FLOODFLOW CHARACTERISTICS OF FILBIN CREEK

AT PROPOSED INTERSTATE HIGHWAY 526,

NORTH CHARLESTON, SOUTH CAROLINA

by Larry R. Bohman

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CONVERSION FACTORS AND ABBREVIATIONS OF UNITS

The following factors may be used to convert the inch-pound units published herein to the International System of units (SI).

Multiply inch-pound units	By	To obtain SI units
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)

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ABSTRACT

A study to determine the impact of two alternative construction plans for proposed interchange between the existing Interstate Highway 26 and Interstate Highway 526 in the Filbin Creek drainage basin near North Charleston, South Carolina was performed by the U.S. Geological Survey, in cooperation with the South Carolina Department of Highways and Public Transportation. A computerized reservoir routing technique was used to route synthetic flood hydrographs through the basin system. Simulation results indicate that the new roadway will cause little or no change in water-surface elevations downstream of Interstate Highway 26. Upstream of Interstate Highway 26, approximately 0.5 foot of backwater will be created by either alternative during a 100-year flood as a result of the Interstate Highway 526 embankments and structures.

INTRODUCTION

The SCDH&PT (South Carolina Department of Highways and Public Transportation) is planning the construction of I-526 (Interstate Highway 526) in the vicinity of North Charleston, South Carolina. The new highway is being built to carry traffic associated with a new airport terminal currently being built. The study area, shown in figure 1, is the Filbin Creek basin from Virginia Avenue to its headwaters upstream of I-26. The proposed interstate construction, shown as a dashed line in figure 1, generally follows the flood plain of Filbin Creek.

The purpose of this study is to determine the impact of two I-526 design alternatives on the water-surface profiles of Filbin Creek for floods with recurrence intervals of 25, 50, and 100 years. Recurrence interval is the average time interval within which a given flood will be equaled or exceeded once. Water-surface profiles and peak discharges are provided for the following conditions:

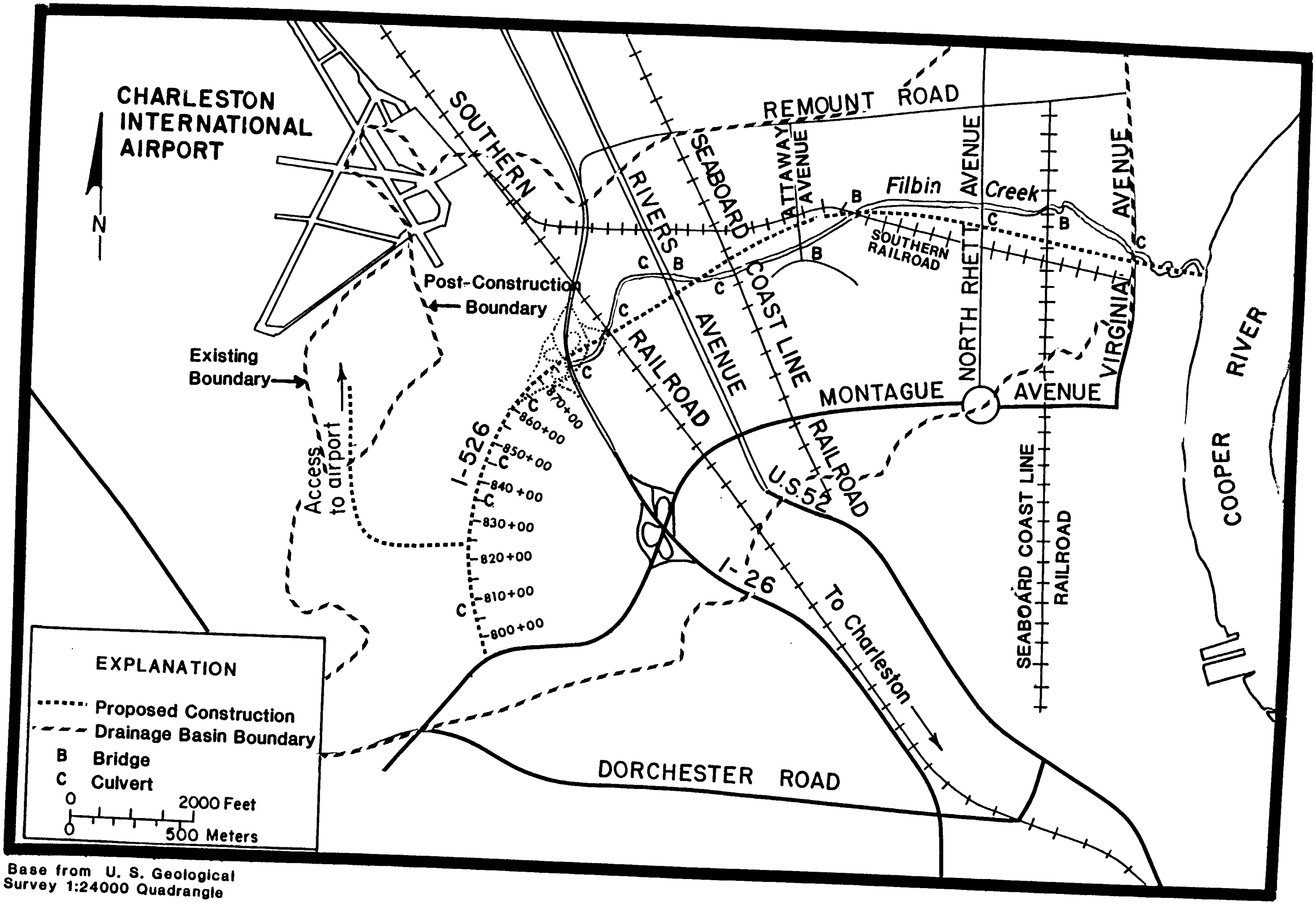

Figure 1.--Filbin Creek basin at North Charleston, South Carolina.

1. Existing conditions as of April 1983 (fig. 2).

2. Construction alternative A (all embankment upstream of I-26). This plan would lengthen the existing culvert under I-26 from 175 feet to 675 feet with a 30° bend at the existing entrance. The entire I-26/I-526 interchange upstream of I-26 would be constructed on embankment (fig. 3). Five RCP (reinforced concrete pipe) culverts will be located under I-526.

3. Construction alternative B (embankment upstream of I-26 with I-26/I-526 interchange elevated on structure). This plan would lengthen the existing culvert under I-26 from 175 feet to 235 feet with a 30° bend at the existing entrance. The I-26/I-526 interchange upstream of I-26 would be elevated above natural ground on piers or piles, except for a part of the most extreme northwest ramp (fig. 4). Five RCP culverts will be located under I-526 at the same sites as in alternative A.

The entire length of the I-526 roadway between I-26 and the Cooper River will be an elevated structure, and there are no plans to improve the existing drainage system.

The analyses performed herein do not consider the uncertain effects of hurricane storm surges. A study of this type was performed for the city of North Charleston by the Federal Emergency Management Agency (1983).

All data and computations supporting the conclusions in this report are available in the files of the District office of the U.S. Geological Survey in Columbia, South Carolina.

GENERAL 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 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 under I-26. Downstream of I-26, Filbin Creek flows eastward for approximately 3.6 miles to the Cooper River and passes through several 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).

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

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

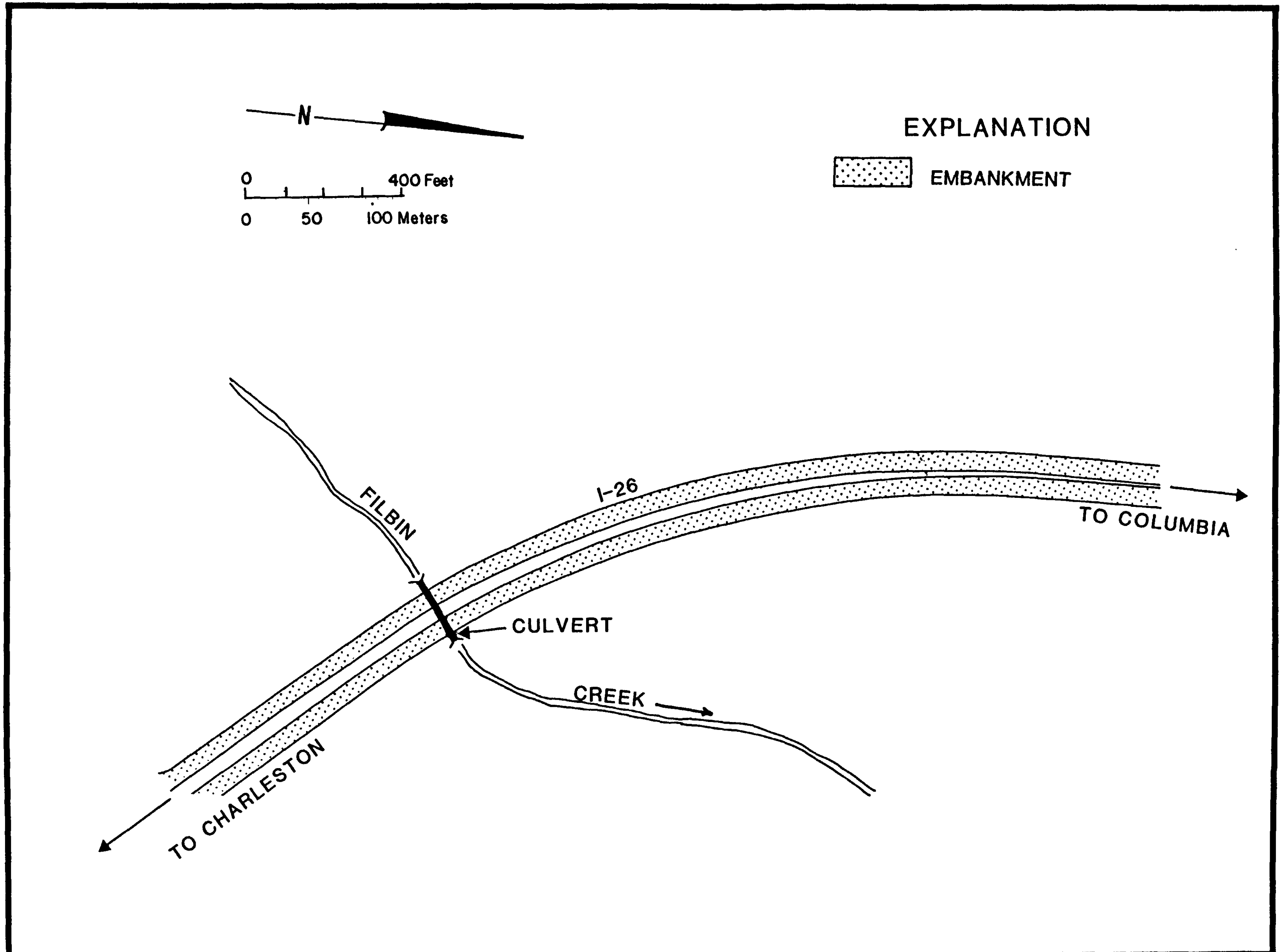

Figure 2.--Filbin Creek at proposed I-26/I-526 interchange, existing conditions.

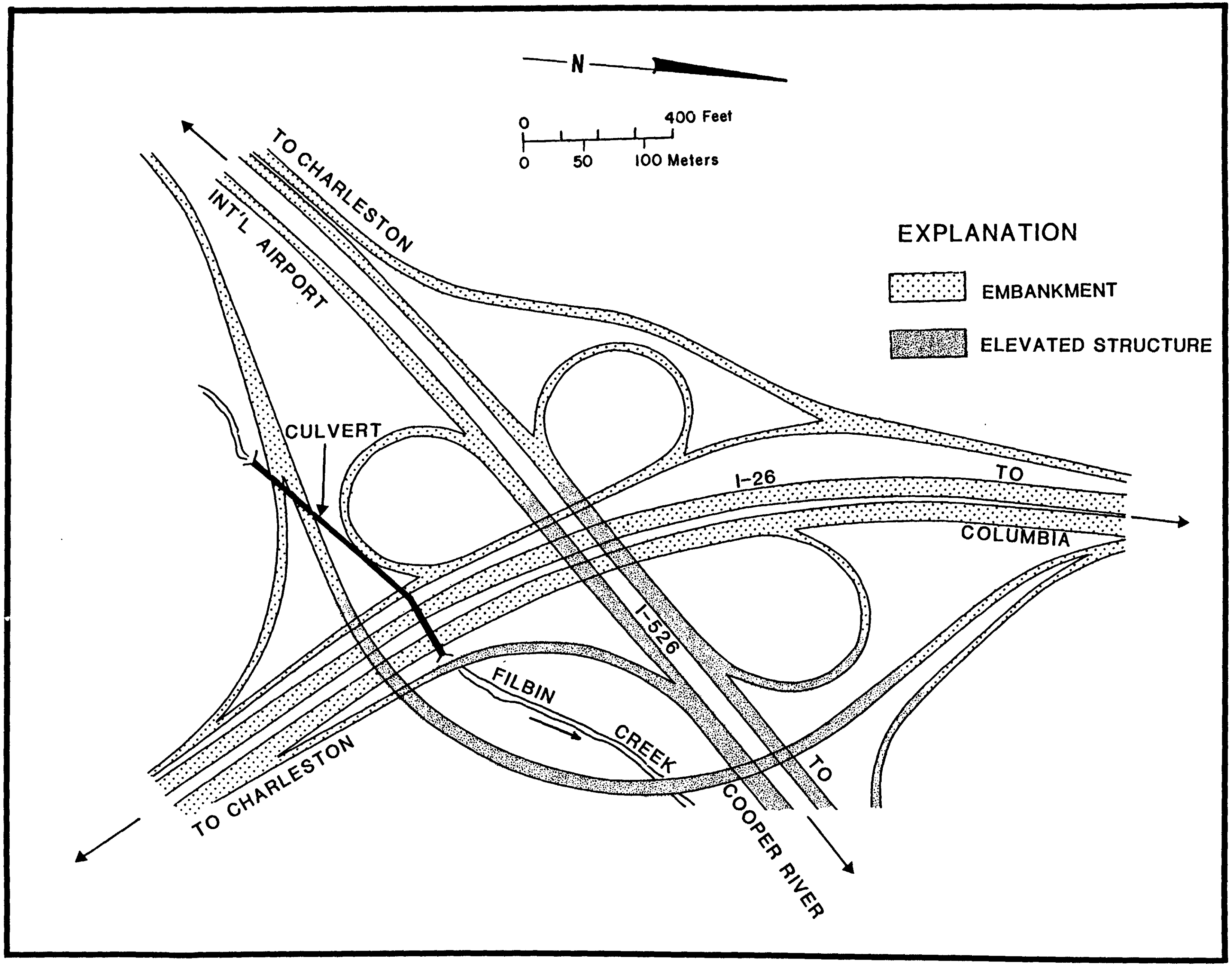

Figure 3.--Filbin Creek at proposed I-26/I-526 interchange, construction alternative A.

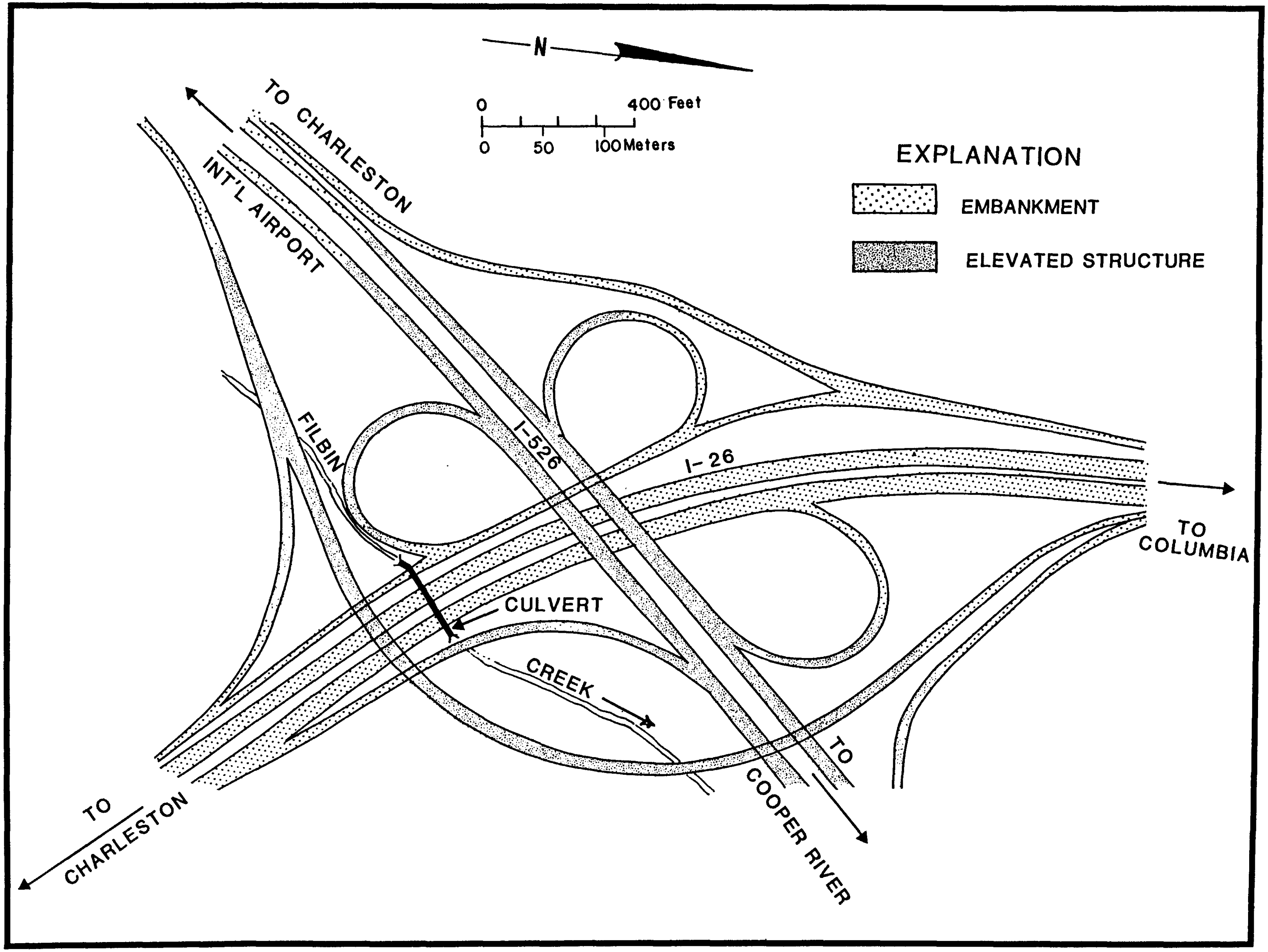

Figure 4.--Filbin Creek at proposed I-26/I-526 interchange, construction alternative B.

The Filbin Creek channel has been dredged and straightened in some places and occupies a relatively broad, flat flood plain. The entire flood plain 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 urbanized, and there are several areas of residential encroachment onto the flood plain.

MODEL DESCRIPTION

The Filbin Creek basin is a dynamic system. It contains numerous sources of variable backwater and areas of significant retention storage. Therefore, a steady-state step-backwater model was not considered to be appropriate for developing water-surface profiles in the basin. The application of a detailed urban watershed model for routing storm runoff was considered, but could not be used because of the paucity of concurrent rainfall and runoff data.

The Filbin Creek system is essentially a series of uncontrolled reservoirs which respond instantaneously to inflows and outflows without manmade interference such as opening or closing of gates. The "reservoir" outflow points in the Filbin Creek basin are culverts for which theoretical stage-discharge relations can be determined. If the reasonable assumption of a level reservoir water surface can be made, then stage-storage relations can also be determined. With these concepts in mind, it is apparent that the modified Puls Method (Soil Conservation Service, 1972) of reservoir routing could be used in this analysis. Therefore, a computer program developed by Jennings (written commun., 1977) which uses the modified Puls method to route inflow hydrographs through an uncontrolled reservoir to obtain an outflow hydrograph was selected to model Filbin Creek.

The modified Puls method solves the continuity equation for the reservoir in the following form:

$$\bar{I} + [(S_1/DT) + (O_1/2)] - O_1 = [(S_2/DT) + (O_2/2)]$$

where: $\bar{I} = (I_1 + I_2)/2$,

S_1, S_2 = storage at times 1 and 2,

DT = a constant computation time interval at which inflow rates are tabulated,

O_1, O_2 = outflow rates at times 1 and 2, and

I_1, I_2 = inflow rates at times 1 and 2.

AVAILABILITY OF INPUT DATA

The input information required for the reservoir routing model is:

1. Inflow hydrographs for selected recurrence intervals,
2. Stage-discharge relations at proposed and existing constrictions, and
3. Stage-storage relations at proposed and existing constrictions.

Continuous water-stage data for short periods of time and a few water-discharge measurements have been obtained at several locations within the Filbin Creek basin. However, due to increases in urbanization, bridge and culvert installations or improvements, and channel dredging, the available stage and discharge data cannot be used to calibrate a model of the existing system, nor can the data be compared to the simulation results.

Flood Hydrographs

Synthetic hydrographs of runoff were developed from a dimensionless hydrograph described by Stricker and Sauer (1982) for each subbasin (that is, the intervening drainage area between structures). Subbasin hydrographs are referred to as intervening area inflow hydrographs in this report. Urban peak discharges and lag time for each subbasin were determined using techniques described by Sauer and others (1983). The following is an example of the urban peak discharge computations and lag time for the intervening area between Seaboard Coast Line Railroad and North Rhett Avenue depicting data type requirements and the regression equations used:

Filbin Creek at North Rhett Avenue, North Charleston, S.C., (intervening area between Seaboard Coast Line Railroad and North Rhett Avenue)

Drainage area, DA = 2.03 square miles
Channel slope, SL = 15.1 feet per mile
2-hour 2-year rainfall intensity, RI2 = 2.6 inches
Basin storage, ST = 12 percent
Basin development factor, BDF = 12
Impervious area, IA = 18.7 percent
Rural peak discharge, $RQ_{25} = 358$
$RQ_{50} = 458$
$RQ_{100} = 565$
Channel length, L = 2.03 miles

Urban peak discharge:

$$UQ_{25} = 2.78A^{0.31}SL^{0.15}(RI2+3)^{1.76}(ST+8)^{-0.55}(13-BDF)^{-0.29}IA^{0.07}RQ_{25}{}^{0.60}$$
$$= 857\ ft^3/s$$

$$UQ_{50} = 2.67A^{0.29}SL^{0.15}(RI2+3)^{1.74}(ST+8)^{-0.53}(13-BDF)^{-0.28}IA^{0.06}RQ_{50}{}^{0.62}$$
$$= 1{,}050\ ft^3/s$$

$$UQ_{100} = 2.50A^{0.29}SL^{0.15}(RI2+3)^{1.76}(ST+8)^{-0.52}(13-BDF)^{-0.28}IA^{0.06}RQ_{100}{}^{0.63}$$
$$= 1{,}290\ ft^3/s$$

Basin lag time, $LT = 0.85(L/SL^{0.50})^{0.62}(13-BDF)^{0.47} = 0.57$ hours

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The recurrence interval peak discharge and basin lag time are multiplied by the discharge and time ratios of the dimensionless hydrograph to produce the subbasin hydrograph as shown on table 1. All hydrographs were extrapolated to zero flow on the rising limb.

Drainage areas were delineated and planimetered on 7-1/2 minute quadrangles using a master drainage plan (Davis & Floyd, Inc., 1980, "City of North Charleston, Drainage Systems and Recommended Improvements": Unpublished data on file in Columbia, S.C., District office of U.S. Geological Survey) to aid in the identification of actual storm sewer boundaries. Upstream of I-26 the watershed boundaries approximately coincide with those used in the airport drainage design report (Howard Needles Tammen & Bergendoff, 1980, "Design Considerations Relating to Stormwater Drainage System for the Proposed Passenger Terminal Complex, Charleston International Airport, South Carolina": Unpublished data on file in Columbia, S.C., District office of U.S. Geological Survey). Channel slope and channel length were determined using 2-foot orthophoto contour maps (Davis & Floyd, Inc., 1980, unpublished data). The rainfall intensity was taken from the rainfall frequency atlas (Hershfield, 1961). The basin storage, basin development factor, and impervious area were estimated using definitions and methods found in the report by Stricker and

Table 1.--100-year flood hydrograph for the Filbin Creek subbasin between North Rhett Avenue and Seaboard Coast Line Railroad, North Charleston, South Carolina

t/LT	X	LT (hours)	=	time (hours)	Qt/Qp	X	Qp (ft^3/s)	=	Discharge (ft^3/s)
0.45		0.57		0.26	0.27		1,290		348
0.50		0.57		0.28	0.37		1,290		477
0.55		0.57		0.31	0.46		1,290		593
0.60		0.57		0.34	0.56		1,290		722
0.65		0.57		0.37	0.67		1,290		864
0.70		0.57		0.40	0.76		1,290		980
0.75		0.57		0.43	0.86		1,290		1,110
0.80		0.57		0.46	0.92		1,290		1,190
0.85		0.57		0.48	0.97		1,290		1,250
0.90		0.57		0.51	1.00		1,290		1,290
0.95		0.57		0.54	1.00		1,290		1,290
1.00		0.57		0.57	0.98		1,290		1,260
1.05		0.57		0.60	0.95		1,290		1,230
1.10		0.57		0.63	0.90		1,290		1,160
1.15		0.57		0.66	0.84		1,290		1,080
1.20		0.57		0.68	0.78		1,290		1,010
1.25		0.57		0.71	0.71		1,290		915
1.30		0.57		0.74	0.65		1,290		838
1.35		0.57		0.77	0.59		1,290		761
1.40		0.57		0.80	0.54		1,290		696
1.45		0.57		0.83	0.48		1,290		619
1.50		0.57		0.86	0.44		1,290		567
1.55		0.57		0.88	0.39		1,290		503
1.60		0.57		0.91	0.36		1,290		464
1.65		0.57		0.94	0.32		1,290		412
1.70		0.57		0.97	0.30		1,290		387

Sauer (1982) and using 7-1/2 minute topographic maps. Rural peak discharges with recurrence intervals of 25, 50, and 100 years were determined using methods described by Whetstone (1982).

Stage-Discharge Relations

The mean high tide elevation, 2.7 feet, was arbitrarily selected as the initial water-surface elevation upstream of Virginia Avenue and the steady-state water-surface elevation downstream of Virginia Avenue. A stage-discharge rating was developed at Virginia Avenue using techniques described by Bodhaine (1969) and adjusting for head loss through the flapper valves.

The computed Virginia Avenue headwater elevations and corresponding discharges were used as the tailwater elevations at North Rhett Avenue. The U.S. Geological Survey program for automatic computation of stage-discharge relations at culverts was used to calculate a discharge rating at North Rhett Avenue. The resultant North Rhett Avenue headwater elevations were used as the tailwater elevations in the Seaboard Coast Line Railroad rating computations. This rating development procedure was repeated for each culvert in an upstream direction. This approach to initial stage-discharge rating development for a structure is appropriate because ponded conditions exist upstream of each culvert for all but extremely low discharges. Estimating a variable stage-discharge relation in this manner is approximate and field verification of such estimates is normally desirable. However, the results for noncomplex structures such as those in this study may be considered fairly reliable even without field verification.

The dimensions and elevations of existing structures in the basin were measured during field inspection. Proposed structure dimensions were taken from construction plans supplied by the SCDH&PT. Dimensions used in the model are shown in table 2. All culverts were assumed to be clean and the entrances free of debris for the purposes of this study.

Preliminary backwater analyses indicate that the backwater attributable to the four bridges within the study area was generally 0.1 foot or less in each case. In view of the complexities associated with simulating unsteady flow, 0.1 foot was not considered to be significant. Accordingly, backwater from the four bridges was not included in the stage-discharge relations and profiles simulated for this report.

Stage-Storage Relations

Stage-storage relations west of I-26 were determined using average cross-sectional end areas taken from 1- and 2-foot contour interval topographic maps. East of I-26, storage was determined by planimetering 2-foot orthophoto contour maps. Stage-storage relations were adjusted to account for vegetation in the channel and flood plain. An average reduction of 25 percent for vegetation volume was estimated by field inspection. Model runs made for subbasins with a 5 or 10 percent storage adjustment resulted in water-surface elevations up to 0.1 foot lower than those with the 25 percent correction. The larger adjustment represents a "worst case" situation and was used in this study because actual vegetation volumes could not be accurately determined.

Table 2.--Dimensions of culverts modeled at Filbin Creek, North Charleston, South Carolina

Roadway (subbasin outlet)	Existing conditions	Alternative A	Alternative B
I-526	N/A	5-4' RCP, lengths and locations identical to Alternative B	5-4' RCP, lengths and locations identical to Alternative A
I-26	2-6'x6' boxes, 175' long	2-6'x6' boxes, 675' long, w/30° angle at existing entrance	2-6'x6' boxes, 235' long, w/30° angle at existing entrance
Southern RR	3-5' CMP, 88' long	*	*
U.S. Highway 52 (South)	2-6'x6' boxes, 80' long 1-5.5' RCP, 125' long	*	*
Seaboard CL RR	1-8'x8' box (arch), 59' long	*	*
N. Rhett Ave.	2-5.5'x8' boxes, 102' long	*	*
Virginia Ave.	5-5' CMP w/gates, 66' long	*	*

*Indicates dimensions unchanged from existing conditions.

FLOOD PROFILE SIMULATIONS AND RESULTS

The Filbin Creek basin was subdivided into several subbasins with outlets at each major flow constriction. For each condition modeled, inflow hydrographs were routed through the most upstream structure using the reservoir routing techniques described earlier. The resultant outflow hydrographs were added to the intervening area inflow hydrograph of the next subbasin downstream and routed through the next constriction, and so on sequentially through the system. It was assumed that all subbasin hydrographs started at time = 0, and each intervening area's runoff created sufficient ponding behind each culvert to satisfy the assumption of routing from reservoir to reservoir. When necessary, initial stage-discharge relations were adjusted by trial and error techniques until tailwater elevation assumptions were not violated.

The assumptions made in modeling the post-construction conditions are:

1. All runoff west of I-526 and south of the airport terminal access road will flow to the twin 4-foot RCP's (reinforced concrete pipe) at station 807+10.

2. All runoff to the west of I-526 and to the north of the airport terminal access road will flow to the 4-foot RCP's at stations 836+50, 847+50, and 862+50.

3. Drainage development within the new airport facility will result in a shift of the natural drainage divide, reducing the drainage area of Filbin Creek by 0.43 mi^2. The drainage divide between the proposed airport facility and I-526 was an estimate between the airport drainage development phases for the years 1985 and 2000.

4. The slope of the existing culvert at I-26 will extend to the new inlet, thus, the inlet invert elevation will be approximately 8.0 feet for construction alternative A.

5. The culverts through I-526 are laid with no slope and their inverts lie at the ditch elevation shown in the highway plans.

Drainage areas, peak inflow discharges, and peak outflow discharges for each condition and recurrence interval are listed in table 3. Water-surface elevations are presented in table 4 and illustrated in figures 5-7.

As shown in table 4, the maximum backwater for any condition modeled was 0.5 foot, which occurs at the upstream side of I-526 for the 100-year flood. The computed water-surface elevations on either side of the airport access road differed because the construction plans did not show relief openings in the access road embankment. The peak water-surface elevations listed in table 4 for a given recurrence interval represent the highest of the two elevations computed, which was always the area to the south of the access roadway. If cross drains are installed at the access road and additional

Table 3.--Drainage areas, peak inflows and outflows at specified subbasin outlets for Filbin Creek, North Charleston, South Carolina (drainage area in square miles, flood inflows and outflows in cubic feet per second)

	I-526	I-26	Southern Railroad	U.S. Hwy. 52	Seaboard CL RR	N. Rhett Avenue	Virginia Avenue
Drainage area							
Existing[1]	--	3.59	0.52	0.76	0.40	2.03	1.17
Alternatives A[2] and B[3]	2.03	1.13	0.52	0.76	0.40	2.03	1.17
25-year flood inflow							
Existing	--	493	402	554	469	1,080	595
Alternative A	461	279	360	*	*	*	*
Alternative B	461	282	361	*	*	*	*
50-year flood inflow							
Existing	--	626	502	684	542	1,310	721
Alternative A	598	359	460	676	*	*	*
Alternative B	598	364	460	677	*	*	*
100-year flood inflow							
Existing	--	764	611	832	656	1,590	880
Alternative A	740	448	569	*	*	*	*
Alternative B	740	453	581	*	*	*	*
25-year flood outflow							
Existing	--	144	192	469	404	333	320
Alternative A	81	98	172	*	*	*	*
Alternative B	93	118	178	*	*	*	*
50-year flood outflow							
Existing	--	158	229	529	470	387	370
Alternative A	101	118	210	524	*	*	*
Alternative B	116	138	213	524	*	*	*
100-year flood outflow							
Existing	--	173	269	584	483	423	403
Alternative A	118	138	260	*	*	*	*
Alternative B	142	162	259	*	*	*	*

[1]Conditions present as of April 1983.
[2]Construction plan with all embankment upstream of I-26.
[3]Construction plan same as A except with I-26/I-526 interchange on structure.
*Values will not change significantly from existing conditions.

Table 4.--Peak water-surface elevations, in feet above sea level, upstream from indicated location for specified recurrence intervals and conditions for Filbin Creek, North Charleston, South Carolina

	I-526	I-26	Southern Railroad	U.S. Hwy. 52 South	Seaboard Coast Line Railroad	North Rhett Avenue	Virginia Avenue
25-year flood							
Existing[1]	--	11.1	10.4	8.9	7.1	3.7	3.0
Alternative A[2]	11.3	11.1	10.3	*	*	*	*
Alternative B[3]	11.3	11.0	10.3	*	*	*	*
50-year flood							
Existing	--	11.2	10.6	9.6	7.8	3.9	3.1
Alternative A	11.6	11.3	10.5	9.5	*	*	*
Alternative B	11.6	11.1	10.5	9.5	*	*	*
100-year flood							
Existing	--	11.3	10.9	10.2	8.3	4.0	3.2
Alternative A	11.8	11.5	10.8	*	*	*	*
Alternative B	11.8	11.3	10.8	*	*	*	*

[1]Conditions present as of April 1983.

[2]Construction plan with all embankment upstream of I-26.

[3]Construction plan same as A except with I-26/I-526 interchange on structure.

*Values will not change significantly from existing conditions.

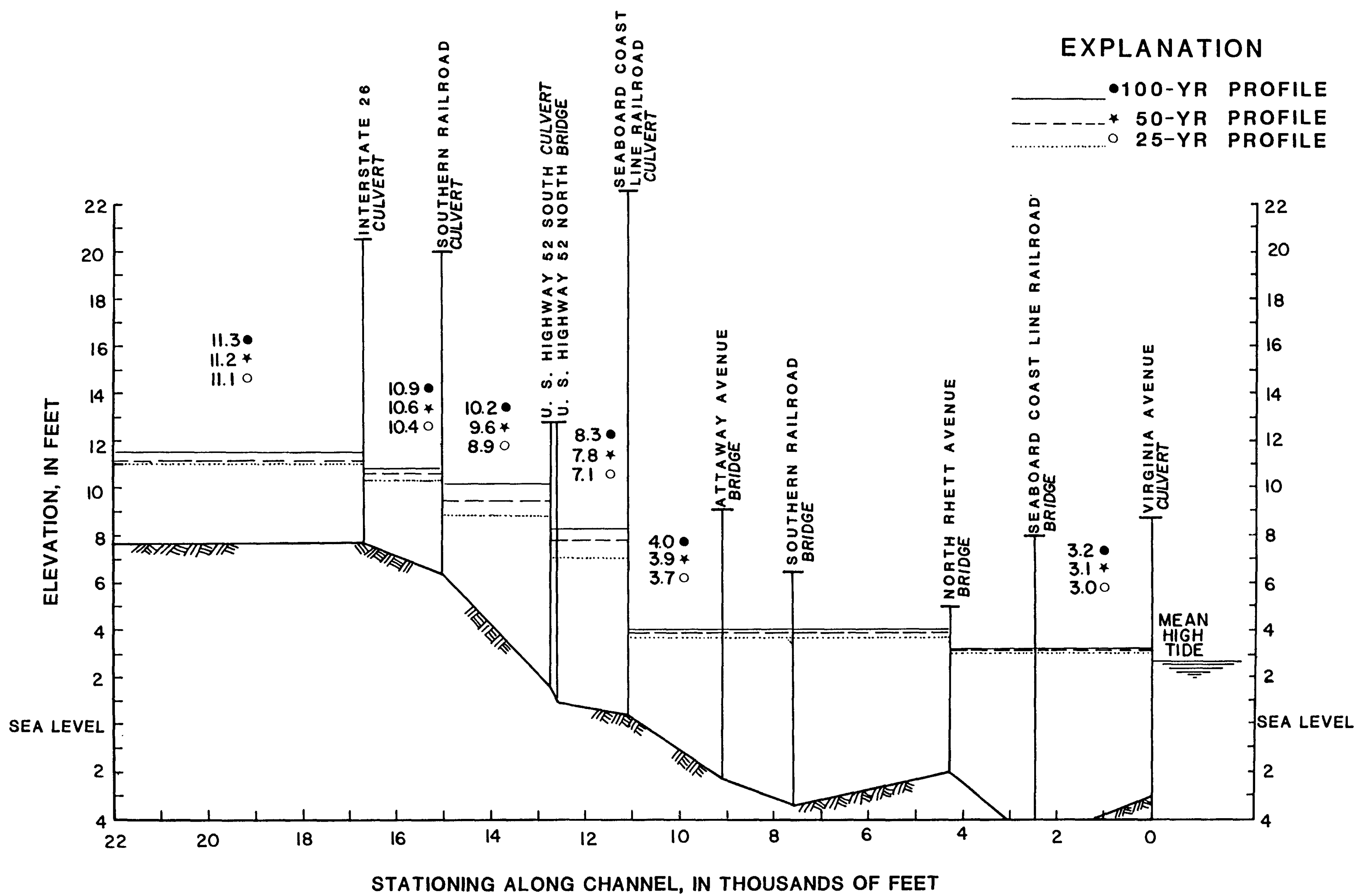

Figure 5.--Existing condition flood profiles, Filbin Creek, North Charleston, South Carolina.

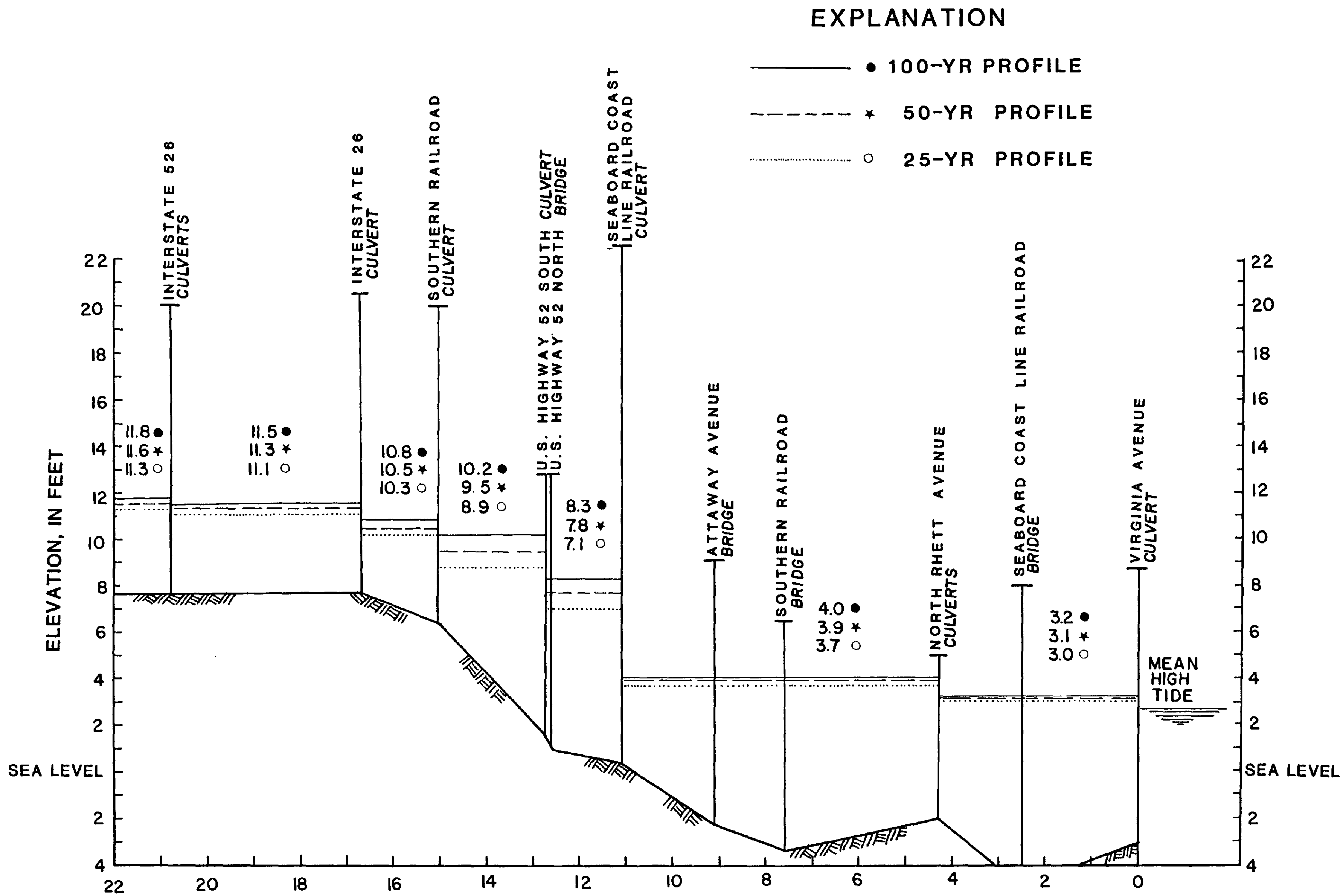

Figure 6.--Construction alternative A (all embankment upstream of I-26) flood profiles, Filbin Creek, North Charleston, South Carolina.

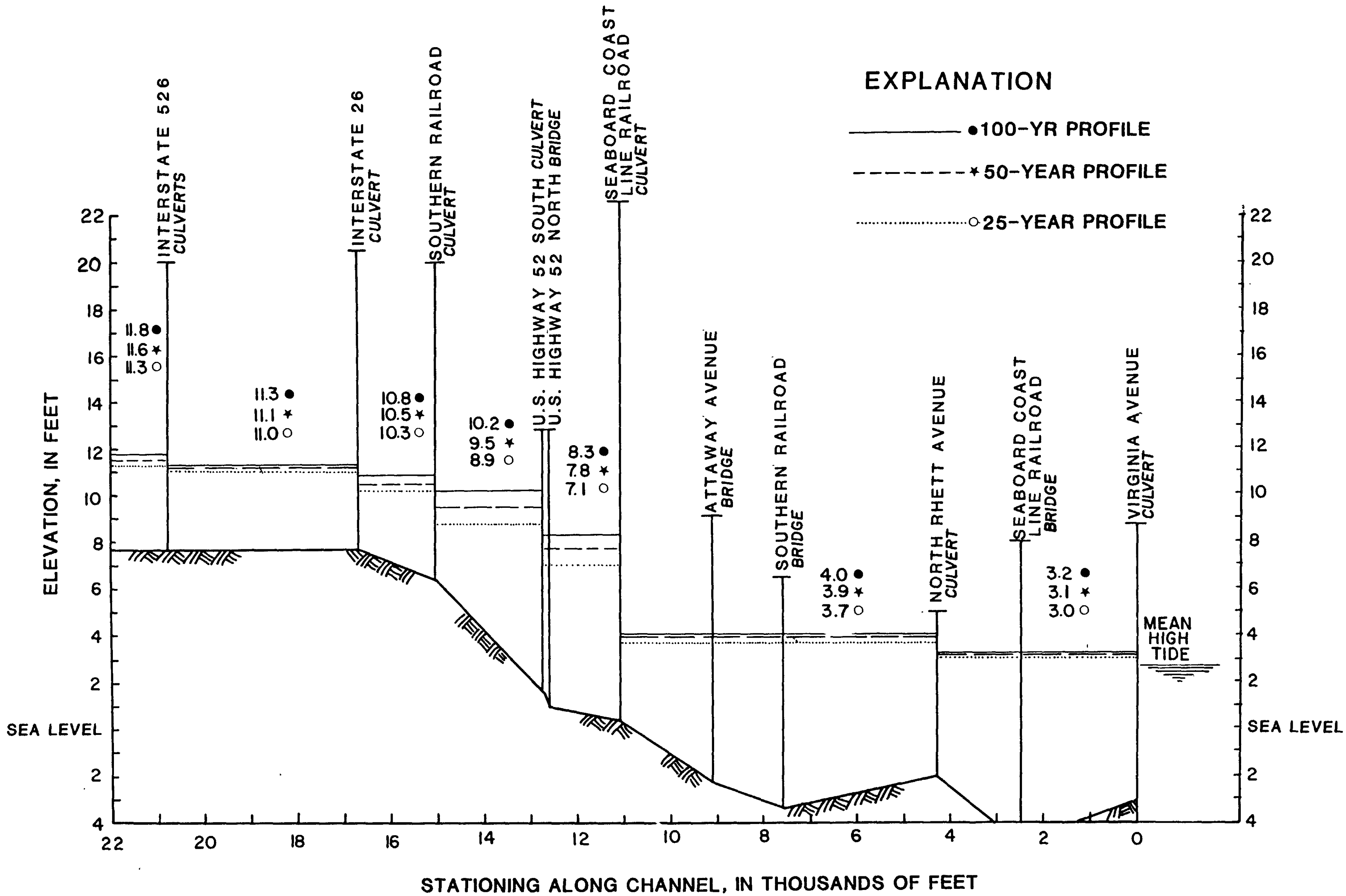

Figure 7.--Construction alternative B (all embankment upstream of I-26 with I-26/I-526 interchange elevated on structure) flood profiles, Filbin Creek, North Charleston, South Carolina.

cross drains included as part of the I-526 plans, the pool elevation may be nearly equal on both sides of I-526 and only slightly higher than the existing conditions pool elevations at I-26.

The water-surface elevations upstream of I-26 are primarily the result of two governing factors: (1) the peak inflow at I-26 and (2) the length of the culvert at I-26. The volume of runoff upstream of I-26 is constant for a given recurrence interval. However, construction of I-526 creates a separate detention basin which effectively reduces the peak inflow discharge for a given recurrence interval at I-26 by as much as 43 percent.

The stage-discharge relation at I-26 for construction alternative A (675-foot culvert) is significantly different from the stage-discharge relation for existing conditions. The discharge for a given pool elevation is significantly reduced for alternative A due to increased friction losses in the longer culvert. For the 100-year flood, the water-surface elevation for alternative A between I-526 and I-26 is 0.2 foot higher than existing conditions despite a lower peak inflow.

The stage-discharge relations at I-26 for existing conditions (175-foot culvert) and construction alternative B (235-foot culvert) are nearly identical. Thus, the pool elevations above I-26 are lower for construction alternative B than for existing conditions because of the reduced peak inflow resulting from detention storage above I-526.

The peak outflow at I-26 for a given recurrence interval will be less after construction of I-526 (table 3). Consequently, peak water-surface elevations downstream of I-26 will also be somewhat lower than those which would occur under existing conditions. For example, a simulation of the 50-year flood at Southern Railroad resulted in a peak water-surface elevation 0.1 foot less than that obtained for the existing condition simulation. Thus, construction of I-526 using either alternative A or B will have a negligible effect on flood profiles in the Filbin Creek basin downstream from I-26.

SUMMARY

The proposed construction alignment of I-526 in North Charleston, South Carolina generally follows the Filbin Creek flood plain. The Filbin Creek study area, which extends from upstream of I-26 to Virginia Avenue, is relatively flat, highly urbanized, and contains several areas of detention storage. A computerized reservoir routing technique was used to simulate water-surface profiles for the 25-, 50-, and 100-year floods for the existing drainage system and two alternative construction plans. Input requirements necessitated the development of synthetic hydrographs, stage-discharge relations, and stage-storage relations at each major constriction to flow.

Simulations indicate that the construction of I-526 will have little effect on the water-surface profiles downstream of I-26. Approximately 0.5 foot of backwater will be created upstream of I-526 with either

construction alternative. The pool elevations on both sides of I-526 will be nearly equal and only slightly higher than those for existing conditions if additional cross drains are installed at I-526 and at the airport access road. Differences in water-surface elevations between the three conditions modeled can be attributed primarily to reduced peak inflow at I-26 resulting from detention storage above I-526 and to the increased friction losses associated with the greater culvert lengths in the construction alternatives at I-26.

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