SECTION 1 INTRODUCTION

1.1 PROJECT AUTHORIZATION

Woolpert LLP was authorized to prepare a Storm Water Master Plan (SWMP) for the Church Creek Watershed under agreement with the City of Charleston. These terms are specified in the Agreement for Engineering Services dated September 2000 between the City of Charleston and Woolpert LLP.

1.2 PURPOSE

The Church Creek Watershed has experienced reoccurring residential flooding within the past 10 years. This watershed is approximately 8.5 square miles in size and drains to the Ashley River. The land use is currently comprised primarily of residential neighborhoods with undeveloped land and some industrial and commercial development. Over the past 10 years, the upper half of the watershed has undergone rapid residential development; there remains more than two square miles of undeveloped land planned for future development. During this same time period, there have been numerous yards that have flooded and several houses inundated with storm water on more than one occasion. As a result, the City of Charleston hired Woolpert to analyze the flooding problems and to create a Storm Water Master Plan for the Church Creek Watershed.

The purpose of this Storm Water Master Plan is to 1) identify existing storm water flooding problems, 2) analyze and recommend potential solutions to those identified problems, 3) analyze flooding impacts due to future development, 4) analyze the current storm water detention requirements, and 5) determine if land use restrictions or modifications to the detention requirements should be made. This process is in accordance with policies adopted by the City of Charleston to implement a comprehensive stormwater management program. The Church Creek Watershed SWMP will be used with other completed watershed SWMP's to establish citywide priorities for capital improvement projects based on the potential benefit versus overall cost (B/C ratio) of each recommended improvement. This SWMP may also be used as the community's source for hydraulic and hydrologic data for the Church Creek Watershed.

A substantial effort was made during the project planning process to make the ICPR model as user friendly as possible. The model created as a result of this study can and should be modified and updated based on improvements and changes made to individual components of the drainage system, as well as the drainage basin as a whole. This report provides the essential background information, methodologies, and assumptions necessary for the future use of the models as a tool in stormwater master planning the City of Charleston.

Upon acceptance of the final Church Creek Watershed SWMP document by the City of Charleston, the hydrologic and floodplain information from this report will be included in a Federal Emergency Management Agency (FEMA) restudy for Charleston County. Revised mapping of the floodplain may allow some property owners to qualify for flood insurance and be able to participate in FEMA programs for floodproofing, structure relocation or structure elevation. In addition, the SWMP will help the City of Charleston manage floodplains not regulated by FEMA.

FIGURE 1-1. – VICINITY MAP

SECTION 2 DESCRIPTION OF WATERSHED

2.1 WATERSHED LOCATION

The Church Creek Watershed is situated in the western part of Charleston in West Ashley with a total drainage area of 8.5 square miles (mi^2) that drains southeast to the Ashley River. Elevations in the watershed range from 35 feet National Geodetic Vertical Datum (NGVD 1929), near the top of the watershed along Ashley River Road (SC-61), to -4 feet NGVD at the confluence with the Ashley River. The upper portion of the watershed is mostly undeveloped land while the middle and lower portions are primarily residential with some commercial development along the major roadways.

2.2 HYDROLOGIC SUBDIVISION OF WATERSHED

For the purposes of this study, the watershed was divided into seven major groups, with a total of 89 sub-basins based on homogeneity of the drainage system, soils, and land use. The sub-basins range in size from two acres to 744 acres with an average size of 61 acres. The smaller sub-basins are located primarily in the residential neighborhoods while the larger sub-basins are located primarily in the undeveloped areas or downstream of the railroad. Each sub-basin was assigned a unique identifier. The naming scheme for the sub-basins consists of a “B-” followed by the node name where the sub-basin enters the model. If there is more than one sub-basin entering at the same node location, then the additional sub-basin’s digit is increased by one (e.g., sub-basins B-D130, B-D131, and B-D132 all enter at node N-D130). Table 2-1 lists the groups and their descriptions while Figure 2-1 shows the locations of the groups and sub-basins in the Church Creek Watershed.

Table 2-1. Group ID’s and Descriptions

Name ID	ICPR Group ID	Description
A	RR	Railroad Area and Below
B	HH	Hickory Hills Area
C	SM1	Shadowmoss #1 – Southeast Portion of Shadowmoss
D	SM2	Shadowmoss #2 – Northwest Portion of Shadowmoss
E	VG	Village Green Area
F	MC	Moss Creek Area
G	BL	Bees Landing Area

FIGURE 2-1 SUB-BASINS

2.3 STREAM CHANNELS

The Church Creek Watershed has a shape factor of 2.2:1 (length to width). There are three typical types of channels in the watershed: 1) channels that meander and branch through the marshy areas and have varying sizes and shapes, 2) wide man-made channels ranging from 30 ft to 50 ft in topwidth, with 4 ft to 10 ft bank heights that always have standing water, and 3) channels ranging from 15 ft to 25 ft in topwidth that travel through the neighborhoods and the upper portions of the watershed. The terrain within the watershed is flat with upstream inverts ranging from 3 ft to 6 ft in elevation. The modeled portion of the watershed consists of the main channel and several branching tributaries that have a combined length of approximately 20.2 miles.

2.4 SOILS

Soils in the Church Creek Watershed are predominantly in the C and D Hydrologic Soil Groups (HSG) and consist primarily of the Yonges (Yo)(HSG=D), Edisto (Ed)(HSG=C) and Hockley (HoA)(HSG=C) soil types. There are also large areas classified as Mine Pits (Mp), because this area was an old phosphate strip mine years ago, that are considered to have a HSG classification of D for this study. Other soils types include Stono (St), Kiawah (Ka), Wadmalaw (Wa), Santee (Se), and Capers (Cg) which are classified as Hydrologic Soil Group D, and Charleston (Ch), Seebrook (Sk), and Quitman (Qu) which are classified as Hydrologic Soil Group C. There are also areas of Wagram (WgB) and Wando (WnB) that fall in the Hydrologic Soil Group A.

The soils located in the Church Creek Watershed area are 5.1 percent HSG A, 0.2 percent HSG B, 25.0 percent HSG C, and 69.8 percent HSG D. Table 2-2 shows the percentage occupied by each Hydrologic Soil Group (HSG) within each sub-basin in the Church Creek Watershed. Figure 2-2 shows the Hydrologic Soil Group locations in the Church Creek Watershed.

Table 2-2. Summary of Hydrologic Soil Groups in Church Creek Watershed

SUB-BASIN NUMBER	HSG A (in percent)	HSG B (in percent)	HSG C (in percent)	HSG D (in percent)
B-A030	8.4	0	25	66.6
B-A040	6	2.3	28.5	63.1
B-A041	3.6	0	11	85.4
B-A060	23.2	0	7.2	69.7
B-A100	29.4	0	10.6	60.1
B-A120	14.7	0	4.5	80.7
B-A140	0.3	0	36.9	62.9
B-B020	0	0	0	100
B-B040	0	0	0	100
B-B060	0	0	61.3	38.7
B-B100	0	0	66.9	33.1
B-B140	0	0	36.2	63.8
B-B160	0	0	14.2	85.8
B-B170	0	0	0	100
B-B230	0	0	0	100
B-C010	0	0	19.3	80.7
B-C050	0	0	52.1	47.9
B-C080	0	0	37.7	62.3
B-C120	0	0	69.4	30.6
B-C130	0	0	34.2	65.8
B-C140	0	0	0	100
B-C150	4.3	0	82.3	13.4
B-C170	0	0	59.8	40.2
B-C190	0	0	26.9	73.1
B-C230	0	0	33.4	66.6
B-C270	0	0	65.8	34.2
B-D010	0	0	63.1	36.9
B-D020	0	0	20.6	79.4
B-D030	0	0	0	100
B-D050	0	0	15.8	84.2
B-D080	0	0	0	100
B-D110	0	0	12.7	87.3
B-D130	0	0	0	100
B-D131	33.1	0	34.7	32.2
B-D132	0	0	0	100

SUB-BASIN NUMBER	HSG A (in percent)	HSG B (in percent)	HSG C (in percent)	HSG D (in percent)
B-D140	0	0	0	100
B-D160	0	0	2.4	97.6
B-D190	0	0	0	100
B-D210	0	0	0	100
B-D220	0	0	0	100
B-E010	14	0	21.6	64.4
B-E020	0	0	0	100
B-E030	0	0	0	100
B-E040	0	0	0	100
B-E050	0	0	0	100
B-E060	0	0	0	100
B-E070	0	0	0	100
B-E080	0	0	0	100
B-E090	0	0	0	100
B-E100	0	0	0	100
B-E110	0	0	0	100
B-E120	0	0	0	100
B-E130	0	0	20.6	79.4
B-E140	0	0	94.6	5.4
B-E150	0	0	0	100
B-E160	0	0	0	100
B-E170	0	0	0	100
B-E180	0	0	0	100
B-E190	0	0	0	100
B-E200	0	0	0	100
B-E210	0	0	0	100
B-E220	0	0	0	100
B-E230	0	0	0	100
B-E231	0	0	0	100
B-E240	0	0	0	100
B-E250	0	0	0	100
B-E251	0	0	0	100
B-E252	0	0	0.5	99.5
B-E260	3.9	0	14.2	81.9
B-E270	0	0	0	100
B-F010	0	0	97	3

SUB-BASIN NUMBER	HSG A (in percent)	HSG B (in percent)	HSG C (in percent)	HSG D (in percent)
B-F030	17.4	0	53	29.6
B-F040	5.4	0	94.6	0
B-F060	50.8	0	48.9	0.3
B-F080	0	0	100	0
B-G020	0	0	55.4	44.6
B-G050	0	0	33.2	66.8
B-G060	0	0	100	0
B-G070	0	0	95.8	4.2
B-G080	0	0	100	0
B-G090	0	0	100	0
B-G110	0	0	100	0
B-G120	0	0	100	0
B-G130	0	0	60.9	39.1
B-G140	0	0	76.6	23.4
B-G150	0	0	7.7	92.3
B-G160	0	0	100	0
B-G180	11.2	0	10.5	78.3
B-G181	11.6	0	35.9	52.5

FIGURE 2-2 HYDROLOGIC SOIL GROUPS

2.5 LAND USE

Land use in the Church Creek Watershed is characterized by marsh, woods, residential and commercial areas in the lower portion of the watershed, mostly residential in the middle portion of the watershed with some commercial and industrial areas along Bees Ferry Road, and mostly undeveloped land in the upper portion of the watershed with limited residential. The existing land use used for this study consisted of December 2000 conditions. For hydrologic modeling purposes, land use in the Church Creek Watershed is defined by the 15 land use categories and they are listed in Table 2-3.

The existing land use was determined using current City zoning, GIS data of the watershed, and field observations. Table 2-3 shows existing land use distribution within the Church Creek Watershed. Table 2-4 shows runoff curve numbers by Land Use Category and Hydrologic Soil Group. The runoff curve numbers presented in Table 2-4 are based on average antecedent runoff conditions (ARC-II).

Table 2-3. Existing TR55 Land Use Distribution in Church Creek Watershed

LAND USE CATEGORY CODE	PERCENT OF WATERSHED	LAND USE DESCRIPTION
ROW	1.0	Impervious Roads, Including Right-of-Way
COM	1.1	Urban Commercial Centers – Malls, Strip Shopping Centers
IND	1.0	Urban Industrial and Manufacturing
OFF	1.8	Office parks and schools
MF	1.4	Multi Family Dwellings – Apartments/Townhomes
R25	1.1	Single Family Residential – 0.25 acre lots
R33	19.5	Single Family Residential – 0.33 acre lots
R50	6.0	Single Family Residential – 0.50 acre lots
R200	2.2	Single Family Residential – 2.00 acre lots
RR	1.0	Rail Road
GOLF	2.9	Golf Courses
OPEN	1.5	Lawns, Parks – Fair condition
WOODS	54.9	Woods/Brush (Good Condition)
MARSH	3.0	Marsh/Swamps
H2O	1.6	Water Bodies

Table 2-4. TR55 Runoff Curve Numbers by Land Use Category and Hydrologic Soil Group

LAND USE CATEGORY CODE	LAND USE DESCRIPTION	Hydrologic Soil Group			
		A	B	C	D
ROW	Impervious Roads Including Right-of-Way	**83**	**89**	**92**	**93**
COM	Urban Commercial Centers – Malls, Strip Shopping Centers	**89**	**92**	**94**	**95**
IND	Urban Industrial and Manufacturing	**81**	**88**	**91**	**93**
OFF	Schools/Colleges/Hospitals & office parks and centers	**72**	**81**	**87**	**90**
MF	Multi Family Dwellings – Apartments/Townhomes	**77**	**85**	**90**	**92**
R25	Single Family Residential – 0.25 acre lots	**61**	**72**	**81**	**85**
R33	Single Family Residential – 0.33 acre lots	**57**	**70**	**80**	**84**
R50	Single Family Residential – 0.50 acre lots	**54**	**68**	**79**	**83**
R200	Single Family Residential – 2.0 acre lots	**46**	**64**	**76**	**81**
RR	Rail Road	**76**	**85**	**89**	**91**
GOLF	Golf Courses	**39**	**61**	**74**	**80**
OPEN	Lawns, Parks – Fair condition	**49**	**69**	**79**	**84**
WOODS	Woods/Brush (Good Condition)	**36**	**60**	**73**	**79**
MARSH	Marsh/Swamps	**99**	**99**	**99**	**99**
H2O	Water Bodies	**99**	**99**	**99**	**99**

Table 2-5. Existing Land Use Distribution Within Each Sub-basin in the Church Creek Watershed

Sub-Basin	ROW	COM	IND	OFF	MF	R25	R33	R50	R200	RR	GOLF	OPEN	WOODS	MARSH	H2O
B-A030	0.0	2.7	0.0	6.2	0.2	3.1	38.6	15.8	2.0	1.6	0.0	2.9	11.3	13.8	1.7
B-A040	0.8	6.6	0.0	0.0	1.8	0.0	0.0	47.3	0.5	0.0	0.0	0.0	28.5	7.7	6.8
B-A041	3.2	0.3	0.0	14.4	7.7	0.0	2.0	10.9	0.0	4.0	0.0	0.0	41.6	14.2	1.6
B-A060	0.0	0.0	0.0	0.0	0.0	15.0	34.7	0.0	0.0	5.7	0.0	0.0	44.2	0.0	0.4
B-A100	4.6	0.0	44.5	0.0	0.0	0.0	0.0	0.0	0.0	4.7	0.0	0.0	45.4	0.0	0.9
B-A120	7.2	0.0	30.6	0.0	0.0	0.0	0.0	0.0	0.0	4.9	0.0	0.1	54.9	0.0	2.3
B-A140	8.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.2	0.0	0.0	86.4	0.0	0.4
B-B020	1.0	0.0	0.0	0.0	0.0	0.0	62.4	0.0	31.7	0.0	0.0	0.2	0.0	0.0	4.7
B-B040	6.5	0.0	0.0	0.0	0.0	0.0	75.7	0.0	0.0	0.0	0.0	2.7	0.0	0.0	15.0
B-B060	3.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.1	0.0	0.0	0.0	86.7	0.0	0.7
B-B100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	81.2	0.0	0.0	0.0	16.5	0.0	2.3
B-B140	0.0	0.0	0.0	0.0	0.0	0.0	8.8	0.0	50.8	0.0	0.0	0.0	39.5	0.0	1.0
B-B160	0.2	0.0	0.0	0.0	0.0	0.0	59.4	0.0	0.0	0.0	17.3	5.3	12.9	0.0	4.9
B-B170	0.0	0.0	0.0	0.0	0.0	0.0	93.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.6
B-B230	3.7	0.0	0.0	0.0	0.0	0.0	71.3	0.0	0.0	0.0	0.0	5.7	0.0	0.0	19.3
B-C010	4.7	0.0	0.0	0.0	0.0	0.0	43.4	0.0	0.0	0.0	37.8	12.9	0.0	0.0	1.2
B-C050	9.9	0.0	0.0	0.0	0.0	0.0	22.7	0.0	0.0	0.0	65.2	0.0	0.0	0.0	2.2
B-C080	5.3	12.3	0.0	6.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	75.7	0.0	0.0
B-C120	1.5	0.0	0.0	0.0	35.8	0.0	32.0	0.0	0.0	0.0	10.0	11.0	6.6	0.0	3.1
B-C130	0.0	0.0	0.0	0.0	0.0	0.0	73.9	0.0	0.1	0.0	19.5	0.0	0.0	0.0	6.4
B-C140	0.0	0.0	0.0	0.0	0.0	0.0	66.7	0.0	0.0	0.0	29.2	0.0	0.0	0.0	4.1
B-C150	0.0	0.0	0.0	0.0	0.1	0.0	43.1	0.0	0.0	0.0	3.4	32.5	20.9	0.0	0.0
B-C170	0.0	0.1	0.0	0.0	0.0	0.0	59.9	0.0	0.0	0.0	4.9	34.5	0.0	0.0	0.6
B-C190	0.0	0.0	0.0	0.0	0.0	0.0	51.4	0.0	0.0	0.0	47.6	0.0	0.0	0.0	1.0
B-C230	0.6	0.0	0.0	0.0	0.0	0.0	85.3	0.0	0.0	0.0	9.6	0.0	0.0	0.0	4.5
B-C270	0.0	2.1	0.0	0.0	0.0	0.0	16.2	0.0	0.0	0.0	76.4	0.0	5.4	0.0	0.0
B-D010	0.0	0.0	0.0	0.0	0.0	0.0	7.9	0.0	0.6	0.0	4.9	0.0	84.9	0.0	1.6
B-D020	0.0	0.0	0.0	0.0	7.4	12.4	0.0	0.0	0.0	0.0	45.3	0.0	30.0	0.0	4.9
B-D030	0.0	0.4	0.0	0.0	16.2	40.2	0.0	0.0	0.0	0.0	6.4	0.0	31.1	0.0	5.8
B-D050	0.0	35.9	0.0	0.0	0.3	0.4	3.4	0.0	0.0	0.0	56.2	0.0	0.0	0.0	3.8
B-D080	0.0	0.0	0.0	0.0	21.3	0.0	7.1	0.0	0.0	0.0	65.0	0.0	0.0	0.0	6.6
B-D110	0.0	0.0	0.0	0.0	8.5	0.6	52.8	0.6	0.0	0.0	32.7	0.0	2.5	0.0	2.2
B-D130	0.0	0.0	0.0	0.0	0.0	0.0	83.4	0.0	0.0	0.0	10.3	0.0	5.1	0.0	1.2
B-D131	0.9	0.0	0.0	0.0	0.0	0.0	1.8	0.0	0.0	0.0	0.0	0.0	97.2	0.0	0.1
B-D132	0.0	0.0	0.0	0.0	0.0	10.0	2.2	0.0	0.0	0.0	33.3	0.0	53.7	0.0	0.8
B-D140	0.0	0.0	0.0	0.0	0.0	0.0	72.6	0.0	0.0	0.0	1.0	0.0	25.1	0.0	1.3
B-D160	0.0	0.0	0.0	0.0	0.0	0.0	95.0	0.0	0.0	0.0	3.8	0.0	0.0	0.0	1.2
B-D190	0.0	0.0	0.0	0.0	0.0	0.0	65.8	0.0	0.0	0.0	21.0	0.0	0.0	0.0	13.2
B-D210	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-D220	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E010	0.0	0.0	0.0	0.0	0.0	0.0	0.9	0.0	0.0	0.0	0.0	0.0	98.8	0.0	0.3

Sub-Basin	ROW	COM	IND	OFF	MF	R25	R33	R50	R200	RR	GOLF	OPEN	WOODS	MARSH	H2O
B-E020	0.0	0.0	0.0	0.0	0.0	0.0	74.5	0.0	0.0	0.0	0.0	0.0	25.5	0.0	0.0
B-E030	0.0	0.0	0.0	0.0	0.0	0.0	91.3	0.0	0.0	0.0	0.0	0.0	8.7	0.0	0.0
B-E040	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E050	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E060	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E070	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E080	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E090	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E100	0.0	0.0	0.0	0.0	0.0	0.0	99.1	0.0	0.0	0.0	0.0	0.0	0.9	0.0	0.0
B-E110	0.0	0.0	0.0	0.0	0.0	0.0	99.9	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0
B-E120	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E130	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E140	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E150	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E160	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E170	0.0	0.0	0.0	0.0	0.0	0.0	97.3	0.0	0.0	0.0	0.0	0.0	2.7	0.0	0.0
B-E180	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E190	0.0	0.0	0.0	0.0	0.0	0.0	98.7	0.0	0.0	0.0	0.0	0.0	1.3	0.0	0.0
B-E200	0.0	0.0	0.0	0.0	0.0	0.0	86.9	0.0	0.0	0.0	0.0	0.0	13.1	0.0	0.0
B-E210	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E220	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E230	0.0	0.0	0.0	0.0	0.0	0.0	98.9	0.0	0.0	0.0	0.0	0.0	1.1	0.0	0.0
B-E231	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0
B-E240	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-E250	0.0	0.0	0.0	0.0	0.0	0.0	63.9	0.0	0.0	0.0	0.0	0.0	36.1	0.0	0.0
B-E251	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	98.7	0.0	1.3
B-E252	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.0	0.0	0.0	0.0	0.0	99.7	0.0	0.0
B-E260	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	99.7	0.0	0.3
B-E270	0.0	0.0	0.0	0.0	0.0	0.0	23.2	0.0	0.0	0.0	0.0	0.0	76.8	0.0	0.0
B-F010	0.0	0.0	0.0	0.0	0.0	0.0	89.0	0.0	0.0	0.0	0.0	0.0	11.0	0.0	0.0
B-F030	0.0	0.0	0.0	0.0	0.0	0.0	64.3	0.0	0.0	0.0	0.0	0.0	35.7	0.0	0.0
B-F040	0.0	0.0	0.0	0.0	0.0	0.0	78.5	0.0	0.0	0.0	0.0	0.0	21.5	0.0	0.0
B-F060	5.4	0.0	0.0	0.0	0.0	0.0	56.9	0.0	0.0	0.0	0.0	0.0	37.7	0.0	0.0
B-F080	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	99.0	0.0	0.0
B-G020	7.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	91.7	0.0	1.2
B-G050	0.0	0.0	0.0	0.0	0.0	0.0	0.8	0.0	0.0	0.0	0.0	0.0	98.3	0.0	1.0
B-G060	0.0	0.0	0.0	0.0	0.0	0.0	92.4	0.0	0.0	0.0	0.0	0.0	7.6	0.0	0.0
B-G070	0.0	0.0	0.0	0.0	0.0	0.0	99.9	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0
B-G080	0.0	0.0	0.0	0.0	0.0	0.0	98.5	0.0	0.0	0.0	0.0	0.0	1.5	0.0	0.0
B-G090	0.2	0.0	0.0	0.0	0.0	0.0	99.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-G110	0.0	0.0	0.0	0.0	0.0	0.0	95.4	0.0	0.0	0.0	0.0	0.0	4.6	0.0	0.0
B-G120	0.0	0.0	0.0	0.0	0.0	0.0	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-G130	0.6	0.0	0.0	0.0	0.0	0.0	99.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B-G140	1.6	0.0	0.0	0.0	0.0	0.0	98.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sub-Basin	ROW	COM	IND	OFF	MF	R25	R33	R50	R200	RR	GOLF	OPEN	WOODS	MARSH	H2O
B-G150	4.5	0.0	0.0	0.0	0.0	0.0	0.8	0.0	0.0	0.0	0.0	0.0	94.8	0.0	0.0
B-G160	0.9	0.0	0.0	0.0	0.0	0.0	78.6	0.0	0.0	0.0	0.0	0.0	20.5	0.0	0.0
B-G180	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	99.5	0.0	0.1
B-G181	2.8	0.0	0.0	0.0	0.0	0.0	1.1	0.0	0.0	0.0	0.0	0.0	95.8	0.0	0.2

FIGURE 2-3 EXISTING LAND USE

SECTION 3 DESCRIPTION OF DATA USED IN ANALYSES

3.1 FIELD RECONNAISSANCE DATA

3.1.1 Stream Evaluations

A field reconnaissance of the Church Creek Watershed was performed by walking the length of the Church Creek mainstem and its tributaries. During this field reconnaissance, channel and overbank Manning's roughness coefficients were recorded with typical channel dimensions, pipe structure sizes and materials, depths of fill cover over the structures, hydraulic characteristics of detention/retention structures, watershed and sub-basin boundaries, and areas of flooding problems. Verification photographs were taken of all the structures and channels that were to be modeled. These photographs are provided on the CD in Appendix D of this report.

3.1.2 Survey Data

Invert elevations and roadway overtopping elevations for key culverts, bridges and detention structures were surveyed with several channel inverts taken in key locations. Finish floor elevations for 44 houses and six townhouse buildings were also surveyed for model calibration and for the alternative analysis.

3.2 RAINFALL DATA

3.2.1 Design Storm Data and Rainfall Frequency Depths

Rainfall depth/duration/frequency data for the 2-, 10-, 25-, 50-, and 100-year frequency storm events was obtained from the South Carolina Stormwater Management and Sediment Control Handbook (1995). This data was used to develop the 500-year, 24-hour rainfall amount using Probability-Log paper. These 24-hour rainfall amounts were used with the SCS TYPE III rainfall distribution in the ICPR model to calculate rainfall runoff amounts. The rainfall depth values are listed in Table 3-1 and the SCS TYPE III distribution is listed in Table 3-2.

Table 3-1. Rainfall Depth/Duration/Frequency Data

Storm Event	Rainfall Depth (inches)
2-year 24-hour	4.6
10-year 24-hour	6.8
25-year 24-hour	7.8
50-year 24-hour	8.8
100-year 24-hour	10.0
500-year 24-hour	11.5

Table 3-2. SCS TYPE III 24-Hour Storm Hydrograph Rainfall Distribution (15-minute intervals, P_{time} / P_{24})

0.000	0.002	0.005	0.007	0.010	0.012	0.015	0.017
0.020	0.023	0.026	0.028	0.031	0.034	0.037	0.040
0.043	0.047	0.050	0.053	0.057	0.060	0.064	0.068
0.072	0.076	0.080	0.085	0.089	0.094	0.100	0.107
0.115	0.122	0.130	0.139	0.148	0.157	0.167	0.178
0.189	0.202	0.216	0.232	0.250	0.271	0.298	0.339
0.500	0.662	0.702	0.729	0.751	0.769	0.785	0.799
0.811	0.823	0.834	0.844	0.853	0.862	0.870	0.878
0.886	0.893	0.900	0.907	0.911	0.916	0.920	0.925
0.929	0.933	0.936	0.940	0.944	0.947	0.951	0.954
0.957	0.960	0.963	0.966	0.969	0.972	0.975	0.978
0.981	0.983	0.986	0.988	0.991	0.993	0.996	0.998
1.000							

3.2.2 Historical Rainfall Data

Rainfall gage data for the Charleston Airport was obtained for the periods from January 1990 through December 2000, and for July 2001. This information was reported in hourly increments and was used to model historical storm events that had reported flooding problems. The storm periods modeled are listed in Table 3-3 along with the total rainfall amounts. All of the gage data obtained is reported in Appendix C.

Table 3-3. Rainfall Depth/Duration/Frequency Data

Year	Month	Day	24-Hour Depth (inches)	48-Hour Depth (inches)
2001	July	27	4.66	4.66
1999	September	29	5.35	5.71
1998	September	21	10.52	10.52
1998	February	17	5.92	5.92
1994	October	3	4.02	4.22

3.3 CITY OF CHARLESTON DATA

3.3.1 Digital Topography and Planimetric Coverages

Topographic and planimetric data were furnished by the City of Charleston in the form of ARCInfo GIS format coverages. This data was used in the watershed boundary delineation, stream connectivity determination, digital cross section generation, delineation of the time-of-concentration flow paths for lag time computations, street name definition, and spatial location of structures located in the floodplains. Digital topographic data was converted to National Geodetic Vertical Datum (NGVD) 1929. A hard copy of the City of Charleston map M-14 (1980, NGVD 1929) was used as supplementary data for locations without topographic data.

3.3.2 Storm Drainage Studies

Storm drainage studies done for sites within the watershed were provided by the City of Charleston. Information from these studies was used to verify invert elevations, detention storage and drainage results. The studies for Village Green, Moss Creek and Bees Landing completed by Seamon, Whiteside & Associates also contained ICPR models. Data from these three models were incorporated into the ICPR model for the entire watershed.

3.3.3 Historical Flooding Information

Areas of known flooding problems within the watershed were provided by the City of Charleston. Additional information was collected from residents that live within the watershed at a public meeting held on November 30, 2000. This information was used to validate and calibrate the ICPR model results.

3.4 FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA)-FIS STUDY

The Flood Insurance Rate Map for the effective FIS was obtained for the Church Creek Watershed. Map (455412 0005D) was used to compare the floodplain elevations in the effective FIS for Church Creek with those developed as a result of this analysis. The effective FIRM shows this area as ZONE A2, A5, or A13, with a WSEL of 13 ft for the area downstream of Ashley River Road (SC-61), a WSEL of 11 ft for the area between the railroad and Ashley River Road, and a WSEL of 8 ft upstream of the railroad.

3.5 UNITED STATES DEPARTMENT OF AGRICULTURE (USDA) SOILS COVERAGES

Soils information was obtained from the Charleston County Soil Survey (US Department of Agriculture, March 1971). This information was used to create a digital Hydrologic Soil Group (HSG) coverage. The soils coverage was used to compute the runoff curve number for each sub-basin.

3.6 NOAA/NOS DATA

Tide gage data for the Cooper River Entrance (8665530) in Charleston, SC was obtained for the period from January 1990 through December 2000. This information contained the monthly Mean High Water (MHW), Mean Low Water (MLW), Mean Higher High Water (MHHW) and Mean Lower Low Water (MLLW) elevations. The maximum MHHW and MLW elevations for this 10-year period were projected up the Ashley River and used as boundary conditions for the ICPR model at the confluence with the Ashley River. The projected MHHW and MLW elevations were 4.8 ft and –1.1 ft respectively. This information is reported in Appendix C.

3.7 UNITED STATES GEOLOGICAL SURVEY (USGS) DATA

Elevation data from the USGS quadrangle for Johns Island, SC (1979) was used for locations where there was no City of Charleston elevation data. (NGVD of 1929)

There are no active U.S. Geological Survey (USGS) streamflow gaging stations or rainfall gaging stations located within the Church Creek Watershed.

3.8 TIME OF CONCENTRATION

A Microsoft Excel spreadsheet, programmed with the Soil Conservation Service (SCS) methodology and equations, was used to calculate the time of concentration for each sub-basin in the Church Creek Watershed. The program contains equations for calculating the travel time associated with the sheet flow, shallow concentrated flow, and channel flow segments of the overall time of concentration flow path within each sub-basin. In areas where there was a previous ICPR model completed, the previous study's time of concentration values were used.

SECTION 4 DESCRIPTION OF HYDROLOGIC AND HYDRAULIC MODELING

4.1 MODEL USED

The ICPR computer model (version 2.2) was used to model the Church Creek Watershed. This model is a link/node computer model that creates rainfall runoff hydrographs and then routes these hydrographs through the watershed. A general explanation of how the computer model works, excerpted from the ICPR User's Manual on page 2-12, is as follows.

"Each node in an ICPR model represents a control volume. Water enters and leaves each node by the links connected to it or from it. Water also enters nodes by way of surface runoff hydrographs and/or base flow. Storage at each node is provided by any link connected to or from it (e.g., channel storage) and/or any user specified supplemental storage (e.g., pond storage or overbank flooding). ICPR calculates the change in storage for each node based on the differences between inflows and outflows at each time step during the simulation period. The changes in storage are used to determine elevations at each node. Flows through each link are calculated from the known elevations at each end of the link and the hydraulic properties of the link (e.g., slope, roughness, geometric configuration, etc)."

4.2 HYDROLOGIC MODELING

4.2.1 Model Assumptions

The Hydrology routine in ICPR is used to compute runoff losses and then generate a runoff hydrograph for each of the sub-basins within the Church Creek Watershed. The following are some of the underlying theoretical assumptions that govern the model's applicability to this watershed:

- The hydrologic process can be represented by the model parameters that reflect average conditions within a catchment area.
- Rainfall and losses are uniformly distributed across the watershed.
- All runoff from a catchment area goes to the same outfall point...eventually.
- The modeling procedure used in this modeling project followed the "SCS Methodology". This terminology is an "umbrella" term used to cover a wide range of procedures relating to rainfall and losses, runoff and hydrograph routing, and use of the SCS Unit Dimensionless Hydrograph to develop runoff hydrographs.
- The 24-hour storm was used for all flood frequency simulations in this project.
- The SCS Type III rainfall distribution was used.

- Adjustments for directly connected impervious areas were not considered because the SCS methodology used in this modeling project assumes “very short” flow paths between the imperviousness and the drainage system for urban condition runoff curve numbers. In the majority of the urban settings in the Church Creek Watershed, the flow path distance from the impervious surface to the storm water collector system is indeed relatively short. Therefore, runoff (particularly during the less frequent storm events) cannot pond long enough for significant infiltration losses to occur.

4.2.2 Model Parameter Development

There are 89 sub-basins within the Church Creek Watershed that were modeled. The parameter data for each of these sub-basins can be found in Section 8 of this report. The following is the list of parameter data used and/or developed for the hydrologic modeling.

Rainfall Data: Rainfall depths from the South Carolina Stormwater Management and Sediment Control Handbook were used along with the SCS Type III rainfall distribution. For this analysis, the 2-, 10-, 25-, 50-, 100-, and 500-year storms were used.

Drainage Area: Drainage basin boundaries for the Church Creek Watershed, and sub-basins were delineated in ARC-Info using a Digital Terrain Model (DTM) of the watershed. The GIS was used to compute the area of each sub-basin and the entire Church Creek Watershed. The average sub-basin area was 61 acres.

Runoff Curve Numbers: A weighted runoff curve number was calculated for each sub-basin by using GIS with soils, land use, and sub-basin boundary coverages. The GIS references a “Look-up Table” of runoff curve numbers for the various soil and land use category combinations and assigns a runoff curve number to each polygon within a sub-basin. For a given sub-basin, the individual runoff curve numbers are multiplied by the drainage area of the polygon they represent and the results are summed and divided by the total drainage area of the sub-basin. The resultant runoff curve number is the weighted runoff curve number for the sub-basin.

Time of Concentration/Lag Time: Time of concentration is the time required for a drop of water (during a 2-year runoff event) to travel from the hydraulically most remote part of a catchment to its outfall. The time of concentration has three associated flow path components:

1) sheet flow,
2) shallow concentrated flow, and
3) channel flow.

In general, the length of the sheet flow segment for a sub-basin was limited to 100 feet for urban areas and 300 feet for undeveloped or rural areas. The shallow concentrated flow segment extended from the downstream end of the sheet flow segment to the first topographically defined swale or pipe on the topographic maps. The channel flow segment extended from the downstream end of the shallow concentrated flow segment to the outfall of the sub-basin. Where appropriate, the channel flow segment was subdivided into circular, triangular, rectangular, and trapezoidal channel sub-reaches.

4.3 HYDRAULIC MODELING

4.3.1 Assumptions

As with the Hydrology routine, it is important to understand the underlying theoretical assumptions with the Hydraulic routine. These assumptions are as follows:

- The stream channel has rigid boundary conditions (the cross sectional area does not change with time).

- Manning's roughness coefficients were selected to represent full summer growth conditions (worst case). In addition, weighted roughness coefficients were used that were representative of the full range of flood discharges being modeled.

- All structures and channels in the Church Creek Watershed were modeled as if they were open and free of debris. The reason for this assumption was to produce water-surface profiles reflecting system design capacity. It should be noted, however, that some of the structures were in reality partially blocked with debris or sediment deposits.

4.3.2 Model Input Parameters

Channel Invert Profiles: The channel invert profiles were developed by combining the inverts derived from topographic maps, surveyed inverts at road crossings that were collected by Woolpert surveyors, information provided in previous drainage studies within the watershed, and information collected during the field reconnaissance.

Cross Sections: Cross sections were created in GIS using topographic and planimetric coverages. Channel dimensions collected during the field reconnaissance were then merged with the DTM cross sections to make a composite cross section that best represents the actual channel section.

Manning's n Values: Manning's roughness coefficients were derived for the channels and floodplains from the field reconnaissance. A Manning's roughness value of 0.013 was used for concrete culverts (pipe and box) and a Manning's roughness value of 0.025 was used for corrugated metal pipe culverts. Manning's roughness values in the channels ranged from 0.045 to 0.075 while the overbanks had values that ranged from 0.020 to 0.180.

Boundary Conditions: The maximum MHHW and MLW elevations for the last 10 years at the Cooper River Entrance were projected up the Ashley River and used as boundary conditions for the ICPR model at the confluence with the Ashley River. The projected MHHW and MLW elevations were 4.8 ft and –1.1 ft respectively, and were used for all the modeled storm events using a linear 12-hour cycle. The start of the cycle was set so that the effects of high tide in the vicinity of the railroad occurred around hour 22, which is approximately the same time as the peak hydrograph from the 24-hour SCS Type III storm event. This was done to produce the largest backwater effects at the railroad crossing.

4.4 MODEL CONNECTIVITY

For the purposes of this study, the watershed was divided into seven major groups to be used in the ICPR model. The ICPR model for the Church Creek Watershed contains 89 sub-basins, 138 nodes, 57 channel links, 84 pipe links, 57 weir links, nine drop-structure links, and two bridges. Table 4-1 lists the group names, the number of structures and number of storage nodes within each group, while Figure 4-1 shows the ICPR link/node network. A large scale map of the ICPR link/node network is also provided in Appendix E. The following is the scheme used to name the nodes, links and templates within the ICPR model:

Nodes: Consist of a "N-" followed by the node name, which is the group name ID and a 3-digit number. (Example: N-A050, N-B110, or N-G150) Nodes were numbered from downstream to upstream in increments of 10 within each group. This allows the possibility of inserting additional nodes at a later time if needed yet still be able to conform to the naming scheme. (e.g., N-A055 can be inserted between nodes N-A050 and N-A060.)

Sub-Basins: Consist of a "B-" followed by the node name where the sub-basin enters the model. If there is more than one sub-basin entering at the same node location, then the additional sub-basins have the digit increased by one (e.g., sub-basins B-D130, B-D131, and B-D132 all enter at node N-D130).

Links: Consist of an "L-" followed by the group name ID, the upstream node number, and the link type and number. The link types are either a "P" for pipe, a "W" for weir, a "C" for channel, a "D" for drop inlet, or a "B" for bridge. For example, if two different size pipes and one weir leave node "N-C060", then the three links would be named L-C060P1, L-C060P2, and L-C060W1.

Cross Section Templates: Consist of an "X-" followed by the group name ID, the upstream node number, another dash (-), and the number. For example, if there are two different cross sections below node N-D130 that are going to be used in link L-D130-C1, then the cross section templates would be named X-D130-1 and X-D130-2.

Table 4-1. Church Creek Watershed ICPR Model Information

Name ID	ICPR Group ID	NUMBER OF LINKS					NUMBER STORAGE NODES
		CHANNEL	PIPE	BRIDGE	DROP STRUCTURE	WEIR	
A	RR	8	12	1	0	4	1
B	HH	15	2	1	4	7	4
C	SM1	16	17	0	0	15	5
D	SM2	11	12	0	0	14	10
E	VG	2	20	0	3	7	26
F	MC	0	7	0	1	0	8
G	BL	5	14	0	1	10	11
	TOTAL	**57**	**84**	**2**	**9**	**57**	**65**

Figure 4-1 ICPR network

SECTION 5 MODEL CALIBRATION/VERIFICATION

5.1 CALIBRATION/VERIFICATION

The ICPR model was calibrated and verified using four different sources of information:

5.1.1 Previous FIS Study

The Flood Insurance Rate Map (FIRM) for the effective FIS was obtained for the Church Creek Watershed. The effective FIRM map (455412 0005D) shows portions of the Church Creek Watershed as either ZONE A2, A5, or A13. These zones have a WSEL of 13 ft for the area downstream of Ashley River Road (SC-61), a WSEL of 11 ft for the area between the railroad and Ashley River Road, and a WSEL of 8 ft upstream of the railroad. These FIS elevations are for storm surge and could not be used to calibrate of this riverine model.

5.1.2 Previous Drainage Models

Storm drainage studies for Village Green, Moss Creek and Bees Landing completed by Seamon, Whiteside & Associates contained ICPR models. Data from these three models were incorporated into the ICPR model for the entire watershed. Model results in these areas from the watershed ICPR model were compared to the results of the individual ICPR models to verify that the models were behaving similarly. The boundary conditions of the individual models varied somewhat from the tailwater conditions that were produced in the watershed model. After accounting for these boundary condition differences, the watershed model produced similar results as the individual models.

5.1.3 USGS Regression Equation

Peak flows were computed based on the USGS regression equations (USGS Report 92-4040 and USGS Report 91-4157) using the lower coastal plain as the hydrologic area. Impervious areas for each sub-basin were estimated based on the land use and by applying the percent impervious values for each land use as reported in TR-55. Table 5-1 compares the ICPR model results with the calculated USGS regression equations for both the rural and urban equations. The locations of these comparison points are shown on Figure 5-1. The urban regression equation is not valid for percent impervious of less than 10 percent. Most of these drainage areas have seven to 18 percent impervious areas, therefore the true regression discharge will fall somewhere between the calculated urban and rural discharges. The ICPR model results showed discharges that were very similar to the rural regression discharges.

Table 5-1. Comparison of Flows (ICPR, and USGS Regression)

#	LOCATION DESCRIPTION	SUB-BASINS ABOVE	AREA (sq mi)	TOTAL IMP. AREA (%)	ICPR 2-yr (cfs)	USGS Rural 2-yr (cfs)	USGS Urban 2-yr (cfs)	ICPR 10-yr (cfs)	USGS Rural 10-yr (cfs)	USGS Urban 10-yr (cfs)
1	Above Village Green	N-E260	1.17	0.0	118	62	---	233	173	---
2	Below Village Green	N-E010	2.01	4.6	160	86	---	338	236	---
3	Dunwoody	N-D130	2.35	7.0	126	95	106	287	259	262
4	Hickory Farms	N-B160	3.63	13.0	302	126	318	507	336	711
5	Bees Ferry – Mainstem	N-B010	4.14	13.2	327	137	358	544	363	794
6	Railroad	B-A110	6.22	12.0	438	177	436	693	461	957
7	SC-61	N-A030	8.47	18.4	550	215	934	874	553	1914
8	Bees Ferry – Location 2	N-G010	1.37	5.1	89	68	---	134	189	---
9	Railroad – Location 2	N-A140	2.07	9.8	123	88	146	181	241	348
10	Hickory Hills	N-B180	1.39	23.2	192	68	316	273	190	690
11	Chippers Pitch and Putt	N-C010	0.58	23.4	84	39	165	100	113	375
12	Shadowmoss Pond #6	N-D020	0.26	24.2	69	24	95	92	71	223

#	LOCATION DESCRIPTION	SUB-BASINS ABOVE	AREA (sq mi)	TOTAL IMP. AREA (%)	ICPR 25-yr (cfs)	USGS Rural 25-yr (cfs)	USGS Urban 25-yr (cfs)	ICPR 100-yr (cfs)	USGS Rural 100-yr (cfs)	USGS Urban 100-yr (cfs)
1	Above Village Green	N-E260	1.17	0.0	297	243	---	442	368	---
2	Below Village Green	N-E010	2.01	4.6	444	333	---	695	502	---
3	Dunwoody	N-D130	2.35	7.0	376	365	369	613	550	573
4	Hickory Farms	N-B160	3.63	13.0	612	473	950	907	708	1365
5	Bees Ferry – Mainstem	N-B010	4.14	13.2	643	511	1057	951	764	1513
6	Railroad	B-A110	6.22	12.0	819	649	1274	1090	966	1824
7	SC-61	N-A030	8.47	18.4	1055	779	2457	1389	1156	3333
8	Bees Ferry – Location 2	N-G010	1.37	5.1	155	266	---	196	402	300
9	Railroad – Location 2	N-A140	2.07	9.8	209	339	480	276	511	721
10	Hickory Hills	N-B180	1.39	23.2	315	268	896	392	405	1236
11	Chippers Pitch and Putt	N-C010	0.58	23.4	102	160	494	109	243	695
12	Shadowmoss Pond #6	N-D020	0.26	24.2	104	100	297	166	154	424

Figure 5-1 regression locations

5.1.4 Historical Structure Flooding

There are several houses and townhouses within the Church Creek Watershed that have flooded on more than one occasion in the past 10 years. Flooding information for these buildings were collected at a Public Meeting held for the residents. Additional information was gathered by interviews that were conducted with some of the residents during the field reconnaissance. The finish floor elevations of these buildings, which are located in three primary locations, were surveyed and used to verify the model results. Figure 5-2 shows the location of these structures.

- Townhouses in Shadowmoss located near node N-D030 have been inundated with one to three inches of water on at least two occasions. The finish floor elevations range from 8.87 ft to 9.19 ft.
- Several houses in the Shadowood neighborhood located near node N-C060 have been inundated with three to 12 inches of water on at least four occasions. One house is reported to have had 18 inches of water during one of those occasions. The four lowest finish floor elevations among these houses are 9.64 ft, 10.02 ft, 10.05 ft and 10.07 ft.
- A house located on Winners Circle near node N-B060 has had water come up to within an inch below the finish floor elevation of 8.55 ft but has not been inundated.

Historical rainfall data for the four largest storm events from the past 10 years were used to calibrate the model results at these three locations. One of the historical rainfall events had a total rainfall depth greater than the 100-year event and showed unreasonable flooding results. The three other storm events had more reasonable results with rainfall depths between the 2- and 10-year events. These results showed slightly higher elevations at the house on Winners Circle (downstream most point), similar elevations near the townhouses (upstream point), and lower elevations near Shadowood where the majority of the flooding occurs.

In order for the model to give the same results as the historical flooding, the elevational difference between the townhouses and the downstream location should be around six inches, while the difference between the Shadowood houses and the downstream location should be around two feet. Several runs were made with some of the model parameters adjusted (i.e., curve numbers, time of concentrations, Manning's n values, antecedent runoff condition, etc.) to try and achieve closer model results in these three locations. Any parameter changes that increased the water surface elevation near Shadowood would also increase the elevation at the downstream location. In order to produce similar flooding in the Shadowood location, parameters had to be adjusted to unreasonable values and in return would produce unreasonable water surface elevations downstream. Therefore, it is assumed that other factors not included in the ICPR model contributed to the flooding in the Shadowood area.

Near the end of this project, a large storm event occurred on July 27, 2001 and flooded several of the houses in the Shadowood area. Rainfall for this storm event was also run in the model and the results did not produce the magnitude of flooding as reported by the residents.

Figure 5–2 house locations

SECTION 6 STORMWATER MANAGEMENT ALTERNATIVES

6.1 SUMMARY OF EXISTING FLOODING

The existing condition ICPR model was used to analyze the performance of the drainage system elements in their current configuration. The model predicts the occurrence of flooding at numerous locations throughout the watershed in response to significant rainfall events. Structures that may potentially be at risk of flood damage in the 100-year flood event were identified using the elevations from the ICPR model. This was done by locating all building footprints that were in or bordering the 100-year floodplain and structures that were identified by the City as having flooding complaints. A drive by survey was performed at each of these structures to determine if the structure was truly at risk. This was done by estimating the depth from the finish floor to the ground around the house and comparing it to the depth of flooding as determined by the ICPR model. Surveyors were then dispatched to obtain accurate finished floor and foundation elevations for 44 houses and six townhouse buildings, containing a total of 32 units, that were determined to be at risk of flooding. This information enabled detailed analysis of flooding impacts on these structures. From this detailed analysis, the depths of flooding under existing conditions were determined for the 2-, 10-, 25-, 50-, 100-, and 500-year flood events. Table 6-1 below provides an overall summary of the results of our analysis of 76 flood prone structures in the watershed. These results are based on existing land use conditions. The model results showed that two houses have finish floor flooding in the 10-year storm event while 23 houses and 32 townhouses have finish floor flooding in the 100-year storm event. The structures surveyed are located in three areas of the watershed; 25 structures are in the Shadowood neighborhood, 19 structures are in the Hickory Farms neighborhood, and the six townhouse buildings are located in Shadowmoss. A detailed list of the flooding at each surveyed structure is provided in APPENDIX C. Maps showing the existing flooding conditions are in Appendix F.

Table 6-1. Summary of Building Flooding

Storm Event	Houses	Townhouse Units
2	0	0
10	2	0
25	8	22
50	15	32
100	23	32
500	24	32
Not flooded	20	0
Total	44	32

6.2 BENEFIT/COST ANALYSIS

Mitigation measures for the three problem areas were identified that would likely be technically feasible, cost effective, and accepted by the local community. These alternatives were focused only on modifications to the City's drainage infrastructure and included such options as culvert improvements, channel improvements, pump stations and temporary flood storage. Buyout of several of the more frequently flooded structures was also considered.

Acceptable alternatives were conceptually designed and inserted into the ICPR model and re-run to determine the impacts on the flooding conditions. If necessary, alternatives were modified or refined to optimize their performance, as predicted by the models. Once a final version of the alternative was modeled and flood improvements were determined, then the flood elevation information resulting from that alternative was analyzed for its reduction in the flood damage to affected structures. For those mitigation alternatives that produced a sufficient benefit, the next step was to develop a cost estimate for construction of the alternative(s). This was done by 1) identifying the items required for construction, 2) determining unit prices, 3) determining quantities of materials, and 4) calculating construction costs.

The next step in the analysis process was to determine the economic impact of flooding. These analyses were conducted by calculating the approximate expected annual damages to these structures by using elevation-frequency and depth-damage relations developed by the Federal Emergency Management Agency (FEMA) and a modification of FEMA's QuattroPro Spreadsheet program Benefit-Cost Analysis of Hazard Mitigation Projects (1996). The Benefit/Cost (B/C) Analysis Methodology consists of using the known flood elevations and depth of flooding in each flood analyzed at each impacted structure and calculating an annual damage cost figure. Annual impacts to individual structures were totaled and converted to a present worth value, based on a useful life of 40 years and an interest rate of 7%. Structure tax values were obtained from the County of Charleston Property Information System.

In general this B/C method requires that each mitigation alternative to solve a storm water problem be analyzed individually. First, a cost of implementing the alternative is developed, then the present worth of annual damages without the mitigation alternative is determined, followed by the present worth of annual damages with the mitigation alternative in place. The difference between these two present worth values is the benefit derived by implementing of the mitigation alternative and this is divided by the cost developed above to determine the B/C ratio. Note that the benefits must be calculated individually for each structure that is helped by the mitigation alternative and then the benefits are summed to determine the total benefit.

This B/C analysis is intended to determine to a rough degree of accuracy, the ratio of dollar value of benefits to the dollar value of costs for a proposed project. The purpose of this analysis is to enable a manager to make an informed decision regarding the relative priority of a particular project as compared to other similar projects. Projects with higher B/C ratios likely justify a higher priority ranking than those with lower ratios. A project with a B/C ratio less than one is not economically feasible based on its merits alone. However, the B/C analysis is only one factor (albeit a major factor) in deciding how to prioritize projects. Other factors enter into this decision that do not lend themselves to economic analysis. These factors may include:

- Direct public endangerment,
- Restriction of emergency access,
- Impact on street system(s), or
- Public inconvenience.

6.3 FLOOD MITIGATION ALTERNATIVES

Once the existing flooding conditions were identified, a total of nine alternatives were investigated to address flooding in the three problem locations. The first level of investigation was to alter the ICPR model in a coarse fashion to see if removing or adding a culvert, enlarging a channel, or provide significant runoff storage would beneficially reduce flooding. If the coarse changes in the models produced a beneficial change in the flooding impacts, then the alternative was pursued to the next level of detail.

If the coarse analysis indicated beneficial results, the alternative was then held for more detailed analysis to determine if it produced a significant reduction in flooding damages as calculated in the benefit portion of the B/C analysis. If significant benefits were achieved, a cost was determined for the alternative and the B/C analysis was performed. Table 6.2 lists those alternatives that were analyzed and their B/C ratio. Table 6.3 lists the number of houses and townhouse units that are inundated during each storm event. Following these tables is a more detailed description of the alternatives. Figures 6-1, 6-2, 6-3, 6-4, 6-5, 6-6, 6-7, 6-8, 6-9 show the location of each of the alternatives. Information used in the B/C calculations (i.e., structures, tax values, costs, benefits, etc.) are provided in Appendix C.

Table 6-2. Summary of Analyzed Alternatives

Alternative Number	Alternative Description	Benefit/Cost Ratio
1	New pipes at primary crossing under Railroad	0.002
2A	New pipes and ditch from Shadowood to Railroad	0.563
2B	New pipes and ditch from Shadowood to Railroad and new culverts under the Railroad	1.182
2C	New ditch along Bees Ferry Road to Railroad and new culverts under the Railroad	1.638
3	Part of Shadowmoss diverted to drain directly to the Ashley River	1.126
4	Drainage from Village Green and above diverted to drain directly to the Ashley River	0.287
5	Channel improvements from Dunwoody to Hickory Farms	0.908
6	Drainage above Village Green diverted to drain directly to the Ashley River	0.288
7	Buyout of frequently flooded structures in Shadowood	0.177

Table 6-3. Number of Structures Inundated (December 2000 Landuse Conditions)

Alternative Number	Houses					Townhouse Units				
	2-Yr	10-Yr	25-Yr	50-Yr	100-Yr	2-Yr	10-Yr	25-Yr	50-Yr	100-Yr
Existing	0	2	8	15	23	0	0	22	32	32
1	0	2	8	15	23	0	0	22	32	32
2A	0	2	6	14	23	0	0	10	32	32
2B	0	0	3	8	18	0	0	10	32	32
2C	0	0	1	2	5	0	0	0	32	32
3	0	0	1	1	2	0	0	0	32	32
4	0	1	7	14	23	0	0	0	0	10
5	0	1	8	15	23	0	0	10	32	32
6	0	1	8	15	23	0	0	0	22	32
7	0	1	1	3	6	0	0	22	32	32

Alternative #1

Location: Main railroad crossing

Description: Add two additional 72-inch diameter steel pipes

Results: This improvement provides more flow area under the culvert but has little to no effect on the water surface elevations upstream of the railroad. The railroad culverts are controlled by the water surface elevation on the downstream side of the railroad.

Present Value Benefit: $722
Cost Estimate: $302,850
Benefit/Cost Ratio: 0.002

Alternative #2A

Location: Shadowood neighborhood

Description: Create a new ditch that drains from the existing 36 inch culvert under Bees Ferry Road to a culvert under the railroad approximately 800 feet away. This would also require adding additional pipes down Wolk Drive to tie into the culvert under Bees Ferry Road, and cleaning out the existing culvert that is almost completely full of sediment. Some improvements to the channel downstream of the railroad may also be required.

Results: This improvement provides very little benefit to the structures in the Shadowood neighborhood. The water surface elevations are reduced by about 0.1ft in the 10-year storm event to about 0.2 ft in the 100-year event.

Present Value Benefit: $120,580
Cost Estimate: $214,021
Benefit/Cost Ratio: 0.563

Alternative #2B

Location: Shadowood neighborhood

Description: Create a new ditch that drains from Bees Ferry Road to the railroad approximately 800 feet away. Install two new 60-inch steel pipes under the railroad at a lower invert than the existing culvert. Replace the existing culvert under Bees Ferry Road with two 36-inch pipes at a lower invert than the existing pipe. This would also require adding pipes down Wolk Drive to tie into the culverts under Bees Ferry Road. Some improvements to the channel downstream of the railroad may also be required.

Results: This improvement provides some benefit to the structures in the Shadowood neighborhood. The water surface elevations are reduced by about 0.75 ft in the 2-year storm event to about 0.4 ft in the 100-year event.

Present Value Benefit: $501,199
Cost Estimate: $424,065
Benefit/Cost Ratio: 1.182

Alternative #2C

Location: Along Bees Ferry Road near the Shadowood neighborhood

Description: Plug the two existing 48-inch pipes that cross under Bees Ferry Road. Create a new ditch that runs along Bees Ferry Road for approximately 1,300 ft and then turns toward the railroad approximately 800 feet away. Install two new 60-inch steel pipes under the railroad at a lower invert than the existing culvert. Some improvements to the channel downstream of the railroad may also be required.

Results: This improvement provides the most benefit to the structures in the Shadowood neighborhood. The water upstream of Shadowood is diverted away from the neighborhood. The water surface elevations are reduced by about 0.8 ft in the 2-year storm event to about 1.0 ft in the 100-year event.

Present Value Benefit: $917,924
Cost Estimate: $560,490
Benefit/Cost Ratio: 1.638

Alternative #3

Location: Shadowmoss near Ashley River Road and Bees Ferry Road

Description: Block the existing weir that is just upstream of the two existing 48-inch pipes that cross under Bees Ferry Road and block the outfall at the 5th fairway. Divert the outfall for the golf course pond near the 6th and 8th holes to outfall directly to the Ashley River. This new outfall will run from Hunters Forest Drive towards Ashley River Road and then towards the Ashley River. This will consist of two new 48-inch pipes beginning at the pond and running approximately 850 feet under Hunters Forest Drive and under Ashley River Road. A new ditch will be created that continues approximately 1,300 ft until it reaches the Ashley River.

Results: This improvement provides benefit to the structures in the Shadowood neighborhood and to several houses within Shadowmoss. The water surface elevations in Shadowood are reduced by about 1.1 ft in the 2-year storm event to about 1.3 ft in the 100-year event.

Present Value Benefit: $1,130,700
Cost Estimate: $1,004,010
Benefit/Cost Ratio: 1.126

Alternative #4

Location: Shadowmoss and Village Green

Description: Divert all the storm water coming to a point just downstream of Village Green directly to the Ashley River. This would consist of several new ditches, pipes and a pump station running between the houses located in Shadowmoss and Village Green.

Results: This improvement provides benefits to many downstream structures including the Shadowood neighborhood. However, this alternative is costly and would require numerous drainage easements.

Present Value Benefit: $1,150,796
Cost Estimate: $4,013,430
Benefit/Cost Ratio: 0.287

Alternative #5

Location: Channel between Shadowmoss and Moss Creek

Description: Increase the size of the channel to provide more flood storage. This would consist of channel improvements for approximately 2,500 feet. The top width of the channel would be increased from the existing average top width of 20 feet to a new top width of approximately 50 feet.

Results: This improvement provides a small benefit to the structures located on Two Loch Place. The water surface elevations are reduced by about 0.2 ft for each of the storm events.

Present Value Benefit: $275,990
Cost Estimate: $303,825
Benefit/Cost Ratio: 0.908

Alternative #6

Location: Village Green

Description: Divert all the storm water coming to a point just upstream of Village Green directly to the Ashley River. This would consist of several new ditches and pipes running just north of Village Green.

Results: This improvement provides some benefit to downstream structures located on Two Loch Place. The water surface elevations are reduced by about 0.2 ft for each of the storm events.

Present Value Benefit: $506,917
Cost Estimate: $1,763,130
Benefit/Cost Ratio: 0.288

Alternative #7

Location: Shadowood neighborhood

Description: The City would buy a section of houses along Wolk Drive and Mowler Drive that borders Bees Ferry Road. This would include the houses that have experienced reoccurring flooding over the last 10 years and some other houses along these streets. This land could then be filled and used by the City (i.e., fire or police station).

Results: This improvement removes a majority of the structures completely from the floodplain that have had repeated flooding over the past 10 years. This alternative does not provide a large financial benefit; however there may be several intangible items that a price cannot be assigned to (i.e., public endangerment, or emergency vehicle access).

Present Value Benefit: $436,478
Cost Estimate: $2,464,200
Benefit/Cost Ratio: 0.177

Figure 6-1

Figure 6-2

Figure 6-3

Figure 6-4

Figure 6-5

Figure 6-6

Figure 6-7

Figure 6-8

Figure 6-9

6.4 PROPOSED ALTERNATIVES

The results of the B/C analysis for reducing building flooding had three positive alternatives. However, all three of those alternatives (#2B, #2C, and #3) provide relief to the Shadowood neighborhood. Alternative 2C provides the largest B/C ratio and is the recommended alternative to reduce flooding in the Shadowood neighborhood. Alternative #5 is the only other alternative that has close to a positive B/C ratio. This alternative is to increase the available channel storage between Dunwoody and Hickory Farms. Table 6-4 lists the prioritized alternatives by B/C ratio and recommends alternatives for the City to consider. The recommended alternatives #2C and #5 have estimated costs of $560,490 and $303,825 respectively. The combined cost for both Alternative #2C and #5 is $864,315. The itemized cost estimates are provided in Appendix C.

Table 6-4. Summary of Recommended Alternatives

Alternative Number	Alternative Description	Benefit/Cost Ratio	Recommended
2C	New ditch along Bees Ferry Road to Railroad and new culverts under the Railroad	1.638	Yes
2B	New pipes and ditch from Shadowood to Railroad and new culverts under the Railroad	1.182	No
3	Part of Shadowmoss diverted to drain directly to the Ashley River	1.126	No
5	Channel improvements from Dunwoody to Hickory Farms	0.908	Yes
2A	New pipes and ditch from Shadowood to Railroad	0.563	No
6	Drainage above Village Green diverted to drain directly to the Ashley River	0.288	No
4	Drainage from Village Green and above diverted to drain directly to the Ashley River	0.287	No
7	Buyout of frequently flooded structures in Shadowood	0.177	No
1	Primary crossing under Railroad	0.002	No

SECTION 7 STORMWATER DETENTION REQUIREMENTS

7.1 CURRENT REGULATIONS

The current detention regulations used by the City of Charleston are those required by the State of South Carolina. These regulations are listed in Section 72-307 and Appendix B of the South Carolina Stormwater Management and Sediment Control Handbook for Land Disturbance Activities (September 1995). The major requirement as pertaining to storm water detention quantity control is that the post-development peak discharge rates shall not exceed pre-development discharge rates for the 2- and 10-year frequency 24-hour duration storm event. This requirement only controls the peak rate at which storm water can leave a site and does not consider the volume of water, or the timing of hydrographs at downstream locations. There is also no control requirement for the larger storm events.

The ICPR model was used to determine what effects controlling only the peak rates might have on hydrograph timing and water surface elevations within the watershed. The sub-basins upstream of Bees Ferry Road were modified to reflect future development land use conditions. These sub-basins were also set so that the peak runoff rates were limited to the existing conditions peak flow rates. This is an option within of ICPR that can be used to modify runoff hydrographs to simulate peak rate controls. ICPR does this by creating a hydrograph that limits the peak rate to a set discharge limit. Any discharge from the runoff hydrograph that is larger than the set discharge limit is set aside until the runoff discharge rate drops below the set limit. At this time, the saved discharge is added back to the hydrograph at a rate that still maintains the set discharge limit. Therefore, any discharge rate greater than a set limit is saved and added to the back end of the runoff hydrograph once the rates drop below the set discharge limit.

The 2- through 100-year storm events were modeled with the peak rate controls in place. The model results showed that there is one additional house that might have finish floor flooding in the 10-, 25- and 100-year storm events while there are three additional houses that may have finish floor flooding in the 50-year storm event. Therefore, the current detention requirement of only controlling peak discharge rates does not protect downstream locations from increased flooding due to new development. Table 7-1 summarizes the flooding impacts with only peak rate controls.

Table 7-1. Flooding Impacts with Only Peak Rate Controls

Policy Modification Alternative	Number of Finish Floors Inundated Per Condition				
	2-year	10-year	25-year	50-year	100-year
Houses					
Existing Conditions	0	2	8	15	23
Future Conditions with Detention That Controls the Peak Rate	0	3	9	18	24
Townhouse Units					
Existing Conditions	0	0	22	32	32
Future Conditions with Detention That Controls the Peak Rate	0	10	32	32	32

7.2 DETENTION REQUIREMENT OPTIONS

Due to the extent of the existing flooding and the potential for future flooding in the watershed, a change in policy and requirements may be a solution to the problem. There were six possible policy modification alternatives investigated. Descriptions of these alternatives are listed below while Table 7-2 shows a comparison of the pros and cons for each alternative.

1) No detention required,
2) Control peak flow rates only,
3) Detain the excess 24-hour, X-year storm rainfall runoff at the peak detention elevation,
4) Detain the excess 24-hour, X-year storm rainfall runoff until Z-time,
5) Detain the excess 24-hour, X-year storm rainfall runoff at the peak detention elevation and control peak discharge rates, and
6) Detain the excess 24-hour, X-year storm rainfall runoff until Z-time and control peak discharge rates.

X-year = given storm frequency (i.e., 2-year, 10-year, 100-year)
Z-time = given time (i.e., 24-hours)

Policy modification alternative #1, No detention required;

- This alternative would not require future development to provide detention, allowing direct release of all runoff.

Policy modification alternative #2, Control peak flow rates only (Current Policy):

- This alternative would implement the current policy of requiring detention facilities to detain runoff and release the post-development peak flow rates for the 2- and 10-year 24-hour storm events to the pre-development peak flow rates. However, the design storm event could be changed to a less frequent storm event (i.e., 25-year, 50-year, or 100-year) to address future storm water quantity problems. See Figure 7-1.

Policy modification alternative #3, Detain the excess 24-hour, X-year storm rainfall runoff at the peak detention elevation,

- This alternative would require detaining the excess runoff volume difference between the pre-development and post-development conditions for a given storm frequency X (i.e., 100-year storm event). This excess volume would occupy the peak storage volume in the detention facility. See Figure 7-2.

Policy modification alternative #4, Detain the excess 24-hour, X-year storm rainfall runoff until Z-time:

- This alternative would require detaining the excess runoff volume difference between the pre-development and post-development conditions for a given storm frequency X (i.e., 100-year storm event) for a certain time period Z (i.e., 24-hours). The storage volume within the detention facility would be required to occupy the excess runoff volume and the volume required to detain this excess volume for the desired time period. See Figure 7-3.

Policy modification alternative #5, Detain the excess 24-hour, X-year storm rainfall runoff at the peak detention elevation and control peak discharge rates:

- This alternative would require detaining the excess runoff volume difference between the pre-development and post-development conditions for a given storm frequency X (i.e., 100-year storm event) and release the post-development peak flow rates for the X-year storm event to the pre-development peak flow rates. The storage volume within the detention facility would be required to occupy the excess runoff volume and the volume required to release the post-development peak flow to the pre-development peak flow rates. See Figure 7-4.

Policy modification alternative #6, Detain the excess 24-hour, X-year storm rainfall runoff until Z-time and control peak discharge rates:

- This alternative would require detaining the excess runoff volume difference between the pre-development and post-development conditions for a given storm frequency X (i.e., 100-year storm event) for a certain time period Z (i.e., 24-hours). The storage volume within the detention facility would be required to occupy the excess runoff volume, the volume required to detain this excess volume for the desired time period and the volume required to release the post-development peak flow to the pre-development peak flow rates. See Figure 7-5.

Table 7-2. Alternative Pros and Cons

Policy Option	Pros	Cons
1	Easiest approach	Results in increased downstream volume, increased flow elevations and increased peak discharges.
2	Current practice, easy understanding for design community	Results in increased downstream volume, and increased flow elevations.
3	Excess runoff volume created from development is captured	Post-peak flow rates could be larger than the pre-rates (excess volume could be captured before peak flow is reached, excess volume may be less than required volume to control peak). Larger post-runoff volume could travel downstream sooner than pre-runoff volume.
4	More than excess runoff volume is captured at peak detention elevation (excess volume + drawdown volume)	Post-peak flow rate could be larger than the pre-rates (excess volume could be captured before peak flow is reached, excess volume may be less than required volume to control peak).
5	Excess volume is captured Peak discharge is controlled	Larger post- runoff volume could travel downstream sooner than pre- runoff volume (post- shape of hydrograph may have centroid sooner). If drawdown time is large, detention facilities could stay full for long periods of time.
6	Same Z-hour volume is released for pre- and post- conditions, and the post- peak flow rates will be equal to or lower than the pre- peak flow rates	Requires the most detention volume of the six options. Detention facilities will stay full for longer periods of time due to smaller outlet control devices.

Figure 7-1 Alternative #2 - Control Peak Flow Rates

Pros: Current Practice
Cons: Increased volume downstream,
Increased elevation downstream,
Increased discharge downstream

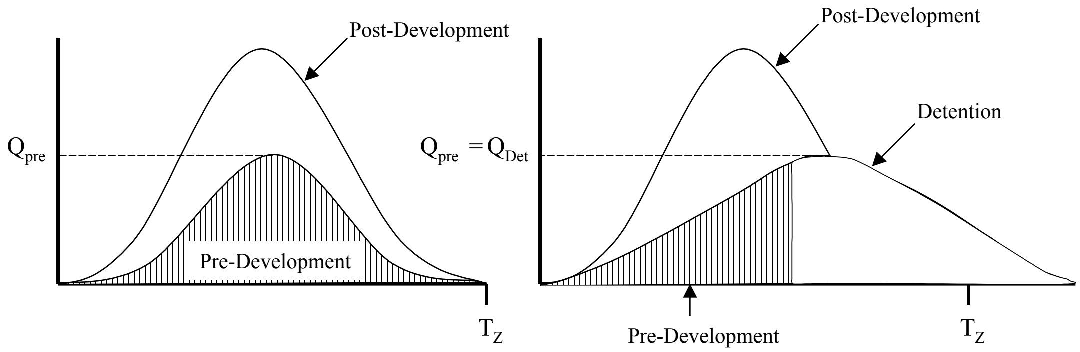
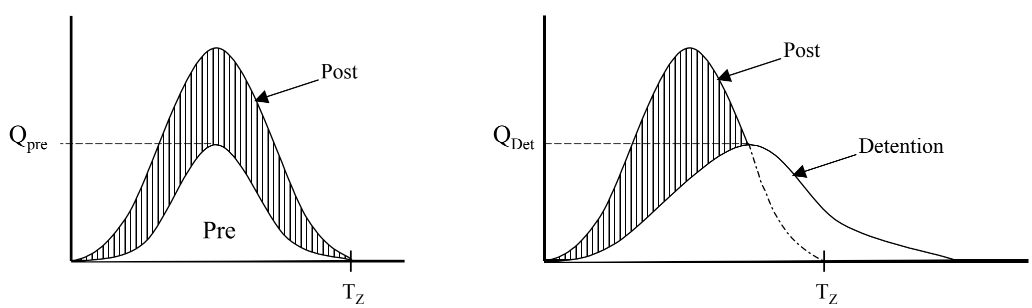


Figure 7-2 Alternative #3 - Detain the excess 24-hour, X-year storm rainfall runoff at the peak detention elevation

Pros: Excess volume is captured
Cons: Peak rates could be larger than existing
*(excess volume is captured before peak)
Larger volume could travel downstream sooner than existing
*(shape of hydrograph may have centroid sooner than existing conditions)

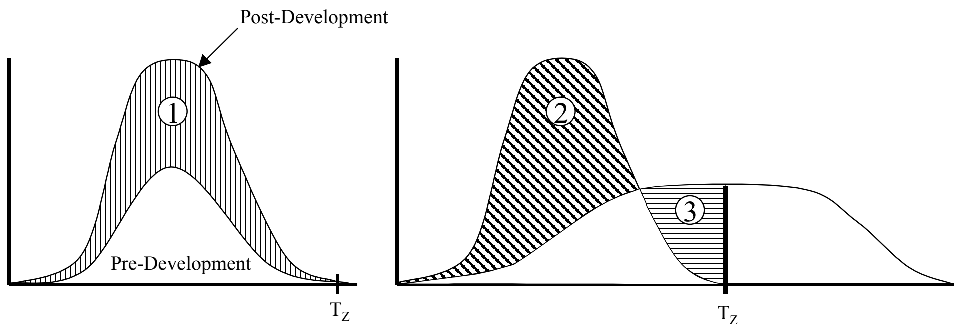


$$\mathbf{A_{Excess} = A_{Storage}}$$

Figure 7-3 Alternative #4 - Detain excess 24-hour X-year rainfall until Z-time

Pros: At peak detention elevation there is more than excess runoff volume
(excess volume + drawdown)

Cons: Peak rates could be larger than existing
*(excess volume is captured before peak)
Excess volume may be less than volume required to control peak



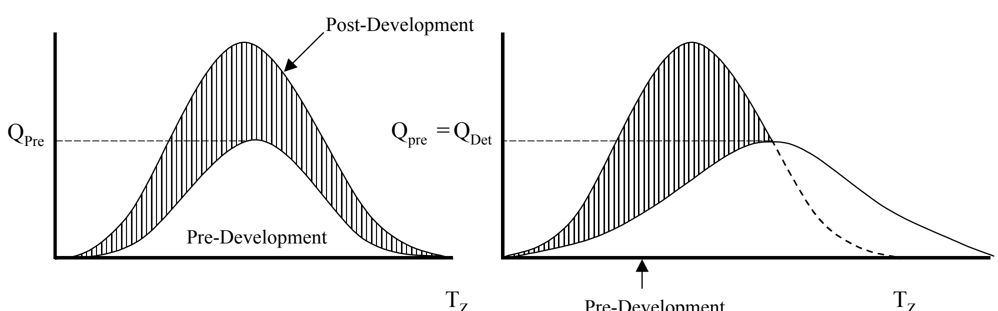
$$A_1 = A_2 - A_3$$

$$A_{Excess} + A_{Drawdown} = A_{Storage}$$

Figure 7-4 Alternative #5 - Detain excess 24-hour X-year at peak detention elevation and control peak discharge

Pros: Excess runoff volume is captured
Peak discharge is controlled

Cons: Larger volume could travel downstream sooner than existing
* (Shape of hydrograph may have centroid sooner)
If drawdown time is large, pond stays full

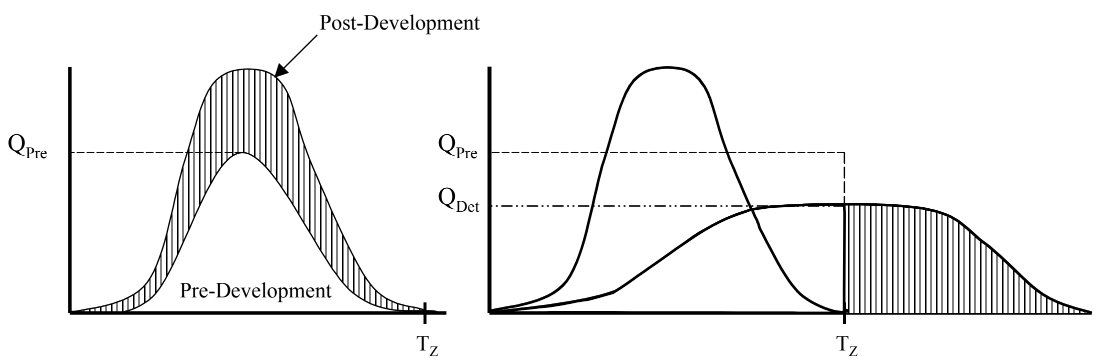


$$Q_{Det} = Q_{Pre}$$

$$A_{Storage} \langle \quad A_{Excess}$$

Figure 7-5 Alternative #6 Detain excess 24-hour X-year until Z-time and control peak discharge

Pros: About the same X-hour volume is released for pre- and post- conditions
Peak discharge will be lower than or equal to existing peak flows
Cons: Requires more detention volume
Ponds may stay full longer due to small outlet devices



Using the ICPR computer model, different detention policy options were applied to future land use conditions for sub-basins located upstream of Bees Ferry Road to determine the resulting impacts on future flood elevations. Changes that were made to the ICPR model to simulate these control options are as follows:

- **Peak Rate Controls**: The future runoff hydrographs were limited to the existing conditions peak rates for each of the storm events.
- **Volume Controls**: The increased volume of runoff from each sub-basin was determined for each storm event. Storage was added to the sub-basin node to account for this volume increase for each storm event. This was done by providing additional storage volume equal to the given year storm event volume increase below the existing water surface elevation for that given year storm event. (i.e., the increase in volume for the 2-year event was created below the existing 2-year water surface elevation for that node).
- **Volume Time Controls**: The volume of runoff at 24-hours was determined for each storm event under existing conditions. The future runoff hydrographs were limited to a discharge rate that would produce the same volume at a time period of 24-hours. A holding period of 24-hours was used due to the fact that the time to peak in the vicinity of the railroad was approximately 25 hours.

Results from the modeled detention options are shown in Table 7-3.

Table 7-3. Policy Modification Alternatives and Future Flooding Impacts

Policy Modification Alternative	Number of Finish Floors Inundated Per Condition				
	2-year	10-year	25-year	50-year	100-year
Houses					
Existing Conditions	0	2	8	15	23
Alt #1 –No Controls	0	4	9	19	24
Alt #2 –Peak Controls	0	3	9	18	24
Alt #3 –Volume Controls	0	2	9	17	24
Alt #6 –Peak and Volume Time Control	0	2 or less	8	15	23
Townhouse Units					
Existing Conditions	0	0	22	32	32
Alt #1 –No Controls	0	22	32	32	32
Alt #2 –Peak Controls	0	10	32	32	32
Alt #3 –Volume Controls	0	4	32	32	32
Alt #6 –Peak and Volume Time Control	0	0	22 or less	32	32

7.3 PROPOSED DETENTION REQUIREMENTS

Based on the results of the computer model simulations it is recommended that detention policy alternative number six be implemented for future development. This alternative was selected because it provides the most protection against flooding for the future land use conditions as shown in Table 7-3. This alternative gives developers the freedom to develop at any impervious density while maintaining no flooding impacts to downstream properties.

Figures 7-6, 7-7, 7-8, and 7-9 show existing stage hydrographs at several locations upstream of the railroad. The peak stages at these locations occur between hours 21 to 25 depending on the location and remain near peak stage for approximately three to six hours. Therefore, it is recommended that the time period for pre-volume release control be set to 24-hours. This should prevent any excess runoff volume due to new development from traveling downstream until after the peak stage at the railroad has begun to reside. It is also recommended that all storm events up to the 100-year storm event should be controlled for both excess volume and peak rates.

Therefore, the recommended detention standard shall require permanent storm water management systems, associated with new development, to be designed and constructed to maintain the post-development peak flow rates at or below the pre-development peak flow rates; and to detain the excess runoff volume difference between the pre-development and post-development conditions for the design storms having a duration of 24-hours and frequencies of 2-, 10-, 25-, 50- and 100- years for a time period of 24-hours. Tolerances for the 25-, and 50- year storm event peak flow rates will be plus or minus 10 percent. All other post-development peak flow rates must be at or below the pre-development peak flow rates. Detention facilities meeting these standards must be designed and constructed to contain the excess volume for the 24-hour period and the volume required to release the post development peak flow at or below the pre-development peak flow rates.

Figure 7-6

Figure 7-7

Figure 7-8

Figure 7-9

SECTION 8 CHURCH CREEK ICPR MODEL RESULTS

8.1 HYDROLOGIC RESULTS

8.2 HYDRAULIC RESULTS

Appendix A -- Bibliography

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South Carolina Stormwater Management and Sediment Control Handbook for Land Disturbance Activities, South Carolina DEHC, September 1995.

Appendix B -- ICPR Input Files (Existing Condition)

Hard Copy Files

Appendix C – Calculations and Additional Data

Structure Flooding Results

Benefit to Cost Output

Construction Cost Estimates

Rainfall Data

Tidal Information

Appendix D -- Digital Files

Digital Files of Models and Reports on CD:

```
CD [Church Creek]
  |
  --------  ICPR
  |            |-----  ChrchEx           (Existing land use files – final version)
  |            |-----  ChrchE2           (Existing land use files used for comparison with
  |            |                         Alternative and Detention analysis)
  |            |-----  Alt1              (Alternative 1 files)
  |            |-----  Alt2              (Alternative 2 files)
  |            |-----  Alt2b             (Alternative 2b files)
  |            |-----  Alt2c             (Alternative 2c files)
  |            |-----  Alt3              (Alternative 3 files)
  |            |-----  Alt4              (Alternative 4 files)
  |            |-----  Alt5              (Alternative 5 files)
  |            |-----  Alt6              (Alternative 6 files)
  |            |-----  FU_BFRY           (Future land use above BeesFerry)
  |            |-----  FU_BF_V           (Future land use above BeesFerry with
  |            |                          existing volume control)
  |            |-----  Pea100y           (100 year Future land use above Bees Ferry with)
  |            |                          existing peak control)
  |            |-----  Peak25y           (25 year Future land use above Bees Ferry with)
  |            |                          existing peak control)
  |            |-----  Peak10y           (10 year Future land use above Bees Ferry with)
  |            |                          existing peak control)
  |            |-----  Peak2y            (2 year Future land use above Bees Ferry with)
  |            |                          existing peak control)
  |            |-----  PV_100y           (100 year Future land use above Bees Ferry with)
  |            |                          existing peak and volume control)
  |            |-----  PV_25y            (25 year Future land use above Bees Ferry with)
  |            |                          existing peak and volume control)
  |            |-----  PV_10y            (10 year Future land use above Bees Ferry with)
  |            |                          existing peak and volume control)
  |            |-----  PV_2y             (2 year Future land use above Bees Ferry with)
  |                                       existing peak and volume control)
  --------  PICTURES
  |            |-----???.JPG             Pictures
  |
  --------  REPORT
  |            |-----  *.DOC             MSWord97 files
  |            |-----  *.XLS             MSExcel97 files
  |
  --------  BC_RATIO
  |            |-----  *.XLS             MSExcel97 files
  |
  --------  GIS
               |-----  *.*               ArcView shape files
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Appendix E – ICPR Network

Appendix F – Floodplain Boundary Map