Church Creek Stormwater Master Plan

Summary Report
(Volume 1)

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1. EXECUTIVE SUMMARY

Background and Scope

The City of Charleston decided to develop a mater plan for the Church Creek watershed. This effort was initiated through homeowner complaints of reoccurring flooding within the basin. The Master Plan was to address existing flooding problems, review current detention policies and recommend any modification to the detention policy required to eliminate future impacts.

Technical Approach

Hydrologic and hydraulic analyses were accomplished using the ICPR software program. Data for the model came from a variety of sources including current soils maps, topographic maps, drainage system maps, and a field reconnaissance of the basin. Hydraulic analyses of the natural stream system were based on DTM cross-sections and field surveys.

Existing Conditions

The existing landuse conditions within the watershed, as of December 2000, were used for the analysis of existing flooding conditions. Areas of known flooding were evaluated such as the Shadowood and Hickory Farms neighborhoods. Existing conditions were derived from mapping and field evaluations. The existing conditions will also be used to update the FIS FEMA maps.

Alternatives Analysis

Flood reduction alternatives were developed to help alleviate reoccurring flooding problems within the watershed. Cost analysis of alternatives were developed and compared to the benefits to determine a benefit to cost ratio. Three basic approaches to flood control were analyzed for the Church Creek Basin.

1. Hydraulic Improvements: This option included enlarging or adding additional culverts to provide more flow area to existing drainage structures.

2. Channel Improvements This option included enlarging existing channel segments or creating new channel/pipe systems to divert the storm water to a new outfall location.

3. Property Buyout This option includes purchasing homes that are within the floodplain and have experienced reoccurring flooding.

Detention Analysis

Future landuse conditions were based on the proposed zoning for the City of Charleston and adjacent existing land use. Future development conditions were used to predict where flooding may occur with the current detention requirements in place. Several modified detention policies were applied to the future landuse to determine which requirements provided the best flood control.

2. SCOPE

Project Scope

The *Church Creek Basin Analysis,* is a 2 Volume storm water management master plan. This is a detailed watershed analysis and master plan that addresses problems which are now occurring in the system and recommends improvements designed to remain stable as areas develop in accordance with current zoning. This report, which is Volume 1, summaries the key points of the analysis and is directed towards the City decision makers. Volume 2 is a technical report and attempts to be very concise. The technical report is directed to the technical professional familiar with the concepts and procedures of stormwater system analysis.

This report is divided into 6 sections as follows:

1. Executive Summary:	Provides a managerial overview of the work.
2. Scope:	Objectives.
3. Technical Approach:	Names methodologies used in this study, describing basic assumptions and limitations.
4. Existing Conditions:	Discusses hydrologic and hydraulic conditions as they currently exist.
5. Alternatives Analysis	Describes alternatives to reduce existing flooding problems.
6. Detention Analysis:	Describes detention policy modifications required to alleviate flooding impacts due to future development.

3. Technical Approach

Project Team

The project team for this study included Flint Holbrook, project leader; Steve Godfrey, JP Johns and Gil Inouye. The project manager for the City was Laura Cabiness.

Hydrologic and Hydraulic Model

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.

Hydrologic Data

Basic hydrologic inputs were developed in accordance with the USDA, SCS publication "Technical Release No. 55, Urban Hydrology for Small Watersheds," Second Edition, June 1986.

The Church Creek drainage basin and sub-basins were delineated using GIS and verified during field visits performed in November 2000. Two engineers examined the basin, photographed significant hydraulic structures, mapped drainage boundaries, and recorded new land use changes.

Existing land use information was determined using current City zoning, GIS data of the watershed and field observations of new development during a reconnaissance of the basin. The existing land use for this study consisted of December 2000 conditions.

Soils were taken from the USDA, SCS "Soil Survey of Charleston County, South Carolina," March 1971.

Topographic data was taken from GIS coverages provided by the City.

Rainfall depths 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.

Hydraulic Data

Hydraulic data was developed from field reconnaissance and detailed technical surveys. Information relative to Manning's "N" value determination was developed from field observations. Channel cross-sections and significant structure elevations were measured by a survey crew. Elevations of driveways, houses, and other potentially flooded structures were taken as needed.

ICPR models from previous drainage studies for Village Green, Moss Creek and Bees Landing neighborhoods completed by Seamon, Whiteside & Associates were incorporated into the watershed model.

Storm Events

The 2-, 10-, 25-, 50-, 100-, and 500-year storm events were modeled. Historical rainfall data from three large storm events along with finish floor elevations of known flooded structures were used to calibrate/validate the model.

4. Existing Conditions

General Description

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. Figure 1 generally indicates basin vicinity.

Land Use

The upper portion of the watershed primarily consists of undeveloped land while the middle and lower portions are primarily residential with some commercial development along the major roadways. Figure 3 shows existing land uses.

Hydrologic and Hydraulic Overview

The watershed is divided into seven major groups with a total of 89 sub-basins for the hydrologic analysis. Figure 2 shows sub-basin delineations. Hydrologic parameters developed for each sub-basin are shown in Table 4, and the soil type / landuse / curve number relationships are shown in Table 3. Hydrologic parameters from the previous study's ICPR models were used except where noted.

24-hour rainfall depths used in this analysis are listed in Table 1 and the SCS Type III rainfall distribution is listed in Table 2.

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) that are considered to have a HSG classification of D for this study.

Current Flooding Concerns

Complaints of flooding have been noted in the three primary locations:

1. Shadowood Neighborhood
2. Townhouses on Two Loch Place
3. Hickory Farms Neighborhood

Surveyors were 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. 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. Table 5 provides an overall summary of the results of our analysis of 76 flood prone structures in the watershed. The floodplain boundaries are shown on Maps 1-4.

5. ALTERNATIVES ANALYSIS

General

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. The flood reduction alternatives discussed in this report have been developed for study purposes only. Actual implementation will require detailed design outside the scope of this work.

Benefit / Cost

Acceptable alternatives were conceptually designed and inserted into the ICPR model and re-run to determine the impacts on the flooding conditions. Construction cost estimates for each alternative were developed along with calculating the approximate expected annual damages to the impacted structures. This was accomplished 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 present worth value of the benefit is divided by the construction cost to determine the B/C ratio. 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. Projects with higher B/C ratios likely justify a higher priority ranking than those with lower ratios.

Alternatives

A total of nine alternatives were investigated to address flooding in the three problem locations. The alternatives analyzed are as follows:

- #1 - New pipes at primary crossing under Railroad
- #2A – New pipes and ditch from Shadowood to Railroad
- #2B – New pipes and ditch from Shadowood to Railroad and new culverts under the Railroad
- #2C – New ditch along Bees Ferry Road to Railroad and new culverts under the Railroad
- #3 - Part of Shadowmoss diverted to drain directly to the Ashley River
- #4 - Drainage from Village Green and above diverted to drain directly to the Ashley River
- #5 – Channel improvements from Dunwoody to Hickory Farms
- #6 – Drainage above Village Green diverted to drain directly to the Ashley River
- #7 – Buyout of frequently flooded structures in Shadowood

The location of the alternatives are shown in Figures 5 through 13 and the Benefit / Cost results are listed in Table 6. There were three alternatives that had positive B/C ratio, 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. 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.

6. DETENTION ANALYSIS

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.

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 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, using the current detention requirement of only controlling peak discharge rates within the Church Creek Watershed does not protect downstream locations from increased flooding due to new development.

Detention Options

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

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)

Recommendation

The ICPR computer model was modified with different detention policy options and applied to future land use conditions for sub-basins located upstream of Bees Ferry Road to determine the resulting impacts on future flood elevations. 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 8. This alternative gives developers the freedom to develop at any impervious density while maintaining no flooding impacts to downstream properties.

It is recommended that the time period for pre-volume release control (Z-time) be set to 24-hours. The peak stages at locations upstream of the railroad occurs between hours 21 to 25 depending on the location and remain near peak stage for approximately three to six hours. This time requirement 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.

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Tables

Table 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 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							

Table 3. 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 4

Table 5. Summary of Current Building Flooding (December 2000 Conditions)

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

Table 6. Summary of Flood Reduction 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

Table 7. Detention 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.

** See page 6-1 for option descriptions.*

Table 8. Future Flooding Impacts from Modeled Detention Alternatives

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

Figures

Maps