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Estimating Sediment Delivery and Yield on Alluvial Fans

Technical Paper No. 130

June 1990

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ESTIMATING SEDIMENT DELIVERY AND YIELD ON ALLUVIAL FANS

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Abstract

This paper summarizes the procedures used for computing the basinwide annual yields and single event sediment production for ephemoral channels located on an incised alluvial fan in Central California. Unique geomorphic characteristics of the basin and alluvial fan are discussed in light of data and analytical methods necessary to compute sediment delivery and yield at a proposed damsite.

Introduction

A Sediment Engineering Investigation (SEI) of the Caliente Creek watershed (470 sq. mi.) in Kern County, California was conducted to determine the watershed sediment yield upstream from a proposed flood detention reservoir located on the Caliente Fan. Previous studies estimated annual sediment yields at the proposed reservoir site based on traditional soil loss methods and sediment accumulation rates observed in impoundments along the Sierra Nevada, Tehachapi and Transverse Mountain Ranges. Initial project feasibility was considered based on preliminary cost/benefit analyses using the rough sediment yield estimates. Further review of the potential annual maintenance requirements led to the conclusion that the economic viability of the project depended heavily on annual O & M costs potentially required to remove the yearly accumulation of sediment within the proposed reservoir. Accurate estimates for the average annual sediment yield and single event sediment delivery were essential.

Further studies were undertaken to (1) identify specific geomorphic characteristics of the stream channels and watersheds upstream from the proposed flood control reservoir that could effect the sediment yield at the damsite, and (2) to relate channel and basin processes to sediment production and yields for various frequency precipitation and flood flow events in the watershed. This paper summarizes the procedures used for computing the basinwide annual yields and single event sediment production, along with conclusions and recommendations for other project design modifications.

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Paper prepared for the International Symposium on the Hydraulics/Hydrology of Arid Lands, ASCE Hydraulics Division, July 30-August 3, 1990, San Diego, CA

Approach

A two element SEI was conducted to address the sediment yield question: (1) geomorphic analyses (Harvey et al., 1990) were conducted to determine those unique characteristics of the basin and channels important to estimating sediment yield, and (2) sedimentation analyses (HEC, 1990) were conducted to determine the sediment yield in light of the findings from the geomorphic analyses.

To determine the amount of sediment that can possibly enter the proposed reservoir during its design life (100 years), both the average annual sediment yield and single event sediment yields are estimated using a variety of sediment engineering procedures as reported in EM 1110-2-4000, "Sediment Investigations of Rivers and Reservoirs," (COE, 1989) and recommended by others. Available scientific and engineering literature was reviewed, a three-day field reconnaissance and sediment data collection investigation was conducted, persons familiar with the Caliente Creek Project and watershed were interviewed, and a series of sediment engineering analyses to determine the possible sedimentation characteristics of the drainage basin at the damsite were carried out. Morphometric data for the alluvial fan in the vicinity of the proposed reservoir site were obtained from 2-foot contour mapping. Sixteen bed and bank material samples and two Wolman Counts were collected at representative locations throughout the drainage basin.

Average Annual Sediment Yield - The possible range of average annual sediment yield at the proposed reservoir site is estimated from the results from eight different sources of data and/or methods for estimating sediment yield. The following sources of data and procedures were used: (1) Previous reports and publications were thoroughly reviewed, (2) U.S.D.A. (1977) reservoir sedimentation rates were examined, (3) recent COE reservoir sedimentation survey data were analyzed, (4) sediment yield maps for the Western United States (U.S.D.A., SCS, 1975) were examined, (5) the average annual sediment yield was estimated from computations of the total event sediment volumes for single events ranging from the 2-year event up to the PMF based on channel transport capacity rather than watershed sediment production and delivery, (6) a similar flow duration and sediment load curve integration method (see EM 1110-2-4000, COE, 1989) was used to estimate the average annual sediment production and yield to the reservoir site, (7) the Pacific Southwest Inter-Agency Committee (PSIAC) method was used to estimate basin-wide sediment yield from the entire watershed, and (8) the Dendy and Bolton (1976) Regional Analysis Method for sediment yield was applied. Results from these analyses are discussed next. Detailed procedures for conducting such investigations are presented in the references cited and in Engineering Manual 1110-2-4000 (COE, 1989).

Table 1 presents the estimated sediment yields computed using the various computational procedures listed above and from measured reservoir surveys conducted by the Corps of Engineers and SCS. Based on measured sediment accumulation rates recorded in the six Tulare, Kings, and Kern County reservoirs, the approximate range of observed sediment yields is from 0.2 AF/sq mi/yr to 2.2 AF/sq mi/yr with an average of approximately 1.0 AF/sq mi/yr. Sediment yield rates determined for the Western United States are reported by the U.S.D.A., SCS (1975). From the mapping of yield rates, it appears that the upper Caliente watershed area has sediment yield rates from 0.2 to 0.5 AF/sq mi/yr, with pockets as high as 0.5 to 1.0 AF/sq mi/yr. In the lower portions of the basin, on the valley floor and on portions of the broad alluvial fan, the estimated yields are reported to be in the 0.1 to 0.2 AF/sq mi/yr range. Using area weighting methods to sum the yields from contributing subbasins, the approximate annual yield appears to range from 0.2 to about 0.75 AF/sq mi/yr, with an average of about 0.47 AF/sq mi/yr for the entire watershed.

Harvey et al., (1990) determined that the sediment delivery and yield at the damsite depends on the channel transport capacity in the fan area upstream from the reservoir rather than the watershed production of sediment. The broad (3,000 to 6,600 feet wide) alluvial fan contains an unlimited supply of easily mobilized sediment materials. This result lead to the following approach based on the transport capacity of the channels in the supply reach. The supply reach is a 4-mile section of the channel considered to be representative of the channel hydraulic conditions and sediment transport characteristics

TABLE 1

Sediment Surveys for Reservoirs in the Vicinity of Caliente Creek, Kern County, California, and Estimated Sediment Yields Based on Various Computational Methods

Data Source	See References	Drainage Basin, Reservoir or Computational Method Used	Drainage Area (sq mi)	Yield (AF/sq mi/yr)
SCS	10	Blackburn	7.1	2.20
SCS	10	Antelope Canyon	4.4	1.50
CESPK	5	Isabella	2,074	0.37
CESPK	9	Pine Flat	1,542	0.20
CESPK	9	Success	393	0.76
CESPK	9	Terminus	560	0.75
SCS	8	SCS Yield Map of Western US (HEC)	470	0.47
Computed	7	Integration of the Event Volume vs. Frequency Curve (HEC)	470	0.55
Computed	7	Flow Duration Method (HEC)	470	0.90
Computed	7	Dendy & Bolton Method (HEC)	470	0.71
Computed	4	PSIAC Method (HEC)	470	0.75
Computed	6	Kern County Water Agency Study (SLA)	470	0.97

upstream from the dam site. Single event total sediment volumes were computed for each of the 2, 5, 10, 20, 50, 100, SPF, and PMF events. The total sediment production for each event was based on the sediment transport capacity of the alluvial channel (supply reach) upstream from the reservoir and the flow hydrographs used for each of the flood events evaluated.

A total sediment load versus percent exceedance curve was developed from these data and the area under the total load frequency curve was computed to give an estimate for the expected average annual sediment delivery to the reservoir based on channel transport capacity upstream from the reservoir. Two different transport relationships were used to develop the total load curves. The resulting average annual sediment delivery ranged from 0.1 AF/sq mi/yr to 1.0 AF/sq mi/yr due to the difference in transport capacity computed with the transport functions. Using these results as a representative range in expected yields based on channel capacity, an average of the two yields seems reasonable. Therefore, based on the channel transport capacity above the reservoir site and the estimated total sediment production from a range of single events, an approximate sediment yield at the reservoir is 0.55 AF/sq mi/yr. This method does not account for the additional contribution of sediment from dry ravel erosion, wind-blown sand transport into the channel or reservoir, channel bank caving, local scour, or toe failure that may occur along the Sand Hills. Therefore, the sediment yield to the reservoir may be as high as the higher of the two transport functions predicts, especially during periods of exceptionally wet years.

The "flow duration sediment discharge rating curve method," (COE, 1989) is a simple method where the flow duration curve is integrated with the sediment discharge rating curve developed for the damsite. It is very similar to the method just described, however, the average annual sediment yield is based on the transport capacity and flow duration relationship at the damsite rather than the total event volume frequency. The resulting annual sediment yield is approximately 438 AF/year, or 0.9 AF/sq mi/yr.

Further examination of the U.S.D.A., SCS (1975) "Sediment Yield Rates for the Western United States" shows areas in the vicinity of the proposed damsite with estimated yields from 0.5 to 1.0 AF/sq mi/yr. These areas may correspond to the broad floodplain channels (4000 to 6500 feet wide) immediately upstream from the proposed reservoir site. If that is the case, then the higher yield values estimated with the channel transport capacity method (1.0 AF/sq mi/yr) and the flow duration method (0.9 AF/sq mi/yr) are supported by SCS yield mapping estimates.

The Dendy and Bolton (1976) method produces an average annual sediment yield of approximately 0.71 AF/sq mi/yr for the Caliente Basin at the Sivert damsite, while the application of PSIAC procedures to the Caliente Creek watershed produces an estimated average annual sediment yield of 0.75 AF/sq mi/yr at the dam site. These values are right in line with the range of values predicted from the channel capacity approach and the measured reservoir accumulation results from Tulare County.

Others (Simons, Li & Associates, 1989) conducted an independent assessment of the proposed Caliente Creek Project. The authors report the arithmetic average of their yield estimates (0.97 AF/sq mi/yr) in Table 1. Figure 1 shows all thirteen yield values and the drainage basin area associated with each yield. A best fit line through these data points gives an average annual sediment yield of 0.75 AF/sq mi/yr. This is more than twice the original annual estimate.

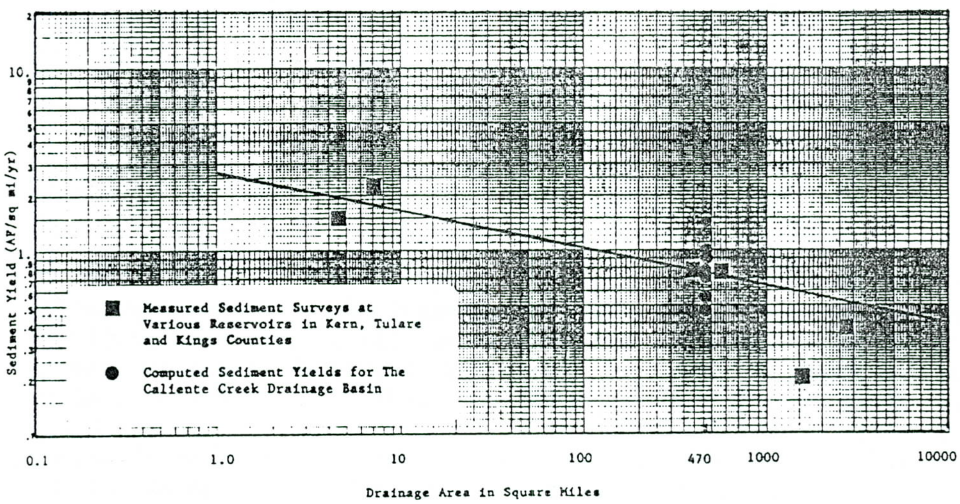

Figure 1

Measured and Computed Values of Average Annual Sediment Yield Versus Drainage Basin Area

It is important to note that arid and semi-arid basins, such as Caliente Creek, are very episodic in nature. During dry years (perhaps even normal years) the sediment production and delivery (and, therefore, annual yield) is small. During large runoff events the sediment production and delivery can produce tremendous loads of sediment in the channels. The annual yield during an excessively wet year can be quite high. Therefore, the presentation of a single average annual yield value may be misleading. For planning purposes, the consideration of the range of possible annual yields is more meaningful.

Single Event Analyses

In addition to the average annual sediment yield, it is important to estimate the sediment production and delivery from possible single events ranging from small 5-year flows to the design event (100 year flood) and, perhaps the SPF and PMF. It is possible that one or more single events during the design life of the project can significantly affect the operation and maintenance of the reservoir.

The study reach upstream from the proposed damsite was partitioned into four different zones or subreaches based on distinct hydraulic and geomorphic characteristics. The transport capacity is computed for each reach and is compared to the others with different hydraulic and geomorphic characteristics. The channel averaged sediment grain size and averaged channel hydraulic conditions for a range of discharges are used with several different total bed material load transport functions to develop representative water discharge versus total bed material load relationships for each of the subreaches and flow conditions.

Table 2 presents the computed sediment inflow to the proposed damsite for the various flood events. The 100 year flood event can possibly produce enough sediment during the single design event to remove 43.7 percent of the gross pool storage capacity (6992 AF). It also suggests that events greater than about the 15 year event can possibly remove 10 percent or more of the gross pool storage in one 5 day period. This indicates that the present design capacity of the reservoir may be undersized. The computed total sediment loads account for the total bed material load with an additional 15 percent estimated for the wash load. Typical wash loads can account for as much as 90 to 95 percent of the total load in most sand bed rivers (Vanoni, 1975). However, in the Caliente River Basin the availability of fines (silts and clays) may be limited due to the nature of the granitic parent materials throughout the basin (see Harvey et al., 1990). The authors postulate that the wash load near the damsite will have an inverted bed load/wash load relationship, and may only account for approximately 15 percent of the total sediment load being transported by each event.

TABLE 2

Computed Single Event Sediment Inflow to the Proposed Reservoir and Comparison to Planned Detention Storage Volume of 16,000 Feet

Event	Total Load Per Event (acre-feet) [dry volume]	Percent of the Planned Detention Storage Volume Associated with Single Event Sediment Delivery
5	245	1.5%
10	760	4.8%
20	1,794	11.2%
50	4,709	29.4%
100	6,992	43.7%
SPF	11,615	72.3%
PMF	29,440	184.0%

Harvey et al., (1990) estimate that there may have been approximately 9 inches of sediment deposited in the reach upstream from the Highway 58 crossing during the 1983 flood event. That event is estimated to be approximately a 50 year event according to the Kern County Water Agency. Comparing the total sediment loads entering and leaving the reach it is seen that there is approximately 575 acre feet more sediment transported into the reach from the upstream supply reach than leaves the reach. The approximate surface area of the reach is one square mile (640 acres). Assuming that the 575 acre feet of sediment deposits uniformly over the reach, this gives an approximate sediment deposition thickness of 10.8 inches. This matches the observed deposition depth for a 50 year event reasonably well.

Large events such as a 50 year flood or greater may produce large amounts of sediment material that enter the water course due to mass wasting, channel bank failure and erosion of prograded alluvial fans that often extend into the channel in the upper basin. It may be that single event sediment production can contribute significant quantities of sediment materials to the reservoir in a short period of time (a few days) and affect the operation and storage characteristics of the project.

Conclusions

The following conclusions are drawn from the results of this investigation:

1) The morphology of the Caliente Creek drainage basin and the nature of the sediments delivered to the channels and the potential for sediment storage within the drainage basin are controlled by the basin geology (Harvey et al., 1990).
2) Sediment transport in the basin is episodic and is governed by the occurrence of large runoff events. Sediment is stored in the broad valley washes (3000 to 6600 feet wide) in the lower portions of the Caliente Basin. There is sufficient material located in these expansive washes to provide sediment supply to the lower fan areas somewhat independently of the production and delivery of sediments from the upper watershed areas. Therefore, sediment yield at the proposed damsite may be more dependent upon the transport capacity of the channels and washes upstream from the damsite, than the watershed production of sediment materials during a flood event.
3) Examination of eight different sources of yield data and methods for estimating yield at the damsite concludes that the approximate average annual sediment yield at the Sivert Reservoir is 0.75 AF/sq mi/yr. This is more than twice the initial yield estimate developed during the planning studies. Annual sediment yields can range from 0.47 AF/sq mi/yr to approximately 1.5 AF/sq mi/yr.
4) Single event floods may produce significantly more sediment per event than the annual sediment yield would indicate. As much as 43 percent of the total gross pool storage volume (16,000 AF) may be lost due to sediment deposition during a 100 year event. This would necessitate the removal of approximately 7,000 AF of sediment material (dry volume) from the reservoir prior to the next flood season. It also indicates that the design capacity of the reservoir may be undersized.

Acknowledgements

The study reported herein was conducted by the Hydrologic Engineering Center, Davis, California at the request of the Sacramento District, Corps of Engineers. The views expressed herein are those of the authors and not necessarily those of the U.S. Army Corps of Engineers.

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4. PERFORMING ORGANIZATION REPORT NUMBER(S): Technical Paper No. 130

5. MONITORING ORGANIZATION REPORT NUMBER(S):

6a. NAME OF PERFORMING ORGANIZATION: Hydrologic Engineering Center

6b. OFFICE SYMBOL *(If applicable)*: CEWRC-HEC

7a. NAME OF MONITORING ORGANIZATION: Water Resources Support Center

6c. ADDRESS *(City, State, and ZIP Code)*:
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8a. NAME OF FUNDING / SPONSORING ORGANIZATION:

8b. OFFICE SYMBOL *(If applicable)*:

9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER:

8c. ADDRESS *(City, State, and ZIP Code)*:

10. SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO.	PROJECT NO.	TASK NO.	WORK UNIT ACCESSION NO.

11. TITLE *(Include Security Classification)*: Estimating Sediment Delivery and Yield on Alluvial Fans

12. PERSONAL AUTHOR(S): Dr. Robert C. MacArthur, Michael D. Harvey and Edward Sing

13a. TYPE OF REPORT: Technical Paper

13b. TIME COVERED: FROM ________ TO ______

14. DATE OF REPORT *(Year, Month, Day)*: June 1990

15. PAGE COUNT: 6

16. SUPPLEMENTARY NOTATION:

17. COSATI CODES

FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS *(Continue on reverse if necessary and identify by block number)*:
Alluvial Fans, Sediment Production, Sediment Yield, Transport Capacity, Geomorophology, Single Event Floods, Episodic Flows, Average Annual Sediment Yield, (continued)

19. ABSTRACT *(Continue on reverse if necessary and identify by block number)*

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20. DISTRIBUTION / AVAILABILITY OF ABSTRACT: UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS

21. ABSTRACT SECURITY CLASSIFICATION: UNCLASSIFIED

22a. NAME OF RESPONSIBLE INDIVIDUAL: DARRYL W. DAVIS, Director, HEC

22b. TELEPHONE *(Include Area Code)*: 916 745-1104

22c. OFFICE SYMBOL: CEWRC-HEC

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Hydrologic Aspects of Flood Warning - Preparedness Programs

Technical Paper No. 131
August 1990

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Hydrologic Aspects of Flood Warning - Preparedness Programs

Harry W. Dotson*, M.ASCE and John C. Peters*, M.ASCE

Abstract

A reliable flood-threat recognition system is a vital component of a sound flood warning-preparedness program. Fundamental questions associated with the development of a flood-threat recognition system are: what warning times can be achieved, and how reliable will the warnings be? Answers to these questions depend on watershed and storm characteristics, and the flood-threat recognition method being considered. The tradeoff between warning time and warning reliability is illustrated, and methods for estimating warning time are discussed.

Introduction

Flood warning and preparedness programs involve flood-threat recognition, warning dissemination, emergency response and post-flood recovery. The design and implementation of a sound, cost-effective program and the determination of the scope of the program depend substantially on the supporting hydrologic analyses. An important aspect of the hydrologic analyses is the development of a flood-threat recognition system. The analysis includes the evaluation of flood warning times, warning criteria, and the reliability of the warning.

Warning Time and Reliability

The concept of warning time is illustrated in Figure 1 (FIACWD, 1989). As indicated, maximum potential warning time (T_{wp}) is the time from the first indication of precipitation to the time flooding begins. Use of time (T_{wp}) as the actual warning time (T_w) would be totally unreliable because it would indicate that it floods every time it rains. There must be a flood recognition time (T_r) which is the time required for specific warning criteria to indicate flooding is imminent. The criteria could be that a specific amount of precipitation has occurred or that a stream has reached a specified stage. The longer the flood recognition time, the

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warning time. However, one must be aware of the tradeoffs between warning time and warning reliability.

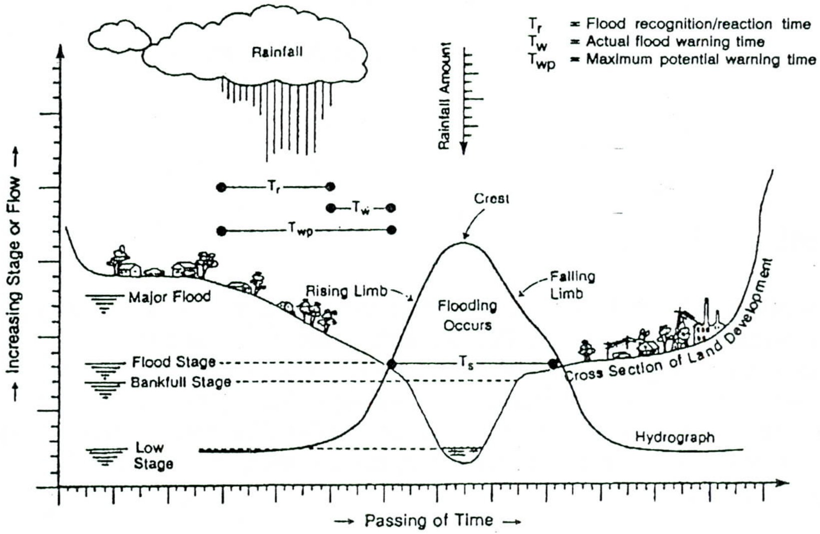

Figure 1. Illustration of Flood Warning Time

Consider Figure 2, which illustrates aspects of reliability. Sets of storm events are labeled {A}, {B}, {C} and {D}, where:

{A} = storm events that cause flooding
{B} = storm events that do not cause flooding
{C} = storm events that cause flooding but for which warnings are not issued
{D} = storm events that do not cause flooding but for which warnings are issued

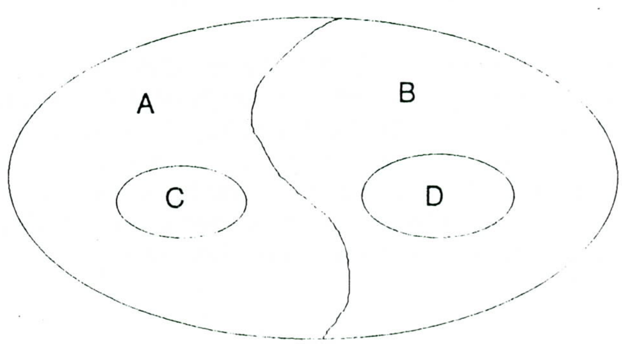

Figure 2. Reliability of Flood Warnings

The goal of a warning system is to minimize both {C} and {D}. Events from {C} can cause damage and loss of life that could possibly be prevented; events from {D} increase the likelihood that future warnings will be ignored. Alternative warning systems will be reflected by different configurations of {C} and {D}. The basis for a warning can range from measured stage at an index gage to results of a rainfall-runoff model that incorporate recent rain data and possibly estimates of future rainfall. Although the more sophisticated warning systems will tend to provide longer lead times, their reliability may not necessarily be greater than that associated with simpler systems. Both warning time and reliability should be evaluated when analyzing alternative warning systems

The tradeoff between lead time (warning time) and warning reliability can be illustrated by considering a simple threshold-stage method of warning, as illustrated in Figure 3. The warning stage is sensed at location A. The primary flood threat is downstream at location B. The problem is to choose a threshold (index) stage for location A such that when that stage is exceeded, a warning for flooding at location B is to be issued. It is desired that the lead time to prepare for the flood threat be as long as possible. The lower the index stage at A, presumably the more lead time will be afforded. However, if the threshold stage is too low, there will be too many false warnings, so that genuine warnings will not be heeded. In terms of Figure 2, as {C} is made smaller, {D} becomes larger.

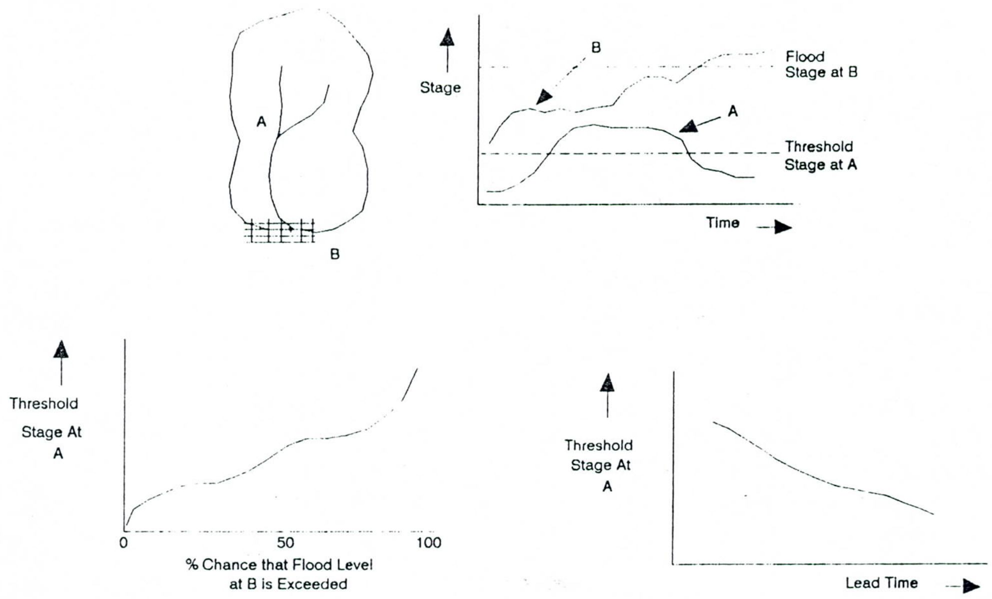

Figure 3. Lead Time Versus Warning Reliability

Illustration of Flood Warning and Reliability

To illustrate the tradeoff between warning time and reliability that is implicit in a flood warning system, consider a situation like that in Figure 3 in which a threshold stage at an index gage is to be used to trigger an alarm that warns of the impending exceedance of flood stage at a damage center. Although most flood warning systems are more sophisticated than this, analysis of a simple system can provide insights that have broader implications.

The basin used in this illustration is part of the Central Great Plains Experimental Watershed near Hastings, Nebraska (USDA, undated). In particular, discharge data collected over a 29-year period (1939-1967) at three gages on the west branch of Beaver Creek were used. The locations are labeled W3, W8 and W11 in Figure 4a. The drainage areas at these locations are very small and warning times will be very short. However, the intent of this analysis is to illustrate concepts rather than a practical design, and the available data is well suited to this purpose.

Assume that location W11 is the damage center for which a warning is to be issued, and that flood stage at W11 corresponds to a discharge of 300 cfs. This discharge was exceeded for 16 events during the 29-year period of record. Locations W3 and W8 will be considered individually as index locations for triggering a warning. That is, when a threshold discharge is exceeded at the index location, a warning is issued. The problem is to determine the threshold discharge to be used, and to assess associated warning time and reliability.

Period-of-record discharge data at a 15-minute interval for the three locations were acquired. The data were processed to determine events that exceed the flood discharge (300 cfs) at W11, and to determine threshold discharge exceedances at W3 and W8. Table

Table 1
Warning Time Analysis for a Threshold Q of 200 cfs at W8

Flood discharge at W11 = 300 cfs.

Date & Time of Flood at W11	Peak Q at W11	Time of Peak Q W11	Thresh. Q at W8 Exceeded?	Time of Exceed. Thresh. Q	Potential Warning Time (hr:min)
12 MAY 44 0315	394	0330	yes	0115	2:00
25 AUG 44 1045	343	1515	yes	1100	-:15
16 JUL 45 2045	333	2100	yes	1745	3:00
9 JUN 49 0030	374	0145	yes	2045 [2]	3:45
20 SEP 50 0115	730	0300	yes	2230	2:45
1 JUL 51 2045	1147	2215	yes	1930	1:15
10 JUL 51 0815	918	0900	yes	0630	1:45
14 JUL 52 0400	1063	0430	yes	0115	2:45
7 JUN 53 1815	680	2000	yes	1745	:30
22 MAY 54 2315	999	0200 [1]	yes	2300	:15
27 MAY 54 0330	325	0345	yes	2345	3:45
15 JUN 57 1730	1459	2115	yes	1215	5:15
29 AUG 57 0045	414	0130	yes	0130	-:45
3 JUL 59 2130	838	2400	yes	2115	:15
27 MAR 60 1645	365	1745	yes	1315	3:30
15 MAY 60 2230	811	0115 [1]	yes	2230	:00

[1] Next day.
[2] Previous day

16 flood events in 29 years
Number of events threshold discharge (200 cfs) was exceeded: 45
Reliability = 16/45 x 100 = 36%

1 illustrates results for a threshold dischage of 200 cfs at W8. The first three columns pertain to the flood event at W11; the last three columns refer to the exceedance of the thresholddischarge at W8. In this illustration, the threshold discharge was exceeded during all 16 flood events. The potential warning time associated with the events is shown in the last column. For two of the events, the time is negative.

As noted at the bottom of Table 1, the threshold discharge was exceeded 45 times during the 29 years of record, which means that a false warning would have been generated 29 times. The realiability of the warning mechanism, that is, the percent of true warnings to total warnings, is 16/45 x 100, or 36 percent. As may be noted from the table, a warning time $\geq$ 1 hour would have been provided for 10 of the 16, or 63 percent of the flood events. A warning time $\geq$ 30 minutes would have been provided for 69 percent of the events. The analysis illustrated in Table 1 was also applied with threshold discharges

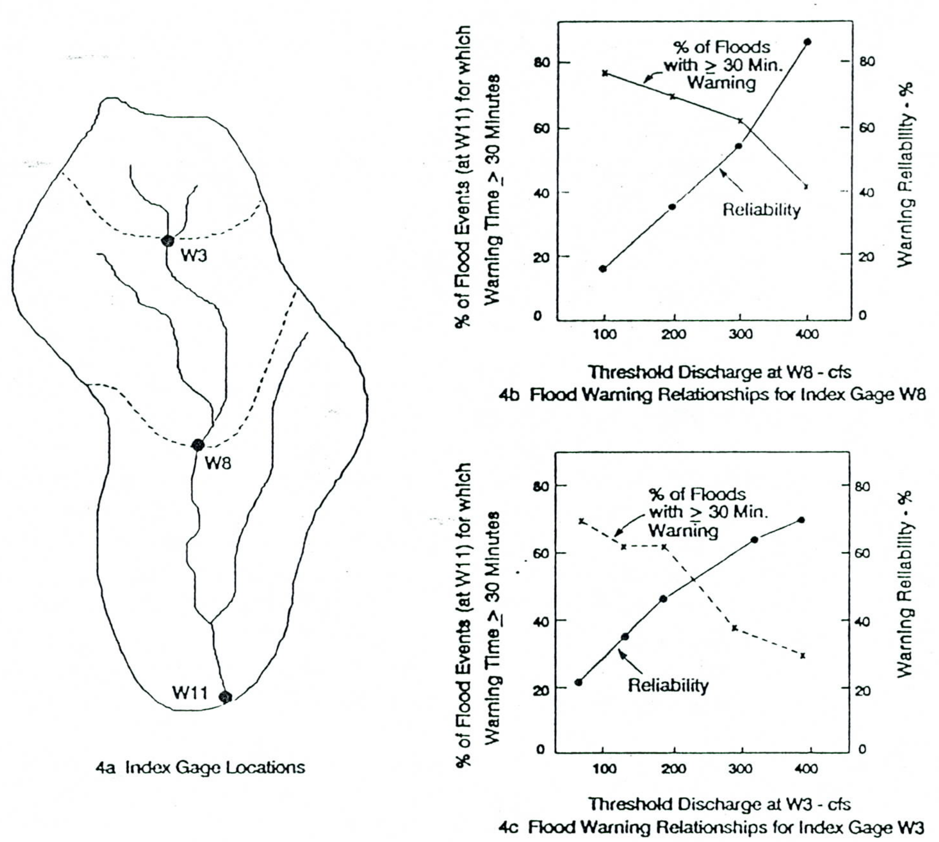

Figure 4. Beaver Creek Watershed

at W8 of 100, 300 and 400 cfs. Figure 4b shows forecast reliability and occurrence of at least a 30-minute warning time, both as a function of threshold discharge at W8. Figure 4c shows results for W3.

The inverse relationship between warning reliability and warning occurrence is readily apparent in Figures 4b and 4c. Suppose that it were desired to have a warning reliability of 70 percent, meaning that 7 out of 10 warnings would be for actual flood events. From Figure 4b, the corresponding threshold discharge at W8 is about 350 cfs and the percent of flood events for which a warning time $\geq$ 30 minutes is provided is 53 percent. That is, a warning time $\geq$ 30 minutes would be provided for only about half the flood events, and 3 out of 10 warnings would be erroneous. These are not very impressive figures, and such a warning system would obviously be far less than adequate.

By comparison, Figure 4c indicates that a 70 percent reliability could be achieved with a threshold discharge of 400 cfs at W3, for which a warning time $\geq$ 30 minutes would be provided for only 31 percent of the flood events. For this level of reliability, index location W8 is the better of the two locations.

Estimation of Flood Warning Time

Flood-threat recognition essentially involves real-time sampling of characteristics of a storm event and forecasting the probable near-term runoff response. The more variability associated with the event being sampled, the more difficulty there is in obtaining an adequate sample and the more uncertain the forecast.

Key variables upon which warning time depends include: (1) spatial variability of precipitation, (2) temporal variability of precipitation, (3) rainfall-runoff response characteristics of the watershed and (4) antecedent soil moisture conditions. Storm rainfall, and consequently warning time, typically exhibit substantial variability. To properly evaluate the potential warning time for a watershed, a set of storms should be analyzed that reflects such variability. Warning time can then be defined in terms of a median value and a standard deviation or some other measure of variability.

Warning time for a specific historical storm event can be estimated using a rainfall-runoff forecast model such as HEC-IF (Peters, 1985). The model accounts for precipitation and streamflow that has occurred up to the specified time-of-forecast and simulates streamflow into the future. Successive times-of-forecast can be evaluated until the simulated future runoff exceeds flood stage. The time between the time-of-forecast and the time when flooding begins represents an estimate of the gross warning time for the event being analyzed. An estimated time for collecting and analyzing real-time data during an actual storm would need to be estimated and subtracted from the gross warning time. If climatological forecasts had indicated a significant probability of future rainfall, such rainfall could be incorporated in the forecast and a longer warning time achieved. However, quantitative estimates of future precipitation are notoriously uncertain.

Ideally the analysis as described would be made for a number of historical events, and the median value and variability of warning determined. If there were no historical precipitation data for the basin, it would be reasonable to transpose rainfall information from within a hydrometeorologically homogeneous region. If no concurrent precipitation and streamflow data were available for a basin, there would, of course, be additional uncertainty associated with lack of data with which to calibrate the rainfall-runoff model.

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REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION UNCLASSIFIED	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION/AVAILABILITY OF REPORT
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE	
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Paper No. 131	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION Hydrologic Engineering Center 6b. OFFICE SYMBOL (If applicable) CEWRC-HEC	7a. NAME OF MONITORING ORGANIZATION Water Resources Support Center
6c. ADDRESS (City, State, and ZIP Code) 609 Second St. Davis, CA 95616	7b. ADDRESS (City, State, and ZIP Code) Casey Building, 2594 Fort Belvoir, Virginia 22060
8a. NAME OF FUNDING/SPONSORING ORGANIZATION 8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS (City, State, and ZIP Code)	10. SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO.	PROJECT NO.	TASK NO.	WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)

Hydrologic Aspects of Flood Warning - Preparedness Programs

12. PERSONAL AUTHOR(S)

Harry W. Dotson and John C. Peters

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Technical Paper	FROM ______ TO ______	August 1990	7

16. SUPPLEMENTARY NOTATION

17. COSATI CODES			18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
FIELD	GROUP	SUB-GROUP	Flood threat recognition, flood warning time, warning reliability, historical storm events, forecasting, rainfall-runoff modeling, HEC-1F program.

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

A reliable flood-threat recognition system is a vital component of a sound flood warning-preparedness program. Fundamental questions associated with the development of a flood-threat recognition system are: what warning times can be achieved, and how reliable will the warnings be? Answers to these questions depend on watershed and storm characteristics, and the flood-threat recognition method being considered. The tradeoff between warning time and warning reliability is illustrated, and methods for estimating warning time are discussed.

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS	21. ABSTRACT SECURITY CLASSIFICATION UNCLASSIFIED
22a. NAME OF RESPONSIBLE INDIVIDUAL Darryl W. Davis, Director, HEC	22b. TELEPHONE (Include Area Code) (916) 756-1104 22c. OFFICE SYMBOL CEWRC-HEC

DD Form 1473, JUN 86 *Previous editions are obsolete.* SECURITY CLASSIFICATION OF THIS PAGE

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TWENTY-FIVE YEARS OF DEVELOPING, DISTRIBUTING, AND SUPPORTING HYDROLOGIC ENGINEERING COMPUTER PROGRAMS[a]

Darryl W. Davis, P.E.[1] and Vernon R. Bonner, P.E.[2]

ABSTRACT: The Hydrologic Engineering Center (HEC) performs research and provides training and technical assistance for the development, deployment, and support of hydrologic engineering methods for Corps field office use. We understood early that to successfully accomplish the task, we needed to evolve a process that would place a family of high quality computer programs in the hands of users and assure that they would be effectively used. Several single purpose programs were released in 1964 and our first major computer program releases were made in 1968. The programs were made available in source code form on punched cards, and were accompanied by user's manuals, source code compilation instructions, and test data sets. The user was offered applications training, direct phone/on-site assistance, and the opportunity to join a network of users. This same philosophy is applied today. Our computer program products are substantially more capable (and complicated) and are in use by a wider variety of professionals in a more diverse computer hardware and operating system environment. The service and support functions, however, are more diffused. This paper presents an overview of the software development, distribution, and support experience of the Hydrologic Engineering Center. Comments are made regarding the future role of HEC and others in the distribution and support of HEC programs.
KEY TERMS: Computer programs, software support, user documentation.

INTRODUCTION

The Hydrologic Engineering Center was established in 1964 to provide technical services to Corps offices engaged in Civil Works activities. The technical areas of responsibility are hydrologic engineering and planning analysis techniques closely associated with hydrologic engineering. Within these technical areas, HEC provides services in research, training, and technical assistance. The Center has a staff of 25 professional engineers and computer scientists working in the executive office or the research, training, planning analysis, or technical assistance divisions. Support staff and a complement of 5 to 10 graduate students in water resources from the nearby University of California campus complete the staff.

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[a]Presented at the Transferring Models to Users Symposium, American Water Resources Association, 4-8 November 1990, Denver, Colorado.

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Applied research is designed to develop systematic methods that can save time and increase the effectiveness of experienced professionals and also enable less experienced persons to perform their duties with minimum start-up time. The products of these efforts are primarily general purpose computer programs and companion user's instructions and study methods guides. HEC distributes and services about 100 computer programs for application in hydrologic engineering and planning analysis (Corps of Engineers 1989). Table 1 summarizes information about the computer programs in the HEC library.

Table 1
Hydrologic Engineering Center
Software Library

Program Category	Major Programs	Other Programs	Editors/ Utilities	Total Programs		Implemented for PC's
Surface Water Hydrology	3	7	-	10	:	3
River Hydraulics	2	3	10	15	:	11
Reservoirs	1	4	5	10	:	5
Statistical Hydrology	1	5	-	6	:	3
Planning Analysis	3	6	1	10	:	7
Water Quality	2	4	1	7	:	4
Data Storage System	2	1	11	14	:	8
Water Control	-	3	14	17	:	-
Miscellaneous	2	1	4	7	:	2
Totals	16	34	46	96	:	43

Training is directed toward reducing the time needed for entry level professionals to become proficient in technical analysis and to familiarize seasoned professionals with new developments. The majority of the training is devoted to teaching effective use of HEC developed computer programs. About 500 student-weeks of training in a dozen courses are conducted annually. About two-thirds of the courses are hydrologic engineering courses and the remainder are planning analysis courses.

HEC works with Corps field offices in the application of new or unfamiliar procedures and in the solution of particularly complex and difficult water resources problems. The technical assistance projects begin with a negotiated reimbursable agreement and typically involve staff of the field office working with staff of HEC. The projects normally conclude with a joint product that solves the field office problem in a way that further assistance from HEC is not needed. The products often provide the basis for an improved general purpose solution that can be further developed with research funding into a product usable by the Corps as a whole.

PROGRAM LIBRARY AND SUPPORT EVOLUTION

The present computer program library, documentation, and support activities are the cumulative result of three eras of HEC activities. The first decade (1964 -1973) was that of single purpose programs, limited types and numbers of mainframe computer systems, and direct engineer - program user support activities. The second decade (1974 - 1983) was that of packaged programs, integration of data management systems, mini/mainframe computers and an expanded user community. The 1984 - 1990 period is that of the personal computer (PC) characterized by many machines, greatly expanded user community, increased attention to user interface and graphics, and diffused program distribution and support.

The First 10 Years (1964 - 1973)

The program development efforts were directed initially to computerizing existing analysis methods documented in Corps Engineer Manuals. The first group of programs released were single purpose, limited scope programs. Separate programs were developed for unit hydrograph computations, basin rainfall excess determination, stream flow routing, and similar functions/purposes. Subsequently, the small single purpose programs were integrated into more complete program packages, represented by such programs as HEC-1 "Flood Hydrograph Package" (Corps of Engineers 1970), HEC-2 "Water Surface Profiles" (Corps of Engineers 1966), and HEC-3 "Reservoir System Analysis" (Corps of Engineers 1971). Input was on punched cards and output was numerical/text with graphics represented as line printer plots. Output could be obtained in punched card format for subsequent input to other analysis programs. User documentation ranged from a limited user's manual (typically less than 10 pages, most of which was a detailed input variable description) for single purpose programs to larger (50 pages or more) for the few major programs. Occasionally short handout papers of a few pages, developed for training courses, were available. Programmer's manuals were developed for a few major programs. An example is the HEC-1 Programmer's Manual (Corps of Engineers 1973). Reference was made to existing Corps technical manuals, mostly dated in the mid- to late 1950's, for technical details about program computations. Incidentally, these technical manuals are just now (1990) in the process of being revised and updated.

Early in this period, computer hardware consisted of IBM 650 and 1620 class machines. Later IBM 7090 class machines became the norm. Programs were distributed as FORTRAN source code in punched card format. Users needed only to be concerned with applications while systems professionals dealt with hardware/operating system issues.

This period was characterized by the concept of an HEC engineer/programmer assigned for each program. The user community was modest from the standpoint that user support was not overly burdensome for HEC staff. High quality user's manuals, direct telephone support for all users, training courses, and systematic computer program maintenance emerged as important and well established principles for assuring effective and efficient use of the program library. The 1973 Annual Report (Corps of Engineers 1973) includes a listing of 28 computer programs presented as available from and supported by HEC. Six of these programs are classified as major including flood runoff, river hydraulics, reservoir systems, and statistical analysis. The remainder are more limited scope, special purpose or minor programs. Fifteen of the 28 programs continue to be maintained in the 1990 HEC program library.

The Second 10 Years (1974 - 1983)

Program development efforts during this period emphasized creating a specialized hydrologic engineering data management system, integrating it with existing programs, and expanding the technical areas addressed by the programs. Major program additions included the HECDSS system (data management), beginning of the real-time water control software family, HEC-5 (reservoir system analysis for flood control and conservation), a package of flood damage analysis programs, and a family of graphics, utilities, and data communications software. Punched cards, (and punched card machines) disappeared from HEC. Data entry was now via remote terminal (creating data files) and output generally went to line printers. Graphics became more important. User documentation became substantially more sophisticated and complete. User's manuals were expanded. A typical manual now comprised near 100 pages including technical descriptions, input preparation assistance, and illustrated examples. Companion applications documents (training documents) were developed for most major programs.

The hardware of this period is typified by a CDC Cyber computer with substantial computing power accessed through inexpensive graphic terminals. Late in this period, most Corps offices installed Harris 500 or 1000 machines - very capable minicomputers. These became a mainstay of the hydrologic engineering community. Within the Corps, programs for the Harris computers were distributed on a mail-out tape containing executable code. This greatly simplified program distribution for ourselves as well as the using offices. For non-Harris sites, programs were distributed as FORTRAN source code via magnetic tape. The programs were accompanied by compilation instructions and test data. With the advent of the HECDSS software, increasingly capable graphics packages, and data communications packages, we had to concern ourselves with the specific hardware and operating system environments in which the programs would be used. These added complications were not welcomed.

The user base for HEC programs was greatly expanded with the wide-spread application of HEC computer programs, particularly HEC-2, in support of the National Flood Insurance program. The daily use of these programs expanded several fold in a matter of two years. The concept of the HEC engineer/programmer assigned for each program became stressed with the advent of a greatly enlarged community of users expecting support. We continued to provide HEC computer program hot-line support to all program users regardless of their affiliation. Training courses were restricted to Corps staff with openings, as available, filled on a first come, first served basis. We presented several classes for private consultants to support our field offices contracting out for flood insurance studies.

The 1983 Annual Report (Corps of Engineers 1983) includes a listing of 66 computer programs presented as available from (a few restricted to Corps offices only) and supported by HEC. Five new major programs were added for a total of 11. Two of these were in the HECDSS data management area and the others were in water quality, statistical analysis and flood damage analysis. The remaining 55 (17 from the first 10 years) are limited scope, special purpose, or minor programs. They span the range from graphics, specialized editors and analysis utilities, but also include limited applications water quality, hydroelectric power, hydraulic and hydrologic analysis programs. Thirty-seven of the 66 programs continue to be maintained in the 1990 HEC program library.

The Personal Computer Era (1984 - present)

HEC released its first personal computer version of an HEC program in 1984. The program was HEC-2. Our business has not been the same since. The number of users expanded several orders of magnitude, and we were nearly overwhelmed with user calls for assistance. Our focus of necessity turned to the user interface, interactive graphics, and preoccupation with operating system details here-to-fore outside the realm of our concern. We ended up explaining MS-DOS to new PC users.

Significant efforts have been devoted to moving existing programs to the PC environment. Forty-three of the 96 programs in our present program library are available in PC versions. We have developed only one new program designed specifically for the PC environment. Another new program (that is considered a major product) is nearing release that is designed specifically for the PC/workstation environment.

We enhanced and released our own text editor to better meet our program development and data entry needs; built menu shells for file, program execution, and display management for our major packages; and became far more expert in the intricacies of the PC than we expected (or hoped) would be needed. We found that user instructions for program application were no longer sufficient. Many users both inside and outside the Corps were just developing PC literacy and therefore needed program installation guides, PC file management standards, and similar information. We needed to handle the explosion of output devices (device drivers), and a multitude of other non-technical items. Without question, our programs were made more useable and widely available but the price was distraction from continued development of new technical products.

We experienced increased non-Corps requests for training course attendance, tapes of course lectures, and course materials. Also occurring at this time were increased offerings of HEC based short courses through university extension programs. A budding private vendor industry for marketing PC based engineering programs also surfaced. After much internal deliberation and several false starts, HEC adopted and published in the public record, a policy of encouraging private vendor distribution and support for HEC programs to non-federal parties. This became effective October 1988. Requestors for programs, training and similar services are referred to a vendor list that is maintained for that purpose. Several thousand HEC program copies have been successfully and professionally provided to the public by the vendor community.

We did not completely ignore mainframe applications. The family of computer programs to support water control (daily operations of existing Corps reservoir projects) was significantly expanded. These programs are supported for the Harris systems dedicated to water control activities within the Corps. Technical features of programs were updated for mainframe versions simultaneously with the intensive PC applications activities. Notable were major additions to HEC-1 (Muskingum-Cunge routing and kinematic wave surface runoff transform), HEC-2 (culvert hydraulics and hydraulic design capabilities), and HEC-5 (power operation algorithms). A few new programs were developed in the traditional batch style. Table 1 summarizes the programs in the 1990 HEC program library. The recent annual report (Corps of Engineers 1989) tabulates the 96 programs shown in Table 1.

HEC PROGRAMS AND SUPPORT IN THE NEXT DECADE

We have (or soon will) have all our major programs assembled into similar PC packages. Program documentation has been updated and upgraded (thanks to today's word processors). Several new training documents provide details on the PC packages and special program applications. We are envisioning this set of releases to be our last major PC (batch programs ported) releases. University short courses and private vendors appear to meet much of the non-federal needs for PC programs distribution, training, and technical support. We now plan to focus on the future.

We view the coming decade (1991 - 2000) as that of the engineering workstation. These machines are very computationally powerful, have exceptional graphic display capability, and will be networked to share mass storage, output devices, and computational resources. The next five years are expected to see transition from DOS to these UNIX based systems. About mid-decade, we anticipate about half of the Corps will use HEC programs on these systems. We are embarked on an intensive developmental effort that will yield successor program packages to the existing major programs. Under development are packages we refer to as the river analysis system, the catchment analysis system, the reservoir analysis system, and the flood damage analysis system. These systems of programs are being developed specifically for the UNIX workstation environment. The program systems will feature new computational algorithms, incorporate imaging and geographic information system capability, and will be executed within an interactive graphic user interface. Computational features of existing programs may be incorporated as proves to be desirable.

The multi-tasking, multi-user, exceptional graphic features of UNIX workstations are compelling in support of engineering applications. We are hopeful that DOS and UNIX will rapidly merge to more common attributes over the coming years so we do not have to maintain significantly different code to service both groups of users. By mid-decade we expect to service the Corps (in the priority listed) software for: UNIX workstation, DOS PC, and Corps mainframe (probably CDC Cyber).

We are committed to the current policy of private vendor distribution and support of HEC programs for non-federal users. We will encourage and support expanded training offerings by vendors and universities. New program development will be emphasized through in-house efforts and increasingly through contract assistance from the private sector. We expect the coming decade to be busy, exciting, and most gratifying in continuing our historical role of developing and servicing a wide array of hydrologic engineering and planning analysis computer programs.

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UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION
UNCLASSIFIED

1b. RESTRICTIVE MARKINGS

2a. SECURITY CLASSIFICATION AUTHORITY

2b. DECLASSIFICATION / DOWNGRADING SCHEDULE

3. DISTRIBUTION / AVAILABILITY OF REPORT

4. PERFORMING ORGANIZATION REPORT NUMBER(S)
Technical Paper No. 132

5. MONITORING ORGANIZATION REPORT NUMBER(S)

6a. NAME OF PERFORMING ORGANIZATION
Hydrologic Engineering Center

6b. OFFICE SYMBOL (If applicable)
CEWRC-HEC

7a. NAME OF MONITORING ORGANIZATION
Water Resources Support Center

6c. ADDRESS (City, State, and ZIP Code)
609 Second Street
Davis, California 95616

7b. ADDRESS (City, State, and ZIP Code)
Casey Building #2594
Fort Belvoir, Virginia 22060

8a. NAME OF FUNDING / SPONSORING ORGANIZATION

8b. OFFICE SYMBOL (If applicable)

9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER

8c. ADDRESS (City, State, and ZIP Code)

10. SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO.	PROJECT NO.	TASK NO.	WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)
Twenty-five Years of Developing, Distributing, and Supporting Hydrologic Engineering Computer Programs

12. PERSONAL AUTHOR(S)
Darryl W. Davis and Vernon R. Bonner

13a. TYPE OF REPORT
Technical Paper

13b. TIME COVERED
FROM ________ TO ________

14. DATE OF REPORT (Year, Month, Day)
November 1990

15. PAGE COUNT
7

16. SUPPLEMENTARY NOTATION

17. COSATI CODES

FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

This paper presents an overview of the software development, distribution, and support experience of the Hydrologic Engineering Center. Comments are made regarding the future role of HEC and others in the distribution and support of HEC programs.

20. DISTRIBUTION / AVAILABILITY OF ABSTRACT
☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS

21. ABSTRACT SECURITY CLASSIFICATION

22a. NAME OF RESPONSIBLE INDIVIDUAL
Darryl W. Davis, Director, HEC

22b. TELEPHONE (Include Area Code)
(916) 756-1104

22c. OFFICE SYMBOL
CEWRC-HEC

DD Form 1473, JUN 86 Previous editions are obsolete. SECURITY CLASSIFICATION OF THIS PAGE

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Predicting Deposition Patterns in Small Basins

Technical Paper No. 133
March 1991

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PREDICTING DEPOSITION PATTERNS IN SMALL BASINS[1]

By D. Michael Gee, Research Hydraulic Engineer, Hydrologic Engineering Center, Davis, CA.

ABSTRACT

A technique for estimating sediment depositional patterns based upon flow patterns is described. Flow patterns are computed using a finite element model for two-dimensional, vertically averaged flow. Once the velocity and depth fields are computed, the bed shear stress distribution can be found. If the annual volume and approximate particle size of the inflowing load is known, anticipated depositional locations and quantities can then be estimated. Use of this technique to forecast the temporal development of the deposits by computing the velocity fields for several steady flow conditions is described. The resulting graphical displays of velocity fields and shear stress contours are very useful to the design engineer. This procedure avoids the complexity associated with use of a two-dimensional sediment transport and dispersion model. Application of the technique to the design of a basin 180 ft. (55 m.) wide by 610 ft. (186 m.) long is described.

INTRODUCTION

Conventional sediment basin design procedures rely on volumetric relationships to determine flow through times, estimated trap efficiency, and average annual deposition rates. Design guidance has been prepared by USACE (1989). These approaches do not necessarily reflect the interaction between changes in bed topography due to scour and/or deposition and the influence of these changes on velocity and shear stress distributions. Some designs have been approached using one-dimensional numerical modeling of flow and sediment such as HEC-6 (USACE-HEC, 1990). Some concerns with these approaches are that complex velocity patterns such as recirculation and short circuiting may not be properly described. These flow patterns may result in uneven distributions of sediment concentration and, therefore, an uneven distribution of sediment deposits (Montgomery, et al. 1983). The use of a fully two-dimensional model for both flow and sediment distribution such as TABS-2 (McAnally et al. 1984) is an attractive approach to improve the prediction of the distribution of sediment deposits. The use of such a model, however, may involve more effort and data acquisition than can be justified for small basin design. The technique described herein represents a midway approach that includes the velocity and shear stress fields in detail, from which the sediment deposition distribution and rates can be inferred. A brief description of this approach was presented by Deering and Larock (1989).

MODEL SELECTION

It is assumed that the salient flow features of small basins can be described in the two horizontal directions and that the variance of velocity in the vertical is the traditional logarithmic velocity distribution for turbulent flow in open channels (French 1985). A widely used model that is suitable for this condition is RMA-2 (King & Norton 1978). RMA-2 has been applied to a wide variety of problems including floodplain analysis (Gee et al. 1990), marsh flooding (MacArthur et al. 1990), sediment basin design (Deering & Larock 1989), has been adapted for bridge design (FHWA 1989), and serves as the hydrodynamic module of the TABS-2 system (McAnally et al. 1984). This model solves the depth integrated Reynolds equations for two-dimensional free-surface flow in the horizontal plane using the finite element method for both steady and unsteady flows. The finite element

[1]Presented at the Fifth Federal Interagency Sedimentation Conference, Las Vegas, Nevada, March 1991.

formulation of RMA-2 allows boundary roughness and geometric resolution to vary spatially to accurately depict topography. It also provides a wide variety of boundary conditions. Wetting and drying of portions of the solution domain is allowed. The two-dimensional approach relieves the engineer from having to construct cross sections that are perpendicular to the flow for all flows, as is required in a one-dimensional analysis.

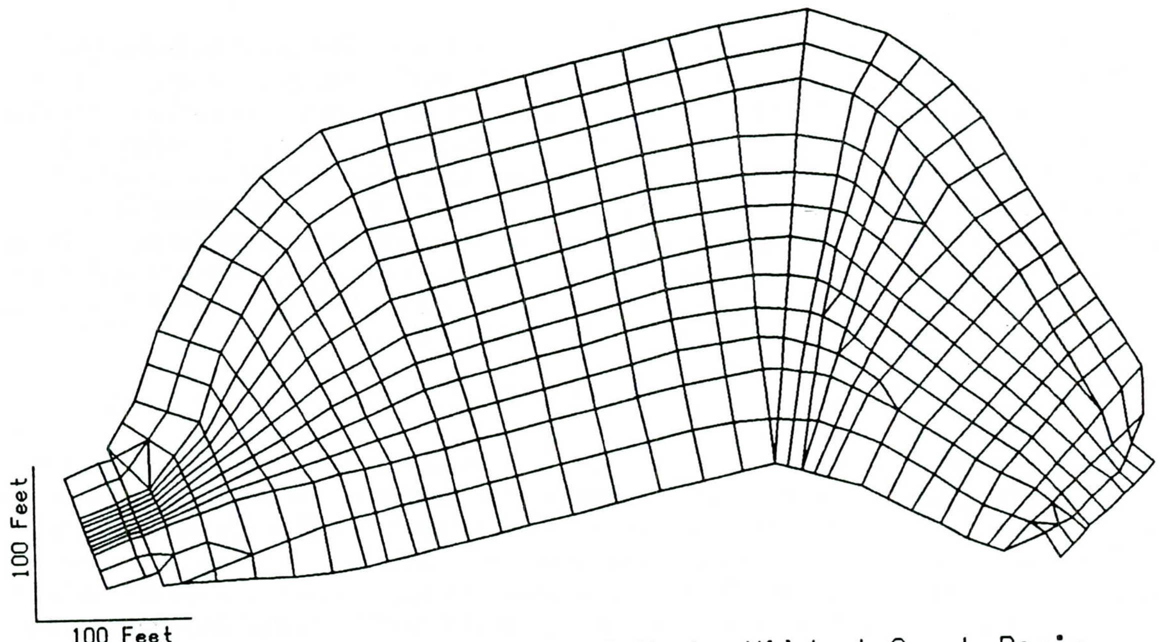


Figure 1. Example Finite Element Mesh, Wildcat Creek Basin.

APPROACH

An example finite element mesh is shown in Fig. 1. Note that the elements are both quadrilateral and triangular. Computational nodes exist at the corners and mid-sides of each element. The bottom elevation is given at each corner node and linearly interpolated for the mid-side nodes. Bed roughness and turbulent exchange coefficients are assigned to groups of elements (not necessarily neighbors) by the user. Solution of the two-dimensional flow equations provides the x- and y-components of the velocity, and the depth, at each computational node. The local shear stress can be calculated from these variables if one assumes that the relation for average shear in a cross section can be applied locally as follows.

$$\tau = \gamma RS \tag{1}$$

Where τ is the bed shear stress, γ is the unit weight of water, R is the hydraulic radius (taken here as the local nodal depth) and S is the friction slope. Now, rewriting Manning's equation in terms of S, we have:

$$S = \frac{n^2u^2}{2.22R^{4/3}} \tag{2}$$

Where u is the resultant of the calculated x and y nodal velocity components, as shown in equation (3) and n is Manning's roughness coefficient.

$$u^2 = u_x^2 + u_y^2 \tag{3}$$

Combining, we can solve for the shear stress:

$$\tau = \gamma \frac{n^2u^2}{2.22R^{1/3}} \tag{4}$$

One must now relate the n-values, which are associated with elements, with the computed values for u and R (depth) which are located at nodes. For this study, the n-value associated with a node was computed as the arithmetic average of the n-values for all elements connected to that node. We have placed these computations in the vector plotting program (VECTOR) which is a post-processor for RMA-2. VECTOR also prepares files of water surface elevation and velocity magnitude for contouring.

AN EXAMPLE

Introduction

The Wildcat Creek sediment basin was designed to trap sediment that potentially could cause excess scour or deposition in a downstream flood control channel. Right-of-way considerations and environmental concerns dictated the bent alignment shown in Fig. 1 (flow is from right to left). Based on cross section average velocity and settling lengths computed from the particle fall velocity, it was estimated that the basin would trap 100% of the sediment larger than fine sand (0.125 mm). A hydrodynamic analysis was performed to ascertain whether the bent alignment would indeed trap the size range and volume of sediment needed and whether high velocities would impinge on the banks requiring some form of bank protection.

Sediment Basin Description

The Wildcat Creek sediment basin was designed to have a maximum width of 180 ft. (55 m.) and length of 610 ft. (186 m.). The bottom slope is 0.0005 and the side slopes 1V:3H. The maximum depth is about 12 ft. (3.7 m.).

Hydraulics

As Wildcat Creek is ephemeral, continuous simulation was not necessary. Therefore, several hydraulic scenarios were studied to verify that the basin would perform as designed. It was planned that deposits would most likely have to be removed from the basin on an annual basis. This led to simplification in the number of conditions to be analyzed because the problem was reduced to evaluation of the interactions between average annual deposition and the occurrence of the design (1% exceedance) event. The results presented here are only for the design event; refer to Deering and Larock (1989) for information on other scenarios. Furthermore, as the basin volume is small relative to the hydrograph volume, the analysis could be performed assuming steady flow. The 1% chance exceedance event is 2300 cfs (65 cms). The drainage area is about 7.8 mi^2 (2000 hectares).

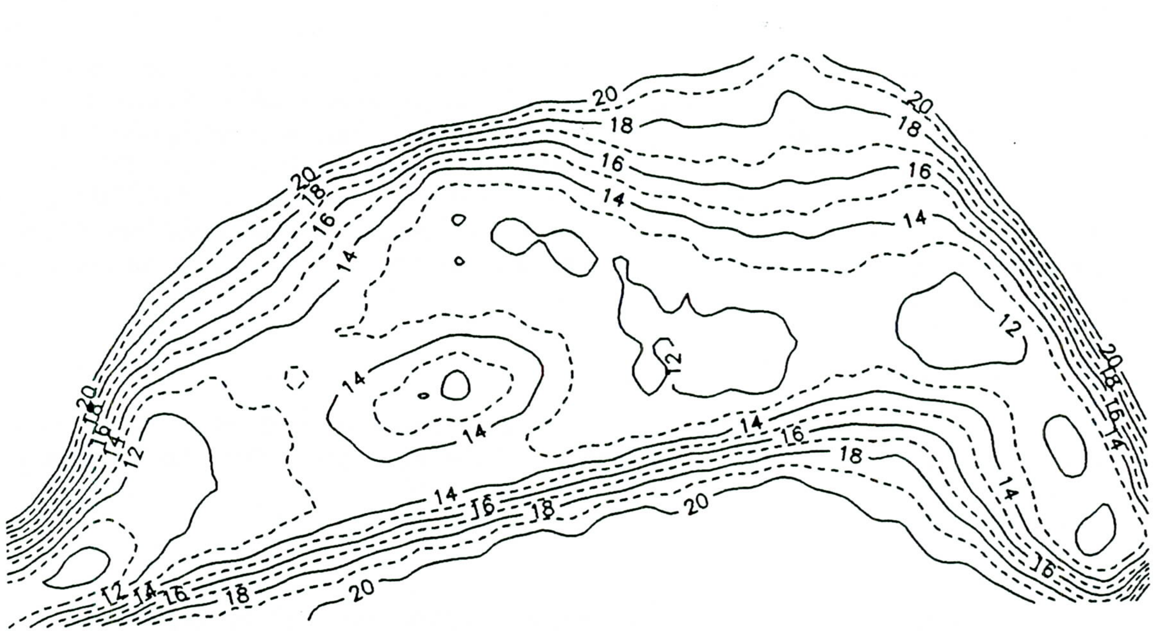
Scenario

The situation presented herein represents the condition of the basin after several years' average annual deposition (not removed). The flow evaluated is that of the design (1% chance exceedance) event. The distribution of the deposits shown in Fig. 2 was created based on simulation of the shear stress distribution in the empty basin and observation of other flood control projects having similar flow and sediment transport conditions. The bar deposits are formed from flows expanding into open areas. Initial deposits will form in the lower velocity areas causing the flow to redistribute, expanding again and reinitiating the bar formation process. This results in bar formation on the left and right banks, immediately downstream of the entrance, and a central bar further downstream. The assumed deposition pattern has a volume equivalent to that of the average annual deposits for the time period selected. The nodal elevations of the finite element mesh that was developed for the design (empty) basin were modified to reflect this hypothetical deposition pattern.

Modeling parameters

The Manning's n-values were set to 0.03 for most of the basin based on it being maintained as smooth earth. The values for one portion of the left bank were set to 0.06 based on maintaining the native heavy vegetation there. The sensitivity of the results to these values, assuming the project is

Figure 2. Bottom Elevations (ft).



not well maintained should be checked and may be significant to the design event water surface elevation. The turbulent exchange coefficients were uniformly set to 10 lb-sec/ft^2 (480 N-sec/m^2) for all elements. This was based on prior experience with finite element meshes of this scale. The sensitivity of the results to variation of these values within reasonable ranges was checked and found to be insignificant.

Boundary conditions

This is a simple 2-D problem in that it is analogous to traditional 1-D backwater computations with regard to boundary conditions. A discharge was specified at the upstream (right) end of the model. In 2-D, however, the direction of the discharge must be given which was selected to be perpendicular to the inflow boundary line (see Fig. 3). The downstream boundary condition was specified as a water surface elevation appropriate for the discharge being analyzed based on design studies of the downstream reach. A rating curve could have been used for the downstream boundary if appropriate. Along all other boundaries, the flow direction is parallel to the boundary.

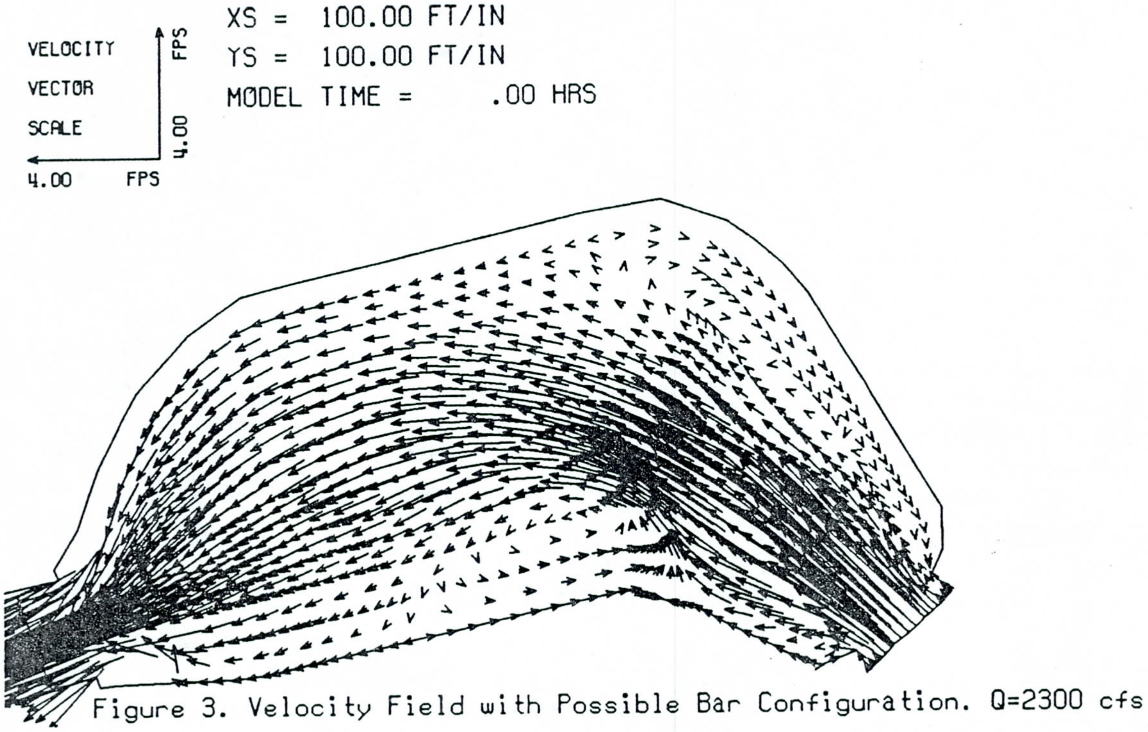


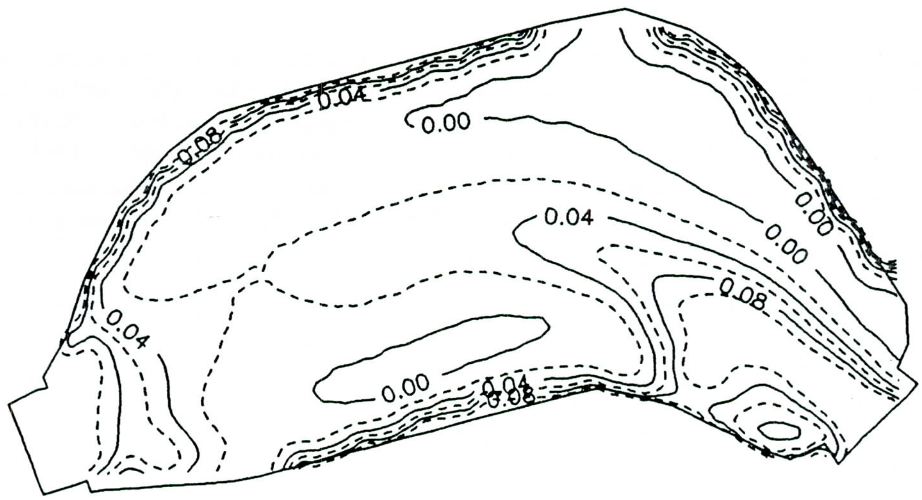
Figure 3. Velocity Field with Possible Bar Configuration. Q=2300 cfs.

Modeling results

The flow field for the design event is depicted on Figure 3. The flow enters the basin as a plume of relatively high velocity. Recirculation zones are seen on each side of the inflow plume. This is obviously not a one-dimensional situation. The hypothetical bar formations do not appear to force the higher velocity jet against either of the banks as originally suspected. The associated shear stress field for this flow and bottom condition is shown in Figure 4.

The shear stress is low enough that sediments of the size of interest will be trapped in the basin. Note particularly the zones of near zero shear that correspond to the recirculation cells near the left and right banks. The clustering of contours near the banks is an artifact of the contouring process.

Figure 4. Shear Stress (lb/sq ft).



CONCLUSIONS

The technique presented herein represents a midway approach to the prediction of spatially complex sediment transport processes. Much can be inferred from viewing the velocity and shear stress distributions. Once the velocity field has been computed, the computation of the shear stress distribution is trivial. If, at this stage, one determines that simulation of the full two-dimensional transport and dispersion of sediment is necessary, the hydrodynamic analysis already performed can be used directly in the sediment transport simulation.

COMPUTATIONAL ASPECTS

The finite element mesh used for this study contains about 550 elements and 1370 nodes. This produces about 2300 simultaneous equations. To solve this system for steady flow using six iterations takes about 15 minutes on a 25 MHz 386 computer. The system can be run within the DOS 640K limitation.

ACKNOWLEDGEMENTS

This study was performed for the U. S. Army Corps of Engineers District, Sacramento with the assistance of the U. S. Army Corps of Engineers Waterways Experiment Station. The opinions expressed herein are those of the author and not necessarily those of the U. S. Army Corps of Engineers.

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2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION/AVAILABILITY OF REPORT
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4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Paper No. 133	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION Hydrologic Engineering Center 6b. OFFICE SYMBOL (If applicable) CEWRC-HEC	7a. NAME OF MONITORING ORGANIZATION U.S. Army Corps of Engineers Water Resources Support Center
6c. ADDRESS (City, State, and ZIP Code) 609 Second Street Davis, California 95616	7b. ADDRESS (City, State, and ZIP Code) Casey Building #2594 Fort Belvoir, Virginia 22060
8a. NAME OF FUNDING/SPONSORING ORGANIZATION 8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS (City, State, and ZIP Code)	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. / PROJECT NO. / TASK NO. / WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)

Predicting Deposition Patterns in Small Basins

12. PERSONAL AUTHOR(S)

D. Michael Gee

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Technical Paper	FROM ______ TO ______	March 1991	7

16. SUPPLEMENTARY NOTATION

17. COSATI CODES			18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
FIELD	GROUP	SUB-GROUP	Two-dimensional Flow, Finite Elements, Sediment Transport Modeling

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

A technique for estimating sediment depositional patterns based upon flow patterns is described. Flow patterns are computed using a finite element model for two-dimensional, vertically averaged flow. Once the velocity and depth fields are computed, the bed shear stress distribution can be found. If the annual volume and approximate particle size of the inflowing load is known, anticipated depositional locations and quantities can then be estimated. Use of this technique to forecast the temporal development of the deposits by computing the velocity fields for several steady flow conditions is described. The resulting graphical displays of velocity fields and shear stress contours are very useful to the design engineer. This procedure avoids the complexity associated with use of a two-dimensional sediment transport and dispersion model. Application of the technique to the design of a basin 180 ft. (55 m.) wide by 610 ft. (186 m.) long is described.

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A Muskingum-Cunge Channel Flow Routing Method for Drainage Networks

November 1991

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TP-135

SUMMARY

A Muskingum-Cunge channel flow routing scheme is modified for application to large drainage networks with compound cross sections and for continuous long-term simulation. The modifications consist of a decoupling and separate routing of main and overbank channel flow, an introduction of a variable time step to increase model efficiency during periods of steady flow, and an internal determination of the numerical increment. The resulting hydrologic model is verified by comparing its flow routing results with those of hydraulic benchmark models solving the full unsteady flow equations. Test conditions consist of hypothetical flood hydrographs, long prismatic channels with simple and compound sections, and a third order drainage network. For the tested conditions, the model produces hydrograph peaks, times to peak and shapes that compare well with those of the hydraulic benchmarks. Hydrograph distortions due to overbank flood plain storage and multiple peaks from complex drainage networks are also well reproduced. The execution time of the model is generally one order of magnitude faster than that of the hydraulic benchmark models.

INTRODUCTION

In drainage networks, channel runoff is continuously transformed as it travels downstream. Within individual channel reaches, hydrograph characteristics change as a result of flow hydraulics, channel storage, subsurface contributions or transmission losses, and lateral inflow. In the presence of active flood plains, overbank storage produces additional attenuation. And, at channel junctions discreet changes in runoff occur as a result of the merging of flows from upstream areas. These flow transformations occur simultaneously throughout the drainage network and reshape the channel-flow hydrographs as they travel downstream. In addition to these in-channel transformations, the spatial distribution of the source areas and the timing of their respective runoff stongly influence the temporal characteristics of the watershed response. To quantify these effects in large watersheds and complex drainage networks, a practical and efficient channel flow routing model is needed. For this purpose, the Muskingum-Cunge flow routing scheme with variable parameters (Koussis, 1978; Laurenson, 1962; Ponce and Yevjevich, 1978) has been modified (Garbrecht and Brunner, 1991). The hydrologic approach greatly improves computational efficiency and speed, and reduces the amount and detail of field data traditionally needed for hydraulic routing (Weinmann and Laurenson, 1979). Such a hydrologic routing scheme is a practical approach to integrate the response from a large number of upstream source areas, to quantify effects of the flow integration processes on watershed runoff characteristics, and to investigate the impact of spatially variable source-area runoff on watershed response.

In this report, the hydrologic Drainage Network Channel Flow Routing model (DNCFR) is presented, followed by a verification for channels with simple and compound sections, and a third order drainage network. The Muskingum-Cunge flow routing scheme with variable parameters is used as the initial base model. It is adapted for separate flow routing in the main and overbank channel portions, and it includes a variable computational time increment. The parallel main and overbank channel flow routing simulates the flow characteristics in each channel portion. Differentiation between main and overbank channel flow is often desirable because sediment mobilization, transport and deposition, transmission losses and water quality parameters vary differently in each channel portion and may require separate treatment. The variable computational time increment is introduced for efficient long-term simulation often required for sediment and water-quality investigations.

This hydrologic channel-flow routing scheme is applied to drainage networks by feeding source-area runoff into the channels, merging the channel flows at network junctions, and routing the flows through the channel network. As for most hydrologic routing schemes, the present scheme does not account for backwater effects and does not provide detailed hydraulic flow conditions along individual channel reaches nor does it reproduce localized effects. The results of the model verification show that the DNCFR is an effective tool for applications to large complex drainage networks and for continuous long-term simulations. To operate DNCFR the user must provide, in addition to the usual channel and drainage network parameters, surface and subsurface inflows into, as well as losses out of the drainage network.

MODEL DNCFR

The flow routing model DNCFR consists of four components: the first component quantifies the drainage network topology; the second the hydraulic properties of the cross sections; the third routes the flow in individual channel reaches; and the fourth is the main driver which controls the execution of individual model components and coordinates the routing within the drainage network. Each component is presented separately.

1 - Drainage Network Topology Component

In a drainage network, it is generally necessary to determine the sequence in which channel flow must be routed. When backwater effects are negligible, it is common practice to route channel flows from upstream to downstream. Such channel flow routing is called cascade routing. In drainage networks there are many upstream channels that simultaneously contribute to the watershed outlet. As a result, there are many possible sequences in which channel flows can be routed. The determination of this routing sequence is often performed manually. This is a tedious and error prone task and is least adaptable to subsequent changes in drainage network resolution. An automated determination of the routing sequence greatly simplifies the engineer's task, insures correctness and consistency, and expedites drainage network evaluation.

One such programmable algorithm was presented by Croley (1980). His algorithm always selects the right most source node as starting point, and it determines the sequence of channel flow routing consistently from right to left on the source nodes, irrespective of drainage network configuration. Under certain conditions this approach can lead to large computer storage needs, as subsequently explained. The algorithm of model DNCFR is a direct solution algorithm (Garbrecht, 1988) based on the drainage network definition by Croley (1980). It determines a channel flow routing sequence that minimizes computer storage needs. Computer storage need is defined as the number of internal arrays required for cascade routing. An array is necessary for temporary storage of runoff results from one channel or network branch, while runoff from another is being evaluated. For example, at a junction node, runoff values from one upstream inflow must be stored, while the other upstream inflow is being evaluated. This corresponds to one storage need. Different drainage network topologies have different computer storage needs. In the following the algorithm of DNCFR to determine the routing sequence is briefly presented.

The drainage network is represented as an arrangement of channels and connecting points called nodes. The type of node is defined by the node code. A node can be a channel source, a tributary junction, a lateral inflow point, or any other special-purpose point, such as a change in channel cross section geometry or longitudinal profile node. A list of node codes that are accepted by the algorithm is given in Table 1. Between two nodes channel cross section geometry and longitudinal slope are assumed constant. It is also assumed that junctions of more than two channels at one point do not occur. However, in the remote chance of occurrence the situation can be simulated by adjacent nodes connected by a short channel. Nodes are numbered in a left hand pattern as shown in Fig. 1a and corresponding node codes are shown in Fig. 1b.

Table 1. Node code definitions.

Node Code	Definition
1	Source node
2	Drainage network outlet node
3	Channel junction node
4	Point lateral inflow node
5	Water withdrawal node
	Change in channel geometry node
	Reservoir inlet node
	Reservoir outlet node
	Water inflow node other than channel junction or point lateral flow

To determine the optimal channel flow routing sequence the Strahler channel orders (Strahler, 1956) at each node is needed. This is accomplished by having the algorithm backtrack from the source nodes downstream and increase the channel order each time a tributary of same order is encountered. When a tributary is of large Strahler order then the latter value is assumed (Fig. 1c). Once Strahler orders are assigned to all nodes, upstream and lateral inflows to each node must be identified. The algorithm identifies upstream inflows into a junction by the node numbers corresponding to the two merging channels, and lateral inflow by the node number it flows into.

With the Strahler channel order and the inflows into nodes defined, the channel flow routing sequence can easily be determined. The flow routing begins at a source node that leads to the minimum storage needs. This source node is one that directly contributes to the network order (Garbrecht, 1988). From the beginning source node, the algorithm backtracks downstream from node to node, and from network subbranch to the next larger subbranch, assigning the appropriate routing sequence to all channels, as shown in Fig. 1d. Information available upon completing the drainage network evaluation includes: 1) channel flow execution sequence, 2) identification of upstream and lateral inflows, 3) Strahler channel order at each node, 4) specification of channel or reservoir segments. This information gives a complete and sufficient description of the drainage network topology to fully automate the management of the channel flow routing process.

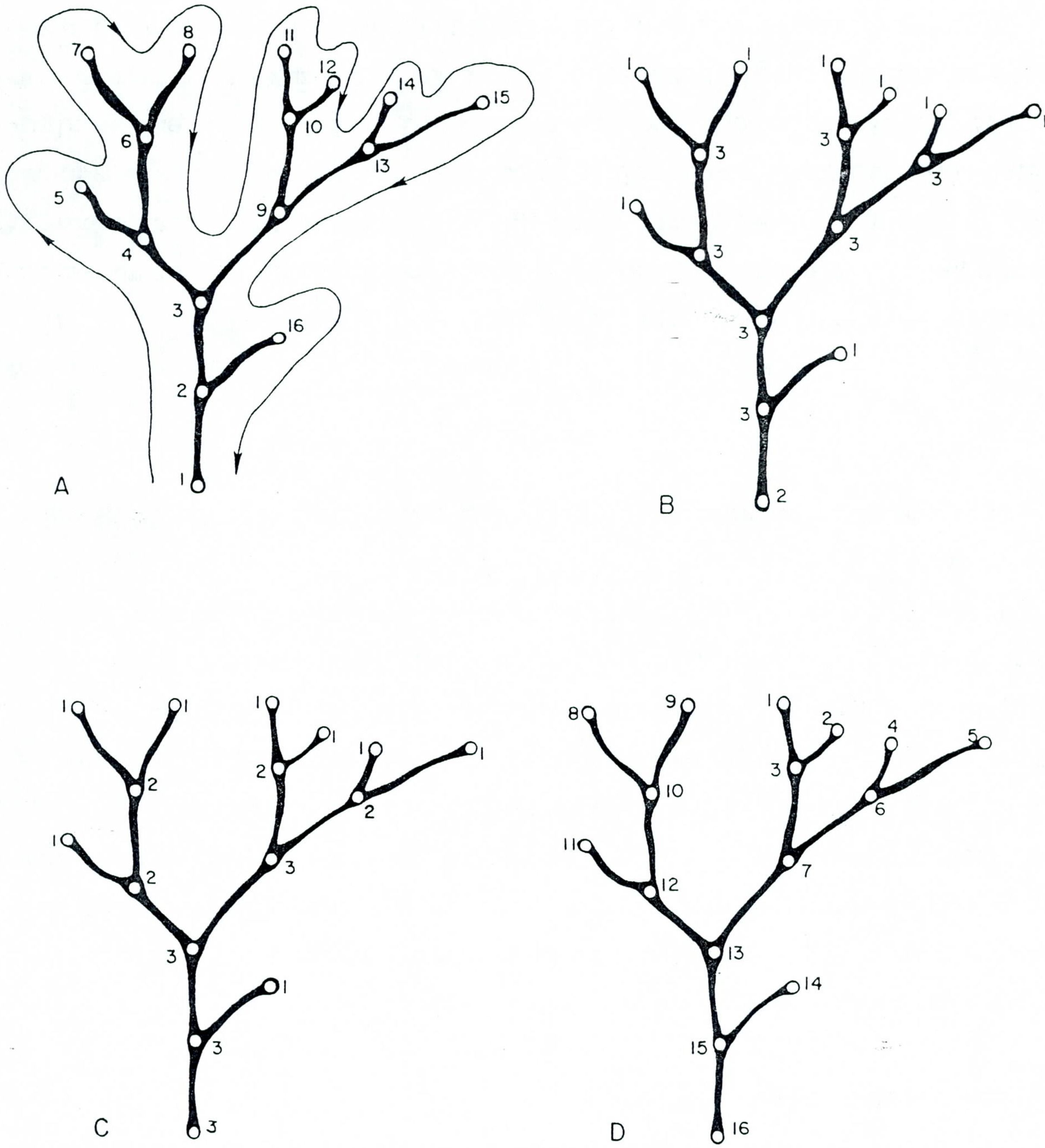

Figure 1. Schematic of a simple drainage network: a) node numbering following left hand pattern (as shown by arrow); b) node codes; c) Strahler's channel orders; d) channel flow routing sequence as determined by DNCFR.

2 - Cross Section Hydraulic Properties Component

Hydraulic properties of channel cross-sections (hereafter referred to as HPs) are required for numerical channel flow routing. HPs of interest are cross-sectional area, top width and conveyance factor. They are a function of stage, and therefore, require repeated evaluation during flow routing as stage varies with discharge. This calls for an efficient scheme to quantify the HPs. Model DNCFR uses the power function approach in which the HPs are approximated by a power function with flow depth as the independent variable (Li et al., 1975; Simons et al., 1982; Brown, 1982).

$$HP = mD^{p} \tag{1}$$

where HP is the hydraulic property, D is flow depth and m and p are coefficients of the power function.

The coefficients of the power functions are computed by a least squared regression through the logarithm of incremental depth and HP data points. This approximation of HPs is computationally effective and generally accurate for simple concave sections. In the case of compound sections, model DNCFR performs the routing separately in the main and overbank channel portions, as previously stated and as discussed subsequently. Therefore, compound sections are broken into two simple sections, and two separate power functions, one for each channel portion, are developed as illustrated in Fig. 2 for the wetted perimeter. It is assumed that the power function accurately represents the rating curve for simple sections.

3 - Channel Flow Routing Component

The channel flow routing is based on the Muskingum-Cunge routing method with variable parameters, with further adaptations to allow for variable time and space increments, and routing in compound sections. Even though these four items are fully integrated, they are, for clarity purposes, presented separately.

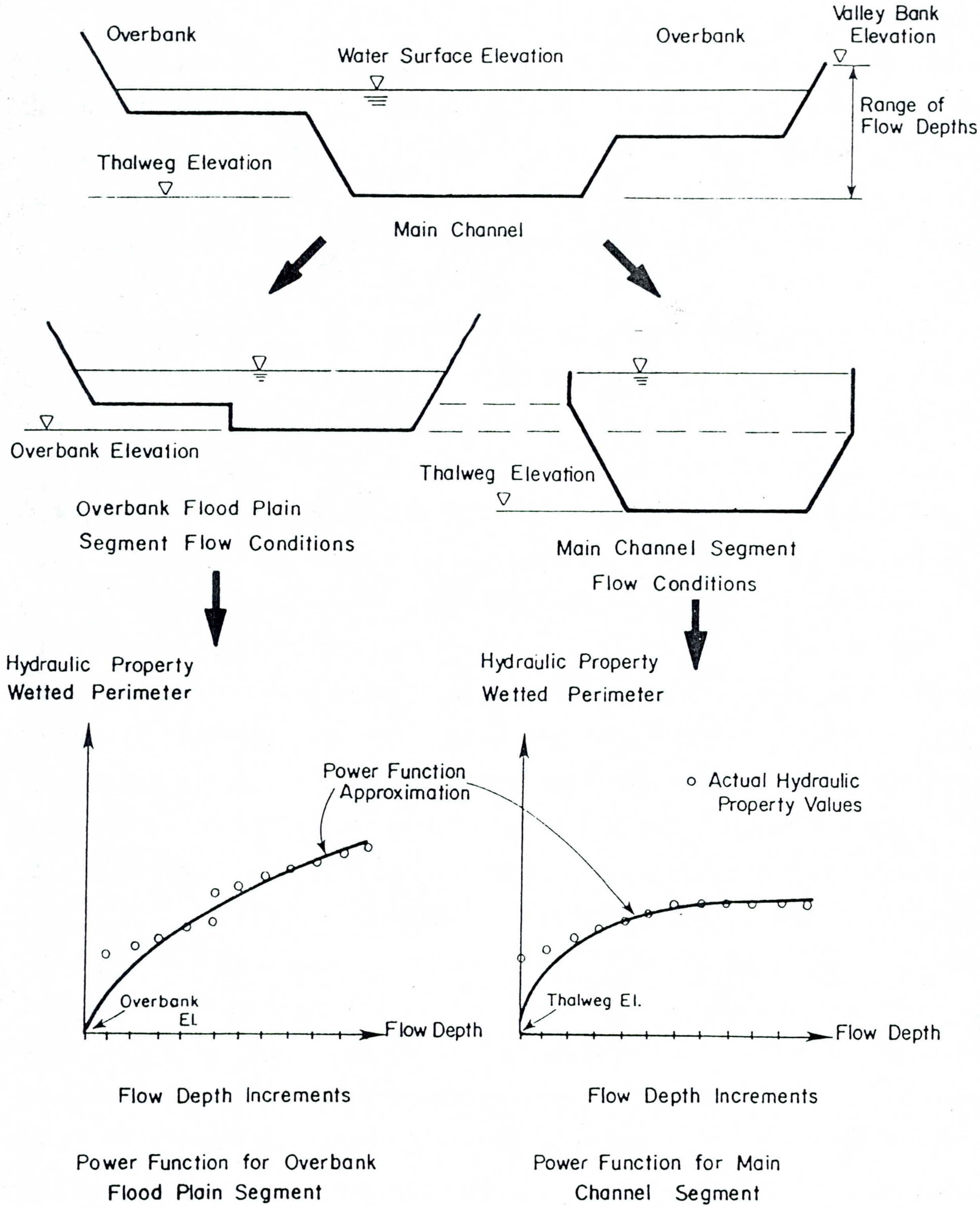

Figure 2. Schematic of wetted perimeter representation for compound cross sections (not to scale).

- Channel Flow Routing Scheme

The channel flow routing scheme is based on the Muskingum-Cunge routing method with variable parameters. The Muskingum-Cunge method, and refinements thereof, have been amply documented in previous work by Cunge (1969), Koussis (1978, 1980), Ponce (1983), Ponce and Yevjevich (1978), Smith (1980), and Weinmann and Laurenson (1979). The method is a kinematic wave routing method. The kinematic wave equation is transformed into a diffusion equation by numerical attenuation of the imperfectly centered finite-difference scheme (Smith, 1980). The method therefore accounts for hydrograph convection and diffusion, i.e. for downstream movement and peak attenuation of the hydrograph. Diffusion is introduced through two weighting coefficients which are determined from physical channel properties and flow characteristics (Cunge, 1969). When these coefficients are varied as a function of flow the method becomes a non-linear coefficient method (Koussis, 1976; Laurenson, 1962). The Muskingum-Cunge routing method with variable parameters accounts for most of the flood wave phenomena when practical applications are considered (Ponce and Theurer, 1982; Weinmann and Laurenson, 1979). The advantages of this method over other hydrologic techniques such as normal depth, Modified Puls, or simple Muskingum method are: (1) the scheme is stable with properly selected coefficients (Smith, 1980; Ponce, 1981; Ponce and Theurer, 1982); (2) it produces consistent results in that the results are reproducible with varying grid resolution (Jones, 1983; Koussis, 1983; Ponce and Theurer, 1982, 1983a and 1983b); (3) it is comparable to the diffusion wave routing (Cunge, 1969; Miller and Cunge, 1975); (4) the coefficients of the method are physically based (Cunge, 1969); (5) the method has been shown to compare well against the full unsteady flow equation over a wide range of flow situations (Ponce, 1981; Younkin and Merkel, 1988a and 1988b); and (6) the solution is largely independent of time and space intervals when these are selected within the spatial and temporal resolution criteria (Ponce, 1981; Ponce and Theurer, 1982). The essential steps of this method are briefly summarized in the following. Detailed formulations and discussions can be found in the above cited literature.

The Muskingum-Cunge routing scheme uses a storage relation to relate inflow and outflow in a channel reach. The storage relation is given by:

$$S = K[XI + (1 - X)O] \qquad (2)$$

where K is a storage coefficient, X is a weighting factor, I is the inflow rate to the reach, and O is the outflow rate from the reach. The finite difference formulation of Eq. 2 results in the Muskingum Equation (Cunge, 1969; Weinmann and Laurenson, 1979):

$$Q_{j+1}^{n+1} = C_1 Q_j^n + C_2 Q_j^{n+1} + C_3 Q_{j+1}^n \qquad (3)$$

with

$$C_1 = \frac{\dfrac{\Delta t}{K} + 2X}{F} \qquad (4)$$

$$C_2 = \frac{\dfrac{\Delta t}{K} - 2X}{F} \qquad (5)$$

$$C_3 = \frac{2(1 - X) - \dfrac{\Delta t}{K}}{F} \qquad (6)$$

$$F = \frac{\Delta t}{K} + 2(1 - X) \qquad (7)$$

where n is time superscript, j is space subscript, Q is discharge, and Δt is the routing time increment of the finite difference cell. In the original Muskingum equation, the value of the storage coefficient K and the weighting factor X are determined by trial and error or by calibration with observed hydrographs (Miller and Cunge, 1975). In the

Muskingum-Cunge approach coefficients K and X are expressed in terms of flow, channel and finite difference cell parameters (Cunge, 1969; Koussis, 1978; Ponce and Theurer, 1982; Weinmann and Lawrence, 1979) as:

$$K = \frac{\Delta x}{c} \tag{8}$$

$$X = \frac{1}{2}\left(1 - \frac{q}{S_o c \Delta x}\right) \tag{9}$$

where Δx is the space increment of the finite difference cell, c is a representative floodwave celerity, q is a representative unit width discharge, and S_o is the channel bed slope. With Eq. 8 and 9, the need of observed hydrographs to calibrate the coefficients K and X is eliminated. Cunge (1969) also demonstrated that the Muskingum-Cunge scheme, given by Eqs. 3 through 9, is equivalent to a convection-diffusion wave model, i.e. accounting for downstream movement and peak attenuation of the hydrograph.

Discharge and flood wave celerity are generally different at various points along a flood wave. To account for some of this observed nonlinearity, Koussis (1976), Weinmann and Laurenson (1980), and Ponce and Yevjevich (1978) presented the concept of variable coefficients. They redefined coefficients K and X for every computational cell as a function of updated values of unit width discharge and wave celerity. The unit width discharge and wave celerity at a grid point (j, n) are defined as:

$$c = \frac{dQ}{dA}\bigg|_{j,n} \tag{10}$$

$$q = \frac{Q}{B}\bigg|_{j,n} \tag{11}$$

where Q is total discharge, A is flow area, B is top width, and c is the floodwave celerity. The celerity is derived from the equation of continuity following the Kleitz-Seddon principle (Chow, 1959). The relation between discharge and flow area (Eq. 10) is based on

Manning's uniform flow equation with energy slope equal to bed slope. It is therefore a kinematic wave celerity. The average flood wave celerity for a computational cell is given as the average value of the celerity at the four nodes of the cell.

For a computational cell, the unknown unit width discharge and wave celerity are evaluated by a four-point iterative approximation (Ponce and Yevjevich, 1978). To begin the iteration an initial estimate of the discharge for the unknown grid point (j+1, n+1) is obtained using a linear projection of the known discharge at points (j, n), (j+1, n) and (j, n+1). Thereafter, a four-point iteration is used to solve for the discharge at the unknown point. The relation between discharge, flow area, top width, and flow depth is defined by power functions which are derived using cross section shape and Manning's uniform flow equation. These power functions represent simple rating curves.

The Muskingum-Cunge method with variable parameters was found to be accurate for a wide range of simple channels and flow conditions (Younkin and Merkel (1988a and 1988b)). Younkin and Merkel (1988b) performed 340 routing tests and compared the results to those from a full dynamic model used as a benchmark. They found that peak discharge, peak area, times to peak, and correlation of hydrograph shapes satisfied over 80 per cent of the Soil Conservation Service (SCS) field conditions covered by their study. The accuracy criteria used in their study were: (1) less than one percent difference for peak discharge and area; (2) one or less time step difference between time of occurrence of discharge and area peaks; and, (3) greater than ninety-five percent shape correlation for discharge and area hydrographs. However, the flow routing scheme does not account for backwater effects and it diverges from the full unsteady flow solution for very rapidly rising hydrographs in flat channels with slope less than 0.0002 (Brunner, 1989).

- Variable Computational Time Increment

Variable computational time increments are introduced to increase numerical efficiency of the routing scheme. Large time increments are used during inter-storm periods when relatively constant discharges prevail. Shorter time increments apply when discharge varies rapidly during rainfall-runoff events. The change in time increment is gradual to assure smooth transitions and to prevent a hydrograph from moving from a region with fine computational cells to one with coarse ones.

Variable time increments are compatible with the finite difference formulation of the Muskingum-Cunge routing scheme (Eqs. 3 through 7) because the latter is an explicit scheme. Flow calculations dependent only on the current space and time increment, and they are independent of any other computational cell. As a result there are no requirements to keep Δx and Δt constant throughout the computational domain, and they may vary within limits established by the accuracy criteria set forth by Ponce and Theurer (1982).

The size of the time increment is determined as a function of rate of change in upstream inflow into the channel reach. The inflow time series is scanned ahead. If a change in discharge above a given threshold value is sensed, the current time increment is reduced; if no change in discharge is found, the size of the next time increment is increased. Upper and lower bounds for the time increment size are one day and five minutes, respectively. These boundaries were found to work well for long-term simulation in drainage networks. In addition to the smooth transition between fine and coarse time increments, the early reduction of the time increment size as an upcoming perturbation is sensed assures an adequate temporal resolution for hydrograph routing that generally satisfies the accuracy criteria of a minimum of 5 time increments on the rising portion of a hydrograph (Ponce and Theurer, 1982).

- Computational Space Increment

Computational space increments, Δx, are subreaches that define the computational cell size at which the numerical flow routing is performed. A computational space increment may be equal to the entire routing reach length or to a fraction of that length. It is initially selected as the entire reach length. If the size of this space increment does not meet the accuracy criteria for flow routing given by Ponce and Theurer (1982), it is reevaluated by subdividing the length of the routing reach into even subreaches that produce Δx's that satisfy the accuracy criteria. Ponce and Theurer's accuracy criteria is given by:

$$\Delta x \le \frac{1}{\lambda}\left(\Delta x_c + \Delta x_D\right) \tag{12}$$

with

$$\lambda = 2 \tag{13a}$$

$$\Delta x_c = c\Delta t \tag{13b}$$

$$\Delta x_D = \frac{q}{S_0}c \tag{13c}$$

where q and c refer to a reference discharge and celerity, respectively, λ is an accuracy parameter and Δt is the minimum time increment. The minimum time increment is used because it is the one applicable during routing of a hydrograph. The reference discharge is generally two thirds of the peak flow above base flow, and the reference celerity is the celerity corresponding to the reference discharge.

The upper limit of the space increment, as given by Eq. 12, becomes quite large for very flat channels and high discharge values. In long channel reaches where such large space increments can be implemented, the flow routing may produce inaccurate hydrographs. First, the time separation between inflow and outflow hydrographs can become large resulting in the computed outflow hydrograph to end up in a region of coarse time increments. In this case the upper limit of the space increment depends on the duration and celerity of the hydrograph. Short duration and fast moving hydrographs require shorter space increments than long duration and slow moving hydrographs. As a rule of thumb the average hydrograph travel time in a space increment should not exceed about one fifth of the duration of the inflow hydrograph.

In the second case, during overbank flow conditions, long space increments may result in the hydrograph in the main channel to significantly outpace the hydrograph on the overbank portion of the cross section. This outpacing is a natural phenomena that changes hydrograph shape. Flow from main channel hydrograph spills onto the flood plains, peak runoff rate decreases, and the recession limb is stretched out as a result of return flow from the overbanks. In the present routing scheme, the uncoupling of the main channel and overbank flow routing in very long space increments may produce an insufficient flow mixing between the two channel portions and, as a result, the effect of flood plain storage on hydrograph shape is not accurately simulated. An upper limit for the space increment of about one twentieth of the wave length was found to provide, in most cases, an adequate flow mixing. Under real world conditions tributary junctions in drainage networks and changes in cross section and flood plain characteristics generally provide short enough channel reach length that do not require limitations on the space increment.

- Routing in Compound Cross Sections

Main and overbank channel portions are separated and modeled as two independent channels. Right and left overbanks are combined into a single overbank channel. At the upstream end of a space increment total inflow discharge is divided into a main channel and an overbank flow component. Each is then routed independently using the previously described routing scheme. Momentum exchange at the flow interface between the two channel portions is neglected and the hydraulic flow characteristics are determined for each channel portion separately. At the downstream end of the space increment both flow components are summed to yield the total outflow discharge. The flow exchange between main channel and overbank channel during routing within a space increment is neglected.

Flow redistribution between main and overbank channels is based on the assumption of a constant energy head perpendicular to the flow direction and on a negligible momentum exchange at the flow interface between the two channel portions. As the stage in the main channel exceeds overbank elevation, the discharge in each channel portion is determined by matching the energy head of the flow. The energy head is computed using Bernoulli's conservation of energy equation and mean flow values for each channel portion:

$$Z_m + d_m + \frac{v_m^2}{2g} = Z_o + d_o + \frac{v_o^2}{2g} \tag{14}$$

$$Q_T = Q_m + Q_o \tag{15}$$

with v = Q/A; A = fct (Q, XSS, N); and d = fct (Q, XSS, N); where Z is elevation above a reference datum, v is flow velocity, XSS is channel section shape, N is channel roughness and d is flow depth. Subscript m stands for main channel, o for overbank channel, and T for total. Flow area, A, and flow depth, d, are determined as a function of cross section shape and Manning's uniform flow equation. The described separation into main and overbank flow components introduces a lateral variation in flow characteristics which make the routing scheme a quasi two dimensional approach. From the point of view of flow routing the uncoupling of main and overbank flow results in a flood wave propagation that is primarily controlled by the faster moving flow in the main channel, and a wave attenuation that is primarily controlled by the storage of the overbank channel.

4 - Coordination Component of Drainage Network Routing

This model component uses the network topology data determined in model component 1 to coordinate the routing in the drainage network. It defines and feeds the source area runoff into the channels, merges appropriate channel flows at network junctions, and executes the flow routing in proper sequence for all channel reaches. Because this model component performs simple bookkeeping tasks, no further explanations are given.

VERIFICATION APPROACH

The original Muskingum-Cunge channel flow routing with variable parameters was found to be accurate for a wide range of channel geometries and flow conditions (Koussis, 1978; Ponce, 1981; Ponce and Theurer, 1982; Younkin and Merkel, 1988a and 1988b). Brunner (1989) indicated that the method diverges from the full unsteady flow solution for very rapidly rising hydrographs in flat channel with slopes less than 0.0002. In the context of this report, DNCFR is tested for channels with simple and compound sections and for complex drainage networks.

Verification is accomplished by comparing flow routing results with corresponding results from models solving the full unsteady flow equations. This benchmark verification approach is preferred over actual field data, because field data often includes processes unrelated to flow routing such as infiltration on overbank flood plains, or variable flow resistance due to the submergence of vegetation. These effects are generally hard to measure, and calibration to site-specific conditions is often required. The latter makes any comparison between model results and field conditions highly subjective. Benchmark verification assures well defined and identical boundary conditions. It provides an objective comparison to state-of-the-art one-dimensional hydraulic modeling capabilities.

The models selected as benchmark are DAMBRK (1988 version) of the National Weather Service (Fread, 1984) and UNET (Version 1.1) by Barkau (1990). The DAMBRK model is used for verification of flow routing in single channels with simple and compound sections. The UNET model is used to verify the flow routing in drainage networks. These models were selected because of the solution to the full dynamic flow equations and no model comparison is intended.

Verification criteria are peak discharge, time to peak, runoff volume and hydrograph shape. Discrepancies from the benchmark are quantified in percent deviation for the first three parameters. Hydrograph shape is verified visually and quantified by the correlation coefficient between computed and benchmark hydrograph values. Finally, the numerical performance of DNCFR versus the hydraulic benchmark models is defined in terms of a reduction in overall execution time.

VERIFICATION TEST CASES

Eighteen verification cases are presented. Of these, ten are single channels with simple cross sections, six are single channels with compound sections, and two are drainage networks with a mix of channels with simple and compound sections. Channel geometry, resistance to flow, and inflow hydrograph characteristics for the single channel tests are given in Table 2. A schematic of the trapezoidal channel cross sections for test cases 2.5 and 2.6 are shown in Fig. 3. The inflow hydrographs are generated using a gamma function (Ponce and Theurer, 1982). The inflow hydrograph of test 1.4 has a dimensionless wave period, τ, greater than 171 (Table 2), and, according to Ponce at al. (1978), it is classified as a kinematic wave. Of the other inflow hydrographs seven are diffusion waves with a τ / F_r factor greater than 30, and eight are dynamic waves with a τ / F_r factor under 30. The predominance of diffusion and dynamic inflow hydrographs makes the selected hydrographs relevant for the testing of DNCFR.

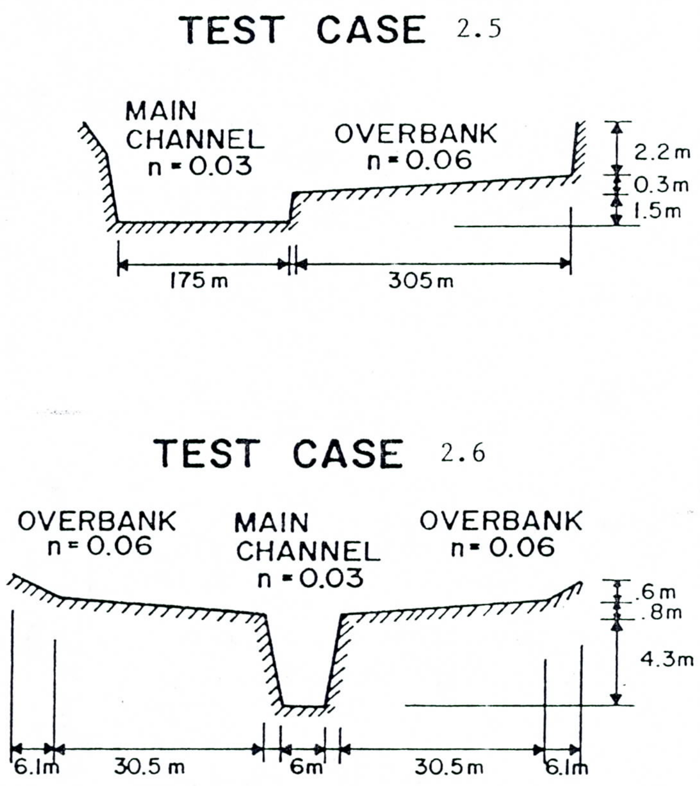

Figure 3. Schematic of cross section for test cases 2.5 and 2.6 (distorted vertical scale).

The hypothetical drainage network for the two network tests are shown in Fig. 4. The watershed is approximately 17 km long and 9 km wide. Channel slopes vary from 0.0016 to 0.004; first and second order channels have no overbank flood plains; third order channels have significant flood plains. The inflow hydrographs are mostly of the

Table 2: Channel geometry and inflow hydrograph characteristics.

	CHANNEL									HYDROGRAPH				
Test number	Length km	Slope -	Shape -	Top width Main m	Top width O.B. m	Depth Main m	Depth O.B. m	Manning's n Main -	Manning's n O.B. -	Peak discharge cms	Base flow cms	Time to peak min	τ	τ/F_r
1.1	25.	.002	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	60	13	32
1.2	25.	.002	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	180	39	94
1.3	25.	.008	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	60	119	154
1.4	25.	.008	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	180	357	463
1.5	25.	.0006	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	60	2	8.5
1.6	25.	.0006	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	180	5.7	24
1.7	25.	.0002	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	60	0.3	2.4
1.8	25.	.0002	Rect	300.0	NA	10.0	NA	.04	NA	2000	1000	180	1	7
1.9	29.	.00038	Rect	175.0	NA	5.0	NA	.03	NA	1250	170	370	8	31
1.10	29.	.00028	Rect	175.0	NA	5.0	NA	.03	NA	1270	170	370	3	18
2.1	25.	.002	Rect	100.0	400.0	3.8	5.0	.04	0.6	2000	300	60	12	29
2.2	25.	.002	Rect	100.0	400.0	3.8	5.0	.04	0.6	2000	300	180	32	75
2.3	25.	.0006	Rect	100.0	400.0	3.8	5.0	.04	0.6	2000	300	60	1.9	8.3
2.4	25.	.0006	Rect	100.0	400.0	3.8	5.0	.04	0.6	2000	300	180	1.7	6.9
2.5	29.	.002	Trap	178.0	305.0	1.5	2.5	.03	0.6	1256	170	360	60	158
2.6	29.	.0019	Trap	9.0	73.2	4.3	1.0	.03	0.6	211	2.8	360	54	132

τ = Dimensionless wave period; F_r = Froud Number.
O.B. = Overbank flood plains; NA = Not applicable.

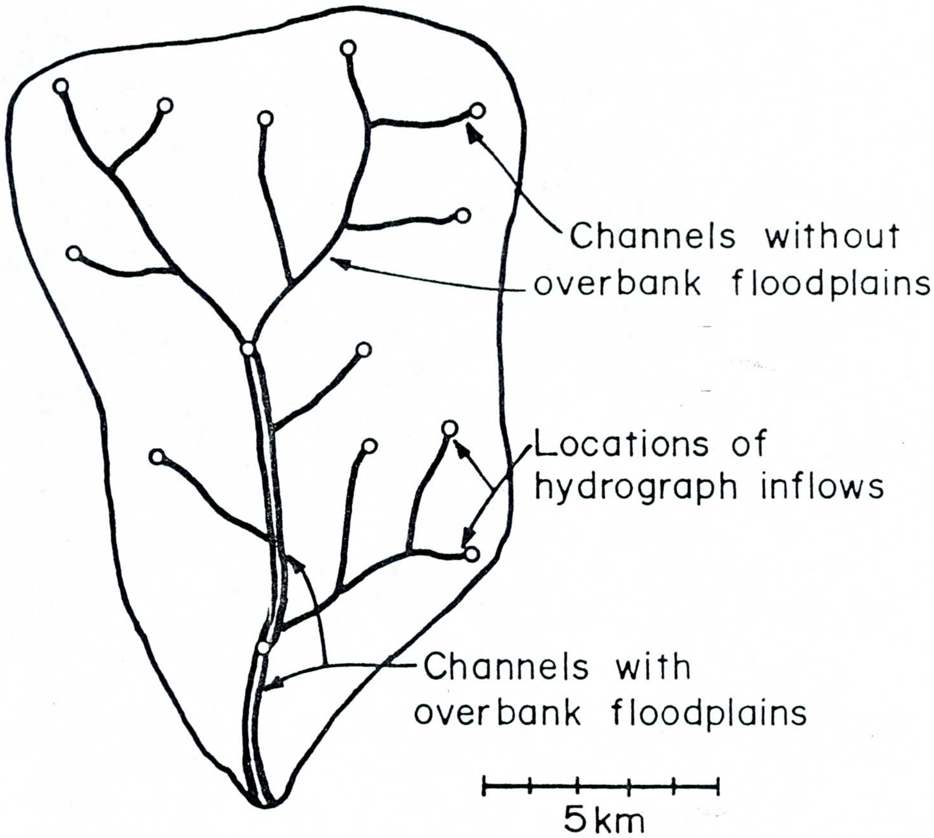

Figure 4. Hypothetical drainage network for tests 3.1 and 3.2.

diffusion type, and are entered at source nodes and selected junction nodes as indicated by hollow circles in Fig. 4. Peak and time to peak of the inflow hydrograph are chosen arbitrarily; they range from 3.0 to 7.5 cms, and 55 to 80 min, respectively, for test 3.1; and from 3.0 to 6.5 cms and 145 to 170 min, respectively, for test 3.2. Hydrograph shapes are generated using a gamma function. A storm movement at about 20 km/hr is assumed from the outlet to the top of the basin.

RESULTS OF VERIFICATION

The results of the verification are shown in Table 3. For channels with simple sections, hydrograph peak and time to peak are, on the average, less than 3% off the

Table 3: Verification results of benchmark and DNCFR, and differences between the two.

	BENCHMARK				DNCFR				DIFFERENCE			
Test	Peak discharge Q cms	Time to peak T_p min	Volume V $10^7 m^3$	Execution time* T_e sec	Peak discharge Q cms	Time to peak T_p min	Volume V $10^7 m^3$	Execution time* T_e sec	ΔQ %	ΔT_p %	ΔT_e %	Shape Correlation *100
1.1	1643.1	186	8.94	ND	1654.0	185	8.94	6	0.7	-0.5	NA	100
1.2	1916.2	300	9.55	ND	1921.4	300	9.55	6	0.3	0.0	NA	100
1.3	1916.5	138	8.94	ND	1968.2	135	8.95	4	2.7	-2.2	NA	100
1.4	1978.5	252	9.55	136	1984.5	255	9.55	6	0.3	1.2	-96	100
1.5	1309.0	226	8.94	ND	1275.3	235	8.94	4	-2.6	4.0	NA	99
1.6	1670.4	348	9.55	ND	1642.2	360	9.55	5	-1.7	3.5	NA	100
1.7	1176.0	210	8.94	74	1120.6	205	8.94	3	-4.8	2.4	-96	94
1.8	1433.9	348	9.55	ND	1337.1	355	9.55	3	-6.7	2.0	NA	97
1.9	1136.1	570	5.65	ND	1108.4	585	5.65	3	-2.4	2.6	NA	100
1.10	1079.4	585	5.65	ND	1024.5	615	5.65	3	-5.1	5.1	NA	100
						Av. of abs. values			2.7	2.3	-96	99
2.1	1058.6	216	3.11	1050	980.1	214	3.11	14	-7.4	-0.9	-99	96
2.2	1588.6	420	4.14	1656	1598.6	425	4.14	22	0.6	1.2	-99	95
2.3	605.5	396	3.11	ND	586.6	425	3.11	18	-3.1	7.3	NA	98
2.4	1017.6	525	4.14	ND	1095.5	544	4.14	18	7.7	3.6	NA	98
2.5	1238.3	605	4.17	396	1210.6	604	4.16	46	-2.2	-0.2	-88	97
2.6	193.9	665	0.6	294	198.1	675	0.58	46	2.2	1.5	-84	98
						Av. of abs. values			3.9	2.5	-93	97
3.1	47.0	325	0.092	623	49.8	315	0.091	56	6.0	-3.0	-83	97
3.2	68.7	370	0.143	985	76.8	364	0.149	62	11.8	-1.7	-94	100
						Av. of abs. values			8.9	2.3	-89	98

* Duration of simulation: 24 hrs; minimum time increments: 5 min.; includes I/O.
ND: No data; NA: not applicable.

benchmark values; for compound sections they are, on the average, less than 4% off the benchmark values. In general, tests having hydrographs of the dynamic type (tests 1.5, 1.6, 1.7, 1.8, 2.1, 2.3, and 2.4) display a larger discrepancy than those having hydrographs of the diffusion type (tests 1.1, 1.2, 1.3, 2.2, 2.5, and 2.6). This is to be expected because DNCFR is equivalent to a diffusion routing method (Cunge, 1969), and it does not account for dynamic effects. Considering the highly dynamic character of some of the hydrographs, the results of DNCFR are good for a hydrologic routing method. For example, in channels with simple sections and dynamic hydrographs (tests 1.5, 1.6, 1.7, and 1.8), about 91% of the attenuation is reproduced by DNCFR. For channels with compound sections, about 94% of the total attenuation (due to diffusion and storage) is reproduced by DNCFR.

Figure 5 show a plot of discharge and time to peak from DNCFR versus corresponding benchmark values. The data represents all tests of Table 1, including the two network applications. Neither discharge nor the time to peak show significant deviations from the line of perfect agreement. In the following peak discharge, time to peak and hydrograph shape are discussed in more detail for selected tests.

With respect to the drainage network tests, the complex hydrographs are the result of the drainage network configuration and movement of the storm up the watershed. The runoff from the lower right network branch arrives first at the outlet followed by the main peak from the upper portion of the watershed. The higher peak values by DNCFR are primarily the result of limitations regarding backwater and reverse flow effects. Indeed, the hydraulic simulations include reverse flow up tributary branches. Reverse flow is the result of high stages in the main channel while low stages prevail on the tributaries. The net effect of this reverse flow is additional storage and attenuation of the peak. Times to peak and hydrograph shapes are well reproduced for both network applications. The hydrographs leaving the drainage network are shown in Fig. 6. Peak, timing and shapes are well reproduced. The correlation coefficient for hydrograph shape is 0.98.

For simple and compound sections, the average correlation coefficient for hydrograph shape is 0.99 and 0.97, respectively. Test 1.6 is a dynamic wave with a 35% attenuation of the peak above base flow. In this case, like in all other dynamic cases, the

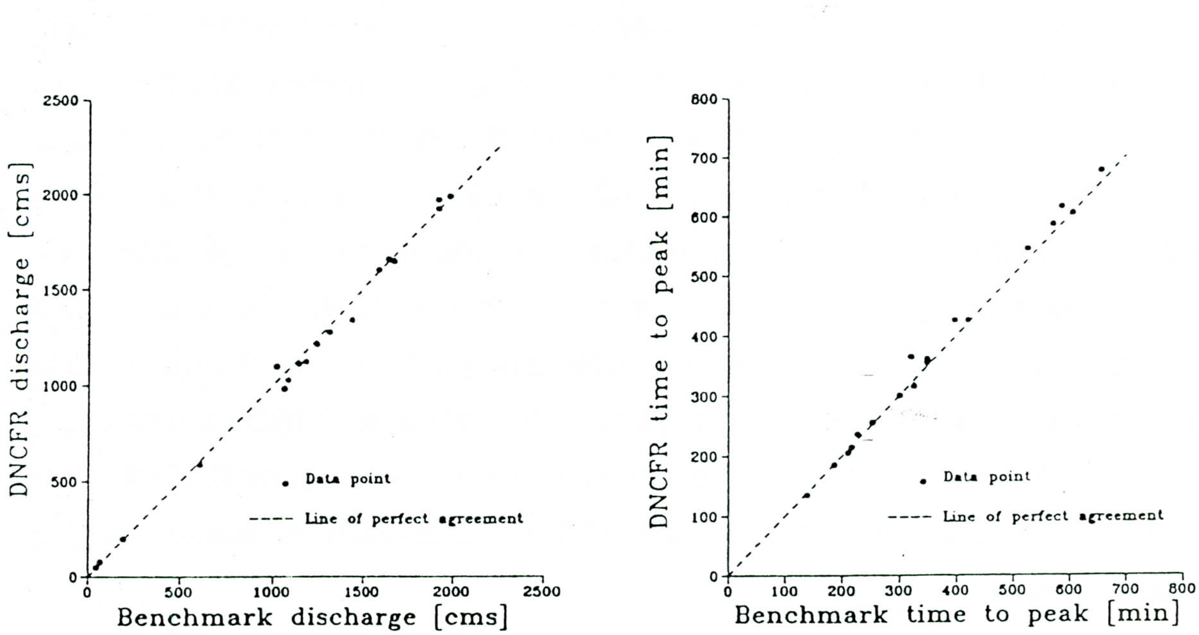

Figure 5. Benchmark vs. computed discharge and time to peak; all verification test results.

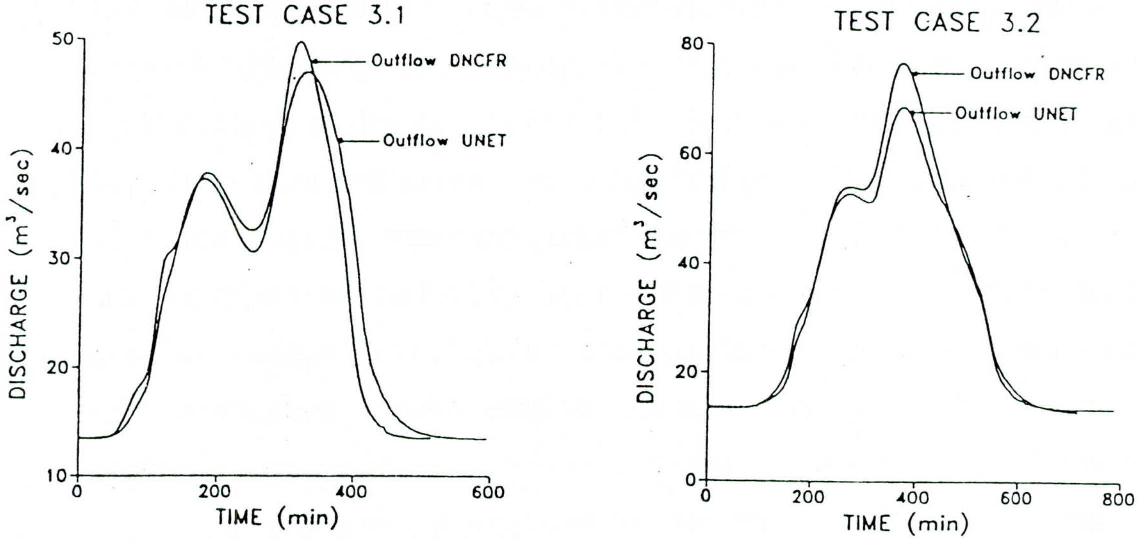

Figure 6. Hydrographs for tests 3.1 and 3.2.

hydrograph computed by DNCFR displays a larger attenuation than the benchmark. As a result of the lower flow, the time to peak lags slightly behind the benchmark value. Test 1.9 is a diffusion wave with 11% attenuation of the peak above base flow. It is typical of most diffusion wave cases with little discrepancy in peak, time to peak and hydrograph shape.

Outflow hydrographs for channels with compound section are given by tests 2.1, 2.2, 2.5 and 2.6 which are shown in Fig. 7. Test 2.1 shows a hydrograph that is attenuated entirely below overbank flood plain elevation. This example also shows the effect of the slower moving overbank flow by producing a longer hydrograph recession limb. In the other three test cases, overbank flow is active along the entire channel length. For these cases the location of beginning of overbank flow is clearly defined by the breaks in the rising limb of the hydrographs. Overall, the distortions of hydrograph shape due to overbank flood plains are consistent with the benchmark shapes.

Finally, the execution time of DNCFR is, on the average, 92% shorter than for the benchmark hydraulic evaluation (Table 3). The comparison is made on an IBM compatible PC, having an 80836 - 20MHZ micro-processor and math co-processor, and using the Microsoft FORTRAN compiler Version 5.0 with optimization. Program I/O and support computations are included in the execution time. Using the average execution time over all tests, the reduction is from 10 minutes to 32 seconds, which is a factor of 20, or about 1.2 orders of magnitude. It is believed that additional reductions in execution time can be achieved for long-term simulations because significantly larger time increments are used by the model for nearly constant discharge values that generally prevail between storm events.

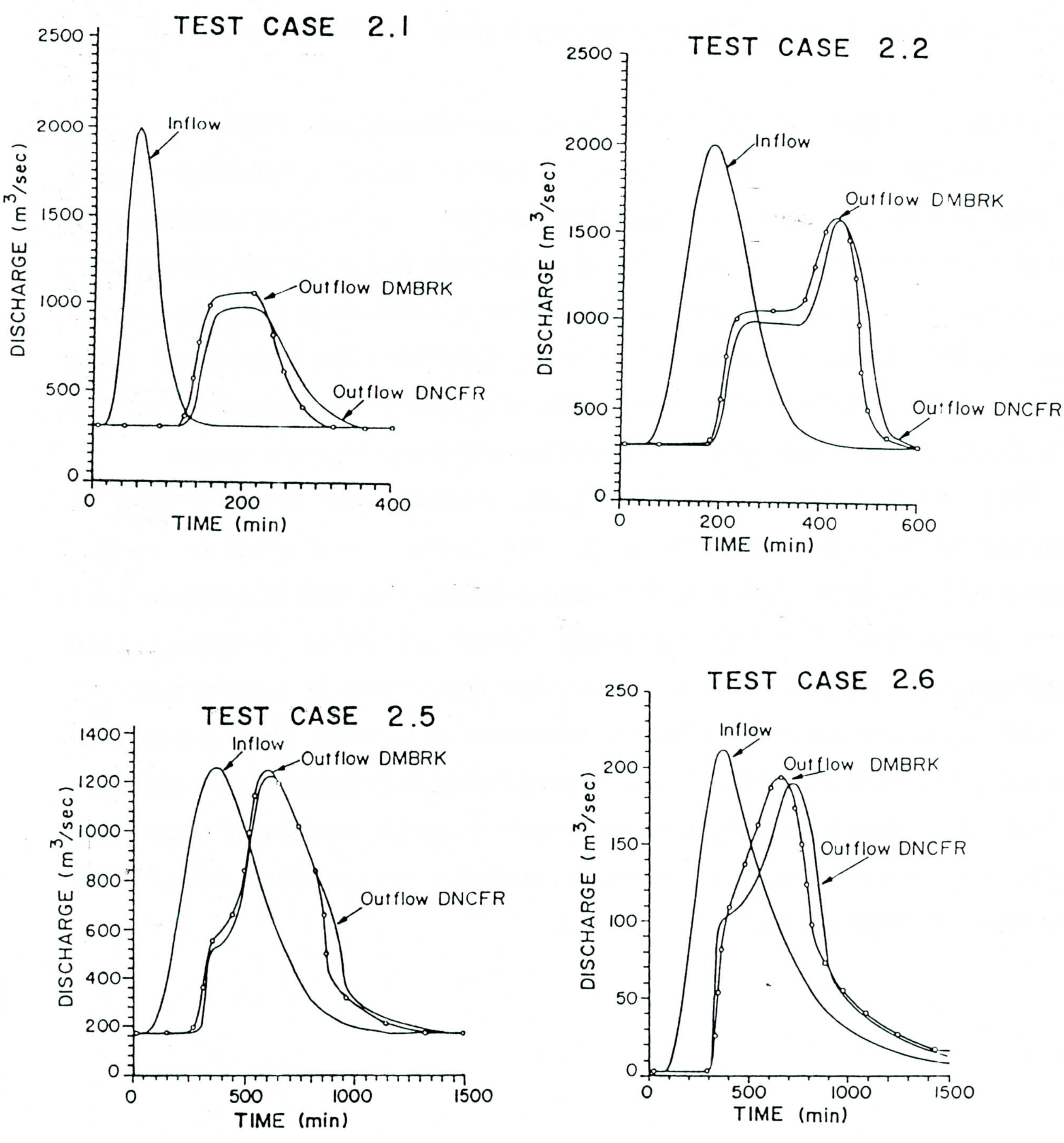

Figure 7. Verification hydrographs for tests 2.1, 2.2, 2.5, and 2.6.

SUMMARY AND CONCLUSIONS

The Muskingum-Cunge channel flow routing scheme with variable parameters is modified to account for compound sections, to include a variable time step, and to determine internally the computational reach increment. The resulting model, DNCFR, is verified for hypothetical channel and flow conditions. Routing results are compared with those of hydraulic models solving the full unsteady flow equations. Ten channels with simple sections, six with compound sections and two drainage network applications are selected for verification. For all tested cases, DNCFR reproduces the peak, time to peak and shape of the benchmark hydrograph with reasonable accuracy. Slight discrepancies (less than 10%) in the drainage network application are due to the limitations of hydrologic models with respect to backwater and reverse flow effects. The size of the discrepancy is well within the usual error of drainage network parameterization and lateral channel inflow determination. Program efficiency, as measured by the reduction in execution time, is, on the average, one order of magnitude faster than the benchmark hydraulic routing. The results of the verification indicate DNCFR to be an effective tool for hydrologic routing applications to large complex drainage networks and for continuous long-term simulations.

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REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

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2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION / AVAILABILITY OF REPORT Distribution of this publication is unlimted
2b. DECLASSIFICATION / DOWNGRADING SCHEDULE	
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Paper No. 135	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION US Army Corps of Engineers Hydrologic Engineering Center 6b. OFFICE SYMBOL *(If applicable)* CEWRC-HEC	7a. NAME OF MONITORING ORGANIZATION
6c. ADDRESS *(City, State, and ZIP Code)* 609 Second Street Davis, California 95616	7b. ADDRESS *(City, State, and ZIP Code)*
8a. NAME OF FUNDING / SPONSORING ORGANIZATION 8b. OFFICE SYMBOL *(If applicable)*	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS *(City, State, and ZIP Code)*	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. / PROJECT NO. / TASK NO. / WORK UNIT ACCESSION NO.

11. TITLE *(Include Security Classification)*
A Muskingum-Cunge Channel Flow Routing Method for Drainage Networks

12. PERSONAL AUTHOR(S)
Jurgen Gorbrecht and Gary W. Brunner

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT *(Year, Month, Day)*	15. PAGE COUNT
Final	FROM ________ TO ______	November 1991	30

16. SUPPLEMENTARY NOTATION
This paper was published in the ASCE Journal of Hydraulics, Volume 117, No. 5, May 1991.

17. COSATI CODES			18. SUBJECT TERMS *(Continue on reverse if necessary and identify by block number)*
FIELD	GROUP	SUB-GROUP	Muskingum-Cunge, Channel Flow Routing, drainage networks, Compound Cross-Sections, variable time step.

19. ABSTRACT *(Continue on reverse if necessary and identify by block number)*
A Muskingum-Cunge channel flow routing scheme is modified for application to large drainage networks with compound cross sections and for continuous long-term simulation. The modifications consist of a decoupling and separate routing of main and overbank channel flow, an introduction of a variable time step to increase model efficiency during periods of steady flow, and an interval determination of the numerical increment. The resulting hydrologic model is verfied by comparing its flow routing results with those of hydraulic benchmark models solving the full unsteady flow equations. Test conditions consist of hypothetical flood hydrographs, long prismatic channels with simple and compound sections, and a third order drainage network. For the tested conditions, the model produces hydrograph peaks, times to peak and shapes that compare well with those of the hydraulic benchmarks. Hydrograph distortions due to overbank flood plain storage and multiple peaks from complex drainage networks are also well reproduced. The execution time of the model is generally one order of magnitude faster than that of the hydraulic benchmark models.

20. DISTRIBUTION / AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS	21. ABSTRACT SECURITY CLASSIFICATION Unclassified
22a. NAME OF RESPONSIBLE INDIVIDUAL Gary W. Brunner	22b. TELEPHONE *(Include Area Code)* 916 756-1104 22c. OFFICE SYMBOL CEWRC-HEC-T

DD Form 1473, JUN 86 *Previous editions are obsolete.* SECURITY CLASSIFICATION OF THIS PAGE

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