

4.2 REVEGETATION CRITERIA

Revegetation of all areas not receiving other stabilization techniques will be required. The revegetation should be installed in conformance with the Guidelines for Vegetation and must meet the approval of the Colorado Mined Land Reclamation Board.

The goal of revegetation is to reestablish the historic ground cover as closely as possible to promote the return of the historic animal population, to protect areas from erosion by wind and water and to improve the aesthetics of the area. To accomplish this, revegetation must be undertaken at the earliest opportunity. Revegetation must begin as soon as the slope has attained its required configuration subject to the seeding criteria specified in Guidelines for Vegetation. When slopes are in areas which will be inactive for periods in excess of six months vegetation must be reestablished.

COMPUTING
DEGRADATION AND
LOCAL SCOUR

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INTRODUCTION

The purpose of this technical guide is to present several methods which can be applied in computing degradation of a stream channel occurring because of changes in flow regimen or reduced sediment load below a dam or diversion dam, and to provide procedures to use in estimating maximum scour depth of channels for design of a structure such as a bridge or siphon crossing.

In this guide, the following definitions have been adopted:

Degradation. - The long-term process by which streambeds and flood plains are lowered in elevation due to the removal of material from the boundary by flowing water.

Aggradation. - The long-term process by which streambeds and flood plains are raised in elevation due to the deposition of material eroded and transported from other areas.

Scour. - The enlargement of a flow section by the removal of boundary material through the action of fluid motion during a single discharge event. The results of the scouring action may or may not be evident after the passing of the flood event.

BACKGROUND ON DEGRADATION

Computations by computer application of some of the more sophisticated mathematical models applied to degradation below a dam are not described in these guidelines. The best known of these solutions is the Corps of Engineers (1977) HEC-6 computer program. A more comprehensive and sophisticated Reclamation (Bureau of Reclamation) computer model, which can deal with uneven scour and deposition across and along a river is being developed and should be available for use in 1985. The objective of most models is to simulate the behavior of an alluvial channel by combining a steady-state backwater computation for defining channel hydraulics with a sediment transport model. It is often difficult to verify the sediment transportation results from models with the total sediment transport of the river under investigation. The desk calculator approach to channel degradation below a dam, developed for Reclamation and described by Lane (1948), was a forerunner to the more sophisticated mathematical models. Although more of the comprehensive mathematical models are becoming available, they are still undergoing development and change. An example of a study to verify one of these mathematical models is described by Mengis (1981). The methods described in this technical guide should be applied before any attempt to use the more sophisticated mathematical models.

Before undertaking any degradation study below a dam, an evaluation is needed of the degree of detail required to complete the study, of the appropriate design data for the dam, and of the future environmental conditions below the dam. The type of study described in this technical guide is considered a minimum requirement before recommending a more sophisticated mathematical model. There is considerable support for these procedures which were applied in studies prepared in the 1950's to channels such as the Colorado River below Glen Canyon Dam, Middle Loup River below Milburn Dam, and Niobrara River below Norden Dam. Observed degradation patterns since construction below Glen Canyon Dam and Milburn Dam have supported the results of the degradation studies. In the case of Niobrara River below Norden Dam, a mathematical model study made by Shen (1981) agreed closely with results of the studies made using the procedure described in this guideline.

Most existing rivers or streams are in a quasi-equilibrium state when considered on a long-term basis. While in this state, the stream sediment processes of degradation and aggradation are relatively at a standstill and, if occurring, are only of localized nature. The state of stream equilibrium as described by Lane (1955) may be expressed qualitatively by the following equation:

$$Q_s D_m = k\ Q_b S_b \qquad (1)$$

where:

Q_s = Bed material discharge
D_m = Effective diameter of bed material mixture
Q_b = Water discharge to determine bedload transport
S_b = Slope of the streambed
k = Constant of proportionality

It is recognized that in some situations other hydraulic parameters may be equally important as slope.

When any one of the four variables is altered, one or more of the other variables must adjust in order to return the stream to a state of equilibrium. An obvious case is when a dam and reservoir are constructed on a stream, eliminating or diminishing the sediment load downstream from the dam. The relatively clear water released to the stream below the dam is capable of eroding both channel bed and banks when released in sufficient quantity. If the exposed bed and banks are composed of sediment particles that can be moved or picked up by the flowing water, degradation will occur. The degradation process can occur vertically (streambed), laterally (streambanks), or both depending upon the stream discharge and the particle size and cohesive properties of the material forming the bed and banks. In the process of establishing a new state of equilibrium, the stream slope will decrease and the sediment particles remaining in the streambed after some time lapse will be the coarser fraction of the original bed material. Equation 1 provides a comparative evaluation which merely indicates an imbalance in channel equilibrium to be expected and that a change in regimen is imminent. To quantify this change requires application of sediment transport equations either in the form of a mathematical model or in less detail by the empirically tested equations and procedures described in this technical guideline. The effect of this change in regimen below a dam is to produce general degradation and lowering of tailwater elevations.

Other examples of change in state of equilibrium are the disturbance created by transbasin diversions, wastewater, or return flows from an irrigation project which increase the water supply of a stream system. The resulting increase in the streamflow component in equation 1 will increase the normal stream velocity which directly influences the sediment transport capacity of the stream. This in turn leads to channel adjustments which if uncontrolled will in time establish a new state of equilibrium.

A closely related problem that is not necessarily associated with the equilibrium relationship defined by equation 1 is the natural scour occurring at the time of a peak-flood discharge. Sufficient channel scour as described by Lane and Borland (1954) may occur during higher floodflows to cause severe damage or threaten the stability of any structure located either along the bank of a river or across the channel. In anticipation of channel scour, a crossing structure such as a siphon or bridge should be designed to withstand any scour which might occur in conjunction with the design flood.

GENERAL DEGRADATION

Basic Factors Influencing Degradation

The two basic factors influencing the extent of degradation in a stream channel are: (1) hydraulic properties including river channel velocities, hydraulic gradient or slope, and depths of flow associated with peak discharges and throughout the range in discharges, and (2) particle size distribution of sediments in the channel bed and banks. A careful evaluation of these factors is essential to any degradation analysis. One additional factor is the combination of streambed and valley controls which may exist in the channel reach subject to degradation. The controls may be rock outcrops, cobbles and boulders in the channel, vegetation growing along the banks, or manmade structures which act to control water levels and retard degradation processes. A control in the channel may in some cases prevent any appreciable degradation from occurring above it. Conversely, a change or removal of an existing control may initiate the degradation process.

The water discharge for the stream channel is essential to the analysis. This requires information on the volume as well as the flow release pattern from an operation study for a reservoir or from any planned increases to the water supply to a stream system. In many stream systems, both the volume and distribution of the change in water supply can be illustrated by use of a flow-duration curve. The flow-duration curve is a cumulative frequency relationship, usually used to represent long-term conditions, that shows the percent of time that specific discharges were equalled or exceeded in a given period. The curves representing a future water supply can be compared directly with historic flow-duration curves for evaluating the significance of any changes. Flow-duration curves are used in computer application of the mathematical modeling for studying river channel degradation. The approach described in this technical guideline for computing degradation utilizes the dominant discharge method for representing water discharge.

The discharge value used in degradation analysis is referred to as the dominant discharge for the stream channel. Dominant discharge is defined as the discharge which, if allowed to flow constantly, would have the same overall channel shaping effect as the natural fluctuating discharges as illustrated by the flow-duration curve. The dominant discharge used in channel stabilization work usually is considered to be either the bank-full discharge or that peak discharge having a recurrence interval of approximately 2 years on an uncontrolled stream. When streamflow is regulated by an upstream dam, the problem of determining the dominant discharge becomes more difficult if detailed data on future reservoir releases are not available. If releases from the reservoir fluctuate considerably due to incoming floods, the mean daily discharge derived from an operation study which is equalled or exceeded on the average of once every 2 years can be considered as the dominant discharge.

The type of sediments forming the bed and banks of the stream channel will influence the extent of degradation. The type of bed material also dictates the approach used in estimating the depth or amount of degradation. In situations where the streambed is composed of transportable material extending to a depth greater than that to which the channel can be expected to degrade, the approach most useful is that of computing a stable channel slope, the volume of expected degradation, and then determining a three-slope channel profile which fits these values. However, in situations where the bed material includes a sufficient quantity of large size or coarse material which cannot be transported by normal river discharges, the best approach is to compute the depth of degradation required to develop an armoring layer. The formation of the armoring layer usually can be anticipated to control vertical degradation when approximately 10 percent or more of the bed material is of armoring size or larger. This layer develops as the finer material is sorted out and transported downstream. Vertical degradation occurs at a progressively slower rate until the armoring layer is of sufficient depth to inhibit the process.

Bed Material Sampling

Bed material samples of the surface layer as well as the underlying sediment should be collected for analysis throughout the reach of the river under investigation. It is important that samples be representative of the material in the zone of anticipated scour, that is vertically, laterally, and longitudinally. Therefore, the number of samples depends on the homogeneity of material in the streambed. If the streambed is fairly uniform, fine-grained material of sand sizes in the range from 0.062 to 2.0 mm, a volumetric or bulk sampling procedure is followed. Bulk samples usually are dug out by shovel from exposed sandbars, or for underwater conditions by a bed material sampler such as the BM-54, BMH-60, or BMH-80 (Federal Interagency Sedimentation Project, 1963). Core samples taken in the stream channel as a part of geologic site investigation may be used if they are considered representative of channel bed material. An example of a sampling program for bulk sampling would be to collect about three samples in each cross section which if located about 0.5 mi (0.8 km) apart for a 5 mi (8-km) reach would provide about 33 samples for sieve analysis. Each sample would contain both surface and subsurface material and an arithmetic average of all 33 samples would provide a composite sieve analysis.

The sampling of riverbeds composed of gravel or cobble material >2.0 mm which may be uniformly mixed through the degradation zone or as a pavement over finer size sediments is more complicated. A good description of sampling procedures under variable types of sediment is given by Wolman (1954), Kellerhals (1967), Leopold (1970), and Kellerhals and Bray (1971).

For a gravel or cobble bed river, the sampling procedure is dependent on the purpose or objectives of the study. If the investigator is conducting a sediment transport study to quantify the bedload movement, then surface samples of the streambed are needed. A degradation or scour study requires samples of both the surface as well as the underlying sediments. In the latter case, it is necessary for the investigator to determine either by sampling or judgment the appropriate procedure for properly weighting the proportion of surface and subsurface sediments.

The procedures for sampling and analysis of samples for gravel and cobble riverbeds can be quite varied depending on river conditions. For "deep water" sampling, a drag bucket technique is used. The size of the bucket is dependent on the size of rocks. A bucket-type "jaw" sampler with jagged edge on the open end of the bucket having a diameter of about 1 foot (0.3 m) has been used with some success by Reclamation for cobble bed material. On many rivers, deep water sampling can be avoided by finding an exposed gravel bar with materials observed to be similar to the underwater material and sampling under dry bed conditions.

The techniques for sampling of bed material on exposed gravel bars or under shallow water are described by Wolman (1954) or Kellerhals and Bray (1971). The most common methods are:

1. Volume or bulk sample collected for sieve analysis by weight.

2. Grid sampling where all material in a specified surface area is collected, usually a square that can vary from 1.5 to 3 ft (0.5 to 0.9 m) on each side.

3. Random sampling of rocks at predetermined distance along a straight line usually by a random step procedure or collecting those at grid intersection points over a large areal coverage such as a 50-ft (15-m) square.

All three methods require an investigator to make a field selection for site selection based on representativeness of the bed material. Method 1 usually is applicable to small size gravels where the sample can be taken to a laboratory for sieve analysis. Methods 2 and 3 are applicable to larger rock where a surface count and measurement of the larger particles can be made and then converted to an equivalent customary bulk sieve analysis. The conversion is especially important if finer material is encountered during the count method which could be analyzed by sieve analysis and combined with the count method for a composite size analysis.

The count method involves the measurement of the intermediate axis of particles larger than about 1/2 inch (13 mm). Each rock is measured and grouped into an appropriate size and class and then thrown away. A minimum of from 75 to 100 rocks usually are considered necessary to have a representative sample. The conversion or weighting factor for each size fraction is directly proportional to D^3 with D being the geometric mean diameter for a size fraction. An example computation for conversion of rock count to sieve analysis by weight is shown in table 1 for sample No. B-2 in the Colorado River. It is advisable to photograph the bed material at all sampling locations. If the surface material is sampled by the count method, a photograph of this material as well as the underlying material is important. Figures 1 and 2 show the surface material and underlying sediments at a sampling location on the Colorado River (Pemberton, 1976). Figure 3 illustrates the results of sampling programs conducted in the Colorado River below Glen Canyon Dam prior to construction of the dam in 1956 and subsequent to construction in 1966 and 1975. The armor material in 1966 and 1975 was analyzed by the count method while all other samples were averaged from a bulk sieve analyses.

Table 1. - Conversion of rock count (grid-by-number) to sieve analysis by weight - Sample No. B-2 Colorado River below Glen Canyon Dam - 1975

Size D 1/			Weighting factor D^3 (mm^3) (10^3)	Count in size range	Count x D^3 (10^6)	Percentage	Percent finer
Size range in	Geometric mean mm	Geometric mean in					
9 to 8	216	8.49	10 100	3	30.3	15.9	100
8 to 6	176	6.93	5 450	14	76.3	40.2	84.1
6 to 4	124	4.90	1 910	28	53.5	28.2	43.9
4 to 2	72	2.83	373	72	26.9	14.1	15.7
2 to 0.75	31	1.22	298	100	2.98	1.6	1.6
				217	189.98	100	

1/ Measurement of intermediate axis.

Hydraulic Properties

The hydraulic properties of the stream channel at the dominant discharge are required in the degradation analysis. These properties include flow area, width, depth, and velocity which usually can be obtained from the water surface profile computations for the tailwater reach downstream from the dam. The accuracy of the field data in defining channel hydraulics as well as location of the proper channel sections is comparable to that given in the criteria for a water surface profile computation described by Reclamation (1957). The hydraulic properties of all the cross sections are averaged for the dominant discharge to determine representative data in the reach where degradation is expected to occur. If a distinct break in slope occurs in the overall reach, a subdivision into one or more reaches selected on the basis of slope should be made for averaging the hydraulic properties. The water surface slope is assumed equal to the energy gradient for all computations.

Upon obtaining data on particle size of bed and bank material and the channel hydraulic properties, a method of analysis is chosen to apply to the stream channel being considered. The two techniques presented in the following discussion, either the armoring or limiting slope method, are recommended as

Figure 1. - Gravel-cobble size armoring in Colorado River below Glen Canyon Dam in July 1975.

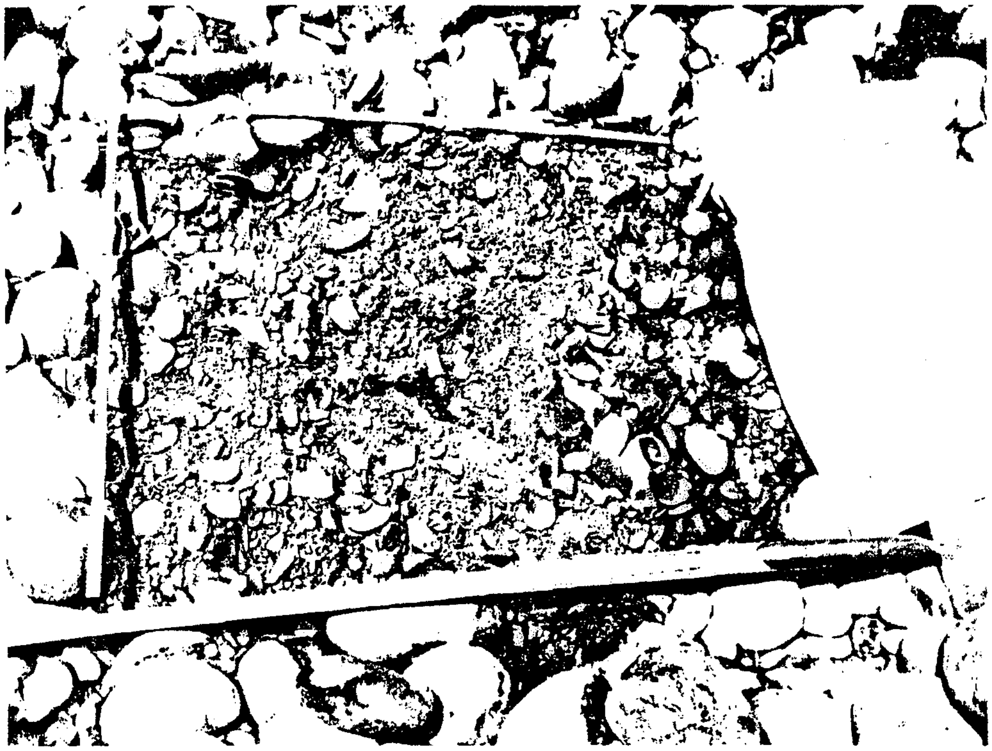


Figure 2. - Material underlying armor layer in Colorado River below Glen Canyon Dam in July 1975.

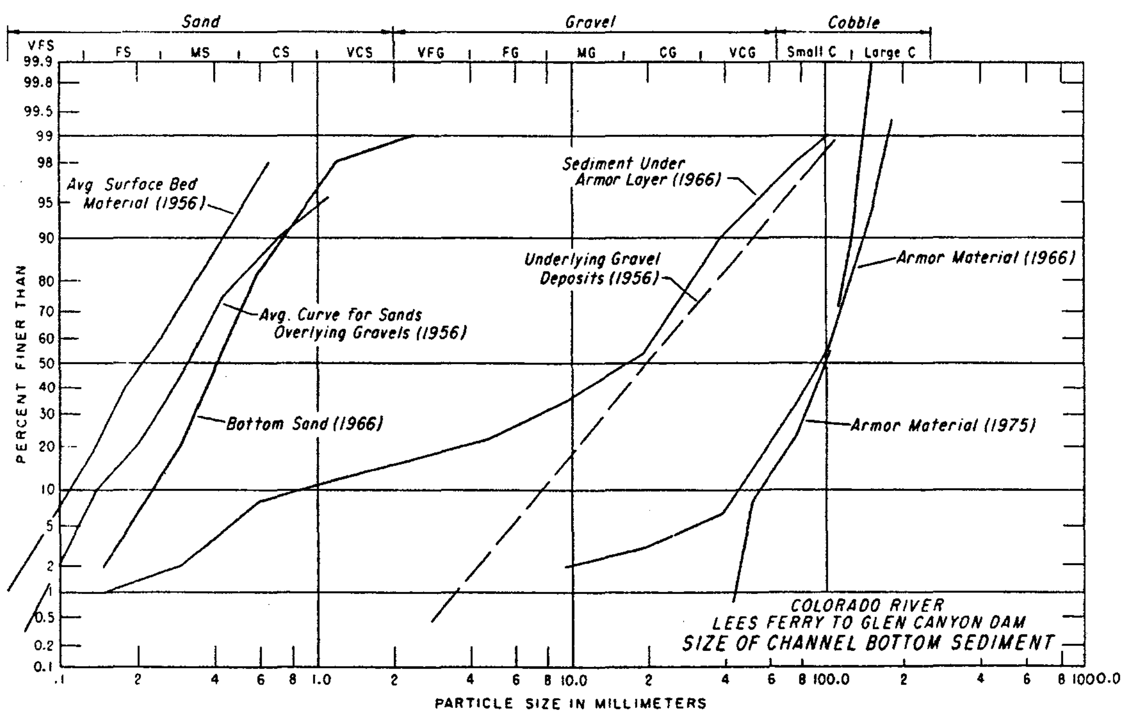


Figure 3. - Size of channel bottom sediment Colorado River, Lees Ferry to Glen Canyon Dam.

alternative choices. For general degradation, the armoring method is tested first because a sediment transport study may not be necessary with a resulting savings in time and cost for computations. If the armoring method is not applicable, then the stable slope method is used.

DEGRADATION LIMITED BY ARMORING

When the channel bed downstream from a dam contains more than 10 percent coarse material which cannot be transported under dominant flow conditions armoring will in time develop. The formation of an armoring layer at the maximum depth of degradation will depend on such factors as reservoir operations, the amount of armoring material available in the scour depth zone below streambed, and the distance to which this material extends downstream.

There are several ways to compute the size of bed material required for armoring and each method is regarded as a check on the others. Each method computes a different armoring size and some judgment may be required in selecting the lower size limitation of nontransportable material. Reclamation recommends the following methods to determine armoring size:

1. Meyer-Peter, Muller (bedload transport equation)
2. Competent bottom velocity
3. Lane's tractive force theory
4. Shields diagram
5. Yang incipient motion

Meyer-Peter, Muller (Bedload Transport Equation)

Bedload transport equations provide a method to compute a nontransportable particle size representing coarse bed material capable of forming an armoring layer. To describe a nontransportable size, the Meyer-Peter, Muller (1948) bedload equation (Sheppard, 1960) for beginning transport of individual particle sizes, may be applied when rewritten in the form:

$$D_c = \frac{dS}{K\left(\frac{n_s}{D_{90}^{1/6}}\right)^{3/2}} \qquad (2)$$

where:

D_c = Individual particle size in millimeters
K = 0.19 inch-pound units (0.058 metric units)
d = Mean water depth at dominant discharge, ft (m)
S = Slope of energy gradient, ft/ft (m/m)
n_s = Manning's "n" for bed of stream
D_{90} = Particle size in millimeter at which 90 percent of bed material by weight is finer

Bedload equations, such as the Schoklitsch equation (Shulits, 1935), that were developed on an experimental basis for material of a uniform size, may also be applied using the individual particle size rather than the mean size. Other bedload equations could also be used to determine the transport rate of various particle size ranges for the dominant discharge condition, selecting that size range where the transport becomes negligible as the representative armoring size. Some judgment is required in choosing the point where the transport is adequately diminished such as to reasonably assume that the particular size range is coarse enough to actually form an armor.

Competent Bottom Velocity

Investigations show that the size of a particle plucked from a streambed is proportional to the velocity of flow near the bed. The particle starts to move at what is called the competent bottom velocity (Mavis and Laushey, 1948) which is approximately 0.7 times V_m, the mean channel velocity. The competent bottom velocity method for determining armoring size is computed from a relationship between mean channel velocity with armoring size by the equation:

$$D_c = 1.88\ V_m^2 \text{ inch-pound units} \qquad (3)$$

$$D_c = 20.2\ V_m^2 \text{ metric units}$$

where:

D_c = Armor size, mm
V_m = Mean channel velocity, ft/s (m/s)

Lane's Tractive Force

The tractive force method is based on the results of a study by Lane (1952). He summarized the results of many studies in a relationship of critical tractive force versus the mean particle size diameter in millimeters, which is reproduced on figure 4. This method entails computing the critical tractive force (equation 4) using the channel hydraulics for dominant discharge. By selecting an appropriate curve on figure 4, usually the recommended set of "curves for canals with clear water in coarse noncohesive material," a critical tractive force gives the lower size limit of the nontransportable material, D_c.

$$\tau_c = \gamma_w d S \qquad (4)$$

where:

τ_c = Critical tractive force, lb/ft^2 (g/m^2)
γ_w = Specific weight)(mass) of water, 62.4 lb/ft^3 ($1\ t/m^3$)
d = Mean water depth, ft (m)
S = Slope, ft/ft (m/m)

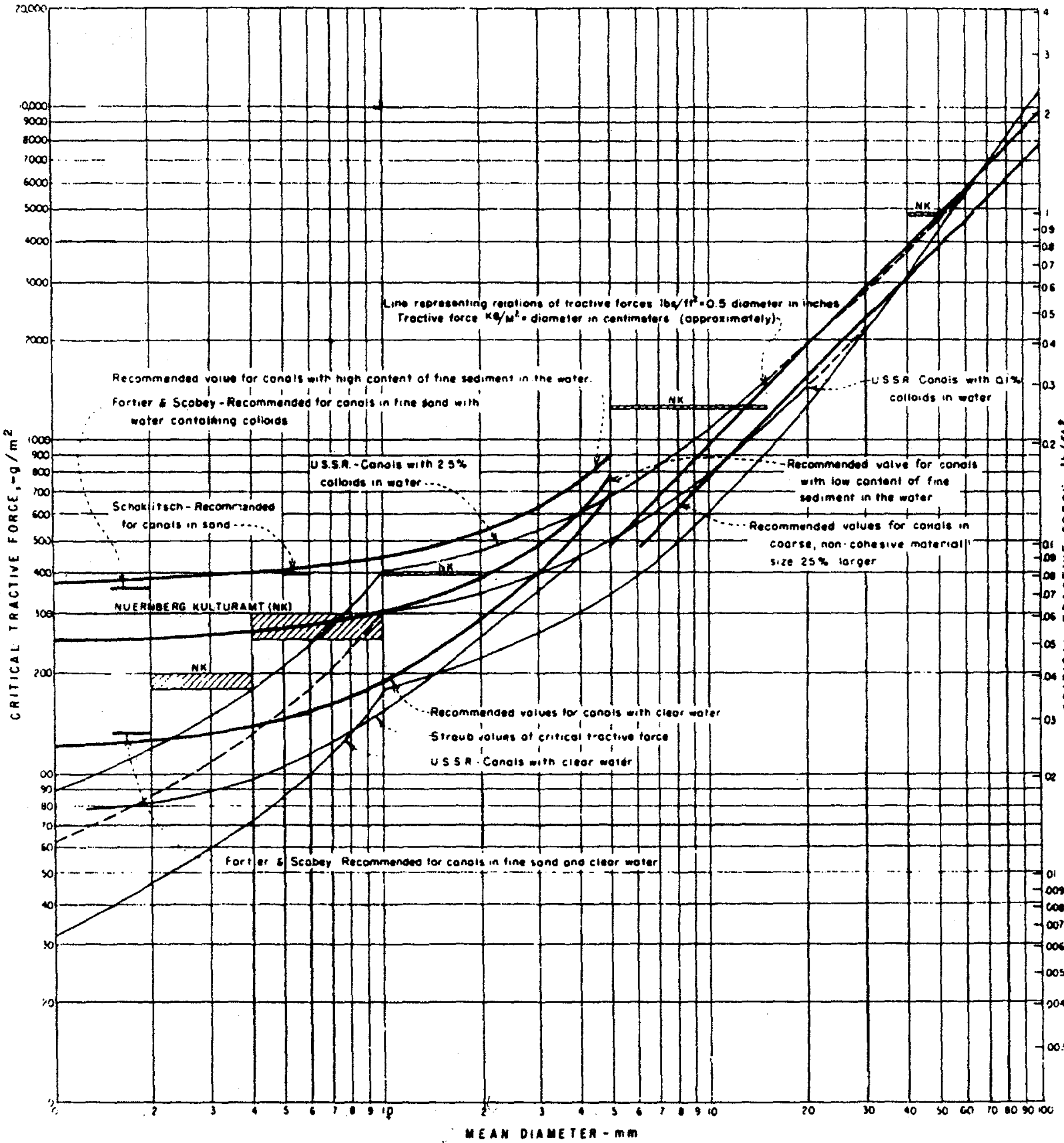


Figure 4. - Tractive force versus transportable sediment size (after Lane, 1952).

Shields Diagram

Many investigators use the Shields diagram (Shields, 1936), figure 5, to define the initiation of motion for various particle sizes. In the process of armoring of a streambed for predominately gravel size material >1.0 mm and high Reynold's number $R_* > 500$, the Shields parameter given below provides a method for determining an armor size.

$$\boxed{\tau_* = \frac{\tau_c}{(\gamma_s - \gamma_w) D_c} = 0.06} \qquad (5)$$

$D_c = \tau_c / 6.1$

where:

τ_* = Dimensionless coeff. of shear stress.
τ_c = Critical shear stress = $\gamma_w dS$, lb/ft^2 (t/m^2)
γ_s = Specific weight (mass) of the particle
γ_w = Specific weight (mass) of water
D_c = Diameter of particle

Inch-pound units	Metric units
$\gamma_w = 62.4\ lb/ft^3$	$\gamma_w = 1.0\ t/m^3$
$\gamma_s = 165\ lb/ft^3$	$\gamma_s = 2.65\ t/m^3$
d = depth, ft	d = depth, m
S = slope, ft/ft	S = slope, m/m
D_c = size, ft	D_c = size, m

Yang Incipient Motion

Yang (1973) developed a relationship between dimensionless critical velocity, V_{cr}/w, and shear velocity Reynold's number, R_*, at incipient motion. Under rough regime conditions where $R_* > 70$, the equation for incipient motion which is considered applicable to bed material size larger than about 2 mm by Reclamation is:

$$\frac{V_{cr}}{w} = 2.05 \qquad (6)$$

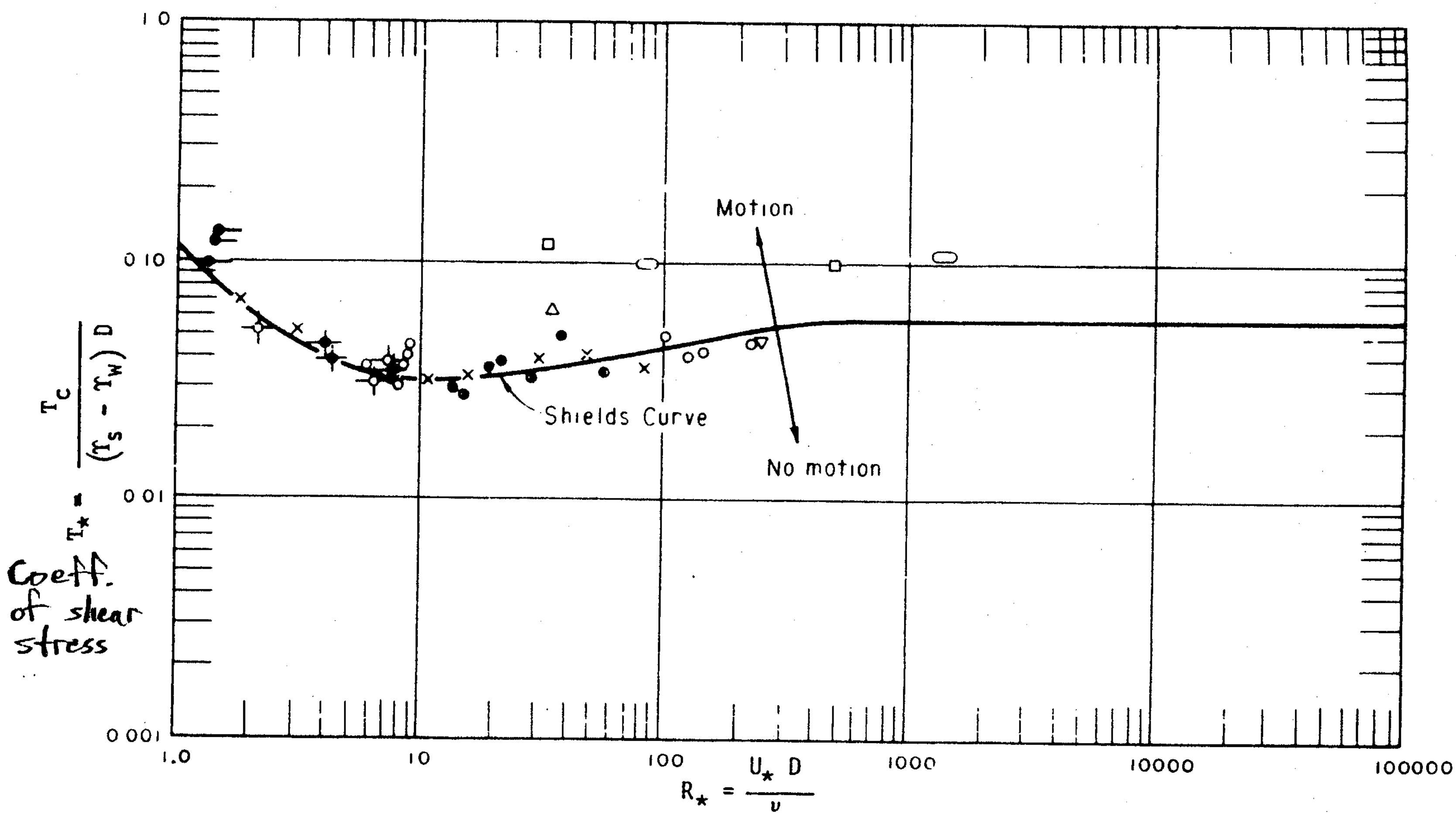
where:

V_{cr} = Critical average water velocity at incipient motion, ft/s (m/s)
w = Terminal fall velocity, ft/s (m/s)

The settling velocity by Rubey (1933) for material larger than 2 mm in diameter will approximate the fall velocity by:

$$w = 6.01\ D_c^{1/2} \text{ inch-pound units} \qquad (7)$$

$$w = 3.32\ D_c^{1/2} \text{ metric units}$$



Fully developed turbulent velocity profile

Sym	Description	γ_s, g/cm3
○	Amber	1.06
●	Lignite (Shields)	1.27
●	Granite	2.7
●	Barite	4.25
×	Sand (Casey)	2.65
✦	Sand (Kramer)	2.65
✦	Sand (U.S.W.E.S.)	2.65
▽	Sand (Gilbert)	2.65

Turbulent boundary layer

●–	Sand (Vanoni)	2.65
–●	Glass beads (Vanoni)	2.49
□	Sand (White)	2.61
⬭	Sand in air (White)	2.10
△	Steel shot (White)	7.9

$U_* =$ Shear Velocity
$= \sqrt{\frac{\tau_c}{\rho}}$ $\quad \rho = \frac{62.4}{32.2} = 1.93$
$= \sqrt{gRS}$

Figure 5. - Shields diagram for initiation of bed material movement

Equations 6 and 7 can be combined to give:

$$D_c = 0.00659\ V_{cr}^2 \text{ inch-pound units} \qquad (8)$$

$$D_c = 0.0216\ V_{cr}^2 \text{ metric units}$$

Depth to Armor and Volume Computations

After determining the size of the material required to armor the streambed, from either an average of the five methods or a judgment decision on the best method, an estimate can be made of the probable vertical degradation before stabilization is reached. The armoring computations assume that an armoring layer will form as shown on figure 6 by the equations:

$$y_a = y - y_d \qquad (9)$$

and

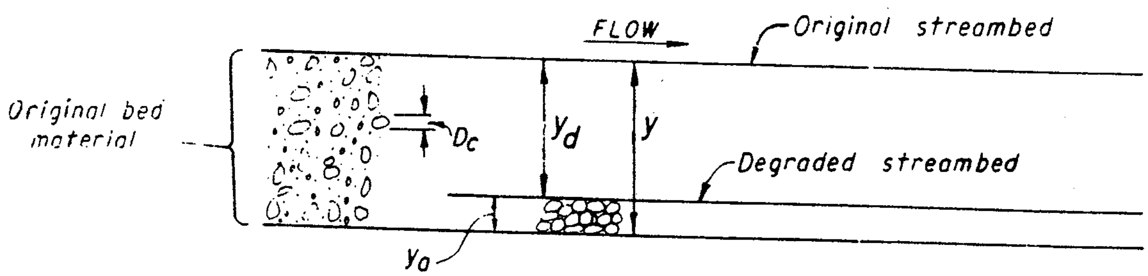
$$y_a = (\Delta p)\ y \qquad (10)$$

which are combined to:

$$\boxed{y_d = y_a\left(\frac{1}{\Delta p} - 1\right)} \qquad (11)$$

where:

y_a = Thickness of armoring layer $\approx 3D_c$
y = Depth from original streambed to bottom of the armoring layer
y_d = Depth from original streambed to top of armoring layer or the depth of degradation
Δp = Decimal percentage of original bed material larger than the armor size, D_c



y = Depth to bottom of the armoring layer

y_d = Depth of degradation

y_a = Armoring layer

D_c = Diameter of armor material

Δp = Decimal percentage of original bed material larger than D_c

Figure 6. - Armoring definition sketch.

The percentage of the bed material equal to or greater than the required armoring size, D_c, can be determined from the bed material size analysis curve from samples collected of the streambed material through the reach involved and at a depth through the anticipated scour zone. This size analysis gives the value Δp to be used in equation 11. The depth, y_a, of the required armoring may vary, depending on the limiting particle size, from a thickness of one particle diameter to three particle diameters or one and three times the armoring size, respectively. A rough guide for use in design is either three armoring particle diameters or 0.5 ft (0.15 m), whichever is smaller. Although armoring has been observed to occur with less than three particle diameters, variability of channel bed material and occurrence of peak discharges dictate the use of a thicker armor layer.

The armoring technique is based on two basic assumptions that may or may not hold for the particular channel studied. The assumptions are: (a) that the degraded channel will have the same hydraulic conditions as the existing channel, and (b) that the ultimate slope of the degraded channel would be equal to the slope of the existing channel. Lateral degradation or erosion of the channel banks may occur simultaneously with armoring of the streambed. A description of the methods for predicting lateral degradation is given in subsequent section "Degradation Limited by a Stable Slope."

An example of the streambed degradation computation limited by armoring using the five recommended methods are given below. The following data are known for the example computations for a channel downstream of a storage dam:

Q = Dominant discharge = 500 ft^3/s (14.2 m^3/s)
B = Channel width = 60 feet (18.3 meters)
d = Mean channel depth = 4 feet (1.22 meters)
V_m = Mean channel velocity = 3.4 ft/s (1.04 m/s)
S = Stream gradient = 0.0021
D_c = Armoring size = diameter in millimeters
n_s = Manning's "n" for bed of stream = 0.03

Meyer-Peter, Muller (bedload transport equation):

Inch-pound units

$$D_c = \frac{dS}{0.19\left(\frac{n_s}{D_{90}^{1/6}}\right)^{3/2}}$$

D_{90} assumed = 34 mm

$$D_c = \frac{4.0\ (0.0021)}{0.19\left(\frac{0.03}{34^{1/6}}\right)^{3/2}}$$

$$D_c = \frac{0.0048}{0.000409} = 20 \text{ mm}$$

Metric units

$$D_c = \frac{dS}{0.058\left(\frac{n_s}{D_{90}^{1/6}}\right)^{3/2}}$$

D_{90} = 34 mm

$$D_c = \frac{1.22\ (0.0021)}{0.058\left(\frac{0.03}{34^{1/6}}\right)^{3/2}}$$

$$D_c = \frac{0.00256}{0.000125} = 20 \text{ mm}$$

Competent bottom velocity:

Inch-pound units

$$D_c = 1.88\ V_m^2$$
$$D_c = 1.88\ (3.4)^2$$
$$D_c = 22\ \text{mm}$$

Metric units

$$D_c = 20.2\ V_m^2$$
$$D_c = 20.2\ (1.04)^2$$
$$D_c = 22\ \text{mm}$$

Lane's tractive force:

Inch-pound units

$$T_c = \Upsilon_w\, dS$$
$$T_c = 62.4\ (4.0)(0.0021)$$
$$T_c = 0.524\ \text{lb/ft}^2$$
D_c from figure 4 = 31 mm

Metric units

$$T_c = \Upsilon_w\, ds$$
$$T_c = 10^6\ \text{g/m}^3\ (1.22)\ (0.0021)$$
$$T_c = 2560\ \text{g/m}^3$$
D_c from figure 4 = 31 mm

Shields diagram:

Inch-pound units

$$D_c = \frac{\Upsilon w d S}{0.06\ (\Upsilon_s - \Upsilon_w)} \approx \frac{\Upsilon R S}{6}$$

$$D_c = \frac{62.4\ (4.0)\ (0.0021)}{0.06\ (165 - 62.4)}$$
$$D_c = 0.0851\ \text{ft}$$
$$D_c = 26\ \text{mm}$$

Metric units

$$D_c = \frac{\Upsilon w d S}{0.06\ (\Upsilon_s - \Upsilon_w)}$$

$$D_c = \frac{1.0\ (1.22)\ (0.0021)}{0.06\ (2.65 - 1)}$$
$$D_c = 0.026\ \text{m}$$
$$D_c = 26\ \text{mm}$$

Yang incipient motion:

Inch-pound units

$$D_c = 0.00659\ V_{cr}^2$$
$$D_c = 0.00659\ (3.4)^2$$
$$D_c = 0.00762\ \text{ft}$$
$$D_c = 23\ \text{mm}$$

Metric units

$$D_c = 0.0216\ V_{cr}^2$$
$$D_c = 0.0216\ (1.04)^2$$
$$D_c = 0.0234\ \text{m}$$
$$D_c = 23\ \text{mm}$$

Mean of the above five methods for computing armoring size is 24 mm, which was adopted as a representative armoring size. By use of equations 10 and 11, a three-layer thickness of nontransportable material to form an armor, and an assumed 17 percent of bed material >24 mm (from size analysis of streambed material), the depth of degradation is:

$$\boxed{y_a = 3D_c} = 3\ (24) = 72\ \text{mm} = 0.236\ \text{ft}\ (0.072\ \text{m})$$

Inch-pound units

$$y_d = y_a\ \left(\frac{1}{\Delta p} - 1\right)$$

$$y_d = 0.236\ \left(\frac{1}{0.17} - 1\right)$$

Metric units

$$y_d = y_a\ \left(\frac{1}{\Delta p} - 1\right)$$

$$y_d = 0.072\ \left(\frac{1}{0.17} - 1\right)$$

y_d = 1.15 ft $\qquad\qquad$ y_d = 0.351 m

It is difficult to determine the distance that degradation will extend downstream when an armoring condition is the limiting factor. With the assumption that the degraded and existing slopes are the same, degradation can be predicted to extend downstream until the volume of material degraded from the channel plus tributary contributions equals the estimated annual volume of eroded material multiplied by some time period usually equal to the economic life of the structure in the following equation form:

$$V_g = V_A\ T \qquad (12)$$

where:

V_g = Total volume of degradation, ft^3 (m^3)
V_A = Estimated annual volume of eroded material, ft^3/yr (m^3/a)
T = Time in years (equals 100 years for most USBR studies)

The actual physical process of degradation begins at the dam and continues downstream with the depth of degradation diminishing in proportion to the sediment load picked up below the dam. As the upstream reach becomes armored, degradation, and, consequently, channel pickup is reduced and the next reach downstream is subjected to a similar degradation process until it armors, after which the process moves on down river.

In the more sophisticated mathematical models degradation computations are made by dividing the stream into reaches. An initial step is to compute the volume of sediment carried out of each reach by the riverflows over a specified time frame. The difference between the volume of material transported out of the reach and that brought into the reach from the immediate upstream reach would determine the degradation in the reach.

DEGRADATION LIMITED BY A STABLE SLOPE

The limiting or stable slope method for computing degradation is based on the degrading process controlled by zero or negligible transport of the material forming the bed of the stream channel. It can be applied to cases where the amount of coarse material is insufficient to form an armoring layer on the channel bed.

The stable slope is determined by application of several methods such as (1) Schoklitsch bedload equation (Shulits, 1935) for conditions of zero bedload transport, (2) Meyer-Peter, Muller (1948) bedload equation for beginning transport, (3) Shields (1936) diagram for no motion, and (4) Lane's (1952) relationship for critical tractive force assuming clear waterflow in canals. Other bedload equations are equally as applicable as the Schoklitsch or Meyer-Peter, Muller equations for zero bedload transport. However, many of these involve trial and error computations until a slope is found to produce negligible bedload transport.

Stable slope computations are made for the dominant discharge which is defined as the flow effecting the ultimate shape and hydraulics of the channel.

Schoklitsch Method

The Schoklitsch equation for zero bedload transport is expressed as follows:

$$S_L = K\left(\frac{DB}{Q}\right)^{3/4} \tag{13}$$

where:

S_L = Stable slope, ft/ft (m/m)
K = 0.00174 inch-pound units (0.000293 metric units)
D = Mean particle size, mm
B = Channel width, ft (m)
Q = Dominant discharge, ft^3/s (m^3/s)

Meyer-Peter, Muller Method

Limiting slope computations by the Meyer-Peter, Muller beginning transport equation are:

$$S_L = \frac{K\left(\frac{Q}{Q_B}\right)\left(\frac{n_s}{D_{90}{}^{1/6}}\right)^{3/2} D}{d} \tag{14}$$

where:

S_L = Stable slope, ft/ft (m/m)
K = 0.19 inch-pound units (0.058 metric units)
$\frac{Q}{Q_B}$ = Ratio of total flow in ft^3/s (m^3/s) to flow over bed of stream in ft^3/s (m^3/s). Usually defined at dominant discharge where $\frac{Q}{Q_B} = 1$ for wide channels
D_{90} = Particle size at which 90 percent of bed material by weight is finer
n_s = Manning's "n" for bed of stream
D = Mean particle size, mm
d = Mean depth, ft (m)

Shields Diagram Method

The use of Shields diagram for computing a stable slope involves the relationship of the boundary Reynold's number R_* varying with the dimensionless shear stress T_* shown on figure 5 as follows:

$$R_* = \frac{U_* D}{\upsilon} \tag{15}$$

where:

R_* = boundary Reynold's number
U_* = Shear velocity $\sqrt{S_L R g}$, ft/s (m/s)
S_L = Slope, ft/ft (m/m)
R = Hydraulic radius or mean depth for wide channels, ft (m)
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)
D = Particle diameter, ft (m)
ν = Kinematic viscosity of water varying with temperature, ft^2/s (m^2/s)

and

$$T_* = \frac{T_c}{(\Upsilon_s - \Upsilon_w)\, D} \longrightarrow D_c = \Upsilon_c / b \qquad (16)$$

where:

T_* = Dimensionless shear stress
T_c = Critical shear stress lb/ft^2 (t/m^2) equal to $\Upsilon_w d S_L$
Υ_s = Specific weight (mass) of particles, 165.4 lb/ft^3 (2.65 t/m^3)
Υ_w = Specific weight (mass), 62.4 lb/ft^3 (1 t/m^3)
d = Mean depth, ft (m)
S_L = Slope, ft/ft (m/m)
D = Particle diameter, ft (m)

Lane's Tractive Force Method

The fourth method suggested for computing the stable slope is to use the critical tractive force relationships shown by Lane (1952). Critical tractive force is defined as the drag or shear acting on the wetted area of the channel bed and is expressed as:

$$T_c = \Upsilon_w d S_L \qquad \Upsilon_c = \Upsilon R S \qquad (17)$$

rewriting in terms of S_L

$$S_L = T_c / \Upsilon_w\, d \qquad (18)$$

where:

T_c = Critical tractive force, lb/ft^2 (t/m^2) (may be read from the curve on figure 4. Enter the abscissa scale with the D_{50} or D_m in millimeters and read the critical tractive force value from the curves for canals with clear water)
Υ_w = Specific weight (mass) of water, lb/ft^3 (t/m^3)
d = Mean water depth for dominant discharge, ft (m)

Example of the Stable Slope Computations

An example problem for a stable or limiting slope, S_L, computation is given below showing the four methods:

Q = Dominant discharge = 780 ft^3/s (22.1 m^3/s)
B = Channel width = 350 ft (107 m)
d = Mean water depth = 1.05 ft (0.32 m)
S = Slope of energy gradient = 0.0014

D = Bed material size D_{50} = 0.000984 ft (0.3 mm)
D_{90} = 0.00315 ft (0.96 mm)
n_s = Manning's "n" for bed of stream = 0.027
$\bar{V}$ = Mean velocity from Manning's equation = 2.13 ft/s (0.649 m/s)
υ = Kinematic viscosity of water = 1×10^{-5} ft^2/s (0.929×10^{-6}) m^2/s

SCHOKLITSCH METHOD:

$$S_L = K \left(\frac{DB}{Q}\right)^{3/4}$$

Inch-pound units	Metric units
$S_L = 0.00174 \left(\frac{0.3 \times 350}{780}\right)^{3/4}$	$S_L = 0.000293 \left(\frac{0.3 \times 107}{22.1}\right)^{3/4}$
$S_L = 0.00174\ (0.222)$	$S_L = 0.000293\ (1.32)$
$S_L = 0.000386$ ft/ft	$S_L = 0.000386$ m/m

MEYER-PETER, MULLER METHOD:

Inch-pound units — Metric units

$$S_L = \frac{K\left(\frac{Q}{Q_B}\right)\left(\frac{n_s}{D_{90}^{\ 1/6}}\right)^{3/2} D}{d}$$

Inch-pound units	Metric units
$S_L = \dfrac{0.19\ (0.3) \left(\dfrac{0.027}{(0.96)^{1/6}}\right)^{3/2}}{1.05}$	$S_L = \dfrac{0.058\ (0.3) \left(\dfrac{0.027}{(0.96)^{1/6}}\right)^{3/2}}{0.32}$
$S_L = \dfrac{0.057\ (0.00448)}{1.05}$	$S_L = \dfrac{0.0174\ (0.00448)}{0.32}$
$S_L = 0.000243$ ft/ft	$S_L = 0.000243$ m/m

SHIELDS DIAGRAM METHOD:

Inch-pound units — Metric units

$$R_* = \frac{U_* D}{\upsilon} \quad \text{vs.} \quad T_* = \frac{T_c}{(\Upsilon_s - \Upsilon_w)\ D} \text{ on figure 5}$$

$$U_* = \sqrt{S\ R\ g}$$

Inch-pound units	Metric units
$R_* = \dfrac{(0.0014 \times 1.05 \times 32.2)^{1/2}\ (0.000984)}{1 \times 10^{-5}}$	$R_* = \dfrac{(0.0014 \times 0.32 \times 9.81)^{1/2}\ (0.0003)}{0.929 \times 10^{-6}}$
$R_* = \dfrac{0.218\ (0.000984)}{0.00001} = 21.5$	$R_* = \dfrac{0.0663\ (0.0003)}{0.929 \times 10^{6}} = 21.4$

Inch-pound units

from figure 5, $T_* = 0.035 = \dfrac{T_c}{(\Upsilon_s - \Upsilon_w)D}$

$$S_L = \frac{0.035\ (165.4 - 62.4)\ (0.000984)}{62.4\ (1.05)}$$

$S_L = 0.0000541$

recompute $R_* = 21.5 \left(\dfrac{0.0000541}{0.0014}\right)^{1/2}$

$R_* = 4.23$

from figure 5, $T_* = 0.039 = \left(\dfrac{T_c}{\Upsilon_s - \Upsilon_w}\right) D$

$$S_L = \frac{0.039\ (103)\ (0.000984)}{62.4\ (1.05)}$$

$S_L = 0.0000603$ ft/ft

Metric units

from diagram, $T_* = 0.035$

$$S_L = \frac{0.035\ (2.65 - 1)\ (0.0003)}{1\ (0.32)}$$

$S_L = 0.0000541$

recompute $R_* = 21.4 \left(\dfrac{0.0000541}{0.0014}\right)^{1/2}$

$R_* = 4.23$

from diagram, $T_* = 0.039 \left(\dfrac{T_c}{\Upsilon_s - \Upsilon_w}\right) D$

$$S_L = \frac{0.039\ (1.65)\ (0.0003)}{1\ (0.32)}$$

$S_L = 0.0000603$ m/m

LANE'S TRACTIVE FORCE METHOD:

$$T_c = \Upsilon_w\, dS_L \text{ or } S_L = T_c/\Upsilon wd$$

Read figure 4 with D = 0.3 mm

Inch-pound units

$T_c = 0.028$ lb/ft^2

$$S_L = \frac{0.028}{62.4\ (1.05)}$$

$S_L = 0.000427$ ft/ft

Metric units

$T_c = 137$ g/m^2

$$S_L = \frac{137}{1 \times 10^6 (0.32)}$$

$S_L = 0.000427$ m/m

The selection of the most appropriate stable or limiting slope can be based on an average of all four methods as shown below in table 2 or can be selected from the technique considered most applicable. In applying any of the methods, some judgmental changes could be made in assumptions of no change in channel hydraulics or bed material particle size analysis. In the example problem, a possible change would be to assume that with degradation the D_{50} could increase to greater than the 0.3 mm. However, this change would be dependent on the characteristics of the particle size distribution curve. In some situations, the stable slope computed by any of the four methods could be equal to or greater than the streambed slope. This would indicate a negligible amount of degradation usually applicable to a streambed that is already armored or the equation is not applicable to this case. Depending on field conditions for an appraisal level investigation, the stable slope could be taken as equal to one-half the streambed slope and used in the computations.

Table 2. - Stable slope

Method	Stable slope ft/ft (m/m)
1. Schoklitsch	0.000386
2. Meyer-Peter, Muller	0.000243
3. Shields diagram	0.0000603
4. Lane's tractive force	0.000427
Average	0.000279

Volume computations

The next step in the degradation computations is to estimate the volume of material expected to be removed from the channel. If there are no downstream controls or bedrock outcrops that would limit the degradation process and little depletion in the steamflow with minor regulation by the reservoir upstream, it can be assumed the stream is capable of picking up a load of coarse sediments (particle sizes greater than 0.0625 mm) equal to that portion of the historic load greater than 0.0625 mm. If, however, the streamflow is depleted or significantly regulated, the sediment load picked up from the channel will be less than the historic load, which is greater than 0.0625 mm. This new sediment load can be determined from a sediment rating curve or plot of stream discharge versus sediment transport specifically for sizes equal to or greater than 0.0625 mm. The rating curve is developed from Modified Einstein computuations described by Colby and Hembree (1955), Bureau of Reclamation (1955), and Bureau of Reclamation (1966) from measured data taken at a section considered representative of the downstream channel degradation reach. If sufficient observed data are not available, a curve can be developed from computed transport values determined by application of appropriate bed material load equations (ASCE, 1975; Simons and Senturk, 1977; and Strand and Pemberton, 1982) that utilize the channel geometry defined by the channel cross sections of the reach being investigated. The annual load determined from this curve by weight (mass) can be converted, through the river density analysis described by Lara and Pemberton (1965), to an annual volume of degradation, V_A.

The annual volume multiplied by a time period T (usually equal to 100 years or the economic life of the structure) gives the total volume of degradation, V_g, in ft^3 (m^3) usually expressed by equation 12.

After determining the stable slope and volume of material removed, the mean depth of degradation applicable to the entire width of the channel at the dam can be computed and the degraded channel profile defined as shown on figure 7. The practical accuracy of the results of this technique improves when the following conditions prevail:

1. The future degraded channel will not differ greatly from the existing channel; thus, the stream channel geometry defined by average cross sections is common to both existing and degraded conditions.

2. The slope of the existing streambed within the expected degraded channel reach is fairly uniform; therefore, an average gradient can be used for the computations.

3. The bed material is considered homogeneous throughout the reach and can be represented by a single particle size gradation curve.

4. The bed is free of any nonerodible barriers that would prevent the stream from degrading to form the average stable section at the stable slope.

The depth of degradation and degraded profiles are determined by the following procedure using the stable slope technique:

First the longitudinal area defined as that area between the existing stream-Bed and degraded streambed (see fig. 7) is computed by the equation:

$$a_g = V_g/B_d \tag{19}$$

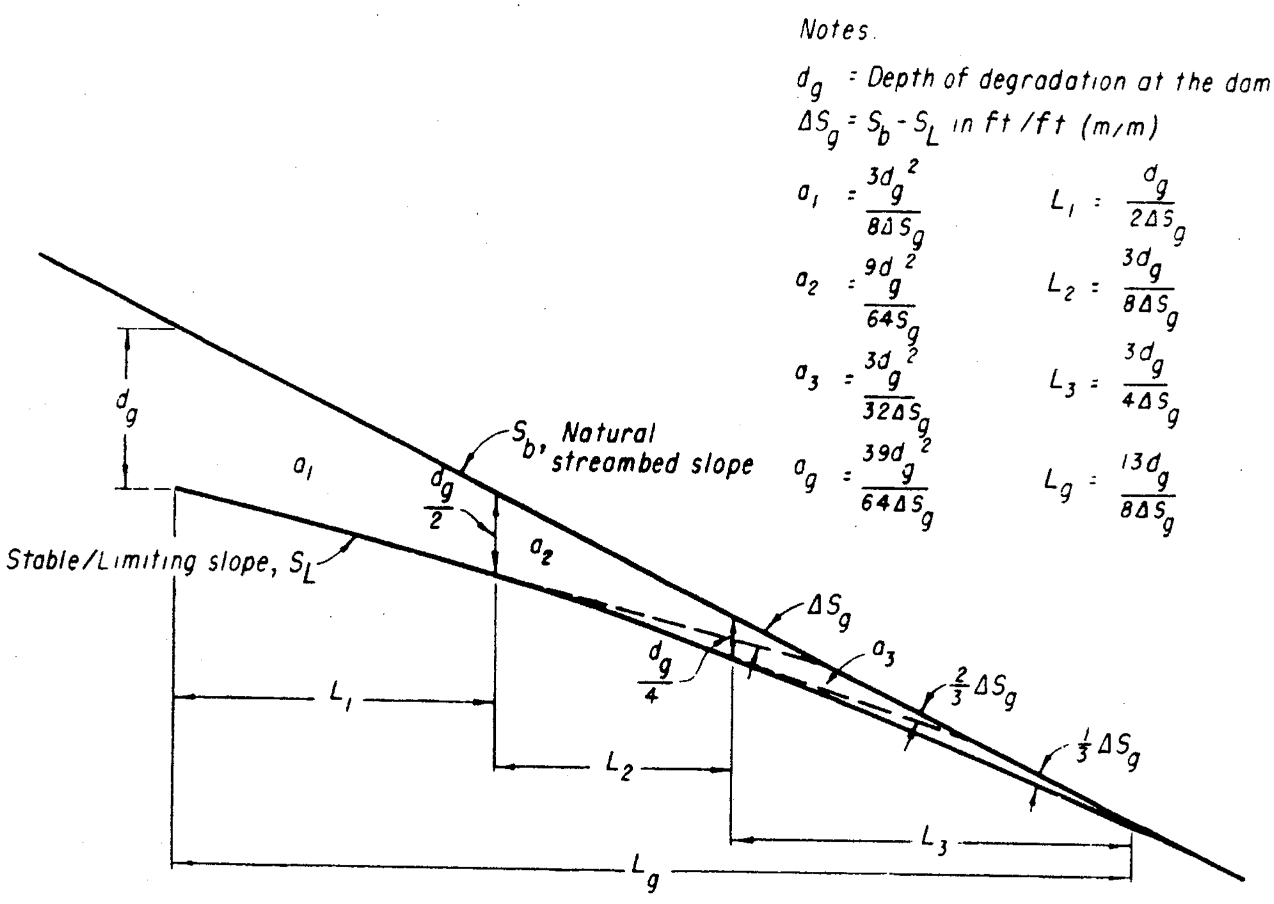


Figure 7. - Degraded channel profile - three-slope method general characteristics.

where:

a_g = Longitudinal area, ft^2 (m^2)
V_g = Volume of degradation, ft^3 (m^3)
B_d = Water surface width for the dominant discharge, ft (m)

The depth of degradation is computed by the equation:

$$d_g = 1.28\ (\Delta S_g a_g)^{0.5} \tag{20}$$

where:

d_g = Depth of degradation, ft (m)
ΔS_g = Difference between the existing streambed slope, S_b, and the stable slope, S_L, ft/ft (m/m)

The length of the degraded channel reach is computed by:

$$L_g = 1.625\ d_g/\Delta S_g \tag{21}$$

where:

L_g = length of the degraded channel reach, ft (m)

Referring to figure 7, the degraded profile can be determined using the diagram and the equations shown to determine the length and slope for each segment of the profile.

If lateral degradation is a significant factor, a special analysis is necessary to determine the degraded channel width. Some lateral movement should be suspected where the banks are composed of similar material as the bed and do not have the necessary vegetation to resist erosion. Where lateral movement is indicated, the extent of vertical degradation generally is not as great because some of the transported material is supplied from the streambanks.

The prediction of bank erosion in a degrading reach of river usually is made by either a permissable velocity and/or tractive force methods. Criteria for determining a degraded width of channel assumes a homogeneous streambank material and that the degradation process eventually will reach a state of equilibrium. The background material for criteria used in application of either method is described by Lane (1952), Lane (1955), and Glover et al. (1951). The procedure outlined by Glover, et al. (1951) requires four basic factors: (a) the tangent of the angle of repose of the bank material, (b) critical tractive force, (c) longitudinal slope of the channel, and (d) a roughness coefficient for use in the Chezy equation. The procedure usually is more applicable to a narrow confined alluvial channel typical of a canal-type section.

The method used for most wider type river channels is to combine the criteria given by Lane (1952) for velocity and critical tractive force with actual field data and Manning's equation in the form:

Inch-pound units	Metric units
$Q = \frac{1.486}{n} B d^{5/3} S_L^{1/2}$	$Q = \frac{1}{n} B d^{5/3} S_L^{1/2}$ metric (22)

The Lane (1952) reference summarizes earlier work by other investigators which included the tabulation by Fortier and Scobey (1926) of limiting velocities compared with values of tractive force for straight channels after aging and is shown in table 3. Table 3 is used primarily in a qualitative manner for comparing tractive forces and velocities for sediment laden channels versus clear water channels.

The first step in the computations for channel widening in a degrading reach of river below an upstream dam is to compute the tractive force and velocity under existing relatively stable channel conditions (with sediment) at a dominant or channel forming discharge. The reduced tractive force or velocity for clear water releases from an upstream dam can then be computed by applying an appropriate adjustment ratio from values given in table 3 or from other criteria such as given in references by Lane (1952) or ASCE (1975). The use of a tractive force adjustment is described in detail in these guidelines, although other techniques involving velocity criteria or regime relationships are considered by many investigators as equally reliable.

In the application of the tractive force method, the reduced tractive force, calculated in accordance with the changes to clear water, is used to predict a new channel cross section by combining equations 4 and 22.

In the previously cited example problem the existing tractive force from equation 4 gives:

Inch-pound units	Metric units
$T_c = 62.4\ (1.05)\ (0.0014)$	$T_c = 1.0\ (0.32\ (0.0014)$
$T_c = 0.092\ \text{lb/ft}^2$	$T_c = 0.000448\ \text{t/m}^2 = 448\ \text{g/m}^2)$

If the material in the banks was "fine sand colloidal", the above tractive forces would, from table 3, be reduced by the ratio of 0.027 ÷ 0.075 (132 ÷ 366 metric) = 0.36. Applying this correction to the existing tractive force gives a clear water tractive force of 0.033 lb/ft^2 (161 g/m^2). This is slightly greater than the tractive force of 0.028 lb/ft^2 (137 g/m^2) read directly from figure 4 for a D = 0.3 mm shown under Lane's tractive force method for computing a stable slope. An average adjustment ratio of 0.5 would apply to most alluvial banks of silt- and sand-size sediments.

In addition to the adjustment for clear water, a correction for sinuosity similar to that described by Lane (1952) for canals is applicable to some rivers as shown in table 4:

Table 3. - Comparison of Fortier and Scobey's limiting velocities with tractive force values (straight channels after aging)
[inch-pound units (metric units)]

Material	Manning's n	For clear water		Water transporting colloidal silts	
		Velocity ft/s (m/s)	Tractive force lb/ft^2 (g/m^2)	Velocity ft/s (m/s)	Tractive force lb/ft^2 (g/m^2)
Fine sand colloidal	0.020	1.50 (0.457)	0.027 (132)	2.50 (0.762)	0.075 (366)
Sandy Loam noncolloidal	0.020	1.75 (0.533)	0.037 (181)	2.50 (0.762)	0.075 (366)
Silt loam noncolloidal	0.020	2.00 (0.610)	0.048 (234)	3.00 (0.914)	0.11 (537)
Alluvial silts noncolloidal	0.020	2.00 (0.610)	0.048 (234)	3.50 (1.07)	0.15 (732)
Ordinary firm loam	0.020	2.50 (0.762)	0.075 (366)	3.50 (1.07)	0.15 (732)
Volcanic ash	0.020	2.50 (0.762)	0.075 (366)	3.50 (1.07)	0.15 (732)
Stiff clay very colloidal	0.025	3.75 (1.14)	0.26 (1270)	5.00 (1.52)	0.46 (2250)
Alluvial silts colloidal	0.025	3.75 (1.14)	0.26 (1270)	5.00 (1.52)	0.46 (2250)
Shales and hardpans	0.025	6.00 (1.83)	0.67 (3270)	6.00 (1.83)	0.67 (3270)
Fine gravel	0.020	2.50 (0.762)	0.075 (366)	5.00 (1.52)	0.32 (1560)
Graded loam to cobbles when noncolloidal	0.030	3.75 (1.14)	0.38 (1860)	5.00 (1.52)	0.66 (3220)
Graded silts to cobbles when colloidal	0.030	4.00 (1.22)	0.43 (2100)	5.50 (1.68)	0.80 (3910)
Coarse gravel noncolloidal	0.025	4.00 (1.22)	0.30 (1460)	6.00 (1.83)	0.67 (3270)
Cobbles and shingles	0.035	5.00 (1.52)	0.91 (4440)	5.50 (1.68)	1.10 (5370)

Table 4. - Sinuosity correction for canals

Degree of sinuosity	Tractive force (%)	Velocity (%)
Straight canals	100	100
Slightly sinuous canals	90	95
Moderately sinuous canals	75	87
Very sinuous canals	60	78

The next step in the width computations by reduced tractive force is to compute the new width, B_1, by combining equation 4 and equation 22 which gives:

Inch-pound units

$$B_1 = \frac{661\ n\ Q\ S_L^{7/6}}{T_C^{5/3}}$$

Metric units

$$B_1 = \frac{n\ Q\ S_L^{7/6} \times 10^{10}}{T_C^{5/3}} \qquad (23)$$

In the example problem for clear water releases using the tractive force method and with no correction for sinuosity,

Inch-pound units

$$B_1 = \frac{661(0.027)(780)(0.000279)^{7/6}}{(0.033)^{5/3}}$$

$$B_1 = \frac{13.9 \times 10^3\ (0.0000713)}{0.00340} = 291 \text{ ft}$$

Metric units

$$B_1 = \frac{0.027(22.1)(0.0000713) \times 10^{10}}{(161)^{5/3}}$$

$$B_1 = 89 \text{ m}$$

The example shows that there would be a reduction in existing width of 350 ft (107 m) to 291 ft (89 m) in the upper reach where the stable slope is 0.000279. However, using figure 7 as an example of the degradation profile and breakdown into subreaches, the degraded width computations from equation 23 are shown in table 5 for the example problem. In table 5, the adjustment in tractive force from clear water to sediment laden water conditions assumes an equal change between reaches as defined by $\frac{\Delta T_C}{3}$.

Table 5. - Degraded width computations by tractive force

Reach	Slope 1/	Tractive force: Adjustment ratio	Tractive force: c lb/ft^2	Tractive force: c (g/m^2)	B_1 = Degraded width from equation 23: ft	B_1 = Degraded width from equation 23: (m)
1	0.000279	0.36	0.033	(151)	291	(89)
2	0.000653	0.57	0.053	(259)	357	(109)
3	0.00103	0.78	0.073	(356)	355	(108)
Natural channel	0.0014	1.00	0.092	(448)	350	(107)

1/ Division of reach into three subreaches with equal change in slope as defined by $\frac{\Delta S_g}{3}$.

The final step for the example problem is the volume computations or the application of equations 12 and 19 through 23 as well as the equations shown on figure 7 for reach lengths. The annual sand (material >0.062 mm) removal is assumed to be equal to the historic sand load of 1×10^6 ft^3/yr (28.3×10^3 m^3/a) and the average width from table 5 equal to 354 ft (108 m). The width in reach 1 was assumed to remain at 350 ft (107 m) rather than reduced to 291 ft (89 m) as shown in table 5.

The longitudinal area in the degraded reach (see fig. 7) is computed for T = 100 years (eq. 12), where $V_{g(100)} = 1 \times 10^8$ ft^3 (2.83×10^6 m^3) by equation 19 as follows:

Inch-pound units

$$a_g = \frac{1 \times 10^8}{354} = 0.282 \times 10^6 \text{ ft}^2$$

Metric units

$$a_g = \frac{2.83 \times 10^6}{108} = 26.2 \times 10^3 \text{ m}^2$$

The depth of degradation is computed by equation 20 as follows:

Inch-pound units

$$d_g = 1.28 \, [(0.0014 - 0.000279) \; 0.282 \times 10^6]^{0.5}$$
$$d_g = 1.28 \, (17.8)$$
$$d_g = 22.8 \text{ ft}$$

Metric units

$$d_g = 1.28 \, [(0.0014 - 0.000279) \; 26.2 \times 10^3]^{0.5}$$
$$d_g = 1.28 \, (5.42)$$
$$d_g = 6.94 \text{ m}$$

The length of the degraded channel reach from equation 21 follows:

$$L_g = \frac{1.625 \, (22.8)}{(0.0014 - 0.000279)}$$

Inch-pound units

$$L_g = \frac{37.05}{0.00112}$$

$$L_g = 33\,100 \text{ ft}$$

Metric units

$$L_g = \frac{1.625\ (6.94)}{0.00112}$$

$$L_g = 10\,100 \text{ m}$$

and for the subreaches:

Inch-pound units

$$L_1 = \frac{22.8}{2\ (0.00112)} = 10\ 200 \text{ ft}$$

$$L_2 = \frac{3\ (22.8)}{8\ (0.00112)} = 7\ 600 \text{ ft}$$

$$L_3 = \frac{3\ (22.8)}{4\ (0.00112)} = 15\ 300 \text{ ft}$$

Metric units

$$L_1 = \frac{6.94}{2\ (0.00112)} = 3\ 100 \text{ m}$$

$$L_2 = \frac{3\ (6.94)}{8\ (0.00112)} = 2\ 300 \text{ m}$$

$$L_3 = \frac{3(6.94)}{4\ (0.00112)} = 4\ 700 \text{ m}$$

CHANNEL SCOUR DURING PEAK FLOODFLOWS

The design of any structure located either along the riverbank and flood plain or across a channel requires a river study to determine the response of the riverbed and banks to large floods. A knowledge of fluvial morphology combined with field experience is important in both the collection of adequate field data and selection of appropriate studies for predicting the erosion potential. In most studies, two processes must be considered, (1) natural channel scour, and (2) scour induced by structures placed by man either in or adjacent to the main river channel.

Natural scour occurs in any moveable bed river but is more severe when associated with restrictions in river widths, caused by morphological channel changes, and influenced by erosive flow patterns resulting from channel alinement such as a bend in a meandering river. Rock outcrops along the bed or banks of a stream can restrict the normal river movement and thus effect any of the above influencing factors. Manmade structures can have varying degrees of influence, usually dependent upon either the restriction placed upon the normal river movement or by turbulence in flow pattern directly related to the structure. Examples of structures that influence river movement would be (1) levees placed to control flood plain flows, thus increasing main channel discharges; (2) spur dikes, groins, riprapped banks, or bridge abutments used to control main channel movement; or (3) pumping plants or headworks to canals placed on a riverbank. Scour of the bed or banks caused by these structures is that created by higher local velocities or excessive turbulence at the strucutre. Structures placed directly in the river consist of (1) piers and piling for either highways or railroad bridges; (2) dams across the river for diversion or storage, (3) grade control structures such as rock cascades, gabion controls or concrete baffled apron drop structures; or (4) occasionally a powerline or tower structure placed in the flood plain but exposed to channel erosion with extreme shifting or movement of a river. All of the above may be subject to higher local velocities, but usually are subject to the more critical local scour caused by turbulence and helicoidal flow patterns.

The prediction of river channel scour due to floods is necessary for the design of many Reclamation structures. These Reclamation guidelines on scour represent a summary of some of the more applicable techniques which are described in greater detail in the reference publications by T. Blench (1969), National Cooperative Highway Research Program Synthesis 5 (1970), C. R. Neill (1973), D. B. Simons and F. Senturk (1977), and S. C. Jain (1981). The paper by S. C. Jain (1981) summarized many of the empirical equations developed for predicting scour of a streambed around a bridge pier. It should be recognized that the many equations are empirically developed from experimental studies. Some are regime-type based on practical conditions and considerable experience and judgment. Because of the complexity of scouring action as related to velocity, turbulence, and bed materials, it is difficult to prescribe a direct procedure. Reclamation practice is to compute scour by several methods and utilize judgment in averaging the results or selection of the most applicable procedures.

The equations for predicting local channel scour usually can be grouped into those applicable to the two previously described processes of either a natural channel scour or scour caused by a manmade structure. A further breakdown of these processes is shown in table 6 where Type A equations are those used for natural river erosion and Types B, C, and D cover various manmade structures.

The importance of experience and judgment in conducting a scour study cannot be overemphasized. It should be recognized that the techniques described in these guidelines merely provide a set of practical tools in guiding the investigator to estimate the amount of scour for use in design. The collection of adequate field data to define channel hydraulics and bed or bank materials to be scoured govern the accuracy of any study. They should be given as much emphasis as the methodology used in the analytical study. Field data are needed to compute water surface profiles for a reach of river in the determination of channel hydraulics for use in a scour study. With no restrictions in channel width, scour is computed from the average channel hydraulics for a reach. If a structure restricts the river width, scour is computed from the channel hydraulics at the restriction. In all cases, scour estimates should be based upon the portion of discharge in and hydraulic characteristics of the main channel only.

Table 6. - Classification of scour equation for various structure designs

	Equation type	Scour	Design
Natural	A	Natural channel for restrictions and bends	Siphon crossing or any buried pipeline. Stability study of a natural bank. Waterway for one-span bridge.
	B	Bankline structures	Abutments to bridge or siphon crossing. Bank slope protection such as riprap, etc. Spur dikes, groins, etc. Pumping plants. Canal headworks.
Man-made	C	Midchannel structures	Piling for bridge. Piers for flume over river. Powerline footings. Riverbed water intake structures.
	D	Hydraulic structures across channel	Dams and diversion dams. Erosion controls. Rock cascade drops, gabion controls, and concrete drops.

Although each scour problem must be analyzed individually, there are some general flow and sediment transport characteristics to be considered in making the judgmental decision on methodology. The general conclusion reached by Lane and Borland (1954) was that floods do not cause a general lowering of streambed, and rivers such as the Rio Grande may scour at the narrow sections but fill up at the wider downstream sections during a major flood. Another general sediment transport characteristic is the influence of a large sediment load on scour which includes the variation of sediment transport associated with a high peak, short duration flood hydrograph. The large sediment concentrations usually of clay and silt size material will occur on the rising stage of the hydrograph up and through the peak of the flood while the falling stage of the flood with deposition of coarser sediments in the bed of the channel may be accompanied by greater scour of the wetted channel banks. Channel scour also occurs when the capacity of streamflow with extreme high velocities in portions of the channel cross section will transport the bed material at a greater rate than replacement materials are supplied. Thus, maximum depth of channel scour during the flood is a function of the channel geometry, obstruction created by a structure (if any), the velocity of flow, turbulence, and size of bed material.

Design Flood

The first step in local scour study for design of a structure is selection of design flood frequency. Reclamation criteria for design of most structures shown in table 6 varies from a design flood estimated on a frequency basis from 50 to 100 years. This pertains to an adequate waterway for passage of the floodflow peak. The scour calculations for these same structures are always made for a 100-year flood peak. The use of the 100-year flood peak for scour is based on variability of channel hydraulics, bed material, and general complexity of the erosive process. The exception in the use of the 100-year flood peak for estimating scour would be the scour hole immediately below a large dam or a major structure where loss of structure could involve lives or represent a catastrophic event. In this case, the scour for use in design should be determined for a flow equal to 50 percent of the structure design flood.

Equation Types A and B (See Table 6)

Natural river channel scour estimates are required in design of a buried pipe, buried canal siphon, or a bankline structure. For most siphon crossings of a river, the cost of burying a siphon will dictate either the selection of a natural narrow reach of river or a restriction in width created by constructing canal bankline levees across a portion of the flood plain. A summary of available methods for computing scour at constrictions is given by Neill (1973). The four methods for estimating general scour at constricted waterways described by Neill (1973) are considered the proper approach for estimating scour for use in either design of a siphon crossing or where general scour is needed of the riverbed for a bankline structure. The four methods supplemented with Reclamation's procedure for application are given below:

Field measurments of scour method. - This method consists of observing or measuring the actual scoured depths either at the river under investigation or a similar type river. The measurements are taken during as high a flow as possible to minimize the influence of extrapolation.

A Reclamation unpublished study by Abbott (1963) analyzed U.S. Geological Survey discharge measurement notes from several streams in the southwestern United States, including the Galisteo Creek at Domingo, New Mexico, and developed an empirical curve enveloping observed scour at the gaging station. This envelope curve for use in siphon design was further supported by observed scour from crest-stage and scour gages on Gallegos, Kutz, Largo, Chaco, and Gobernador Canyons in northwest New Mexico collected during the period from 1963 to 1969. The scour gages consisted of a series of deeply anchored buried flexible tapes across the channel section that were resurveyed after a flood to determine the depth of scour at a specific location. The results of these measurements are shown on figure 8 along with the envelope curve for Galisteo Creek that support scour estimates for wide sandbed (D_{50} varying from 0.5 to 0.7 mm) ephemeral streams in the southwestern United States by the equation.

$$d_s = K\ (q)^{0.24} \qquad (24)$$

where:

d_s = Depth of scour below streambed, ft (m)
K = 2.45 inch-pound units (1.32 metric units)
q = Unit water discharge, ft^3/s per ft of width (m^3/s per m of width)

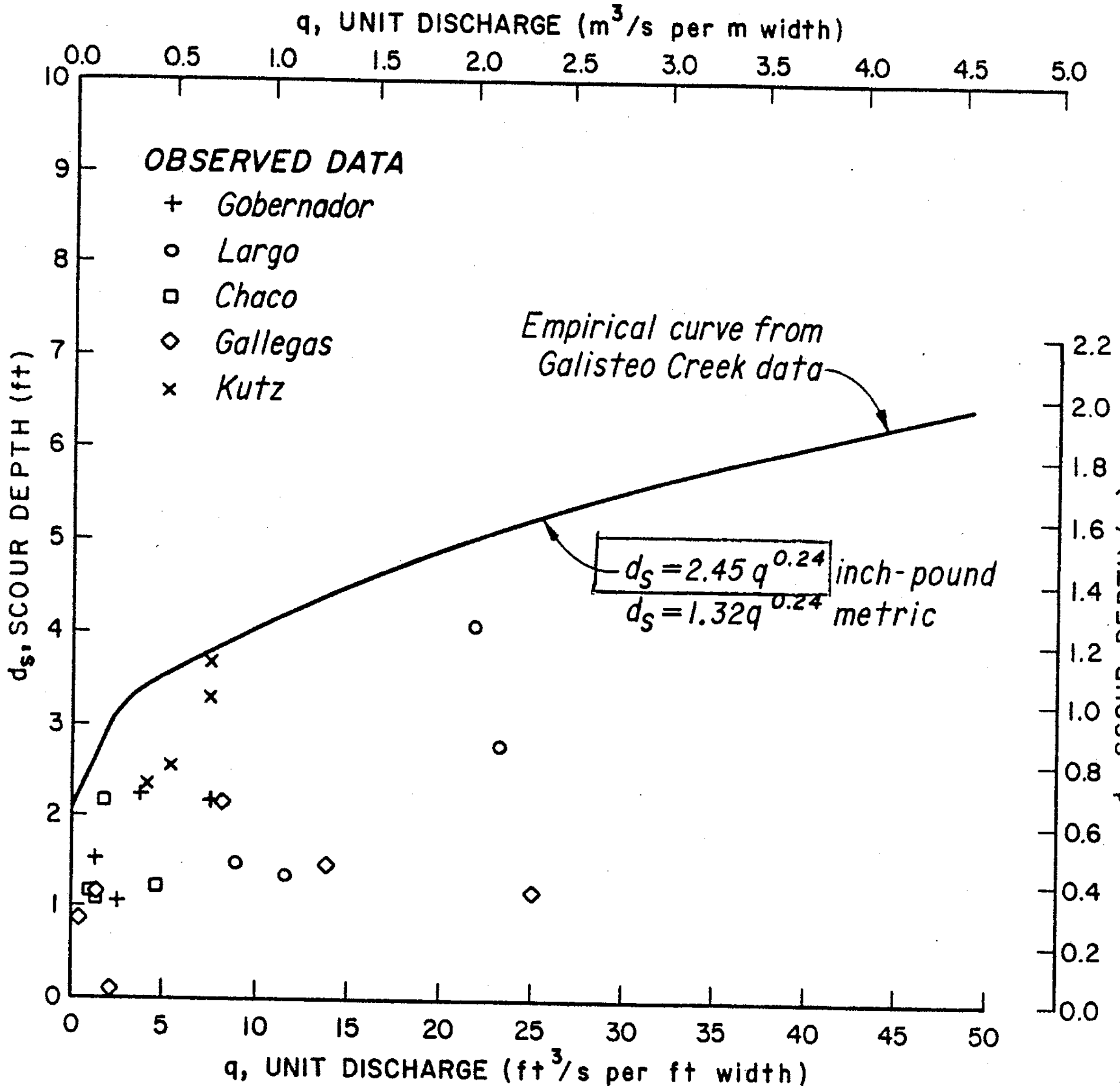


Figure 8. - Navajo Indian Irrigation Project - scour versus unit discharge.

The use of equation 24 except as a check on other methods would be limited to channels similar to those observed on relatively steep slopes ranging from 0.004 to 0.008 ft/ft (m/m). Because of shallow depths of flow and medium to coarse sand size bed material the bedload transport should also be very high.

<u>Regime equations supported by field measurements method</u>. - This approach as suggested by Neill (1973) on recommendations by Blench (1969) involves obtaining field measurements in an incised reach of river from which the bankfull discharge and hydraluics can be determined. From the bankfull hydraulics in the incised reach of river, the flood depths can be computed by:

$$d_f = d_i \left(\frac{q_f}{q_i}\right)^m \tag{25}$$

where:

d_f = Scoured depth below design floodwater level
d_i = Average depth at bankfull discharge in incised reach
q_f = Design flood discharge per unit width
q_i = Bankfull discharge in incised reach per unit width
m = Exponent varying from 0.67 for sand to 0.85 for coarse gravel

This method has been expanded for Reclamation use to include the empirical regime equation by Lacey (1930) and the method of zero bed-sediment transport by Blench (1969) in the form of the Lacey equation:

$$d_m = 0.47 \left(\frac{\overset{q^2}{\cancel{Q}}}{f}\right)^{1/3} = .47\left(\frac{q^2}{f}\right)^{\frac{1}{3}} \tag{26}$$

where:

d_m = Mean depth at design discharge, ft (m)
Q = Design discharge, ft^3/s (m^3/s)
f = Lacey's silt factor equals $1.76\ (D_m)^{1/2}$ where D_m equal mean grain size of bed material in millimeters

and the Blench equation for "zero bed factor":

$$d_{fo} = \frac{q_f^{\ 2/3}}{F_{bo}^{\ 1/3}} \tag{27}$$

where:

d_{fo} = Depth for zero bed sediment transport, ft (m)
q_f = Design flood discharge per unit width, ft^3/s per ft (m^3/s per m)
F_{bo} = Blench's "zero bed factor" in ft/s^2 (m/s^2) from figure 9

The maximum natural channel scour depth for design of any structure placed below the streambed (i.e., siphon) or along the bank of a channel must

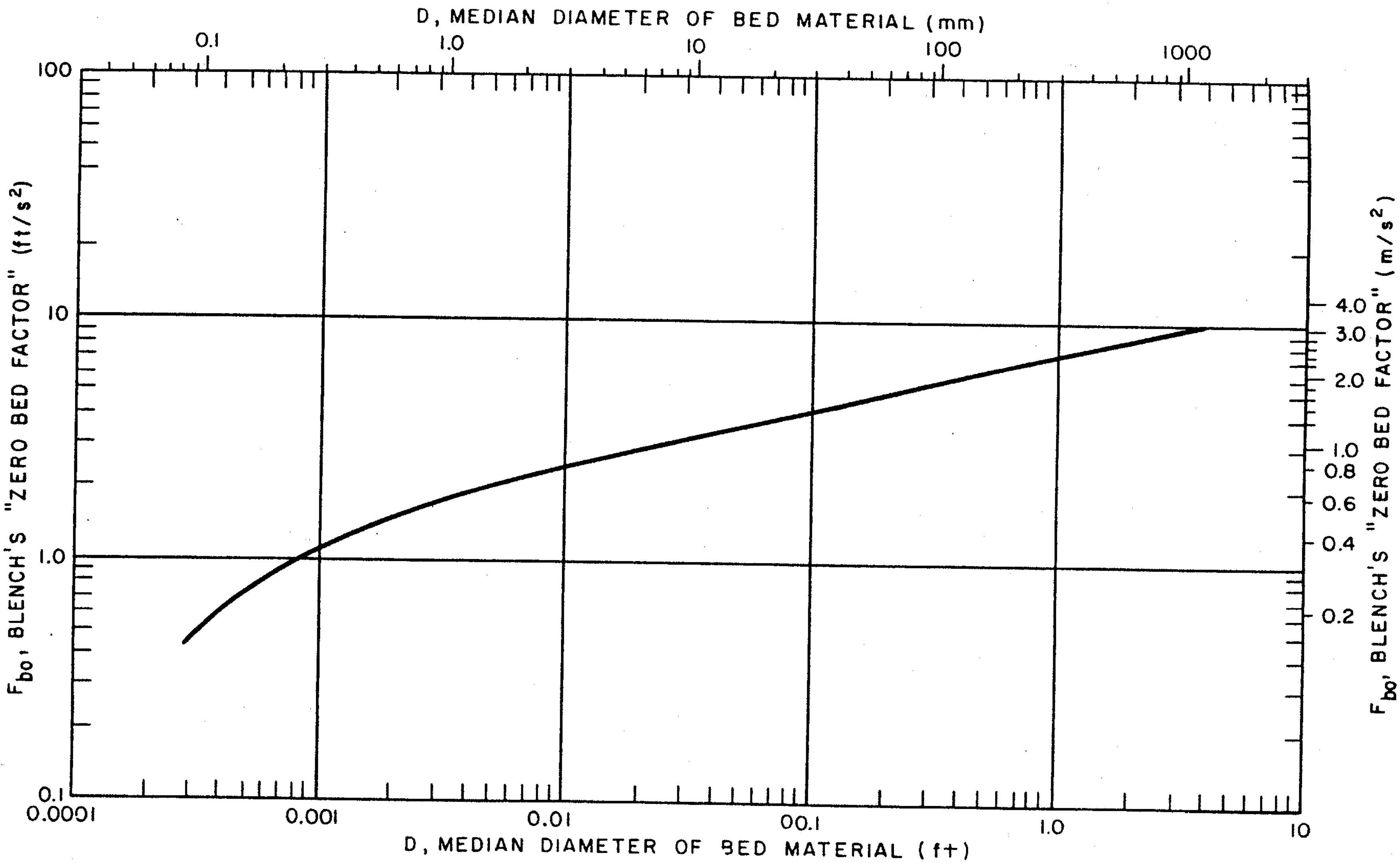


CHART FOR ESTIMATING F_{bo} (AFTER BLENCH)

Figure 9. - Chart for estimating F_{bo} (after Blench, 1969).

consider the probable concentration of floodflows in some portion of the natural channel. Equations 25, 26, or 27 for predicting this maximum depth are to be adjusted by the empirical multiplying factors, Z, shown for formula Types A and B (table 6), in table 7. An illustration of maximum scour depth associated with a flood discharge is shown in a sketch of a natural channel, figure 10. As shown in table 7 and on figure 10, the d_s equals depth of scour below streambed.

$$d_s = Z\ d_f \tag{28}$$

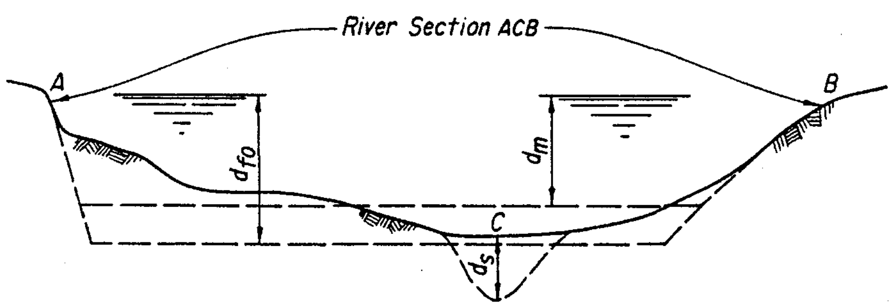
$$d_s = Z\ d_m \tag{29}$$

$$d_s = Z\ d_{fo} \tag{30}$$

Table 7. - Multiplying factors, Z, for use in scour depths by regime equations

Condition	Value of Z		
	Neill $d_s = Z\ d_f$	Lacey $d_s = Z\ d_m$	Blench $d_s = Z\ d_{fo}$
Equation Types A and B			
Straight reach	0.5	0.25	1/ 0.6
Moderate bend	0.6	0.5	
Severe bend	0.7	0.75	
Right angle bends		1.0	1.25
Vertical rock bank or wall		1.25	
Equation Types C and D			
Nose of piers	1.0		0.5 to 1.0
Nose of guide banks	0.4 to 0.7	1.50 to 1.75	1.0 to 1.75
Small dam or control across river		1.5	0.75 to 1.25

1/ Z value selected by USBR for use on bends in river.



NOTE: $d_{fo} > d_f > d_m$. Point C is low point of natural section.

Figure 10. - Sketch of natural channel scour by regime method.

Although not shown on figure 10, the d_f from Neill's equation 25 is usually less than the d_{fo} from Blench's equation 27 but greater than the d_m from Lacey's equation 26.

The design of a structure under a river channel such as a siphon is based on applying the scoured depth, d_s, as obtained from table 7 to the low point in a surveyed section, as shown by point C on figure 10. This criteria is considered by Reclamation as an adequate safety factor for use in design. In an alluvial streambed, designs should also be based on scour occuring at any location in order to provide for channel shifting with time.

Mean velocity from field measurements method. - This approach represents an adjustment in surveyed channel geometry based on an extrapolated design flow velocity. In Reclamation's application of this method, a series of at least four cross sections are surveyed and backwater computations made for the design discharge by use of Reclamation's Water Surface Profile Computer Program. In addition to the surveyed cross sections observed, water surface elevations at a known or measured discharge are needed to provide a check on Manning's "n" channel roughness coefficient. This procedure allows for any proposed waterway restrictions to be analyzed for channel hydraulic characteristics including mean velocity at the design discharge. The usual Reclamation application of this method is to determine the mean channel depth, d_m, from the computer output data and apply the Z values defined by Lacey in table 7 to compute a scour depth, d_s, by equation 29 where $d_s = Z\ d_m$.

Examples of more unique solutions to scour problems were Reclamation studies on the Colorado River near Parker, Arizona, and Salt River near Granite Reef Diversion Dam, Arizona, where an adjustment in "n" based on particle size along with a Z value from table 7 provided a method of computing bed scour. The selection of a particle size "n" associated with scour in the above two examples was computed from the Strickler (1923) equation for roughness of a channel based on diameter of particles where:

$$K = \frac{C}{D_{90}^{1/6}} \tag{31}$$

$C \approx 26$ from Nikuradse (1933) and "n" = 1/K. The appropriate "n" values for the two rivers based on particle size and engineering judgment were selected as follows:

River	D (mm)	Particle size "n"	Selected "n"
Colorado	0.2	0.01	0.014
Salt	18	0.02	0.02

In the Colorado River study, the existing channel "n" value of 0.022 was adjusted down to 0.014 due to bed material particle size to give a computed water surface at design discharge representative of a scoured channel. With a Z value of 0.5, the scoured section in the form of a trianglular section combined with the accepted "n" of 0.022 provided a close check on the water surface computed without scour. An illustration of this technique is shown in sketch on figure 11a. Another example is shown on figure 11b for a Salt River scour study where the particle size "n" of 0.02 gave a reduced mean depth. Scour was assumed to be in the shape of a triangle where the average depth of scour would be equal the depth at an "n" equal to 0.02 subtracted from depth at an "n" equal to 0.03. (See example problem in subsequent paragraph.)

Competent or limiting velocity control to scour method. - This method assumes that scour will occur in the channel cross section until the mean velocity is reduced to that where little or no movement of bed material is taking place. It gives the maximum limit to scour existing in only the deep scour hole portion of the channel cross section and is similar to the Blench equation 27 for a "zero bed factor."

The empirical curves, figure 12, derived by Neill (1973) for competent velocity with sand or coarser bed material (>0.30 mm) represent a combining of regime criteria, Shields (1936) criterion for material >1.0 mm, and a mean velocity formula relating mean velocity V_m to the shear velocity. The competent velocities for erosion of cohesive materials recommended by Neill (1973) are given in table 8. The scour depth or increase in area of scoured channel section with corresponding increase in depth for competent velocity, V_c, is determined by relationship of mean velocity, V_m, to V_c in the equation:

$$d_s = d_m \left(\frac{V_m}{V_c} - 1 \right) \tag{32}$$

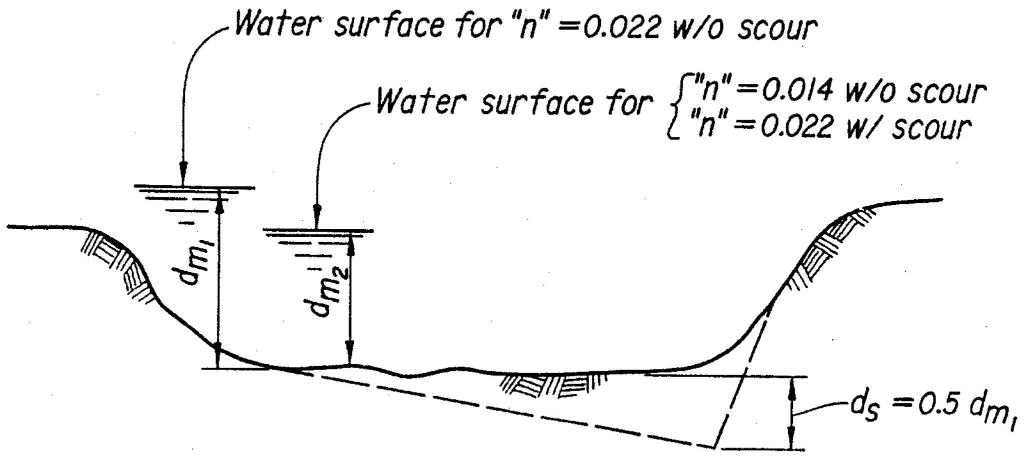
where:

d_s = Scour depth below streambed, ft (m)
d_m = Mean depth, ft (m)

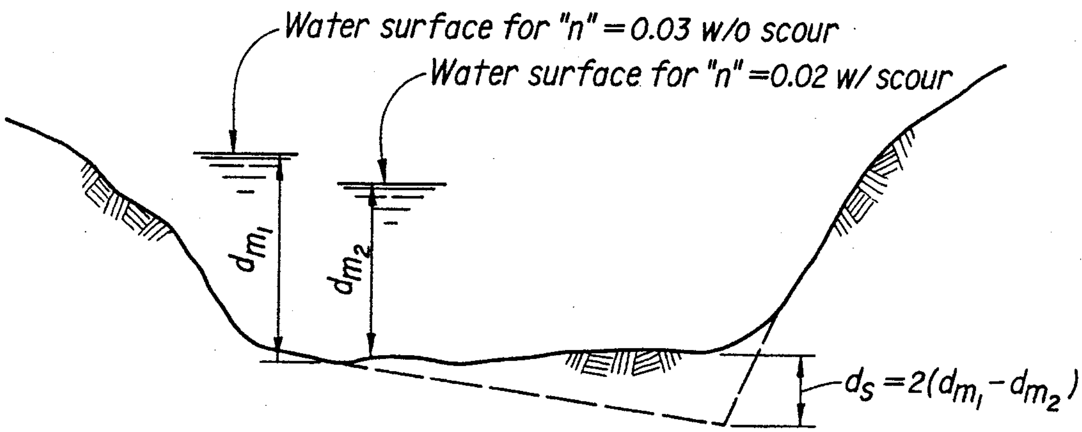
Table 8. - Tentative guide to competent velocities for erosion of cohesive materials* (after Neill, 1973)

Depth of flow		Competent mean velocity					
		Low values - easily erodible material		Average values		High values - resistant material	
ft	m	ft/s	m/s	ft/s	m/s	ft/s	m/s
5	1.5	1.9	0.6	3.4	1.0	5.9	1.8
10	3	2.1	0.65	3.9	1.2	6.6	2.0
20	6	2.3	0.7	4.3	1.3	7.4	2.3
50	15	2.7	0.8	5.0	1.5	8.6	2.6

* Notes: (1) This table is to be regarded as a rough guide only, in the absence of data based on local experience. Account must be taken of the expected condition of the material after exposure to weathering and saturation. (2) It is not considered advisable to relate the suggested low, average, and high values to soil shear strength or other conventional indices, because of the predominating effects of weathering and saturation on the erodibility of many cohesive soils.



a. Colorado River Study



b. Salt River Study

Figure 11. - Sketch of scour from water surface profile computations and reduced "n" for scour.

The use of figure 12 and table 8 recommended by Neill (1973) has had limited application in Reclamation, but appears to be a potential useful technique for many Reclamation studies on scour and armoring of the channel.

Equation Type C (See Table 6)

The principal references for design of midchannel structures for scour such as at bridge piers are National Cooperative Highway Research Program Synthesis 5 (1970), C. R. Neill (1973), Federal Highway Administration, Training and Design Manual (1975), Federal Highway Administration (1980), and S. C. Jain (1981). The numerous empirical relationships for computing scour at bridge piers include one or more of the following hydraulic parameters: pier width and skewness, flow depth, velocity, and size of sediment. The many relations available were further broken down by Jain (1981) to two different approaches: (1) regime, and (2) rational.

The Federal Highway Administration has funded numerous research projects to assist in improving their designs of bridge piers. This research has not resulted in any one recommended procedure. Reclamation's need for scour estimates at midchannel structures is limited. The procedures adopted are to try at least two techniques and apply engineering judgment in selecting an average or most reliable method. The regime approach is to use either equations 26, 27, 28, or 30 and a Z value from table 7. An appropriate Z value to use for piers is 1.0 as found for the railway bridge piers applied to the Lacey equation 29 reported by Central Board of Irrigation and Power (1971).

The rational equation selected for scour at piers is described by Jain (1981) in the form:

$$\frac{d_s}{b} = 1.84 \left(\frac{d}{b}\right)^{0.3} \left(F_c\right)^{0.25} \qquad (33)$$

where:

d_s = Depth of scour below streambed, ft (m)
b = Pier size, ft (m)
d = Flow depth, ft (m)
$F_c = V_c/\sqrt{gd}$ = Threshold Froude number
V_c = Threshold velocity, ft/s (m/s) from figure 12
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)

Equation Type D (See Table 6)

Immediately downstream from any hydraulic structure the riverbed is subject to the erosive action created by the structure. Some type of stilling basin or energy dissipator as described by Reclamation (1977) is provided in the design of such structures to dissipate the energy thereby reducing the erosion potential. There still remains at most structures, below the point where the structure ends and the natural riverbed material begins, a potential for scour. The magnitude of this scour hole will depend on a combination of flow velocity, turbulence, and vortices generated by the structure. Simons and Senturk (1977) describe many of the available equations.

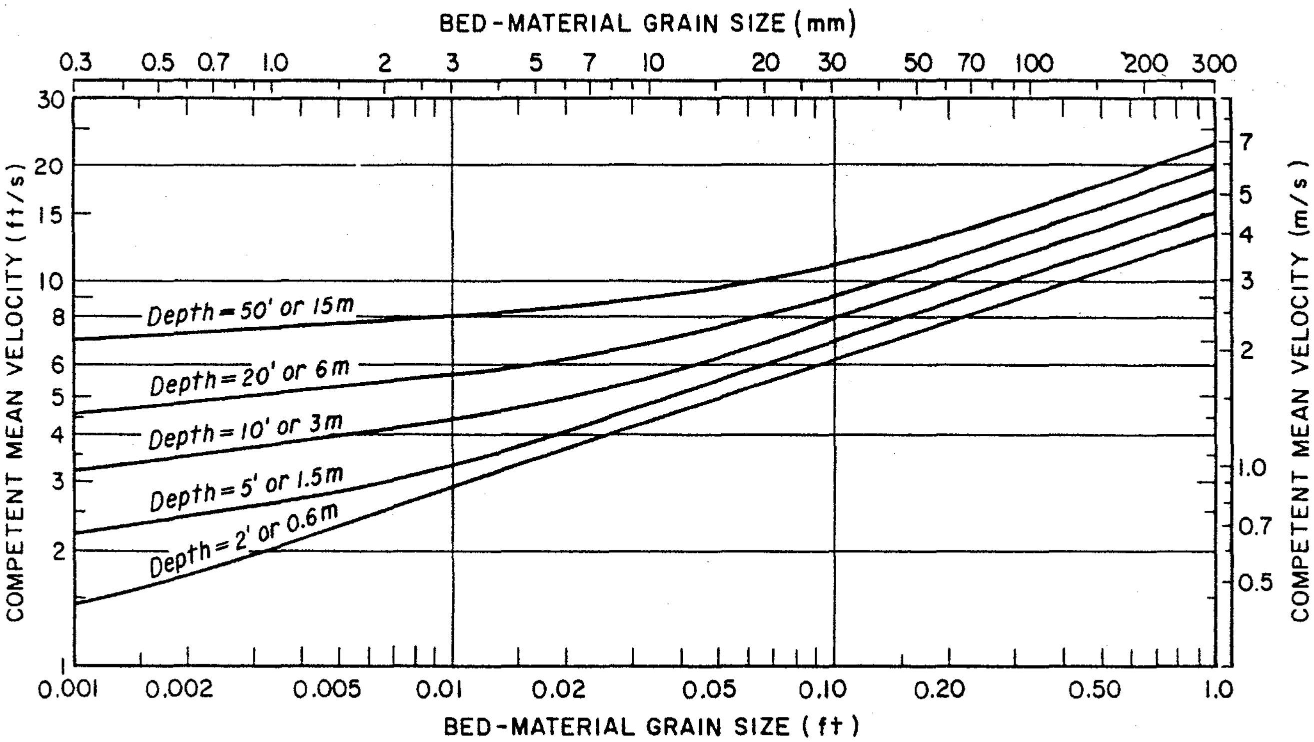


Figure 12. - Suggested competent mean velocities for significant bed movement of cohesionless materials, in terms of grain size and depth of flow (after Neill, 1973).

Methods adopted by Reclamation for computing local scour below a hydraulic structure across the river channel are based on either the regime or rational approach. Scour computations should be made by several methods and engineering judgment used to select the most appropriate. In the regime approach, the Lacey or Blench equations 26, 27, 29, and 30, respectively, with Z values from table 7 are applicable.

The most appropriate empirically developed rational methods for scour below a structure are those by Schoklitsch (1932), Veronese (1937), or Zimmerman and Maniak (1967). Scour computations by Schoklitsch are made by:

$$d_s = \frac{K\ (H)^{0.2}\ q^{0.57}}{D_{90}{}^{0.32}} - d_m \tag{34}$$

where:

d_s = Depth of scour below streambed, ft (m)
K = 3.15 inch-pound units (K = 4.70 metric units)
H = Vertical distance between the water level upstream and downstream of the structure, ft (m)
q = Design discharge per unit width, ft^3/s per ft (m^3/s per m)
D_{90} = Particle size for which 90 percent is finer than, mm
d_m = Downstream mean water depth, ft (m)

The Veronese (1937) equation for computing the scour hole depth below a low head stilling basin design is as follows:

$$d_s = K\ H_T{}^{0.225}\ q^{0.54} - d_m \tag{35}$$

where:

d_s = Maximum depth of scour below streambed, ft (m)
K = 1.32 inch-pound units (K = 1.90 metric units)
H_T = The head from upstream reservoir to tailwater level, ft (m)
q = Design discharge per unit width, ft^3/s per ft (m^3/s per m)
d_m = Downstream mean water depth, ft (m)

The Zimmerman and Maniak (1967) equation for local scour below a stilling basin can be calculated by:

$$d_s = K \left(\frac{q^{0.82}}{D_{85}{}^{0.23}}\right) \left(\frac{d_m}{q^{2/3}}\right)^{0.93} - d_m \tag{36}$$

where:

d_s = Depth of scour below streambed, ft (m)
K = 1.95 inch-pound units (K = 2.89 metric units)
q = Design discharge per unit width, ft^3/s per ft (m^3/s per m)
D_{85} = Particle size for which 85 percent is finer than, mm
d_m = Downstream mean water depth, ft (m)

Example Problem

A scour study was prepared for a reach of the Salt River channel downstream from the existing Granite Reef Diversion Dam and near the Granite Reef Aqueduct which serves as an example of the different methods for computing scour during a design peak flood. These example computations are shown in table 9. The channel hydraulics represent an arithmetic average from water surface profile computations using six sections on the river defining a reach length of 6850 ft (2090 m). To show the many different methods for computing local scour occuring during a flood, several hypothetical situations are used such as a bridge pier, 10-ft (3.05-m) wide and a control structure with a design head, H = 5 ft (1.52 m). A summary of the results is given in table 10.

Table 10. - Summary of channel scour during a floodflow on Salt River

Design	d_s - scour below streambed	
	ft	m
Siphon or bankline structure with minor restriction (A and B)	8.99	2.74
Bridge pier or spur dike from bank (C)	12.2	3.72
Below control structure across river (D)	11.6	3.54

CONCLUSIONS

These guidelines describe the procedures available for computing general river channel degradation and local scour during peak floodflows for use in design of Reclamation structures. Recommendation of a specific method for prediction of either channel degradation or local scour is difficult because of the complexity and variability of the many parameters influencing the erosive action of a river channel. Factors such as river discharges, channel hydraulic characteristics, velocities, turbulence, bedload transport, suspended sediment, bed material size, gradation, and natural rock controls all affect the degradation and erosion process. Most procedures described are empirically developed in laboratory studies with a limited amount of field data on measurement of scour to verify the results. Because of the complexities involved in defining the parameters to use in the equations and variability in results, Reclamation recommended procedure is to try several methods and from experience and engineering judgment select the techniques and results most applicable to the problem.

The field data needed to define the parameters in many equations are critical in the selection and application of a specific procedure. Because of the importance in collection of field data, these guidelines include a description of the appropriate bed material sampling techniques. Through experience investigators continue to emphasize the importance of collecting appropriate field data which governs the accuracy of any analytical study and should be given as much emphasis as the methodology used in the analytical study.

Table 9. - Example problem - Salt River scour study below Granite Reef Diversion Dam

Given data:

	Inch-pound units	Metric units
Q, design discharge	= 110,000 ft^3/s	($3110\ m^3/s$)
B, channel width	= 990 ft	(302 m)
d_m, mean water depth	= 12.4 ft	(3.78 m)
A, water area	= 12,300 ft^2	(1140 m^2)
V_m, mean velocity	= 8.94 ft/s	(2.73 m/s)
q, discharge per unit width	= 111 $ft^3/s/ft$	(10.3 $m^3/s/m$)
D, bed material size	D_{50} = 18 mm D_{85} = 23.5 mm D_{90} = 25 mm	

Equation type	Method	No.	Equations	Computations	d_s - scour ft (m)
A and B	USBR	(24)	(Not considered applicable because of bed material size and extrapolation of curve in figure 8.)		
	Lacey	(26)	$d_m = 0.47 \left(\frac{Q}{f}\right)^{1/3}$ $f = 1.76\ (D_{50})^{1/2}$ $f = 7.47$	$d_m = 0.47 \left(\frac{110\ 000}{7.47}\right)^{1/3} = 11.5$ $(d_m = 0.47 \left(\frac{3110}{7.47}\right)^{1/3} = 3.51)$	
		(29)	$d_s = 0.75\ d_m$ (Severe bend - table 7) $Z = 0.75$	$d_s = 0.75\ (11.5)$ $(d_s = 0.75\ (3.51))$	8.63 (2.63)
	Blench	(27)	$d_{fo} = K\left(\frac{q^{2/3}}{F_{bo}^{1/3}}\right)$ $F_{bo} = 3.6$ (fig. 9) $(F_{bo} = 1.1)$ $F_{bo}^{1/3} = 1.53$ $(F_{bo}^{1/3} = 1.03)$	$d_{fo} = \left(\frac{23.1}{1.53}\right) = 15.1$ $(d_{fo} = \left(\frac{4.73}{1.03}\right) = 4.59)$	
		(30)	$d_s = 0.6\ d_{fo}$	$d_s = 0.6\ (15.1)$ $(d_s = 0.6\ (4.59))$	9.06 (2.75)
	USBR	(29)	$d_s = Z\ d_m$	$d_s = 0.75\ (12.4)$ $(d_s = 0.75\ (3.78))$	9.30 (2.84)
	Neill	(32)	$d_s = d\left(\frac{V_m}{V_c} - 1\right)$ V_c from figure 12 $V_c = 7.0$ ft/s (2.13 m/s)	$d_s = 12.4\left(\frac{8.94}{7} - 1\right)$ $(d_s = 3.78\left(\frac{2.73}{2.13} - 1\right))$	3.4 (0.45)
	Average = 8.63 + 9.06 + 9.30 + 3.4* ÷ 3 (2.63 + 2.75 + 2.84 + 1.06* ÷ 3) * disregard in averaging				8.99 (2.74)
C Bridge pier with assumed pier width b = 10 ft (3.05 m)	Lacey	(29)	$d_s = 1.0\ d_m$	$d_s = 1.0\ (11.5)$ $(d_s = 1.0\ (3.51))$	11.5 (3.51)
	Blench	(30)	$d_s = 0.7\ d_{fo}$ $Z = 0.7$ for pier	$d_s = 0.7\ (15.1)$ $(d_s = 0.7\ (4.59))$	10.6 (3.21)
	Jain	(34)	$d_s = b\ [1.84 \left(\frac{d_m}{b}\right)^{0.3} \left(F_c\right)^{0.25}]$ $F_c = \frac{V_c}{(gd)^{1/2}}$ V_c from figure 12 $F_c = \frac{7.0}{(32.2 \times 21.4)^{1/2}} = 0.3$	$d_s = 10\ [1.84\ (1.24)^{0.3}\ (0.3)^{0.25}]$ $d_s = 10\ (1.46)$ $(d_s = 3.05\ [1.84\ (1.24)^{0.3}\ (0.3)^{0.25}])$	14.6 (4.45)
	Average = 11.5 + 10.6 + 14.6 ÷ 3 (3.51 + 3.21 + 4.45 ÷ 3)				12.2 (3.72)
D	Schoklitsch Assume H = 5 ft (H = 1.52 m)	(34)	$d_s = \frac{K\ (H)^{0.2}\ q^{0.57}}{D_{90}^{\ 3/2}} - d_m$	$d_s = \frac{3.15\ (5)^{0.2}\ (111)^{0.57}}{(25)^{0.32}} - 12.4$ $d_s = 3.15\ (7.20) - 12.4$ $d_s = 22.7 - 12.4$ $(d_s = \frac{4.7\ (1.52)^{0.2}\ (10.3)^{0.57}}{2.8} - 3.78)$	10.3 (3.14)
	Veronese	(35)	$d_s = K\ H_T^{0.225}\ q^{0.54} - d_m$	$d_s = 1.32\ (5)^{0.225}\ (111)^{0.54} - 12.4$ $d_s = 24.2 - 12.4$ $(d_s = 1.9\ (1.52)^{0.225}\ (10.3)^{0.54} - 3.78)$	11.8 (3.58)
	Zimmerman and Maniak	(36)	$d_s = K\left(\frac{q^{0.82}}{D_{85}^{0.23}}\right)\left(\frac{d_m}{q^{2/3}}\right)^{0.93} - d_m$	$d_s = 1.95\left(\frac{111^{0.82}}{23.5^{0.23}}\right)\left(\frac{12.4}{111^{2/3}}\right)^{0.93} - 12.4$ $d_s = 1.95\left(\frac{47.6}{2.07}\right)\left(\frac{12.4}{23.1}\right)^{0.93} - 12.4$ $d_s = 1.95\ (23.0)\ (0.561) - 12.4$ $(d_s = 2.89\left(\frac{10.3^{0.82}}{2.07}\right)\left(\frac{3.78}{4.73}\right)^{0.93} - 3.78)$ $(d_s = 2.89\ (3.27)\ (0.812) - 3.78)$	12.8 (3.89)
	Average = 10.3 + 11.8 + 12.8 ÷ 3 (3.14 + 3.58 + 3.89 ÷ 3)				11.6 (3.54)

1/ All computations given in inch-pound units except those given in parenthesis, which indicate metric units.

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SKUNK CREEK, NEW AND AGUA FRIA RIVERS

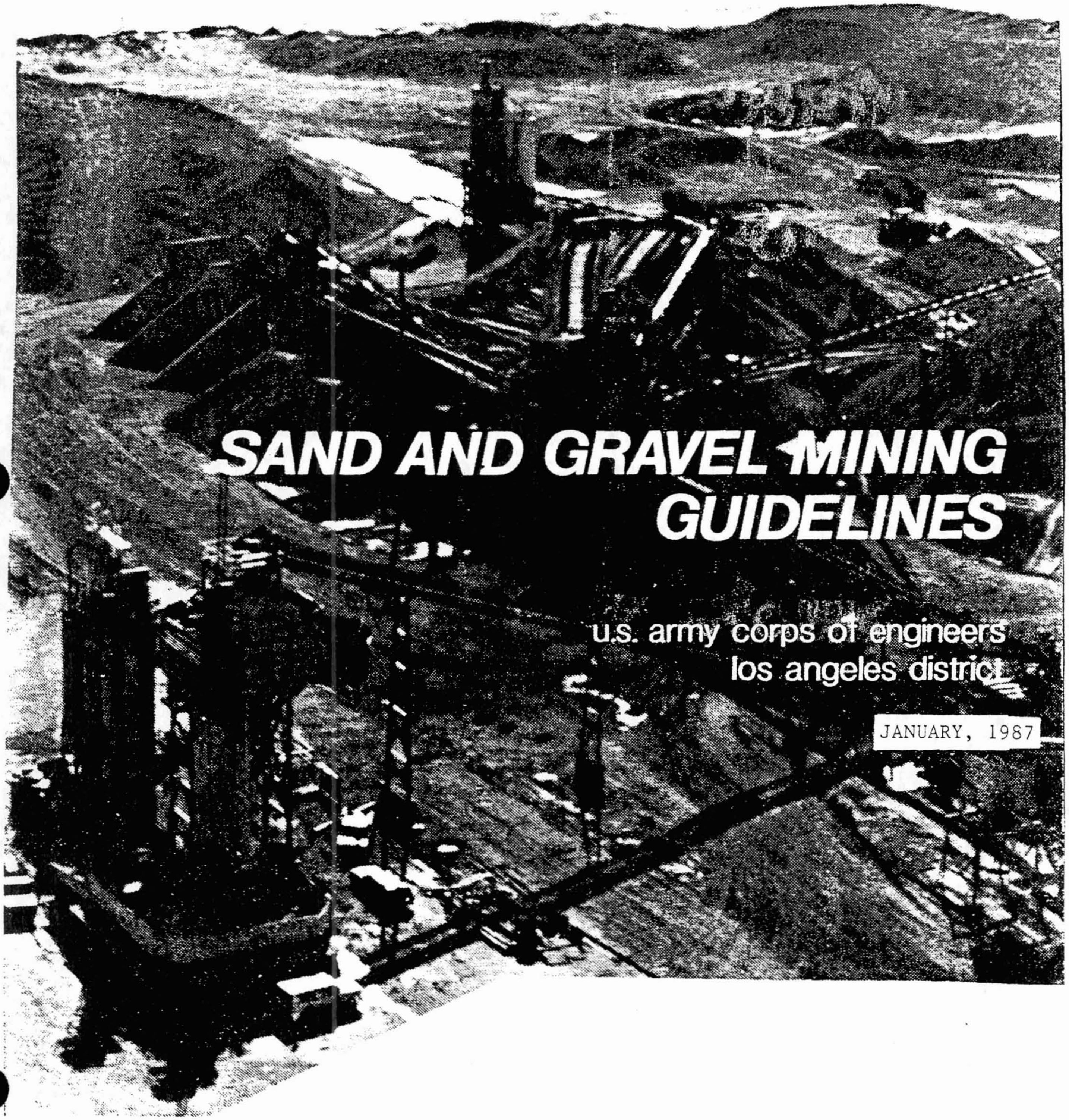

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Table of Contents

I. INTRODUCTION

This document presents guidelines for sand and gravel mining operations in selected reaches of Skunk Creek, New River, and the Agua River near Phoenix, Arizona. The objective of these guidelines is to delineate the extent of permissible mining activity which is consistent with the design of the federal flood control improvements along these streams. Implementation of these guidelines by local interests would ensure the structural integrity of those flood control improvements during storm and flood events.

The guidelines were developed by first conducting a literature search to learn how previous engineering studies have approached similar problems. Based on those findings an engineering analysis was performed to address the site specific characteristics of the Phoenix, Arizona area. The objective was to establish acceptable mining practices that would not result in a compromise of the flood control features which provide protection from floods and erosion damages. These guidelines do not consider the other potential environmental impacts of sand and gravel mining. Support documentation is contained in the appendixes.

II. GRAVEL MINING OPERATION GUIDELINES IN THE VICINITY OF CORPS OF ENGINEERS STRUCTURES.

1. All extraction of streambed and overbank materials should be conducted in accordance with plans that have received prior official approval of the regulatory agency Flood Control District Maricopa County (FCDMC).

2. All excavation operations should be conducted in such a manner as to cause no obstruction of the natural flow in waterways, and cause no damage to adjacent structures or properties. No excavation operation, no stockpiling of any kind, and no other obstructions are to be permitted in the floodway during the months of highest flood risk which are June through September and December through March.

3. The extraction operation will be required to limit its streambed influence to the extraction property boundaries as per the approved extraction plan. Instream extraction will be limited to an extraction depth that is controlled and defined by an extraction flow line elevation profile. The extraction flow line elevation profile is identical to the thalweg elevation profile as shown on the applicable Plan and Profile sheet and table 3 and 4 in the Attachment to Appendix 1 "Floodplain Delineation Document" as contained in Skunk Creek and the New and Agua Fria Rivers Design Memorandum #3 dated May 1986. Present and future channel inverts are also defined as extraction flow lines. For Corps of Engineers (COE) projects in which bank stabilization or flood protection are to be constructed, adequate depths of toe protection will be provided below the extraction flow line. No instream extraction will be permitted within 5 feet of the bank stabilization and levee slopes as shown in figure 1a. These areas for which the extraction controls are reduced apply to the following reaches: (1) Skunk Creek east stabilized bank upstream of 83rd Avenue; (2) New River both stabilized banks from Grand to Olive Avenues; and (3) Agua Fria River west levee from Buckeye Road to about 3900 feet downstream of Lower Buckeye Road. However, an exception to the permissible extraction criteria occurs at the terminus of the Skunk Creek bank stabilization about 977 feet upstream of the 83rd Avenue bridge. In this Skunk Creek reach, a minimum extraction boundary

divergence angle of 1 lateral on 4 longitudinal from the longitudinal centerline would be required. The divergence would start at a point on the top of the bank 200 feet downstream of the grouted stone terminus would be required. The 200 foot buffer zone is required to prevent undercutting of the grouted stone tieback by potential future gravel operations. Figure 1b illustrates this modified extraction criteria.

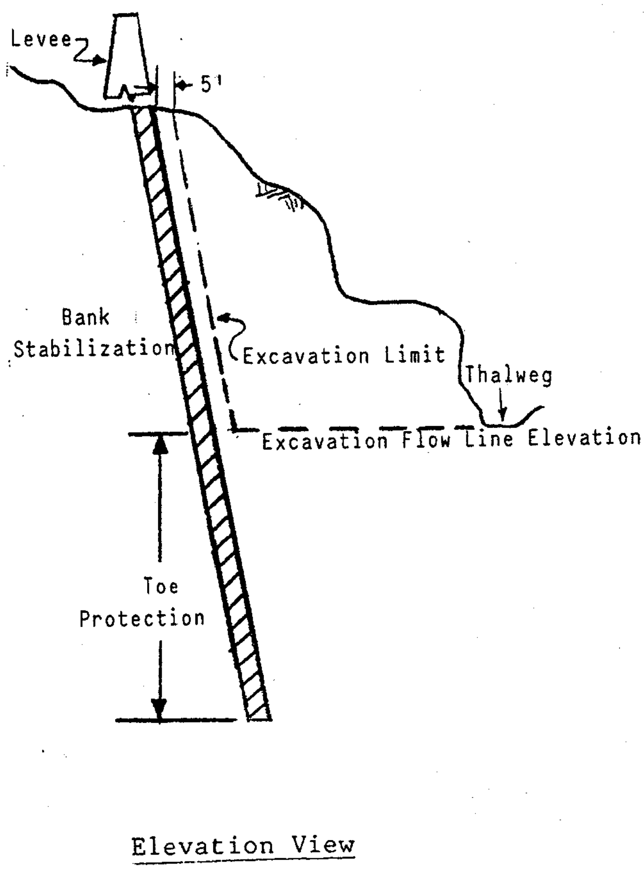


Figure 1a.

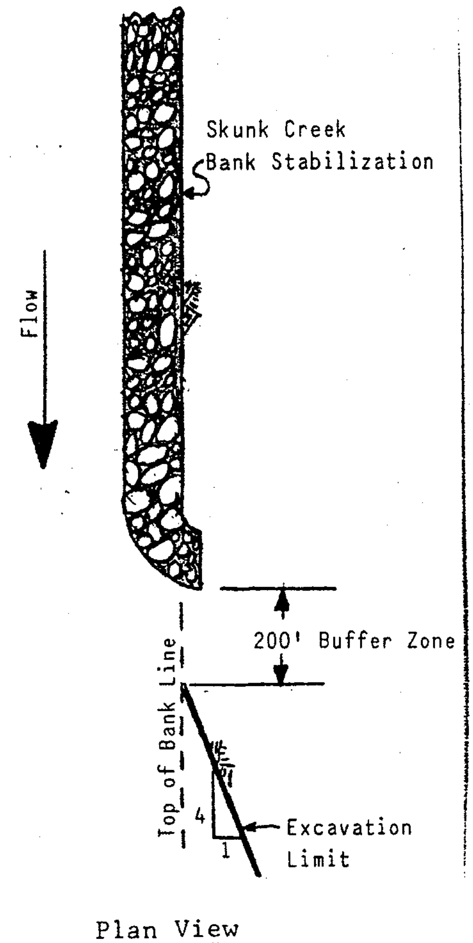


Figure. 1b.

4. Overbank extraction operations on the land side of the COE stabilized banks and levees shall be controlled to prevent floodwaters from damaging project structures. Cut off walls protecting the pit operation, may be required as a local option in order to prevent the floodflows from causing upstream head cutting. Thus excavation would be prohibited within a strip extending 200 feet landward and below a plane made by a 1V on 5H slope as shown in figure 2.

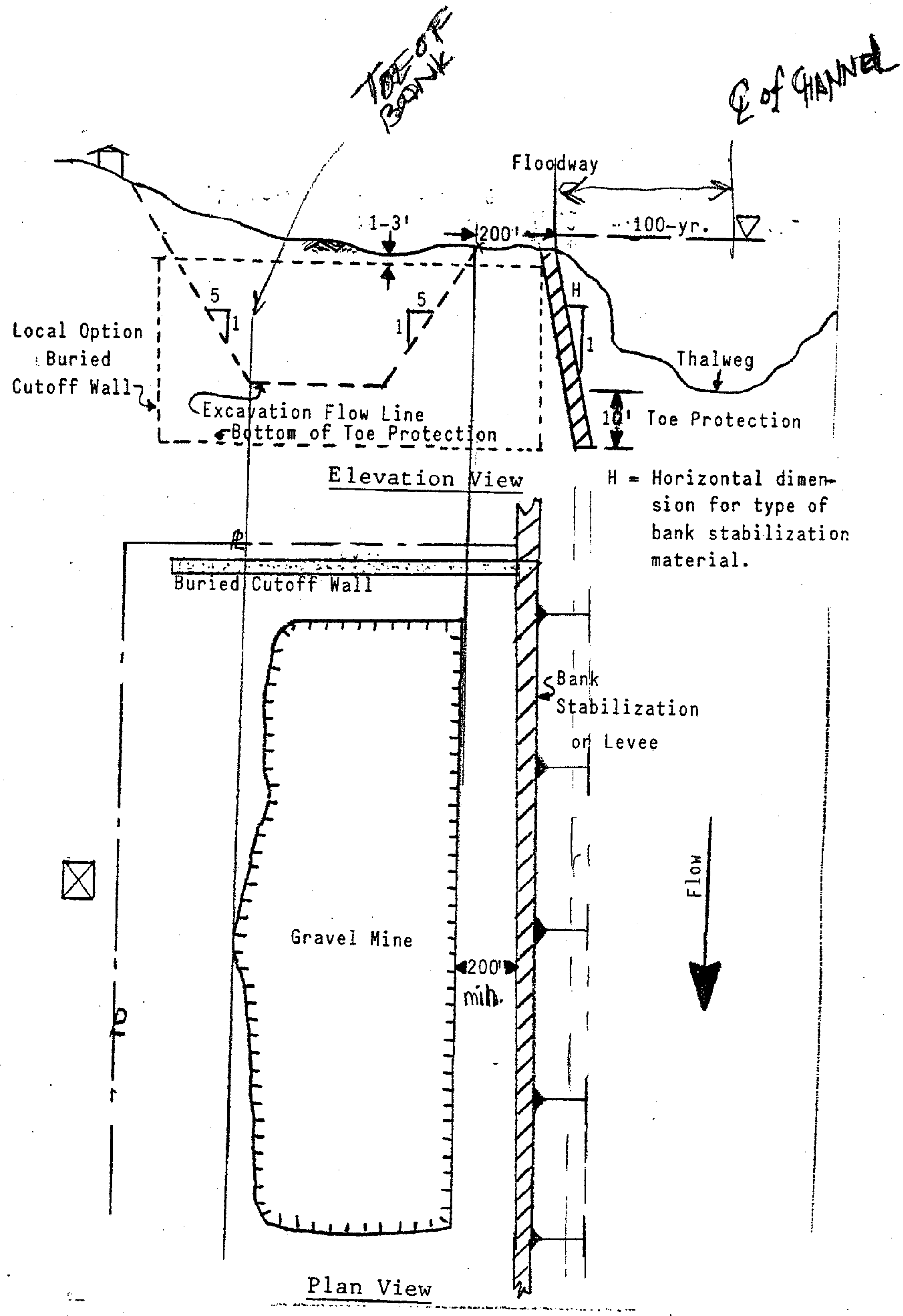


Figure 2. Overbank Gravel Mining in Reaches with Bank Stabilization.

5. No excavation will be permitted below the extraction flow line elevation.

6. All extraction operations must be performed on the basis of a continuous pit within the property of any one operation. Leapfrog operations will not be permitted; and the continuous pit must not be sinuous with respect to either the alignment or grade of the stream.

7. In cases where there are potential adverse hydraulic effects from an extraction operation, the owner will provide the regulatory agency with the necessary engineering analysis, performed by a qualified engineer, showing that there are no significant adverse effects, or if there are, that they can and will be mitigated.

8. COE flood control features must not be damaged by the extraction machinery or processes. Any inadvertant damage will be promptly repaired at the extraction operators expense. Repairs must meet original specifications and to the complete satisfaction and approval of the COE or its representatives.

III. SUGGESTED GRAVEL MINING OPERATIONS GUIDELINES AT LOCATIONS NOT ADJACENT TO COE STRUCTURES

It is suggested that guidelines adopted by local regulatory agencies acknowledge the economic value of aggregate mining, as well as protecting other values and activities in the flood plain. The adopted guidelines should be implemented through a permit process which considers existing, as well as, future intended uses of the flood plain. Sand and gravel operations would be liable for damages resulting from failure to adhere to permit requirements.

1. All extraction of streambed material should be conducted in accordance with plans that have received prior official approval of the regulatory agency.

2. All excavation operations should be conducted in such a manner as to cause no obstruction of the natural flow in waterways, and cause no damage to adjacent structures or properties. No excavation operations, no stockpiling of any kind, and no other obstructions should be permitted in the floodway during the months of highest flood risk which are June through September and December through March.

3. The extraction operation should limit its streambed influence to the extraction property boundaries as per the approved plan. The upstream face of mines which predate the established excavation flow line depth of excavation, should be provided with drop structures or invert stabilizers to preserve the natural stream grade and to prevent head cutting during all floodflows. The downstream end of the pit should also be provided with an invert stabilizer to maintain the pre-extraction operation natural invert elevations during all floodflows. An approximately 500 foot long transition channel should be made

an integral part of the downstream interface between the instream gravel mine and the existing riverbed. The transition channel would permit the reestablishment of the natural river flow regime to prevent downstream riverbed degradation.

4. An alternative to the upstream pit face invert stabilizers would be to control the pit excavation maximum depth so that the upstream grade cannot exceed one percent as measured between the midpoint elevation of the upstream pit face and the nearest point in the streambed 500 feet downstream of an existing structure or utility crossing. This alternative is illustrated in figure 3. If it can be shown by engineering analysis that the excavations would have no adverse effect on the upstream structure or utility crossing, then the upstream length constraint may be relaxed.

5. Instream gravel mines, with unprotected natural river banks, should have a 500 foot buffer zone that projects into the stream from the top of the bank or floodway line and then extended at a side slope of 1V to 10H to the established flow line depth. Lateral extension of the instream gravel mine may be permitted where the gravel mine operator's property also include the overbank mineral rights. <u>But for unprotected gravel mine banks, no mining should be permitted within a 500 foot minimum buffer zone and a plane extending to the flow line depth on a 1V on 10H slope relative to the lateral property line</u>. However, no lateral buffer zone and sloped plane should be required where the gravel mine banks are stabilized in a manner approved by the responsible regulatory agency and incorporate a minimum depth of toe protection of 10 feet below the thalweg.

The upstream and downstream buffer zone should be a minimum distance of 200 feet from the gravel mine operator's property lines. In addition the excavation operation should include a gradual expansion of the upstream incoming banks. Specifically, relative to the stream, the modified banks should expand at a ratio of 1 to 4 for each side. Similarly, the downstream end of the pit contraction ratio should be 1 on 2. Figures 4 and 5 illustrate several suggested gravel mine operational plans.

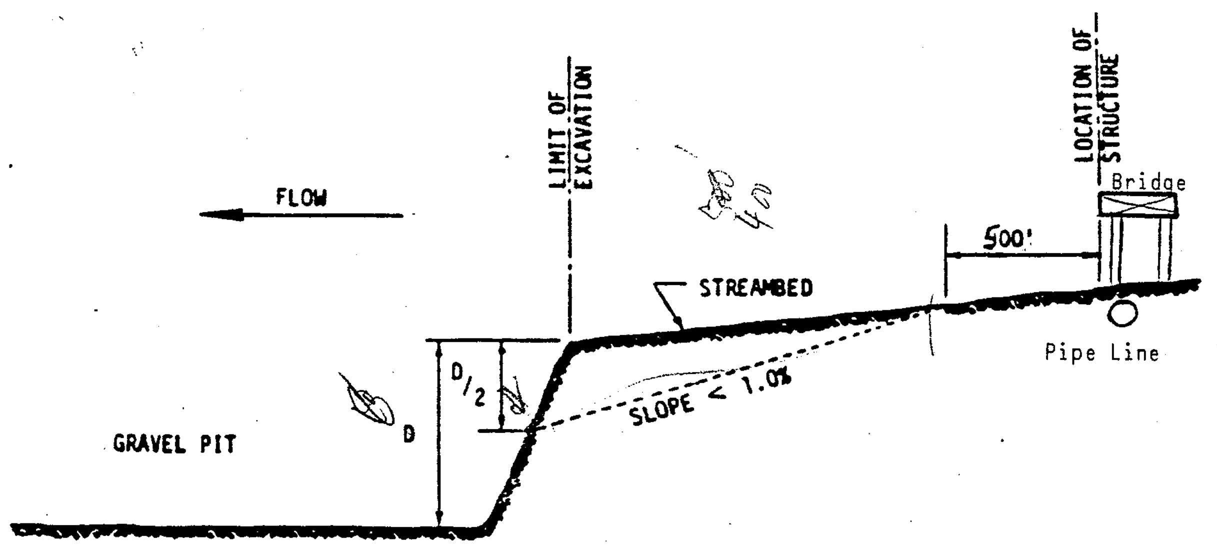


Figure 3. Limit of Excavations Downstream of a Hydraulic Structure.

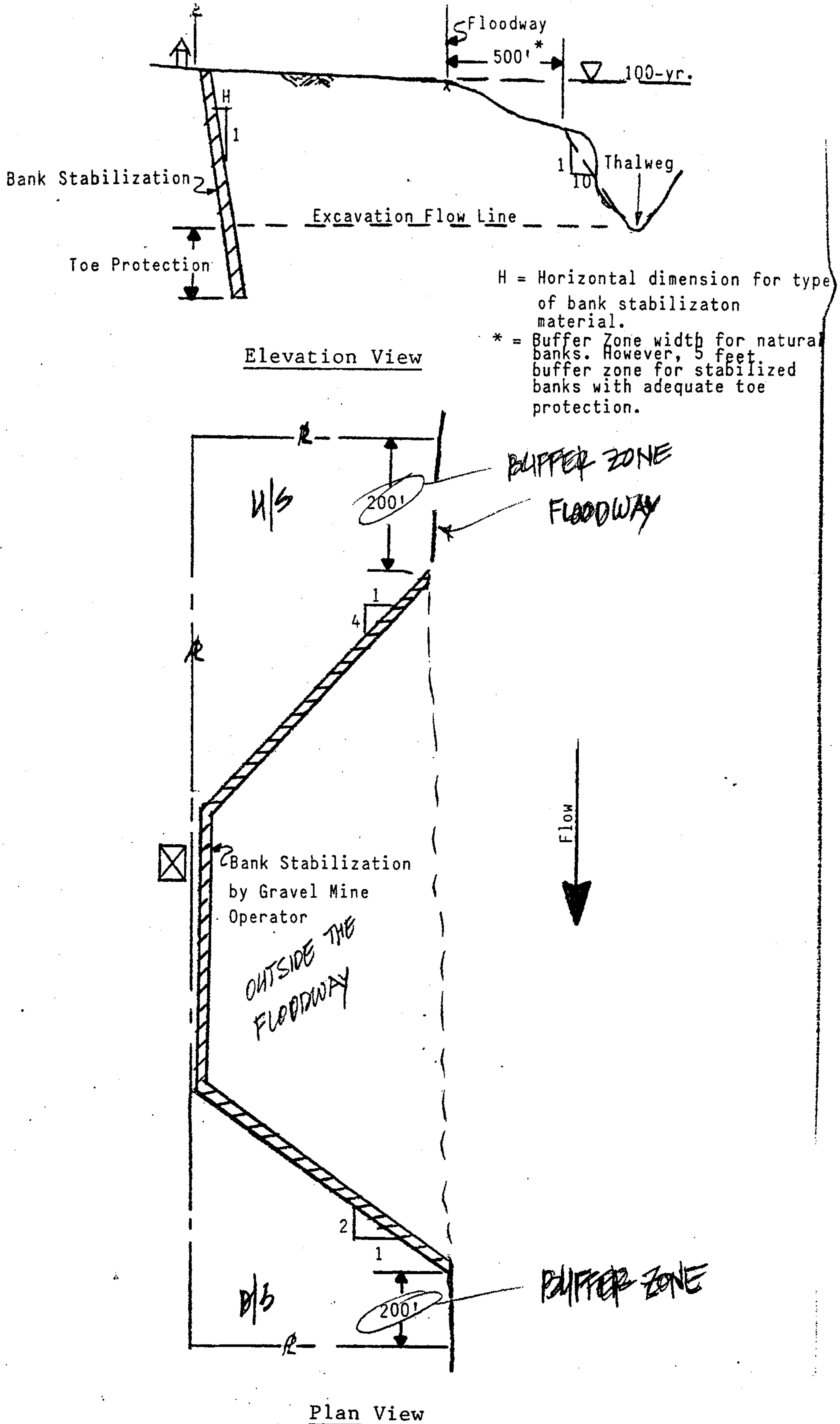
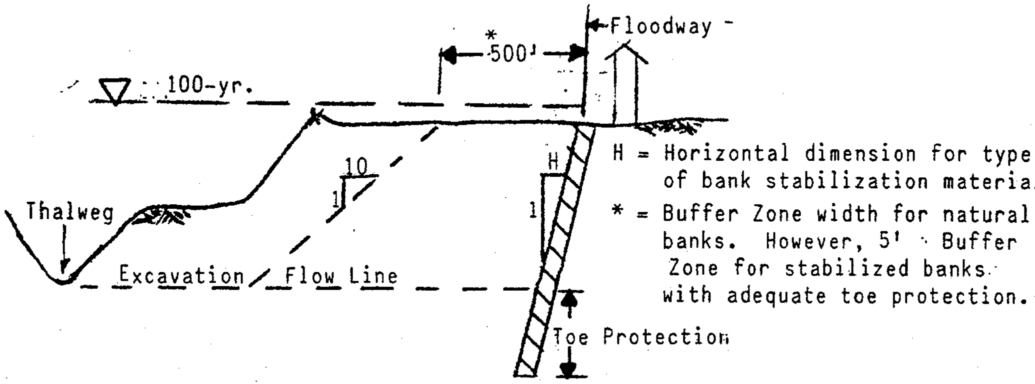
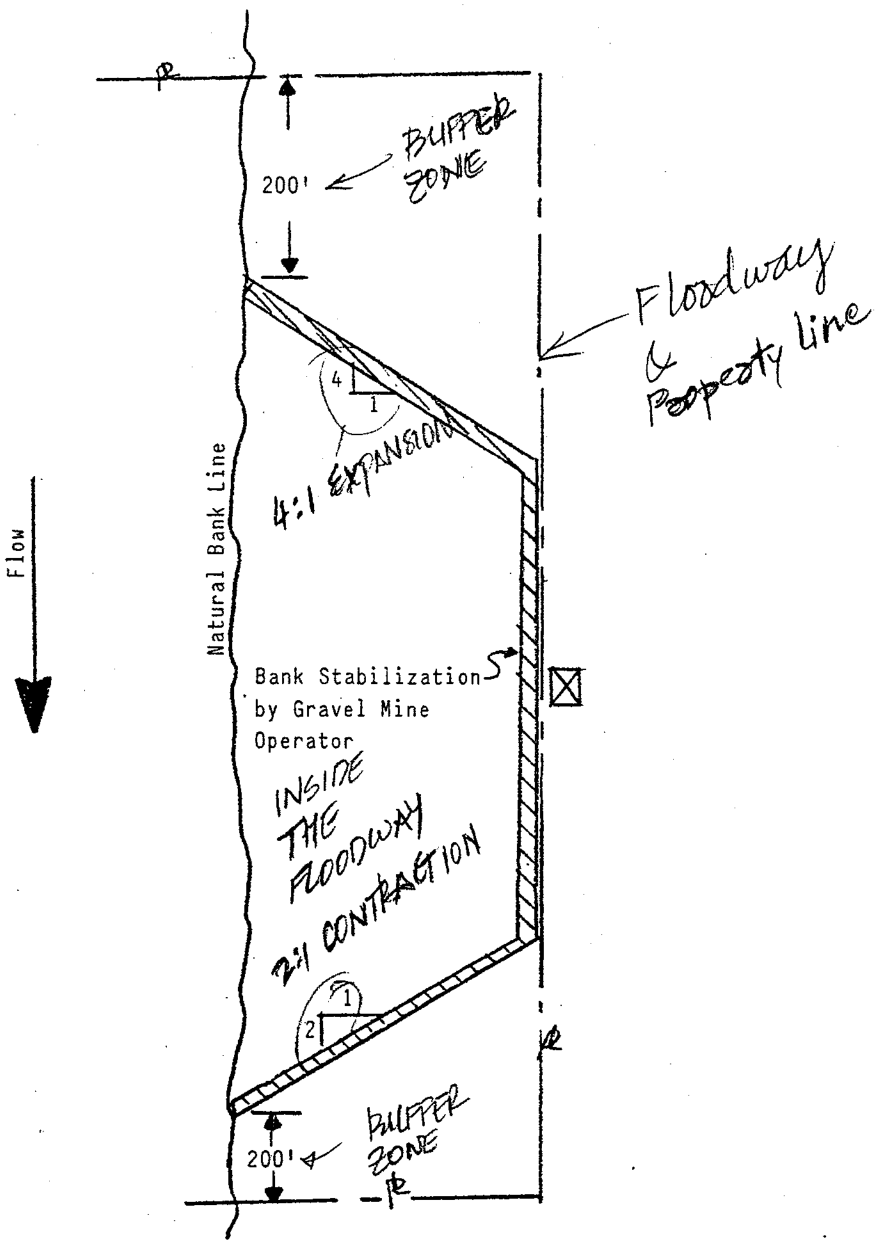


Figure 4. Suggested Expansion of Instream Gravel Mine Operation.



Elevation View



Plan View

Figure 5. Suggested Expansion of Instream Gravel Mine Operation.

6. Overbank extraction operations should be designed to prevent flood waters from causing migration of the gravel mine into adjoining property, either by head cutting or lateral migration. For the general case where the gravel operation controls the overbank area to the centerline of the stream; bank stabilization or buffer zones be required to protect the adjacent property owners. No lateral buffer zone and sloped plane should be required where the gravel mine banks are stabilized in a manner approved by the responsible regulatory agency and incorporate a minimum depth of toe protection of 10 feet below the thalweg. The upstream and downstream buffer zone should be a minimum distance of 200 feet from the gravel mine operator's property lines. The excavation operation should include a gradual expansion of the upstream incoming banks. The modified bank should expand at a ratio of 1 to 4. Similarly, the downstream end of the pit contraction ratio should be 1 on 2. Figure 6 illustrates the suggested gravel mine operational plan. It should be noted that the unprotected stream bank would be subject to erosion by the lateral migration of the gravel mine during flood flows so that the floodway would be ineffective when the stream bank is overtopped and eroded.

For a second general case where the gravel operation does not control the river bank and immediate overbank, but is still in the floodplain, an upstream and downstream submerged cutoff walls and both side bank stabilizations would be required to protect the adjacent property owner from headcutting and lateral migration, respectively. No lateral buffer zone and sloped plane should be required where the gravel mine banks are stabilized in a manner approved by the responsible regulatory agency and incorporated a minimum depth of toe protection of 10 feet below the thalweg. Where structural stabilization is not provided, the upstream, downstream and side buffer zones should be a minimum distance of 200 feet inside the gravel mine operator's property lines. Figure 7 illustrates the surrounded gravel mine in the floodplain.

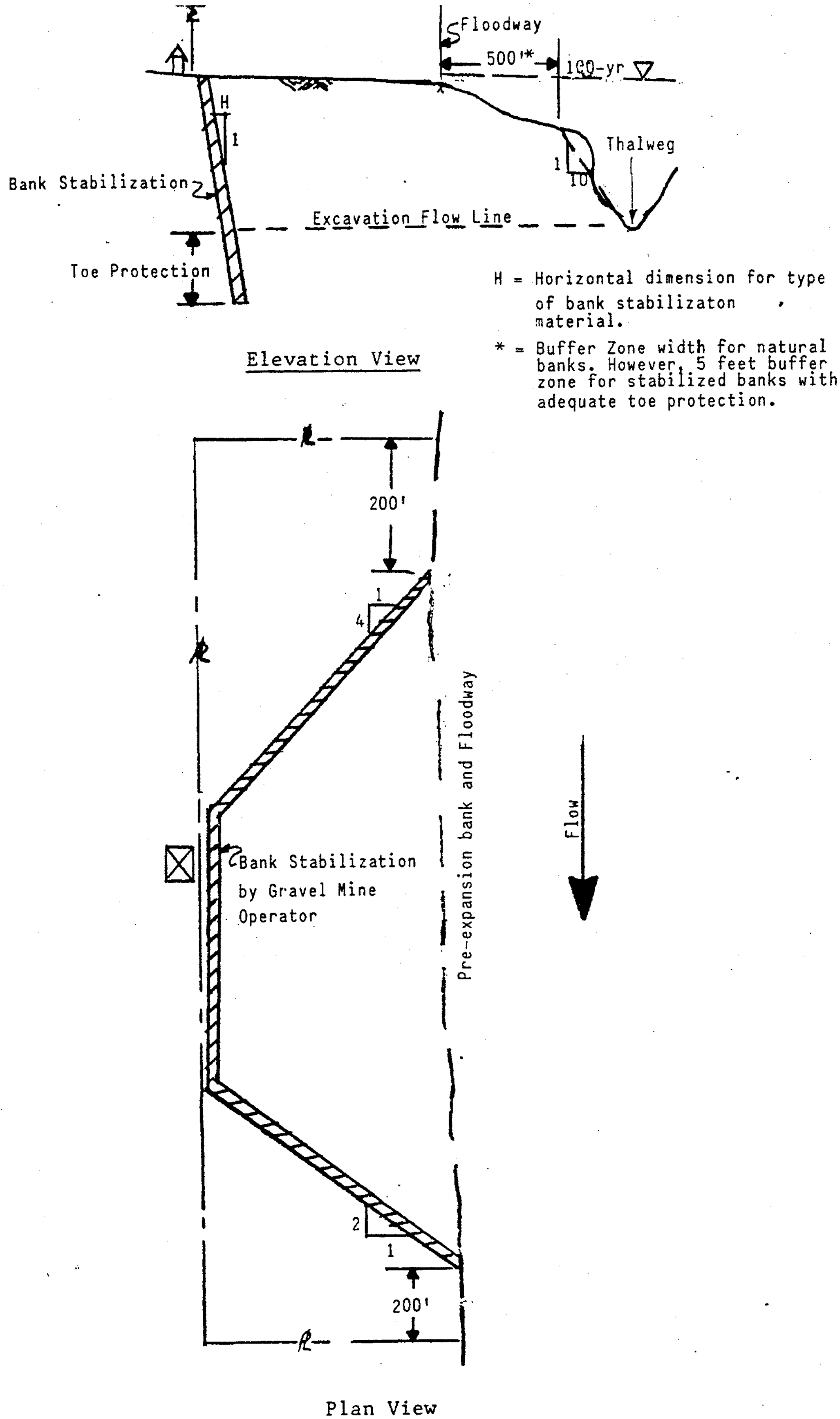
7. No excavation should be permitted below the established excavation flow line elevation. Those mines that were operational before the adoption of the suggested guidelines should be given special evaluation and considerations.

8. All extracting operations should be performed on the basis of a continuous pit within the property of any one operation. Leapfrog operations should not be permitted; and continuous pit excavation should not be sinuous with respect to either the alignment or grade of the stream.

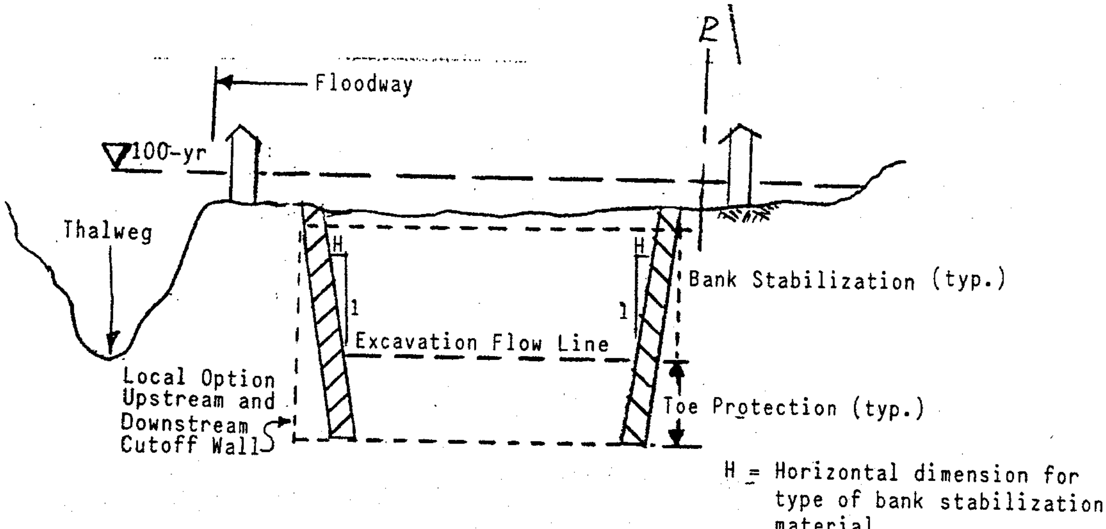
9. In cases where there are potential adverse hydraulic effects from an extraction operation, the owner should provide the regulatory agency with the necessary engineering analysis, performed by a qualified engineer, showing that there are no significant adverse effects, or if there are, that they can be mitigated.

SUGGESTED GRAVEL MINING OPERATION GUIDELINE OUTSIDE THE 100-YEAR FLOODPLAIN

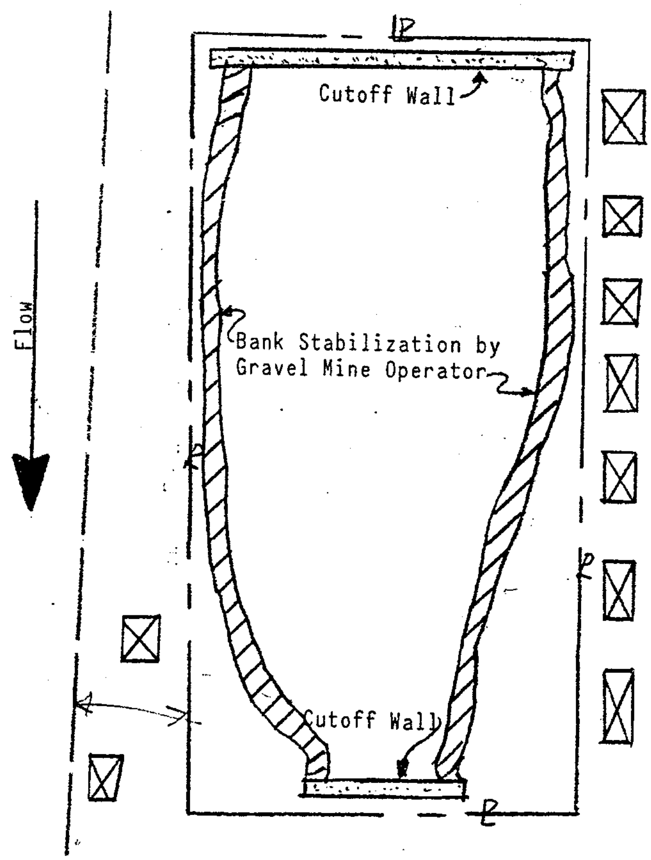
A minimum 200 foot wide buffer zone should be established outside the 100-year floodplain to prevent floodflows from causing gravel mine bank migration back into the channel. To prevent piping between the river thalweg and the gravel mine, the gravel pit depth should be limited by a 2-1/2 percent grade plane from the estalished flow line. Figures 8 represent typical illustration of this condition.



Figures 6. Suggested Overbank Gravel Mine Operation.



Elevation View



Plan View

Figure 7. Suggested Overbank Gravel Mine Operation.

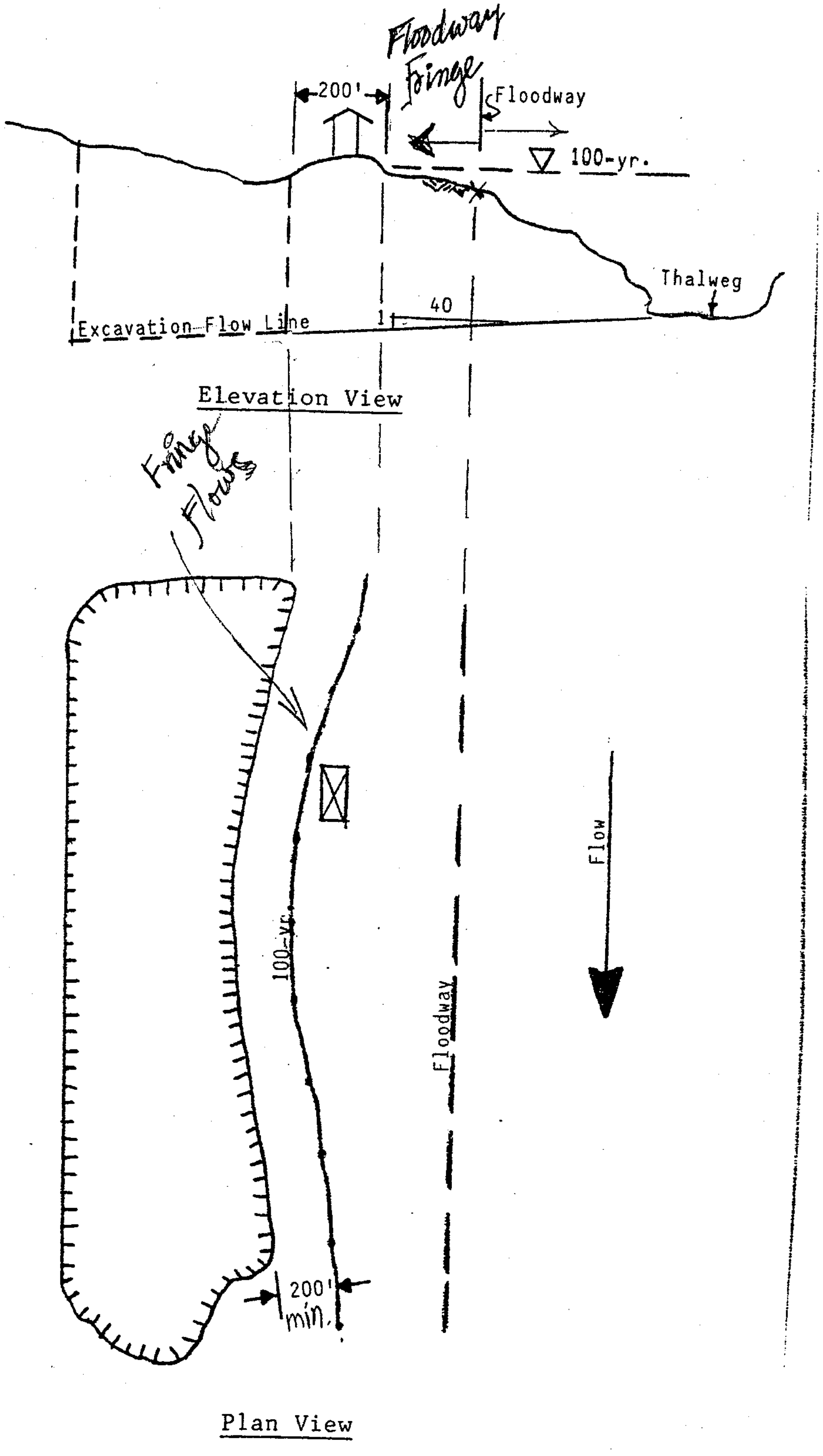


Figure 8. Suggested Gravel Mine Operation Outside the 100-Year Floodplain.

IV. RECLAMATION

1. Streambanks affected by a sand and gravel mining operation should be rehabilitated according to procedures acceptable to the regulatory agency.

2. Any piles of mining waste, and any equipment should be removed from the flood plain after excavation is completed. Certain materials may be used for the backfilling of the excavated pits provided that there is no adverse environmental effect. No toxic material or organic solid waste should be allowed in the backfill. Fill material or weathered waste should be graded and covered with coarse hard material, where practical, to prevent scouring.

3. The final side slopes of the pits should take into consideration slope stability and the effects of river hydraulics. In all cases, the side slopes should be flatter than the critical gradient (angle of repose) for the type of soil involved.

4. All streambanks that have been disturbed by mining operations should be stabilized to prevent erosion and sloughing.

5. Access to abandoned pits should be prevented by structures such as fences or berms constructed outside the floodway.

V. ADMINISTRATION

1. The regulatory agency should establish and maintain in-house measures and procedures to ensure organized record keeping, monitoring of gravel mining operations, and reclamation under its jurisdiction.

2. The regulatory agency should suspend permits for sand and gravel mining operations when significant adverse effects are likely to occur as a result of such operations.

3. The regulatory agency should assure that the objectives of the operation and reclamation plan will be accomplished. This may include provisions for liens, performance bonds, or other security to guarantee reclamation in accordance with the approved reclamation plan.

4. The regulatory agency should act with diligence in reviewing and ruling on applications for extraction permits, and on proposed reclamation plans for existing pits. The agency should integrate the requirements of these guidelines with other planning, and institute environmental review procedures required by law and administrative practice.

5. The use of HEC-6 and other computer mathematical models are encouraged to verify the effects of the operational plan submitted using the aggregate size distribution from the gravel mine location.

6. If the proposed sand and gravel mining operation deviates from these identified guidelines, then there should be a requirement to support the new operational plan with a detailed hydraulic analysis.

APPENDIX A

Abstract of Los Angeles District Sand and Gravel Mining Activities That Have Caused Structural Damage During Floodflows.

SAND AND GRAVEL MINING ACTIVITIES REVIEWED THAT HAVE CAUSED STRUCTURAL DAMAGE DURING FLOOD FLOWS

A.1 The reports on river and channel damages that were the direct result of aggregate extraction activities in the Los Angeles District were reviewed to ascertain common modes of failure and to identify preventative measures. The pertinent reports are summarized below.

A.2 Banning West Levee-Riverside Co., CA. There were two gravel mines that impacted upon the levee, (1) a 60 foot deep abandoned gravel mine located in mid-river about 400 feet downstream of the levee and (2) a second gravel pit of about the same magnitude located 400 feet directly downstream of the end of the west levee. The 1965 flood caused a 20 percent grade head cut to a depth of about 20-25 feet on the east side of the center gravel pit. The extent of the head cut progressed upstream to a point opposite the downstream end of the levee. But there was no levee damage since only the middle of the streambed was affected. The 1966 and the January 1969 floods caused the center gravel pit to continue to experience head cutting. However, it wasn't until the February '69 flood that the streambed experienced degradation of about 20 feet and which extended upstream for about 1700 feet. As a result, the center and west pits combined and caused damage to the downstream 600 feet of levee. The damage to the levee was initiated by toe exposure, caused by floodflows which undermined the grouted stone protection, causing it to collaspe under its own weight. The slope within the leveed reach increased from 4.8 percent to 5.1 percent. However, even though the gravel pits have been completely filled with sediment, the river bed has been stable, even with the increased grade.

A.3 Lytle and Cajon Creek Levees, San Bernardino Co., CA. The Lytle and Cajon Creek levees were built in 1956. As the result of uncontrolled instream sand and gravel mining operations, floodflows have caused serious degradation of the streambed; even small flows have caused problems. The 1965 flood caused gravel pit head cutting. This head cutting resulted in levee damages in terms of toe and revetment undercutting and dip crossing undermining. A flood in 1966 caused two dip crossings to wash out because of continued head cutting. However, the most severe damages to the levees and channels were caused during the floods of 1969. During this flooding period, gravel mine head cutting caused the streambed to degrade and migrate over to the levees and groins which in turn caused their structural failure. In summary, the gravel pits accelerated the meander qualities of the streams. During the flood flows, head cutting action was initiated which inturn scoured the streambed in the upstream direction and attacked nearby flood control structures. The net result was that the levees and groins failed through undermining and loss of toe protection.

A.4 Santa Clara Rivers, Ventura Co., CA. The riverbed from Highway 101 to the City of Santa Paula has been continuously degrading over the years; predominantly due to the unrestricted instream gravel mining operations. The river thalweg, in this 4.7 mile reach, has degraded by about 20 feet. About 10 feet of this degradation has occurred within an eight year period along the Corps of Engineers (COE) east side levee. This period started from when the levee was constructed in 1961 and extended through the 1969 flood. The levee and gravel mines have increased the grade and confined the floodflows within the streambanks. Thus, the discharge per unit width has increased while the sediment transport capacity in the gravel mining reach is high in comparison to the braided upstream supply reach. The resultant instability of the streambed damaged the levee by: (1) undercutting the toe; (2) caused bridge failures by exposing the pier footing; (3) caused the uncovering and rupture of pipe lines because of streambed degradation and (4) caused flow diversion works to be extended upstream because of the degradation of the natural thalweg. Sespe Creek is the major source of sediment; however, bed replenishment is relatively insignificant compared to the documented gravel mining extraction quantities. It has been estimated that replenishment of the subject reach will require more than 100 years assuming that no additional headwater detention basins are constructed. Unrestricted gravel mining has also affected the ground water recharge, riparian habitat, and the ocean beach sand supply.

Since 1979, major degradation of the Santa Clara riverbed has ceased because sand and gravel extraction regulations have been applied and enforced. Conditional and special use permits are issued by Ventura County only after individual review and approval of the EIR and extraction plan. Ventura County requires a phased removal of the aggregate in width lifts along the direction of streamflow in order to increase flow conveyance during the extraction operation. Also, the County developed an optimum "red line standard" (maximum depth of excavation) which is based on: (1) structural safety of hydraulic structures (bridge footings, levee toe depth and irrigation intake works); (2) sand and gravel replenishment rate; and (3) streambed impact.

Further, Ventura County uses a computer mathematical model (PITS) to update and to optimize the "red line standard" in order to control future degradation near critical structures while allowing gravel mining activities where more balanced sediment conditions can be achieved. The computer model indicates areas of streambed instability and indentifies conditions at bridges where pier scour protection is not adequate to permit future gravel mining. In addressing lateral gravel mining (overbank extension of the instream excavation), operations, Ventura County regulates with the intent to: (1) widen a low flow channel to increase channel capacity and decrease flood stage; (2) promote more uniform sediment flow along the entire reach and (3) to provide an adequate buffer zone to prevent head cutting when normal buffer zones are breached in major floods. In summary, Ventura County operates with guidelines that generally conform to those previously suggested. However, Ventura County requires the following exceptions: (1) 200-foot buffer zone streamward from toe of bank at levee; (2) 20:1 side slope for limit of excavation plane; and (3) "red line standard" for depth of excavation control.

A.5 Rillito River, Pima Co., AZ. The river reach from La Cholla Boulevard to the Southern Pacific Railroad (SPRR) bridge has mostly unstable banks with very limited bank stabilization. The dominant discharge is generated from about a 2-year frequency flood. Future streambed degradation has been estimated at 4 feet. However, 2 feet of degradation has been measured in the La Cholla Boulevard to La Canada Drive reach for the period of 1967 to 1979. Historic information indicates that from 1941 to 1964, floodflows of less than a 10-year frequency have laterally shifted the streambed over 1300 feet in the vicinity of the La Cholla Boulevard reach. During 1965, a 10-year freqency flood caused a 700-foot shift in the streambed at Swan Road and in 1978 a similar 10-year frequency flood shifted the streambed 800 feet just below La Cholla Boulevard. More recently, 1983 floodflows, generally, widened the streambed from 200 to 500 feet. This bank erosion translated into an approximate loss of 100 acres of land along the river banks.

Past gravel mining operations appear to be the most probable cause of lateral river bank instability for the La Cholla Boulevard to the SPRR reach. Currently, there are two instream and two overbank gravel pits; however, all gravel mining is presently prohibited. The predominant overbank floodflows cause lateral migration into overbank gravel pits and into the historical meander riverbed. Segmented low flow bank stabilization and shifting river bends have caused flow impingement and aggravated lateral scour in the coarse sand streambed alluvial cone. A 100-year stabilized channel bank with several drop structures is currently being considered for the reach. As noted above, Pima County has prohibited active instream gravel mining and has instituted a regulation requiring a 500 to 1000 foot wide setback buffer zone for new developments that have unstabilized banks.

A.6 Santa Cruz River, Pima Co., Az. Because of man's direct influence, the Santa Cruz River is undergoing the process of having its natural braided multiple channel confined into a single well defined channel. This process has caused increased floodflow velocities with a high sediment transport capacity. Noticeable streambed degradation has been traced back to 1890's when development began to encroach into the riverbed. Problems of bank erosion and bank sloughing began to occur because instream gravel mines captured sediment and thereby reduced downstream sediment supply which in turn caused streambed degradation. Along with increased development in the river basin, property damages have also increased because of the erosion and sediment related problems caused by unregulated gravel mine operations, particularly during the 1950's and 60's. As a direct result, local governing agencies began to develop regulations to control gravel excavation operations. Historically, the Santa Cruz riverbed has undergone significant lateral shifting. For example, during the 1983 floods, lateral headcutting into overbank gravel pits caused the Santa Cruz River to shift by as much as 2000 feet. In addition, in certain areas, dense phreatophyte growth along the banks due to sewage effluent has limited the channel capacity and natural bank erosion process. This in turn forced the floodflows to overtop its banks and shift the streambed to a historical meander channel and into a line of overbank gravel mines. Contributing to this lateral movement of the streambed are landfills that are composed of highly erodible materials. Finally, gravel mine head cutting has also been identified as the cause of several bridge instability problems and partial failures on the Santa Cruz River.

A.7 Salt and Gila Rivers, Maricopa Co., AZ. Gravel mine operators in these rivers have suffered from flood damage to their equipment. However, their operations have also has been accused of causing, or extending, damage to adjacent property and structures. In the 1980 floods a main pier footing of the 1,500-foot, Maricopa freeway (I-10) bridge over the Salt River was undercut as a result of riverbed shifting and scouring. Part of the problem was caused by sand and gravel operations excavating large areas in the riverbed, both upstream and downstream of the bridge. It appeared that both the downstream and upstream excavations caused the shifting of the main channel, creating scouring at the piers. The scour problem was aggravated by the headcutting of the downstream excavation.

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Erosion problems similar to those of the I-10 bridge were noted on the old Oak Street crossing on the Salt River Reservation. Presence of an abandoned gravel pit located about 200 feet from the road caused undercutting of the road foundations, and collapse of the paved roadway.

Another problem related to in-channel sand and gravel operations on the Salt and Gila Rivers was the obstruction of the floodway by stockpiles, levees, and dikes built to protect equipment and pits. These obstructions diverted runoff and changed the course of the streams; thereby endangering adjacent property. In addition, the constriction of flow increased velocities, which increased the erosive capacity and further damaged the streambed and banks.

Mining-related damges were also observed in earlier floods. However, local agencies indicated that flood-related complaints against sand gravel operators are increasing. Examples include damage to the south bank of the Salt River between 16th and 24th Streets and to the southeast corner of 19th Avenue. The extent to which sand and gravel mining is responsible for these damages has not been determined and quantified. However, the potential damages are severe enough that the present pattern of extraction is considered to be a flood-related problem. In May 1986 the Arizona Department of Transportation awarded an 18-month study contract to: (1) determine the extent of damages caused by gravel mining operations on all highway related structures throughout Arizona; and (2) to define preventative measures to protect structures during future floods.

APPENDIX B
SEDIMENT TRANSPORT ANALYSIS OF GRAVEL PITS

B.1.1 <u>General Sediment Transport Theory and its Application to Sand and Gravel Mining</u>. The amount of material transported, eroded, or deposited in a channel is a function of sediment supply and channel transport capacity. Sediment supply includes the quality and quantity of sediment brought to a given reach. Transport capacity involves the size of bed material, flow rate, and geometric and hydraulic properties of the channel. Both the supply rate and the transport capacity may limit the actual sediment transport rate in a given reach.

The total sediment load in a stream is the sum of bed material load and wash load. The bed material load is that part of the total sediment discharge which is composed of grain sizes found in the bed. The wash load is that part composed of particle sizes finer than those found in appreciable quantities in the bed. Wash load can increase bank stability, reduce seepage and increase bed material transport, and can be transported easily in large quantities by the stream, but is usually limited by availability from the watershed and banks. The bed material load is more difficult for the stream to move, and is limited in quantity by the transport capacity of the channel.

Sediment particles are transported by the flow in one or more of the following ways: (1) surface creep; (2) saltation; and (3) suspension. Surface creep is the rolling or sliding of particles along the bed. Saltation is the cycle of motion above the bed with resting periods on the bed. Suspension involves the sediment particle being supported by the water during its entire motion. Sediments transported by surface creep, sliding, rolling, and saltation are referred to as bed load, and those transported by suspension are called suspended load. The suspended load consists of sands, silts, and clays. The bed material load is the sum of bed load and suspended bed material load.

Under proper management, sand and gravel removal can increase the stability of a river system that is overloaded with sediment (supply greater than transport capacity). The overloaded condition can exist as a result of the natural characteristics of the watershed, or from abnormal events. These events could include land conversion changes in the watershed, construction, seismic activities, climatic conditions, and wildfire. The overload of sands and gravels can form large gravel bars and also provide material to form an armored layer of coarse particles on the streambed. Armoring encourages lateral migration due to the shifting of the thalweg in response to the development and movement of the bars and the relatively erodible bank material. With this condition, controlled removal of gravel bars by extraction and limited mining may actually enhance channel bank stability. Hence, careful river management is required to maintain equilibrium between excess production of sand and gravel, and extraction of sand and gravel.

~~Excessive sand and gravel removal~~ (removal greater than supply in any given reach) ~~can endanger the stability of the river system and bridges by inducing general degradation and headcutting.~~ For example, during recent floods several bridges over the Salt, Gila and Agua Fria were endangered by significant bed erosion and/or lateral migration of channels. Sand and gravel mining in the river system has been identified as one of the major causes of bridge instability and/or failure. Analysis of the effects of sand and gravel mining activities on the stability of a river system and bridges is important. Protection of the bridges may be required where the sand and gravel mining activities are of significant magnitude. (Bib. #2)

B.1.2 Physical Processes Governing Response Mechanisms Near Gravel Extraction

In an alluvial river the most significant riverbed changes are generally experienced during the peak flow of a major flood; however, previous studies indicate that in the vicinity of gravel extraction significant channel geometry changes are more often associated with the initial period of the flood. Additionally, significant changes near gravel extraction areas can occur during low-flow periods when other reaches of the river are relatively stable. The effect of gravel extraction in the riverbed can add energy to the system by increasing the water-surface slope, or energy slope, just upstream of the extraction. The steeper slope has greater erosive power and can initiate bank erosion and headcutting. These processes supply additional sediment to the river in quantities greater than it is capable of carrying locally, resulting in deposition. The upstream headcutting and deposition immediately downstream transforms the abrupt transition at the upstream face of the excavation to a more gradual, smooth transition. After this occurs, erosion will proceed at a much slower rate. In contrast, at high flows the river is generally already transporting near capacity and the influence of an increased water-surface slope near the excavation is relatively smaller due to backwater effects and channel control. Furthermore, during flood peak flows which have been preceded by low flows, the abrupt face may have already been completely transformed to a smooth transition. Therefore, low flows can cause significant erosion and may even have a higher erosion potential than high flows for local situations involving gravel extraction areas.

The significance of this unexpected situation, where low flows are potentially more destructive than high flows, depends on the size and volume of the excavation and the characteristics of the inflow hydrograph. For a small excavation the increased water-surface slope would not be nearly as significant as for a large excavation. The volume of the excavation controls how long it takes to fill with sediment, or to reach a new equilibrium.

While the "cut and fill" process is occurring near the upper face of the gravel excavation, the center reach of the gravel pit (which has lowest velocity and lowest transport rate within the gravel mining area) will experience deposition. The deposition potential in this area can be significant during low, medium, and high flow as long as the exit-channel area (downstream portion of the gravel pit in which the gradient is nearly zero) is long enough to establish a low-velocity backwater area.

The effects of a gravel operation are not limited to the upstream headcutting described above. Downstream erosion can also be significant. This is due mainly to the sediment trapping in the low-velocity backwater area at the center of the excavation. Lateral erosion can also occur along the sides of the excavation (especially at the upstream end), if lateral inflow is significant.

The above discussion applies to a gravel excavation located in a river reach with fairly uniform sediment transport characteristics throughout the reach. In an area of high sediment inflow, the headcutting may not be significant and the pit has a high potential for filling. On the other hand, if the excavation happened to be located in an area of significant degradation, the backfill rate will be extremely slow and the headcutting may extend far upstream. Similarly, downstream erosion potential also depends on transport rate in the downstream reach. If the downstream reach has a significantly low transport capacity, erosion in this reach may not occur.

The depth of scour occurring at bridge crossings as a result of a headcut, changes as the hydrograph passes through the river system. During the rising limb of the hydrograph scour occurs and potentially endangers the structural stability of the bridge by undermining the bridge footings. After the peak has passed (during the falling limb), the scour hole partially refills as sediments drop out. Therefore, the critical time for the structural stability of the bridge is during the storm, near the peak flow. Soundings made of scour holes after the storm do not indicate the potentially dangerous situation that might have existed during the storm. (Bib. #3)

B.1.3 <u>Problem Solving Techniques and Examples of Gravel Pit Analysis</u>. The degradation and aggradation problems associated with sand and gravel mining are very complicated. Simplifying assumptions are needed to obtain a practical and economical solution. The dominant physical processes include water runoff, sediment transport, sediment routing by size fractions, degradation, aggradation, and breaking and forming of the armor layer. These processes are unsteady and complicated in nature.

Recently, a number of computer models have been developed to analyze sediment and erosion problems associated with gravel mining operations occurring along rivers. A water and sediment routing method developed by Simons, Li and Associates (1979) has been applied to analyze headcutting problems associated with the Consolidated Rock (Conrock) gravel mining operation in San Juan Creek and Bell Canyon of Orange County, California. The model evaluated the erosional and depositional responses of the stream when subjected to different hydrologic inputs. In order to simplify the analysis, a known discharge water routing approach is used. The known discharge solution utilizes the data base developed for the HEC-2 flood level analysis. This method is feasible for gravel pit problems because of the short distances involved in the analysis. Three storms in January, February and March 1978 induced significant degradation and headcutting, and provided an excellent test for the model. The evaluation was made using time steps of 4 hours. The time lapse change of bed elevation at the original gravel pit boundary (Station 16+00) is given in figure B-1.

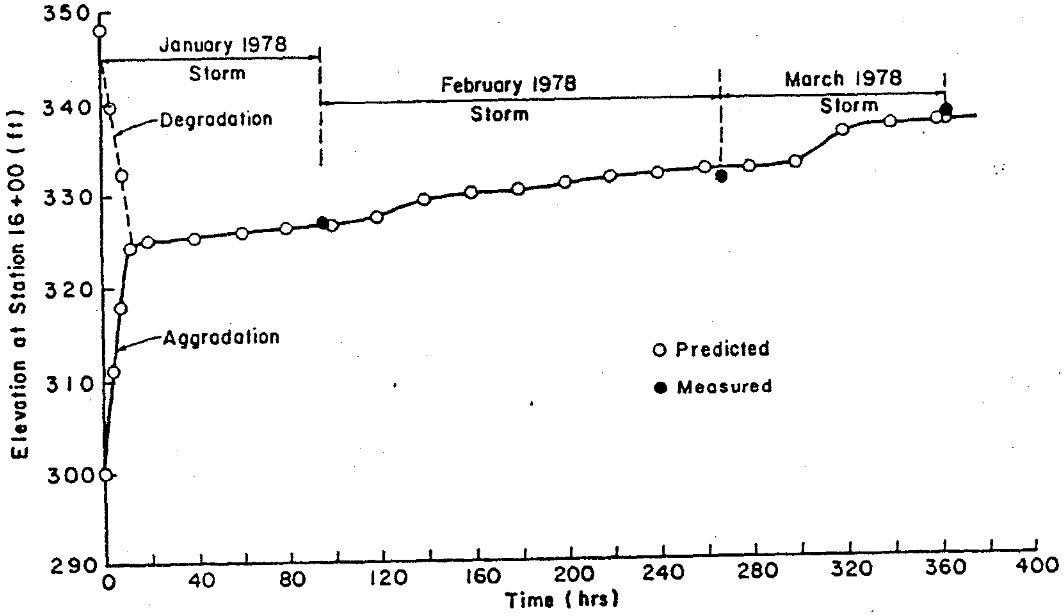

Figure B-1. Time lapse changes of elevation at the original gravel pit boundary (Station 16+00).

A second example involves sand and gravel mining activities just downstream of the Oracle Highway bridge over Rillito Creek in Tucson, Arizona. The reach length studied was approximately 2 miles (river mile 4.00 to 6.1). The bridge is located at river mile 5.05, and a gravel pit extends from river mile 4.65 to 5.03. The assumed dimensions of the pit for computer modeling were 10 feet deep by 400 feet wide by approximately 2000 feet long. Upstream of the bridge, the channel is 350 feet wide. Five cross sections were used within the pit during the analysis to define the geometric conditions.

The hydrograph used for testing was the 2-year flood event with a peak discharge of 7000 cfs. The 18-hour duration was divided into six time steps of 3 hours each. The changes occurring in the geometry of the upstream edge of the pit were defined at each of these time increments.

The initial condition was for a dry riverbed and an empty gravel pit. Prior to filling the pit with water or sediment, a normal depth approximation is used, rather than the HEC-2 analysis, to determine the hydraulic conditions and sediment transport rate. After the pit fills with water, the HEC-2 analysis is used to define the hydraulic conditions. The inflow occurring in the first time step (3 hours) initiates the headcut by eroding the corner off the upstream edge of the pit and depositing sediment in the bottom of the pit at the upstsream end (see fig. B-2). The slope of the headcut and deposited material is 0.050, however, a discontinuity of 2.40 feet exists. At time 5.20

hours the discontinuity between the headcut and deposition slope disappears, and a continuous slope of 0.050 exists. Table B-1 summarizes the changes occurring throughout the hydrograph. The pivot point actually shifts upstream 18 feet, although the resolution on the figure does not illustrate this. The calculated degradation (scour) occurring at the bridge as a result of the headcut is 4.66 feet at the end of the storm, which agrees with actual soundings that indicated approximately 5 feet of scour for this event.

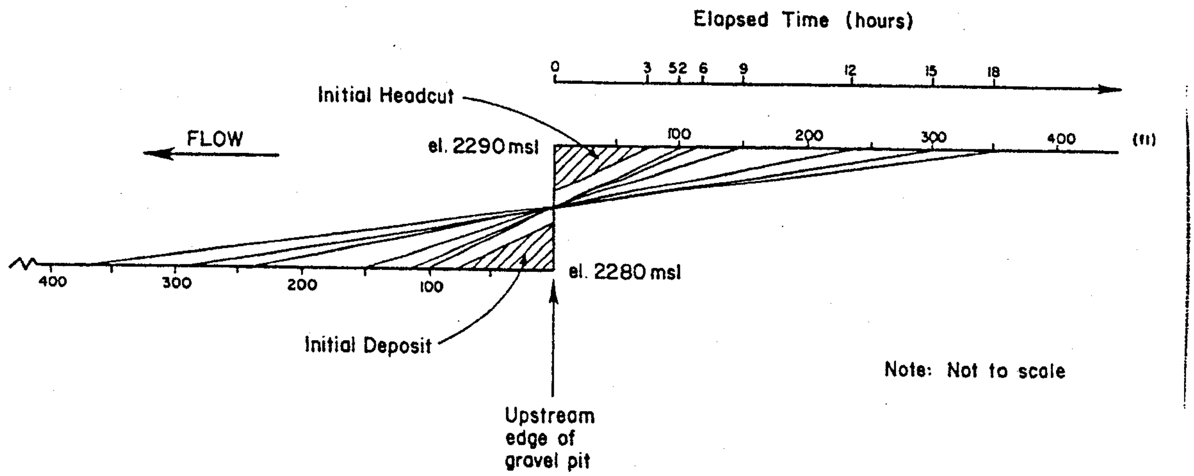

Figure B-2. Definition sketch of the temporal changes at the upstsream edge of a gravel pit.

RAJ SHAH

Table B-1. Calculated Headcut Distance and Slope

Time (hrs)	Headcut Distance (ft)	Headcut Slope
3	76	0.050
5.2	100	0.050
6	116	0.044
9	176	0.029
12	237	0.022
15	299	0.018
18	363	0.015

The U.S. Army Corps of Engineers developed the HEC-6 computer model to simulate scour and deposition in rivers and reservoirs. The model has been revised to simulate the effects of sand and gravel mining operations, and tested on the Kansas River in Missouri (U.S. Army Corps of Engineers, 1980). The results indicate that the model may be useful in future predictions of changes in bed load movement resulting from instream extraction.

Another computer program that may be used for simulation of sand and gravel mining operations is that developed by Chang for San Diego County (1976). The model has been applied a number of times to analyze erosion and sedimentation problems associated with sand and gravel mining operations as part of the requirements for a San Diego County use permit.

The models mentioned above, as well as other models, may be useful tools to evaluate river management practices or special problems, resulting from sand and gravel mining operations. Selection of an appropriate model should be based on the quantity and quality of available data, stream characteristics, and the special problems to be analyzed. Some of the models may be complex and expensive. If sufficient information is not available, the results could be misleading and the cost of using those models may not warranted. (Bib. #2)

Appendix C

Aggregate Extraction Guidelines Bibliography

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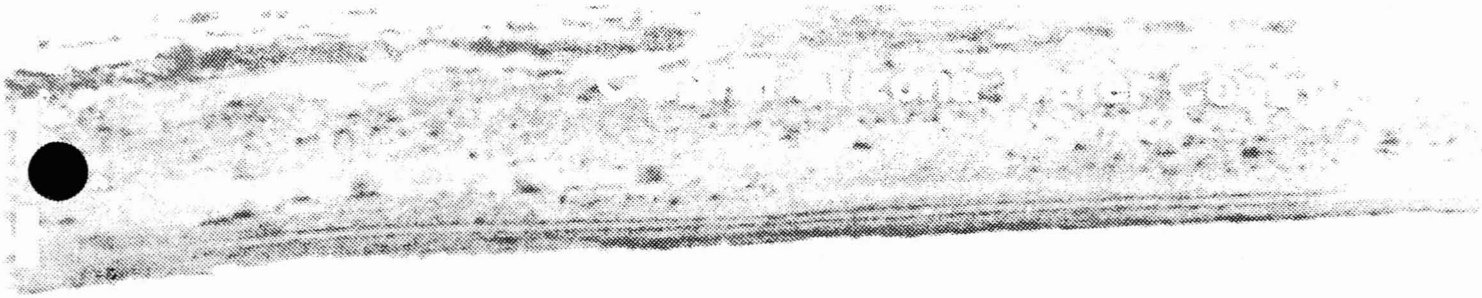

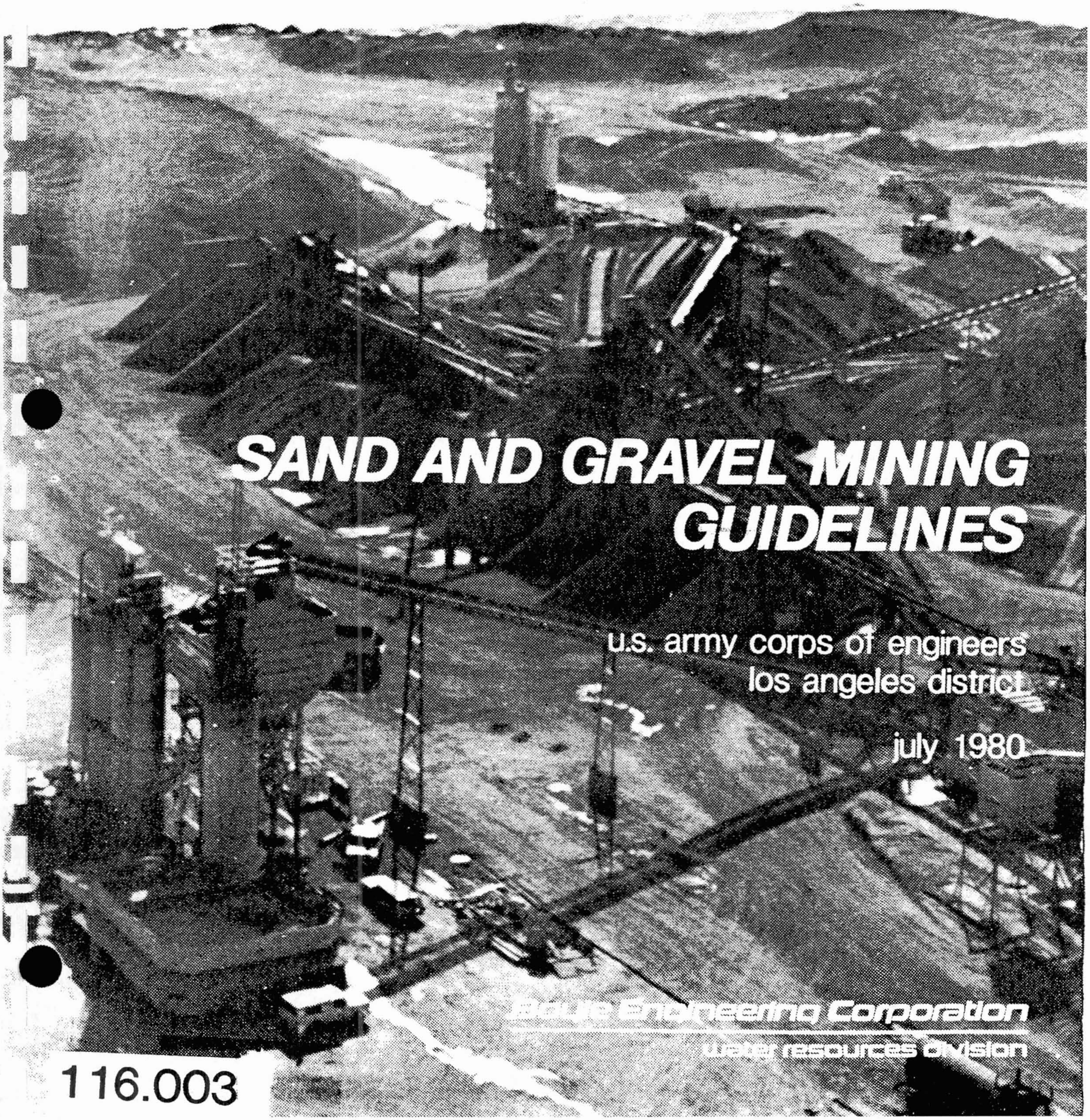

SAND AND GRAVEL MINING GUIDELINES

u.s. army corps of engineers
los angeles district

july 1980

Engineering Corporation
water resources division

Boyle Engineering Corporation

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consulting engineers architects

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Los Angeles District
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Chief, Engineering Division
300 N. Los Angeles Street
P.O. Box 2711
Los Angeles, CA 90053

August 8, 1980

Central Arizona Water Control Study
Sand and Gravel Mining Guidelines

We are pleased to submit the accompanying report concerning sand and gravel mining guidelines on the Salt and Gila Rivers, Maricopa County, Arizona. In accordance with Contract DACW09-80-C-0002, the report includes the following elements:

1. hydraulic effects of in-stream sand and gravel mining;
2. flooding problems associated with sand and gravel mining; and
3. mitigation measures, including sand and gravel mining guidelines, to reduce flood-related problems.

During the preparation of this report we had the pleasure of working closely with members of your staff, all of whom were very cooperative. We have appreciated the opportunity to study this important problem, and to prepare the report.

BOYLE ENGINEERING CORPORATION
Water Resources Division

Young Yoon

Young S. Yoon, Ph.D.
Senior Hydrologist

blc

Enclosure

X-C38-100-02

CENTRAL ARIZONA WATER CONTROL STUDY

SAND AND GRAVEL MINING GUIDELINES

PREPARED FOR

U.S. ARMY CORPS OF ENGINEERS

LOS ANGELES DISTRICT

BY

BOYLE ENGINEERING CORPORATION

WATER RESOURCES DIVISION

JULY 1980

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1.0 INTRODUCTION

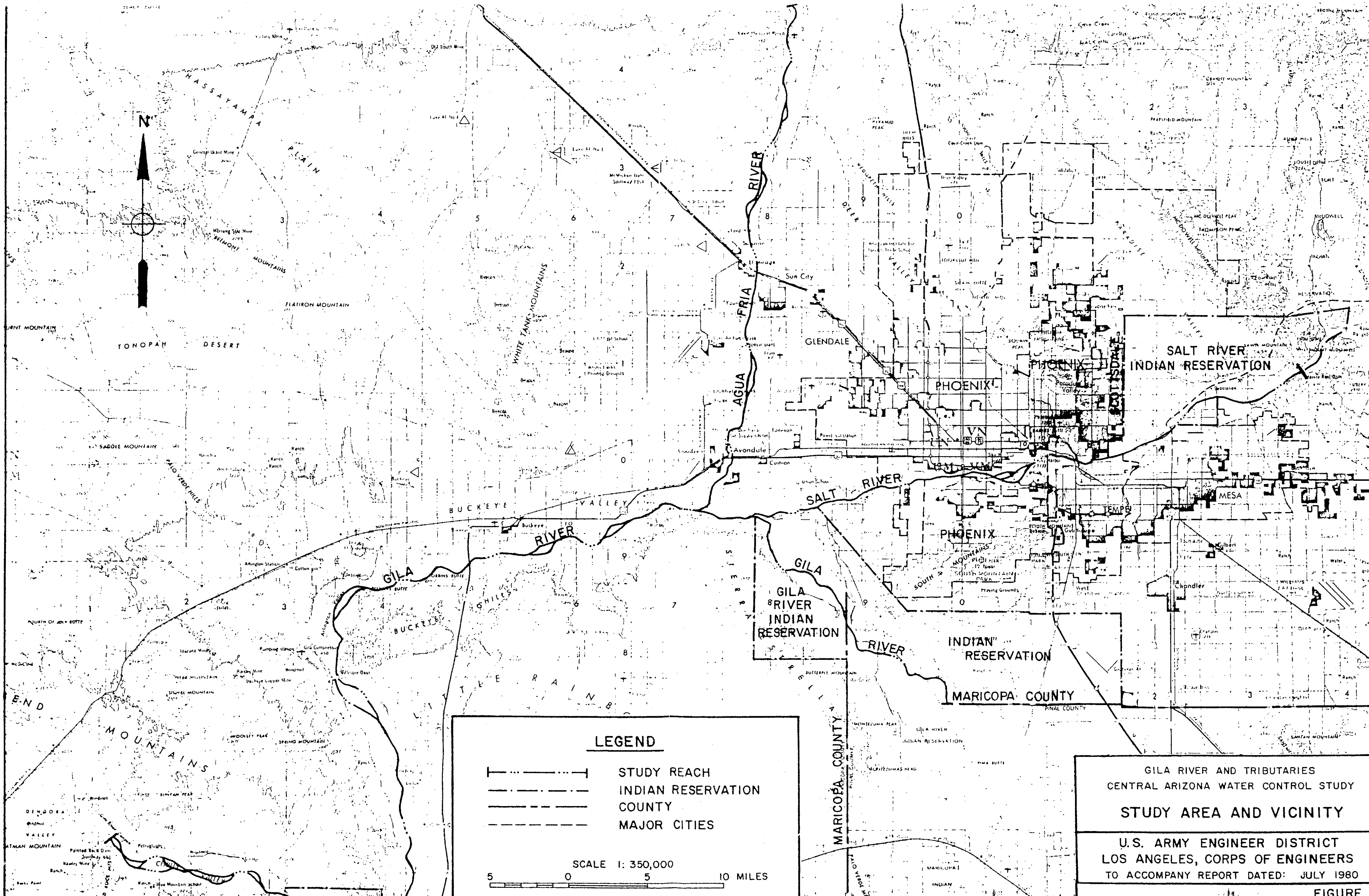
1.1 Background

The purpose of this study is to develop guidelines for sand and gravel extraction from the Salt and Gila Rivers that would reduce flood damages associated with sand and gravel mining. The study reach extends from Granite Reef Dam to Painted Rock Dam, Maricopa County, Arizona (Figure 1). The report is based on data obtained from field investigations, informal interviews, a literature review, and computer modeling. The report discusses impacts of sand and gravel mining activities, especially on the hydraulic processes of degradation/aggradation, headward erosion, and lateral migration. It outlines mitigation measures that may reduce the adverse hydraulic impacts of extraction activities, and proposes guidelines for management of future in-channel mining of sand and gravel. These guidelines are developed as part of the nonstructural measures under study by the Central Arizona Water Control Study.

1.2 Definition of the Problem

Private and public property has been damaged during recent, severe floods on the Salt and Gila Rivers. An undetermined portion of that damage resulted from in-channel sand and gravel mining. Mining activities change the pattern of flow in the flood plain. These changes can cause damages to structures adjacent to mining operations.

The problem of regulating sand and gravel mining is twofold. The current flood plain ordinances of Phoenix and Maricopa County could reduce some flood damages resulting from mining. However, most mining operations are not subject to the ordinances because a state law exempts from regulation all



LEGEND
STUDY REACH
INDIAN RESERVATION
COUNTY
MAJOR CITIES
SCALE 1: 350,000
5
0
5
10 MILES
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY
STUDY AREA AND VICINITY
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED: JULY 1980
FIGURE 1
N
HASSAYAMPA PLAIN
BELMONT MOUNTAINS
FLATIRON MOUNTAIN
TONOPAH DESERT
SADDLE MOUNTAIN
PALO VERDE HILLS
WHITE TANK MOUNTAINS
AGUA FRIA RIVER
Sun City
GLENDALE
PHOENIX
SCOTTSDALE
SALT RIVER INDIAN RESERVATION
Avondale
BUCKEYE VALLEY
Buckeye
GILA RIVER
SALT RIVER
BUCKEYE HILLS
MESA
TEMPE
Chandler
Gilbert
SOUTH MOUNTAINS
GILA RIVER INDIAN RESERVATION
INDIAN RESERVATION
MARICOPA COUNTY
PINAL COUNTY
SIERRA ESTRELLA
MOUNTAINS
GILA BEND

flood plain uses existing prior to enactment. Another hindrance to uniform enforcement of regulations is the presence of multiple jurisdictions. City, county and federal governments have authority in different areas of the Salt River Valley flood plains.

The guidelines in this report are based on engineering analyses of the hydraulic effects of sand and gravel mining. A comparative analysis was made of existing and proposed regulations in other areas. The purpose of the guidelines is to reduce flood damages sustained and caused by in-channel mining operations. The proposed provisions are more comprehensive and more specific than the existing flood plain regulations. The effect of the guidelines on the industry itself depends on the method of enactment, and the specific provisions and their enforcement. The governments involved should develop among themselves the most effective and equitable strategy for implementation.

1.3 Conclusions and Recommendations

-- Sand and gravel mining in the Salt and Gila Rivers has caused hydraulic changes in the channels. As a result of these changes, nearby lands and structures have been threatened or damaged during floods.

-- The most severe problem caused by in-channel extraction in the study area is headward erosion upstream of a pit. In addition, the presence of dikes and stockpiles in the floodplain may divert flood flows, causing erosion of the opposite bank and lateral migration of the channel.

-- Sand and gravel mining can be managed so that it enhances the capacity of the river system and reduces flooding, while accomplishing the objective of mineral extraction. However, this process requires the adoption of a river basin management plan.

-- The guidelines proposed in this report should be considered for adoption whether or not a basinwide plan is implemented.

-- The appropriate agencies should commit more resources to the enforcement of the existing regulations that apply to sand and gravel mining, and to the bringing of all existing operations under the regulations.

1.4 Acknowledgments

We acknowledge with appreciation the information and cooperation we received from the following groups during this study: Arizona Department of Transportation, Maricopa County, City of Phoenix, Salt River Pima-Maricopa Indian Community, Sonoma County Planning Department, Dames & Moore, Arthur Beard Engineering, Natelson Company, and CDM, Inc. Special thanks go to Dr. Daryl B. Simons and Dr. Ruh-Ming Li of Colorado State University, who provided material used throughout the report.

2.0 GUIDELINES

The guidelines developed in this section are based on the engineering analyses and references discussed in the following chapters. The guidelines are considered to be minimum acceptable practices that, if followed, will reduce flood and erosion damages associated with sand and gravel mining operations on the Salt and Gila Rivers. The guidelines do not consider the other potential environmental impacts of sand and gravel mining.

Any guidelines that are adopted should acknowledge the economic importance of sand and gravel mining, while protecting other values and activities in the flood plain. The guidelines could be implemented through a permit process applying to existing, as well as new, operations. Sand and gravel operations would be liable for damages resulting from failure to adhere to permit requirements.

<u>Operation</u>

-- All extraction should be conducted in accordance with plans that have received prior, official approval of regulatory agencies.

-- All excavation operations should be conducted in such a manner as to cause no obstruction of the natural flow in waterways, and cause no damage to adjacent structures or properties. No excavations, no stockpiling of any kind, and no other obstructions should be permitted in the floodway during the months of highest flood risk.

-- Excavations should be located so that the grade cannot exceed one percent between the midpoint elevation of the upstream pit face and the nearest point in the streambed 200 feet downstream of an existing structure or utility crossing (see Figure 2), unless it is shown that the excavations would have no effect on the upstream structure or utility crossing.

-- Excavations within a strip extending 100 feet streamward from the toe of river banks, or below a plane extending streamward at a 10 to 1 slope (horizontal to vertical), should not be permitted if there is a potential for such excavations to cause significant bank sloughing that would endanger structures or property within or adjacent to the flood plain.

-- No excavation should be permitted below the existing elevation of the flow line of the channel unless it is shown that the excavation would not cause significant damage to bank stability or to nearby structures.

-- All extraction operations should be performed on the basis of a continuous pit within the property of any one operation. Leapfrog operations should not be permitted; and a continuous pit should not be sinuous with respect to either the line or grade of the stream.

-- In cases where there are potential adverse hydraulic effects from an extraction operation, the owner should provide the regulatory agency with the necessary engineering analysis, performed by a qualified engineer, showing that there are no significant adverse effects, or if there are, that they can be mitigated.

3.0 SAND AND GRAVEL MINING ACTIVITIES

3.1 Resources

The quality of aggregate is measured in terms of size and shape distributions, flexibility, durability, chemical stability, and cleanness. Specifications in each category may vary with aggregate use. Such uses include concrete, asphaltic concrete, road base and subbase, trench backfill and pipeline bedding, riprap, and road surfaces (Sonoma County, 1980).

There are three major sources of aggregate: hardrock quarries, terrace (out-of-channel alluvium) excavations, and in-channel excavations. Processing and transportation are the two major costs associated with aggregate materials. Therefore, the source of supply that is exploited depends on minimizing total costs. A high quality source that is farther from the potential market may be competitive with a poorer source closer in. However, a distant, low-quality source will not be competitive with either.

Although no data are available for Maricopa County, it is generally accepted that in-channel operations on the Salt River are the most important sources of aggregate (Hollingsworth, 1970). There are four reasons for this preeminence. First, in-channel sources have no overburden and, therefore, are cheaper to extract. Second, the sand and gravel deposits in the Salt River are of excellent quality for all purposes. Third, the river sites are close to major urban markets and transportation routes. The fourth advantage results from the arid climate of the region. Because the Salt River flows intermittently, sand and gravel operators have access to the entire riverbed for most of the year. In addition, the lack of groundwater close to the surface increases the depth to which pits may be dug.

In 1970, Hollingsworth estimated the volume of available sand and gravel from Granite Reef Dam to 67th Avenue to be 368,000,000 cubic yards. Approximately half of that has been reserved by the Water and Power Resources Service for use in constructing the Central Arizona Project (Mariscal, 1973). Aggregate resources downstream of 67th Avenue are estimated to be approximately the same as in the upstream reach. Hollingsworth estimated the reserves of the Agua Fria River channel to be approximately 8,500,000 cubic yards. Other in-channel sources currently are not as important as the Salt and Agua Fria Rivers because of quality, quantity, or distance constraints.

Since Hollingsworth's study, there have been no estimates of the annual extraction and renewal rates. Both figures are necessary to estimate the existing in-channel reserves.

Empirical information suggests that not all of the in-channel resources are renewable. The sand portion of recently deposited aggregate is too fine to be used for ready-mix concrete (Bureau of Indian Affairs, undated). Further changes in size distribution are likely if additional upstream dams are built.

3.2 Historical and Present Patterns of Extraction

Sand and gravel extraction has followed the patterns of urbanization in the Salt River Valley. Extensive excavation activities near central Phoenix and near Tempe have expanded upstream and downstream with development. There are not as yet any major extraction operations downstream of the confluence of the Salt and Gila Rivers (see Plate 1). The location of gravel pits shown on Plates 1 through 3 was identified from USGS 1:24,000 topographic maps, and from photos taken before the 1978 floods (information provided by Natelson Company, Inc.). Also shown on the sheets is the 100-year flood boundary determined by the U.S. Army Corps of Engineers in 1979.

The streambeds of the Salt and Gila Rivers, and their tributaries, are in both public and private ownership. Jurisdictional authority in the study area is fragmented. Federal law and tribal regulations apply in the Indian reservations. The municipalities have control over the area within their corporate boundaries. Maricopa County has jurisdiction in all non-federal, unincorporated areas. State-owned land is not a significant factor in the study area.

Sand and gravel operations can be divided into three major categories: (1) those producing only sand and gravel; (2) those producing sand and gravel, and ready-mix concrete; and (3) those producing sand and gravel, and ready-mix and asphaltic concrete (Mariscal, 1973). The four largest companies in the greater Phoenix area fall into the third category, and have approximately 71 percent of the market (Mariscal, 1973). It was estimated in 1970 that each of the four companies had production capacities of at least 1,000 tons per hour; whereas the smaller companies had capacities of less than 200 tons per hour (Hollingsworth, 1970).

3.3 Flood Damages

Historically, sand and gravel operations have experienced the greatest industrial losses from flooding in Maricopa County, because they are situated in or near the riverbeds (U.S. Army Corps of Engineers, Feb. & Sept. 1979).

Damages to sand and gravel operations are mainly in the form of damaged conveyors, flooded materials, water-filled pits, and interrupted business. In the February-March 1978 floods, sand and gravel industrial losses were estimated to be $2.5 million (U.S. Army Corps of Engineers, Feb. 1979). This loss is about eight percent of the total flood damages. Floods in the following year caused damage to sand and gravel operations estimated at $5.2 million, about 10 percent of the total damages.

While the sand and gravel industry has incurred flood damages, it also has been accused of causing, or extending, damage to adjacent property and structures. In the most recent floods a main pier footing of the 1,500-foot, Maricopa freeway (I-10) bridge over the Salt River was undercut as a result of riverbed shifting and scouring. Part of the problem was caused by sand and gravel operations excavating large areas in the riverbed, both upstream and downstream of the bridge. It is alleged that both the downstream and upstream excavations caused the shifting of the main channel, creating scouring at the piers. It also is alleged that the problem was aggravated by the headcutting of the downstream excavation. (Bishop 1980)

Erosion problems similar to those of the I-10 bridge were noted on the old Oak Street crossing on the Salt River Reservation (Bureau of Indian Affairs, undated). Presence of an abandoned gravel pit located about 200 feet from the road caused undercutting of the road foundations, and collapse of the paved roadway.

Another problem related to in-channel sand and gravel operations has been the obstruction of the floodway by stockpiles, or by levees and dikes built to protect equipment and pits. These obstructions can divert runoff and change the course of streams, thereby endangering adjacent property. In addition, the constriction of flow increases velocity, which increases erosive capacity, endangering streambed and banks.

Mining-related damages were also observed in earlier floods (Aldridge, 1970). However, interviews with local agency personnel indicate that flood-related complaints against sand and gravel operators are increasing. Examples include damage to the south bank of the Salt River between 16th and 24th Streets and to the southeast corner of 19th Avenue. The extent to which sand and gravel mining is responsible for these damages has not been determined and quantified. However, the potential damages are severe enough that the present pattern of extraction is considered to be a flood-related problem (U.S. Dept. of Interior, Nov. 1979).

4.0 POTENTIAL IMPACTS OF SAND AND GRAVEL MINING

Any consumptive use of resources will cause impacts on natural and cultural systems. Some effects are direct and relatively easy to quantify, predict, and assess. Other impacts may be indirect or may affect dynamic processes that are either poorly understood or, in turn, are affected by random events outside human control. The following discussion of potential effects of in-stream sand and gravel mining indicates that the least understood and least predictable impacts may be the most severe. Consequently, the discussion focuses on hydraulic impacts of extraction. Although not within the scope of this study, other potential impacts are summarized in section 3.2 for general reference.

4.1 Hydraulics

A characteristic of flowing water is to seek and maintain an even gradient (thalweg) of flow. A stream in this condition is said to be in equilibrium. Natural or human-related forces can disturb this equilibrium by creating a knickpoint, or sudden change in gradient. The stream will respond with various processes which tend to return it to equilibrium--in effect making a stair-step into a ramp. A knickpoint may develop indirectly as a result of stream disturbances such as a sudden increase in flow at a particular point or in erosive capacity by removal of sediment.

The hydraulic processes that tend to recover and maintain equilibrium are sedimentation (aggradation), erosion (degradation), and lateral migration. Because equilibrium is a dynamic process, not a steady state, these hydraulic processes are present to some degree in every stream. However, their dominance and potential adverse effects increase as the amount of disturbance increases. Sand and gravel mining inevitably affects stream hydraulics by removing material from the streambed. The impact, however, need not be negative. In deepening a channel, the capacity is increased, thus reducing the amount of overbank flow during floods. On the other hand, with all other factors remaining the same, velocity is also increased, so that actual damage caused by flooding may not be reduced. The natural replenishment rate of extracted material will also affect the extent of impacts. The higher the replenishment rate, the faster equilibrium will be restored. In watersheds where dams and storage reservoirs exist upstream of mining operations, replenishment will be limited, and the channel may be subject to severe instability. In addition, the rate and amount of extraction will affect the duration of impacts.

Some impacts of sand and gravel mining are related to the creation of a knick-point. Examples are headcutting upstream of a pit, and aggradation at the pit and downstream. Headcutting is the erosive process by which a drop in gradient moves upstream. Field investigations revealed that headcutting of a pit is one of the most severe threats to hydraulic structures on the Salt River. Mechanisms causing headcutting, and typical examples, are illustrated in the appendix. In addition if a pit is extensive, it may cause water to slow and drop its sediment load, giving the stream increased erosive capacity below the pit. This process ceases when the pit fills with sediment.

To analyze the extent of headcutting associated with gravel pits, a computer model (Simons and Li, 1979) was applied to a reach of the Salt River. The model uses the Meyer-Peter, Muller bed load equation, coupled with an adaptation of Einstein's suspended sediment integration method. This integrated procedure determines the total bed material load by size fractions, based on the hydraulic parameters determined from HEC-2 analysis, and the measured bed material size distribution. The reach that was modeled extended 2.8 miles upstream of the I-10 bridge and 3.2 miles downstream of the bridge. This study reach was chosen because of readily available information. The gravel pit was assumed to be located downstream of the bridge with a surface area of 60 acres (1,200 feet wide and 2,200 feet long). The analysis used an 11-day, synthesized hydrograph with a peak flow of 176,000 cubic feet per second, approximately the 100-year flood (Figure 3). The maximum headcut distances and associated bed slopes for pits 15-feet, 30-feet and 50-feet deep are presented in Table 1. When the maximum headcut distance occurs, the depth at the upstream pit face is about half of the pit depth, and the bed slope is approximately one percent. This pattern has been used in the proposed guidelines as a standard for protecting upstream structures.

Additional hydraulic impacts are related to other aspects of mining operations, such as the creation of stockpiles, or the building of levees and dikes to protect equipment and active pits. These obstructions may deflect a stream during high flows and cause it to alter course. This can be a particularly serious problem if urban lands or high-value agricultural land is threatened by channel encroachment. Heavy economic losses will be sustained if channel migration undermines and destroys buildings, roads or bridges. If flood protection structures, such as levees, are breached by channel migration during floods, then loss of life may occur as well as property loss. However, lateral migration is a natural process among braided streams of the Southwest. The extent that sand and gravel mining may increase or accelerate this process is difficult to determine, especially because of the random element present in the natural migration process.

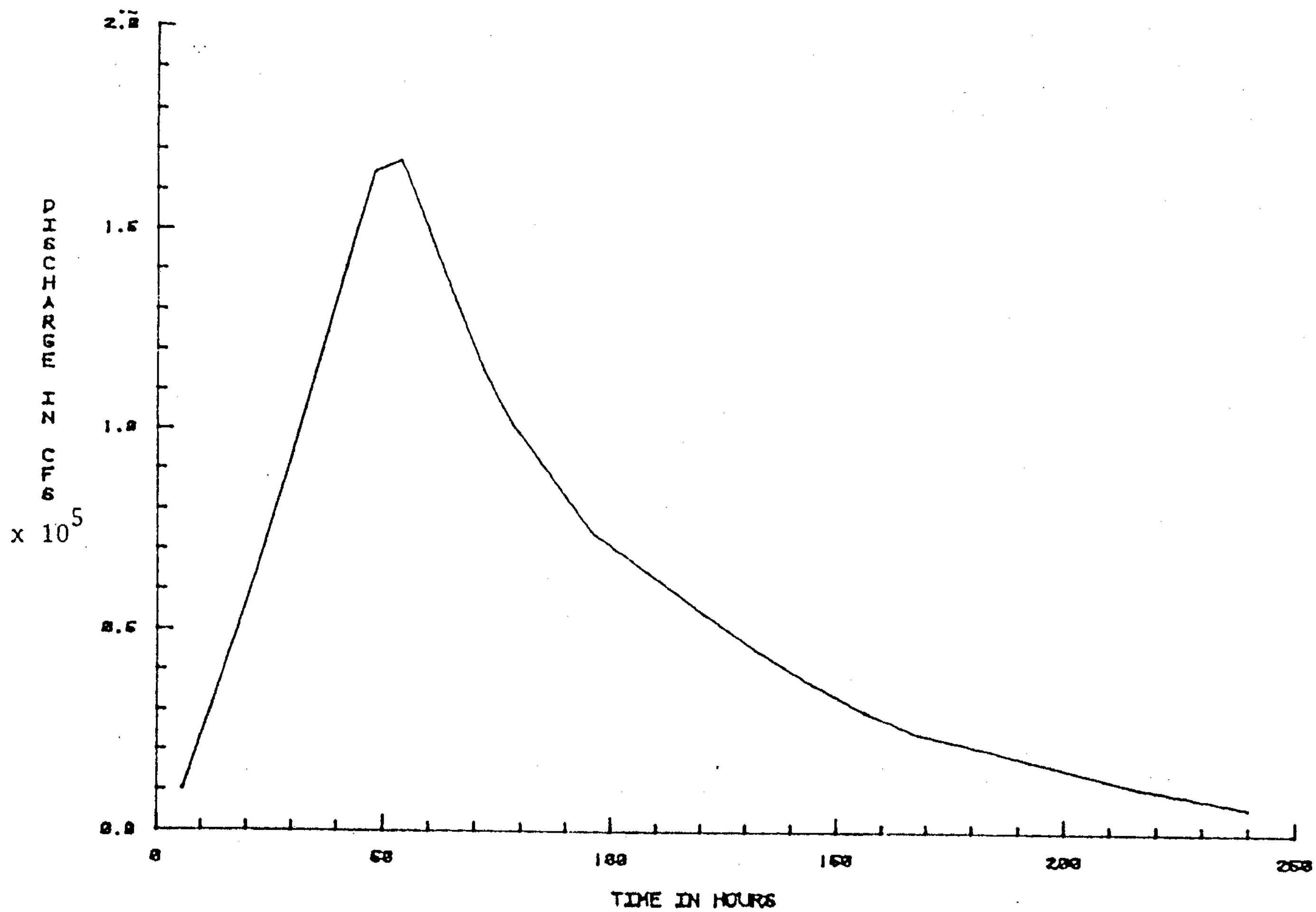


Figure 3 Synthetic Hydrograph, Salt River

Table 1. Computed Maximum Headcut Distance and the Associated Bed Slope for Various Assumed Pit Sizes in the Salt River.

Depth of Pit (ft)	Volume of Pit (Acre-ft)	Maximum Headcut Distance (ft)	Depth of Headcut at the Pit Boundary Associated With Maximum Headcut Distance (ft)	Bed Slope Associated With Maximum Headcut Distance (ft/ft)
15	900	940	9	0.0121
30	1,800	1,300	11	0.0105
50	3,000	2,500	20	0.0100

Finally, gravel pit location and alignment can affect channel location. For example, if a long, deep pit is excavated parallel to the natural channel, the stream may begin eroding a new channel through the pit, and eventually abandon its old channel (Bureau of Indian Affairs, undated). This may cause adverse impacts if the new channel is close to the bank or to a flood control structure such as a levee. In another case, if the pits are deep and "leapfrog," then increased velocity results, in turn causing increased erosion and channel instability downstream (Sonoma County, 1980).

Studies of other areas confirm that channel degradation has occurred as a result of sand and gravel operations (Envicom, 1979). Although not a negative impact in itself, degradation indicates instability which may cause problems in the stream and adjacent flood plain. The long-term effects of sand and gravel mining on the Salt and Gila Rivers cannot be determined from the inadequate data currently available. However, the channel has exhibited extreme local fluctuations in gradient. As an example, the profiles shown in Figure 4 are based on measurements taken before and after the 1980 floods. The profiles extend from 2.8 miles downstream of the I-10 bridge to 2.3 miles upstream of the bridge. Conclusions about the effect of mining operations on lateral migration must await additional data and studies.

4.2 Other Impacts

4.2.1 Groundwater. Groundwater recharge may be enhanced by gravel pits which retain flood waters and allow infiltration over a longer period of time (Dames & Moore, 1979). However, finer sediments often replace permeable river gravel after extraction, thereby reducing infiltration through the streambed. Groundwater quality may be degraded by the reduction of this permeable layer

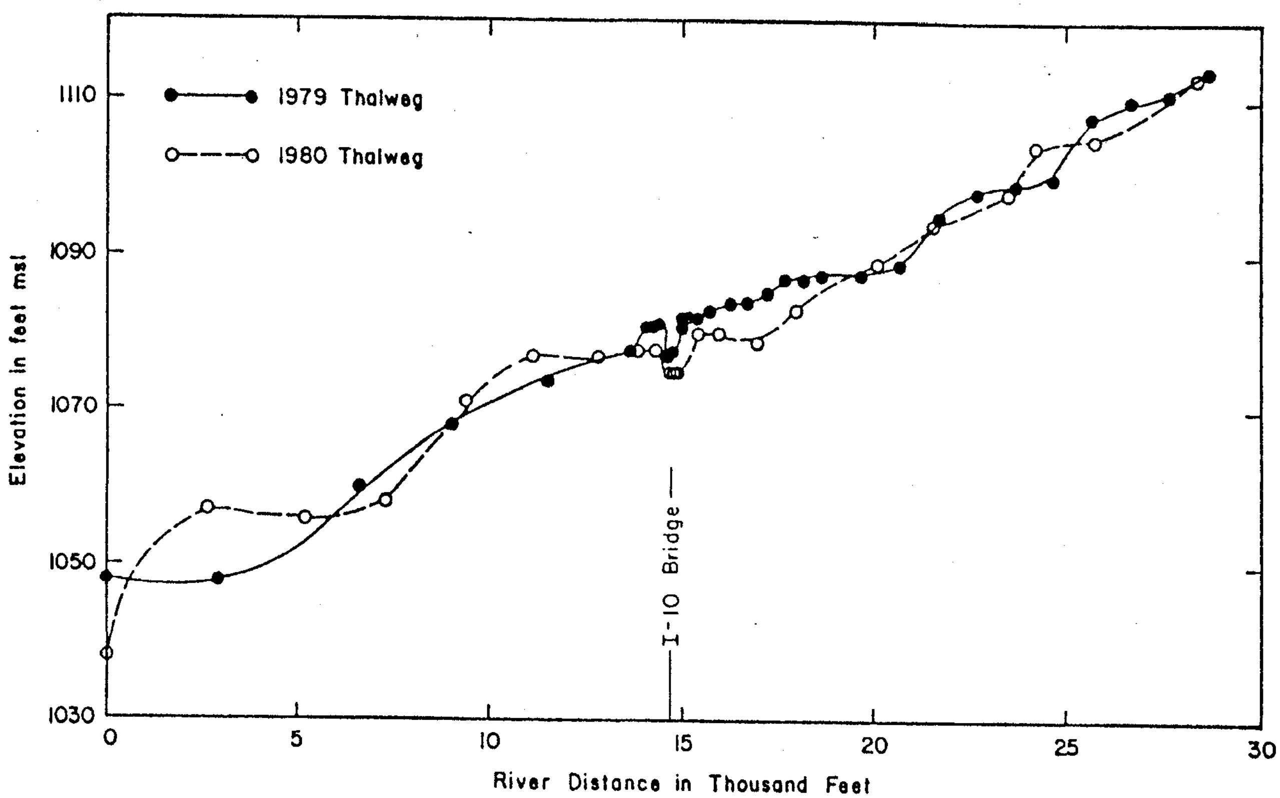


Figure 4 Salt River Thalweg Profile.

and its filtering qualities, or by exposure of the water table. This impact may be more important on smaller tributaries than on the Salt River where the groundwater table is at least 100 feet below the surface (U.S. Army Corps of Engineers, Sept. 1979, Draft Technical Appendix). Finally, a small negative effect on groundwater will occur if it is used for processing the aggregate.

4.2.2 Surface Water Quality. Turbidity and downstream sedimentation increase when mining operations disturb silts and sediments trapped among river gravels, or when water from washing operations is released into the stream. However, this is a relatively minor problem on intermittent streams such as those in the study area. A more important impact on water quality in the Salt River occurs when abandoned pits, filled with landfill, have been exposed, creating a public nuisance and health hazard.

4.2.3 Air Quality. Both stationary and mobile emissions occur during mining and processing operations. In an urban setting, the impacts of these emissions have been judged to be relatively minor (Dames & Moore, 1979). Dust can be a nuisance to adjacent property and will degrade habitat by coating vegetation with dust particles.

4.2.4 Acoustic. The major effects of noise associated with sand and gravel extraction and processing are lessening of aesthetics and recreational value of the area, and disruption of animal use of the riparian zone. The longer the diurnal period of operations, the more severe are these impacts.

4.2.5 Biologic. Removal of riparian vegetation reduces aquatic and terrestrial habitat by reducing cover and feeding areas for animals, and by reducing diversity of vegetation (Sonoma County, 1980).

4.2.6 Economic. The economic benefits of sand and gravel mining are employment and income generation, and lower costs of supplying aggregates over alternative sources. However, public costs are increased by the need for additional access routes and road maintenance required by heavy vehicles used to transport aggregate. In addition, mining extraction can cause irretrievable loss of the resource if extraction exceeds replenishment.

4.2.7 Recreation. Sand and gravel operations may temporarily interfere with some recreational uses of a stream and the riparian zone. However, it is possible to reclaim abandoned sites in such a way as to make them more attractive to recreation than before mining. Visual and aesthetic qualities are affected negatively during operations. Negative impacts may continue after termination if sites are not reclaimed.

4.2.8 Archaeological. In the Phoenix area, the development of access roads and processing locations may disturb archaeological sites.

4.2.9 Additional Impacts. Other impacts may occur as a result of sand and gravel mining, but are considered negligible in the Phoenix area. These impacts are alterations of topography, loss of topsoil, reduction of beach-sand formation, and reduction of aquatic habitat quality.

5.0 MITIGATION MEASURES AND MANAGEMENT PLAN

Mitigation measures are aimed at reducing either the temporal or spatial extent of adverse impacts. Such measures can be structural, non-structural, or a combination of both. Their cost may be borne by public agencies or private sources. The following discussion presents various mitigation measures related to flooding and erosion problems that may apply to the study area.

5.1 Structural Measures

5.1.1 Grade-Control Structures. A grade control structure can be an effective means of controlling general scour. Such structures can prevent headward erosion if the gravel pit initiating the headcut is shallow. The structure can be placed upstream of the gravel pit or downstream of the threatened structure (bridge, road, utility crossing).

Considering the use of control structures to limit headward erosion, two types are feasible: (1) a relatively economical structure formed of rock riprap reinforced with steel rods that will require minimum maintenance, or (2) a conventional reinforced concrete drop structure which can more effectively accommodate large differences in head, but is much more expensive to construct and maintain.

The rock riprap control structure should be constructed in a trapezoidal form with a downstream slope of approximately 1:4 with a stilling basin formed of adequate-size riprap extending approximately 15 feet downstream for a 2- to 3-foot differential in head. The top width of the structure would be approximately 10 feet and the upstream slope should be approximately 1:2. To improve the stability of the structure, reinforcing rods can be placed strategically in the rock riprap as construction proceeds. After the base layer of rock riprap is laid, steel rods can be laid horizontally and parallel to the direction of flow. These rods should extend through the rock riprap on the upstream and downstream faces of the structure. This procedure should be repeated at approximately each 4-foot change in elevation. Simultaneously with the placement of the first layer of rock riprap, vertical rods with large washers would be installed extending upward through the rock riprap. These reinforcing rods terminate in a steel bolt with a thread diameter of approximately 1-1/2 inches. Upon completion of the rock structure, longitudinal steel members would be welded to those rods extending through the structure parallel to the flow. Subsequently, as the structure settles, these longitudinal and horizontal rods are stressed by this settling action. Continuous steel elements would be drilled and placed over the steel rods extending through the top of the rock riprap and nuts would be tightened to stress these steel elements, compressing the rock and simultaneously increasing the tension in the horizontal steel members. The vertical rods extending through the top of the structure should be spaced at approximately 10-foot intervals along the axis of the control structure (see Figure 5). Constructing a rock riprap control using this methodology adds to the stability of the structure without adding significantly to its cost.

If a reinforced concrete retaining wall is built, a typical dimension for a single drop is as shown in Figure 6a. The riprap placed downstream should be designed to resist the forces exerted on the surface by the flowing water.

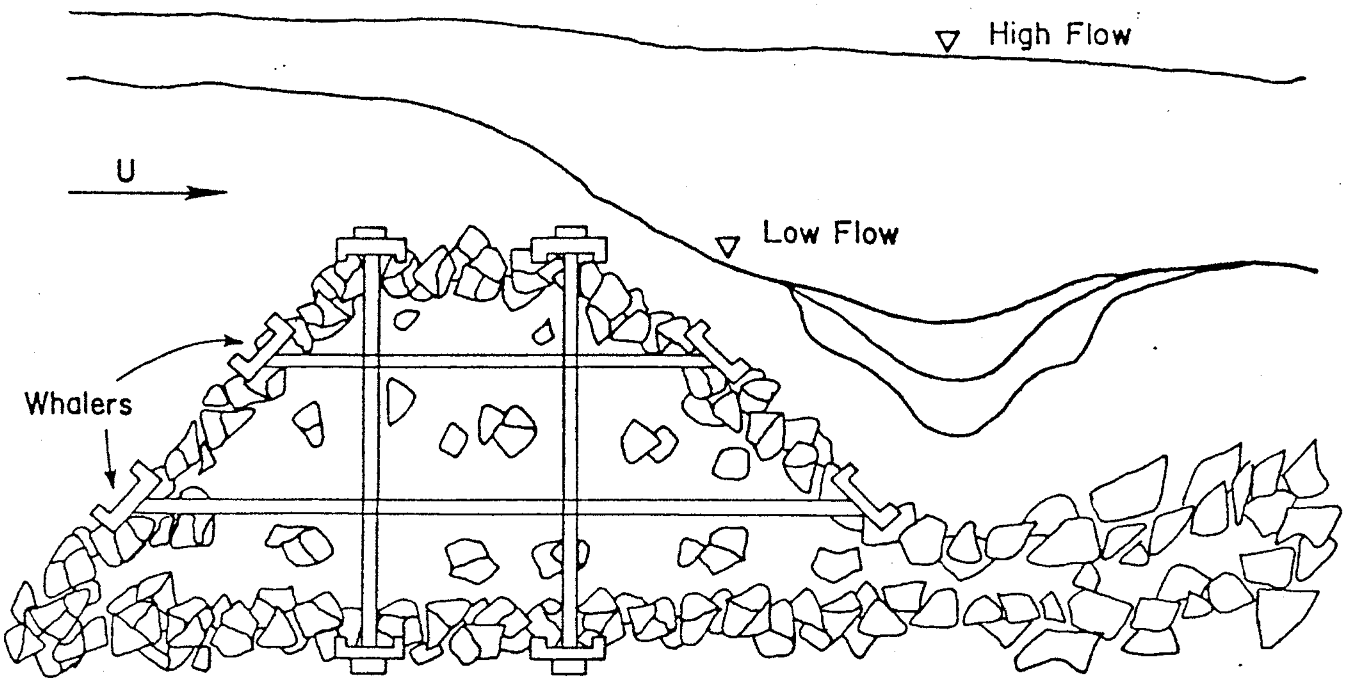
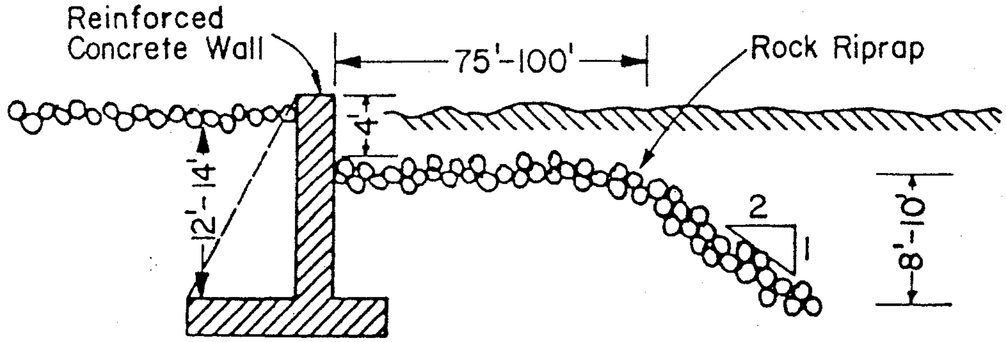
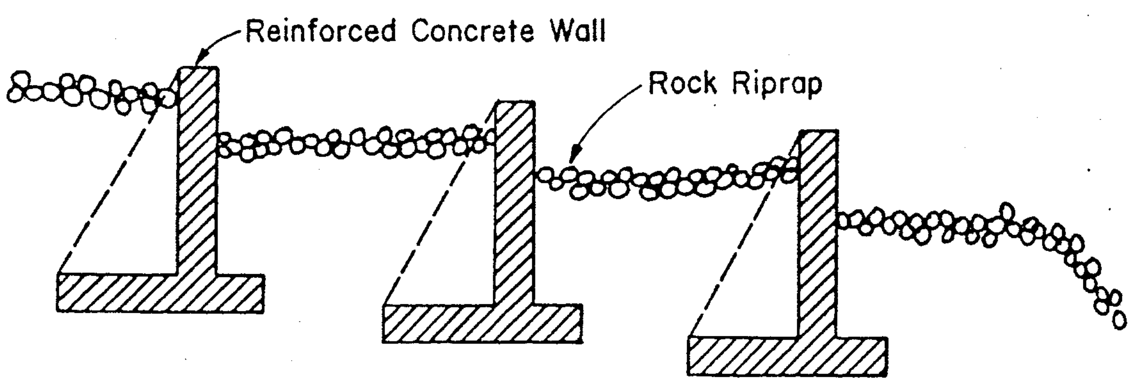


Figure 5 A Proposed Grade-Control Structure.



(a) Single Drop



(b) Multiple Drop

Figure 6 Reinforced Concrete Wall Drop Structure.

The riprap control structure can be effective if the potential drop across the structure is on the order of two to three feet. It is usually impractical to use a dumped riprap drop structure for a potential head drop equal to or greater than 4 feet. Another restriction is that a grade-control structure must extend across the full width of the channel to be effective. In many sections of the Salt and Gila Rivers, such a structure would be a half mile or more in length, greatly increasing its cost. Additional costs are incurred in maintaining the structure. Maintenance is required even after mining activities have ceased.

5.1.2 Flow Control Structure. The effects of dynamic scour during the storm, and the acceleration of lateral channel migration are major problems associated with sand and gravel mining. The shifting of the river thalweg is a major problem in the Salt River. Many existing bridges were designed without taking channel migration into consideration. An appropriate measure to mitigate the problems is implementation of a channelization scheme that controls the location and direction of the flow.

Guide banks have often been used to guide the flow of water through a bridge opening, and to control the position of scour and protect the abutments. Guide banks have been used effectively on both sand- and gravel-bed streams. Principal factors that must be included in the design of guide banks include controlled convergence of the flow normal to the opening, plan shape, upstream and downstream lengths, cross section, crest elevation, scour, and riprap protection. A common practice in the United States is to give the guide banks an elliptical form convergent to the opening; whereas in Pakistan and India the banks are straight and parallel to the opening, with a curved section at the upstream and downstream ends. The form of the short, elliptical guide bank was illustrated by Karaki (1959). The design layout for straight guide banks is given in Figure 7 (from Control Board, 1956).

Guide banks require specialized engineering experience in design and construction. Guide banks can be effective in protecting existing bridges, but their cost would prohibit use at every river crossing.

5.1.3 Additional Structural Measures. Riprap may be used in emergencies to prevent immediate collapse of bridges, levees or banks. The failure of the I-10 bridge across the Salt River was prevented during the February 1980 flood by placing 3,000 cubic yards of boulders in the channel (Bishop, 1980). However, such measures are temporary and do not solve the problems of erosion and lateral migration.

The effects of sand and gravel extraction may be lessened by reducing flood peaks. Existing dams on the Salt River were constructed for the purpose of water supply, not flood control. Construction of a flood control dam upstream will change design requirements of downstream structures, and may reduce lateral migration.

Structural measures are limited in their effectiveness by engineering and economic considerations. The wider the channel, the deeper and more extensive the excavation, the larger the flows, or the higher their velocities, then the less effective the structure. In addition, there must be a minimum working distance between the excavation and threatened structure in which to

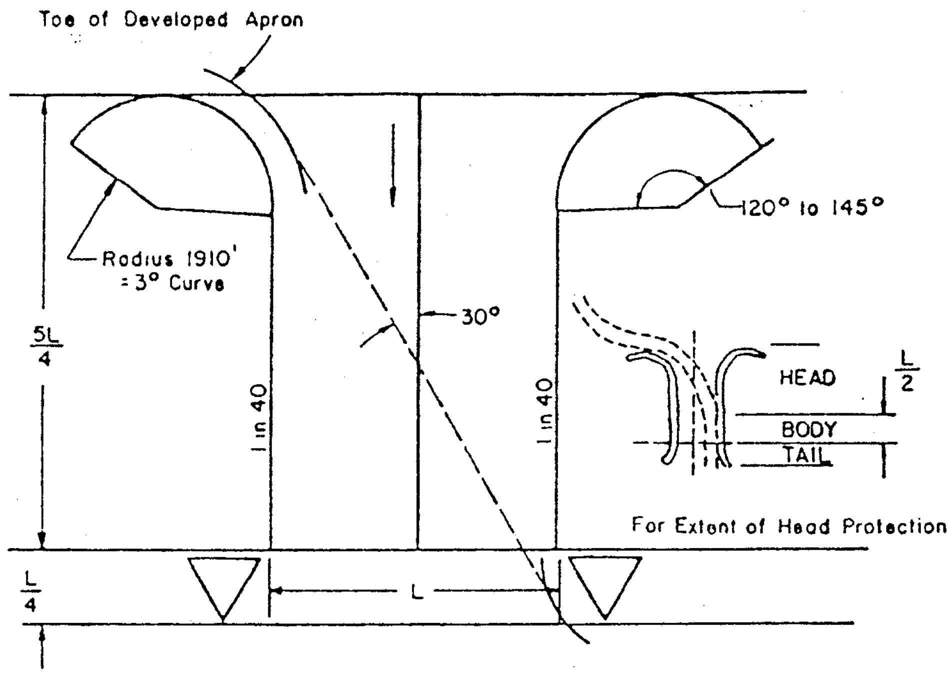


Figure 7 Straight Guide Bank Design to Protect Bridge From Misalignment of the Flow.

construct protection works. Clearly, the cost of and need for mitigating structures can be reduced by implementing non-structural mitigation measures.

5.2 Non-Structural Measures

5.2.1 Operation Standards. Operation standards would regulate the manner in which sand and gravel are mined. These standards could be enacted and enforced under the authority of zoning, flood control, or surface mining regulations. The standard would require that a permit be obtained before excavation begins. The permit process would include considerations of the effects of the particular excavation plan, and a review by public agencies and utilities that could be affected by these impacts. The degree of scrutiny applied to each permit request would depend on the extent of the proposed excavation and on its additive effect in relation to existing sand and gravel mining.

Possible operation standards include: restrictions as to rate and extent of extraction; setback and slope restrictions; limitations on pit location, phasing and configuration; separation of mining and processing phases; wetting requirements to reduce dust; and seasonal and diurnal shutdown requirements (Envicom, 1979).

The advantages of operation standards are that they reduce adverse impacts of in-stream sand and gravel mining while allowing continued use of the resource. They also can reduce the size and cost of necessary structural mitigation measures. However, for standards to be effective, they must be applied uniformly along an extensive reach of river. Operation standards that could be applied to sand and gravel mining on the Salt and Gila Rivers are suggested in Chapter 2 of this report. It is emphasized that these standards should be enacted so as to apply to existing sand and gravel operations, as well as to those established after enactment.

If enacted, the guidelines would have a number of effects. Slope and setback restrictions would reduce the area that otherwise could be excavated for sand and gravel mining purposes, thereby reducing the available resource. Restrictions on stockpiling, and requirements for reclamation may increase operational cost of sand and gravel mining.

As previously mentioned, the damages associated with sand and gravel operations accompany the natural processes of flooding and erosion which are random in nature. There are no quantitative data that indicate how much sand and gravel mining increases damage. Therefore, it is extremely difficult, if not impossible, to quantify the degree to which the proposed guidelines would reduce flood and erosion damage to public and private property. However, the guidelines are considered to be minimum acceptable practices that will reduce flood and erosion damages associated with sand and gravel mining operations.

5.2.2 Reclamation Standards. Reclamation standards could be enacted and enforced in a similar manner to operation standards. The reclamation standards would apply when an excavation was abandoned permanently. Such standards might require that all stockpiles, equipment, or waste heaps be removed or leveled; that the low flow channel be excavated to the depth of the pit, reducing the chance of channel migration; that side slopes be graded to at least five degrees less than the angle of repose for the remaining bed material; and that access to an abandoned pit be restricted by berms, fences, or other structures, reducing the chance of unauthorized dumping into the pit.

5.2.3 Right-of-Way Acquisition. When public right-of-way is acquired in the stream channel for a bridge or utility crossing, an additional area downstream could be obtained. Within that area, sand and gravel mining would be prohibited or severely restricted, thus reducing headward erosion problems. However, the right-of-way required to protect a structure, such as a bridge, is dependent on the location, size, and depth of the pit, and on the flow rates and sequences of flow. Right-of-way requirements for various assumed gravel pit depths and sizes can be estimated using a technique presented in Section 4.1. For example, the headcut distance for a pit 50 feet deep may be as much as 2,500 feet. If a safety factor of 500 feet is added, the right-of-way requirements would be 3,000 feet. The economic and legal problems of acquiring as much as 3,000 feet of riverbed could be extraordinary for agencies in the study area.

5.2.4 Extraction Fee. If prevention of headcutting or lateral migration is not feasible, then a jurisdiction may choose to impose an extraction fee. The fees would be set aside in a fund to finance repair or replacement of structures damaged by mining-related impacts. The advantage of this approach is that the cost of repairing public facilities is paid by those responsible. However, the cost will be passed on to consumers. In addition, the timing of destructive flows cannot be predicted. On one extreme, the fund could remain unused for many years. On the other hand, it could be depleted in less than one year.

5.2.5 Monitoring and Modeling Programs. Programs such as annual topographic mapping, aerial surveillance, and aggregate sampling would provide a better understanding of the effects of aggregate extraction. With increased understanding, better predictive models could be developed and applied to specific situations.

An accurate inventory of quantity, quality, and replenishment rate of sand and gravel resources in the study area is necessary. This base information, combined with improved information about extraction quantities and a refined model, would provide data about resource availability required by planners and private enterprise.

5.2.6 Moratorium of In-Stream Extraction. Some effects of gravel extraction, such as resource depletion and channel alterations, are long term and irreversible. Termination of in-stream extraction would be the only way to avoid these impacts. However, such a moratorium would undoubtedly increase costs of aggregates to consumers, and might create severe market disruptions. Therefore, it is assumed in the proposed guidelines that in-channel sand and gravel mining will continue in Maricopa County, and that its adverse effects can be reduced significantly.

5.2.7 Enforcement of Existing Flood Plain Regulations. The existing flood plain regulations for the unincorporated area of Maricopa County (1977) permit extraction of sand, gravel, and other materials from the flood plain on the condition that such activities

> do not require permanent structures, fill or other obstructions to the flow of flood water in the Floodway District, and provided that they do not adversely affect the capacity of the channels or floodways of any tributary to the main stream, drainage ditch, or any other drainage facility or system.

In addition, permitted activities are allowed only after the issuance of a flood plain use permit issued by the county flood plain administrator. A similar regulation in the Phoenix city ordinance prevents stockpiling in the floodway.

The county regulations divide the flood plain into two districts, following Federal Emergency Management Agency guidelines. Development is allowed at the edge of the flood plain until the cumulative effect of the structures raises the flood elevation by one foot. At that point no additional development is allowed. This area of limited development is called the Floodway Fringe District. The area in the center of the flood plain, where no development is allowed, is called the Floodway District.

Flood damages incurred and caused by sand and gravel operations would be reduced if the appropriate jurisdictions in the study area adopted and enforced regulations suggested by the federal guidelines. An obstacle to enforcing such regulations is the state requirement that they not apply to existing land uses or facilities installed or constructed prior to enactment. However, it may be possible to develop use permit requirements that would bring existing operations within the flood plain guidelines.

5.2.8 Existing Regulations in Other Areas. In 1939 Virginia became the first state to enact a surface mining law applicable to the sand and gravel industry (Newport & Moyer, 1974). In 1974, Newport and Moyer reported that 21 states had enacted similar legislation. Most state laws address the quality of water runoff from mining sites, and the reclamation of sites. Many states require periodic reports by the mining operator, and the posting of a performance bond. Some states impose criminal penalties for failure to comply.

At the local level, sand and gravel operators may be required to obtain use permits which are issued on a case-by-case basis. This is the policy in San Diego and San Bernardino Counties, California. Some jurisdictions, such as Sonoma County, California, are now considering ordinances that formally regulate many aspects of sand and gravel mining. The proposed Sonoma County ordinance includes requirements of in-stream operations relating to setbacks, extraction volumes, processing, seasonal operations, in-stream crossings, erosion, and sedimentation.

5.3 Management and Channelization Plans

The dynamic nature of river and watershed systems requires that local problems and their solutions be considered in terms of the entire system. Natural and man-induced changes in a river frequently initiate responses that can be propagated for long distances both upstream and downstream. Sand and gravel mining activities affect the sediment movement and supply in a channel system. Such operations can be beneficial or detrimental depending on watershed and river characteristics, and the mining and management practices followed. Therefore, a thorough understanding of the river system is necessary to evaluate the effects of mining activities.

In many rivers sand and gravel bars, formed by the natural processes of moving water and sediment, force the subsequent flows to meander around them. Consequently, rapid bank erosion can be expected opposite these large deposits. These features move slowly downstream, resulting in continued attack to opposite banks. With managed removal of excess sediments by sand and gravel mining, an adequate channel can be maintained, flooding can be reduced, bank erosion can be reduced, and an extremely valuable renewable resource can continue to be utilized.

The sand and gravel mining industry can assist local agencies in development of a flood control channel. It was estimated that construction of a flood control channel on the Salt River capable of carrying the 100-year flood (approximately 200,000 cfs) would require excavation of approximately 2.7 million tons per mile on the average. This estimate is based on a trapezoidal channel having a side slope of 3 to 1 with the depth of 15 feet, a bottom width of 1,000 feet, and an average channel slope of 0.0024. The apparent specific weight of the channel material is assumed to be 93 pounds per cubic foot. It is assumed that about 70 percent of the trapozoidal cross-sectional area would have to be excavated given the existing channel area.

A construction plan for such a channel that relies solely on sand and gravel operators may encounter problems. Annual demand for sand and gravel in Maricopa County during 1978 was estimated to be 10 million tons (Buehler & Best, 1979). Assuming that 20 river miles would be channelized through the urban area, the total volume excavated would be more than 5 year's demand. More importantly, not all excavated material would be of market quality. Fine materials, composed of silt or clay, are typically deposited on the gravel and sand bars in the middle of the wash or in the downstream river reaches. These materials may not be suitable for concrete aggregate.

Other problems that may discourage commercial excavation of a channel are access to public roads, proximity to aggregate markets, difficulty in shaping the channel, existence of impermeable layers, and shallow water tables causing pit flooding during excavation. Other considerations are land ownership of the reach to be channelized, continuity of the channel, administration of construction, and problems of maintenance.

Although problems may be encountered in developing a plan, nevertheless opportunities may exist for the sand and gravel mining industry to excavate a channel while fulfilling its aggregate extraction objectives. Currently, the Salt River Pima-Maricopa Indian Community is planning to have sand and gravel operators excavate a trapezoidal channel with a capacity of 150,000 cfs. If the outcome is successful, downstream jurisdictions may implement similar plans.

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APPENDIX

SEDIMENT TRANSPORT ANALYSIS OF GRAVEL PITS

A.1.1 General Sediment Transport Theory And Its Application To Sand And Gravel Mining. The amount of material transported, eroded, or deposited in a channel is a function of sediment supply and channel transport capacity. Sediment supply includes the quality and quantity of sediment brought to a given reach. Transport capacity involves the size of bed material, flow rate, and geometric and hydraulic properties of the channel. Both the supply rate and the transport capacity may limit the actual sediment transport rate in a given reach.

The total sediment load in a stream is the sum of bed material load and wash load. The bed material load is that part of the total sediment discharge which is composed of grain sizes found in the bed. The wash load is that part composed of particle sizes finer than those found in appreciable quantities in the bed (Simons and Senturk, 1977). Wash load can increase bank stability, reduce seepage and increase bed material transport, and can be transported easily in large quantities by the stream, but is usually limited by availability from the watershed and banks. The bed material load is more difficult for the stream to move, and is limited in quantity by the transport capacity of the channel.

Sediment particles are transported by the flow in one or more of the following ways: (1) surface creep, (2) saltation, and (3) suspension. Surface creep is the rolling or sliding of particles along the bed. Saltation is the cycle of motion above the bed with resting periods on the bed. Suspension involves the sediment particle being supported by the water during its entire motion. Sediments transported by surface creep, sliding, rolling and saltation are referred to as bed load, and those transported by suspension are called

A.1.2 Physical Processes Governing the Mechanisms of a Gravel Pit. The extent of damage to the system that can result from the potential headcut induced by the sand and gravel mining activities is a function of volume and depth of the gravel pit, location of the pit, bed material size, flood hydrographs, and sediment inflow rates and volume. During a low flow period the sediment supply to the river and to any given reach is generally less than the transporting capacity. Under these conditions the flow in that reach is capable of producing degradation by picking up additional sediment from the bed. The presence of a gravel pit can add energy to the system by increasing the water surface slope, or energy slope, just upstream of the pit. The steeper slope has greater erosive power and can initiate bank erosion and headcutting. These processes supply additional sediment to the river in quantities greater than it is capable of carrying locally. In contrast, at high flows the river is generally already transporting near capacity, and the influence of an increased water surface slope near the gravel pit is relatively smaller due to backwater effects and channel control. In addition, the velocity in the scoured portion may be reduced because of the increased depth when the pit is filled with water. Therefore, low flows can cause significant erosion and may even have a higher erosion potential than high flows for local situations involving gravel pits.

The significance of this unexpected situation, where low flows are potentially more destructive than high flows, depends on the size and volume of the gravel pit and the characteristics of the inflow hydrograph. For a small pit the increased water surface slope would not be nearly as significant as for a large pit. In a low flow event, the pit will not fill or reach equilibrium as soon as it will for a high flow event. During a high flow event the rising limb of the hydrograph fills the pit with water rapidly, and quickly drowns out the effect of a steeper energy slope. This concept is illustrated in Figure A-1 for representative low and high flow hydrographs. The cross-hatching indicates the relative times required to fill the pits to the level where channel hydraulics control the flow conditions.

The depth of scour occurring at bridge crossings as a result of a headcut changes as the hydrograph passes through the river system. During the rising limb of the hydrograph scour occurs and potentially endangers the structural stability of the bridge by undermining the bridge footings. After the peak has passed (during the falling limb), the scour hole partially refills as sediments drop out. Therefore, the critical time for the structural stability of the bridge is during the storm, near the peak flow (see Figure A-2). Soundings made of scour holes after the storm do not indicate the potentially dangerous situation that might have existed during the storm.

A.1.3 Problem Solving Techniques and Examples of Gravel Pit Analysis. The degradation and aggradation problems associated with sand and gravel mining are very complicated. Simplifying assumptions are needed to obtain a practical and economical solution. The dominant physical processes include water runoff, sediment transport, sediment routing by size fractions, degradation, aggradation, and breaking and forming of the armor layer. These processes are unsteady and complicated in nature.

Recently, a number of computer models have been developed to analyze sediment and erosion problems associated with gravel mining operations occurring along rivers. A water and sediment routing method developed by Simons and Li (1979) has been applied to analyze headcutting problems associated with the Consolidated Rock (Conrock) gravel mining operation in San Juan Creek and Bell Canyon of Orange County, California. The model evaluated the erosional and depositional

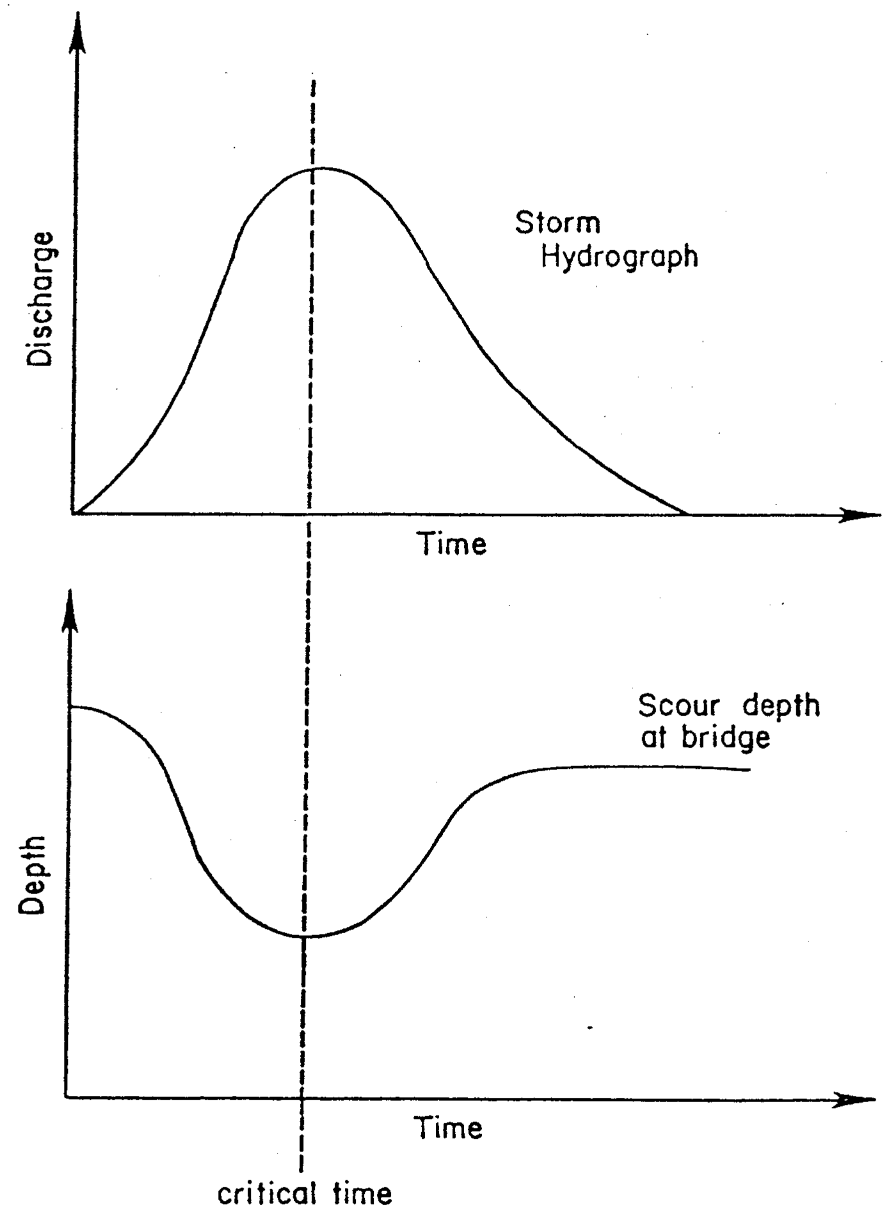


Figure A-2. Temporal change of scour hole depth at a bridge during a storm.

responses of the stream when subjected to different hydrologic inputs. In order to simplify the analysis, a known discharge water routing approach is used. The known discharge solution utilizes the data base developed for the HEC-2 flood level analysis. This method is feasible for gravel pit problems because of the short distances involved in the analysis. Three storms in January, February and March 1978 induced significant degradation and headcutting, and provided an excellent test for the model. The evaluation was made using time steps of four hours. The time lapse change of bed elevation at the original gravel pit boundary (Station 16+00) is given in Figure A-3.

A second example involves sand and gravel mining activities just downstream of the Oracle Highway bridge over Rillito Creek in Tucson, Arizona. The reach length studied was approximately 2 miles (river mile 4.00 to 6.1). The bridge is located at river mile 5.05, and a gravel pit extends from river mile 4.65 to 5.03. The assumed dimensions of the pit for computer modeling were 10 feet deep by 400 feet wide by approximately 2000 feet long. Upstream of the bridge, the channel is 350 feet wide. Five cross sections were used within the pit during the analysis to define the geometric conditions.

The hydrograph used for testing was the 2-year flood event with a peak discharge of 7000 cfs. The 18-hour duration was divided into six time steps of three hours each. The changes occurring in the geometry of the upstream edge of the pit were defined at each of these time increments.

The initial condition was for a dry riverbed and an empty gravel pit. Prior to filling the pit with water or sediment, a normal depth approximation is used, rather than the HEC-2 analysis, to determine the hydraulic conditions and sediment transport rate. After the pit fills with water, the HEC-2

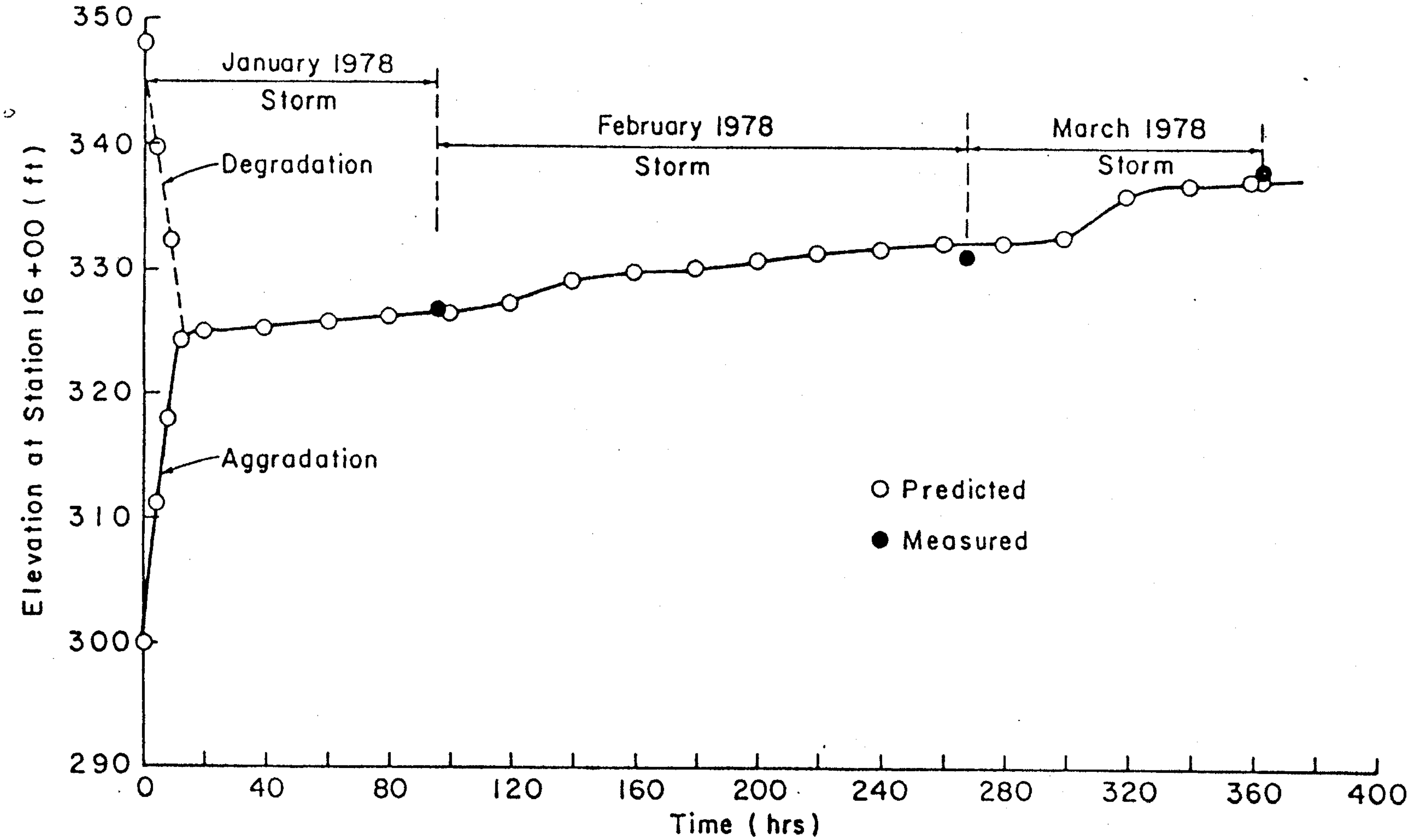


Figure A-3. Time lapse changes of elevation at the original gravel pit boundary (Station 16+00).

analysis is used to define the hydraulic conditions. The inflow occurring in the first time step (3 hours) initiates the headcut by eroding the corner off the upstream edge of the pit and depositing sediment in the bottom of the pit at the upstream end (see Figure A-4). The slope of the headcut and deposited material is 0.050; however, a discontinuity of 2.40 feet exists. At time 5.20 hours the discontinuity between the headcut and deposition slope disappears, and a continuous slope of 0.050 exists. Table A-1 summarizes the changes occurring throughout the hydrograph. The pivot point actually shifts upstream 18 feet, although the resolution on the figure does not illustrate this. The calculated degradation (scour) occurring at the bridge as a result of the headcut is 4.66 feet at the end of the storm, which agrees with actual soundings that indicated approximately 5 feet of scour for this event.

The U.S. Army Corps of Engineers developed the HEC-6 computer model to simulate scour and deposition in rivers and reservoirs. The model has been revised to simulate the effects of sand and gravel mining operations, and tested on the Kansas River in Missouri (U.S. Army Corps of Engineers, 1980). The results indicate that the model may be useful in future predictions of changes in bed load movement resulting from instream extraction.

Another computer program that may be used for simulation of sand and gravel mining operations is that developed by Chang for San Diego County (1976). The model has been applied a number of times to analyze erosion and sedimentation problems associated with sand and gravel mining operations as part of the requirements for a county use permit.

The models mentioned above, as well as other models, may be useful tools to evaluate river management practices or special problems, resulting from sand

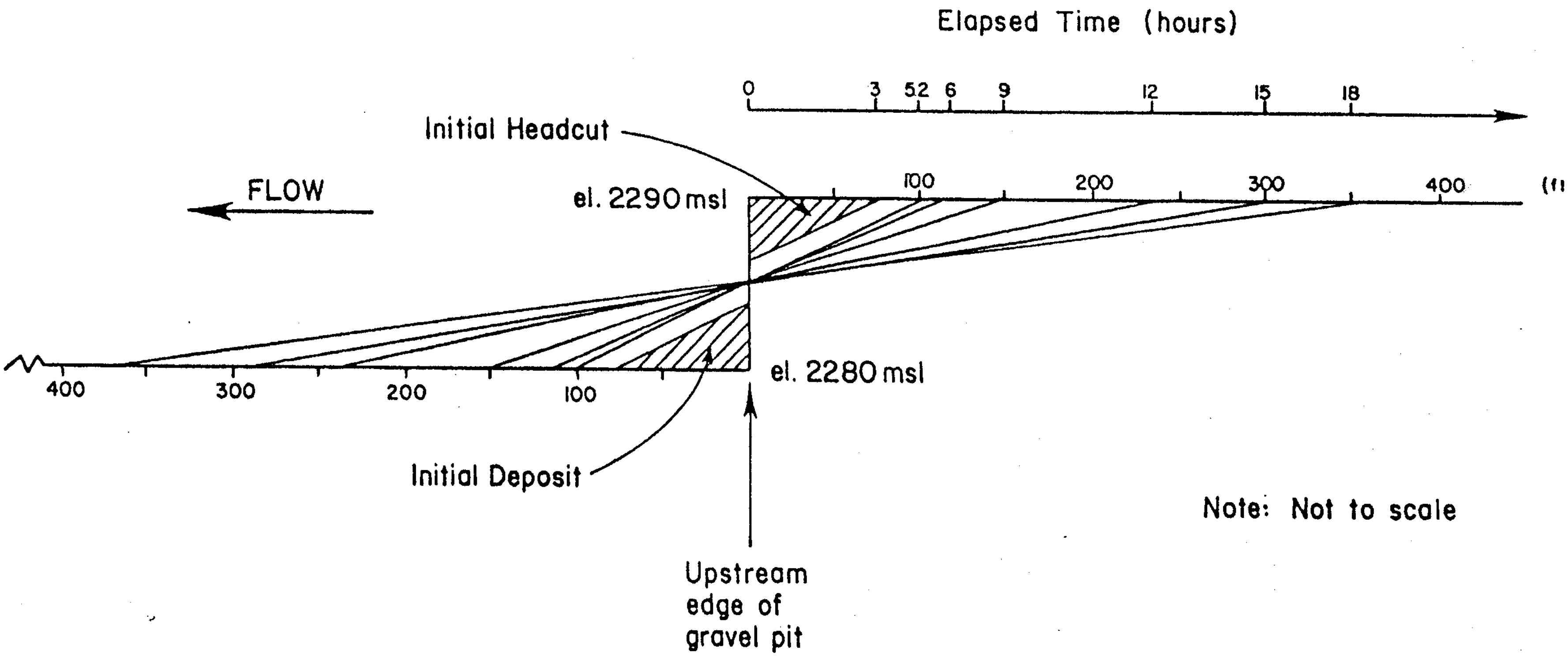


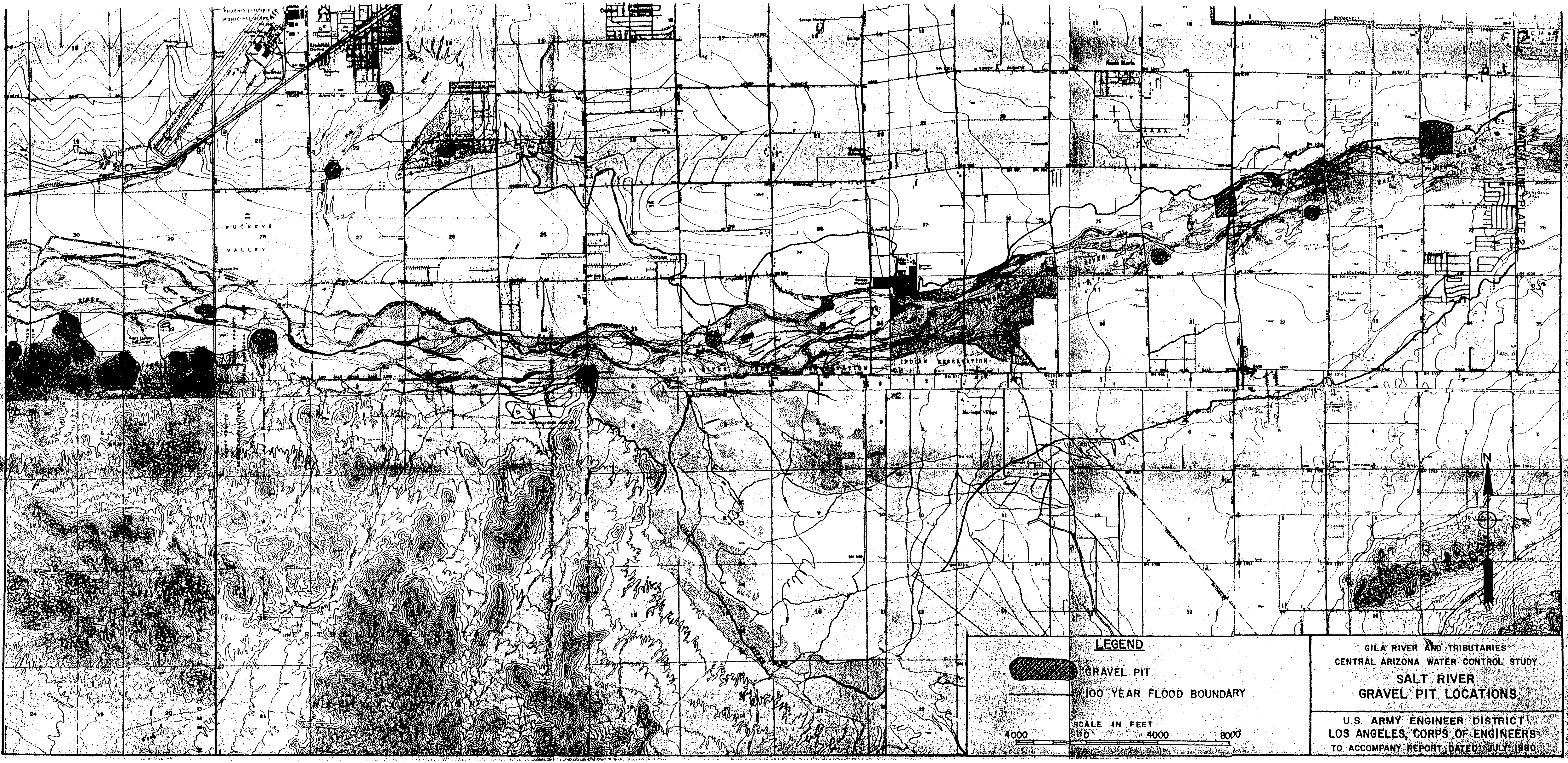
Figure A-4. Definition sketch of the temporal changes at the upstream edge of a gravel pit.

Table A-1. Calculated Headcut Distance and Slope.

Time (hrs)	Headcut Distance (ft)	Headcut Slope
3	76	0.050
5.2	100	0.050
6	116	0.044
9	176	0.029
12	237	0.022
15	299	0.018
18	363	0.015

and gravel mining operations. Selection of an approprdiate model should be based on the quantity and quality of available data, stream characteristics, and the special problems to be analyzed. Some of the models may be complex and expensive. If sufficient information is not available, the results could be misleading and the cost of using those models may not be warranted.

PLATES



BUCKEYE
VALLEY
GILA RIVER INDIAN RESERVATION
SALT RIVER
LEGEND
GRAVEL PIT
100 YEAR FLOOD BOUNDARY
SCALE IN FEET
4000 0 4000 8000
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY
SALT RIVER
GRAVEL PIT LOCATIONS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED: JULY 1980

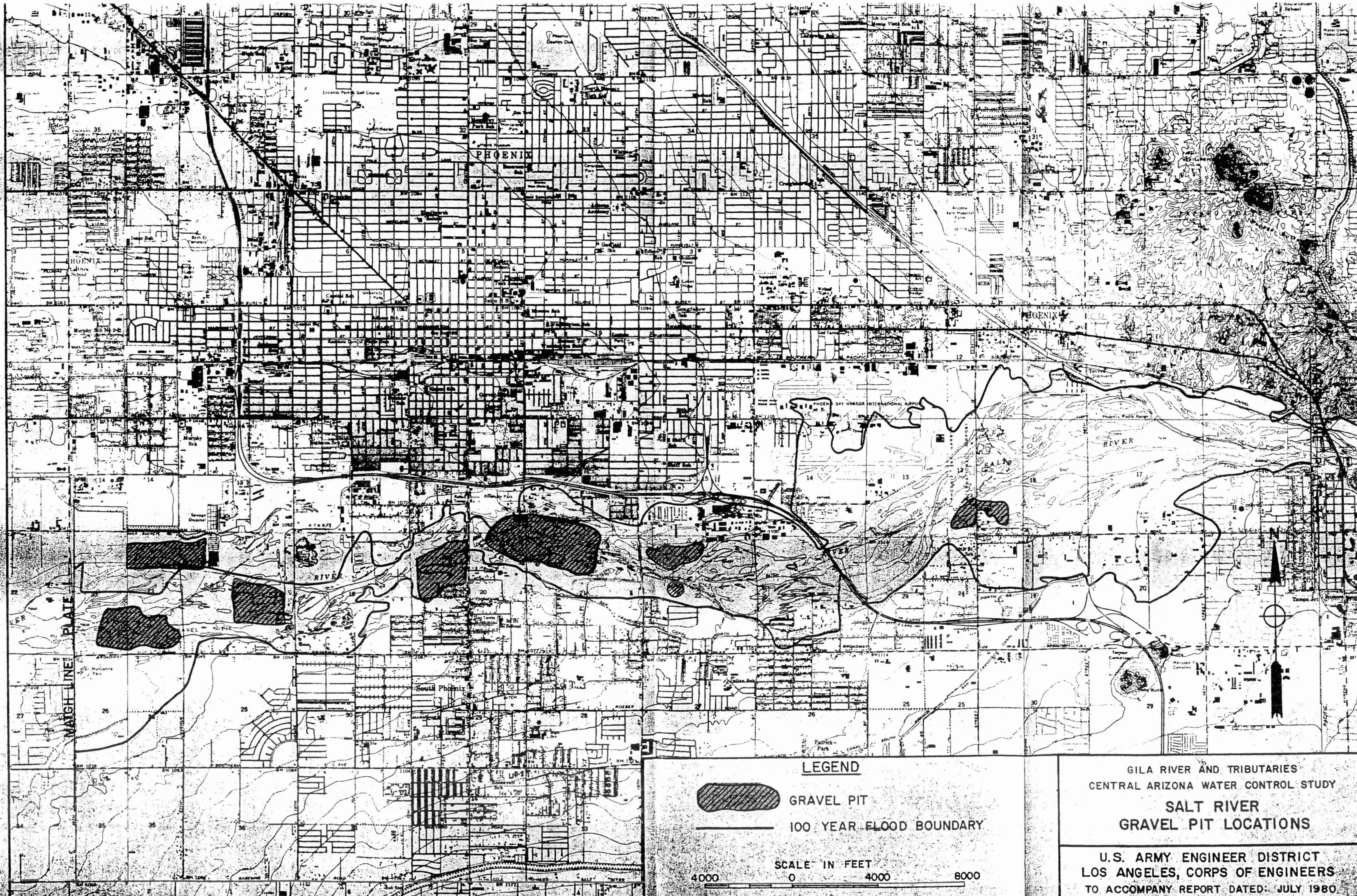

PHOENIX
Encanto Park & Golf Course
PHOENIX SKY HARBOR INTERNATIONAL AIRPORT
RIVER
SALT
South Phoenix
MATCH LINE PLATE 1
LEGEND
GRAVEL PIT
100 YEAR FLOOD BOUNDARY
SCALE IN FEET
4000 0 4000 8000
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY
SALT RIVER
GRAVEL PIT LOCATIONS
U.S. ARMY ENGINEER DISTRICT
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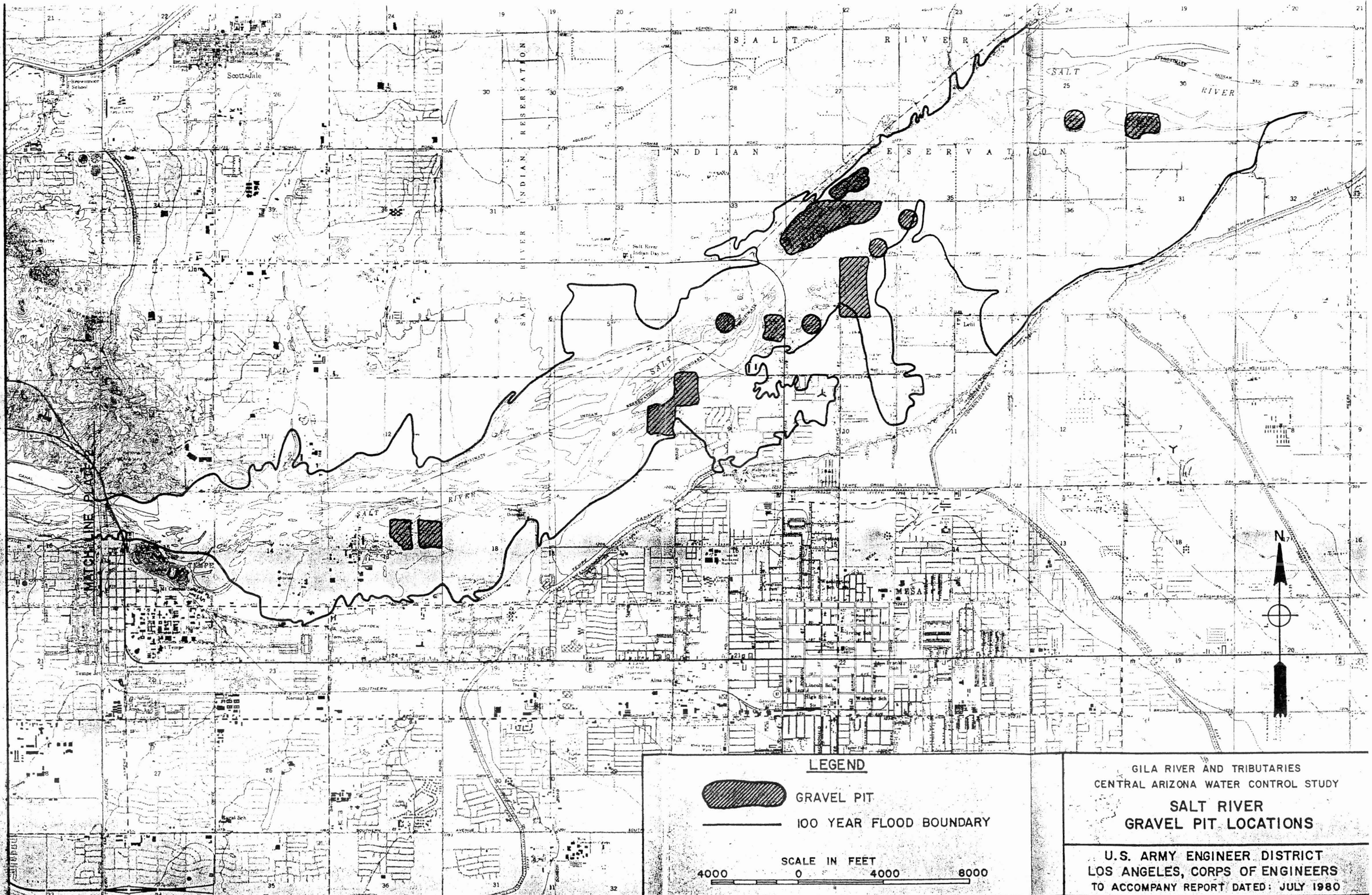
PLATE 2


LEGEND
GRAVEL PIT
100 YEAR FLOOD BOUNDARY
SCALE IN FEET
4000 0 4000 8000
GILA RIVER AND TRIBUTARIES
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SALT RIVER
INDIAN RESERVATION
SALT RIVER INDIAN RESERVATION
Scottsdale
TEMPE
MESA
Lehi
MATCH LINE PLATE
N