STANLEY CONSULTANTS, INC.

3117 North 16th Street • Phoenix, AZ 85016 Tel: 602/279-0901 Fax: 602/230-7655

December 27, 1991

Mr. David R. Johnson
FLOOD CONTROL DISTRICT OF
MARICOPA COUNTY
2801 West Durango
Phoenix, Arizona 85009

RE: Hydrologic Design Manual

Dear Mr. Johnson:

I have recently been using the District's new Hydrology manual in the design of drainage for the proposed expansion of Sun City West north of Deer Valley Drive. We are currently putting together a master drainage report for submittal in January 1992.

My exposure to the manual thus far has been primarily from the private development perspective although we did use a portion of the manual to develop initial and uniform rainfall loss rates for the Waterman Wash Hydrology Study and FIS we are just now completing for the District. The Sun City West drainage we are currently designing is based mainly on those parts of the manual dealing with the rational method, Clark unit hydrograph and Green-Ampt rainfall losses. I haven't dealt much with the "S" graph portion of the manual yet.

I understand that one of the primary purposes of the manual is to establish a standard methodology with a wide range of applications. This is a considerable objective with the wide range of both users and potential applications. My overall reaction to the manual is mixed. Considering the subjective nature of hydrology, I think the manual does a generally credible job of documenting its methods and presents a good text of background, theory and technical information. However, it departs from the more convenient and conventional rational and SCS unit hydrograph methods.

I know it's not possible to anticipate every circumstance and potential application of the manual when it is first developed. As the manual is used, there will be oversights corrected and additions made. I understand there are a number of things in flux at this moment and that a revision of the manual is due sometime in the spring of 1992. I would like to take this opportunity to point out a number of things that I have found in using the manual for your consideration in this up-coming revision.

1. The rainfall distributions for the 6 hour duration storm are significantly less intense than the 2 hour storm resulting in significantly lower peak flows.

2. Intensities from the rational method I-D-F curves, when compared with results from Chapter 2, "Rainfall", show good agreement for durations longer than 30 minutes but not very good agreement for shorter durations.

3. The manual states when discussing time of concentration that a flood wave travels faster than actual flow velocity. Yet, for several natural sub-basins, we observed wave velocities that were significantly slower than corresponding flow velocities.

4. The time-area relations in the manual for urban and natural watersheds do not yield significantly different results for small contributing areas.

5. Times of concentration for small contributing areas are significantly longer based on average rainfall intensity (as with the rational method) than based on average excess intensity (as with the Clark unit hydrograph method).

6. The manual is fairly comprehensive when dealing with Green-Ampt loss rates for natural soils. However, it contains very little regarding urbanized soil losses or the selection of either natural of urbanized percent impervious.

7. The Hydrology Manual is inconsistent with the selection of rational runoff coefficients and does not present any correlation of these coefficients with loss rates and percent impervious used with the Clark unit hydrograph method.

8. Our preliminary results indicate that the Clark unit hydrograph method tends to under estimate peak flows from our smallest urbanized watersheds and over estimate peak flows from our larger ones compared to results from the rational method using essentially the same data base.

9. In one particular case, the Clark unit hydrograph method yielded a significantly different peak discharge for a single natural contributing area when that area was divided into 2 sub-basins and their hydrographs added.

10. The manual does not include any guidelines or recommendations for storage routing of hydrographs through detention basins.

We intend to use the Clark unit hydrograph method to develop the hydrology for existing conditions and proposed onsite development for the Sun City West expansion master drainage report. This hydrology will be used in the design of open channels, conveyance of flows in the golf course, onsite detention basins, storm drains and conveyance of flow in major streets. However, future drainage reports which will accompany individual units will base their hydrology on the rational method. It is important, then, that the two methods are in reasonable agreement in both peak flow and volume. Our effort to accomplish this has resulted in many of the afore mentioned observations.

I have discussed many of these items with Steve Waters and Felicia Terry. I am available to discuss them in more detail and provide copies of our preliminary work from which we have made these observations. If there is any type of forum planned to gain input from others in the private sector, I would be interested in participation. In the mean time, we are proceeding on with our master drainage report for the expansion of Sun City West based on our response from Steve and Felicia.

Sincerely,

STANLEY CONSULTANTS, INC.

Scott Buchanan

Scott Buchanan, Hydrologist

SB/vm:DEC002

cc: Timothy D. Crall, Stanley Consultants
Joe Rumann, George V. Sabol, Consulting Engineers

PART 6

REVIEW AND COMMENTS FROM MANUALS INTRODUCTORY CONFERENCE

On May 1st, 1990, the Flood Control District sponsored a one day symposium for a number of invited guests for the purpose of introducing the Hydrologic Design Manual for Maricopa County, as well as to solicit comments on its technical merit and acceptability. The presentation was made by the authors and it was held at the Executive Park Hotel in Phoenix, AZ..

Those in attendance were asked to provide any written comments within 30 days, after which we would respond and make any necessary changes. Included in the appendices of this part are the following items.

Appendix	Material
6-A	List of conference attendees
6-B	Conference Questionaires
6-C	Mr. Gohmert's (SCS) Letter FCD Response Dr. Sabol's Response
6-D	Dr. May's (ASU) Letter FCD Response Dr. Sabol's Response
6-E	Mr. Creighton's (ADWR) Memo FCD Response Dr. Sabol's Response
6-F	Mr. Ramirez' (Mesa) Letter FCD Response
6-G	Mr. Bond's (Tempe) Letter FCD Response
6-H	Mr. Brunner (HEC) Letter FCD Response
6-I	Mr. Duren's (JMM) Letter FCD Response Dr. Sabol's Response
6-J	Mr. Miller's (JRJ) Letter FCD Response Dr. Sabol's Response
6-K	Mr. Kao's (AGK) Letter FCD Response

Overall, the letters expressed concerns that could not be immediately and effectively addressed in the Hydrologic Design Manual. These comments were very helpful by indicating where further research is needed. Otherwise, the comments were editorial in nature.

Drainage Design Manual
Volume 1
Hydrologic Design Manual
Promulgation Conference
Executive Park Hotel
Tuesday, May 1, 1990

Name	Affiliation
THOMAS E. SONNEMANN	MARICOPA CO. HIGHWAY DEPT.
Ray A Brown	City of Tucson
Dan Sherwood	City of Glendale
Shaja Amani	Cella Barr Assoc.
DAVID E. CREIGHTON JR.	AZ DWR.
STEVE CASILLAS	Pima County DOT & FCD
George V. Sabol	G.V. Sabol Consulting Eng. Inc.
Al Pfahl	City of Chandler
HARRY MILLSAPS	USDA-SCS
RAY JORDAN	ADOT
David Dust	Coe & VanLoo
PAUL KIENOW	CITY OF PHOENIX
Laurie Miller	James M. Montgomery
David Ramirez	City of Mesa
V. OTTOZAWA CHATUPRON	ADOT/ATRC
Larry W. Mays	ASU
Jesús Romero	Yavapai County Flood Control District
DAVID KOHE	CITY OF PEORIA
DAN NISSEN	City of Peoria
DON SNYDER	CITY OF TEMPE
JIM BOND	" " "
Samuel Kao	AGK Engineers
Jeff S. Erickson	WLB Group Inc
KEN QUARTERMAIN	HOME BUILDERS ASSOC. OF CENTRAL AZ.
PETER MILLER	JERRY R. JONES & ASSOC.

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Executive Park Hotel
Tuesday, May 1, 1990

Name	Affiliation
DENNIS HUSTEAD	NBS/LOWRY
Greg Crossman	City of Scottsdale
Ray Acuña	City of Phoenix
DALLAS REIGLE	SALT RIVER PROJECT

Drainage Design Manual
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Promulgation Conference
Executive Park Hotel
Tuesday, May 1, 1990

Name	Affiliation
DENNIS HUSTEAD	NBS/LOWRY
Greg Crossman	City of Scottsdale
Ray Acuña	City of Phoenix
HAL MARRON	AGK Engrs

Drainage Design Manual
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Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   YES - GOOD OVERVIEW OF INTENT AND RATIONAL FOR DEVELOPMENT OF MANUAL

2. Do you think this manual is useful?

   VERY USEFUL - IT WILL SIMPLIFY DRAINAGE CALCS & ENABLE CONSULTANTS TO ANTICIPATE ACCEPTABLE METHODS OF ANALYSIS.

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   YES

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

   NONE

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

   N/A

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   YES FOR DRAINAGE ENGINEERS AND TECHNICIANS WHO WORK WITH P.E.'s.

7. What kind of background do you think a person should have to use this manual? USERS OF THE MANUAL SHOULD EITHER BE PROFESSIONAL ENGRS WITH DRAINAGE EXPERIENCE OR TECHNICIANS WORKING UNDER THE DIRECTION OF PROF ENGR

8. Were the facilities for this conference acceptable? Why or why not?

   GOOD.

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?
   Yes, it will Take Time to absorb the Info.

2. Do you think this manual is useful?
   when I Get the Imformation thought out enough to utilize

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why? Need to Check it Out.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas. "C" Values.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction? Need to look into it more

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend? maybe later.

7. What kind of background do you think a person should have to use this manual? Engineering.
   Needs to know HEC-1

8. Were the facilities for this conference acceptable? Why or why not?
   Yes. Very Good.

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?
   good overview presentation; lack detail due to time limits

2. Do you think this manual is useful?
   need to study it further, but i part g Tc is good

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?
   N/A

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.
   comparison with past FCD methods

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?
   N/A

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend? YES IT WOULD BE HELPFUL
   & I WOULD GLADLY ATTEND

7. What kind of background do you think a person should have to use this manual? HYDROLOGY DEGREE OR LOT OF EXPERIENCE

8. Were the facilities for this conference acceptable? Why or why not?
   OK

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   Yes

2. Do you think this manual is useful? Yes

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why? Yes, with reservations - need time and experience to really determine effect.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas. Verification procedures for users to check reasonableness of results. Include some regression equations and curve envelopes.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction? 1 year Trial Period to prove manual and see results.

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend? Yes - Yes

7. What kind of background do you think a person should have to use this manual? BS Degree in Engineering or Hydrology

8. Were the facilities for this conference acceptable? Why or why not?

   Yes -

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   Yes, well organized.

2. Do you think this manual is useful?

   Yes.

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   Yes

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   Yes

7. What kind of background do you think a person should have to use this manual?

8. Were the facilities for this conference acceptable? Why or why not?

   OK

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not? typically, although many different things were presented with the result of being an overview of available methods and not an overview of the MHDM

2. Do you think this manual is useful?

   yes

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   N/A

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

   N/A

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   yes / yes

7. What kind of background do you think a person should have to use this manual? hydrology or engineering

8. Were the facilities for this conference acceptable? Why or why not?

   yes - very good

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

YES

2. Do you think this manual is useful?

YES

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

YES, BUT HAVE TO REQUIRE "C" VALUE FOR RETENTION TO BE .95 FOR ALL SURFACES.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

INCLUDE HAND METHOD FOR YOUNGTOWN WATERSHED IN FINAL ISSUE - THE PART REFERENCED IN THE AFTERNOON SESSION (FOR US WHO DON'T TRUST COMPUTERS)

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

C=.95 FOR ALL RETENTION ~~CALCS~~. CALCS.
+ GO THROUGH MAG - MAKES ACCEPTANCE BY ALL CITIES EASIER, DOES NOT REQUIRE NEW ORDINANCE

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

YES, YES

7. What kind of background do you think a person should have to use this manual?

SOME PREVIOUS TRAINING, ~~OR~~ EDUCATION, OR EXPERIENCE

8. Were the facilities for this conference acceptable? Why or why not?

YES

(OVER)

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   technical to late in day, Clark Unit & S-graph seemed to leave out some key steps

2. Do you think this manual is useful?

   Extremely if include more intermediate steps

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   Maybe Depends if applicable to are

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   Yes No only if jurisdiction adopts

7. What kind of background do you think a person should have to use this manual?

8. Were the facilities for this conference acceptable? Why or why not?

   Yes No umbrellas for tables

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

Yes, the material was well presented and there appears to have been a significant amount thought and logic which went into the manual. The procedures presented seem to be generally accepted methods adapted for use in Maricopa County. The staff/consultants presenting this material appear to be very knowledgeable in the hydrology field.

2. Do you think this manual is useful?

Yes, I think the manual will be useful in standardizing procedures and methods used in hydrologic studies throughout the county.

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

I would like to use the methods and procedures in this manual for areas within the City of Scottsdale where we have existing studies and compare the results before making a final determination.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

I would like to take a more in-depth look @ the manual and then provide comments.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

Same comment as #4.

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

Yes, I think that if this manual becomes "the" accepted standard, I feel that expanded training would be appropriate and would attend.

7. What kind of background do you think a person should have to use this manual?

Basic hydrology/engineering background.

8. Were the facilities for this conference acceptable? Why or why not?

Yes.

Greg Crossman
City of Scottsdale
994-2326

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   I need time to digest. This is not my daily involvement.

2. Do you think this manual is useful?

   Very useful.

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   Possibly

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

   /

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

   Possible later comment after thorough review.

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   Yes

7. What kind of background do you think a person should have to use this manual?

   /

8. Were the facilities for this conference acceptable? Why or why not?

   Very acceptable.

DSP

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   Yes - The presentation was excellent. Further follow-up or study of the manual will be required to understand it fully.

2. Do you think this manual is useful?

   yes

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   If they exceed SCS criteria, we will use local guidelines

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

   Will need to study the manual more fully to provide further comment

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

   N/A

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   yes

7. What kind of background do you think a person should have to use this manual?

   A working knowledge of hydrology & its processes

8. Were the facilities for this conference acceptable? Why or why not?

   Very Good

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?
   Yes - The examples will help clarify

2. Do you think this manual is useful?
   Extremely

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?
   Yes - It is important to develop a consistant, uniform method to start handling drainage.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.
   Emphasize the advantages to the City's in adopting this manual.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?
   Technicians will be using this manual - this should be considered in the format.

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?
   Yes - Yes

7. What kind of background do you think a person should have to use this manual?
   Basic physics.

8. Were the facilities for this conference acceptable? Why or why not?
   Yes

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?
   yes, understandable as to process and goals.

2. Do you think this manual is useful?
   As a guide for methodology and after revision of draft and/or preparation of ERRATA SHEET.

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why? NO. As presented, the exclusion of normal depth routing method precludes percolation loss inclusion in routing. This skews the lower volume (higher frequency) discharges and runoff.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas. Yes. See 3 above, and broad US Data from Eastern US without identification of Arizona - New Mexico data separately for possible differentiation - Green Ampt Values from Rawls - Table 2

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction? not applicable

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend? Yes, or substitute.

7. What kind of background do you think a person should have to use this manual? Adequate experience to recognize output abnormalities that may result, ~~when~~ and basic background in engineering and hydrology to use logical input concepts rather than illogical.

8. Were the facilities for this conference acceptable? Why or why not?
   Yes

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?
   Need page #'s on all pages
   Presentation seemed to jump around at times

2. Do you think this manual is useful?

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why? N/A

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction? N/A

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?
   Hands on workshop
   with computers

7. What kind of background do you think a person should have to use this manual?
   A strong understanding of Drainage principles, It is doubtful most Engr. firms will have personnel that will understand most of the concepts

8. Were the facilities for this conference acceptable? Why or why not?
   yes

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   yes (examples were a little difficult to follow)

2. Do you think this manual is useful?

   definitely

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   N/A

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

   Sedimentation/Erosion issues as they relate to flows/volumes

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

   N/A

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   yes, yes

7. What kind of background do you think a person should have to use this manual?

   some hydrologic experience as a minimum

8. Were the facilities for this conference acceptable? Why or why not?

   yes

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not? Yes, was a clear summary of purpose and applications of manual.

2. Do you think this manual is useful? yes

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why? yes I believe so.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas. –

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction? will have to be reviewed in more detail by other staff members before this is known.

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend? Possibly after we've had the opportunity to review the manual in detail.

7. What kind of background do you think a person should have to use this manual? ?

8. Were the facilities for this conference acceptable? Why or why not?
Some of the overheads were difficult to read.

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not? 1) TABE 2 WAS CONFUSING. (OVER)

   2) ~~NEED HANDS ON WITH COMPUTERS & REAL TIME OVERHEAD FROM P.C. SCREEN~~

   3) MORE GENERAL HEC1 GUIDELINES NEEDED

2. Do you think this manual is useful?

   YES

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   DON'T KNOW

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

   TOO SOON TO TELL

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   YES

   YES

7. What kind of background do you think a person should have to use this manual?

   4 YRS + EXPERIENCE IN HYDRO/HYDRA DESIGN

8. Were the facilities for this conference acceptable? Why or why not?

   YES

Drainage Design Manual
Volume 1
Hydrologic Design Manual

Conference Questionnaire

1. Do you think that the material presented to you today was understandable? Why or why not?

   Yes, considering the scope of the audience.

2. Do you think this manual is useful?

   Yes - It appears to be a mean between oversimple approaches (ADOTs manual) and overdetailed ones (SCS - NEH-4).

3. Would you be willing to use the Drainage Design Manuals in your jurisdiction? If not, why?

   Yes, we have been needing a uniform method for consultants. ADOTs manual is obsolete.

4. Are there areas of concern that were not addressed today? If so, please indicate which areas.

   For drainage areas which are at the small end of the range (say for storm sewers) is there ~~are~~ a minimum value for t_c?

5. What should be added or deleted to make the Drainage Design Manual acceptable to your jurisdiction?

   . Expanded 'C' value tables (Rational Method).

6. Would an expanded training conference on the Drainage Design Manual be helpful? Would you attend?

   Yes - I would not personally attend as I worked on it (Vol. II), but someone from MCHD would.

7. What kind of background do you think a person should have to use this manual?

   B.S. in Civil Engineering or Hydrology
   Some experience in Hydrology / Hydraulics.
   NOT FOR TECHNICIANS!

8. Were the facilities for this conference acceptable? Why or why not?

   Yes, better than average
   (First time I have had lunch at a agency-sponsored seminar!)

United States Department of Agriculture

Soil Conservation Service

201 E. Indianola Avenue
Suite 200
Phoenix, Arizona 85012

FLOOD CONTROL DISTRICT
RECEIVED
JUN 05 '90

CH ENG		P & PM	
DEF		HYDRO	
ADMIN		LMGT	
FINANCE		FILE	
C & O	/	HSC	
ENGR			

REMARKS

June 1, 1990

Dan Sagramoso
Chief, Maricopa County
Flood Control District
3335 W. Durango Street
Phoenix, Arizona 85009

Dear Mr. Sagramoso:

We have completed our review of the Draft Hydrologic Design Manual for Maricopa County, and have found items both of interest and concern. We have found the manual to be well written and easy to understand. It should help the county to reach its goal of consistent analysis of drainage requirements within the county. It is good that the authors of the manual have recognized that it is not an all encompassing document, but that good engineering judgement and reasoning will be needed in its application. Of course, this is true of most, if not all, hydrologic procedures.

One item we found of interest in the manual is the use of the Green and Ampt Inflitration Model to estimate rainfall loss rates. The Soil Conservation Service (SCS) has considered this model for several years as an "alternative-to" but not necessarily "a-replacement-for" the curve number procedure. The main drawback has been insufficient soils data (on a national basis) to adequately estimate the required runoff parameters. Also, there is a need for a much greater understanding of the hydrologic cycle in order to apply the Green and Ampt Model. The difficulty of application is also much greater than that of the CN method. The latter considerations are especially important when the procedures are to be used by non-engineering types of employees, such as engineering technicians located in SCS field offices.

For these reasons the SCS will continue to use the CN procedure for estimating on-site runoff, and we are therefore concerned about how it is depicted in the hydrologic manual. Although we recognize the limitations of the CN procedure and that the runoff equation might not stand up under a rigorous theoretical analysis, it is, based on empirical and historical data, and provides reasonable answers when used with good judgement and knowledge of the procedure. As noted in the design manual, it has been well received within Maricopa County, and has withstood the test of time and use. We are not necessarily suggesting that the CN procedure be selected as one of the preferred methods of runoff prediction, but at the same time, the authors of the manual should refrain from trying to discredit this procedure. The non-acceptability of the procedure is not a common opinion among the engineering profession, but we think is simply a bias of the manual's authors.

One of the major weaknesses we see in both the SCS and the Hydrologic Design Manual procedures is the lack of any method to account for transmission losses. This can be seen in the results of the Cave Creek Wash example problem presented in the manual. Both the SCS and the Hydrologic Design Manual procedures over-estimated the 10-year frequency discharge for this stream. This probably results from an over-estimation of runoff volume at the gaged site, not because of too high a curve number or underestimating the Green and Ampt loss rate parameters, but due to ignoring transmission losses. As noted in the example, the predicted 100-year flood by both the SCS and HDM procedures agree more closely with the theoretical discharge than that for the 10-year flood. It is theorized that the effect of transmission losses is much greater on the more frequent (10-year) flood(s) than on the 100-year or less frequent flood(s). It is in the area of transmission losses that the SCS feels major improvements are needed in trying to estimate runoff from arid and semiarid watersheds, rather than trying to improve the "on-site" prediction models.

We hope these comments will be of benefit to you. We also hope that you will consider removing from the manual the effort to discredit the SCS Runoff Curve Number Procedure.

Sincerely,

Donald W. Gohmert

DONALD W. GOHMERT
State Conservationist

FLOOD CONTROL DISTRICT of MARICOPA COUNTY 1959

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 30 1990

Mr. Donald W. Gohmert
State Conservationist
U.S. Department of Agriculture
Soil Conservation Service
201 E. Indianola Avenue, Suite 200
Phoenix, Arizona 85012

SUBJECT: Comments on Hydrologic Design Manual

Dear Mr. Gohmert:

Thank you for taking the time to review and comment on the District's Hydrologic Design Manual. Your comments have been carefully considered and changes have been made to the final draft as a result of your correspondence. The final draft should be available by the middle of August.

When we were putting together the criteria to be used in the manual, we realized that, since a majority of the engineering community uses the Curve Number method, it might be necessary to list the limitations of this method. This was not done in an attempt to discredit the procedure, but rather as a justification for using the Green and Ampt Infiltration equations. We did not deliberately intend to discredit the Curve Number method and apologize for any misunderstanding. Disparaging comments will be removed from the final draft.

We also agree with you concerning the weakness of both methods to account for transmission losses. We anticipate that as additional data becomes available with which criteria can be developed, later editions of the manual will include some guidelines for transmission losses.

If you have any other comments or would like to meet with the authors to discuss the data used to develop the manual, please call. Once again I would like to thank you for you input and apologize for any misunderstanding of our intent with regard to the Curve Number method.

Sincerely,

STANLEY L. SMITH JR., P.E
DEPUTY CHIEF ENGINEER
D. E. Sagramoso, P.E.

TLB/JMR/ag

* See reverse for filing information.

George V. Sabol Consulting Engineers, Inc.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

13 August 1990

Mr. Joe Rumann
Hydrologist
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Subject: Hydrologic Design Manual for Maricopa County
review comments by Mr. Donald W. Gohmert, SCS

Dear Mr. Rumann:

I have reviewed the comments by Mr. Gohmert in his letter of 1 June 1990, and the Flood Control District's response in its letter to Mr. Gohmert of 30 July 1990. I agree with the District's response, and I offer the following additional comments that you may wish to consider.

As a coauthor of the manual, I would like to reassure Mr. Gohmert that there was no intent to discredit the CN method and I apologize for that regrettable misunderstanding. The authors of the manual previously have agreed to remove such disparaging comments from the manual, as the District's letter states. The limitations to the CN method were originally contained in the manual because it was our perception that it would be necessary to provide a technical justification to the engineering community as to why the CN method was not selected. It no longer appears to be necessary to provide this justification, which has been taken as undue criticism.

In support of our selection of the Green and Ampt infiltration equation with an estimate of surface retention losses, I believe that adequate soils data is available in Maricopa County to adequately estimate rainfall losses by this method. The parameters may need to be revised in the future as more and better data become available. I don't think that the method to estimate rainfall losses by the Green and Ampt equation with the HEC-1 program and using the guidance that is provided in the manual is any more difficult than is the application of the CN method.

Furthermore, I don't think that we need to be as concerned about use of the manual by non-engineering types of employees or those without a hydrology background as the SCS is, particularly in SCS field offices. The authors of the manual have written the manual for use by qualified and experienced professionals.

Mr. Gohmert's comments are appreciated, and I hope that this response of mine is of assistance to the District in resolving some of Mr. Gohmert's concerns about the manual. It may be of value to meet with Mr. Gohmert after the next revision to the manual is available to discuss these and other related matters with him. I would be available for such a meeting, if desired.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo Sabol

George V. Sabol

Arizona State University
College of Engineering and Applied Sciences
Department of Civil Engineering
Tempe, AZ 85287-5306
602/965-3589

May 30, 1990

Mr. Dan Sagramoso, P.E.
General Manager and Chief Engineer
Maricopa County Flood Control District
3335 West Durango Street
Phoenix, Arizona 85000

Dear Dan:

I was quite pleased to be invited to attend your seminar and asked to review the Hydrology Manual. After some review of the manual I have quickly put together the following comments that may be of use to you and your staff. These comments are not meant to criticize anyones work, but to point out some things that may cause concern or problems in the future. If you and your staff would like to discuss these comments further, I would be pleased to do so. In the following discussion I have given several references to the book Applied Hydrology by V.T. Chow, D.R. Maidment, and L.W. Mays. Not knowing whether or not you had a copy, I've enclosed one that you can now consider your own.

To begin with, I applaud you and your staff for using the HEC-1 computer code as the major vehicle for performing rainfall-runoff studies. I feel that this is a major first step in any manual such as this.

a) <u>Rational Method</u>

The rational formula is presented in equation (1) followed by a definition of the variables. The definition of T_c is given which is not in equation (1) and is then given again after equation (2). The rational formula might better be stated as it is actually used as

$$Q = i\sum_{j=1}^{m} C_j A_j$$

where m is the number of subcatchments. One comment is that there are much more extensive tables for the C coefficients that should be incorporated. It is an accepted fact that C varies with many factors, one of which is the return period which is not reflected in this table. The City of Austin, Texas developed a very detailed table that would certainly be applicable.

Has any verification been made of equation (2) for Maricopa County? Based upon equation (4) on page 35 your detention requirements are to store the 100-year-2hour storm. Is this requirement for any condition: residential, commercial and industrial? Do other municipalities use this criteria? There are much more extensive approaches using the modified rational method to determine the optimal storm duration, i.e. the duration that causes the maximum volume for a particular frequency at a particular location. The optimal duration varies for different situations. We have one discussion in Applied Hydrology and I'm sending you another that is in the manuscript of a new book, <u>Hydrosystems Engineering and Managment</u> by L.W. Mays and Y.K. Tung to be published by McGraw-Hill. The upper limit of 160 acres for the rational method seems to be high, especially when we really think about the assumptions.

Has anyone developed the I-D-F equation for the curves in Figure 3.2? This is a rather simple task and would be very helpful to engineers.

b) <u>Unit Hydrographs</u>

I was somewhat surprised to see recommendations of the Clark Unit Hydrograph Method and not the allowance of Gray's or the SCS. Using the Snyder's method automatically develops the Clark Unit hydrograph parameters. In teaching the shortcourse, Flood Plain Hydrology using HEC-1, we do not cover application of the Clark unit hydrograph method. Dr. David Ford, formerly of HEC and I have never covered this method in detail because engineers around the U.S. simply don't use it. Instead the Snyder's coefficients are read in to determine the Clark unit hydrograph parameters. I doubt many engineers in Maricopa county understand the Clark Unit hydrograph method. In the book Applied Hydrology, we chose not to even present the method.

What is the basis for the time-area curves for urban watersheds and natural watersheds in Table 5.2? These obviously would vary for each watershed. What is the basis to use a general relationship for all situations? Many factors affect this, of which one of the most obvious is shape of the watershed.

One procedure for synthetic unit hydrograph generation has been to relate the Snyder's coefficients to watershed parameters such as water course length and slope, percent imperviousness, watershed area, and watershed conveyance. This was originally started in Texas and now has been used and accepted in other parts of the country. A good reference on this is the City of Austin, Texas Drainage Criteria Manual.

c) Rainfall Losses

I am extremely pleased to see that use of the Green-Ampt method has been recommended. This is the only physically based method in HEC-1 that can be derived from fundamental principles using the governing equation of flow. A very detailed theoretical derivation and discussion is provided in Applied Hydrology.

I do think that some of the statements in the Hydrology Manual concerning the SCS method for infiltration are misleading and incorrect. The statement, "rainfall losses are independent of the duration of rainfall", is simply not true of the SCS infiltration method. This is true of the SCS graph and equation that relates runoff to rainfall for various curve numbers, but is not true for the time variation infiltration method in HEC-1. Refer to the discussion and derivations of this method in Applied Hydrology. In most parts of the country the curve number, CN can be selected from county soil survey maps which are based upon soil types in a particular area. I personally feel that the selection of CN values are certainly less subjective than the selection of coefficients for the rational method. The statement, "the initial abstraction is equal 0.2S", is not complete. This is the accepted for antecedent moisture condition of II, but is only the default in HEC-1. Other initial abstractions can be used in HEC-1 besides the 0.2S.

d) Channel Routing

I am somewhat concerned about recommending the use of the kinematic wave for channel routing of urbanized channels. Obviously the most accurate one-dimensional routing approach is a full dynamic wave model solved by an implicit scheme such as that found in the U.S. National Weather Service DAMBRK and DWOPER codes. However this method is not included in the HEC-1 computer code as one of the channel routing approaches. If you do include the kinematic wave, I feel that the manual needs more explanation of the method and warnings for users. The statement on page 91, "the routing process involves minimal attenuation", is not completely correct. The kinematic method solves a simplified form of the Saint Venant equations. The simplification is that the local acceleration, convective acceleration, and pressure terms in the momentum equation are ignored. These terms provide the mechanism to describe back water effects, and describe the energy slope which defines the frictional effects. A full dynamic wave model includes all of these terms. For the kinematic wave method the energy slope is simplified to be the channel bottom slope for purposes of computing friction effects. So theoretically the kinematic wave procedure provides for no flood wave attenuation and is a pure translation of the wave. The attenuation that does occur when applying this method is an artificial attenuation due to numerical error that varies with the reach length and time step. This discussion leads to the fact that the kinematic wave could be used where backwater conditions are insignificant and on relatively steep slopes where frictional effects are minimal. I guess the question is, "Do the urban channels in Maricopa County satisfy these conditions"? Unfortunately the numerical scheme used in the HEC-1 is a linear-explicit scheme which is not the most desirable. A non-linear implicit scheme would have much more desirable. This is discussed in detail in Applied Hydrology. Other methods more properly account for numerical dispersion.

When we teach the use of the kinematic wave in the shortcourse, Floodplain Hydrology using HEC-1, I try to bring out these points as warnings. This method can be used in rather limited situations where backwater conditions are insignificant and on relatively steep channels. I have found that most consulting engineers are not aware of these factors and basically do not understand the kinematic wave method.

If the kinematic wave is used, a great deal of care will need to be taken both in the application by consulting engineers and by Maricopa County Flood Control District in checking the work of consultants. I enclosed an article concerning the kinematic wave method in HEC-1 and the discussion that resulted. As you can see this method is a somewhat controversial subject. A much better method is the Muskingum-Cunge method which is a diffusion wave approach. Unfortunately though HEC did not put this into the HEC-1 computer program.

e) One major concern that I have about the manual is that it does not address the hydrology and hydraulics of alluvial plains and of alluvial fans. I'm finding out that the local engineering community simply does not have an understanding of the hydrologic and hydraulic aspects of alluvial situations. No one has a real good handle on how we should hydrologically and hydraulically model active alluvial fans. The FEMA procedure for active alluvial fans simply is not physically based on any sound hydraulic principles and should be used with extreme caution, if at all. I would think that a good addition to this manual would be to address some of the engineering (hydrologic and hydraulic) issues associated with alluvial fan and alluvial plains.

f) The use of worked out examples in the appendices is excellent. This was a very good idea. Overall I think this is a very good manual and should serve the community very well.

If you have questions about my comments, I would be very pleased to discuss them. I would like to get a copy of your Hydraulics manual and a final copy of the Hydrology Manual when they are completed. My best regards and good luck on this manual.

Sincerely yours,

Larry

Larry W. Mays, Ph.D., P.E., P.H.
Chairman and Professor

LWM:slh

From: Hydrosystems Engineering and Management by L.W. Mays, & Y.K. Tung, To be published by McGraw-Hill Book Co., 1991,

assimilated and/or transported by urban runoff from land, pavements and other surfaces. Several different methods exist for the detention of stormwater runoff including underground storage, basins and ponds on ground surface, parking lot storage, and rooftop detention.

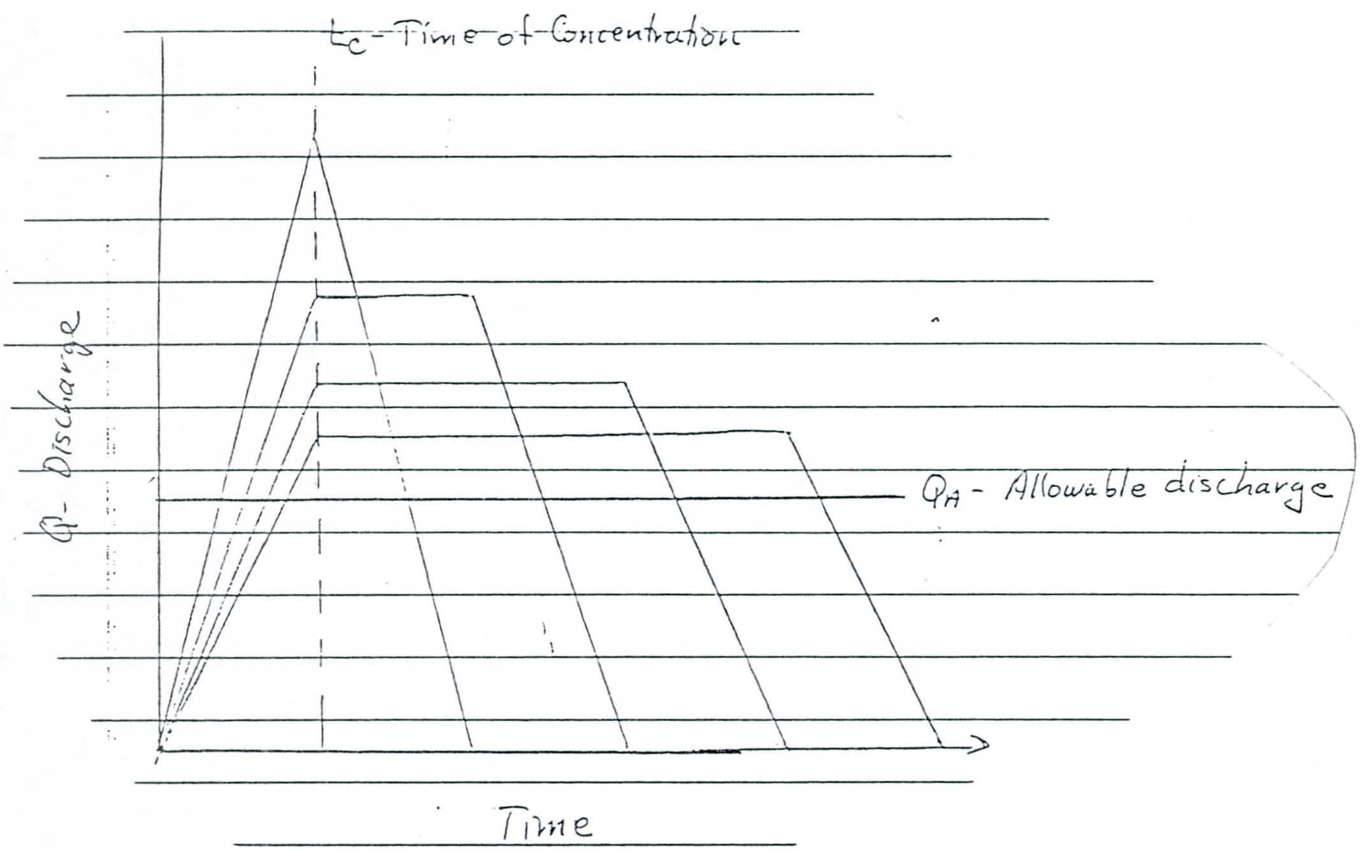
11.6.2 Selection of Detention Pond Size - Modified Rational Method

The modified rational method can be used to determine the preliminary design, which is the detention pond volume requirements for contributing drainage areas of 30 acres or less. For larger contributing areas a more detailed rainfall-runoff analysis with a detention basin flow routing procedure should be used. The modified rational method is an extension of the rational method to develop hydrographs for storage design, rather than just flood peak discharges for storm sewer design. The shape of hydrographs produced by the modified rational method is either a triangular or trapezoidal shape constructed by setting the duration of the rising and recession limbs equal to the time of concentration, t_c, and computing the peak discharge assuming various durations. Figure 11.6.3 illustrates modified rational method hydrographs.

An allowable discharge, Q_A, from a proposed detention basin can be the requirement that the peak discharge from the pond be equal to the peak of the runoff hydrograph for predeveloped conditions. The required detention storage, V_s, for each rainfall duration can be approximated as the cumulative volume of inflow minus the outflow as shown in Figure 11.6.4.

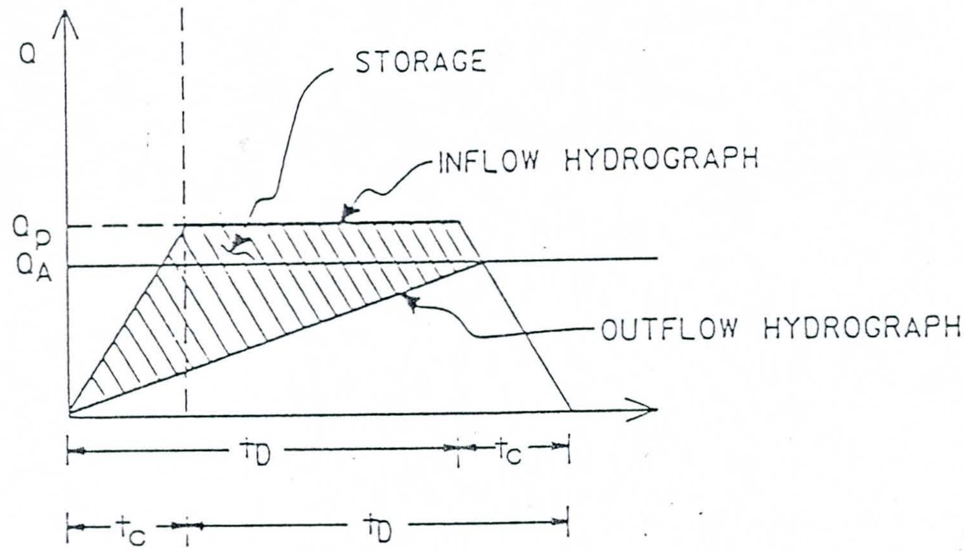
The assumptions of the modified rational method include:

1) the same assumptions as the rational method (Section 11.2.2),
2) the period of rainfall intensity averaging is equal to the duration of the storm;



t_c - Time of Concentration
Q - Discharge
Q_A - Allowable discharge
Time

Figure 11.6.3



t_C : TIME OF CONCENTRATION (MINUTES)

t_D : TIME OF DURATION (MINUTES)

Q_P : PEAK FLOWS (CFS)

Q_A : MAXIMUM ALLOWABLE RELEASE RATE (CFS)

Figure 11.6.4

3) because the outflow hydrograph is either triangular or trapezoidal, then the effective contributing drainage area increases linearly with respect to time.

An equation for the critical storm duration, i.e., the storm duration which provides the largest storage volume can be determined for small watersheds based upon the modified rational method. Consider a rainfall intensity-duration-frequency equation of the general form

$$i_D = \frac{a}{(t_D + b)^c} \tag{11.6.1}$$

where i_D is the average rainfall intensity (in/hr) for the specific duration and return period; t_D is storm duration in minutes; and a, b, and c are coefficients for a specific return period and location. Consider the trapezoidal shaped inflow hydrograph and outflow hydrograph shown in Figure 11.6.4.

Using the rational formula the peak discharge can be expressed in terms of the storm duration

$$Q_p = C_p \, i_D A$$

$$= C_p \left[\frac{a}{(t_p + b)^c} \right] A \tag{11.6.2}$$

The inflow hydrograph volume V_i in ft^3, is expressed as

$$V_i = 60 \, (0.5) \, Q_p \left| \left(t_D - t_C \right) + \left(t_D + t_C \right) \right| \tag{11.6.3}$$

where t_C is the time of concentration for proposed conditions. The outflow hydrograph volume V_0 in ft^3, is expressed as

$$V_0 = 60\,(0.5)\,Q_A\left(t_D + t_C\right) \qquad (11.6.4)$$

where Q_A is the allowable peak flow release in ft^3 The storage volume V_S in ft^3 is computed using the above expressions for V_i and V_0

$$\begin{aligned} V_S &= V_i - V_0 \\ &= 60\,(0.5)\,Q_p\left[\left(t_D - t_C\right) + \left(t_D + t_C\right)\right] - 60\,(0.5)\,Q_A\left(t_D + t_C\right) \\ &= 60\,Q_p\,t_D - 30\,Q_A\left(t_D + t_C\right) \qquad (11.6.5) \end{aligned}$$

The duration for the maximum detention is determined by differentiating (11.6.5) with respect to t_D and setting the derivative equal to zero:

$$\begin{aligned} \frac{dV_S}{dt_D} &= 0 = 60\,t_D\,\frac{dQ_D}{dt_D} + 60\,Q_p - 30\,Q_A \\ &= 60\,t_D\,C_p\,A\,\frac{di_D}{dt_D} + 60\,C_p\,i_D\,A - 30\,Q_A \qquad (11.6.6) \end{aligned}$$

where

$$\frac{di_D}{dt_D} = \frac{d}{dt_D}\left[\frac{a}{(t_D + b)^c}\right] = \frac{-ac}{(t_D + b)^{c+1}} \qquad (11.6.7)$$

so

$$\frac{dV_S}{dt_D} = 0 = 60\,C_p\,A\,(-ac)\,\frac{t_D}{(t_D+b)^{c+1}} + 60\,C_p\left[\frac{a}{(t_D+b)^c}\right] - 30\,Q_A \qquad (11.6.8)$$

Simplifying results in

$$\frac{t_D\,(1-c) + b}{(t_D+b)^{c+1}} - \frac{Q_A}{2C_pA} = 0 \qquad (11.6.9)$$

which can be solved for t_D using Newton's iteration technique,

$$t_{D_{i+1}} = t_{D_i} - \frac{F(t_{D_i})}{F'(t_{D_i})} \qquad (11.6.10$$

where

$$F(t_{D_i}) = \frac{\left[t_D (1-c) + b\right]}{(t_D + b)^{c+1}} - \frac{Q_A}{2C_pA} = 0 \qquad (11.6.11)$$

and

$$F'(t_{D_i}) = \frac{d\left|F(t_{D_i})\right|}{dt_D} = -\frac{\left[t_D (1-c) + b\right](c+1)}{(t_D+b)^{c+2}} + \frac{(1-c)}{(t_D+b)^{c+1}} \qquad (11.6.12)$$

Example 11.6.1

Determine the critical duration, t_D, for a 15.24 ac watershed with a developed runoff coefficient of $C_p = 0.85$. The allowable discharge is the predevelopment discharge of $Q_A = 32.17$ cfs. The time of concentration for proposed conditions is 21.2 min. The applicable rainfall - intensity - duration relationship is

$$i_D = \frac{97.86}{(t_D + 16.4)^{0.76}}$$

Solution

The critical storm duration is found from solving equation (11.6.9) by use of Newtons method with an initial guess of the duration of 30 min. The computerized procedure begins by using Eq. (11.6.11)

$$F\left(t_{D_{i=1}}\right) = \frac{\left|30(1-0.76) + 16.4\right|}{(30 + 16.4)^{0.76}} - \frac{32.17}{2\,(0.85)\,(15.24)}$$

$$= 0.0148$$

and Eq. (11.6.12)

$$F^{1}\left(t_{D_{i=1}}\right) = -\frac{\left|30(1-0.76) + 16.4\right|(0.76 + 1)}{(30 + 16.44)^{0.76+2}} + \frac{(1-0.76)}{(30 + 16.4)^{0.76+1}}$$

$$= -0.00076$$

Applying Eq. (11.6.10)

$$t_{D_{i=2}} = 30 - \frac{0.0148}{-0.00076}$$

$$= 49.42 \text{ min}$$

The procedure continues using $t_{D_{i=2}} = 49.42$ min in the next iteration. The results are presented below in Table 11.6.1.

Table 11.6.1 Application of Newton's Method (Example 11.6.1)

Iteration	t_{D_i}	$F(t_{D_i})$	$F'(t_{D_i})$	$t_{D_{i=1}}$
1	30.0000	0.0148436	-0.0007643	49.4203
2	49.4203	0.0051300	-0.0003251	65.1982
3	65.1982	0.0011547	-0.0001949	71.1222
4	71.1222	0.0000911	-0.0001653	71.6732
5	71.6732	0.0000006	-0.0001629	71.6772

The procedure actually converged to a duration of 71.6772 min (or 71.68 min.) after five iterations using a convergence criteria of

$$\frac{F\left(t_{D_i}\right)}{F'\left(t_{D_{i+1}}\right)} < 0.5$$

Example 11.6.2

Determine the maximum detention storage for the watershed in Example 11.6.1.

Solution

The peak discharge for the duration of $t_D = 71.68$ min is

$$Q_p = C_p A \left[\frac{a}{(t_D+b)^c}\right]$$

$$= 0.85\ (15.24) \left[\frac{97.86}{(71.68 + 16.4)^{0.76}}\right]$$

$$= 42.16 \text{ cfs}$$

Using equations (11.6.5) the maximum detention storage is

$$V_S = 60\ Q_P\ t_D - 30\ Q_A\ (t_D + t_C)$$

$$= 60\ (42.16)\ (71.68) - 30\ (32.17)\ (71.68 + 21.2)$$

$$= \text{ft}^3 = \text{ac-ft}$$

$$= 91674.\ \text{ft}^3$$

Alternatively the volume of storage, V_S, needed for a detention basin is the accumulated volume of inflow minus outflow during the period when the inflow rate exceeds the outflow rate as shown in Figure 11.6.5. Using a rainfall-intensity-duration relationship of the form

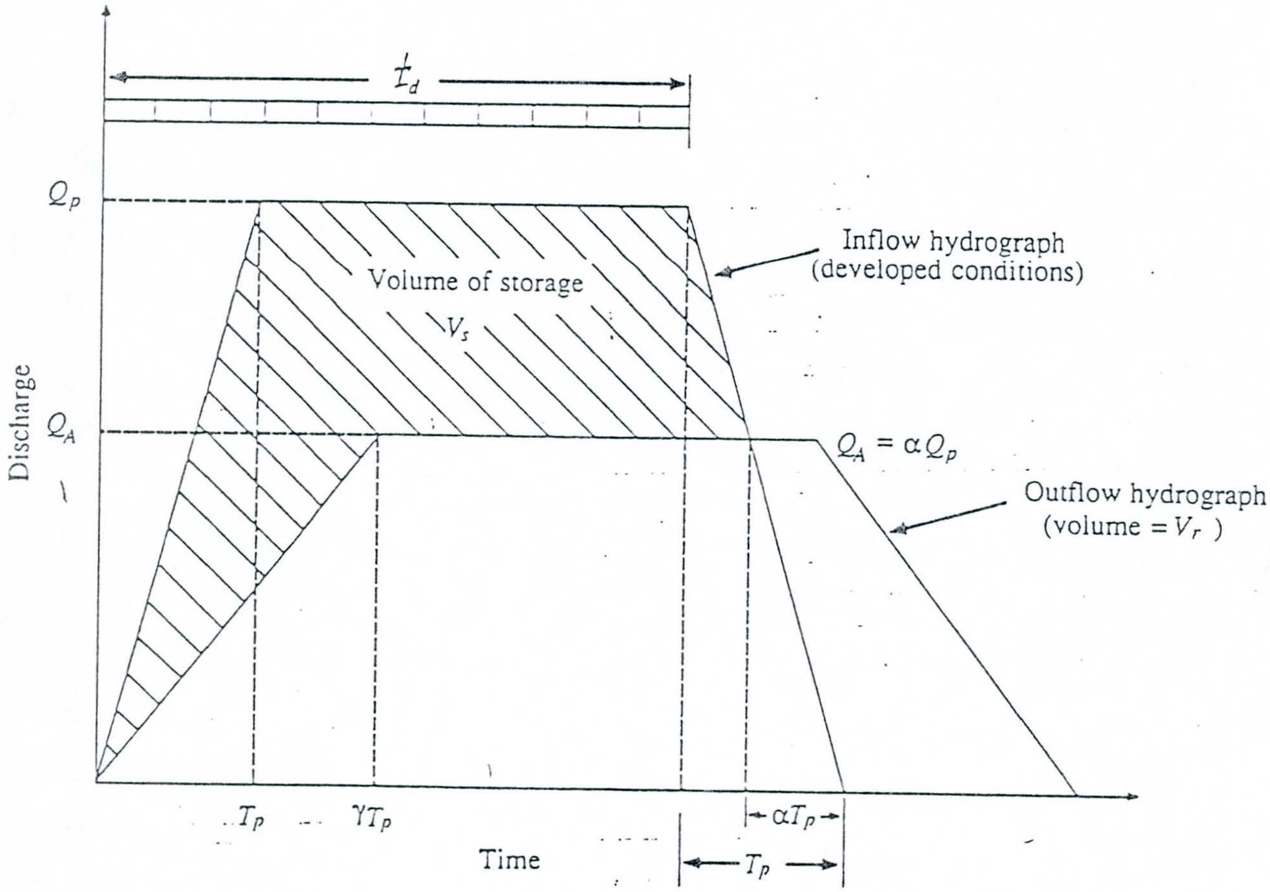


Figure 11.6.5 Inflow and outflow hydrographs for detention design. The outflow hydrograph is based on the inflow hydrograph for predeveloped conditions or on other more restrictive outflow criteria. (Source: Donahue, McCuen, and Bondelid, 1981. Used with permission.)

$$i_D = \frac{e}{t_D + f} \tag{11.6.13}$$

the volume of storage is

$$V_S = t_D Q_P - Q_A t_D - Q_A t_P + \frac{\gamma Q_A t_P}{2} + \frac{Q_A^2 T_P}{2Q_P} \tag{11.6.14}$$

The duration that results in the maximum detention is

$$t_D = \left(\frac{fcAe}{Q_A - \frac{Q_A^2 t_P}{2ce}} \right)^{1/2} - f \tag{11.6.15}$$

The time to peak, t_P, is set equal to the time of concentration.

11.6.3 Hydrograph Design Method

There are several major design determinations involved in the engineering design of stormwater detention facilities. These are: (1) the selection of a design rainfall event, (2) the volume of storage needed, (3) the maximum permitted release rate, (4) pollution control requirements and opportunities, and (5) identification of practical detention methods and techniques for the specific project.

A simple design procedure for detention basins is now outlined that is useful in practice.

1. Determine the watershed characteristics and location of the detention basin.

Kinematic Wave Routing and Computational Error

By T. V. Hromadka II[1] and J. J. DeVries[2]

Abstract: The standard kinematic wave (KW) method used in many models of open channel flow routing of runoff hydrographs in watershed models is examined as to the significance of the computational errors due to numerical-diffusion and the selection of computational effort. It is shown that a wide range of modeling results are possible from a KW model depending on the choice of computational reach length and time-step size used in the KW approximation. In comparison, the simple convex hydrologic routing method demonstrates only a small fraction of the variation in results demonstrated by the KW model. It is recommended that use of the KW method for channel routing in watershed models be reconsidered.

Introduction

Models of watershed runoff typically include a submodel for approximating the effects of unsteady flow in open channels (i.e., channel routing) for routing a runoff hydrograph through a channel reach. The various methods used to approximate the unsteady flow routing process can be grouped primarily into two categories: hydraulic routing methods which approximate the governing flow equations of continuity, energy, or momentum; or hydrologic routing methods which represent the effects of translation and channel storage on the inflow runoff hydrograph. By far, the most popular hydraulic method used in watershed models is the kinematic wave approach. One of the most popular hydrologic channel routing models is the convex method.

In this paper, the standard kinematic wave routing method is compared to the standard convex routing method such as described and employed in the HEC-1 kinematic wave (KW) program (HEC 1979) and the SCS Engineering Handbook (1972), respectively. Several watershed models use the KW method for channel routing such as used in the HEC-1 KW program and, therefore, the results of this study apply to KW programs in general. The focus of this paper is not toward the accuracy of either routing method in the approximation of flow routing effects but rather the computational errors that are associated to either method. It is shown that except for those conditions where there is no attenuation or subsidence of the runoff hydrograph peak flow rate due to channel storage effects and where the inflow hydrograph includes a mild rising and falling limb, the KW model exhibits significant computational error and numerical-diffusion effects which depend on the user-specification of the KW modeling reach

[1]Dir. of Water Resour. Engrg., Williamson and Schmid, 17782 Sky Park Blvd., Irvine, CA 92714; and Res. Assoc., Dept. of Civ. Engrg., Princeton Univ., Princeton, NJ 08544.

[2]Assoc. Dir. of the Water Resour. Ctr., Univ. of California, Davis, CA 95616.

Note. Discussion open until July 1, 1988. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on May 16, 1986. This paper is part of the *Journal of Hydraulic Engineering*, Vol. 114, No. 2, February, 1988. ©ASCE, ISSN 0733-9420/88/0002-0207/$1.00 + $.15 per page. Paper No. 22212.

length and time-step sizes. In comparison, however, the simple convex hydrologic routing method shows only a small fraction of the irregularities associated with the KW modeling results.

As a result of the identified inconsistencies, use of a KW model to approximate channel routing effects may be questionable for both hydrologic design studies where there is no model calibration, and for watershed model calibration studies where the errors in the KW channel routing models is accounted for in the watershed model by modifying the runoff hydrograph subarea parameters (e.g., modifying the overland flow-plane roughness factors).

BACKGROUND

Use of the KW channel routing technique is popular among many of the watershed models developed during the last decade. [To avoid confusion, the KW routing method is defined to be the technique described in the HEC TD-10 (1979)]. However, the literature contains several examples of KW channel routing performance that indicate that this procedure may be of limited value in comparison to other methods. For example, Akan and Yen (1981) show that their comparisons of KW routing results to the diffusion and fully dynamic computational solutions indicate that the KW peak flow-rate estimates and hydrograph timing differ significantly from the other comparable modeling results. Similar results were obtained by Katapodes and Schamber (1983) that demonstrate the significant errors developed from the KW models where the standard KW model is "corrected" for dynamic routing effects. Weinmann and Laurenson (1979) demonstrate the significant errors developed from the standard KW approach.

The source of the KW routing errors can be grouped into two categories: (1) Errors in the KW model fundamental assumptions; and (2) computational errors from the finite-difference numerical solution of the KW approach. Typically, both errors are "seen" together, and comparisons are reported in the literature that do not isolate the two sources of error. For example, Doyle et al. (1983) write that, "It has been shown repeatedly in flow-routing applications that the kinematic wave approximation always predicts a steeper wave with less dispersion and attenuation than may actually occur." Generally speaking, however, the KW does not attenuate the peak flow rate, i.e., modeled attenuation of the hydrograph peak flow rate is under most circumstances a result of the computational errors in approximating the KW flow equations. Ponce et al. (1978) write, " . . . the kinematic model, by definition, does not allow for subsidence." In consideration of solving the KW flow equations by using the method of characteristics, Strelkoff (HEC 1980) writes that " . . . kinematic waves can attenuate under certain conditions. Such attenuation is enhanced by overflow into flood plains, but can occur when kinematic shocks (as distinguished from bores) are formed in the channel at the intersection of the characteristics."

Therefore, attenuation of the hydrograph peak flow rate when using the KW technique is essentially the result of computational errors including numerical-diffusion and not due to the application of the KW flow equations. This paper focuses on the magnitude and significance of these

computational errors as produced by the well-known HEC-1 KW program. In this way, the second category of errors associated with the KW method are evaluated. The first category of KW errors (i.e., the appropriateness of the use of the KW flow equations) is essentially addressed by the statement in Li et al. (1975) regarding the limitations of the kinematic wave approximation: "local and convective accelerations must be negligible, and the water surface slope is nearly equal to the channel bed slope."

STUDY PROCEDURE

The reported difficulties in the referenced KW model were investigated during the course of a study to evaluate the accuracy of hydraulic and hydrologic channel routing models. During the course of that study, the significance of the KW computational errors were evaluated and then separately studied to identify the implications, if any, in the use of a KW channel routing model in a hydrologic model setting.

Several test cases were considered involving various rectangular channel reach lengths, slopes, friction coefficients, and base widths. In all cases, a runoff hydrograph shape typical of those anticipated for flood control studies was used. Use of a more peaked runoff hydrograph worsened the computational errors identified for the set of test cases reported in this paper.

Each test case involved a total channel length of 25,000 ft (7,620 m). Throughout the length, all channel properties are held constant. The inflow hydrograph was then routed through the channel using various (constant) channel segment (Δx) and time-step (Δt) sizes in the KW model. The convex method was then applied to the same problem conditions using identical channel segment sizes used for the KW model test, but with a constant time-step size of five min.

In the following are presented the set of test results involving the rectangular channel of 40-ft (12-m) base width, a bottom slope of 0.0010, and a Manning's friction factor of n = 0.050. In this test, the largest magnitude of computational error was noted for the set of tests considered in our study.

Typically, depending on hydrograph shape, the slower the flow velocities, the more significant the computational errors. However, for steep or peaked hydrographs, the errors were of the significance reported herein. It is repeated that the errors reported herein are due to computational errors, e.g., numerical-diffusion, and not due to the model's underlying assumptions as to hydraulics of the flow. It is also noted that although the HEC-1 KW model is used for KW modeling purposes, other similar KW models will also exhibit the properties described herein. The HEC-1 KW model is used for KW routing demonstration purposes only, and because this particular KW model is well known and is one of the most frequently used KW model programs.

CASE STUDY RESULTS

In HEC-1, the program selects Δx on the basis of Δt, or Δt is chosen on the basis of Δx. The routing reach is always divided into at least two segments, so that the maximum Δx is 1/2 the reach length. Because the finite-difference solution used in the kinematic wave routing equations

TABLE 1. 10,000-ft Channel Length KW Model Results

t (min) (1)	x (ft) (2)	Q_{peak} outflow (cfs) (3)	Time of peak (hr) (4)
1	2,000	769	2.13
2	3,333	705	2.17
3	5,000	650	2.15
4	5,000	658	2.13
5	5,000	665	2.17
6	5,000	677	2.10

introduces numerical diffusion into computational results, noticeable differences in routed hydrographs occur as Δt is varied in the reach. Table 1 gives the results of routing a hydrograph (see the inflow hydrograph shown in Fig. 2) through a 10,000-ft (3,048-m) long channel reach using various values of Δt.

From Table 1, as Δt gets smaller, the Δx value used decreases such as to satisfy the well-known Courant condition. As $\Delta t \rightarrow 0$, $\Delta x \rightarrow 0$ and Q_{peak} (outflow) = Q_{peak} (inflow) = 940 cfs (26.2 m^3/s) (see Fig. 5) where outflow and inflow indicate the corresponding runoff hydrograph values. Thus, the KW model results vary between 677–940 cfs (19.2–26.2 m^3/s) based on the selection of the model's computational effort to be used.

Fig. 1 contains KW model results for channel lengths of L = 5,000 and 10,000 ft (1,528 and 3,048 m) for two modeling attempts each. For L = 10,000 ft (3,048 m), it is seen that depending on whether Δx = 2,500 or 5,000 ft (762 or 1,724 m), Q_{peak} (outflow) is 840 cfs or 680 cfs (23.8 or 18.3 m^3/s), respectively. Again, a smaller Δx would result in a higher Q_{peak} (outflow) until the 940-cfs Q_{peak} (inflow) value is reached.

Fig. 2 shows the KW model outflow hydrographs for various channel lengths L from L = 0 ft (0 m) (i.e., the inflow hydrograph) to L = 25,000 ft (7,620 m). In all cases, Δx = 2,500 ft (762 m) and Δt = 6 min. Again, the Q_{peak} (outflow) values of Fig. 2 would raise (or lower) should a smaller (or larger) Δx value be specified in the KW model. This is demonstrated by using a Δx = 500 ft (172 m) and Δt = 2 min such as shown in Fig. 3. Comparing Figs. 2 and 3, it is seen that using more computational effort in the KW model [i.e., decreasing Δx from 2,500 to 500 ft (762 to 152 m)] increases the Q_{peak} (outflow) and also changes the hydrograph shape and time-to-peak.

Fig. 4 summarizes the KW modeling results for the total channel length of 25,000 ft (7,620 m). From the figure it is seen that depending on whether Δx = 2,500 ft or 8,333 ft (762 or 2,540 m), Q_{peak} (outflow) = 640 or 400 cfs (48.1 or 11.3 m^3/s), respectively. Recalling the Fig. 3 value for L = 25,000 ft (7,620 m) using Δx = 500 ft (152 m), Q_{peak} (outflow) = 800 cfs (22.7 m^3/s). Again, use of still smaller Δx would increase Q_{peak} (outflow) to the 940 cfs Q_{peak} (inflow) value.

Should the HEC-1 KW model user input Δt, the results of the L = 25,000 ft (7,620 m) case study vary according to Fig. 5. Again, as $\Delta t \rightarrow 0$, then $\Delta x \rightarrow 0$ and Q_{peak} (outflow) $\rightarrow$ Q_{peak} (inflow).

Fig. 6 shows the HEC-1 KW channel routing Q_{peak} (outflow) values for

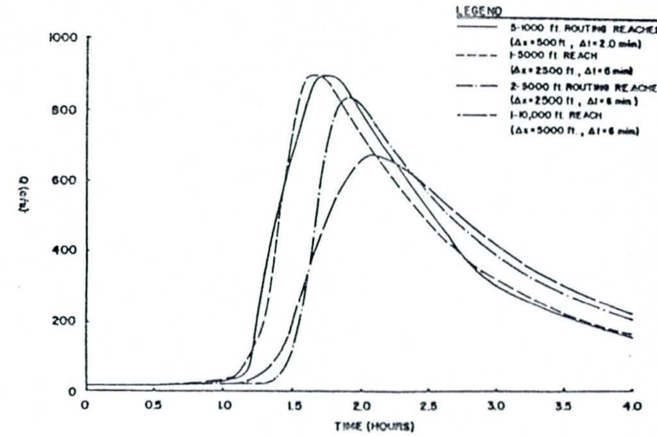


FIG. 1. KW Outflow Hydrographs for L = 5,000 ft and 10,000 ft (Inflow Hydrograph Shown In Fig. 2)

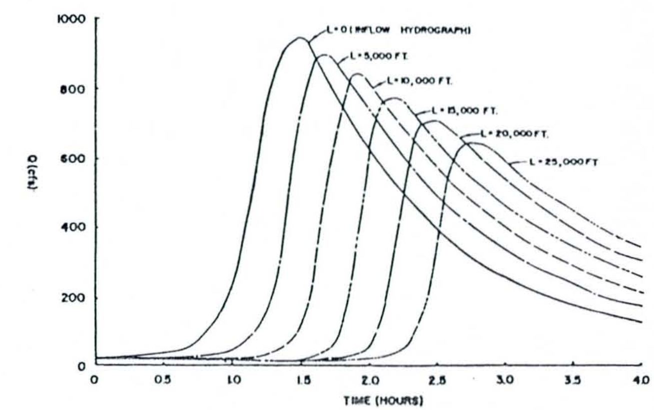


FIG. 2. Outflow Hydrographs for Δx = 2,500 ft, Δt = 6 min for Various Channel Lengths (L)

various L lengths and for an input Δt value of 6 min. Recalling that Q_{peak} (inflow) = 940 cfs (26.2 m^3/s), the shaded area shown in Fig. 6 is the KW Q_{peak} (outflow) values possible depending on the Δx value chosen.

The convex routing model was also used to approximate the unsteady flow problems attempted by the KW model. Typically, the convex model performed most "poorly" when the KW model did and, therefore, examination of the computational error for the same set of test problems described for the KW model is appropriate. Because the convex model demonstrated only a small fraction of the variation in results that the KW model demonstrated, the convex modeling results are shown in table form.

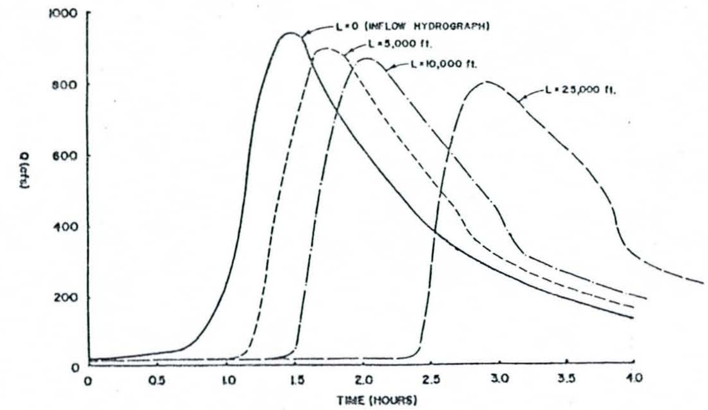


FIG. 3. Using Δx = 500 ft and Δt = 2 min in KW Model Test of Fig. 2

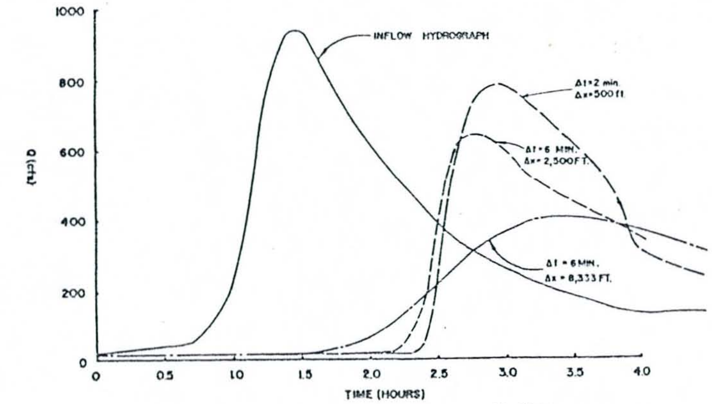


FIG. 4. KW Results for L = 25,000 ft

In Table 2 are contained the Q_{peak} (outflow) values from use of the convex model for the inflow hydrograph of Fig. 2 and for various values of L. Three cases are considered for Δx values; namely, $Q1$ values indicate three reaches composed of two 10,000-ft (3,048-m) lengths and one 5,000-ft (1,524-m) length; $Q2$ values indicate five 5,000-ft (1,524-m) lengths; and $Q3$ values indicate twenty-five 1,000-ft (305-m) lengths. For all tests, a Δt of 5 min was used. Also included in Table 2 is an additional convex test case for a different set of channel conditions that results in considerably higher channel-flow velocities. It is readily seen that after 25,000 ft (7,620 m), the convex routing method involves computational errors due to the selection of Δx values of the order of 5%.

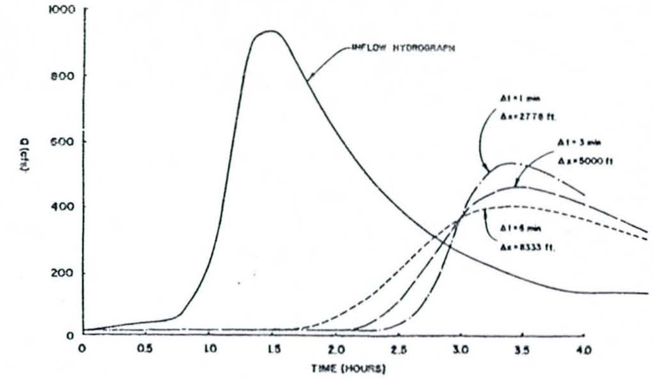


FIG. 5. Effect of Δt Input in HEC-1 KW Model (L = 25,000 ft) (as Selected by HEC-1 Program)

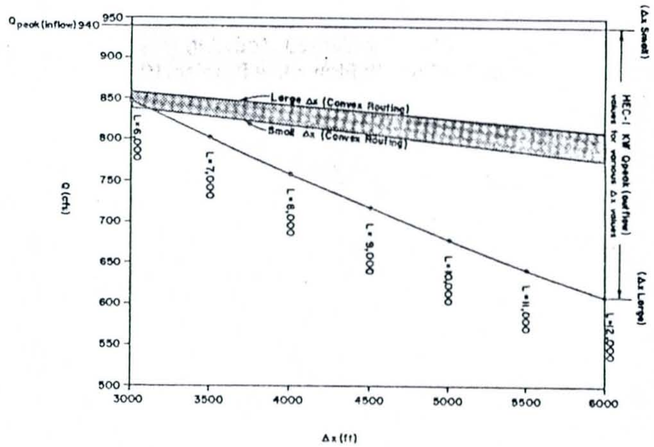


FIG. 6. Variation in KW and Convex Method Modeling Results of Q_{peak} (Outflow) for Various L Values from 6,000–12,000 ft

Fig. 7 shows the range of computational results from the HEC-1 KW model (where the program selects the computational parameters) and the convex routing method (for a constant time step of five min). The illustrated range of channel lengths vary from 0–25,000 ft (7,620 m). From the figure, the convex method shows a variation of 5%. In contrast, the KW model shows a variation of over 130% for L = 25,000 ft (7,620 m) depending on the Δx values selected.

Fig. 8 compares the KW-produced range of results and the convex routing results for the fast-flow problem of Table 2.

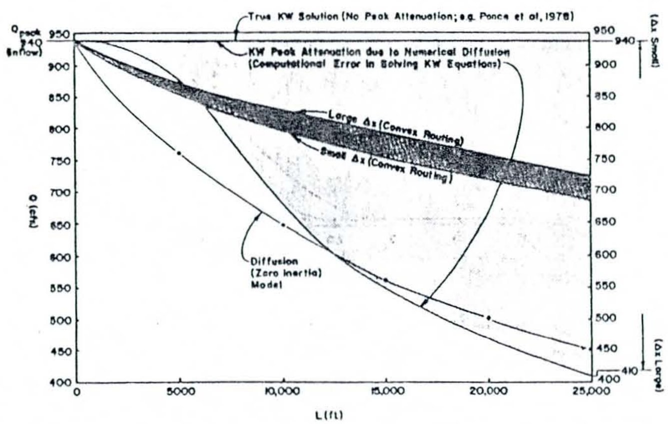


FIG. 7. Variation in KW and Convex Method Modeling Results of Q_{peak} (Outflow) for Various L Values from 0–25,000 ft; Slow-Flow Problem (Diffusion Model Results Shown by *)

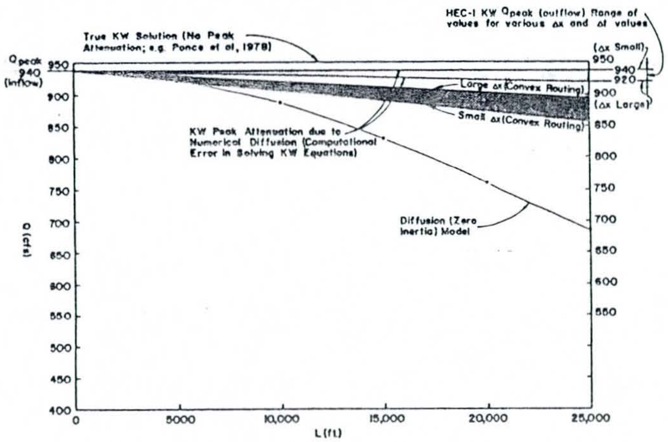


FIG. 8. Variation in KW and Convex Method Modeling Results of Q_{peak} (Outflow) for Various L Values from 0–25,000 ft; Fast-Flow Problem (Diffusion Method Results Shown by *)

TABLE 2. Convex Model and Diffusion Model Q_{peak} (Outflow) Results

	SLOW-FLOW B = 40 ft; So = 0.001 ft; n = 0.050				FAST-FLOW B = 10 ft; So = 0.010 ft; n = 0.015			
	Convex				Convex			
L (ft) (1)	$Q1$ (2)	$Q2$ (3)	$Q3$ (4)	Diffusion (5)	$Q1$ (6)	$Q2$ (7)	$Q3$ (8)	Diffusion (9)
1,000			922				938	
2,000			903				935	
3,000			332				931	
4,000			869				926	
5,000		858	855	761		935	920	922
6,000			838				920	
7,000			828				918	
8,000			817				915	
9,000			804				910	
10,000	831	795	795	647	929	930	904	885
11,000			786				904	
12,000			775				903	
13,000			768				900	
14,000			760				895	
15,000		750	751	560		925	889	829
16,000			745				889	
17,000			737				888	
18,000			730				885	
19,000			725				881	
20,000	757	716	717	500	910	921	875	757
21,000			712				875	
22,000			706				874	
23,000			699				871	
24,000			695				867	
25,000	721	689	689	450	907	916	862	627

Comparison to Diffusion (Zero Inertia) Model

The next level of sophistication above the KW technique is the zero inertia or the diffusion routing method. Akan and Yen (1981), Tingsanchali and Manandhar (1985), Katopodes and Schamber (1983), Ponce et al. (1978), Weinmann and Laurenson (1979), Li et al. (1975), and Doyle et al. (1983), among others, have shown the significant improvement in computational accuracy using the diffusion analog in comparison to the KW technique.

Included in Table 2 are peak flow-rate values at 500-ft intervals obtained from a one-dimensional diffusion model of the test inflow hydrograph for both the considered slow-flow and fast-flow problems. The diffusion model results are also plotted on Figs. 7 and 8.

From Fig. 7, it appears that the lower curve of values associated to the KW approximation are close to the diffusion modeling results. However, it must be remembered these KW results are strictly due to the algorithmic errors (numerical-diffusion) in solving the KW equations. Had the KW equations been solved exactly, then the top line [i.e., a constant Q = 940 cfs (26.6 m^3/s)] would be the KW modeling results.

Analysis of Results

From the preceding results it is seen that the arbitrary use of the KW method to model unsteady flow in open channels is subject to considerable scrutiny due to the potentially wide variation in results possible by the selection of Δx or Δt values. This "range of results" impacts the very credibility in using KW or channel routing hydrologic models. A possible remedy in using the standard KW approach (such as in HEC-1) may be to require that all users choose Δx values sufficiently small as to guarantee a good solution of the KW assumption, but in that case, Q_{peak} (outflow) = Q_{peak} (inflow) due the lack of subsidence of the peak flow-rate fundamental to the KW formulation. But many channel routing conditions do exhibit peak attenuation due to channel storage effects, and, therefore, use of the KW would contradict the fundamental channel routing characteristics. Possibly, KW should only be used when there is negligible peak attenuation in the channel. In that case, simple hydrograph translation would be a simpler method to use than KW.

The convex routing method, on the other hand, is simple to apply, does not demonstrate the computational deficiencies to the magnitude exhibited by the HEC-1 KW model, contains peak attenuation, and performs translation for high velocity flows.

Based on the observed computational errors of the KW channel routing method, the limitations fundamental to use of the KW method, and the computational effort needed to approach a true KW hydrograph routing approximation, we submit that use of the KW method for channel routing needs a re-evaluation for use in hydrologic models unless guidelines are developed to control the arbitrary use of KW in design studies.

It is not implied by this study that the simple convex routing technique should be used as the standard flow routing method, but rather that the uncertainty in the selection of KW discretization values for space and time needs further attention from the program developers. Even though the KW technique is conceptually more physically based than the convex method and can potentially achieve the "correct" routing effects, the typical general purpose computer program does not provide internal computational checks to optimize the time step and spatial increment sizes to achieve this "correct" solution. Indeed, the typical goal of most canned-program users is to simply achieve a successful run of the computer program. With the demonstrated range of results possible from a widely used program (i.e., HEC-1) based on KW, a policy statement regarding use of such programs should indicate supplemental procedures required to reach to the "correct" solution. Otherwise, such a policy statement may need to eliminate the use of routing techniques such as KW in favor of a crude convex approximation simply due to its reproducibility by the average practicing floor control engineer in industry and local government agencies involved with flood control planning.

Conclusions

The HEC-1 KW model is studied to evaluate the significance of computational errors due to the choice of the computational effort used to approximate the unsteady flow effects in channel routing. It is shown that the selection of Δx and Δt values may have a significant impact on the KW

modeling results, and that the simple convex hydrologic routing method demonstrates but a small fraction of the variations in results demonstrated by the KW model used. It is recommended that hydrologic models that use the standard KW method for channel routing be re-evaluated as to their credibility and reliability in their use in the typical flood control design setting of practicing engineers. Guidelines are needed in KW routing models in order to eliminate the possible range of values due to computation error, or KW channel routing programs need internal checks to select Δx and Δt such that an accurate solution of the KW equation is achieved. KW programs also need internal checks to notify program users when the KW flow equations may be inappropriate due to channel storage effects becoming significant.

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Kinematic Wave Routing and Computational Error[a]

Discussion by David R. Dawdy,[3] Associate Member, ASCE

The authors were made aware that their results were incorrect in December 1986 (Dawdy and Saluja 1986).

The kinematic wave is not recommended for channel routing to model attenuation of the flood wave, which it does not do when properly used. Kinematic wave routing is recommended for use in basin modeling for urban master planning. The difference in timing of the flows from different parts of the urban basin is more important than attenuation of the flood wave. Most of the authors' paper is not germane to the intended use of kinematic wave models in basin modeling for urban planning and flood simulation.

The Courant condition should be understood for kinematic wave models to be used properly. The authors neither explain the Courant condition nor give an understanding of its implications. The Courant condition requires that

$$\frac{\Delta x}{\Delta t} = \alpha m A^{m-1} \quad (1)$$

which for the case used by the authors gives

$$A = \left(\frac{940}{0.0806}\right)^{0.6} = 275.5 \quad (2)$$

$$\Delta x = 300\,(0.0806)(1.667A^{0.667}) = 1{,}710 \text{ ft } (520 \text{ m}) \quad (3)$$

With 5 min fixed as the time interval, the routing interval should be approximately 1,710 ft (520 m). Table 5 shows the results for the rectangular channel chosen by the authors. The attenuation of the peak is only 1/2% in 10,000 ft (3,050 m). Part of that may be interpolation error. Table 1 should have been constructed as Table 6.

Table 6 demonstrates that various values of Δt can be used, if the Courant condition is met and the proper Δx is chosen. In the hands of a skilled modeler the kinematic wave model does what it is supposed to do.

An example of incorrect use of the kinematic wave solution is the example in the *HEC-1 Training Document No. 10* (DeVries 1979). That example uses a 5-min interval for Δt. The sensitivity of variation of computed peak discharge to Δx and Δt is shown in Table 7.

In conclusion, the kinematic wave is a good method for use in models for urban design and planning. The small basin sizes and the often fairly steeper prismatic channels in the urban environment cause the major attenuation of the hydrograph to result from the timing of the flows from the various parts of the basin. One advantage of the kinematic wave method is that it can be developed with little or no streamflow data.

Any model in the wrong hands can result in misuse and incorrect answers. This paper illustrates that point. The problem has been exacerbated by the wide use of computers. The inexperienced hydrologist can still make mistakes in hydrology, but can now also have the hydrology correct and mess up with a computer application. The mathematically naive can develop answers that common sense says are wrong. No matter how much the results of mathematical naïvete are to be deplored, the method should not be blamed. Properly used, the tool of kinematic wave routing is a powerful one.

[a]February, 1988, Vol. 114, No. 2, by T. V. Hromadka II and J. J. DeVries (Paper 22212).

[3]Consulting Hydrologist, 3055 23rd Ave., San Francisco, CA 94132.

TABLE 5. Result of Routing 940-cfs Flood Peak down 40-ft- (12-m-) Wide Rectangular Channel with Slope 0.001 and n = 0.05

Reach (1)	Distance (ft) (2)	Peak (cfs) (3)	Time of peak (4)
0	0	940	1:15
1	1,667	939	1:20
2	3,333	938	1:25
3	5,000	937	1:30
4	6,667	937	1:35
5	8,333	936	1:40
6	10,000	935	1:45

TABLE 6. Relation of Δt to Δx for Proper Kinematic Wave Channel Routing

Δt (min) (1)	Δx_* (ft) (2)	Δx (ft) (3)	N_x (4)	Q (cfs) (5)	Δt_m (min) (6)
1	341	333–334	30	937	5
2	683	666–667	15	935	5
3	1,024	1,000	10	935	5
4	1,366	1,250	8	927	5
5	1,707	1,666–1,667	6	935	5
6	2,048	2,000	5	936	5
10	3,414	3,333–3,334	3	939	10

TABLE 7. Demonstration of Incorrect Choice of Δx and Δt in DeVries (1979)

Δt (min) (1)	Nx (2)	$Q1$ (3)	$C1$ (4)	$\Delta x/\Delta t1$ (5)	$Q2$ (6)	$C2$ (7)	$\Delta x/\Delta t2$ (8)	% increase (9)
5	2	2,017	10	2.9	3,527	15	3.3	0
4	2	2,054	10	3.6	3,654	15	4.2	3.6
3	2	2,064	10	4.9	3,759	15	5.6	6.6
2	2	2,092	10	7.3	3,961	15	8.3	12.3
1	2	2,033	10	14.6	3,749	15	16.7	6.3
1	4	2,107	10	7.3	3,968	15	8.3	12.5

Appendix I. References

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Appendix II. Notation

The following symbols are used in this discussion:

N_x = number of length subdivisions used in solution;
Q = discharge;
α, m = parameters of kinematic wave equation;
Δt = time increment used in solution;
Δt_{in} = input time increment;
Δx = reach length used in solution; and
Δx_* = optimal reach length.

Discussion by David Goldman,[4] Associate Member, ASCE

Introduction

The authors have provided some useful comments on an error in the Hydrologic Engineering Center (HEC)–1 kinematic wave (KW) channel routing scheme. However, their routing example misrepresents the usefulness of the KW approach.

HEC-1 Kinematic Wave Routing

The current version of HEC-1 (January 1985) does not properly reduce the finite difference distance, *DX*, increment for a given time step, *DT*, to obtain a reasonable solution to the kinematic-wave equations. A new version of HEC-1 (July 1988) will automatically select a reasonable value of *DX* and *DT* to obtain a solution of the KW equations with a scheme proposed by Leclerc and Schaake (1973).

Kinematic Wave Modeling in Headwater Watershed Models

The authors have chosen an example that is unrepresentative of the problem facing most drainage engineers and that shows the KW technique at its worst. Their example is for a 5-mi- (8-km-) long channel, where the absence of lateral inflow and the length of channel causes a KW shock to form. This is not typical of most urban drainage problems, where there is lateral inflow for short channel length. The presence of the shock is apparent in the characteristic solution shown in Table 8 at a distance of 20,000 ft (6.1 km).

A comparison is given in Table 8 of the characteristic and finite difference solution obtained with the new version of HEC-1 for the authors' example. Two different finite difference solutions are shown, one using the automatically selected *DX* and *DT*, and another with a user-defined *DX* and program-selected *DT*. The solutions compare well with the method of characteristics, at least until the shock forms. (Notice that there is some inconsistency between the discharges at 5,000 and 10,000 ft for program-selected *DX* and *DT*. This resulted because individual channels had to be used to obtain flows

[4]Hydr. Engr., Hydrol. Engrg. Ctr., 609 2nd St., Davis, CA 95616.

TABLE 8. Reproduction of Table 2 Using New Example

Distance (ft) (1)	KINEMATIC WAVE[a]: HEC-1 Finite Difference Solution				KINEMATIC WAVE: Characteristic Solution		DIFFUSION: Cell Integrated Solution			
	DX = 330 ft[b] (Q) (cfs) (2)	DT = 1.4 min (TP) (hr) (3)	DX = 125 ft[c] (Q) (cfs) (4)	DT = 0.5 min (TP) (hr) (5)	DX = 250 ft (Q) (cfs) (6)	(TP) (hr) (7)	DX = 250 ft[d] (Q) (cfs) (8)	DT = 3 min (TP) (hr) (9)	DX = 250 ft[e] (Q) (cfs) (10)	DT = 3 min (TP) (hr) (11)
5,000	939	1.8	947	1.8	950	1.8	758	1.8	757	1.8
10,000	944	2.2	946	2.2	950	2.2	651	2.2	650	2.2
15,000	940	2.5	945	2.5	950	2.5	575	2.6	575	2.6
20,000	933	2.8	932	2.8	913	2.8	525	3.1	521	3.1
25,000	928	3.2	932	3.2	829	3.2	489	3.6	500	3.6

[a]Kinematic wave used five channels at lengths shown.
[b]*DX*, *DT* chosen by program, values of *DX*, *DT* are average for all channel lengths.
[c]*DX* specified by user.
[d]Normal depth downstream BC, discharge BC upstream.
[e]Critical depth downstream BC, discharge BC upstream.

at different distances. The automatically selected *DX* and *DT* values were somewhat different for each channel.)

The authors are not correct in saying that numerical diffusion is a significant part of KW routing schemes, at least when the scheme is applied properly. The presence of a kinematic shock will cause some problems for a finite difference scheme, but this situation should not occur for problems where the KW method is of interest.

Diffusion Routing

Presumably, the authors presented diffusion routing results to show their superiority for flood routing. Certainly, for a single channel, with specified boundary conditions, the diffusion model will in general produce a more reasonable hydrograph then a KW model. In their diffusion routing example (Table 2), they do not specify the boundary conditions assumed. In Table 1, the difference between a cell-integrated numerical solution of the diffusion equations (Strelkoff 1982) for normal depth and critical depth boundary conditions is shown to be small for their example. However, would this difference always be small independent of reach length? Before diffusion models are used in a headwater model, like HEC-1, one probably should evaluate the impact of the assumed boundary conditions for each reach length on the hydrographs routed through the whole watershed.

Conclusion

The authors are correct that the KW channel routing solution in the January 1985 version of HEC-1 needs improvement. This error has been corrected in the July 1988 version. The KW method is most useful in quick-responding (urban) headwater watershed situations where lateral inflows and short channels prevent the formation of the kinematic shock.

The authors are mistaken in claiming that the KW technique should be reevaluated for drainage design. Furthermore, they do not offer the useful simple alternative that they claim should be used instead.

Appendix. References

Leclerc, G., and Schaake, J. J., Jr. (1973). "Methodology for assessing the potential impact of urban development on runoff control alternatives." *Dept. No. 167*, Ralph M. Parsons Lab., Massachusetts Inst. of Tech., Cambridge, Mass.

Strelkoff, T. (1982). *A computer program for performing numerical experiments in comparative flood routing*. Hydrol. Engrg. Ctr., Davis, Calif.

Discussion by William H. Merkel[5]

The authors have demonstrated the negative aspect of finite difference solutions to the kinematic wave (KW) equations very well. The writer agrees with the observations that the finite difference solution exhibits sensitivity to the selection of Δx and Δt. A very wide range of routing results is ob-

[5]Hydr. Engr., U.S. Soil Conservation Service, Engrg. Div., P.O. Box 2890, Washington, DC 20013.

TABLE 9. Illustration of Convex-Routing Characteristic

Δt (hours) (inflow) (1)	Peak outflow [cfs (cm)] (2)	Percent of peak (3)
0.1	357 (10.1)	91.5
0.2	332 (9.4)	85.1
0.3	315 (8.9)	80.8
0.4	301 (8.5)	77.2
0.5	295 (8.35)	75.6

tained by varying Δx and Δt. These results diverge from the results of the solution by the method of characteristics (Eagleson 1970). In the absence of kinematic shock, the solution by the method of characteristics (MOC) indicates no attenuation and steepening of the wave front. The MOC is a solution technique that rigorously satisfies the KW mathematical model.

The authors have overlooked a major problem associated with the Soil Conservation Service (SCS) convex routing procedure. The convex routing is not very sensitive to the Δx selection by the user as indicated by the authors. However, it is sensitive to variation in Δt (also selected by the user). The basic computation steps involved in the SCS convex routing procedure are: (1) Compute the routing distance increment based on a flow velocity and user selected Δt; and (2) route the inflow hydrograph through the reach at steps equal to this distance increment. Thus, by changing Δt, the number of routing steps is changed. If Δt is short, then Δx is also short which results in less attenuation of the peak discharge. Conversely, if Δt is long, more attenuation is computed.

This characteristic of the convex routing is illustrated in Table 9.

The Soil Conservation Service (SCS) TR-20 computer program ("Computer Program" 1965) was used to route an inflow hydrograph through the reach at different Δts.

Basic data used to develop the information include drainage area equal to 1 sq mi (2.6 km^2), time of concentration of 2 hr, runoff curve number of 80, type II rainfall of 4 in. (102 mm), reach length equal to 200 ft (2,195 m), and flow velocity of 1.5 ft/sec (0.46 m/s).

The values used to develop Table 9 are all within the range of field applications. Increasing Δt over 0.5 hr and decreasing Δt less than 0.1 hr will widen the range of results. Theoretically, as Δt approaches zero, the peak outflow will approach the peak inflow.

Because of this sensitivity to Δt and the large range in results, SCS replaced the convex routing procedure with the modified Att-Kin routing procedure ("Computer Program" 1983). The modified Att-Kin procedure is not sensitive to changes of Δt, but it is sensitive to the reach length selected by the user.

In the interim, SCS has been investigating linear and variable parameter diffusion models (Ponce 1978, 1979). Routing coefficients are based on the Δx and Δt selected by the user and the models do not exhibit significant sensitivity.

Sufficient computer technology and numerical techniques are available to evaluate the accuracy and applicability of approximate flood routing models. Several nondimensional parameters have been developed that indicate the applicability of approximate flood routing models ("Computer Program" 1983; Ponce et al. 1978).

The SCS is currently comparing selected approximate flood routing models with a full dynamic solution of the routing problem. The linear and variable parameter diffusion models show very promising results for conditions present in many SCS flood routing applications. These models have the potential to replace both the convex and modified Att-Kin procedures for SCS use.

Appendix. References

Eagleson, P. S. (1970). *Dynamic hydrology*. McGraw-Hill Book Co., New York, N.Y.

"Computer program for project formulation—hydrology." (1965). *Technical Release 20*, U.S. Soil Conservation Service, Washington, D.C., May.

"Computer program for project formulation—hydrology." (1983). *Draft Technical Release 20*, U.S. Soil Conservation Service, Washington, D.C., May.

Ponce, V. M. (1978). "Muskingum-Cunge method with variable parameters." *J. Hydr. Div.*, ASCE, 104(12).

Ponce, V. M. (1979). "Simplified Muskingum routing equation." *J. Hydr. Div.*, ASCE, 105(1).

Ponce, V. M., Li, R.-M., and Simons D. B. (1978). "Applicability of kinematic and diffusion models." *J. Hydr. Div.*, ASCE, 104(3).

Discussion by Carl L. Unkrich[6] and David A. Woolhiser,[7] Member, ASCE

The authors used the HEC-1 computer program to evaluate the "standard kinematic wave (finite difference) routing method" for several open channel test cases. They reported mainly on the slow-flow case, and concluded that the method is far too sensitive to the choice of computational increments. However, two major assumptions are incorrect: (1) The slow-flow case, contrary to the authors' claim, does indeed attenuate due to kinematic shock; and (2) not all kinematic wave finite-difference algorithms are as sensitive to the choice of dx and/or dt as HEC-1. To illustrate the second point, the Kineros program (Smith 1981), which uses a four-point implicit numerical scheme with centered time differences and a weighting factor of 0.8 for the space differences at the advance time step, was used to simulate the same slow-flow case. Solutions were obtained for the same combinations of dx and dt as shown in Fig. 4. These solutions are shown in Fig. 9 along with the HEC-1 solution for dt = 6 min and dx = 8,333 ft (2,540 m). The Kineros solution for dt = 2 min and dx = 500 ft (152 m) plots virtually on top of the partially analytic solution obtained by the method of characteristics with a shock following scheme. It is clear that the finite-difference scheme in Kineros exhibits much less numerical diffusion than that in HEC-1 and that with a reasonable number of dx increments (>10) it provides quite acceptable numerical results.

The ranges of peak discharge obtained from Kineros for dx ranging from 1,000 to 8,333 ft (3.5 to 2,540 m) and dt = 5 min are shown in Fig. 10

[6]Hydrol. Asst., Univ. of Arizona, Tucson, AZ 85721.

[7]Res. Hydr. Engr., U.S. Dept. of Agric., Agric. Res. Service, 2000 E. Allen Road, Tucson, AZ 85719.

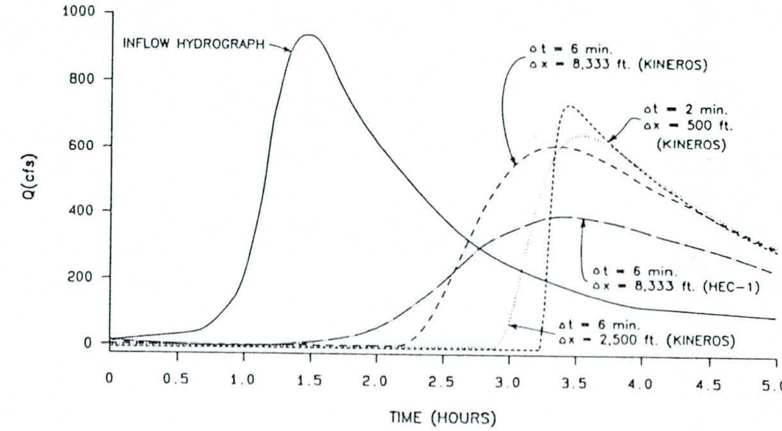


FIG. 9. Kineros Results for L = 25,000 ft (7,620 m)

along with the range for the convex method and a line showing the true KW solution (obtained by the method of characteristics with a shock following scheme). The resulting variation for peak flow rates is 8% at L = 25,000 ft (7,620 m) compared with the 130% (really 80% because the authors incorrectly assumed that there would be no attenuation) for HEC-1.

It should be noted that Kineros has built-in limits on the number of spatial increments (which were overridden for this study). The size of dx increments is based on an input "characteristic length" for the watershed defined as the length of the longest cascade of overland flow planes or the longest channel, whichever is greater. This characteristic length is divided into 15 dx increments and shorter elements have proportionally fewer, with a minimum of five dx increments. Therefore the allowable range of dx increments for Kineros is from 1,786 to 6,250 ft (544 to 1,905 m) for a 25,000-ft (7,620-m)

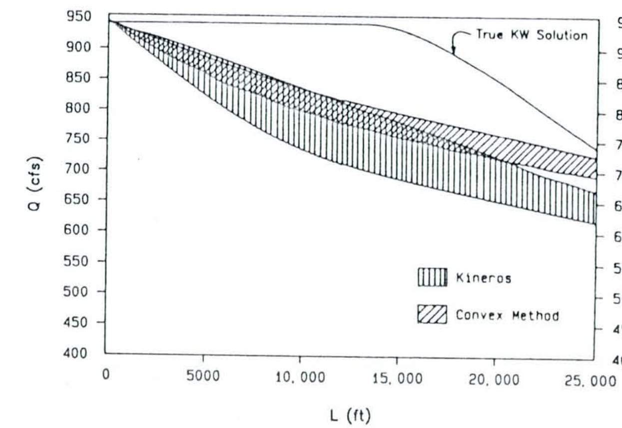


FIG. 10. Variation in Kineros and Convex Method Modeling Results of Qp for Various L Values from 0 to 25,000 ft (7,620 m)—Slow-Flow Case

reach. For a dt of 5 min, which is sufficient to define the inflow hydrograph, the range of Qp was 626–666 cfs (17.7–18.9 m^3/s), a variation of only 6.4%.

The authors' choice of references regarding KW channel routing performance is highly selective and may lead to incorrect conclusions by the reader. For example, the cases examined by Akan and Yen (1981) involved backwater conditions so it is not surprising that KW routing procedures did not work well. Katapodes and Schamber (1983) were investigating dam-break problems that lead to kinematic shocks. Since it is precisely in the vicinity of shocks that the kinematic approximation breaks down, KW routing would normally not be recommended for this problem. Weinmann and Laurenson (1979) point out that for slowly rising hydrographs and moderately steep slopes KW routing will give results that compare favorably with solutions of the St. Venant equations. Ferrick (1985) provided a comprehensive analysis of river wave types, developed a set of scaling parameters and used case studies to define the appropriate scaling parameter range for each wave type. His criteria should provide useful guidelines for choosing the appropriate approximation for a given channel.

Appendix. References

Ferrick, M. G. (1985). "Analysis of river wave types." *Water Resour. Res.*, 21(2), 209–220.

Smith, R. E. (1981). "A kinematic model for surface mine sediment yield." *Trans.*, Am. Soc. of Agric. Engrs., 24(6), 1508–1514.

Discussion by David A. Woolhiser,[8] Member, and David C. Goodrich,[9] Student Member, ASCE

Although we agree with the authors' conclusion that there is a need for better guidelines for choosing Δx and Δt in some kinematic wave (KW) routing models and for internal checks regarding the appropriateness of the KW formulation from the physical point of view, we find that their analysis is misleading. An analysis of their "slow-flow" case, which leads to the information shown in Figs. 1–7, reveals that according to current criteria the (KW) model should not be used for this case. Ponce et al. (1978) showed that for 95% accuracy of the kinematic wave solution after one propagation period, the dimensionless period $\hat{\tau}$ should be greater than 171. This translates into

$$T \geq \frac{171 d_0}{(U_0 S_0)} \quad (4)$$

where T = the wave period of the perturbation to steady uniform flow; U_0 = the steady velocity; d_0 = the steady depth; and S_0 is the slope. If we relate U_0 and d_0 to the mean variables at the upper boundary, we find that T should

[8]Res. Hydr. Engr., U.S. Dept. of Agric., Agric. Res. Service, 2000 E. Allen Road, Tucson, AZ 85719.

[9]Grad. Student, Dept. of Hydrol. and Water Resour., Univ. of Arizona, Tucson, AZ 85721.

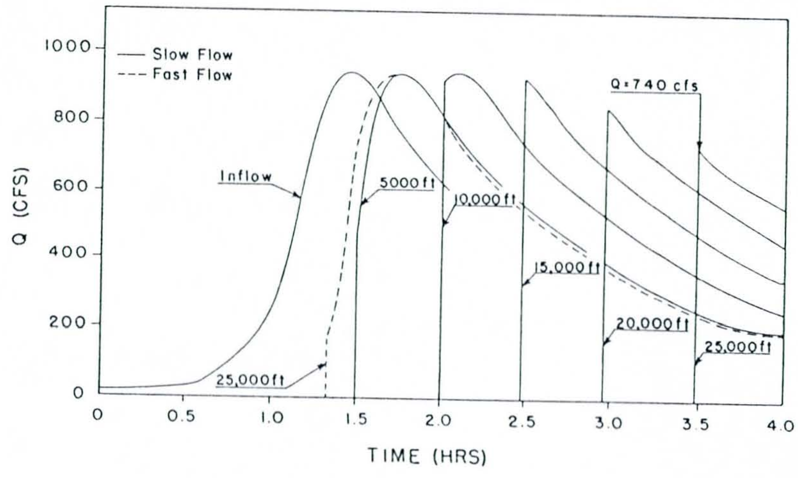


FIG. 11. Characteristic Solutions for Hydrographs

be greater than three days, yet the period of input at the upper boundary is approximately 4 hr. The "fast-flow" case meets the preceding criterion and we see that the numerical errors introduced by the KW model are in fact smaller than those in the convex method (Fig. 8).

The authors state that they are evaluating only numerical errors introduced by the HEC-1 KW program rather than those due to the KW fundamental assumptions. Yet by using an example that violates the fundamental assumptions they leave the reader with the impression that the numerical errors are more serious than they really are. If the finite difference equations in the HEC-1 model are expanded in Taylor series, we find, for example, that the error terms for the conservation form are $(\Delta x/2)(\partial^2 Q/\partial x^2) + \Delta t(\partial^2 Q/\partial x \partial t) + (\Delta t/2)(\partial^2 A/\partial t^2) + O(\Delta x)^2$.

If the Courant condition is exactly satisfied, this scheme can give exact results, but in general it is of first-order accuracy and is more dispersive than some alternative schemes. It is worth noting that for flows meeting the criteria for kinematic flow, the second-derivative terms are very small over most of the solution domain and, if reasonable Δx and Δt increments are chosen, this finite difference scheme will give quite accurate results.

When performing an empirical examination of the accuracy of rectangular grid finite-difference schemes it is always wise to have a more accurate solution for comparison. Both examples used will lead to a kinematic shock emanating from $x = 0$, $t = 0$, and traversing the channel with a shock velocity equal to the local velocity. Kinematic characteristics will be straight lines emanating from the line $x = 0$ and some will intersect the shock front. A numerical shock following scheme similar to that used by Kibler and Woolhiser (1970) was used to develop accurate hydrographs for both examples. Discharge hydrographs at various distances along the channel are shown in Fig. 11. For the slow flow case, the hydrograph peak overtakes the shock front at x = 12,490 ft (3,797 m), so for this case the peak does attenuate due to the peak overtaking the shock. Therefore, the line shown in Fig. 7 as the true kinematic solution is incorrect after that distance and

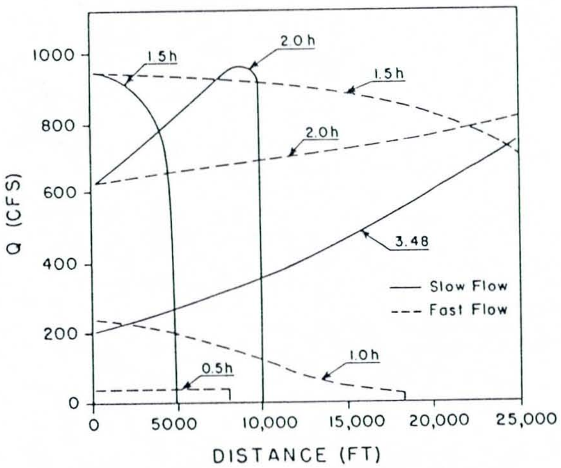


FIG. 12. Discharge Profiles at Various Times

the variation at 25,000 ft (7,600 m) is 80% rather than 130%. Discharge profiles at various times for both cases are shown in Fig. 12. An examination of this figure provides more insight into the numerical dispersion effects of the finite-difference scheme. Numerical diffusion is significant in the vicinity of the shock fronts where $\partial^2 Q/\partial x^2$ is large. Although a shock front forms for the fast flow case, it is small and does not affect the peak discharge. Numerical accuracy will be very good for this case because the second derivative terms are small over most of the solution domain.

Although the authors restrict their analysis to channels, it should be emphasized that kinematic modeling does an excllent job for overland flow and for channels with overland lateral inflow, provided adequate numerical models are used. For channels dominated by lateral inflow, the wavelength will be larger than the channel length, and criteria developed for overland flow (Woolhiser and Liggett 1967; Morris and Woolhiser 1980) are appropriate to determine if the kinematic wave formulation is adequate.

Appendix. References

Kibler, D. F., and Woolhiser, D. A. (1970). "The kinematic cascade as a hydrologic model." *Hydrology Paper No. 39,* Colorado State Univ., Fort Collins, Colo.

Morris, E. M., and Woolhiser, D. A. (1980). "Unsteady one-dimensional flow over a plane; partial equilibrium and recession hydrographs." *Water Resour. Res.*, 16(2), 355–360.

Woolhiser, D. A., and Liggett, J. A. (1967). "Unsteady, one-dimensional flow over a plane—the rising hydrograph." *Water Resour. Res.*, 3(3), 753–771.

Closure by T. V. Hromadka II[10] and J. J. DeVries[11]

The authors appreciate the time and effort spent by the many individuals who provided discussions on this paper. Their contributions will further enlighten the journal's audience as to the levels of imprecision that exist in the considered routing techniques, and the lack of agreement among hydrologists as to any method that should be used.

The authors pointed out in the paper that the focus of the paper is not directed toward defining the accuracy of either routing method (i.e., kinematic wave or convex routing), but rather toward illustrating the computational errors that are associated with either method. The comments further demonstrate the range of results made possible by either routing method. In fact, the Soil Conservation Service has modified the kinematic wave method through the modified Att-Kin procedure (discussion by Merkel), and the Hydrologic Engineering Center has also modified HEC-1 ("HEC-1" 1988) to "select a reasonable value of *DX* and *DT* to obtain a solution of the KW equations" (Goldman). It is of some interest that Dawdy earlier wrote that he "does not really feel 'comfortable' with HEC-1," because he is sure that "there is a bug in the stability criterion in the kinematic wave option, and the model sometimes produces anomalous results. However, even wrong, it is better than some other methods" (Dawdy 1987).

In the authors' experience, such as in short courses involving HEC-1, the general users of the program often attribute the apparent attenuation of the peak discharge in the KW option in HEC-1 to channel storage effects. Because of the widespread use of HEC-1, it is important that engineers who use HEC-1, or other general-purpose computer programs, be aware of the mathematical underpinnings. The KW routing option is a case in point. Program users often assume such computer programs to have considerably more capabilities than they actually contain.

The authors assume that there is no debate over the assumption that it is important to demonstrate to users possible computer-program spurious results. It simply isn't proper to use a computer program with the philosophy that "even wrong, it is better than some other methods."

Appendix. References

Dawdy, D. R. (1987). Discussion on "Evaluations of hydrologic models used to quantify major land-use change effects." *J. Irrig. and Drain. Engrg.*, 113(3).

"HEC-1 flood hydrograph package computer program." (1988). Hydrol. Engrg. Ctr., U.S. Army Corp of Engrs., Davis, Calif.

[10]Dir. of Water Resour. Engrg., Williamson and Schmid, 17782 Sky Park Blvd., Irvine, CA 92714.

[11]Assoc. Dir. of the Water Resour. Ctr., Univ. of California, Davis, CA 95616.

Division of Flow in Short Open Channel Branches[a]

Discussion by Ramesh N. Ingle[3] and Anant M. Mahankal[4]

The authors are to be commended for developing a theoretical model, which has been shown to be in good agreement with the experimental data,

[a]April, 1988, Vol. 114, No. 4, by Amruthur S. Ramamurthy and Mysore G. Satish (Paper 22384).

[3]Prof. of Civ. Engrg., Visvesvaraya Regional College of Engrg., Nagpur—440 011, India.

[4]Grad. Student, Visvesvaraya Regional College of Engrg., Nagpur, India.

FLOOD CONTROL DISTRICT of MARICOPA COUNTY 1959

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 31 1990

Dr. Larry W. Mays, P.E., P.H.
Chairman and Professor
Department of Civil Engineering
College of Engineering and Applied Sciences
Arizona State University
Tempe, Arizona 85287-5306

SUBJECT: Comments on Hydrologic Design Manual

Dear Dr. Mays:

Thank you for your comments on the Hydrologic Design Manual. Your input as well as input from others, has helped us to improve the material contained in the manual.

Our staff has put together the following response to your comments and indicate where changes will be made if necessary. The final draft of the manual will be completed by the middle of August.

A. In trying to define the variables in the rational equation, we recognized that there is a difference in how i, excess rainfall is defined. However, it was determined that for practical purposes the difference is not significant enough to invalidate its use. Using another form of the equation may be more theoretically correct, but may confuse some individuals, since the more commonly used form of the equation is the one presented in the manual.

We recognize that the C coefficients should be expanded; however, rather than provide all possible values, it would be better handled by leaving this to the various municipalities to define through Maricopa Association of Governments (MAG).

Several of the criteria required for hydrologic analysis are mandated by the Uniform Drainage Policies and Standards. A copy of this document was included as part of the information provided at the seminar. The criteria will be referenced as such in the final text, but will not be, and in most cases, cannot be technically supported. This includes the retention requirements and the 160 acre limit for the Rational Equation.

At some point in the future, the IDF curves will be tabulated and incorporated into the software provided. You wouldn't happen to have a graduate student available for this, would you?

B. The selection of the Clark Unit Hydrograph was based on the three criteria used to put the manual together: reproducibility, practicality, and relative accuracy. The Clark method is very flexible and with the proper guidance given in the chapter on application, its use is as simple as reading a textbook. Because the HEC-1 program converts Snyder UH to Clark, we felt it would be better not to lump parameters by using Snyder.

The basis for the time-area curves is from the analysis of data from similar hydrologic conditions. The results of these analyses will be presented in the Supporting Documentation Volumes of the Hydrologic Design Manual, which will be available by the end of this year.

C. Since the Curve Number method is widely used, we erroneously felt it was necessary to explain why we were not using Curve Numbers. The comments that you found to be false or misleading in our explanation will be removed.

D. We recognize that there are other routing techniques available, but they did not meet our criteria for practicality. Other methods investigated did not improve the final analysis significantly to justify their inclusion in the manual. The flexible nature of the manual does not preclude the use of other methods, should there be sufficient justification.

E. The District recognizes the art of hydrologic and hydraulic analysis on distributive systems is advancing, but did not find enough data applicable to Maricopa County to formulate criteria. We determined that a chapter on alluvial fans that addresses some of the hydrologic and hydraulic issues was outside the purpose of this manual.

I hope these responses clear up some of the issues you addressed in your correspondence. I recognize that until the actual data used for calibration is available you may still have questions. If, in the meantime, you would like to meet with our consultant or staff, please feel free to call. Once again I'd like to thank you for you input.

Sincerely,

STANLEY L. SMITH JR., P.E.
DEPUTY CHIEF ENGINEER
for D. E. Sagramoso, P.E.

TLB/JMR/ag

COORD: ~~DES,~~ ~~SLS,~~ 7/30 ~~DRJ,~~ ~~JMR,~~ ~~DDK,~~ ~~SDW~~

INFO: ~~JMR,~~ ~~FT,~~ ~~CHD~~ CHD

FILE: JMR

MFR: This letter was written in response to the above-referenced comments that were submitted for review.

GEORGE V. SABOL CONSULTING ENGINEERS, INC.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

13 August 1990

Mr. Joe Rumann
Hydrologist
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Subject: Hydrologic Design Manual for Maricopa County
review comments by Dr. Larry W. Mays, P.E., P.H.,
Arizona State University

Dear Mr. Rumann:

I have reviewed the comments by Dr. Mays in his letter of 30 May 1990, and the Flood Control District's response in its letter to Dr. Mays of 30 July 1990. I agree with the District's response, and I offer the additional response comments that you may wish to consider.

The District has already responded to most of Dr. Mays' comments on the use of the Clark unit hydrograph, but I have one general and one specific additional responses to add. First, I don't think that previous use of this method or the absence of its use, need be a criterion in selecting the Clark unit hydrograph or any other procedure for the manual. The Clark unit hydrograph was selected and procedures for its use were developed after extensive research and evaluation. Second, general time-area relations are not recommended for all situations. Rather, two default relations (one urban and one natural) are provided along with shaded zones where the time-area relations are expected to lie. For unusually shaped watersheds, it is recommended that the time-area relation be developed and that the derived relation should lie in the appropriate shaded zone.

Dr. Mays is obviously in agreement with our recommendation to use the Green and Ampt infiltration equation; however, he has concerns about statements in the manual concerning the SCS CN method that he believes are misleading and incorrect. There is no intent to mislead anyone and the authors of the manual certainly would not attempt such deception. I don't believe that any of the statements are incorrect. It probably is not possible to resolve the disagreements between Dr. Mays and me in a letter, but for the record, I will attempt to clarify a few points.

First, the statement, "rainfall losses are independent of the duration of rainfall" is true for both the SCS CN equation and also as it is coded into the HEC-1 program. A few simple HEC-1 input files using the CN option and changing only the duration of the rainfall will illustrate that the rainfall excess is identical for any selection of storm duration.

Second, Dr. Mays is correct that other assumptions about the initial abstraction can be selected when using the CN option of the HEC-1 program. The assumption that initial abstraction equals 0.2S is the usual assumption when using this method, and although that assumption is not required when using HEC-1, no guidance is available for selecting the appropriate value for the initial abstraction. The 0.2S assumption for initial abstraction is not related to the selection of antecedent moisture conditions (AMC) II or other AMC. When converting to another AMC from the usual assumption of AMC II, the CN is either adjusted up or down, but I know of no other adjustment that corresponds to a modification to the assumption of 0.2S as a function of the assumed AMC.

Third, although the selection of CN may be less subjective than the selection of the runoff coefficient of the Rational Method, I don't see how that statement is germane to the overall evaluation of the CN method.

Concerning channel routing, we recognized that kinematic wave routing is not the most accurate method that is available (but not necessarily contained as an option in the HEC-1 program). While that routing method has deficiencies, it is a method that is available in HEC-1 and for which general guidance can be provided. The user of the manual is not constrained to use the methods that are presented in the manual and this is so stated in the Introduction. Muskingum-Cunge routing will probably be incorporated into the manual when it becomes available as an option in HEC-1. I share Dr. Mays' concern about the kinematic wave method and I hope that he has noticed that the kinematic wave method for overland runoff routing has not been recommended.

A "major concern" of Dr. Mays is that modeling of alluvial fans is not addressed while he later states that, "No one has a real good handle on how we should hydrologically and hydraulically model active alluvial fans." I believe that the District has also noted this last deficiency and it is undertaking the first step in resolving that deficiency; that is, implementing a data collection program. I'm confident that guidance on alluvial fan analysis will be incorporated into the manual when reliable guidance is available.

I appreciate Dr. Mays' interest in the manual and that he took time to share his concerns with us. However, I had trouble identifying a consistent message in his comments: The Clark unit hydrograph was criticized because it is not extensively used; the Green and Ampt procedure was endorsed although it has been used only recently to any extent, and simultaneously the CN method was defended; the use of kinematic wave routing in channels was questioned while not providing a practical and readily available alternative; and he is concerned about not addressing alluvial fans while indicating that the technology is apparently not presently available.

In defense of the manual, the authors have tried to walk a course "between the devil and the deep blue sea." We have tried to prepare a manual that represents a reasonable expression of the state-of-the-art for which the theoritician and researcher may find a comfort zone, while simultaneously not jeopardizing the usability of the manual by the practicing engineer and hydrologist. I believe that the manual has been fairly successful in this regard.

I hope that my comments are of service to you. It is difficult to respond to Dr. Mays' comments in a letter, and Dr. Mays similarly may have had difficulty in conveying his concerns. A meeting to resolve these matters may be needed. I am available to meet with you and Dr. Mays to discuss the manual and to address Dr. Mays' concerns, if desired.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo Sabol

George V. Sabol

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JAN 1 1 1990

Mr. Jim Morris
Arizona Department of Water Resources
15 South 15th Avenue
Phoenix, AZ. 85007

Dear Mr. Morris:

The purpose of this correspondence is to request that the Arizona Department of Water Resources (ADWR) review and comment on the use of the Hydrologic Design Manual for Maricopa County (HDMMC) for floodplain studies. Our staff is confident that the preliminary draft now completed, will generate design hydrology which is more representative of conditions in Maricopa County than what has been used in the past.

I have enclosed a copy of the manual for your use, as well as an example of its' application. Our goal is to have the manual ultimately published as part of the Uniform Drainage Policy and Standards Manual by the Maricopa Association of Governments (MAG). In addition, we would like ADWR and FEMA to accept it as an appropriate methodology for study and design work.

Should you have any questions, or if you would like to meet and discuss this request, please feel free to contact either myself or Mr. Joe Rumann here at the District, thank you.

Sincerely,

STANLEY L SMITH JR., P.E
DEPUTY CHIEF ENGINEER
D. E. Sagramoso P.E.

Enclosures (2)

JMR/eal

Coord: JMR

Info: DJP

JJT

LCL

DRJ

SLS 1/10

File: Hydrology (JMR)

DDK COMMENTS

DEPARTMENT OF WATER RESOURCES
ENGINEERING DIVISION
FLOODPLAIN MANAGEMENT

File: mcfcd.man

TO: James R. Morris, PE.

FROM: David E. Creighton, Jr., PE.

DATE: April 18, 1990

SUBJECT: Review comments on Flood Control District of Maricopa County, Hydrology Design Manual, Preliminary Draft December 1989

Review of subject draft prompts the following comments for improving the adequacy of the material as a standard for flood control studies for FEMA FIS reports, CLOMR, and LOMA's.

1. List of Figures - Correct the spelling (isopluvial, precipitation). OK

2. Introduction - - Section 1.2 What I interpret here is don't try to calibrate watershed events relying upon the guides and parameters in this manual. THAT IS CORRECT.

3. The first paragraph of Section 1.3 - Using the Manual, has three very significant sentences in the 2nd, 3rd, and 4th (last) sentences. The last sentence in Par. 3 is also pertinent to the comments which follow.

4. Several of the introductory statements in topic sections might best be compared to the description of Professor Harold Hill in "Music Man", 'he doesn't know the territory'. NEITHER DOES STATE OF THE ART HYDROLOGY!

5. Section 2.1 General -- The mean annual precipitation of "more than 30" in the mountain regions of northern Maricopa County is highly suspect. From known precipitation maps and average annual station precipitation data, it appears questionable whether any Maricopa County location has an average which exceeds about 25" with a possible exception of a very small area on Mt. Ord near Sunflower (average precip= 23.13" in 29 years). O.K., WILL CHANGE TO 25", ORIGINALLY, AREAS OF BRADSHAWS WERE INCLUDED WHICH WOULD BE OUTSIDE MARICOPA COUNTY

6. Section 2.1.2 General Summer Storms -- The named tropical storms such as Katrina (9-5-67), Norma (Sept. 5, 1970), Joanne (10-7-72), post-Kathleen (9-26-76), and Octave (9-27 to 10-3,-83) appear to have possible cyclic-climatic association with ENSO (El Nino) periods. The direction southeast to northwest appears misleading. The southwest to northeast flow of the storm centers and cells needs to be recognized appropriately. LET'S INCLUDE

7. Section 2.1.3 Local Storms -- One of the major significant problems of periodic and damaging small drainage area flooding

events is the short-duration high-intensity rainfall events which happen over differing segments of gaged watersheds and which produce varying peaks and volumes at the downstream gage. Due to channel percolation losses the statistical frequency analysis attempts based on the gaging station record are highly distorted. A specific example is the history of Centennial Wash and the Harquahala Valley. The storm of August 26, 1964 inundated 8,400 ~13mi² cultivated acres. Precipitation recorded totaled 1.50" at Harquahala Plains, with 2.5" purported on the south slopes of Burnt Mountain. 7,000 cfs was estimated in the flood area. Volume of flooding may have ranged between 3,000- to 9,000+/- acre-feet. The USGS Centennial Wash gaging station record shows a mean daily flow of 11 cfs on 8/26, and 4.8 cfs on 8/27, for a total runoff of 31.3 acre-feet. The gaging station is about 21 miles downstream. This storm event was used as part of the project justification for Saddleback and Harquahala FRS's. The peak discharge at the gage for this event was less than the station base (1,000 cfs) for reporting. This event will be referred to again under section 6. AGREE, BUT CAN NOT DO MUCH ABOUT IT!

8. Section 2.5 Design Storm Distributions -- Table 2.2 and Figure 2.17 appear to be good guidance for 6-hour storm areal variations.

9. Section 4.1 General - pg 39. -- The vegetation-interception tabulation for hardwood tree can be very misleading. Cottonwood and willow are also 'hardwoods' and have a very extensive canopy during the growing season. A range of interception values should be shown related to size of plant growth from seedling-mature-harvested, or mowed (alfalfa, grass). THIS IS JUST FOR INFORMATION. WILL NOT BE USED IN THE MANUAL.

10. Section 4.3 Infiltration pg 42-last paragraph, pg 43-last par. Good idea.

11. Section 4.4.1 Green-Ampt -- This section is based on soil-moisture tension relationships derived from commercial farm crops with particular orientation toward irrigation and irrigation applications prior to soil-moisture reaching or falling below the wilting range and wilting point. These review comments are oriented toward placing this manual in an arid-land stance rather than a humid-region/irrigated crop orientation.

WHILE THIS IS TRUE, HEC-1 CAN HANDLE THIS PROBLEM BY "A TIME OF PONDING" CALCULATION SO INITIALLY DRY SOILS CAN BE ACCOMODATED.

11.1 Pg 53, 1st Para.: "Maricopa County has a large segment of its land area under irrigated agriculture...." In 1988, 5.2% was reported as irrigated cropland (306,205 acres in 9,225 sm). With 95% of the county being non-agricultural lands, the emphasis on wilting point as derived from agricultural studies for association and adoption of guiding Green-Ampt parameters is misleading. In 1961, Maricopa County had 555,240 acres of productive land (included irrigated, fallow, and idle).

SHOULD GET RID OF THIS PARAGRAPH.

11.2 The concept of "dry" relating to vegetation wilting point may have been based upon an irrigated agricultural crop/plant mix. The display of "wilting point" data should be against creosote bush, bursage, ironwood, palo verde, cholla, saguaro, ocotillo, mesquite and other xerophyte vegetative types. The concept of dry used may have some applicability (wilting point at a soil-moisture tension of -15 BAR for the 5% of cropped lands. Double cropped acreage may be included in the acreage. Abandoned farm lands should be rated as desert or rangeland. The 95% non-cropped will include another small amount of urbanized lands which are a mixture of irrigated landscaping and impervious.

11.3 Desert/arid-land vegetation varieties and wilting percentage levels were discussed on 4/2/90 with Dr. William Ehrler of the Desert Botanical Garden, Phoenix. From his background in plant physiology and knowledge of studies on soil-moisture tension with different sensors such as thermocouple psychrometers, membrane pressure plate, neutron probe meter and various specific and comparative plants such as agave, guayule, wild sunflower, Cucurbita digitata (gourd), jojoba, and corn, the soil moisture levels were expressed in terms of atmospheres (BAR). The SCS NEH manual, Section 15 - Irrigation, Chapter 1 Soil-Plant-Water Relationships, in a general way relates moisture content to soil-moisture tension (atmosphere) by soil texture (clay, loam, sand), and available and unavailable to plants (hygroscopic water) up to 15BAR in relation to wilting range and permanent and ultimate wilting point.

11.4 Desert plants have capabilities to survive at moisture-tension levels at about 25BAR for wild sunflower to below 40 BAR for guayule. The SCS indicates that 60 BAR may approximate the lower moisture limit. Definitive field studies for most of the Sonoran desert and other Arizona species have not been obtained. Whether they are available and the sources should be explored.

11.5 The function and possible correlation of the soil-moisture tension to the wetting front capillary suctions (psi of PSIF) and the volumetric soil moisture deficit (theta of DTHETA) need to be examined. In relation to PSIF (inches), the FCD manual Table 4.2, column 3 may require revision to reflect soil-moisture tensions in the range experience in desert soils (25 to 40 BAR). The role that stronger soil-moisture tensions may have in maintaining an increased infiltration rate deeper in the soil prism due to the root systems impact and depleted reservoir capacity should be considered. COMMENTS 11.2 TO 11.5 ARE GOOD FUTURE RESEARCH PROJECTS, BUT BEYOND THE SCOPE OF MANUAL.

11.6. With a 10 day to 2 week irrigation rotation on most agricultural land, other than some vegetables, 'saturated' does not appear to be a reasonable guide. Parks and golf courses with daily sprinkler system waterings may be close to normal for the greens, and fairways with other than bermuda grass. The narrow grass fairways through desert may have a net 'dry' DTHETA. The DTHETA discussion appears divorced from water conservation and restrictions on groundwater use on lands outside the operating irrigation districts. GET REAL !!!

11.7 On pg 49, par. ~~1~~ 3, 1st sent. "Under natural conditions, soil seldom reaches a state of soil moisture less than the wilting

point of vegetation". This sentence for most of Arizona and the arid west must be specifically related to the vegetation considered to be having the wilting point. This misunderstanding of desert soils and vegetation, desert and irrigated crop, reflects upon the DTHETA values and possibly the PSIF. The lumping of nationwide data including a preponderance of data from the humid eastern states may adversely bias the value derived from arid land samples. Moisture percentage samples reported for the McMicken Dam repair investigation (SHB for FCDMC) showed soil moistures near surface, and at depth, upstream, downstream, and on the dam axis, which were mostly in the range of 1% to 6 to 7%. The vegegation had not died at these soil-moisture levels.
FUTURE RESEARCH !!

11.8 Table 4.2 -- Green - Ampt Loss Rate Parameters -- The Table 4.2 values for XKSAT (Col. 2), and PSIF (Col. 3) appear to have been derived from Table 2 (Cols. 7 & 6) of the Rawls, Brakensiek & Miller (1983) report in HYD 109:1 ASCE pg 62, as converted from centimeters to inches. These values are from 1200 soils, (5000 horizons) in 34 states. Table 2 shows the sample size for each of 11 soil texture classes. Silt is not included in Table 2, although it is in FCD Table 4.2. Twenty-six soils were sampled in Arizona. SO ?

11.9 In reviewing the Rawls (1983), Table 2, it is noted that the psi value with a reported range of +/- one standard deviation values show an extremely wide spread with respect to the mean value. This range ratio varies from about 0.15 to 6.7 of the class mean. A display of arid land values within the +/- 1 SD range needs to be examined for trends possibly present for Arizona, and the possible area of Arizona, New Mexico and western Texas-Oklahoma west of the 102 degree west meridian. The relative role of expanding and non-expanding type clay minerals should also be identified. NO CURRENT ARIZONA DATA

11.10 For an Arizona and Maricopa County site specific parameter value chart, it would be most appropriate to obtain Arizona specific data as the initial data base before adjusting for national data. This is particularly appropriate for considering the possible (probable) impact of organic material on the values. The desert Arizona soils which make up 95% of Maricopa County are particularly noted for their low organic content. IF YOU CAN

(+) 11.11. The Table 2 XKSAT values for loam and silty loam appear to have been switched. The PSIF values for sandy-loam and loam appear to have been switched. The XKSAT value for silt which appears to have been obtained by averaging the silty-loam and sandy-clay-loam values is thus miscalculated.

WE NEED TO CHECK THIS

11.12 The source and basis for modifications for Figures 4.3, 4.4, 4.5, 4.6, 4.7, 4.8 and 4.9 should be more clearly identified and discussed. The values obtainable from figures 4.3 and 4.4 from centroids of soil classes most often do not appear close to the Table 4.2 parameter values. The relation to the spread of values may be relatable to the standard deviation range.

I AGREE !!

11.13 Figure 4.10 -- For the Ck scale, change rate to ratio, correct the vegetation cover illustrative point from 2.5 to 25. YES

11.14 Figure 4.12 - - With revision of Table 4.2, the configuration of this relation function needs to be checked against the corrected Table 4.2 values of XKSAT, and PSIF if Table 4.2 was the basis for Figure 4.12. Spelling and XKSAT example ordinate should be corrected. This is no longer included

(*) 11.15 Table 4.3, pg 63. Texture Classification (Col.1) Silt which was included in Table 4.2 is not included in at least 2 of the Brakensiek or Rawls references. Some checking needs to be done for loam and silty loam in the loss rate. The Hydrologic Soil Groups from the pg 58 table may need to be reorganized to include the texture omissions. A separate diagram developed from the NEH Handbook and Soils Reports data and glossary should be added to show the overlap in ranges for permeabilities/soil groups. NEED TO CHECK THIS CAREFULLY

11.16 Table 4.4 - - With the Hydrologic Soil Group, Soil Texture listing on pg 58, and the correlation to Table 4.3, the CNSTL values appear to be inconsistently low. The logic for the possibly conflicting display of values should be displayed.
SOIL TEXTURE OF PG.58 IS FOR INFORMATON ONLY!

11.17 Consideration needs to be given to the impact of soil classification on derived flood-frequency values as displayed in Shen, Koch, and Obeysekera (1990) (JHEND, 116:4 pg 495-514). The skewing of runoff frequency relationships between frequencies assigned to station record discharges and precipitation-runoff modeled discharges is further distorted from the absence of channel routing percolation in the modeling.

NOT REALLY RAINFALL TIME/AREA IS FOR UNIFORM DISTRIBUTION OF RAINFALL. CLARK TIME/AREA IS FOR WATERSHED RESPONSE AS A FUNCTION OF SHAPE.

12.1 Section 5. - - Unit Hydrograph Procedures.
The time-area relationships (pp 70-71) for unit hydrograph methods, and the rainfall-areal distribution have co-dependent relationships. A clearer enunciation should be made concerning the extent of the historic precipitation which has been used for parameter development and calibration. The reliability of most hydrographs, S-curves, and Lag time equations appear to hinge upon an assumption that a whole (total) watershed event occurred to produce the gaging station data basic to the specific station lag. Archive retrieval and re-publication of the specific dated events used to develop lag curves is needed, with revision as necessary for events which have occurred during the last 40-50 years since the earliest lag curves were presented in the 1940's. The basic USCE 1940's data appears to have remained unchanged and unreviewed. With the amount of evidence being developed that discharges and runoff are cyclic, climatic, and non-random, review of the data upon which lag curves have been developed is overdue.

YES BUT!!

12.2 Section 5.3 Limitations and Applications. - - The 5 and 10 sm values as guides, thresholds or ceilings appears to be a parameter not often followed on large watershed and floodway studies. A check list of priority of acceptable and preferred applications for the elements of this manual is suggested, with attention to size of watershed area and subareas, routing length, and routing method related to positively modeling the channel percolation losses. IT HAS BEEN INDICATED THAT SMALL AREAS SHALL USE CLARK PROCEDURE, LARGE AREAS USE S-GRAPH.

12.3 Section 5.4, 5.5, & 5.5.1 Time of Concentration. The Papadakis and Kazan (1987) adaptation is a conversion from feet to mile length and slope, and minutes to hour time parameters. The watershed size basis for the formula appears to be based on: 1. USCE airfield series of small areas, 2. University laboratories using very small artificial watersheds, and USDA-ARS data from 84 small rural watersheds. The 3 laboratory condition watershed lengths varied for 40 feet to 500 feet. The field watershed criteria are not identifiable from the reference report, <u>per se</u>. The applicability to Maricopa County general, unrestriced, unqualified use for areas such as Cave Creek, Centennial Wash, Waterman Wash, Jackrabbit Wash, and numerous other medium to large watersheds should be verified. Should watershed size be classified using a logarithmic scale from 1 to 10,000 sm? DOCUMENTATION MANUAL.

12.4 The supplemental data base from AZ, NM, CO, & WY may be helpful for further review. Figure 5.4 provides an interesting commentary on the applied over the theoretical Papadakis & Kazan equation. At slopes of 300, 400, 500, & 600 ft/mi, corrections of 11, 28, 38, & 46% need to be applied.

(*) 12.5 Section 5.6 S-Graphs , & 5.6.2 Sources of S-Graphs.
The earliest S-graphs and Lag curves retrieved from DWR files and library are dated 1946 and 1948. The parallel development of the two charts and subsequent editions indicate that the additional stations displayed for the Lag diagram (Figure 5.11) has been for the purpose of supporting the original, rather than possible revision and/or updating. NEED TO UPDATE

12.6 The inclusion of Verde River, Tonto Creek, and Salt River above Phoenix with the reservoir system complex through which the upper watershed runoff flows must be reservoir routed to reach the Phoenix area causes a question to be raised about the blanket appropriateness of the Phoenix Mountain S-Graph. The numerals for figures 5.09 and 5.10 need to be large enough to be readable, and the missing scale values added. The charts may need to be replotted. The basic data for some of the stations in the upper Gila River, especially the Lag curve, may not have been updated for the major runoff events which occurred after 1947 (or 1945). Only 3 of the 10 highest ranked discharge events for the Gila River at the head of Safford Valley occurred during the record for the 1948, 1952 reports. Those events currently are ranked 2, 5, and 7.

(*) 12.7 Section 5.6.4 - Estimation of Lag + Fig 5.11.
Corrections are needed for the following items: UPDATE
#18 - change Connor to Conner, and AZ to NM,
#19 - change length 30.0 to 130.0

and on the chart, #7 has been mis-identified as 17,
#1 appears to have been omitted,
#10 appears to have been mis-plotted,
check plotting of points #3 & #6.

#18 and #19 are both non-record locations for streamflow and hydrograph records. #18 approximates Gila River near Redrock, NM. As a minimum, stations in Arizona and New Mexico which should have their Lag time data updated for revision or re-verification are:

No.	Place	Update for Events of
18	Gila R.(Conner) nr Redrock	Dec. 19, 1978, Dec. 28, 1984
19	San Francisco R }	Dec 23, 1965, Aug 12, 1967, Oct 20
20	Blue R. nr Clifton }	1972, Sept 8-9, 1975, Dec 19, 1978,
	}	Oct 2, 1983
21	Salt R nr Roosevelt}	1-18-52, 12-26-59, 12-23-65, 10-20--72, 3-2-78, 12-19-78, 2-15-80

13.1 Section 6 - Channel Routing -- The exclusive identification of only Kinematic Wave and Muskingum Routing and excluding any reference to the Normal Depth (HEC-1 RS, RC, RX, RY records) guarantees that the 10-year, 50-year, and other low year frequency discharges will artificially skew all model derived frequency curves by excluding the use of the program capability to include channel percolation losses. The Centennial Wash August 26, 1964 example, previously referred to, demonstrates even by the limited record available, the fallacy and hazard of doing studies using less than the capability of the programs available and the historic streamflow records for calibration efforts. The Hassayampa River nr Morristown, Cave Creek, and New River records provide further gage evidence of significant to total streamflow loss due to percolation.

SHOULD WE INCLUDE NORMAL DEPTH ROUTING

14.1 Section 7 - Application -

Section 7.3.2. Interesting to find the term 'channery' (sic) used in Scotland and Ireland for gravel to be included with, etc.

14.2 #4. Hydrologic Soil Groups: Fractured rock, bouldery block talus slopes, and basalt covers need to be recognized for their differences from solid or impervious rock areas. OK INCLUDE

14.3 <u>Section 7.4 - Parameters for Clark Unit Hydrograph.</u>

#4. The caution is important that Tc is a function of the excess rainfall intensity and varies with the duration and magnitude 'frequency' of the storm. This is often overlooked, or not even recognized when models are set up for differing frequencies (5, 10, 25, 50, or 100 years).

14.4 <u>Section 7.7 Notes, Applications of S-Graphs.</u> - The S-graphs and Lag computations implies that the lag time has no variation regardless of storm magnitude and differing storm characteristics, one being location of storm cell in relation to the gaging (or analysis) station. With major ranked discharges showing indications of occurrence with cyclic-climatic trends, the dates of events for assessment of lag times need to be reviewed. The lack of change between original publication about

1945 for most of the lag curve ~~plotted~~ points indicates examination of criteria and data are needed. YES !!!

15. <u>Appendix</u> <u>A,</u> <u>B,</u> <u>&</u> <u>C.</u> The Loss-Rate Parameters need to be checked for XKSAT & PSIF after Table 4.2 is reviewed and modified as necessary. Occasional mispellings mar the professional quality of the material.

16. <u>Examples.</u> - - Correction to agree with Table 4.2 revisions.

FLOOD CONTROL DISTRICT
1959
FLOOD CONTROL DISTRICT
of
Maricopa County
3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501
D. E. Sagramoso, P.E., Chief Engineer and General Manager

MAY 16 1990

Mr. David E. Creighton Jr., P.E.
Arizona Department of Water Resources
Engineering Division, Floodplain Management
15 S. 15th Ave.
Phoenix, Arizona 85007

RE: Hydrologic Design Manual for Maricopa County, December 1989 Draft Response to Comments Dated April 18, 1990 from D.E. Creighton to J.R. Morris

Dear Mr. Creighton:

My staff and I of the Special Projects Branch, Hydrology Division, have reviewed your comments concerning the subject draft manual. Our responses are listed below, according to the numbering system in your memo:

1. We will correct this.

2. The point we are trying to make is that calibration requires a more complex set of parameters (i.e. recorded rainfall depths and aerial averaging, losses due to channel percolation, etc.) than is required for design work.

3. We assume that this is a comment to James R. Morris.

4. We do not understand the intent of this statement. Please clarify if you feel it is warranted.

5. We will change to 25 inches. Originally, the intent was to include areas of the Bradshaw Mountains which contribute runoff to the Agua Fria and Hassayampa watersheds in Maricopa County.

6. We will correct the direction of storm travel.

7. We again assume that this comment is directed to Mr. Morris. It is true that the frequency analysis in this case is not representative of the upstream event when one considers what the discharge might have been if 1.5 inches of rain had been distributed over the entire watershed above the gauging station. However, isn't the discharge record at the gauge an indicator of how the watershed responds to all historic rainfall events, including allowances for channel losses and aerial rainfall extent? Again as a reminder, the Hydrologic Design Manual (HDM) methods will be used as presented only for drainage areas less than 100 square miles.

9. The table was cited for information purposes only. The values will not be used in any calculations.

11.1 The word "large" will be removed.

11.2 - 11.6
We consider these comments to be good future research projects, and data from such projects will be incorporated into the HDM if/when it becomes available. Our intention was to use DTHETA (saturated) for irrigated agricultural lands, DTHETA (normal) for soils that are usually maintained in a state of high soil moisture, and DTHETA (dry) for all other soils. These soil moisture states apply to design applications only. We will include the observation that abandoned farmland should be treated as rangeland.

11.7 - 11.11
We are presently reviewing the loss rate parameter tables for inconsistencies relative to their sources. Sources provided by you will be considered.

11.12 Figures 4.3 and 4.4 will be removed from the final draft.

11.13 We will make these changes.

11.14 Figures 4.11 and 4.12, and their associated procedures, have been deleted from the April 1990 draft version.

11.15 - 11.16
See comments for 11.7 - 11.11. The Hydrologic Soil Group table on page 58 has been deleted.

11.17 See comments for #7, page 1 of this letter.

12.1 We agree, although time-area relationships are more a function of watershed shape and hydraulic efficiency than rainfall distribution. Part of the reason for limiting application of the Clark Unit Hydrograph to areas less than 5 square miles is because the data came from small watersheds where complete aerial rainfall coverage is assumed. Parameter development and testing will be referenced in the Documentation Manual as explained at the Promulgation Conference.

12.2 We will consider inserting a summary table similar to the following:

Drainage Area	Method of Determining Peak Discharge
0 - 160 acres	Rational Method or Clark Unit Hydrograph
160 ac. - 5 sq.mi.	Clark Unit Hydrograph
5 sq.mi. to 100 sq.mi.	Phx. Valley or Phx. Mountain S-Graph (or subdivide and use Clark U.H.)

Channel percolation losses will not be addressed because of the lack of a practical, reliable, and reproducible method of determining loss rates.

12.3 As stated in section 5.3 of the HDM, use of the Papadakis Tc, as a component of the Clark Unit Hydrograph, is limited to watersheds less than 5 sq.mi. in size. This procedure will not be used for "medium to large watersheds" unless they are broken down into sufficiently small subbasins. Also, in Figure 5.5, watershed area is in acres.

12.4 Figure 5.4 provides a means of adjusting watershed slope based on the non-linear relationship between slope and floodwave velocity, and has been applied to other Tc equations.

12.5 We assume this is a comment to James R. Morris, however, we would like to review your information if possible.

12.6 The S-Graph figures will be improved. A major hydrologic study of the Verde/Salt River system would require special consideration outside the scope of the HDM.

13.1 In some of our model calibrations, we have used the RL card very successfully in combination with Muskingum Routing. This combination simulates peak attenuation due to both channel losses and wedge storage. Again, no guidelines are given for use of the RL card due to lack of an easily applicable procedure for determining channel percolation losses. Also, special consideration would be given to modeling these watersheds outside that provided by the HDM.

14.2 We will include a statement addressing this.

14.4 We agree that more data are needed. Unfortunately, a convenient procedure for adjusting lag time based on return frequency does not exist. Also, because we are applying a design storm, it must be assumed that the entire watershed is covered.

15. We are in the process of reviewing these tables concurrently with the Chapter 4 loss rate tables.

16. We will correct as necessary.

Thank you for your helpful and sometimes enlightening observations. Please find enclosed your original redlined copy of the HDM as requested. If you feel more comments are necessary concerning the April, 1990 version of the HDM, please feel free to send them, or call.

Sincerely,

Joe Rumann
Hydrologist

Enclosure

SDW/eal

Coord: ~~DRC~~

Info: ~~SDW~~ SDW ~~DDK~~

File: Hydrology (JMR)

George V. Sabol Consulting Engineers, Inc.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

17 July 1990

Mr. Joe Rumann
Hydrologist
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Subject: Hydrologic Design Manual for Maricopa County, review comments by Mr. David E. Creighton Jr., ADWR

Dear Mr. Rumann:

I have reviewed the comments of Mr. David E. Creighton Jr., in his memorandum to Mr. James R. Morris in the ADWR memorandum of 18 April 1990, and also the Flood Control District's responses in its letter of 16 May 1990. Mr. Creighton has performed a very thorough review in which he has helped us to identify several errors and deficiencies in the December 1989 draft of the manual and he has also prompted us to substantiate the content of the manual. The District's response to Mr. Creighton was very thorough and I agree with those; however, I have provided additional comments to some of Mr. Creighton's concerns that you may wish to consider. My responses, which follow, are numbered according to the numbering system used by Mr. Creighton:

7. Two problems in performing flood frequency analyses using discharge data for gaged watersheds are correctly identified. First, often the storms produce only partial areal rainfall, and second, channel transmission losses can be significant in reducing downstream discharges. Of course these, along with other reasons, are why we are using a rainfall-runoff procedure to estimate flood magnitudes for selected rainfall frequencies rather than performing flood frequency analyses of gaged watershed data. In that regard, we assume that for the type of <u>severe</u> storms that we are attempting to model that the rainfall does cover the entire watershed (with appropriate reduction of both rainfall depth and intensity as a function of watershed area). I believe that our design storm assumptions are appropriate in this regard which negates a concern about partial area rainfall for local storms on smaller (less than 100 square mile) watersheds. The second concern is valid, and although we have not provided guidance on how channel transmission losses are to be estimated, we do not necessarily intend that transmission losses are not to be estimated. I believe that we all agree that the engineer or hydrologist is responsible for estimating transmission losses if it is believed that such losses are a critical element of the analysis. Possibly we need to provide adequate discussion of this concept in the manual to avoid future concerns about this subject.
9. As the District correctly noted in its response, the interception values are for information only. Interception is included in surface retention loss. The magnitude of surface retention loss estimates could be increased beyond the guidance provided in the manual for good cause. As such, more extensive guidance on interception is probably not warranted.

11. I disagree with this broad statement about the derivation and application in Maricopa County of the published Green and Ampt equation parameters. Soil-moisture tension influences only the DTHETA parameter and not either the hydraulic conductivity (XKSAT) or the capillary pressure (PSIF), and these two parameters are far more important in severe storm rainfall losses than is DTHETA. The implication that XKSAT and PSIF estimated from agricultural lands are not applicable to other lands (arid or humid) does not seem valid. I think the question should be, can Green and Ampt equation parameters be estimated as a function of soil texture? If the answer is yes, and several researchers in this area of study seem to think it is, then the question becomes, do the parameter values (XKSAT and PSIF) vary for a given soil texture whether that soil exists in a humid or an arid environment? The answer to the second question would seem to be no for bare soil. However, vegetation differences that exist between arid and humid zones will affect the infiltration rates for a given soil texture. We recognize that fact and we have tried to account for that as best we can in Figure 4.10. Since the parameter values by Rawls and other are for bare ground, I would argue that they are more appropriate for use in western rangelands than eastern and midwestern pastures, forests, etc. where vegetation influences could dramatically alter the effective hydraulic conductivity.

11.2-11.5 and 11.7

In preparing the guidance for DTHETA, I referred to several publications including the SCS NEH, Section 15 - Irrigation manual and talked to Dr. Herman Bouwer of the ARS Water Conservation Lab in Phoenix. My conclusion from those investigations was that the information on wilting point and soil-moisture tension that are to be <u>assumed</u> for a <u>design storm</u> are not available. Mr. Creighton's statement that, "Definitive field studies for most of the Sonoran desert and other Arizona species have not been obtained." may be because this information does not exist. His next statement, "Whether they are available and the sources should be explored." was done to my satisfaction. I did not believe that further effort is warranted.

Mr. Creighton's concern seems to be centered on the wilting point for native vegetation in Maricopa County and how that wilting point would affect the Green and Ampt parameter values. Again, wilting point affects the assumption for selecting DTHETA but should not affect either XKSAT or PSIF. The assumption on DTHETA should be made with due regard to likely soil moistures that may exist prior to a <u>design storm</u>. First, there is very little volumetric difference in soil moistures for most soils (clay is rather unusual in this regard but not of great significance to us because of the low occurrence of this soil texture in Maricopa County) whether the wilting point is 15 bars or 60 bars. I think that we would all agree that within this range that the soil is very dry and that the potential storage in the soil matrix is something less than the soil porosity. I believe that the DTHETA for the Dry condition of Table 4.2 adequately represents that assumption regardless of what the actual wilting point is.

11.6

The "Saturated" condition for DTHETA was recommended for irrigated agricultural land because we believe that it is prudent to assume that the design storm could occur shortly after irrigation application. This is conservative, but defensible. In regard to water conservation and other considerations concerning the selection of DTHETA, we recognize that good judgement should be used in this selection and the engineer or hydrologist should consider all reasonable factors in that selection.

11.8

The Green and Ampt parameter values for silt were taken from the Rawls and Brakensiek (1983) paper (not the Rawls, Brakensiek, and Miller, 1983 paper) and were selected to fit between the adjacent soil texture values.

11.9 and 11.10

These comments will be considered and discussed with other professionals in Arizona in regard to the need for this additional research/information gathering. Further action should be undertaken, if deemed necessary.

11.11

There was an error in the source of this information (Rawls, Brakensiek, and Miller, 1983). This was confirmed by Rawls in April 1989. I have recently communicated to Rawls and suggested that he publish a correction.

12.1

A certain reliance has been placed on both the Corps' and the USBR's analysis of data in the presentation of lag relations. The data base certainly has not remained unchanged since the 1940's (see <u>Design of Small Dams</u>, Third Edition, 1987), and I don't believe that the data has remained unreviewed by either of these agencies or others. I investigated other forms of lag relations (S-Graph Report, 1987) but found no definitive result that was preferable to those already in use.

12.3

A certain amount of testing and verification has been performed and results will be presented in the Documentation Manual. The T_c method is not recommended for use with watersheds larger than 5 square miles and therefore verification on medium to large watersheds is not necessary.

12.5

Further analysis of S-graphs and lag relations may be of value, however, a need for revision has not been identified. Updating has been done (see my response to 12.1).

12.7

Mr. Creighton's suggested corrections should be investigated and changes made, as necessary. The need to expand or update the lag relation for the identified flood events should be considered.

13.1

The manual does not preclude the user from any option within the HEC-1 program or other valid hydrologic analysis. This may need to be more clearly stated in the manual.

14.3

To further expand on Mr. Creighton's correct observation about the T_c equation being a function of rainfall excess, I would like to add that this has the effect of incorporating nonlinearity into the unit hydrograph approach which is often criticized for being linear. This will aid in the best estimation of the smaller, more frequent events.

14.4

The lag equation is not a function of storm characteristics, however the authors of the manual have not implied that "lag time has no variation regardless of storm magnitude and differing storm characteristics." On the contrary, on page 87 (April 1990 draft) is contained a discussion of the need to adjust K_n to reflect different hydraulic efficiencies for differing magnitudes of runoff, and it is stated that this is due to varying rainfall depths and intensities.

Mr. Creighton's comments have been helpful in sorting out some errors, and have challenged me to critically reconsider many of my assumptions and decisions. In this regard, his review has been appreciated. I hope that my response is of service to you.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

George Sabol

George V. Sabol

GEORGE V. SABOL Ph.D., P.E.
CONSULTING ENGINEER
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-~~0989~~ 4015

17 July 1990

Dr. Walter Rawls
ARS, U.S. Dept. of Agriculture
Agri. Research Center-West
Bldg. 007, Rm. 137
Beltsville, MD 20705

Subject: Maricopa County Hydrologic Design Manual

Dear Walter:

Last April I wrote to you concerning values of Green and Ampt infiltration equation parameters that were published in the January 1983 Hydraulics Division journal, and a copy of that letter with your response is enclosed to help refresh your memory about this "old" correspondence. Maricopa County has completed its Hydrologic Design Manual and the manual is being reviewed in the community. We have had several inquires about why we have reversed some of the Green and Ampt parameter values from those that are published in the reference for those values. It is important that we are confident that the information in the manual is defensible and is from the best available source. Therefore, I am asking for a confirmation and possibly clarification of the data from that ASCE paper.

I have enclosed a copy of a table (Table 4.2) from the Maricopa County Manual that shows the values of the Green and Ampt parameters that we are recommending. The XKSAT values and the PSIF values are english unit equivalents of the hydraulic conductivity and wetted front capillary pressure, respectively, from the ASCE paper. Notice that I have reversed the XKSAT values for loam and silty loam which is in agreement with my April 1989 correspondence and your reply.

However, notice that I have reversed the PSIF values for sandy loam and loam from those reported in the ASCE paper. We may have talked about that change at some time, but I don't have any record of such discussion. The values from Table 4.2 seem to be correct, especially when compared to graphical presentation of these parameter values (Rawls and Brakensiek, 1983), and copies of those figures (from the manual) are enclosed. I have added parameter values from the ASCE paper and other notes to those figures.

Complicating all of this is that the Green and Ampt parameter values from the ASCE paper may be pervading the literature. Notice in another enclosure that Dodson & Associates, Inc. (1989) are using the ASCE paper parameter values in a commercially available version of HEC-1, and that Dodson references an intermediate source (Chow, Maidment, and Mays, 1988). The trail of use of the values from the ASCE paper is getting long!

I have one request and one suggestion: First, please verify or correct the values of XKSAT and PSIF that we are showing in Table 4.2 considering that we are using the ASCE paper as the major reference. Do you have another reference that we should use? Your comments and suggestions would be appreciated. Notice that I added values for silt because that soil texture class is not contained in the ASCE paper. I "selected" the values for silt based on the figures in Rawls and Brakensiek (1983) that generally fit between silty loam and sandy clay loam.

Second, it seems that with the growing popularity and use of the Green and Ampt equation and the proliferation of a possible error or errors in the ASCE paper, that another ASCE paper or technical note would be appropriate to get us all on the same page. An expansion to include your best interpretation of Green and Ampt parameter values would be of great value to the profession.

I thank you for your continued cooperation in our preparation of the Maricopa County Hydrologic Design Manual. I hope that this doesn't cause you much inconvenience. Please call me after you have had time to think about the contents of this letter. I will be attending the ASCE San Diego meeting, and possibly we could talk there if you also plan to attend.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

George V. Sabol

Copy: Mr. Joe Rumann, Flood Control District of Maricopa County
w/enclosures

Enclosures:
1. Copy of 3 April 1989 letter w/enclosure,
2. Table 4.2 from Maricopa County Hydrologic Design Manual,
3. XKSAT and PSIF figures, and
4. Green and Ampt parameter values from Dodson & Associates, Inc. (1989)

GEORGE V. SABOL Ph.D., P.E.
CONSULTING ENGINEER
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-0989

CONSULTING ENGINEERS C A E C COUNCIL

3 April 1989

Mr. Walter Rawls
ARS, U.S. Dept. of Agriculture
Agri. Research Center-West
Bldg. 007, Rm. 137
Beltsville, MD 20705

Subject: FCDMC Hydrology Manual

Dear Walter:

Thank you for your review comments of 23 March 1989. I will incorporate your suggestions into the manual. I have one question for which I would like your response. This is in regard to the values of the Green and Ampt parameters for loam and silt loam in Table 2 of Green-Ampt Infiltration Parameters from Soils Data by Rawls, Brakensiek, and Miller (ASCE, Hyd. Engineering, Vol. 109, No. 1). Specifically, on the table the value of hydraulic conductivity for loam is 0.34 cm/hour and for silt loam is 0.65 cm/hour, and the capillary pressures are 8.89 cm and 16.68 cm, respectively. Since loam generally has a higher sand content than silt loam I would think that these values should be reversed, and that loam would have a greater hydraulic conductivity than silt loam. Is there some reason for this anomaly, or is there a possible error in this table?

Sincerely yours,

George V. Sabol

George V. Sabol

Enclosure: Copy of Table 2

Copy: Mr. Joe Rumann, Flood Control District of maricopa County

George
you are correct the numbers were transposed.
Walter

FIG. 1.—Distribution of Soils

in which θ = soil water content, in cubic centimeters per cubic centimeter; θ_r = residual saturation, in cubic centimeters per cubic centimeter; ϕ = total porosity, in cubic centimeters per cubic centimeter; ψ_b = bubbling pressure, in centimeters; ψ = capillary pressure, in centimeters; and λ = the pore-size distribution index.

The Green and Ampt parameters can be calculated from the estimated Brooks and Corey constants as follows: The wetting front capillary pressure term, ψ_f, is calculated by (2)

$$\psi_f = \frac{2\lambda + 3}{2\lambda + 2}\left(\frac{\psi_b}{2}\right) \qquad (4)$$

The effective porosity, θ_e, is calculated as

$$\theta_e = \phi - \theta_r \qquad (5)$$

in which ϕ = the total porosity, in cubic centimeters per cubic centimeter, and is calculated from bulk density and particle density; and θ_r = the residual soil-water content, in cubic centimeters per cubic centimeter. The Green and Ampt hydraulic conductivity, K, based on Bouwer's (4) findings that it is one-half the saturated hydraulic conductivity, is calculated as

$$K = \frac{K_s}{2} \qquad (6)$$

in which the saturated conductivity, K_s, is calculated by an equation (Ref. 5) derived by substituting the Brooks and Corey equation into the Childs, Collis-George permeability integral (6) given by

$$K_s = a\frac{\phi_e^2}{\psi_b^2}\left[\frac{\lambda^2}{(\lambda + 1)(\lambda + 2)}\right] \qquad (7)$$

in which a = a constant representing the effects of various fluid con-

TABLE 2.—Green and Ampt Parameters According to Soil Texture Classes and Horizons

Soil texture class (1)	Horizon (2)	Sample size (3)	Total porosity, ϕ, in cubic centimeters per cubic centimeters (4)	Effective porosity, θ_e, in cubic centimeters per cubic centimeters (5)	Wetted front capillary pressure, ψ_f,[d] in centimeters (6)	Hydraulic conductivity, K,[b] in centimeters per hour (7)
Sand[c]		762	0.437 (0.374–0.500)[d]	0.417 (0.354–0.480)	4.95 (0.97–25.36)	11.78
	A	370	0.452 (0.396–0.508)	0.431 (0.375–0.487)	5.34 (1.24–23.06)	
	B	185	0.440 (0.385–0.495)	0.421 (0.365–0.477)	6.38 (1.31–31.06)	
	C	127	0.424 (0.385–0.463)	0.408 (0.365–0.451)	2.07 (0.32–13.26)	
Loamy sand		338	0.437 (0.363–0.506)	0.401 (0.329–0.473)	6.13 (1.35–27.94)	2.99
	A	110	0.457 (0.385–0.529)	0.424 (0.347–0.501)	6.01 (1.58–22.87)	
	B	49	0.447 (0.379–0.515)	0.412 (0.334–0.490)	4.21 (1.03–17.24)	
	C	36	0.424 (0.372–0.476)	0.385 (0.323–0.447)	5.16 (0.76–34.85)	
Sandy loam		666	0.453 (0.351–0.555)	0.412 (0.283–0.541)	11.01 (2.67–45.47)	1.09
	A	119	0.505 (0.399–0.611)	0.469 (0.330–0.608)	15.24 (5.56–41.76)	
	B	219	0.466 (0.352–0.580)	0.428 (0.271–0.585)	8.89 (2.02–39.06)	
	C	66	0.418 (0.352–0.484)	0.389 (0.310–0.468)	6.79 (1.16–39.65)	
Loam		383	0.463 (0.375–0.551)	0.434 (0.334–0.534)	8.89 (1.33–59.38)	0.34
	A	76	0.512 (0.427–0.597)	0.476 (0.376–0.576)	10.01 (2.14–46.81)	
	B	67	0.512 (0.408–0.616)	0.498 (0.382–0.614)	6.40 (1.01–40.49)	
	C	47	0.412 (0.350–0.474)	0.382 (0.305–0.459)	9.27 (0.87–99.29)	
Silt loam		1,206	0.501 (0.420–0.582)	0.486 (0.394–0.578)	16.68 (2.92–95.39)	0.65
	A	361	0.527 (0.444–0.610)	0.514 (0.425–0.603)	10.91 (1.89–63.05)	
	B	267	0.533 (0.430–0.636)	0.515 (0.387–0.643)	7.21 (0.86–60.82)	
	C	73	0.470 (0.409–0.531)	0.460 (0.396–0.524)	12.62 (3.94–40.45)	
Sandy clay loam		498	0.398 (0.332–0.464)	0.330 (0.235–0.425)	21.85 (4.42–108.0)	0.15
	A	—[e]	—	—	—	
	B	198	0.393 (0.310–0.476)	0.330 (0.223–0.437)	26.10 (4.79–142.30)	
	C	32	0.407 (0.359–0.455)	0.332 (0.251–0.413)	23.90 (5.51–103.75)	
Clay loam		366	0.464 (0.409–0.519)	0.309 (0.279–0.501)	20.88 (4.79–91.10)	0.10
	A	28	0.497 (0.434–0.560)	0.430 (0.328–0.532)	27.00 (6.13–118.9)	
	B	99	0.451 (0.401–0.501)	0.397 (0.228–0.530)	18.52 (4.36–78.73)	
	C	55	0.452 (0.412–0.492)	0.400 (0.320–0.480)	15.21 (3.79–61.01)	
Silty clay loam		689	0.471 (0.418–0.524)	0.432 (0.347–0.517)	27.30 (5.67–131.50)	0.10
	A	65	0.509 (0.449–0.569)	0.477 (0.410–0.544)	13.97 (4.20–46.53)	
	B	191	0.469 (0.423–0.515)	0.441 (0.374–0.508)	18.56 (4.08–84.44)	
	C	39	0.475 (0.436–0.514)	0.451 (0.386–0.516)	21.54 (4.56–101.7)	
Sandy clay		45	0.430 (0.370–0.490)	0.321 (0.207–0.435)	23.90 (4.08–140.2)	0.06
	A	—	—	—	—	
	B	23	0.435 (0.371–0.499)	0.335 (0.220–0.450)	36.74 (8.33–162.1)	
	C	—	—	—	—	
Silty clay		127	0.479 (0.425–0.533)	0.423 (0.334–0.512)	29.22 (6.13–139.4)	0.05
	A	—	—	—	—	
	B	38	0.476 (0.445–0.507)	0.424 (0.345–0.503)	30.66 (7.15–131.5)	
	C	21	0.464 (0.430–0.498)	0.416 (0.346–0.486)	45.65 (18.27–114.1)	
Clay		291	0.475 (0.427–0.523)	0.385 (0.269–0.501)	31.63 (6.39–156.5)	0.03
	A	—	—	—	—	
	B	70	0.470 (0.426–0.514)	0.412 (0.309–0.515)	27.72 (6.21–123.7)	
	C	23	0.483 (0.441–0.525)	0.419 (0.294–0.544)	54.65 (10.59–282.0)	

[a]Antilog of the log mean and standard deviation.
[b]Values for Rawls, et al. (13).
[c]Values for the texture class.
[d]Numbers in ( ) = one standard deviation.
[e]Insufficient sample to determine parameters.

general soil texture classification of the drainage area is available. The values of XKSAT and PSIF from Figures 4.3 and 4.4 can be used if more specific soil texture classification is available from a detailed soil survey for which the percentages of sand and clay have been determined by an appropriate field soil survey. The use of the information in Figures 4.3 and 4.4 will require an extensive study of the soil for the drainage area, and for most drainage studies only general soil texture classification will be known so the values from Table 4.2 should be used.

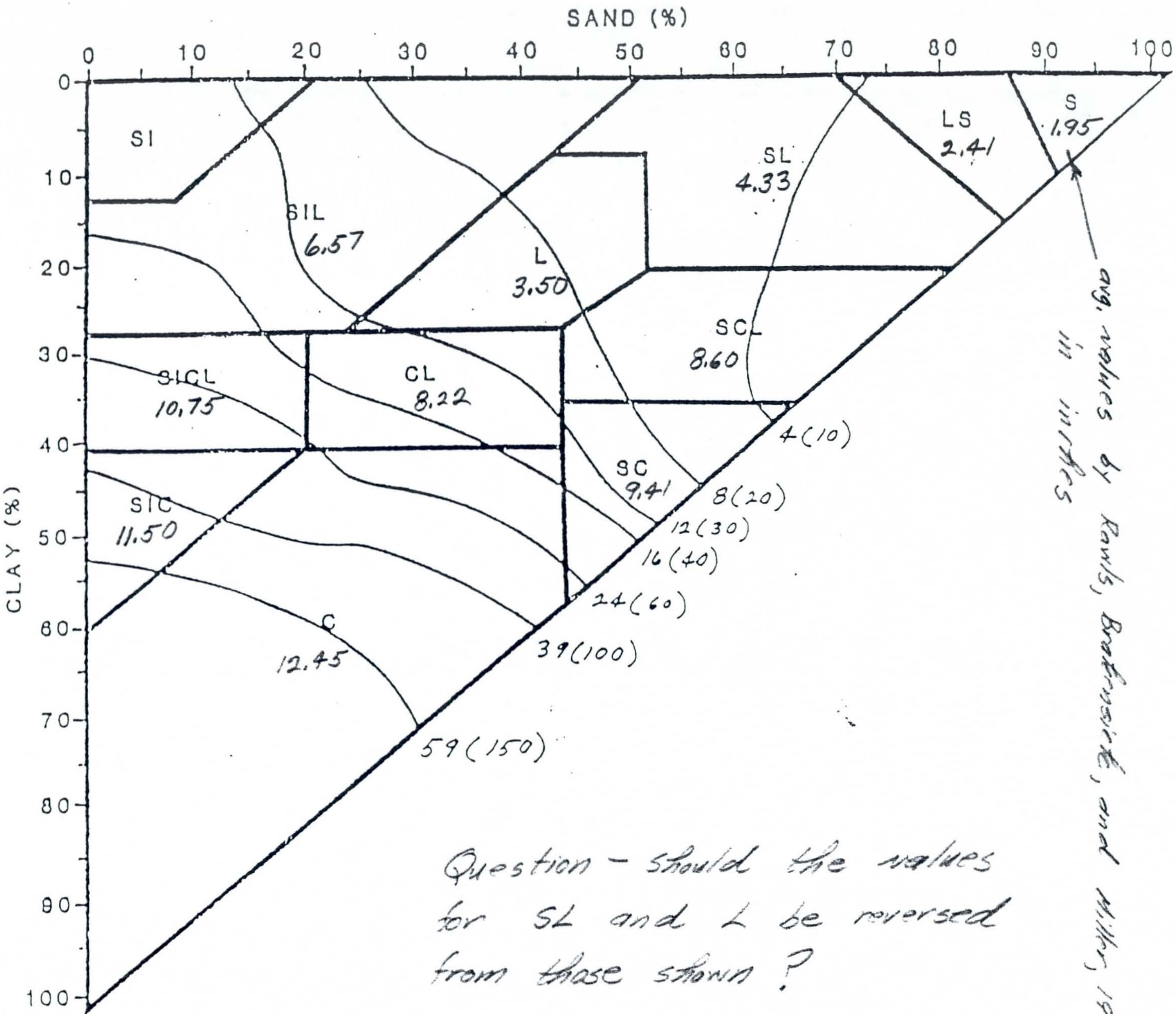
The soil moisture deficit (DTHETA) is a volumetric measure of the soil moisture storage capacity that is available at the start of the rainfall. DTHETA is a function of the effective porosity of the soil. The range of DTHETA is 0.0 to the effective porosity. If the soil is effectively saturated at the start of rainfall then DTHETA equals 0.0; if the soil is devoid of moisture at the start of rainfall the DTHETA equals the effective porosity of the soil. The porosity of soil as a function of soil texture (percent of sand and percent of clay) is shown in Figure 4.5 (Brakensiek and others, 1984).

Under natural conditions, soil seldom reaches a state of soil moisture less than the wilting point of vegetation. Figure 4.6 is a graph of volumetric soil moisture at wilting point as a function of soil texture. Due to the rapid drainage capacity of most soils in Maricopa County, at the start of a design storm the soil would not be expected to be in a state of soil moisture greater than the field capacity. Figure 4.7 is a graph of volumetric soil moisture at field capacity as a function of soil texture.

Table 4.2
Green and Ampt Loss Rate Parameter Values for Bare Ground

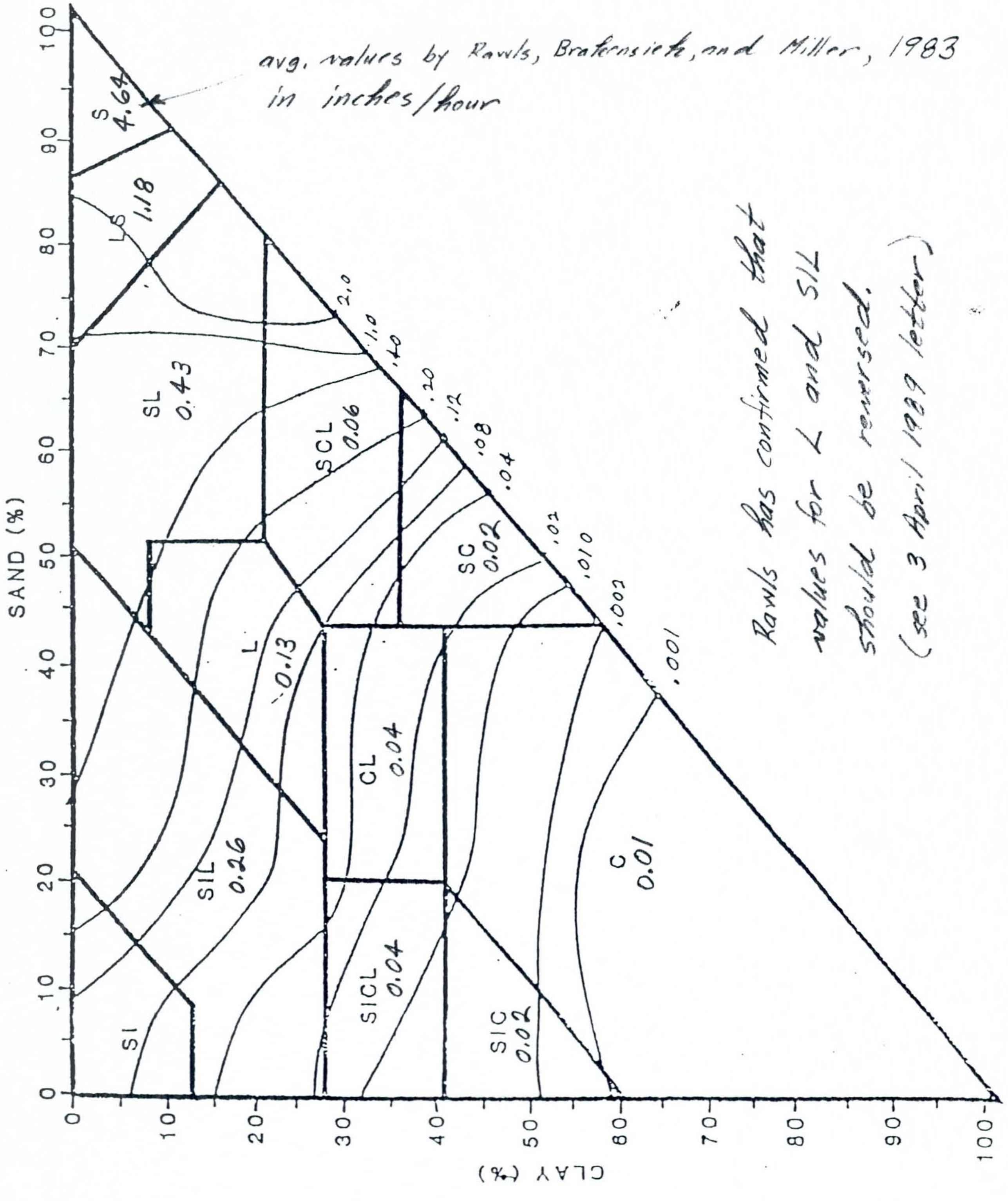
Soil Texture Classification (1)	XKSAT Inches/hour (2)	PSIF Inches (3)	DTHETA[1]		
			Dry (4)	Normal (5)	Saturated (6)
sand	4.6	1.9	0.35	0.30	0
loamy sand	1.2	2.4	0.35	0.30	0
sandy loam	0.40	3.5	0.35	0.25	0
loam	0.25	4.3	0.35	0.25	0
silty loam	0.15	6.6	0.40	0.25	0
silt	0.10	7.5	0.35	0.15	0
sandy clay loam	0.06	8.6	0.25	0.15	0
clay loam	0.04	8.2	0.25	0.15	0
silty clay loam	0.04	10.8	0.30	0.15	0
sandy clay	0.02	9.4	0.20	0.10	0
silty clay	0.02	11.5	0.20	0.10	0
clay	0.01	12.4	0.15	0.05	0

[1] Selection of DTHETA:
Dry = Nonirrigated lands, such as desert and rangeland;
Normal = Irrigated lawn, turf, and permanent pasture;
Saturated = Irrigated agricultural land.



Question - should the values for SL and L be reversed from those shown?

Figure 4.4
Green and Ampt Loss Rate
Wetting Front Suction in Inches (cm)
PSIF



Rawls has confirmed that values for L and SIL should be reversed.
(see 3 April 1989 letter)

Figure 4.3
Green and Ampt Loss Rate
Hydraulic Conductivity at natural saturation in inches/hour
XKSAT (0.5% organic matter)

TABLE 4.17 Green-Ampt Infiltration Parameters for Various Soil Classes

Soil Class	Effective Porosity DTHETA	Wetting Front Suction PSIF (in)	Hydraulic Conductivity XKSAT (in/hr)
Sand	0.417	1.95	4.64
Loamy Sand	0.401	2.41	1.18
Sandy Loam	0.412	4.33	0.43
Loam	0.434	3.50	0.13
Silt Loam	0.486	6.57	0.26
Sand Clay Loam	0.330	8.60	0.06
Clay Loam	0.309	8.72	0.04
Silty Clay Loam	0.432	10.75	0.04
Sandy Clay	0.321	9.41	0.02
Silty Clay	0.423	11.50	0.02
Clay	0.385	12.45	0.01

Source: CHOW, MAIDMENT AND MAYS [1988]

4.3 INFILTRATION WORKSHOP

1) Modify the LF.IH1 file to use the Initial and Uniform loss method, with the data listed in Table 4.18. Save the modified data set under file name **LFLUUSRS.IH1**.
2) Run HEC-1 using the **LFLUUSRS.IH1** file for input. Use **LFLUUSRS.OH1** as the output file name.
3) Tabulate the results in the correct location on Table 4.23.

TABLE 4.18 Rainfall Loss Data for Initial and Uniform Loss Method

SUBAREA:	1	2	3	4	5	6	7	8	9
INITIAL LOSS (in)	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.78	0.83
UNIFORM LOSS (in/hr)	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.13	0.14
PERCENT IMPERVIOUS	10	15	20	10	10	20	5	30	30

4) Modify the LF.IH1 file to use the SCS Curve Number loss method, using the data listed in Table 4.19. Save the modified data set under file name **LFLSUSRS.IH1**.
5) Run HEC-1 using the **LFLSUSRS.IH1** file for input. Use **LFLSUSRS.OH1** as the output file name.
6) Tabulate the results in the correct location on Table 4.23.

TABLE 4.19 Rainfall Loss Data for SCS Curve Number Loss Method

SUBAREA:	1	2	3	4	5	6	7	8	9
INITIAL ABSTRACTION	0.2*S	0.2*S	0.2*S	0.2*S	0.2*S	0.2*S	0.2*S	0.2*S	0.2*S
CURVE NUMBER	78	78	78	78	78	78	78	77	76
PERCENT IMPERVIOUS	10	15	20	10	10	20	5	30	30

7) Modify the LF.IH1 file to use the Holtan loss method, using the data listed in Table 4.20. Save the modified data set under file name **LFLHUSRS.IH1**.
8) Run HEC-1 using the **LFLHUSRS.IH1** file for input. Use **LFLHUSRS.OH1** as the output file name.
9) Tabulate the results in the correct location on Table 4.23.

City of Mesa

Engineering Division

LETTER OF TRANSMITTAL

FLOOD CONTROL DISTRICT
RECEIVED
JUN 05 '90

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REMARKS

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55 NORTH CENTER STREET • P.O. BOX 1466 • 85201-0904 • 834-2251

DATE 5-31-90

TO: Flood Control District
3335 W. Durango
Phoenix, AZ. 85009

RE: Hydrologic Design Manual

ATTENTION: Joe Rumann

GENTLEMEN:

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THE FOLLOWING ITEMS:

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☐ PRINTS ☐ CHANGE ORDER ☐ DETAILS ☐ ______
☐ PLANS ☐ ADDENDUM ☐ SPECIFICATIONS ☐ ______

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SIGNED: David Ramirez
TITLE: Senior Civil Engineer

Executive Summary

The objective of the *Hydrologic Design Manual of Maricopa County* is to provide technical procedures for the estimation of flood hydrology for the purpose of designing stormwater drainage facilities in Maricopa County. Two methodologies are defined for the development of flood hydrology; the Rational Method, and rainfall-runoff modeling using a design storm. For small, urban watersheds, less than 160 acres and fairly uniform land-use, the Rational Method is acceptable. Use of this method will only produce peak discharges and runoff volume and this method should not be used if a complete runoff hydrograph is needed, such as for routing through detention facilities. For larger, more complex watersheds or drainage networks, a rainfall-runoff model should be developed. The *Hydrologic Design Manual of Maricopa County* provides guidance in the development of such a model and the estimation of the necessary input parameters to the model. Although not necessarily required, the use of the U.S. Army Corps of Engineers' HEC-1 Flood Hydrology Program facilitates the use of the procedures that are contained in the *Hydrologic Design Manual of Maricopa County*. (The *Hydrologic Design Manual* was written to supplement the *HEC-1 Users Manual*.)

Expand upon or clarify this statement for cases where the Rational Method and not the Design Storm is used.

The *Hydrologic Design Manual* can be used to develop design hydrology magnitudes for storms of frequencies up to and including the 100-year event. The design storm is of 6-hour duration and that storm is to be used for the design of all stormwater drainage facilities except detention and retention basins. According to the *Uniform Drainage Policies and Standards for Maricopa County, Arizona* (February 25, 1987), all development shall make provisions to retain the peak flow and volume of runoff from rainfall events up to and including the 100-year, 2-hour duration storm falling within the boundaries of the proposed development. Accordingly, the criteria to be applied to the 2-hour storm is also provided in the *Hydrologic Design Manual*.

The rainfall-runoff modeling procedure that is contained in the manual is physically based, that is, the procedures are based—to the extent practical—on the physical processes that occur during the generation of storm runoff from rainfall. While the basic procedure is physically based this does not assure that the rigorous application of the procedures will, in fact, reproduce the actual rainfall-runoff phenomenon of any storm that has occurred or may occur in the future. However, the procedure,

2.3 Depth–Area Relation

The problem of spatial variability of rainfall is quite difficult to handle because of an irregular, limited network of rain gauges. Work in the southwest by the United States Department of Agriculture, Agricultural Research Service, indicates that high intensity storms do not have large areal extent. Most runoff producing thunderstorms south of Tombstone cover less than twenty square miles.

The above argument supports development of areal reduction curves which reflect the nature of the thunderstorms in the southwest. However, drainage facilities (storm drains, channels, and culverts) should be sized to handle the peak discharge resulting from the design storm critically centered above them to create the worst case discharge. Retention/detention facilities serving as an outfall for a small contributing area of up to 10 square miles would not appear to justify areal reduction of the depth. In all other applications, areal reduction seems appropriate for runoff calculations of contributing areas of any size.

See comment on p. 7, Table 2.1.

2.3.1 Procedure for Depth-Area Adjustments

Use the Depth-Area Reduction Curve developed for the historic storm of 1954 over the Queen Creek area (U.S. Army Corps of Engineers, 1974). This curve was developed for a major peak producing event within Maricopa County and should be representative of local conditions for design purposes.

a. Determine the size of the drainage area, and decide if areal reduction is necessary.

b. Use Section 2.4 to calculate depth for the design frequency.

c. If more than one isoline is shown over the drainage area, calculate average depth.

d. Use Figure 2.1 and Table 2.1 to select the reduction coefficient.

e. Multiply average rainfall by the depth reduction coefficient.

Table 2.1
Depth-Area Reduction Factors
for 6-Hour Duration Rainfall

Area, Square Miles	Ratio to Point of Rainfall
0	1.0
1	0.987
5	0.96
10	0.94
20	0.91
30	0.89
40	0.87
50	0.86
100	0.80
200	0.72
300	0.66
400	0.61
500	0.57

No area reduction per statement on p.5 Sec. 2.3?

2.4 Selection of Appropriate Design Storm

The design hydrologist must specify the appropriate rainfall frequency, duration, depth and the corresponding time distribution for any design purposes which require calculation of runoff volume and peak discharge. Application of the Rational Formula does not require a time distribution. The Hydrology Manual applies the NOAA procedures which led to the 100-year, 6-hour mass curve for small areas up to 0.5 $miles^2$. This mass curve is also known as Pattern 1, and will be discussed later. If a particular application requires that a mass curve should be developed, the following procedures (NOAA) or, alternatively, a program referred to as PREFRE by the National Weather Service can be used:

1. Using Figures 2.2 through 2.13, read rainfall depths for 2-, 5-, 10-, 25-, 50, and 100-year return periods for 6- and 24-hour durations, employing linear interpolation between isolines when required. The numbers on the isolines show tenths of inches of rainfall (i.e., 23 = 2.3 inches).

2. Plot the values from 1 for each duration on a separate line on Figure 2.14, look for any deviation from a straight line and make corrections on the line. This process will minimize any error due to transposition of values on the maps. Also, any error due to reading and interpolating values between the isolines will be minimized. Note that these numbers are already in partial-duration series, so there is no need for annual to partial-duration conversion.

FLOOD CONTROL DISTRICT of MARICOPA COUNTY 1959

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 30 1990

Mr. David Ramirez, P.E.
Senior Civil Engineer
City of Mesa
P. O. Box 1466
Mesa, Arizona 85301-0904

SUBJECT: Comments on Hydrologic Design Manual

Dear Mr. Ramirez:

Thank you for your timely response to our request for review and comment. The Special Projects Branch of the Hydrology Division has taken your comments into consideration, and offers the following responses:

1. You requested that we expand upon or clarify our position in the Executive Summary on cases where the Rational Method and not the Design Storm is used. The Statement in question will be changed as follows:

   "The design storm is of 6-hour duration and when utilized in the Maricopa County Unit Hydrograph Procedure, may be used for the design of all stormwater drainage facilities. In certain special cases, such as very small watersheds and detention/retention basins, alternative methods and rainfall distributions may also be used."

2. Table 2.1 and Figure 2.1 were developed from the 6-hour data for the 1954 Queen Creek storm, and we feel are not applicable to a 2-hour storm. Because we did not have a reliable aerial reduction curve for a 2-hour storm and because the majority of retention/detention basins have contributing watersheds less than 10 square miles, a policy decision was made to ignore aerial reduction when using the 2-hour, 100-year storm for retention/detention design.

We are proceeding with plans to have the Drainage Design Manual adopted by MAG and portions incorporated within municipal ordinances by April, 1991. Additional training sessions will be offered later this year.

Thank you again for attending our conference and for your comments. We will contact you when the final version of the manual is available. If you have any questions or comments, feel free to call me at 262-1501.

Sincerely,

J. M. Rumann
Hydrologist

TLB/JMR/ag

COORD: ~~DES~~, ~~SLS~~, 7/30 ~~DRJ~~, ~~JMR~~, ~~DDK~~, ~~SDW~~

INFO: ~~JMR~~ ~~FT~~, ~~GHD~~ CHD

FILE: JMR

City of Tempe
P.O. Box 5002
31 East Fifth Street
Tempe, AZ 85280
602-350-8371

Tempe

Public Works
Department

May 31, 1990

Maricopa County Flood Control District
3335 W. Durango St.
Phoenix, AZ 85009

ATTN: D.E. SAGRAMOSO, P.E. CHIEF ENGINEERING AND GENERAL MANAGER

RE: COMMENTS ON MCFCD HYDROLOGIC DESIGN MANUAL

Dear Mr. Sagramoso:

Thank you for the opportunity to review and comment on the Hydrologic Design Manual. You and your staff have put together a very impressive manual.

After reviewing the method for determining flows from a 100 year storm, the results are very similar to the Rational Method that the City of Tempe currently employs. The results are acceptable and the City will accept either method.

However, the County's retention requirements are a different story. Since the City adopted a storm drain ordinance in 1977, it has required new developments to retain the 100 year, one hour storm runoff. Additionally, variable "C" Factors are allowed only for sizing storm drains. A "C" Factor of .95 is required for calculating all retention quantities since roof area, paved parking areas and retention basins take up a large portion of a typical development. In a 100 year, one hour storm, the runoff from roofs and parking areas would hit the retention basin so hard and fast that percolation would be relatively insignificant, which justifies the .95 "C" Factor for retention basins. Where central storage basins are designed 1' of freeboard is also required.

In light of the fact that Tempe has very little vacant ground left to develop and since most of Tempe was developed using the above criteria, this criteria will remain in effect.

Since cities in the valley have already adopted MAG Details and Specifications, issuing the new manuals through MAG would make it much easier for cities to adopt the new manuals. Since each city has supplements to the MAG, it would also make it much easier for each city to add criteria peculiar to their area. This would result in more widespread usage of the manuals.

If you have any questions or comments, please call me at 350-8897.

Sincerely,

CITY OF TEMPE

James E. Bond, P.E.

James E. Bond., P.E.
Senior Civil Engineer

JEB:11

cc: Lee Quaas
Bill Coughlin

FLOOD CONTROL DISTRICT of MARICOPA COUNTY 1959

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 30 1990

Mr. James E. Bond, P.E.
Senior Civil Engineer
City of Tempe, Public Works Department
P. O. Box 5002
Tempe, Arizona 85280

SUBJECT: Comments on Hydrologic Design Manual

Dear Mr. Bond:

Thank you for your timely response to our request for review and comment. We are pleased that your review was a favorable one and that you will accept use of the design methods presented in the manual.

On the subject of "C" Factors for retention design, it seems we disagree on a specific design criteria. Our feeling is that in areas where rooftops, driveways, paved streets, etc., predominate, the user will calculate an appropriate "C" Factor from the guidelines presented. In areas where these types of impervious covers are not as widespread, the 0.95 "C" Factor presents an unrealistically conservative value. However, for the reasons outlined in your correspondence, use of this value in Tempe can be justified.

We are proceeding with plans to have the Drainage Design Manual adopted by MAG and portions incorporated within municipal ordinances by April, 1991. Additional training sessions will be offered later this year.

Again, thank you for attending our conference and for you comments. We will contact you when the final version of the Drainage Design Manual is available. If you have any questions, please feel free to contact Joe Rumann at 262-1501.

Sincerely:

STANLEY L. SMITH JR., P.E
DEPUTY CHIEF ENGINEER

D. E. Sagramoso, P.E.

TLB/JMR/ag

* See reverse for filing information.

FLOOD CONTROL DISTRICT
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REMARKS

11 May, 1990

Mr. Joe Rumann
Flood Control District of Maricopa County
Hydrologist III
3335 West Durango
Phoenix Arizona 85009

Dear Mr. Rumann;

I was recently asked to review the draft Maricopa County Hydrologic Design Manual. This request came from Mr. Jeff Erichson, who is with the W.L.B. group in Phoenix. Mr. Erichson sent me a copy of the manual, along with the computer program.

In general, I think the manual is a very good document and is one of the best county hydrology manuals that I have seen. The manual is well organized, it provides good information on the development of model parameters, and it also has some good examples. However, there are a few sections in the manual that could be improved. The following is a list of concerns that I had as I reviewed the manual:

1. Sections 2.4 and 2.5, which deal with the development of design storms, are not clear in describing what level of design storm (return period and duration of the event) should be used in specific types of studies. Specifically, section 2.5 states "The 2-hour storm is used for detention and retention design purposes". No return period is attached to this event (e.g. 10, 50, or 100 yr event). likewise, it is stated that the 6-hour event should be used for all other hydrologic analyses for watersheds up to 100 sq. miles. Again the return period associated with the event is not mentioned. Also, I have great concern over the statement that detention and retention facilities should be designed with a 2-hour event. Detention and retention facility design is much more sensitive to the volume of the hydrograph than the peak flow. Traditionally these types of facilities are designed for 24-hour events. This may be conservative for your area, but it is not unrealistic. If there are specific reasons for the 2-hour event, it should be stated clearly in section 2.5. I would suggest re-evaluating this criteria very closely. What ever is put in this manual will be used throughout your county. I would suggest the use of 24-hour storms for detention and retention facilities.

2. Section 5.5 deals with the development of coefficients to be used in the Clark Unit Hydrograph procedure. In this section, regression equations for T_c and R are shown. The data used to derive these equations is not presented. When ever it is suggested that model parameters should be derived from regression equations, the user should be fully aware of the limits of the equations. I would suggest that the data used to derive these equations be

summarized and published directly in the document. Also, a discussion of the limitations of the equations would be pertinent. I would also suggest that a discussion of model calibration be included, even though there is a limited amount of gaged data in that area. And finally I would provide alternative ways to calculate T_c. The method that I most often use is to map out the longest water course of each subbasin. Then break that into sections of overland flow, shallow channel flow, and main channel flow. Manning's equation can then be used to calculate bank full flow velocities, and a time of concentration would be the sum of the three travel times. This is documented in the most recent TR-55 manual from the SCS.

3. My last concern is in the area of channel routing. Of all the methods currently available in the HEC-1 computer program, I would suggest using the modified puls method. The best application of this method would be to develop storage-discharge relationships for each channel reach by using the HEC-2 backwater program. If cross-section data is available, storage-discharge information could be developed by running a range of discharges through HEC-2 (any flows ranging from low to high). Then this information could be used in HEC-1 to perform the channel routing with the modified puls method. Specific return period events could then be generated with HEC-1. The peak flows from those specific events could then be entered into HEC-2 in order to develop a flood profile of each event. Also, a new version of HEC-1 is going to be released this summer. In this new version, there is a new channel routing technique called Muskingum-Cunge routing. This method is physically based, in that it only requires a representative cross-section, channel slope, length, and an estimate of Manning's n values. The cross-sections can be either a simple prismatic shape (trapezoids, rectangles, and circular pipes) or an 8-point cross-section that has overbanks and a main channel. The method provides for hydrograph attenuation based on the physical cross-section properties and the magnitude of the inflowing hydrograph. I have compared it to models that use the full unsteady flow equations, and it compares very well over a wide range of channel configurations and flow situations.

If you have any questions about my comments or would like to discuss this with me further, please phone me at your earliest convenience (916-756-1104). I hope these comments will help you in the development of this manual.

Sincerely,

Gary W. Brunner

Gary W. Brunner M.S., P.E.

FLOOD CONTROL DISTRICT of MARICOPA COUNTY 1959

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 30 1990

Mr. Gary W. Brunner, M.S., P.E.
Department of the Army
The Hydrologic Engineering Center
The Corps of Engineers
609 Second Street
Davis, California 95616

SUBJECT: Comments on Hydrologic Design Manual

Dear Mr. Brunner:

Thank you for your letter of May 11, 1990, including your review comments on the Hydrologic Design Manual for Maricopa County. I appreciate your input and apologize for the delay in responding. We recently had a seminar introducing the manual to the community and asking for input. We are currently in the process of responding to those comments and including changes in the final draft. We anticipate that the final draft will be available by the middle of August.

In response to your list of concerns, we include the following discussion:

1. We have clarified Sections 2.4 and 2.5 to instruct use of the 100-year, 2-hour event for retention calculations and use of the 6-hour distribution, with the appropriate return frequency for all other applications. Certain criteria in the manual have been mandated by the Uniform Drainage Policies and Standards and the final draft of the Hydrologic Design Manual will reference these criteria as such.

2. The analyses conducted in Section 5.5 for the estimation of parameters will be presented in our Supporting Documentation Volumes to the Hydrologic Design Manual, which will be available by the end of this year. Guidance and limitations for the use of these parameters were included and expanded upon in the Applications chapter of the final draft. After an evaluation of various Tc equations, we selected a hydrologic equation over a hydraulic equation.

3. We recognize that there are other methods for routing available, and will accept their use if there is sufficient justification. However, for day to day use, we feel the methods presented are straightforward and somewhat conservative. Subsequent editions of the manual will more than likely include additional routing techniques.

If you would like to further discuss the data or methods used in the manual, please call me at 262-1501. Once again I'd like to thank you for your comments, which have been very helpful.

Sincerely,

J. M. Rumann
Hydrologist

TLB/JMR/ag

COORD: ~~DES~~, ~~SLS~~, 7/30 ~~DRJ~~, ~~JMR~~, ~~DDK~~, ~~SDW~~

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REMARKS

May 31, 1990

Flood Control District
of Maricopa County
3335 W. Durango
Phoenix, AZ 85007

ATTN: Mr. J.M. Rumann
Hydrologist

SUBJECT: Hydrologic Manual Review Comments

Gentlemen:

We appreciate the opportunity to attend the recent seminar on the draft Hydrologic Design Manual and to review the manual prior to its finalization. Several hydrologists within JMM have reviewed the manual, and we have found it to be clearly written and concise. We believe the manual will be a useful tool in hydrologic investigations within Maricopa County.

In the process of our review, we have developed comments for District consideration prior to finalizing the manual. These comments are provided below.

1. Pages 5 and 6 describe the use of depth-area reduction factors based on the 1954 Queen Creek storm. We would prefer use of depth-area reduction factors based on average regional conditions, rather than on a single storm. We believe that averaging storm histories from throughout the hydrologic region in which Maricopa County is located would have more likelihood of producing a "representative" or "typical" depth-area relationship than use of a single storm event. It is noted that the Queen Creek depth-area relationship gives values which are higher than those developed in the U.S. Weather Bureau publication "Hydro 40" or those used by the Corps of Engineers in Las Vegas Valley based on regional storm studies. Thus, the Maricopa County method will produce higher rainfall (and runoff) than would be produced by these other regional methods.

2. On page 29, use of the Rational Method is limited to drainage areas of 160 acres or less. Although this limitation is reasonable for a single-subarea analysis, many flood control agencies allow for use of a "modified Rational Method" for areas up to 1.0 square mile. The modified Rational Method provides an approach for developing a link-node system of subareas, each analyzed by the Rational Method, connected by flow paths (e.g., streets or channels). This method is easy to program and works well in urban areas. It is easily applied by development engineers and meets the District criteria of accuracy, practicality, and reproducibility. Various forms of this method are included in the Hydrology Manuals of San Bernardino County Flood Control District, Riverside County Flood Control District, Orange County Flood Control District, Los

Angeles County Flood Control District, and Clark County Regional Flood Control District. It is suggested that the District consider including the modified Rational Method in its hydrologic design procedures for urban areas up to 1.0 square mile.

3. Equation (2) and Table 3.1 on page 30 refer to the watershed resistance coefficient as K_b, but Figure 3.1 denotes this parameter as "n". It is recommended that Figure 3.1 be revised to agree with the text and table.

4. On page 35, it may be advisable to note in the text that equation (4) gives the 100-year, 2-hour hydrograph volume.

5. A general note regarding rainfall losses (Chapter 4) is that losses during flood events are not well known or documented; therefore, any modeling must necessarily lump numerous different and unrelated physical processes into only a few parameters. For this reason, loss rates are often a calibration tool when comparing simulated versus recorded streamflows and runoff volumes. Therefore, loss rates generated using any procedure are estimates only and subject to change based on available calibration data (if any).

6. On page 44, six reasons for rejecting use of the SCS Curve Number (CN) method of computing rainfall losses are given. The first four reasons are theoretical deficiencies; however, it could also be noted that all of the methods discussed have some inherent theoretical limitations due to the simulation of complex hydrologic processes with relatively simple mathematical expressions.

   Deficiency Number 5 suggests that selection of CN values is too subjective. However, use of tables published in numerous SCS reports and in other sources can provide reproducibility to this selection not unlike that for selection of Manning's "n" values for hydraulic analyses. In addition, it should be admitted that selection of parameters for any loss rate method is subjective given the lack of "real" data and the complex nature of the modeling problem.

   Deficiency Number 6 states that the estimate of rainfall loss is very sensitive to CN at low rainfalls. This condition is true for any loss method because the total rainfall is not much larger than the maximum soil infiltration capacity.

   We believe it may be premature to reject use of the popular CN method based on these reasons, given its widespread use in hydrology studies throughout the country and flood control agencies in the desert Southwest. The critical test is whether CN-based models can be calibrated to historical rainfall-runoff events in Maricopa County. Information in this regard was not included in the manual.

   An alternative approach would be to calibrate CN values to Green and Ampt simulations. This would involve determining a table of "modified" CN values for the Green and Ampt parameters, based on numerous simulations of different watershed conditions. If this could be accomplished, it would simplify the modeling process (since only one loss parameter would have to be estimated for model input), yet would provide results consistent with the desired Green and Ampt assumptions. However, one potential problem with this approach would be that a "modified" CN developed from

Green and Ampt parameters could become confused with a CN developed by the standard approach.

7. Regarding unit hydrograph procedures discussed in Chapter 5, it is noted that considerable recent research comparing various hydrologic modeling methods indicates that more complex modeling methods, even though "physically based", do not necessarily guarantee more accurate results. This is primarily due to the lack of calibration data and the need to describe complex processes with simple equations applied to lumped areas. Thus, the three-parameter Clark Unit Hydrograph Method will not necessarily be more accurate than the one-parameter S-Graph Method. This might justify use of the S-graph Method for areas smaller than the 5-square-mile limit given in the manual. It is noted that several other agencies use this method for areas as small as 1.0 square mile.

8. It may be beneficial to move the information provided on selection of proper Muskingum routing parameters in Section 7.6 into Section 6.3 The Chapter 6 information on this method is of limited help on its own.

9. It is suggested that a section on model calibration be added to the manual. This step is critical for use of any hydrologic model to assure reasonable results. Even if actual calibration data are not available, guidance on reasonable discharge values based on other studies (e.g., typical cfs per square mile, acre-ft per acre, etc.) would assist the user in knowing whether model results are "in the ballpark".

10. MCUHP1 and MCUHP2 greatly simplify the formulation of a HEC-1 input file. It should be noted in the program documentation and in the Hydrology Manual that the most recent version of HEC-1 must be used (June 1988). Earlier HEC-1 versions do not contain the Green and Ampt loss rate parameter option, and the use of Green and Ampt with an earlier version will cause the program to abort. A user who is unfamiliar with the HEC-1 program and its most recent update would find it difficult to discover why a successful run could not be obtained.

Thank you again for the opportunity to review the draft manual. We hope you find our comments useful. Should you have any questions on these comments, please do not hesitate to contact us.

Very truly yours,

Fred K Duren, Jr.

Fred K. Duren, Jr., P.E., P.G.
Acting Regional Office Manager

FLOOD CONTROL DISTRICT of MARICOPA COUNTY 1959

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 30 1990

Mr. Fred K. Duren, Jr., P.E., P.G.
James M. Montgomery
Consulting Engineers, Inc.
6245 North 24th Parkway, Suite 208
Phoenix, Arizona 85016

SUBJECT: Comments on Hydrologic Design Manual

Dear Mr. Duren:

We would like to thank your staff for attending the seminar on the draft Hydrologic Design Manual and for your thorough review of the draft copy. Your comments have been quite useful and are being addressed individually below, as follow:

1. You suggested that a depth-area relation based on the average of storm histories would be more "representative" or "typical" for Maricopa County. This is a practical approach if the data base is extensive enough to include a variety of storm events over several years. However, we have found that, due to data limitations, an average depth-area relation may result in "under-designing" of projects. In addition, design hydrology should be flexible for application to different watershed sizes. Since our design rainfall was based on the 1954 Queen Creek storm and since this storm was the most extreme local event in Maricopa County, it was used to develop the depth-area relation. A comparison with the "Hydro 40" may not be appropriate since the database used in "Hydro 40" was not from Maricopa County.

2. Application of Rational Method for drainage areas of up to 160 acres was an administrative decision policy for peak flow calculations and retention/detention requirements. Furthermore, the policy has required application of a hydrograph procedure and, thus, a rainfall distribution input for areas larger than 160 acres. As a result, the "Modified Rational Method" cannot be considered.

3. Corrections will be made to the final draft.

4. Such notation will be made in the final draft.

5. Correct.

6. The six reasons for our not choosing the SCS Curve Number (CN) method were included only to emphasize the need for a more physically-based approach. Although lack of data has a major impact on loss computations, application of a physical model, i.e., Green-Ampt, would describe the process in a more realistic fashion. We are revising the discussion on the SCS Curve Number method to avoid future confusion.

7. Due to lack of data, comparison of a physically-based method, such as the Clark Unit Hydrograph, with that of a one-parameter S-Graph is difficult to achieve. However, since the Clark procedure offers a better description of the individual variables which impact runoff, it should be considered as an improvement over the S-Graph approach.

8. Will note.

9. Good suggestion.

10. Thank You!

Again, we thank you for attending the seminar and also for your comments. If you have any further questions, please do not hesitate to call me at 262-1501.

Sincerely,

J. M. Rumann
Hydrologist

TLB/JMR/ag

COORD: ~~DES~~, SLS, 7/30 DRJ, JMR, DDK, SDW

INFO: ~~JMP~~, ~~FT~~, CHD

FILE: JMR

George V. Sabol Consulting Engineers, Inc.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

13 August 1990

Mr. Joe Rumann
Hydrologist
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Subject: Hydrologic Design Manual for Maricopa County review comments by Mr. Fred K. Duren, Jr., P.E., P.G., James M. Montgomery Consulting Engineers, Inc.

Dear Mr. Rumann:

I have reviewed the comments of Mr. Duren in his letter of 31 May 1990, and the Flood Control District's response in its letter to Mr. Duren of 30 July 1990. I agree with the District's response, and I offer the additional response comments that you may wish to consider.

1. In regard to Mr. Duren's comment 6, it is not likely that the CN method could be calibrated to Green and Ampt simulations and give equivalent results. The two methods are not amenable to such calibration, and would probably result in considerable confusion.
2. In regard to Mr. Duren's comment 7, I believe that the "recent research comparing various hydrologic modeling methods" was probably a comparison of kinematic wave modeling as compared to unit hydrograph methods. The three parameter Clark unit-hydrograph may not be more accurate than the one parameter S-Graph (then again, it may), but the three parameter method is certainly more sensitive to watershed changes, and that was a major factor in selection of the Clark unit hydrograph method.

Mr. Duren's comments and corrections are appreciated. I hope that this response of mine is of assistance to the District.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo Sabol

George V. Sabol

JERRY R. JONES & ASSOCIATES, INC.
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June 8, 1990

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Thomas J. Lute, R.L.S.

Mr. Joe Rumann, Hydrologist
Branch Supervisor
Special Project Branch
Hydrology Division
Flood Control District of Maricopa County
3335 W. Durango Street
Phoenix, Arizona 85009

RE: Drainage Design Manual KOL I
Hydrologic Design Manual
JJA Job No. 290-018

Dear Joe,

Roger Baele and I appreciate the opportunity to participate in the Promulgation Conference for the Hydrologic Design Manual. I felt that the best way to familiarize myself with the new hydrology manual would be to work through some examples. JJA is finalizing an Area Drainage Master Study on the Apache Wash for the City of Phoenix. In this study six different computer models/lag relationships were tested to see which would generate 'reasonable' runoff values. Model I.2, which utilized the Pima County time of concentration method with the SCS HEC 1 hydrograph was chosen because it consistently produced the highest runoff values.

Every effort was made to use comparable data values from model to model but, there was some minor descripencies inherent between the models. The 100 year 24 hour storm with a type IIA rainfall distribution was used for the modeling. The attached Table III summarized 3 basins which were studied.

I used this basin data and Clark Unit Hydrograph from the FCD new hydrology method for the computation of runoff for the three basins. The Green and Ampt, IL and ULR soil texture and the IL and ULR hydrologic soil group rainfall loss methods were used for each basin and compared to the Runoff Summary Table from the Apache Wash Report. The results of my computations are listed here.

	Basin 12			Basin 30			Basin 50		
	LAG	VEL.	Q	LAG	VEL.	Q	LAG	VEL.	Q
G&A	0.26	13.3	4,268	0.32	10.6	718	0.40	12.2	1,613
Soil Texture	0.25	13.9	4,259	0.32	10.6	707	0.40	12.2	1,562
HSG	0.26	13.3	4,494	0.32	10.6	751	0.40	12.2	1,560

Corporate Office: 3845 N. Business Center Dr. • P.O. Box 1830 • Tucson, AZ 85702-1830 • (602) 292-2160 • FAX 292-9131
Anchor Centre Two: 2207 E. Camelback Rd. • Suite 302 • Phoenix, AZ 85016-3446 • (602) 956-9850 • FAX 956-9853

*These methods use the time of concentration term and are converted to a 'LAG' by multipling by 0.6 TL=0.6 TC. Velocity is basin length/LAG.

The computed flow (FCD) was substantially higher than Model I.2 for Basin 12 which also has rainfall intensity term in its time of concentration relationship. The higher flow values (3775 CFS - 4,268 CFS to 4,494 CFS) had the same lag time values which may be caused by the difference in losses produced by using RCNs and losses used in the FCD hydrology manual. Basin 12 consists of mountainous and flatter desert areas.

Basin 30 computed runoff values (FCD) were very similar to Model I.2 while the lag values were less (FCD). This basin is an 'average' undeveloped desert watershed. The computed values (FCD) were somewhat higher with an accompanying lower lag time as one would expect. However, the velocity values seem a little excessive when compared to Model I.2.

My feeling from working with this manual (although briefly) is that it is fairly sensitive to the slope of the watershed. This may or may not be of consequence since I do not have a good feel for the value differences of the RCN and the FCD loss values. The differences in the flow value may not be related to these parameters. The following is a table which shows that for the FCD IL and VLR HGS for example, the rainfall excess is less than Model I.2 even the peak flow of the FCD calculations is higher.

	Basin 12			Basin 30			Basin 50		
	LOSS (IN)	EXCESS (IN)	Q (CFS)	LOSS (IN)	EXCESS (IN)	Q (CFS)	LOSS (IN)	EXCESS (IN)	Q (CFS)
Model I.2	1.47	3.13	3,775	1.65	2.95	708	1.64	2.96	1,229
HGS	1.84	2.76	4,494	1.96	2.64	751	1.94	2.66	1,560

Since the flow values are larger with less excess runoff, the difference must be in the unit hydrograph relationships and probably the time-area distribution for the Clark Unit Hydrograph. I used the HEC 1 default time-area distribution because the manual provided only 6-hour distributions. I am not sure if these differences will be of significance if a 6 hour storm was used for the Apach Wash Study.

I did like the Green and Ampt Loss Method from a conceptual standpoint, the ave. excess intensity varied through the peak of the runoff where the soil texture, and HSG methods had at least two time periods which did not change. Intuitively, there should be some changes especially near the peak.

Overall the manual was pretty clear and easy to follow. The computations are lengthy compared to existing methods. One suggestion would be to include a soil loss summary table in the manual. It will help guide the user to fill in the proper blanks, and also help the reviewer to receive information in a standard format. The left hand side of the soil data table needs to be rotated 180 degrees so the user could read both sheets from one orientation. I had to turn the book around several times to read the tables, hey how often does a private sector person get an opportunity to knit pick the public sector.

Please review my computations and assumptions about the higher flow values and if you need any other information or clarification please call me. Again, thanks for seminar and opportunity to comment on this manual.

Sincerely,
JERRY R. JONES & ASSOCIATES, INC.

Pet Mill

Peter S. Miller, P.E.
Project Manager

PSM:jee
290-018c/PHX-1

Computer Model Verification

The results of this model were verified through independant hydrologic calculations. The hydrologic models used for the verifications are:

Model I.1) HEC 1 Soil Conservation Service (SCS) Hydrograph SCS Lag Method
Model I.2) HEC 1 SCS runoff model, (utilizing Pima County Department of Transportation (PCDOT)- Time of Concentration Method)
Model I.3) The PCDOT hydrology method

Model I.4) U.S. Army Corps of Engineers (USACOE) Flood Data Phoenix area (Empirical Data)
Model I.5) U.S. Department of Agriculture (USDA) Walnut Gulch Regional Flood Frequency Relationships
Model I.6) U.S. Bureau of Reclamation/Flood Control District of Maricopa County (Preliminary Hydrology Manual) S Graph Method.

Model I.1 was used as a basis for comparison. The first method resulted in the lowest flow values. The HEC I, PCDOT Model I.2 method produced the highest results. The next highest was I.3 PCDOT which produced flow values an average of about 25% less than Model I.2 HEC I PCDOT. This was followed closely by Models I.6, I.5 and I.4. The SCS HEC 1 (PCDOT) (Model I.2) was chosen to model the existing hydrology conditions because it produced the highest flow values. This method yielded travel velocities in the order of 9 feet/ second which consequently produces slightly higher flow values. Table III summarizes the results of this study. See Figure 2 for a plot of these computed values.

The flow values quantified by the hydrology analysis were used to develop approximate floodplain boundaries for washes with flow values above 1,000 CFS as requested by the Flood Control District of Maricopa County (FCDMC). These approximate floodplain limits closely match the reports of flooding in the area. The majority of the development in the watersheds is above Carefree Highway, within Maricopa County.

TABLE III
RUNOFF SUMMARY

MODEL	Basin 12 (DA=1.89 Sq. Mi.) LAG (hrs)	VEL. (fps)	Q (cfs)	Basin 30 (DA=0.5 Sq Mi.) LAG (hrs)	VEL. (fps)	Q (cfs)	Basin 50 (DA=1.15 Sq. Mi.) LAG (hrs)	VEL. (fps)	Q (cfs)
I.1) HEC1 (SCS)	0.72	4.8	2,023	1.35	2.5	315	0.78	6.2	445
I.2) HEC1 (PCDOT)	0.26	13.4	3,775	0.39	8.7	718	0.66	7.4	1,229
I.3) PCDOT	0.29	12.0	2,495	0.34	10	595	0.66	7.4	876
I.4) USACOE	N/A	N/A	1,417	N/A	N/A	500	N/A	N/A	920
I.5) USDA	N/A	N/A	1,360	N/A	N/A	600	N/A	N/A	1,014
I.6) USBR/FCDMC	0.61	5.7	2,430	0.57	6.0	577	0.80	6.1	1,076

Basin 12 consists of steep mountainous and flat valley sections. Model I.1 uses an average slope value 0.07 ft/ft. and the others use a weighted slope value. The flow computation for Model I.1 is artificially high and should compare with relative value differences as shown by basins 30 and 50 for the same methods. The runoff value for Model I.1 would be close to 1200 CFS if a weighted slope value is used in the lag calculation. This method does not account for variable slope components in the watershed. Velocity - Basin Length/Lag Time.

Model I.1 HEC1 SCS Lag=i**0.8 (S+1)**0.7/1900y**0.5 Where i=Hydraulic Length S=1,000/CN-10 y=Slope in %: (see reference 18, page 19)

Model I.2 & I.3 HEC1 (PCDOT) - Uses Lag (TL) Calculated From (PCDOT) Hydrology Method and TL=0.6 TC

Model I.4 USACOE - Regression Analysis See Plot in Figure No. 2.

Model I.5 USDA-Walnut Gulch - Regression Analysis From Flood Frequency Estimates in Southeastern Arizona by Boughton, Renard, Stone 1987 Q100 Env.=31.60**(0.77-0.00664LOGD) Where D=Drainage Area in Kilometers Square. QENV=Meters **3/sec

Model I.6 USBR/MCFCD Lag=26(n)(LcxLca)/(S)**1/2),**0.33,S.in Ft/Mile

FLOOD CONTROL DISTRICT of MARICOPA COUNTY 1959

FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 31 1990

Mr. Peter S. Miller, P.E.
Jerry R. Jones & Associates, Inc.
Anchor Centre Two
2207 E. Camelback Road, Suite 302
Phoenix, Arizona 85016-3446

SUBJECT: Comments on Hydrologic Design Manual

Dear Mr. Miller:

Thank you for attending the seminar on the draft Hydrologic Design Manual and for your review comments/comparison of the methodologies. In response, we would like to call your attention to the following:

1. You indicated that the methods outlined in the manual provided substantially higher peak values when compared with your model I.2. This is mainly due to the selected rainfall distribution. The SCS Type-IIA was not recommended in the manual since it does not account for rainfall intensity as a function of the drainage area. Based on available data, rainfall intensity decreases as the watershed size becomes larger. For this reason, the rainfall pattern distributions were developed.

   Furthermore, the 24-hour duration of the SCS Type-IIA distribution is more indicative of a general storm. Data for Maricopa County indicates that the peak producing events, typically occur during the monsoon season and do not exceed a 6-hour duration.

   We anticipate that the 6-hour pattern distribution will result in lower peak values if applied to your project. It should be noted that since there is no actual way to verify the peak runoff, a more conservative value is not necessarily unjustifiable.

2. The velocities calculated in your example appear to be rather high when the methods recommended in the manual are used. This is attributed to the method used for velocity calculation, i.e., Vel. = length/LAG. The term LAG is defined as the time from the center of mass of rainfall excess to the peak of the hydrograph, and normally is not used to explain travel time of water. Time of concentration, Tc, is sometimes used for this purpose. However, Tc by itself is not representative of travel time either, as the calculated velocity should be adjusted to account for wave propagation speed.

3. You indicated that the methods outlined in the manual are fairly sensitive to the slope of the watershed. This is a good observation, since every attempt has been made to make the manual sensitive to the hydrologic characteristics and physical conditions of the watersheds.

4. A comment was made regarding the use of time-area distributions with the 6-hour rainfall distribution. Please note that a time-area curve is only a function of watershed geometry and can be used with any rainfall distribution regardless of the duration.

5. You mentioned that the manual methods are time consuming and tedious. Software programs MCUHP1 and MCUHP2 were developed for the purpose of simplifying the analyses.

We would like to thank you for participating in the seminar and for your review comments. If you have any further questions, please call me at 262-1501.

Sincerely,

J. M. Rumann
Hydrologist

TLB/JMR/ag

COORD: ~~DES~~, ~~SLS~~ 7/30 ~~DRJ~~, ~~JMR~~, ~~DDK~~, ~~SDW~~

INFO: ~~JMP~~, ~~FT~~, ~~CHD~~ CHD

FILE: JMR

GEORGE V. SABOL CONSULTING ENGINEERS, INC.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

13 August 1990

Mr. Joe Rumann
Hydrologist
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Subject: Hydrologic Design Manual for Maricopa County review comments by Mr. Peter S. Miller, P.E., Jerry R. Jones & Associates, Inc.

Dear Mr. Rumann:

I have reviewed the comments of Mr. Miller in his letter of 8 June 1990, and also the Flood Control District's response in its letter to Mr. Miller of 30 July 1990. I agree with the District's response, and I offer the additional response comments that you may wish to consider.

1. Using the peak discharges from Mr. Miller's letter and comparing the results of Model I.2 to the results designated G&A, I note the following:

Basin	Ratio of Peak Discharges (I.2/G&A)	% Increase
12	4,268/3,775 = 1.13	13
30	718/708 = 1.06	6
50	1,613/1,229 = 1.27	27

   Considering the significant differences in model input; that is, rainfall losses and unit hydrographs, the differences in magnitudes of peak discharge are not surprising. The SCS Type II distribution was used in all comparisons and it would be interesting to see how the results would compare if the entire procedure from the manual, including design rainfall criteria, were used.

2. On page 2, second paragraph from the bottom, is stated "the difference must be in the unit hydrograph relationships." It should be noted that the difference can be due to differences in intensity of rainfall excess. The Green and Ampt infiltration equation usually results in higher rainfall excess intensities as compared to the CN method.

Mr. Miller's comparison to results by several methods has been informative and generally reassuring. I hope that this response of mine is of assistance to the District.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo Sabol

George V. Sabol

AGK ENGINEERS, INC.
2255 N. 44th St. Suite 330 • Phoenix, Az. 85008 • Phone (602) 244-2566

Edward A. Adair, P.E.
R. Gerald Green, P.E.
Samuel E. Kao, Ph.D., P.E.

May 30, 1990

Mr. Dan E. Sagramoso, P.E.
Chief Engineer and General Manager
Flood Control District of Maricopa County
3335 W. Durango
Phoenix, Arizona 85009

Reference: Hydrologic Design Manual

Dear Mr. Sagramoso:

Thank you for inviting us to attend the symposium of the Hydrologic Design Manual.

After carefully reviewing the manual, we feel that the manual provides significant improvement to the technical procedures for the estimation of drainage hydrology.

We look forward to applying this manual to our drainage projects.

Very truly yours,

AGK ENGINEERS, INC.

Samuel Kao

Samuel E. Kao, Ph.D., P.E.
Vice President

SEK/ap

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of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager

JUL 30 1990

Samuel E. Kao, Ph.D., P.E.
Vice President
AGK Engineers, Inc.
2255 North 44th St., Suite 330
Phoenix, Arizona 85008

SUBJECT: Comments on Hydrologic Design Manual

Dear Dr. Kao:

We would like to thank you for attending the seminar on the Hydrologic Design Manual and for your recent correspondence. We are happy to hear that you feel the manual will improve the technical procedures currently being used for hydrological analyses.

If you have any questions regarding the manual, please do not hesitate to call Joe Rumann of my staff at 262-1501.

Sincerely,

D. E. Sagramoso, P.E.

TLB/JMR/ag

COORD: ~~DES~~, ~~SLS~~, 7/30 ~~DRJ~~, ~~JMR~~, ~~DDK~~, ~~SDW~~

INFO: ~~JMR~~, ~~FT~~, ~~CHD~~ CHD

FILE: JMR

MFR: This letter was written in response to the above-referenced comments that were submitted for review.