FLOODFLOW CHARACTERISTICS OF FILBIN CREEK FOR PRE- AND POST-CONSTRUCTION CONDITIONS, 1986, AT NORTH CHARLESTON, SOUTH CAROLINA

By Curtis L. Sanders, Jr.

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CONTENTS

ILLUSTRATIONS

TABLES

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ABSTRACT

A study to determine the effect of the construction of a shopping and business center, and of the construction and improvement of several highways on floodflow in the Filbin Creek drainage basin near North Charleston, South Carolina was performed. Discharge hydrographs were synthesized using computerized U.S. Soil Conservation Service unit hydrograph methods and routed using reservoir, step backwater, and culvert flow programs. Construction of the shopping and business center, according to plans of July 1986, will raise the water-surface elevations upstream of Interstate Highway 26 by about 2.0 feet for runoff from 100-year rainfall. Structures at Seaboard Railroad downstream of U.S. Highway 52, U.S. Highway 52, and Virginia Avenue would cause about 2.0, 2.6, and 4.1 feet of backwater, respectively.

INTRODUCTION

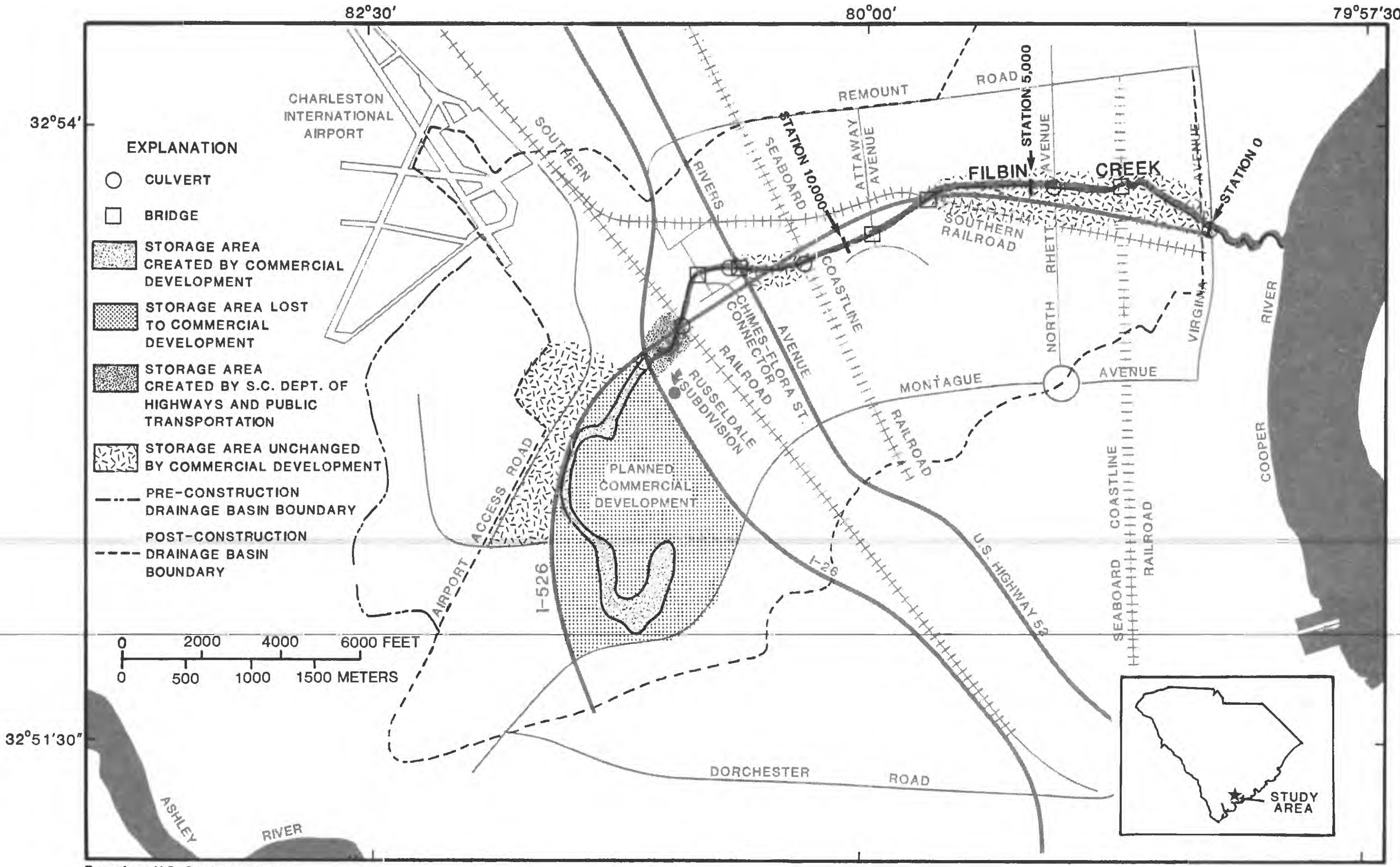
The South Carolina Department of Highways and Public Transportation (SCDHPT) is currently (1987) constructing Interstate Highway 526 (I-526) near Filbin Creek at North Charleston, South Carolina (fig.1). As part of this project, the SCDHPT will also excavate the area between I-26 and the Southern Railroad culvert to provide storage for runoff from I-526, will construct a bridge over Filbin Creek to connect Chimes and Flora Streets, and will modify the culverts at North Rhett and Virginia Avenues.

There are plans to fill an old phosphate mining area between I-26 and I-526 and to construct a shopping and business center on the fill. The phosphate mining area currently provides significant reduction of downstream flooding by temporarily storing storm runoff. Plans of July 1986 are to create new storage areas within the commercial development to compensate for the decreased storage and increased runoff that may result from basin development.

The City of North Charleston desires to alleviate the frequent flooding of residences between U.S. Highway 52 and Southern Railroad culvert and to prevent overtopping of the embankment at North Rhett Avenue.

Purpose and Scope

The purposes of this report, which was prepared by the U.S. Geological Survey in cooperation with the South Carolina Department of Highways and Public Transportation, are to:



Base from U.S. Geological Survey 1:24,000 Charleston Quadrangle

Figure 1.--Filbin Creek basin at North Charleston, South Carolina.

1. Define the profile of the 100-year water-surface elevations from Virginia Avenue to the area upstream of I-26 for pre-construction conditions, which excludes I-526, the airport access road, proposed fill and compensating storage at Centre Pointe, the added storage area excavated between I-26 and the Southern Railroad culvert, the Chimes-Flora Street connector bridge, and proposed modifications to culverts at North Rhett and Virginia Avenues. This profile includes the part of the Filbin Creek drainage basin modified by construction of the airport to drain part of the Filbin Creek basin to the Ashley River (figure 1) instead of the Cooper River. The Ashley River is 0.6 miles west of the upstream boundary of the Filbin Creek basin.

2. Define the profile of the 100-year water-surface elevations from Virginia Avenue to the area upstream of I-26 for post-construction conditions, which includes I-526, the airport access road, the proposed fill and compensating storage at the commercial development, the added storage area excavated between I-26 and the Southern Railroad culvert, the new Chimes-Flora Street connector bridge, and proposed modifications to North Rhett and Virginia Avenues. This profile excludes that part of the Filbin Creek drainage basin modified by construction of the airport to drain part of the Filbin Creek basin to the Ashley River instead of the Cooper River.

These flood profiles show the effects of currently (1987) planned land-use changes on the basin and will provide a reference against which the effects of additional changes can be compared. These profiles were computed without the effect of hurricane storm surges. Hurricane storm surge elevations (Federal Emergency Management Agency, 1986) are discussed briefly in the "Hurricane Storm Surge Profile" section of this report.

Site Description

Filbin Creek is a tributary of the Cooper River in Charleston County. The creek originates near the Charleston Airport in an abandoned phosphate mine area. The creek has a drainage area of about 2.4 square miles at I-26 and about 7.3 square miles at Virginia Avenue. The channel is not well defined upstream of I-26 where runoff collects in the strip-mined ridge-and-valley network and drains slowly toward the existing culvert at I-26. Downstream from I-26, Filbin Creek flows eastward for approximately 3.6 miles to the Cooper River; it passes through culverts at Southern Railroad, U.S. Highway 52 (South), Seaboard Coast Line Railroad, North Rhett Avenue and Virginia Avenue. The culverts at Virginia Avenue have been fitted with flapper valves (tide gates) to prevent reverse flow during high tides.

In addition to these culverts, Filbin Creek flows through four bridges in the study area. The location and length of these bridges are listed below:

Bridge site	Length (feet)	Distance upstream from Virginia Avenue (feet)
U.S. Highway 52 (north)	45.5	12,600
Attaway Avenue	50.5	9,100
Southern Railroad	53.0	7,600
Seaboard Coast Line Railroad	75.0	2,450

The Filbin Creek channel has been dredged and straightened in some places and occupies a relatively broad, flat flood plain. The entire floodplain is densely vegetated with brush and trees in the upper reaches and marshland vegetation near the downstream boundary of the study area. The basin is urban, and has several areas of residential encroachment on the flood plain.

Previous Investigations

Davis and Floyd, Inc. (1980) computed profiles for the 5-year recurrence interval flood on Filbin Creek as part of a study of drainage systems of North Charleston. Hydrographs were synthesized using Soil Conservation Service (SCS) unit hydrographs and the SCS recommended 5-year, 24-hour rainfall distribution. Hydrographs were routed downstream using SCS methodology.

Bohman (1984) computed flood profiles for conditions before and after construction of I-526 and the diversion of some of the Filbin Creek drainage basin to the Ashley River by Charleston Air Force Base. Hydrographs were determined using dimensionless hydrograph techniques developed by Stricker and Sauer (1982). High-water marks obtained after the study indicated that the computed profiles were too low. The SCS unit hydrograph methods using several uniform rainfall distributions were determined to be more applicable for Filbin Creek than the Stricker and Sauer dimensionless hydrographs or the SCS 24-hour rainfall distribution. This report updates the Bohman report.

METHODS OF ANALYSIS

The Filbin Creek basin contains several sources of variable backwater and areas of significant detention storage. Therefore, a steady-state step-backwater model and peak discharges from regional flood frequency relations were not considered to be appropriate for developing surface-water profiles in the basin. Profiles of 100-year water surface elevations for Filbin Creek were computed by using a rainfall-runoff model to generate hydrographs and by using hydrologic and hydraulic models to route hydrograph discharges. The profile of the 100-year water-surface elevations is the composite of the highest elevations computed from routing hydrographs from several 100-year durations of rainfall through the basin. Several durations of rainfall were used to fully define the combined effects of large storage areas and urbanized areas on discharges and water-surface elevations.

Because 100-year discharges and water-surface elevations may or may not result from 100-year rainfalls, the water-surface elevations presented in this report may not have a frequency of 100 years. However, as described below, peak discharges computed for small, homogeneous sub-basins of the study area using 100-year rainfalls agreed very closely with 100-year peak discharges computed using flood frequency relations. The term "100-year water-surface elevations" was used in this report rather than the more accurate, but more cumbersome term "water-surface elevations resulting from rainfall durations of 100-year frequency".

Hydrograph Generation

Hydrographs can be generated by several different U.S. Geological Survey rainfall-runoff models, but parameters must be determined by calibration from field-collected data or by estimation from data collected in a similar hydrologic area. However, the data available for Filbin Creek were not sufficient for calibration or for estimation of parameters.

The SCS (Soil Conservation Service) has developed methods for estimating runoff from rainfall, soil classifications, time of concentration of runoff in a basin, and degree of urbanization. Hydrographs are generated by applying rainfall-time data to an SCS dimensionless unit hydrograph. The SCS method is described in detail by McCuen (1982) and was used in this study.

Computer programs that use the SCS runoff curve number, drainage area, time of concentration, total rainfall amount, and duration of rainfall were written to generate hydrographs by the SCS method.

With little or no adjustment of parameters (table 1), peak discharges computed for subbasins without large storage areas by use of the SCS method compared well with peak discharges for South Carolina computed by Whetstone, (1982), and Sauer and others (1983). Rainfall was assumed to be uniformly distributed with respect to time and area.

The SCS method was tested by comparing SCS peak discharges computed using two different methods of rainfall distribution with peak discharges computed using methods developed by Whetstone (1982) and Sauer and others (1983). In the method used by Whetstone (1982), rural peak discharges are estimated from regression models that relate peak flow and drainage area. The rural discharges are subsequently adjusted for effects of urbanization by use of regression models that relate rural peak discharges to various parameters of urbanization (Sauer and others, 1983).

Table 1.--Subbasins, hydrograph numbers, Soil Conservation Service (SCS) unit hydrograph parameters, and comparison of U.S. Geological Survey peak discharges with SCS peak discharges

Subbasin	Hydrograph Number	Drainage Area (mi^2)	Runoff curve number	U.S. Geological Survey TC[1] (hrs)	Used TC[1] (hrs)	U.S. Geological Survey Discharge (ft^3/sec)	Soil Conservation Service Discharge (ft^3/sec)	Percent Difference
Upstream of airport access road	HYD.1	0.68	75	1.15	1.15	666	650	-2.4
Airport access road to I-26, pre-construction	HYD.2	1.67	56	6.50	6.50	543	356	-34.4
Airport access road to I-26, post-construction	HYD.2	1.67	78	4.25	4.25	959	865	-9.8
I-26 to Southern Railroad culvert	HYD.3	0.52	74	1.12	1.12	458	498	+8.7
I-26 to U.S. Route 52 culvert	HYD.4	1.28	74	1.88	1.88	835	960	+14.9
I-26 to Seaboard Railroad culvert	HYD.5	1.68	74	1.58	1.58	1190	1380	+16.0
Southern Railroad culvert to U.S. Route 52 culvert	HYD.4.A	0.76	74	1.10	1.10	677	722	+6.6
Southern Railroad culvert to Seaboard Railroad culvert	HYD.5.A	1.16	74	1.12	1.12	1040	1090	+4.8
Seaboard Railroad culvert to Attaway Avenue	HYD.6	0.41	74	0.30	0.30	665	534	-19.7
Seaboard Railroad culvert to Southern Railroad Bridge	HYD.7	0.98	74	0.53	0.53	1030	1110	+7.8
Seaboard Railroad culvert to Virginia Avenue	HYD.9	3.30	74	1.70	2.00	2080	2410	+15.9

[1] Time of concentration

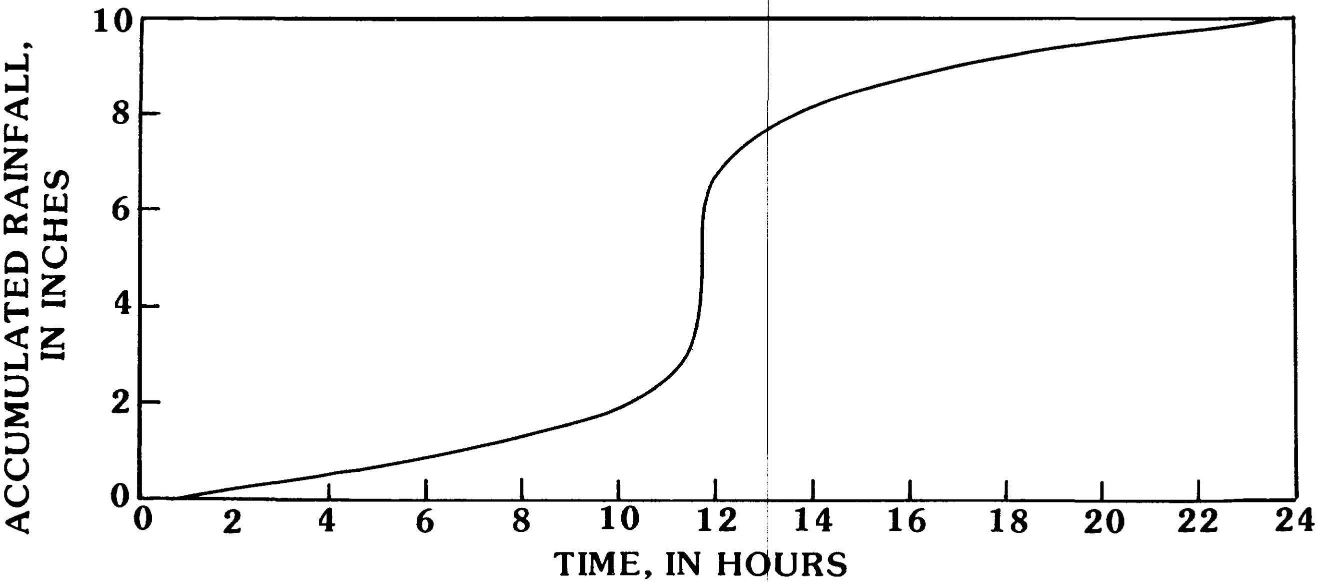
The two methods of distributing rainfall with time for the SCS unit hydrograph method were tested as described below:

1. The SCS Type II 24-hour rainfall storm distribution nests rainfall frequencies within each other (fig. 2a). For example, the 30-minute, 100-year rainfall is assumed to happen within the 60-minute, 100-year rainfall. The probability of all frequencies nesting or occurring at the same time should be very small. The method produced very high discharges in comparison with discharges computed using methodology by Whetstone (1982) and Sauer and others (1983).

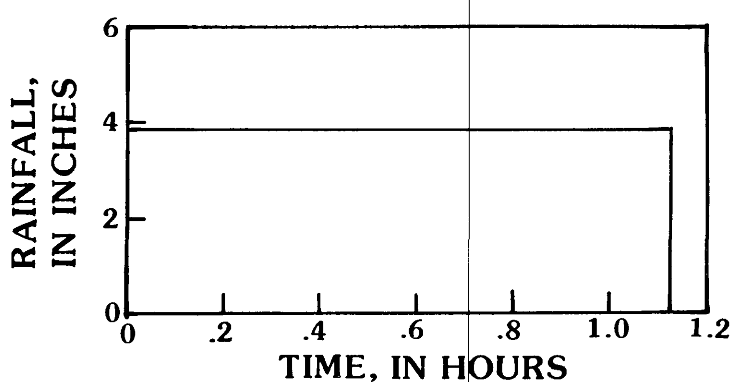
2. The 100-year rainfall for a specific duration was considered to be uniformly distributed with respect to time and area (fig. 2b). The SCS runoff curve number was selected as described by McCuen (1982) and a time of concentration was computed by dividing 0.6 into lag time calculated by methods described by Sauer, and others, (1983). Then a 100-year rainfall, uniformly distributed, with a duration equal to the computed time of concentration was used to compute discharges with the SCS unit hydrograph method. The time of concentration for the hydrograph for the sub-basin Seaboard Railroad culvert to Virginia Avenue was adjusted from 1.70 to 2.00 hours so that the SCS peak discharge would agree more closely with those computed by Whetstone (1982) and Sauer and others (1983). Peak discharges obtained in this manner compared favorably with those computed using Whetstone-Sauer methods (see table 1) and therefore, it was assumed that the SCS method was useable for the study area and for longer durations of uniform rainfall.

Figure 3 represents the sub-basins for which hydrographs were computed and accumulated in the downstream direction. The arrows in figure 3 represent the sub-basins for which hydrographs were computed. For example, the hydrograph identified as HYD.4 represents a hydrograph generated by the SCS method for a sub-basin from I-26 to U.S. Route 52 and includes the outflow from the I-26 reservoir. Subdivision of the Filbin Creek basin was minimized to avoid inaccuracies which might result from accumulating and routing hydrographs from many small sub-basins. For example, the hydrograph for the sub-basin between I-26 and U.S. Route 52 could have been estimated by accumulating the hydrographs from three smaller sub-basins within the larger one, but a single hydrograph computed using the whole sub-basin was considered to be more realistic with regard to shape.

Hydrographs HYD.1 and HYD.2 are both input to the I-26 reservoir for pre- and post-construction conditions. For short duration rainfalls, HYD.3 was also input to the I-26 reservoir, HYD.4.A was used at U.S. Route 52, and HYD.5.A was used at Seaboard Railroad culvert. For long duration rainfalls, HYD.3 was used at Southern Railroad culvert, HYD.4 was used at U.S. Route 52, and HYD.5 was used at Seaboard Railroad culvert. The outflow hydrograph from the I-26 reservoir on the post-construction Southern Railroad reservoir was added to each of these hydrographs. The differential storage at Southern Railroad culvert was added to the storage above I-26 for post-construction conditions and short duration rainfalls.



A. Soil conservation service type II 100 year rainfall distribution for Filbin Creek



B. Uniform rainfall distribution for Filbin Creek for a time of concentration and 100 year rainfall duration of 1.12 hours.

Figure 2--Two methods for distributing rainfall with time for the Soil Conservation Service unit hydrograph method.

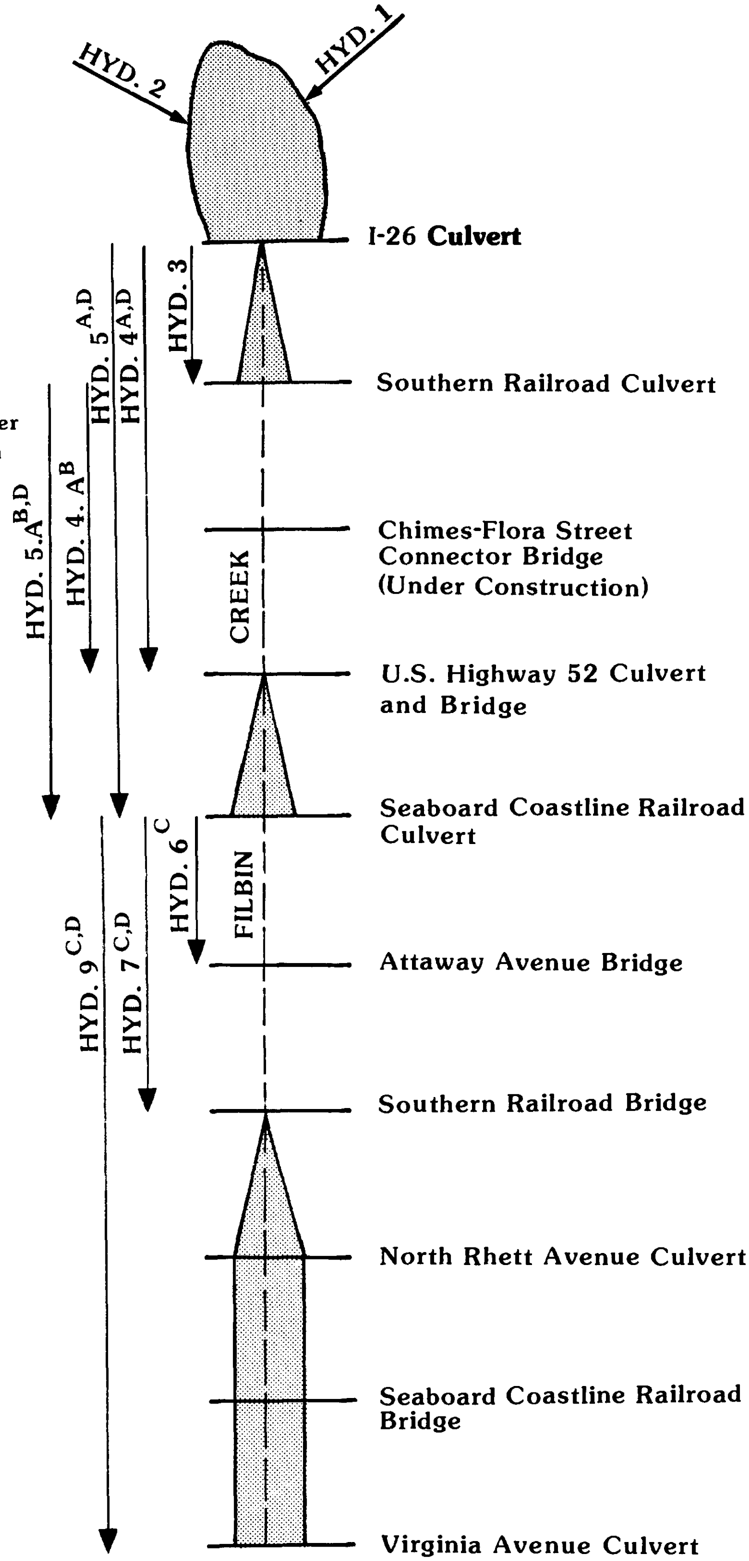
Notes:

A- Outflow from I-26 reservoir was added to hydrograph for pre-construction condition.

B- Post-construction outflow from Southern Railroad reservoir or I-26 reservoir was added to hydrograph

C- Outflow from Seaboard Railroad reservoir was added to hydrograph.

D- Hydrograph was computed using cumulated drainage areas rather than cumulated hydrographs to attain more realistic hydrograph shape.



EXPLANATION

HYD.3 Extent of sub-basin and hydrograph identification for hydrographs computed by the Soil Conservation Service method

Reservoir

Figure 3.--Schematic representation of subbasins for which hydrographs were computed for Filbin Creek, North Charleston, South Carolina.

Hydrograph HYD.6 was used at Attaway Avenue, HYD.7 was used at Southern Railroad Bridge, and HYD.9 was used at Virginia Avenue. The outflow hydrograph from Seaboard Railroad culvert reservoir was added to each of these three hydrographs.

SCS hydrograph parameters of drainage area, runoff curve number, and time of concentration are shown in table 1. Runoff curve numbers were determined assuming wet antecedent conditions from SCS tables described by McCuen (1982) and soil types identified by Davis and Floyd, Inc., (1980). Times of concentration were estimated by dividing 0.6 into lag times computed by methods developed by Sauer and others (1983). Drainage area and imperviousness estimates were determined from 7.5-minute topographic maps (U.S. Geological Survey, 1958, 1971, and 1979). Rainfall duration and amounts determined by Hershfield (1961) and Miller (1964) that were used to create hydrographs are shown in table 2.

Table 2.--Duration and amount of rainfall used to generate inflow hydrographs by Soil Conservation Service unit hydrograph method

Duration of rainfall (hours)	Amount of rainfall[1] (inches)
2	5.2
6	7.0
12	8.5
24	10.0
48	12.2

[1] Data from Hershfield (1961) and Miller (1964).

Flow Routing

No single hydraulic model was found that would accurately define flood profiles for Filbin Creek. Several U.S. Geological Survey two-dimensional unsteady-flow models exist, but these models currently do not route flow through multiple culverts with embankment overflow. U.S. Geological Survey programs exist to accurately compute steady-state flow through culverts, bridges, and channels, and to route flow through resevoirs, but most are designed to work in stand-alone mode only. A program, "Interconnected Pond Routing Program" by Advanced Engineering Technologies Inc.,[2] of Orlando, Florida, will compute water-surface elevations for interconnected ponds, but lacks accurate open-channel, bridge, and culvert flow routing methodology.

[2] Use of the brand/trade names in this report is for identification purposes only and does not constitute endorsement by the U.S. Geological Survey.

Attempts to model Filbin Creek one time increment at a time through the whole basin using step-backwater programs, culvert programs, and an adaptation of the Puls reservoir routing method (Jennings, 1977) proved unsuccessful.

Filbin Creek was hydraulically and hydrologically modeled by simplifying the modeling concept and by using existing programs in conjunction with specially developed Fortran 77 and Command Procedure Language (CPL) programs. These programs all process an entire hydrograph for a subreach. CPL programs, Fortran 77 programs, and operating commands residing on the U.S. Geological Survey's PRIME computer in Columbia, South Carolina, may be linked together using CPL programs. The modeling concept was simplified by minimizing the number of reservoirs in the model, based on high-water data.

Filbin Creek was conceptually modeled using the following sequence of steps:

1. Tributary inflow hydrographs resulting from 100-year rainfall durations of 2, 6, 12, 24, and 48 hours were computed using the SCS method. The hydrographs serve as input to Geological Survey methods as described below in detail.

2. Hydrographs for each rainfall duration were routed through storage areas upstream of constrictions using the U.S. Geological Survey program A697, which uses the modified Puls reservoir routing method described by Jennings (1977). The program generates an outflow hydrograph from an inflow hydrograph, a stage-storage relation, and a stage-outflow relation. Stage-storage ratings were computed from 1- and 2-foot contour interval topographic maps prepared by Davis and Floyd (1980) and the SCDHPT.

3. Tributary inflow hydrographs and outflow hydrographs from the storage areas were accumulated in the downstream direction to produce hydrographs upstream and downstream of all road crossings.

4. Discharges at selected time intervals from the accumulated hydrographs were routed upstream by the U.S. Geological Survey step-backwater program E431 (Shearman, 1976) and specially developed Fortran 77 programs which perform rating conversions using three-parameter ratings (headwater, tailwater, and discharge) developed by program E431 and U.S. Geological Survey culvert flow program A526 (Mathai and others, written commun.). The upstream routing produced profiles of water surface elevations at the selected time intervals for the associated discharges. Cross-sections, bridge geometry, and culvert geometry were determined from SCDHPT highway plans, field survey, and 1-foot and 2-foot contour maps prepared by Davis and Floyd (1980) and the SCDHPT.

5. On the first trial in step 2, stage-outflow ratings were estimated for each storage area and then maximum pool elevations were computed using the modified Puls method. Pool elevations were also produced

by the upstream routing in step 4. If the two sets of pool elevations differed by more than about 0.10 feet, new stage-outflow ratings were prepared using water-surface elevations computed in step 4, and steps 2-5 were repeated. If the two sets of pool elevations agreed within about 0.10 feet, the profile established in step 4 was accepted as the profile of a flood resulting from the particular rainfall duration used to generate the hydrographs being routed.

COMPUTATIONS OF FLOOD FLOW PROFILES UNDER PRE- AND POST-CONSTRUCTION CONDITIONS

Nonsurge Storm Profile

Profiles were computed for storm runoff without the effect of hurricane storm surges (fig. 4). Filbin Creek is hydrologically complex because of its interspersed large storage areas and urbanization. For example, maximum peak flow upstream from Seaboard Railroad culvert could result either from runoff from nearby highly urbanized areas after rainfall of short duration and high intensity, or from outflow from the large storage area upstream from I-26 after rainfall of long duration and high volume. The effects of interspersed large storage areas and urbanization on peak flow and hydrograph shape can be determined by using rainfalls of varying durations in the analytical methods described. Flood profiles resulting from the 100-year rainfall of 2-, 6-, 12-, 24-, and 48-hour durations were computed. The highest elevations of the computed profiles were used. Profiles shown in figure 4 are a composite of the highest water-surface elevations computed using these five durations of rainfall. A flood profile for runoff from a 2-hour duration rainfall for the whole stream would be much lower than the profile in figure 4 upstream from I-26 and Virginia Avenue. Rainfall durations and amounts are shown in table 2.

Profiles for each rainfall duration were computed using the iterative trial and error process described in the section "Methods of Analysis."

High-water mark data were used to evaluate computed profiles. The data and evaluations are described below in detail.

High-water marks were obtained on April 9, 1986 by the author and L.R. Bohman at Attaway Avenue, downstream of Seaboard Railroad culvert, and under the downstream bridge of U.S. Highway 52. High-water marks were also obtained from citizens downstream of Southern Railroad culvert and in the Russelldale subdivision area. Elevations of high-water marks are shown in figure 4. These data were used to evaluate computed profiles.

High-water marks at Attaway Avenue and U.S. Highway 52 were faint stain lines on wooden piles under the bridges. The mark downstream of Seaboard Railroad culvert was pointed out by Mr. Brisbon of 1832 Wasp Drive. He said the flood occurred in 1962 or 1963 and that it was the highest he had seen in 23 years. In 24 hours, the Weather Bureau airport rain gage (U.S. Weather Bureau, 1962 and 1963) measured 4.6 inches on August 3, 1962 and 5.2 inches on June 23, 1963. If these rainfall amounts fell in 6 hours, the return period of these events would be about 16 years, which indicates that this flood was not exceptionally rare.

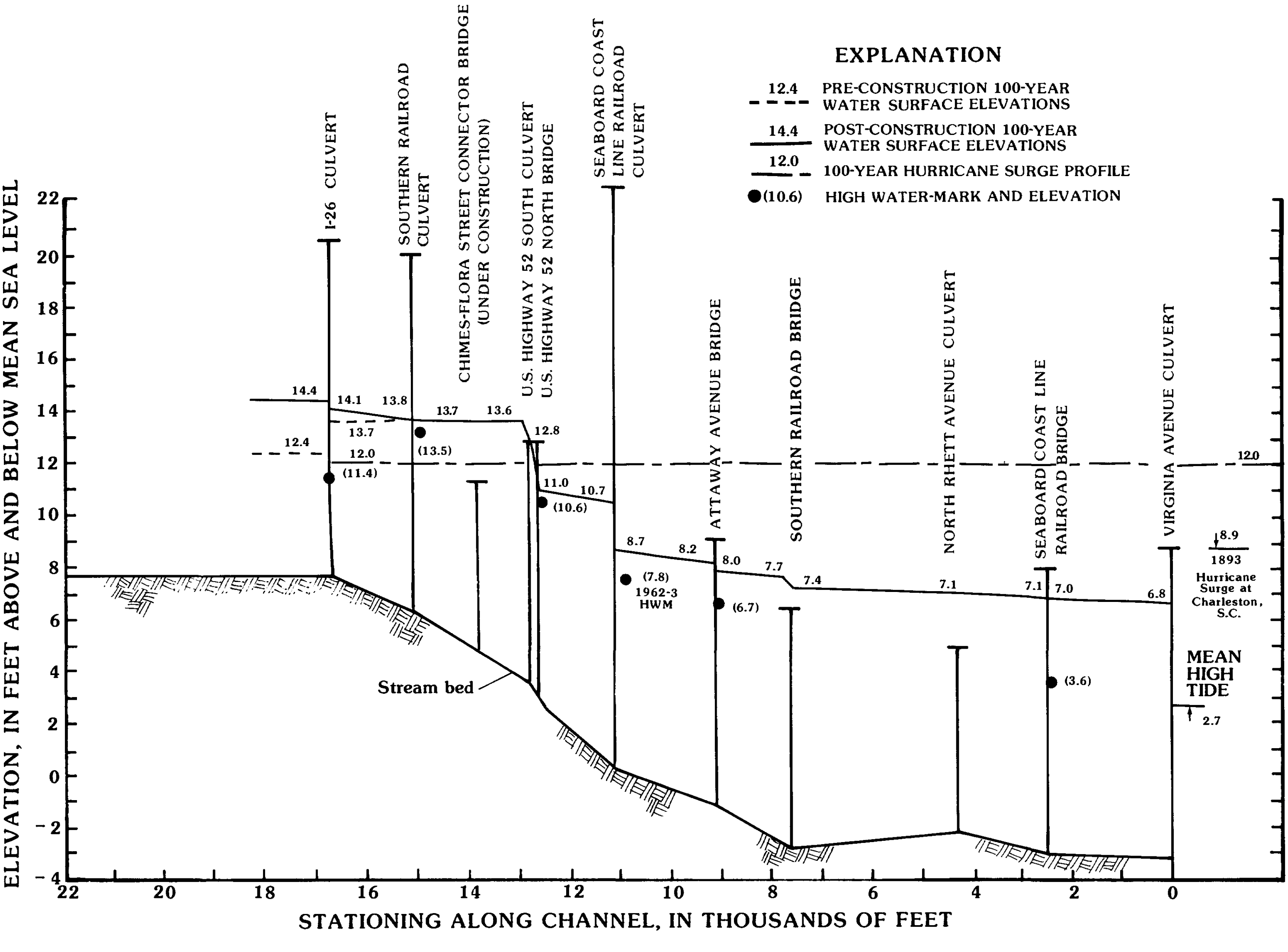


Figure 4.--High-water marks and 100-year water-surface elevations for pre- and post-construction conditions for Filbin Creek, North Charleston, South Carolina.

On April 9, 1986, the author observed a fresh high-water mark inside the I-26 culvert at about 11.4 feet, and a water surface of about 8.9 feet. The high-water mark could have resulted from construction debris downstream.

The mean high tide elevation of 2.7 feet downstream of Virginia Avenue was used as the starting elevation of all upstream routing from Cooper River. Mean high tide was arbitrarily selected as the starting elevation at the downstream boundary. Any change in the starting elevation will significantly affect computed water surface elevations downstream of the Seaboard Coastline Railroad culvert.

Storage areas were modeled in the following reaches of Filbin Creek:

1. Virginia Avenue to Southern Railroad bridge:

   Storage areas downstream of the Southern Railroad bridge were combined, with Virginia Avenue as the control. This consolidation was valid, because the final profile shows very little fall over the reach. An attempt to adjust for storage upstream from North Rhett Avenue showed storage effects until flow went over the road, after which outflow equaled inflow. Storage was determined directly from topograhic maps.

2. Seaboard Railroad Culvert to U.S. Highway 52 culvert:

   Above 8.5 feet, the Seaboard Railroad culvert starts causing enough backwater to affect outflow discharges. The lower end of the stage-storage rating for the area affected by the Seaboard Coastline Railroad culvert was begun at 8.5 feet to adjust for the effect of storage.

3. U.S. Highway 52 to I-26:

   When storage corrections were made for pre-construction conditions between I-26 and U.S. Highway 52, preliminary computations resulted in 100-year water surface elevations lower than observed high-water marks, all of which were probably the highest in the 23-year period. For this reason, storage adjustments were not made between I-26 and U.S. Highway 52 for pre-construction conditions.

   The pre-construction profile of 100-year water-surface elevations for this reach shows only about 2 feet of backwater; therefore, storage upstream of U.S. Highway 52 is predominantly channel storage. Because U.S. Geological Survey regression equations and SCS hydrographs already have channel storage built into them, it would be improper, as indicated by the high-water marks, to make additional adjustments for predominantly natural channel storage by the Puls method.

   Adjustment was made for post-construction conditions upstream from the Southern Railroad culvert by using only the difference in storage before and after excavation, to avoid over-adjustment for storage. For short-duration, low volume 100-year rainfalls where flow was upstream through the I-26 culvert, post-construction storages upstream of Southern Railroad culvert were lumped together with storage upstream of I-26, as described in the following section.

4. Upstream of I-26:

From figure 1 and as described below, it appears that the post-construction storage system upstream of I-26 is complex. Four sets of culverts exist through I-526 and four sets of culverts exist under the airport access road. These culverts are not shown in figure 1. The storage area between the airport access road and I-526 is long and rough as a result of the phosphate mining.

For the commercial development, total storage, general shape, selected cross-sections, and the proposed outflow structure to restrict runoff are being designed for the 10-year storm. A minimum parking lot elevation of 12 feet and a general elevation of about 13 feet are planned for the development, according to plans of July 1986. (A fresh high water mark, however, of 11.4 feet was observed by the U.S. Geological Survey inside the I-26 culvert on April 9, 1986). Plans of July 1986 include zero acre-feet of storage at 7 feet elevation, 200 acre-feet at 12 feet, and 250 acre-feet at 13 feet. These figures were extrapolated to 400 acre-feet at 16 feet. No information was available on culvert sizes.

As shown in figure 1, the Centre Pointe storage areas lie in a half circle around the western and southern parts of Centre Pointe. A two-dimensional, unsteady flow model that could handle general culvert flow is needed to route hydrographs through this complex system, but such a model could not be found. Therefore, the system had to be greatly simplified as described below.

The final analysis showed that the highest pool elevations upstream from I-26 for post-construction conditions would result from a 12-hour duration rainfall. Total openings under I-26, I-526, and the airport access road are 72, 140, and 28 square feet, respectively. Because of the exceptionally broad and flat outflow hydrograph resulting from the 12-hour rain, and because several pipe culverts through I-526 would act as equalizers, the whole system would be ponded for all practical purposes. Therefore, storages for the commercial development, from I-526 to the airport access road, and upstream of the airport access road were added together, and treated as one reservoir, with the control at I-26 for rainfall durations equal to and greater than 12 hours.

For both pre- and post-construction conditions, however, runoff caused by short-duration, low-volume, 100-year rainfall would flow upstream through the I-26 culvert because runoff in the upper part of the basin would not be enough to satisfy storage upstream of I-26 and raise pool elevations enough to cause downstream flow.

Profiles in figure 4 are composites of the maximum elevations of several flood profiles generated from rainfalls of different durations. Profiles of floods caused by short-duration storms resemble the profile of pre-construction conditions in figure 4. Upstream flow through the I-26 culvert requires some sort of "divide" between I-26 and U.S. Highway 52. This "divide" was assumed to exist at the Southern Railroad culvert because it is approximately midway in location, culvert opening size, and intervening drainage area. Openings for the I-26, Southern Railroad, and U.S. Highway 52 culverts are 72, 71, and 76 square feet, respectively. The drainage areas from I-26 to the Southern Railroad culvert and Southern Railroad culvert to U.S. Highway 52 are about 0.52 and 0.76 square miles respectively.

Therefore, for rainfall durations shorter than 12 and 24 hours for post- and pre-construction conditions, respectively, the control for the storage areas was assumed to be at Southern Railroad culvert, rather than at I-26. The inflow from I-26 to Southern Railroad was added to the reservoir, and for post-construction conditions, the differential post-construction storage downstream of I-26 was added to the reservoir storage.

Outflow from the I-26 reservoir was for all practical purposes prevented by means of the stage-outflow discharge rating at Southern Railroad. Thus, the model generated a stage hydrograph for the I-26 reservoir for a no-outflow case and hydrographs and profiles downstream of Southern Railroad culvert as if a divide existed at the culvert.

For the shorter duration rainfalls, elevations generated in the I-26 reservoir were lower than elevations generated at Southern Railroad culvert, even with hydrographs HYD.1, HYD.2, and HYD.3 flowing into the I-26 reservoir. HYD.3 was made to flow upstream into the I-26 reservoir for short duration rainfalls.

The validity of the "flow divide" assumed to exist at Southern Railroad culvert was tested by routing flows from the I-26 reservoir toward the Southern Railroad culvert and also from the Cooper River toward the Southern Railroad culvert. Water-surface elevations between I-26 and Southern Railroad culvert computed by these two routings agreed within about 0.4 feet. Therefore, because the computed water surface in the vicinity of the "divide" was reasonably level by routing from opposite directions, the assumption of a "flow divide" in this area was judged valid.

For rainfall durations greater than 12 and 6 hours for pre- and post-construction conditions, respectively, the control was assumed to be at the I-26 culvert, and all flow would be downstream. For the profile of 100-year water-surface elevations, about 2.0, 2.6, and 4.1 feet of fall will exist across the Seaboard Coastline Railroad culvert, the structure at U.S. Highway 52, and the Virginia Avenue culvert, respectively.

Hurricane Storm-Surge Profiles

A Federal Emergency Management Agency (FEMA) report (1986) indicated that hurricane storm surge elevations for Filbin Creek from Virginia Avenue to I-26 would be 9.6, 10.9, 12.0, and 12.9 feet respectively for the 10-, 50-, 100-, and 500-year storm surges. The water-surface profile of the 100-year storm surge is shown in figure 4.

To determine these elevations, FEMA generated many theoretical storms using combinations of five parameters (central pressure depression, radius to maximum winds, forward speed of the storm, shoreline crossing point, and crossing angle). These storms were input to the FEMA storm surge model, which computed surge elevations for each of them. The FEMA storm surge model takes into account effects of bottom configuration, tide, wind setup and wave actions.

Each of the five parameters that define the storms is associated with frequency of occurrence, from which the joint probability of each storm can be computed. The joint probabilities of all the modeled storms are accummulated to define the final stage-frequency relation.

FEMA reports that the highest recorded storm surges at Charleston, S.C. to be 8.9, 7.5, and 8.0 feet respectively on the 1893, 1911, and 1940 hurricanes. It should be noted that the maximum recorded storm surge in 92 years is 8.9 feet at Charleston and that the elevation of road overflow at Virginia Avenue, where tidal flapper gates are located is also at about 8.9 feet. Virginia Avenue crosses Filbin Creek about 0.4 miles from its junction with the Cooper River, which flows about seven miles to the shorefront of Charleston, S.C.

Combined Profiles

The scope of this study did not include the statistical combination of profiles from both storm runoff and hurricane storm surges.

If the occurrences of storm runoff can be considered independent of rainfall from hurricanes, the higher of the two profiles that result from storm runoff and hurricane surge in figure 4 would probably be close to results that would be obtained by rigorous adding of joint probabilities of storm runoff profiles and hurricane storm surge profiles.

If the two occurrences were completely dependent, such that the 100-year storm runoff always occurs at the peak of the 100-year hurricane storm surge, the higher of the two profiles upstream from Seaboard Railroad culvert may be too low, because of backwater from the storm surge below the railroad. If the storm runoff is partially dependent on rainfall from hurricanes, an accurate combined profile would be very difficult to determine.

SUMMARY

Profiles of 100-year water-surface elevations for pre-construction and post-construction conditions, according to July 1986 plans, for Filbin Creek in North Charleston, South Carolina were computed using SCS hydrograph methodology and U.S. Geological Survey programs for routing flow through open channels, bridges, culverts, and reservoirs. Adjustments were made for storage upstream of I-26, Southern Railroad culvert, Seaboard Railroad culvert, and Virginia Avenue.

Hydrographs were generated using SCS methods for the 100-year rainfall for 2-, 6-, 12-, 24-, and 48-hour rainfall durations.

These hydrographs were routed downstream by addition and by routing through reservoirs using the Puls method. Flow was routed upstream using U.S. Geological Survey step-backwater and culvert flow programs to compute profiles and refine stage-outflow relations used in the reservoir routing programs. Upstream and downstream routing was iterated until reservoir pool elevations changed less than about 0.10 foot.

Highest elevations downstream of Southern Railroad bridge were caused by the 12-hour duration rainfall. Between Southern Railroad bridge and I-26, the highest elevations were caused by 2-hour rainfall durations. Above I-26, highest elevations were caused by 24- and 12-hour durations respectively for pre- and post-construction conditions.

The highest elevations computed from the various durations of rainfall were used for the final profiles of 100-year water-surface elevations, which looked reasonable when compared to reliable high-water mark data.

The proposed commercial construction, according to July 1986 plans, will raise the 100-year water-surface elevations upstream of I-26 by about two feet, from 12.4 to 14.4 feet. Seaboard Railroad culvert, the U.S. Highway 52 structures, and Virginia Avenue culvert will cause about 2.0, 2.6, and 4.1 feet of fall across the structures, respectively, for floods caused by storm runoff without effect of hurricane surge. A previous study by the Federal Emergency Management Agency indicated that a water-surface elevation of 12.0 feet will occur between Virginia Avenue and I-26 during a 100-year hurricane storm surge.

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