Appendix 8
Interior Drainage Analysis – Algiers and English Turn Districts of Orleans Parish West Bank

Introduction

Study Purpose

To answer the questions regarding the performance of the hurricane protection system (HPS), the interior drainage analysis focused on the filling and unwatering of the separate areas protected by levees and pump stations, referred to as basins. Interior drainage models were developed for Jefferson, Orleans, St. Bernard and Plaquemines Parishes to simulate water levels for what happened during Hurricane Katrina and what would have happened had all the hurricane protection facilities remained intact and functioned as intended.

The primary components of the hurricane protection system are the levees and floodwalls designed and constructed by the US Army Corps of Engineers (USACE). Other drainage and flood control features (land topography, streets, culverts, bridges, storm sewers, roadside ditches, canals, and pump stations) work in concert with the USACE levees and floodwalls as an integral part of the overall drainage and flood damage reduction system and are included in the models.

Interior drainage models are needed for estimating water elevations inside leveed areas, or basins, for a catastrophic condition such as Hurricane Katrina and for understanding the relationship between HPS components. Results from the interior drainage models can be used to determine the extent, depth and duration of flooding for multiple failure and non-failure scenarios. The models can also be used to:

- Support the risk modeling effort
- Estimate time needed to unwater an area
- Support evacuation planning
- Evaluate design options of the HPS to include multiple interior drainage scenarios

This appendix will provide details of the development of the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS) models for the Algiers and English Turn districts of the West Bank of Orleans Parish. In summary, an HEC-HMS model was developed to transform the Katrina precipitation into runoff for input to the HEC-RAS models. HEC-RAS models were developed to simulate the two conditions discussed below.

This model was developed to help answer questions 3 and 4 listed on page 1 of Volume VI. Question 3 is answered by the Katrina simulation listed below. Question 4 is a more difficult one to answer. This is mainly due to the variety of possible combinations of system features, especially pumps. It was decided to bracket these combinations with the hypothetical combinations listed below.

One of the major difficulties is determining what pumps may have continuing operating. There are many potential factors that can cause pump stations to not operate during a hurricane event. Some of these are power failures, pump equipment failures, clogged pump intakes, flooding of the pump equipment, loss of municipal water supply used to cool pump equipment and no safe housing for operators at the pump stations resulting in pump abandonment. At both Algiers and English Turn, the pump operator logs are relatively complete during the event, providing a good understanding of actual operations. For the purposes of the IPET analysis, it was decided to operate the pumps two ways. (1) As they actually operated during hurricane Katrina and (2) As if the pumps operated throughout the hurricane.

Described below are the 5 scenarios generally applied in the Interior Drainage and Pumping Technical Appendices. For Algiers and English Turn in Orleans West Bank, only hypothetical 2, resilient pump stations applies, as no overtopping or breaching of floodwalls or levees occurred in these districts.

Katrina

Simulated what happened during Hurricane Katrina with the hurricane protection facilities and pump stations performing as actually occurred. Compared results to observed and measured high water marks. Pre-Katrina elevations were used for top of floodwalls and levees.

Hypothetical 1 – Resilient Levees and Floodwalls

Simulate what would have happened during Hurricane Katrina had all levees and floodwalls remained intact. There are no levee or floodwalls breaches or failures for this scenario even where overtopping occurs. Pump stations operate as they did in the Katrina event. Pre-Katrina elevations are used for top of floodwalls and levees. This scenario is meant to simulate what could have happened if all levees and floodwall had erosion protection that would allow them to be overtopped but not breached. For Orleans West Bank, since there were no levee or floodwall breaches, the results of this scenario are the same as the Katrina scenario.

Hypothetical 2 – Resilient Floodwalls, Levees and Pump Stations

Simulated what would have happened during Hurricane Katrina had all levees, floodwalls and pump stations remained intact and operating. This scenario was simulated to provide an upper limit on what could have been the best possible scenario had no failures occurred. There were no levee or floodwall breaches or failures in Orleans West Bank. However, power outages did limit pumping at both Algiers and English Turn.

Hypothetical 3 – Resilient Floodwalls

Simulate what would have happened during Hurricane Katrina had all floodwalls, which failed from foundation failures, remained intact. All other areas are modeled as they actually functioned. Pump station operate as they did in the Katrina event. Pre-Katrina elevations used for top of floodwalls and levees. Since there were no levee or floodwall breaches in Orleans West Bank, the results of this scenario are the same as the Katrina Scenario.

Hypothetical 4 – Resilient Floodwalls and Levees at Authorized Design Grade

Simulate what would have happened during Hurricane Katrina had all levees and floodwalls remained intact and the crest of all levees and floodwalls were at the authorized design grade elevation. There are no levee or floodwall breaches or failures for this scenario even where overtopping occurs. Pump stations operate as they did during the Katrina event.

Table 8-1 lists the simulation scenarios in a matrix format.

Table 8-1
Katrina Scenarios

	Katrina	Hypothetical 1	Hypothetical 2	Hypothetical 3	Hypothetical 4
Pumps operate as during Katrina	X	X		X	X
Pumps operate throughout Katrina			X		
Levee and floodwall breaches occur everywhere as during Katrina	X				
Levee and floodwall breaches occur on West wall of IHNC and in, St Bernard, New Orleans East and Plaquemines as during Katrina				X	
Levee and floodwalls overtop but do not breach		X	X		X
No failures on 17th Street and London Ave		X	X	X	X
Levee and floodwall elevations based on pre-Katrina elevations	X	X	X	X	
Levees and Floodwalls elevations are at authorized elevation					X

Review of Existing Data

Prior to Hurricane Katrina, a steady state HEC-RAS model existed for a portion of Algiers. This model was constructed to analyze culvert improvements along Victory Park Canal in the median of General DeGaulle Drive and only included enough detail for those ends. No hydraulic model existed for English Turn. LIDAR data flown in 2004 was available for both Algiers and English Turn. Water conditions in the canals during the LIDAR flights were at normal operating levels. The operation logs at both pump station 13 (Algiers) and 11 (English Turn) provided stage observations at both the intake and outlet sides of the stations as well as a log of pump operations during the event.

General Modeling Approach

The unsteady flow HEC-RAS program developed by the USACE Hydrologic Engineering Center (HEC) was used to develop the hydraulic model for Orleans West Bank. The modeling approach was to identify storage areas bounded by ridges or elevated roads and then calculate flow between these storage areas. The Orleans West Bank HEC-RAS model consists of 109 storage areas connected by storm drains, open channels and overtopping ridges. The rainfall-runoff relationship for each storage area was developed using HEC-HMS. This model produced flow hydrographs for each of the storage areas from gridded precipitation data captured during the Katrina event. These modeled outflows from each storage area defined the inflow into the HEC-RAS model. Pump station discharges were simulated in the model to account for expulsion of water from Algiers and English Turn into the Gulf Intracoastal Waterway (GIWW). In this manner all of the storage areas were interconnected and populated with inflow data appropriate to the Katrina event. The network was provided with expulsion rules and operations as they occurred to model the event. Hypothetical conditions can be considered by modifying any of these factors.

Hydrologic Model Development

Background

HEC-HMS version 3.0.0 was used to model the rainfall-runoff response for the Hurricane Katrina event for sub-basins in Orleans West Bank. Sub-basin boundaries in the HEC-HMS model correspond to storage areas defined in the HEC-RAS model. Rainfall for each sub-basin was determined using radar-rainfall estimates from the National Weather Service. The Soil Conservation Service (SCS) curve number (CN) and the SCS dimensionless unit hydrograph methods were used to compute runoff hydrographs given basin average precipitation. Landuse and soil data were used to estimate SCS curve numbers and lag times.

Development of GIS Watershed Model

Sub-basin boundaries for the Orleans West Bank HEC-HMS model are shown in Figure 8-1. Basin boundaries correspond to storage areas defined in the HEC-RAS model for this area. Delineation of sub-basin boundaries is described in RAS Interior Modeling Section later in this appendix. A shapefile of sub-basin boundaries was generated and used for estimating HEC-HMS model parameters.

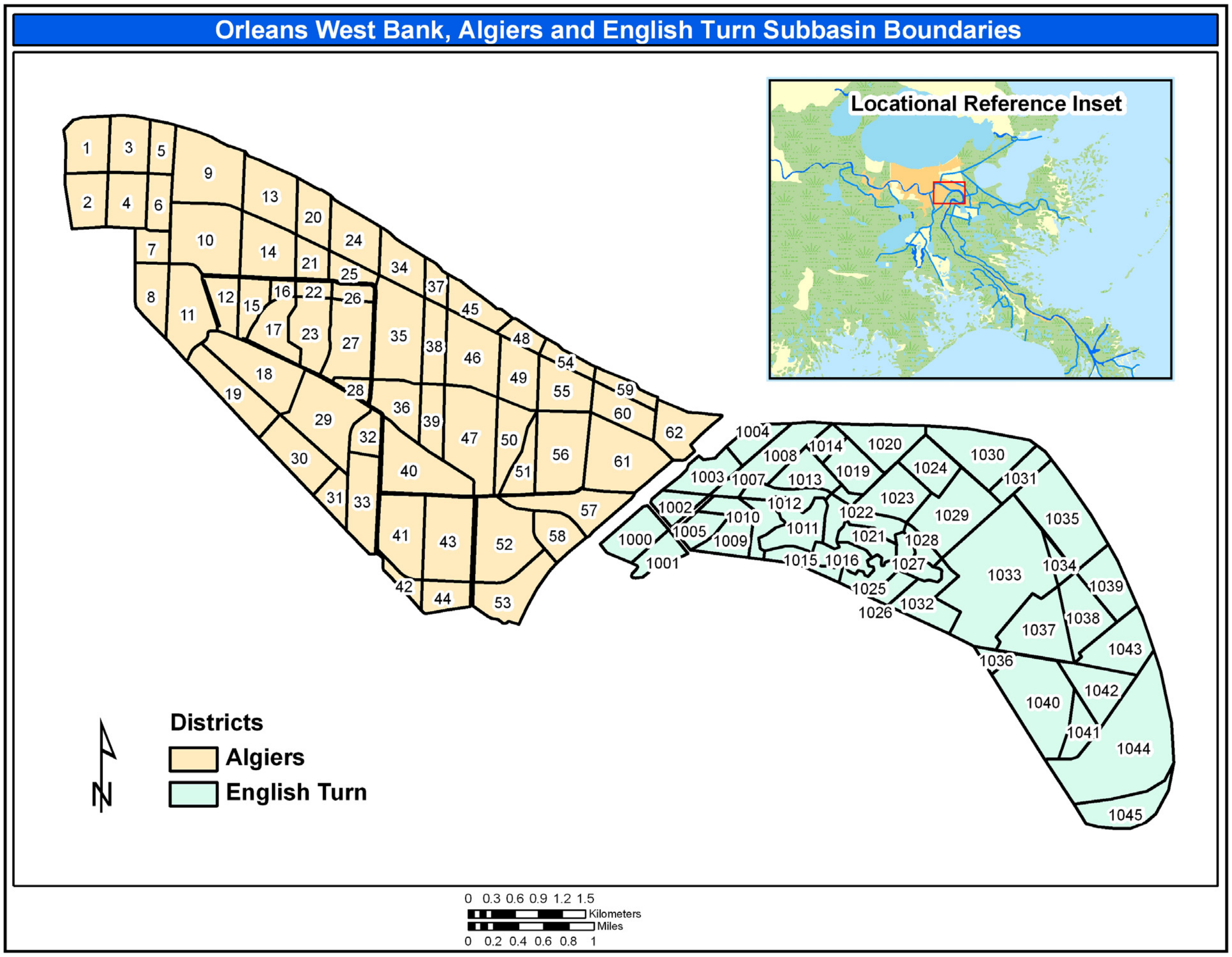

Figure 8-1. Orleans West Bank, Algiers and English Turn Sub-basin Boundaries

Model Parameters

Landuse and soil data. Landuse and soil data were used to estimate SCS curve numbers. Landuse data was obtained from the New Orleans District (MVN). The landuse data was a raster coverage of 24 different landuse types, 9 of which are represented in Orleans West Bank (Table 8-2). Soil data, contained in the Soil Survey Geographic (SSURGO) Database, was downloaded from the following National Resources Conservation Service (NRCS) website: http://www.ncgc.nrcs.usda.gov/products/datasets/ssurgo/. The SSURGO dataset is a digital copy of county soil survey maps and provides the most level of detail for digital soil maps from the NRCS.

Loss rates. Loss rates were computed by determining the amount of precipitation intercepted by the canopy and depressions on the land surface and the amount of precipitation that infiltrates into the soil. Precipitation that is not lost to interception or infiltration is called "excess precipitation" and becomes direct runoff. SCS CN method was used to model interception and infiltration. This method estimates precipitation loss and excess as a function of cumulative precipitation, soil cover, landuse, and antecedent moisture. This method uses a single parameter, a curve number, to estimate the amount of precipitation excess/loss from a storm event. Studies have been conducted to suggest appropriate curve number values for combinations of landuse type and condition, soil type, and the moisture state of the watershed. While these studies provide guidance on curve number selection, variability exists in each parameter and each watershed. Calibration of a hydrologic model may require some adjustment of the curve number in order to accurately depict the observed loss rate.

Table 8-2 was used to estimate an initial curve number value for each combination of landuse and soil type in the study area. The hydrologic soil group (A, B, C, or D) is one of the soil properties contained in the SSURGO database. The percent impervious cover for developed areas is already included in the curve number value in Table 8-2. More information about the background and use of the SCS curve number method can be found in SCS (1971, 1986). Figure 8-2 and Figure 8-3 show landuse types and hydrologic soil groups, respectively, in Orleans West Bank, Algiers and English Turn.

Table 8-2
Landuse Categories

LANDUSE	A	B	C	D
Fresh Marsh	39	61	74	80
Wetland Forest-Deciduous	43	65	76	82
Upland Forest-Deciduous	32	58	72	79
Upland Forest-Mixed	39	61	74	80
Agriculture-Cropland-Grassland	49	69	79	84
Vegetated Urban	49	69	79	84
Non-Vegetated Urban	71	80	87	91
Wetland Complex	85	85	85	85
Water	100	100	100	100

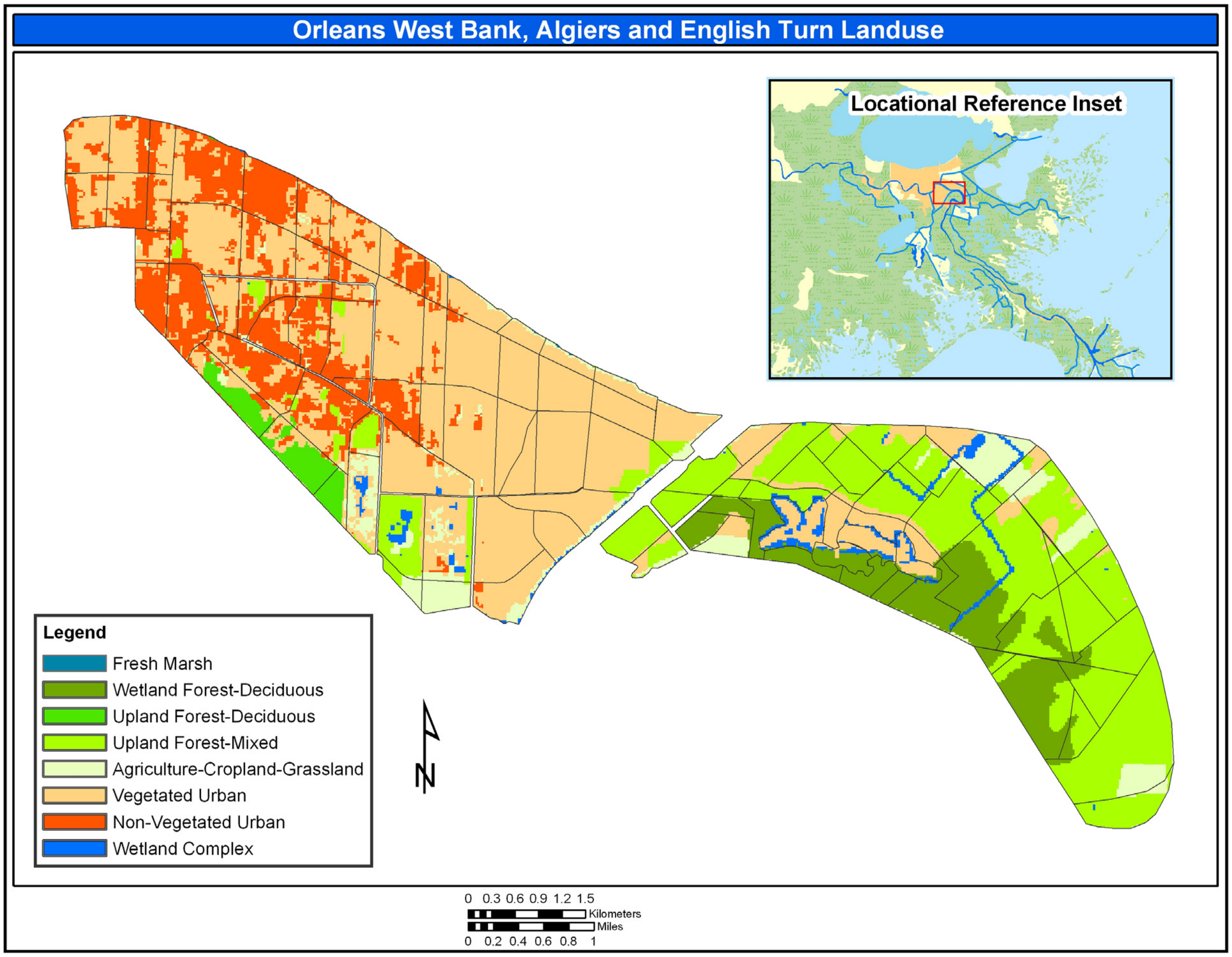

Figure 8-2. Landuse Types in Orleans West Bank, Algiers and English Turn

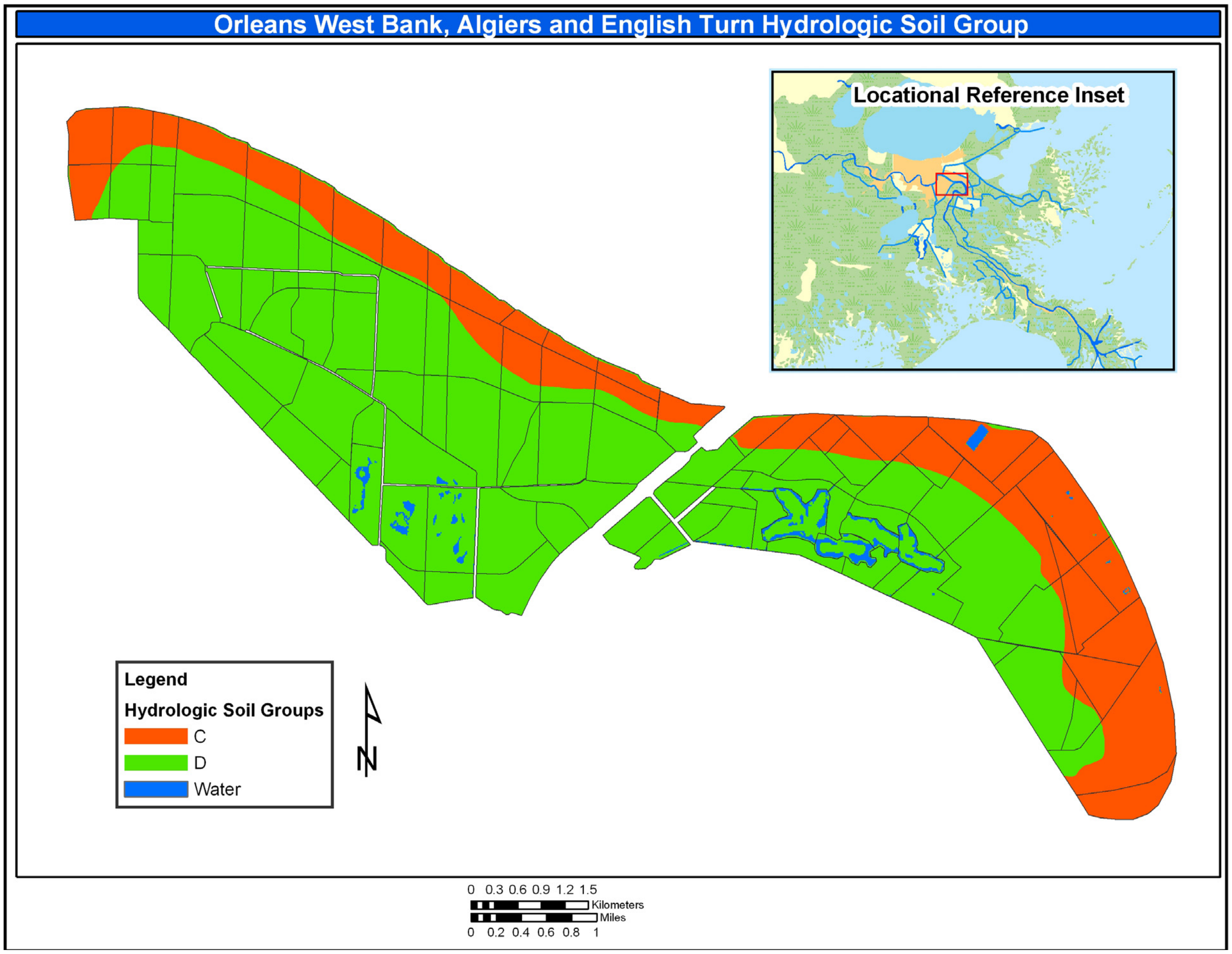

Figure 8-3. Hydrologic Soil Groups in Orleans West Bank, Algiers and English Turn

The ArcGIS map calculator was used to create a raster coverage of curve numbers from the two data sets and the curve number lookup table (Figure 8-4). Average curve numbers were computed for each sub-basin using the sub-basin boundary shapefile and the curve number raster coverage. During the course of calibration, it was determined that the English Turn district was experiencing inadequate hydrologic loss. The curve numbers for all English Turn sub-basins were decreased by 5 to model additional hydrologic loss and lag. Table 8-3 shows both the initial estimate for curve number as well as the final calibration values.

Transform

Excess precipitation was transformed to a runoff hydrograph using the SCS unit hydrograph method. The SCS developed a dimensionless unit hydrograph after analyzing unit hydrographs from a number of small, gaged watersheds. The dimensionless unit hydrograph is used to develop a unit hydrograph given drainage area and lag time. A detailed description of the SCS dimensionless unit hydrograph can be found in SCS Technical Report 55 (1986) and the National Engineering Handbook (1971).

Lag times for the SCS unit hydrograph method were estimated using the following equation:

$$t_l = \frac{L^{0.8} * (1000 - 9CN)^{0.7}}{1900 * CN^{0.7} * Y^{0.5}}$$

where t_l is the sub-basin lag (hr), L is the hydraulic length (ft), CN is the sub-basin average curve number, and Y is the average sub-basin land slope (percent). The hydraulic length was determined visually using topographic maps developed from lidar data of Orleans West Bank, Algiers and English Turn. Lidar data was also used to compute the average land slope for each sub-basin. Lag times computed using the formula above are shown in Table 8-3. The same formula is used for both the initial and calibration lag time calculation with only the curve number being altered.

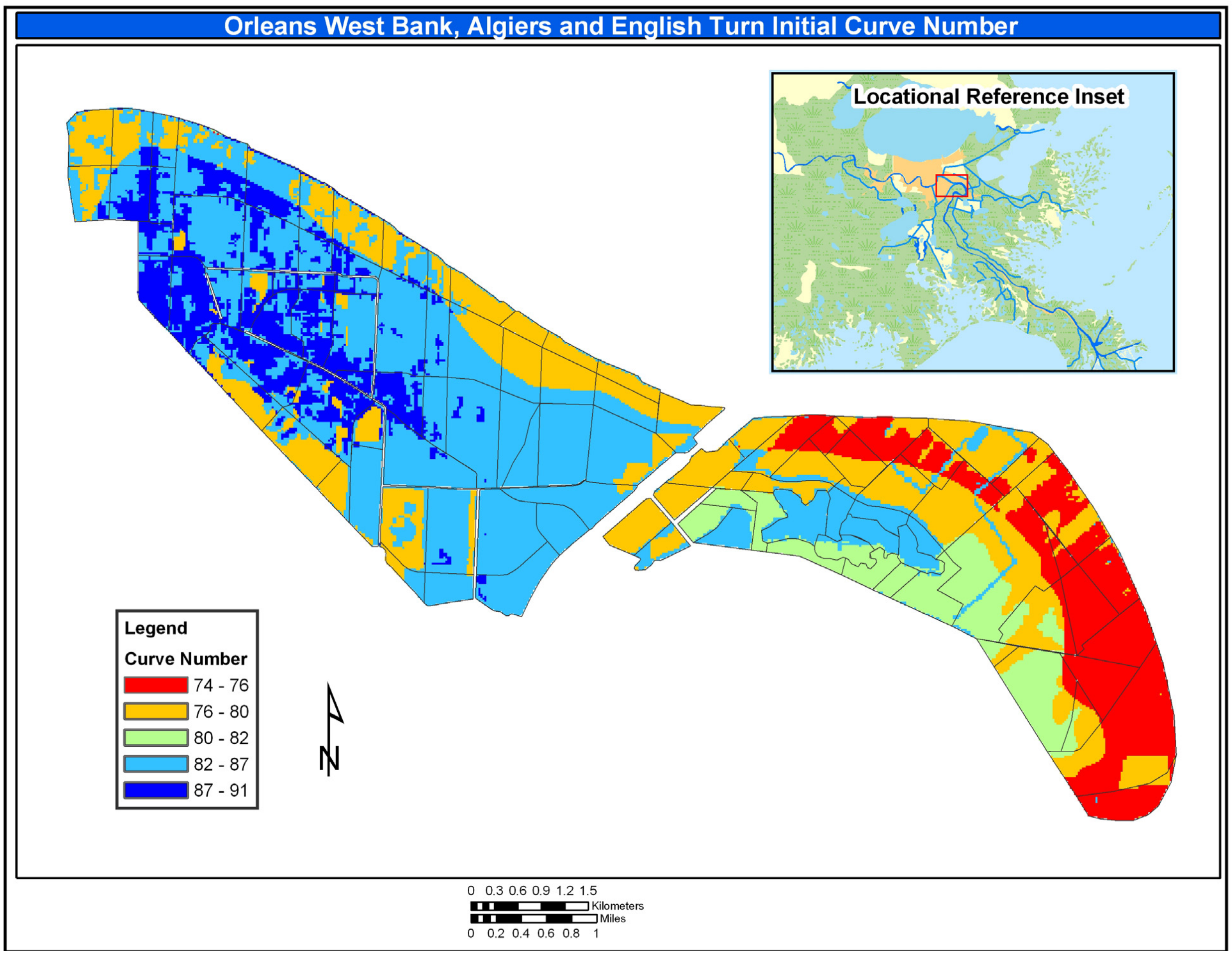

Figure 8-4. Grid of the Initial Estimate of Curve Number

Table 8-3
Sub-basin Hydrologic Parameters

Sub-basin Name	Sub-basin Area (mi^2)	Average Slope (%)	Flow Length (ft)	Initial Estimate: Average Curve Number	Initial Estimate: Lag Time (minutes)	Calibration Values: Average Curve Number	Calibration Values: Lag Time (minutes)
1	0.15	1.90	2761	81	30	same	same
2	0.14	1.53	2839	83	32	same	same
3	0.14	1.60	2395	83	28	same	same
4	0.14	1.71	2560	87	24	same	same
5	0.08	1.98	2450	84	24	same	same
6	0.09	1.42	2921	87	30	same	same
7	0.07	2.78	1523	87	12	same	same
8	0.12	1.98	3181	90	24	same	same
9	0.30	1.79	3344	87	30	same	same
10	0.30	1.52	4156	85	41	same	same
11	0.22	2.10	3724	90	27	same	same
12	0.09	1.79	2580	86	25	same	same
13	0.19	2.04	3701	88	29	same	same
14	0.19	1.54	2836	86	29	same	same
15	0.09	1.59	2780	86	28	same	same
16	0.03	1.97	928	87	10	same	same
17	0.13	1.77	3175	89	26	same	same
18	0.24	1.99	1941	88	17	same	same
19	0.18	1.79	1346	82	17	same	same
20	0.12	2.05	2466	83	25	same	same
21	0.08	1.41	1796	85	22	same	same
22	0.04	2.22	873	87	9	same	same
23	0.16	1.67	3601	89	30	same	same
24	0.16	2.50	2223	81	22	same	same
25	0.06	1.75	1202	87	13	same	same
26	0.04	2.74	746	86	7	same	same
27	0.22	1.80	3560	86	33	same	same
28	0.03	2.37	1018	89	9	same	same
29	0.28	1.56	3716	87	34	same	same
30	0.17	1.58	1308	81	18	same	same
31	0.07	1.96	1891	80	23	same	same
32	0.07	1.92	1140	83	14	same	same
33	0.17	2.45	1862	84	18	same	same
34	0.12	2.85	1969	84	17	same	same
35	0.29	2.71	3312	84	26	same	same
36	0.14	1.72	3502	89	29	same	same
37	0.05	2.93	1597	82	15	same	same
38	0.12	1.83	3538	85	33	same	same
39	0.10	1.71	3050	85	30	same	same
40	0.26	2.13	2583	85	24	same	same
41	0.22	2.37	2015	81	21	same	same
42	0.05	3.79	2544	82	19	same	same
43	0.26	2.64	2082	84	19	same	same
44	0.09	2.13	1369	84	15	same	same
45	0.11	3.26	1905	80	18	same	same

(continued)

Table 8-3 (Continued)

Sub-basin Name	Sub-basin Area (mi^2)	Average Slope (%)	Flow Length (ft)	Initial Estimate		Calibration Values	
				Average Curve Number	Lag Time (minutes)	Average Curve Number	Lag Time (minutes)
46	0.23	2.26	4254	82	37	same	same
47	0.30	1.93	4242	84	38	same	same
48	0.05	3.39	927	79	10	same	same
49	0.15	3.07	3859	81	31	same	same
50	0.14	2.23	2857	84	26	same	same
51	0.09	2.64	2563	84	22	same	same
52	0.28	2.42	4205	84	34	same	same
53	0.18	2.72	3791	84	29	same	same
54	0.05	3.91	1861	79	16	same	same
55	0.15	2.07	2511	81	27	same	same
56	0.26	1.94	4613	84	41	same	same
57	0.18	2.32	1989	84	19	same	same
58	0.11	2.24	1963	84	19	same	same
59	0.06	4.59	1251	80	11	same	same
60	0.13	2.19	1417	81	16	same	same
61	0.29	2.10	3999	83	36	same	same
62	0.14	1.69	3116	81	36	same	same
1000	0.12	2.39	3269	80	32	75	37
1001	0.08	3.03	2582	82	22	77	26
1002	0.07	2.59	1408	80	16	75	18
1003	0.12	2.45	1700	80	19	75	22
1004	0.08	3.50	2657	80	22	75	26
1005	0.07	1.84	2214	82	25	77	29
1006	0.05	1.84	1173	82	15	77	17
1007	0.03	2.41	1667	80	18	75	21
1008	0.15	1.87	3719	78	43	73	49
1009	0.09	1.96	1681	84	18	79	21
1010	0.07	2.43	1224	82	13	77	16
1011	0.15	4.84	995	84	8	79	9
1012	0.08	2.22	1502	81	17	76	20
1013	0.11	1.32	2612	80	36	75	41
1014	0.05	1.22	1561	75	29	70	33
1015	0.07	3.03	684	82	8	77	9
1016	0.04	3.40	473	83	5	78	6
1017	0.06	2.84	1106	84	11	79	13
1018	0.02	4.10	775	82	7	77	8
1019	0.12	1.76	1661	77	23	72	27
1020	0.15	1.61	2535	76	36	71	41
1021	0.08	4.21	1368	84	10	79	12
1022	0.06	1.83	1284	84	15	79	18
1023	0.16	1.50	3510	81	42	76	49
1024	0.13	1.34	1733	76	29	71	33
1025	0.07	2.28	1394	82	15	77	18
1026	0.05	2.73	1563	82	15	77	18
1027	0.07	5.12	877	84	7	79	8
1028	0.04	2.26	1394	84	15	79	17
1029	0.26	1.32	3870	79	51	74	59
1030	0.23	1.72	2782	80	33	75	38

(continued)

Table 8-3 (Concluded)							
Sub-basin Name	Sub-basin Area (mi^2)	Average Slope (%)	Flow Length (ft)	Initial Estimate		Calibration Values	
				Average Curve Number	Lag Time (minutes)	Average Curve Number	Lag Time (minutes)
1031	0.08	1.16	3239	78	49	73	56
1032	0.15	2.11	1842	82	20	77	23
1033	0.67	1.06	5317	80	70	75	82
1034	0.04	1.56	903	75	16	70	19
1035	0.30	1.43	2766	76	41	71	47
1036	0.03	2.29	1082	81	13	76	15
1037	0.23	1.15	2747	80	40	75	47
1038	0.17	1.13	2191	74	40	69	46
1039	0.15	1.64	1821	74	28	69	33
1040	0.33	1.33	3396	81	42	76	49
1041	0.09	1.14	2285	79	35	74	41
1042	0.15	1.53	3577	74	51	69	58
1043	0.19	1.26	3743	74	58	69	67
1044	0.61	1.51	6471	75	79	70	91
1045	0.16	2.43	3990	74	44	69	50
1046	0.04	3.40	473	83	5	78	6

Rainfall Data

Radar rainfall data, referred to as Multisensor Precipitation Estimator (MPE), was used as a boundary condition in the hydrologic models to determine runoff hydrographs produced by the Hurricane Katrina event. MPE data from the Lower Mississippi River Forecast Center (LMRFC) was downloaded from the following website: http://dipper.nws.noaa.gov/hdsb/data/nexrad/lmrfc_mpe.php. Raw radar data is adjusted using rain gage measurements and possibly satellite data to produce the MPE product.

The radar-rainfall data was imported into a GIS program as a series of gridded coverages of hourly precipitation. The GIS program was used to compute sub-basin average precipitation for each hour of the event. These hourly averages were combined to form the hyetographs for each sub-basin. The individual hyetographs were imported into an HEC-DSS file where they were read by HEC-HMS. Total rainfall from Hurricane Katrina, as estimated by MPE data and the GIS exercise described above, varied from 9.8 to 10.9 inches across the sub-basins in Orleans West Bank, Algiers and English Turn (Figure 8-5). The precipitation hyetograph for sub-basin 47 is shown in Figure 8-6. This figure shows the time distribution of rainfall from Hurricane Katrina.

During calibration it was determined that the volume of water entering the English Turn district exceeded the volume being pumped out. Hydrologic loss rates were increased to accommodate for some of the discrepancy, but additional measures were required to reasonably calibrate the model. The method described above for determining precipitation rates from radar provides the best available estimate of rainfall as well as the variability of precipitation across the affected region, but uncertainty exists. Operator notes at Orleans Pump Station #11 in

English Turn indicate less observed precipitation than estimated by the MPE at that location. Precipitation rates for the English Turn district were reduced by 10% for the calibration and hypothetical model runs.

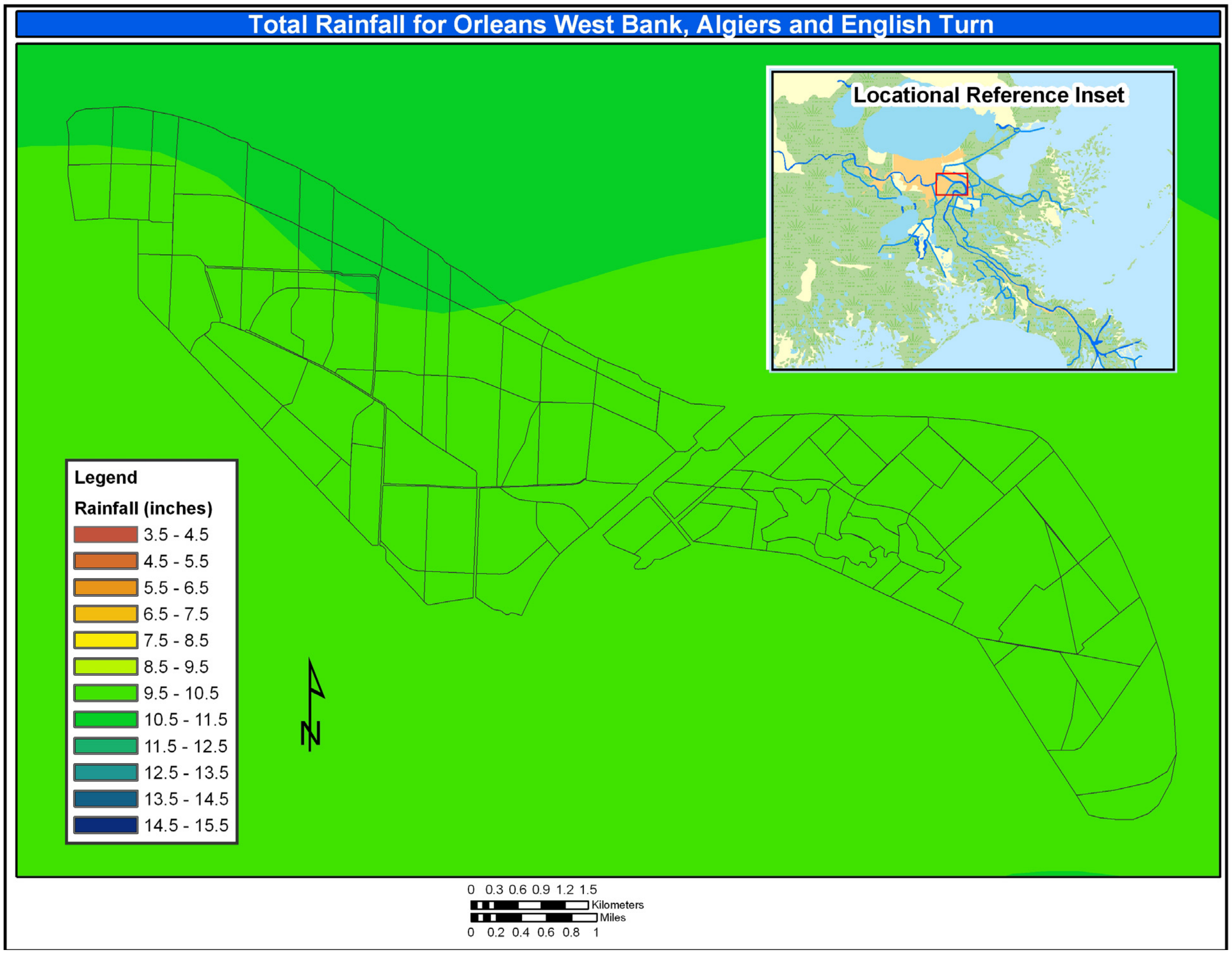

Figure 8-5. Total Storm Rainfall

The frequency of the 24-hour rainfall from Katrina varied across the New Orleans region. 24-hour precipitation values, shown in Table 8-4, ranged from 7 to 15 inches. Table 8-5 shows the range of rainfall frequencies over the area. Rainfall frequency values were estimated from charts in National Weather Service Technical Paper No. 40.

Table 8-4 **Katrina 24-Hour Precipitation Values**	
Area	Katrina 24-Hr Precipitation Range (inches)
Jefferson Parish	9 - 12
Orleans East Bank	10 - 15
Orleans West Bank	10 - 11
New Orleans East	8 - 14
St. Bernard	7 - 12
Plaquemines	7 - 10

Table 8-5 **TP-40 24-Hour Precipitation Frequency**	
Event	TP-40 24-Hr Precipitation Range (inches)
2-Year	5.5 – 6.0
5-Year	7.5 – 8.5
10-Year	9 – 10
25-Year	10 – 12
50-Year	11 – 13
100-Year	13 - 15

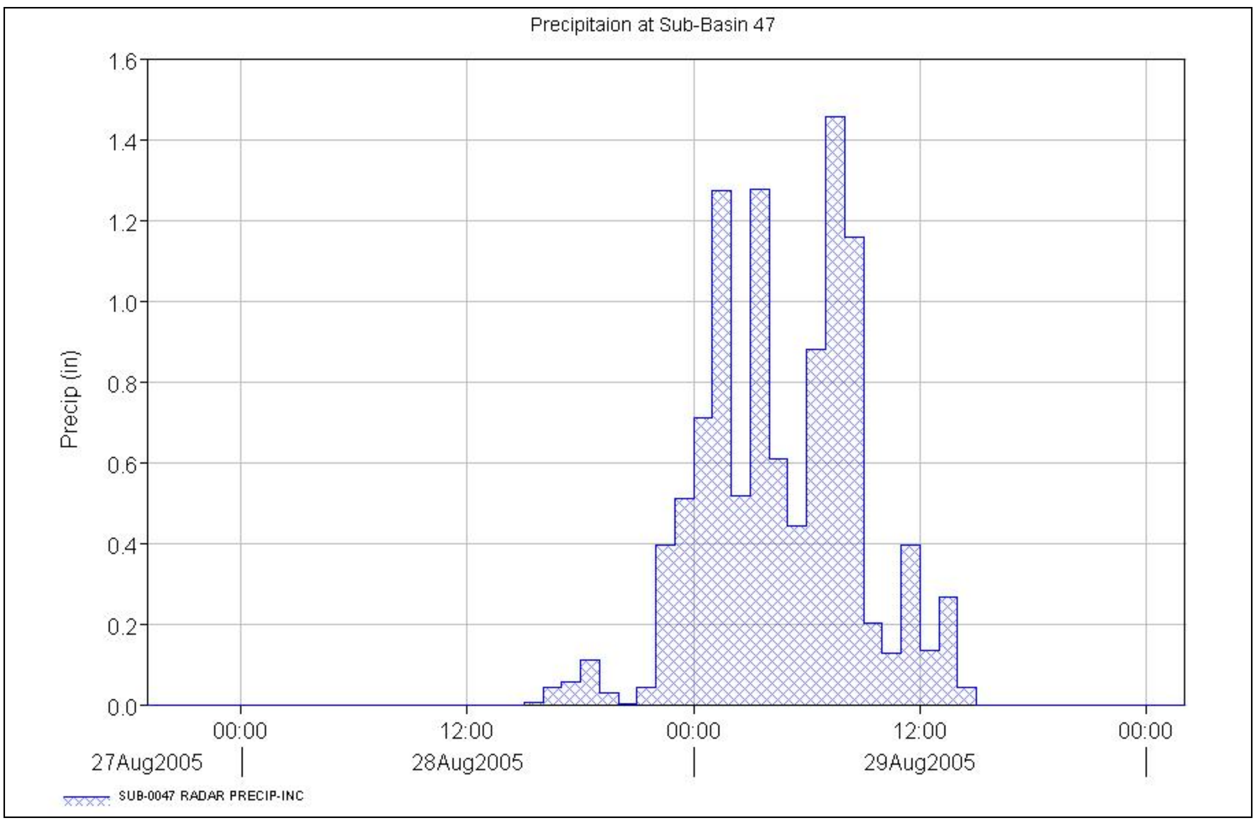

Figure 8-6. Average Rainfall for Sub-basin 47

Model Results

Summary output from the HEC-HMS model is available in Table 8-6. A complete runoff hydrograph was also computed by the hydrologic program. This information was stored in an HEC-DSS file and provided part of the boundary condition for the HEC-RAS model of Orleans West Bank, Algiers and English Turn.

Table 8-6
Summary Output from HEC-HMS Model

Sub-basin Name	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
1	152	29Aug2005, 08:12	8.2
2	137	29Aug2005, 08:14	8.2
3	145	29Aug2005, 08:10	8.4
4	147	29Aug2005, 08:06	8.7
5	85	29Aug2005, 08:06	8.6
6	95	29Aug2005, 08:12	8.9
7	72	29Aug2005, 08:00	8.5
8	122	29Aug2005, 08:06	8.8
9	316	29Aug2005, 08:12	8.9
10	282	29Aug2005, 08:22	8.4
11	221	29Aug2005, 08:10	8.8
12	89	29Aug2005, 08:08	8.3
13	202	29Aug2005, 08:10	9.1
14	190	29Aug2005, 08:12	8.5
15	88	29Aug2005, 08:10	8.3
16	31	29Aug2005, 08:00	8.5
17	130	29Aug2005, 08:08	8.7
18	246	29Aug2005, 08:02	8.6
19	178	29Aug2005, 08:02	7.8
20	126	29Aug2005, 08:08	8.4
21	81	29Aug2005, 08:06	8.4
22	40	29Aug2005, 08:00	8.6
23	149	29Aug2005, 08:12	9.1
24	150	29Aug2005, 08:08	8.5
25	55	29Aug2005, 08:00	9.1
26	36	29Aug2005, 08:00	8.9
27	191	29Aug2005, 08:16	8.9
28	28	29Aug2005, 08:00	9.3
29	243	29Aug2005, 08:16	9.1
30	153	29Aug2005, 08:02	8.2
31	60	29Aug2005, 08:06	8.2
32	63	29Aug2005, 08:00	8.6
33	152	29Aug2005, 08:02	8.7
34	112	29Aug2005, 08:04	8.9
35	255	29Aug2005, 08:08	8.7
36	125	29Aug2005, 08:12	9.3
37	46	29Aug2005, 08:02	8.6
38	103	29Aug2005, 08:16	8.8
39	87	29Aug2005, 08:12	8.8
40	231	29Aug2005, 08:06	8.8
41	192	29Aug2005, 08:04	8.3
42	42	29Aug2005, 08:04	8.0
43	232	29Aug2005, 08:04	8.7
44	76	29Aug2005, 08:02	8.2
45	97	29Aug2005, 08:04	8.2
46	192	29Aug2005, 08:20	8.4
47	253	29Aug2005, 08:20	8.7
48	44	29Aug2005, 08:00	8.0

(Continued)

Table 8-6 (Concluded)			
Sub-basin Name	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
49	127	29Aug2005, 08:14	8.3
50	123	29Aug2005, 08:08	8.7
51	80	29Aug2005, 08:06	8.7
52	239	29Aug2005, 08:16	8.7
53	153	29Aug2005, 08:12	8.5
54	44	29Aug2005, 08:02	8.0
55	129	29Aug2005, 08:10	8.3
56	217	29Aug2005, 08:24	8.7
57	140	29Aug2005, 08:06	8.3
58	94	29Aug2005, 08:04	8.6
59	44	29Aug2005, 09:00	7.6
60	96	29Aug2005, 09:00	7.7
61	214	29Aug2005, 09:04	7.9
62	102	29Aug2005, 09:04	7.6
1000	64	29Aug2005, 09:06	5.0
1001	44	29Aug2005, 09:02	5.3
1002	38	29Aug2005, 09:00	5.0
1003	65	29Aug2005, 09:02	5.0
1004	43	29Aug2005, 09:02	5.0
1005	39	29Aug2005, 09:02	5.2
1006	28	29Aug2005, 09:00	5.2
1007	16	29Aug2005, 09:00	5.0
1008	75	29Aug2005, 09:12	4.8
1009	51	29Aug2005, 09:00	5.5
1010	39	29Aug2005, 09:00	5.2
1011	86	29Aug2005, 09:00	5.5
1012	44	29Aug2005, 09:00	5.1
1013	58	29Aug2005, 09:08	5.0
1014	25	29Aug2005, 09:04	4.4
1015	39	29Aug2005, 09:00	5.2
1016	23	29Aug2005, 09:00	5.5
1017	34	29Aug2005, 09:00	5.5
1018	11	29Aug2005, 09:00	5.2
1019	63	29Aug2005, 09:02	4.7
1020	73	29Aug2005, 09:08	4.5
1021	44	29Aug2005, 09:00	5.5
1022	32	29Aug2005, 09:00	5.4
1023	76	29Aug2005, 09:12	5.0
1024	57	29Aug2005, 09:04	4.4
1025	36	29Aug2005, 09:00	5.2
1026	24	29Aug2005, 09:00	5.1
1027	35	29Aug2005, 09:00	5.4
1028	20	29Aug2005, 09:00	5.3
1029	112	29Aug2005, 09:18	4.8
1030	106	29Aug2005, 09:06	4.9
1031	34	29Aug2005, 09:16	4.6
1032	72	29Aug2005, 09:02	5.1
1033	276	29Aug2005, 09:30	4.9
1034	18	29Aug2005, 09:00	4.3
1035	128	29Aug2005, 09:12	4.4
1036	14	29Aug2005, 09:00	5.0
1037	103	29Aug2005, 09:12	4.9
1038	70	29Aug2005, 09:12	4.2
1039	64	29Aug2005, 09:06	4.2
1040	149	29Aug2005, 09:12	5.0
1041	40	29Aug2005, 09:08	4.8
1042	60	29Aug2005, 09:18	4.2
1043	74	29Aug2005, 09:22	4.2
1044	215	29Aug2005, 09:38	4.5
1045	60	29Aug2005, 09:12	4.8
1046	23	29Aug2005, 09:00	5.5

RAS Interior Modeling

Background

The leveed areas of Orleans West Bank, Algiers and English Turn districts are subject to ponding of runoff and shallow flooding due to inadequate subsurface drainage and the sheet flow associated with overland travel of excess water that cannot enter the subsurface system. This excess water collects in depressions and may remain trapped for hours or even days before finally being carried away by the drainage system. Disruption to the power supply may reduce pumping capacity such that the system is overwhelmed by storm inflows resulting in ponding in the lower portions of the districts. Extreme tropical storm events may overwhelm the flood protection system through wave-overtopping, free-flow over the line of protection, and structural failure of the levees, however no breach or overtopping of the flood protection system was experienced in Algiers of English Turn during the Katrina Event. Because no breaching or overtopping occurred, the Katrina model geometry does not include the bounding Mississippi River or the levees separating the Mississippi from Orleans West Bank.

Datum Reconciliation

The Katrina modeling utilizes the NAVD88 1994, 1996 datum. Construction of the model utilized various sources of data in various vertical datum. The U.S. Army Corps of Engineers New Orleans District had previously modeled a portion of canal system of Algiers District, as described in the Review of Existing Data section, in the Cairo datum. Both pumping stations in Orleans West Bank report stage (used for calibration and boundary condition) in Cairo datum. Data in the Cairo datum was corrected to the final project datum of NAVD88 1994, 1996 by subtracting a constant of 20.43 ft. For the rest of the study area, the topographic data was taken from the LiDAR surveys that were performed in the final model datum. Surveys for the remaining channels and structures in Algiers district were not available. No existing models or survey data was available for Algiers District. Where no survey data or prior models were available, channel cross sections and structure inverts were based on aerial photography, site visits, and the Master Drainage Plan.

Terrain Model

The primary source of topographic data in the ponding areas was a LiDAR survey of South Louisiana taken for the Federal Emergency Management Agency in 2004. The data collected during this LiDAR survey was processed using Geographic Information System (GIS) to produce the stage-volume curves for each of the 109 storage areas in the study area. Additional information from a visit to the site was used to supplement data.

Basic Geometric Data Using GIS

The LIDAR data set was used to determine the heights of the drainage divides, such as levees, roads, and railroad grades, for the RAS model. It was also used in determining the heights of the lateral weirs that connect the storage areas to the drainage canals or reaches. While the existing HEC-RAS model for a portion of Algiers was georeferenced, the model geometry was recreated in GeoRAS in order to easily incorporate the partial existing model into the full modeling effort. Aerial photography, raster topographic data and the drainage master plan guided the development of the full river network in GeoRAS. Placement of cross sections was likewise determined. The river network, cross sections, bridge decks, lateral structures, bridge deck and storage areas were developed in GeoRAS and cut from the 2004 LiDAR data. This was imported to HEC-RAS as a fully georeferenced system. The existing model was merged into the new georeferenced data set in entirety to preserve its integrity. The existing model represents only the portion of the Katrina model where survey data is used for channel bathymetry. Only a portion of Algiers district and none of English Turn were included in the existing model. No additional survey data would be available.

This LiDAR data set consistently reflected the water surface at the time the data was collected. In English Turn, where no channel and structure survey was available, the water level shown in the LiDAR data was at normal operating levels near -7.4 ft NAVD88 1994, 1996. This water level proved useful during the site visit as it allowed for determination of structure elevations relative to one another as well as overall vertical placement of cross section thalwegs and structures in the model. When compared with aerial photography, this water level also allowed for the determination of thalweg elevation when the channel bottoms rose above the water level. In areas where the canal or ditch is relatively small compare to the raster cell size of the LiDAR terrain data, accurate channel thalwegs can not be positively determined. A significant improvement in the English Turn model outputs may be realized by incorporating survey data of cross sections and structures into the numerical model.

Inundation maps were prepared showing the hurricane Katrina event and a hypothetical scenario where the pump stations operate with full capacity throughout the event.

Manning's *n*-Values

The Manning's *n*-value used for an earthen channel was 0.075 to 0.025 with 0.045 being the most common value used. For concrete lined channels and culverts the Manning's value used was .019. Overbank areas were modeled with Manning's *n*-values between 0.100 and 0.035 with values matching the channel being most common. These values were used consistently throughout the study area.

Bridges and Culverts

Bridges and box culverts were analyzed as part of the HEC-RAS model for the whole basin. HEC-RAS computes flow through the modeled bridge or culvert using the Bernoulli or Energy Equation. Entrance and exit losses are also computed using coefficients input for each structure. Hydraulic losses in large concrete box culverts and arch pipes were computed using entrance and exit loss coefficients recommended in the HEC-RAS Reference Manual. These were 0.3 to 0.5 and 0.5 to 1.0 respectively, depending on what local conditions required.

Many of the bridges and box culverts in the model were measured in the field and entered into the model without survey data. Water in the canals in Algiers provided a means to compare the elevation of structures in the existing model to unsurveyed structures. However, some portions of the district canal system were dry leaving an assumption that the channel thalweg and structure inverts were higher than the water level, but unknown without survey. English Turn contains significantly fewer structures, but they are significant in their affect. As described in the Basic Geometry Data Using GIS section, water levels in the LiDAR data, and field observation of the structures were used to place the bridges, culverts and inline structures vertically in the model, however, a significant improvement in the English Turn model outputs may be realized by incorporating survey data of structures into the numerical model.

Ineffective Flow Areas

Ineffective flow areas were set for bridges and culverts to simulate the slack water found in the contraction and expansion of the channel upstream and downstream of the structure. Many of the structures in this model are almost as wide as the canals; therefore no ineffective flow areas were placed on the cross sections outside of these structures.

Storage Areas

The study area was divided up into 109 storage areas. LiDAR data was used to determine the stage-volume relationship for each storage area by extracting it from the GIS data set using GeoRAS. The storage areas were defined by the drainage divides such as roads, railroad embankments, drainage canals, and/or levees. In the English Turn district where relatively little development has occurred, storage areas were also defined by regions of similar elevation. The reason behind this approach was to better model shallow inundation throughout the entire region. Storage areas were hydraulically connected to the canals by using lateral weirs. Additionally, storage areas were connected to each other with a weir or the HEC-RAS linear routing option.

Lateral Structures and Storage Area Connections

For the weirs connecting storage areas to the canals, weir coefficients between 0.6 and 1.0 were used. These values are lower than one might think of for a traditional lateral weir that is designed to remove flow from a stream to an overbank area. However, lateral weirs as used in this model are to allow water in a storage area to flow overland and get into the canals. This is not really a physical weir situation, and therefore using traditional weir coefficients would transfer the water too quickly from the storage area to the canal. It has been found through experience and model calibration with other models that values around 1.0 seem to provide the appropriate transfer of flow between canals and storage areas. The model is sensitive to this factor in that the shape of the rising and falling limb as well as the maximum stage and timing of the flood wave can be modified to some degree. All water passing from storage areas to the pump station must pass through a lateral structure increasing their importance in this model.

Weir coefficients for storage area connections that represent higher ground between storage areas were also set with coefficients near 1.0. Due to the lack of perimeter levee breaching or overtopping, the Orleans West Bank models perform as low flow models relative to many of the other parishes. Storage area connection weirs in this model operate more as sheet flow described above and require weir coefficients lower than traditional values.

Linear routing was used in regions where grade breaks or inconsistent higher ground separated two storage areas. Coefficients were set to 0.01 for the storage area connections in which linear routing was used. The English Turn model approximated hydrologic lag in its network of storage areas using storage area connections. The linear flow coefficient was an important tool in calibration in the controlling the peak elevation and detaining water in the system. The linear routing equation is as follows:

$$Q = k\left(\Delta S\right)/Hour$$

where:

Q = Flow

k = Linear Routing Coefficient (Varies from 0.0 to 1.0)

ΔS = Available Storage (Difference in head times multiplied by surface area of receiving storage area)

Because this equation computes a rate per hour the magnitude is divided by the time step to get flow per time step. A minimum elevation for flow to pass between storage areas must also be entered. If both storage areas are below this elevation no flow is exchanged. If one storage area has a stage greater than the minimum elevation, the head difference is the elevation of the storage area minus the user entered minimum elevation for passing flow.

Pump Stations

2 pump stations operated by the New Orleans Sewerage and Water Board (NOS&W) drain Orleans West Bank, one in Algiers and one in English Turn. Both stations discharge into the GIWW. Table 8-7 shows basic data for the two pump stations. Further information on the operation of the pump stations is described in Appendix 7 of this Volume. The RAS model attempted to model the pump operation as close to what actually occurred as possible, such as pump failures caused by power outages and cycling due to draw down. The Operators' Logs for both stations proved indispensable in understanding how these pumps operated during the event. Both stations experienced power outages that limited the station capacity during the event, however English Turn reported that the flood waters were kept within the channels despite the limited pumping capacity. Algiers experienced minor flooding as the flood water came out of the canal banks in the lowest parts of the district.

Pump geometry and operations were coded in the model in such a way that the individual pumps operating during the event could be turned on or off as detailed in the Operators' Log regardless of stage at the inlet. The desire to optimize pump start and stop elevations was left for the hypothetical condition of ideal pumping where operational limits of the pumps were taken into account. The pumps are allowed to operate on their curves with the exception of the draw down period before the storm when initial conditions in the drainage system are being set.

Table 8-7
Pump Station Information

Pump Station Name	Pump From	Pump To	Capacity (cfs)
Orleans P.S. # 11	Donner Canal	Intercoastal Waterway	4700
Orleans P.S. # 13	Nolan and East Donner Canals	Intercoastal Waterway	1690

Storm Drain System

The drainage system for Algiers consists of many features that are typical of large urban cities in the United States. As in any urbanized area, catch basins and drop-inlets receive surface runoff from yards and streets, and excess runoff runs down slope in the streets and/or overland to areas of lower elevation. Runoff that can enter drop-inlets proceeds underground in small pipes, 21 inches or less in diameter, called the tertiary system that collect local flows and convey them to the secondary system, 21 inches to 30 inches in diameter, where several of these local flows combine. Generally pipes or box culverts that are larger than 30 inches in diameter are considered to be part of the secondary system. The primary drainage system is largely composed of man-made mainly prismatic, trapezoidal open channels, though some of the historic open drainage channels have been put underground in some areas. The primary drainage system, the larger pipes in the secondary drainage system and the pump station were modeled in the HEC-RAS unsteady model for Algiers.

The drainage system in English Turn is a stark contrast to that found in Algiers and much of the rest of urbanized New Orleans. English Turn is largely undeveloped and the development that has occurred consists of large lots with considerable green space. An unusual feature that affects the overall performance of the drainage system is a series of water features excavated during the construction of the English Turn Golf and Country Club. These water features are large enough to provide additional flood water storage thereby attenuating the flood peak for the whole lower portion of the district. Secondary and tertiary drainage is largely absent in the district. The distal ends of the open primary drainage system typically terminate with roadside ditches in the highest ground throughout most of the district. Only the larger canals and ditches were modeled in HEC-RAS unsteady model for English Turn.

Flow Data and Boundary Conditions

The rainfall-runoff hydrographs developed by the HMS model were applied to the appropriate storage area as inflow hydrographs. The upstream boundary condition for the internal drainage canals was a minimal flow condition that was considered the base flow condition. That flow was determined by running the model with the minimum flow that would allow the model to run. The pump stations act as internal boundary conditions. The downstream boundary condition, the receiving waters for the pumps, was based on the stage hydrographs obtained from the Operators' Log. The Mississippi River and associated levees, which bounds Orleans West Bank on three sides, were not modeled since no overtopping or breaching was observed in the area.

Model Calibration

The Operators' Logs provided the only calibration data for Algiers and English Turn. No other high water marks in the districts were available. The logs did provide an excellent stage hydrograph for calibration in lieu of crest marks. With the stage hydrograph, total storm volume as well as timing of the event could be addressed. With such detailed calibration data, pump operations became critical in understanding how the system operated during the event. Again, the Operators' Logs provided ample detail. The first major hurtle in calibration was do decipher the notation of the operators during the event. Stage at both the intake and outlet of the pump,

pump operations and rainfall were written in longhand in the log. These notes do not agree with operations provided in Appendix 7 as diesel pumps were used in both stations when line power was lost. In particular, many pump starts and stops were recorded at pump station #11 when the system was being held at very low stages before the event and through a period when the line power was lost and diesel backup power was being started.

Algiers calibrated easily when accurate pump data was entered into the model. The rising limb, peak and falling limb all match observed data with the observed peak within 0.2 ft of the modeled peak. The modeled stage falls away from the observed stage in the falling limb of the hydrograph. This discrepancy can be accounted for by looking at the observed stage data before and after intake screen at pump station #13. The screen appears to plug quickly creating a head difference across the screen that would decrease the overall pumping rate of the station. Since HEC-RAS uses the intake pond level as part of the calculation to determine pumping rates, the model calculates a pumping rate higher than occurred during the event and draws the district down to a lower elevation. Modeling the screen plug does not affect the high water calculations and was not done.

English Turn did not calibrate easily and proved difficult to match the observed data. The modeled maximum stage at pump station #11 exceeds the observed data by 1.2 ft. While observed stage data is incomplete during the rising limb of the flood wave, the existing data indicates that the modeled stage rises too quickly. The modeled peak, while 1.2 ft higher, does match the timing of the observed peak, though this may be attributed as much to pump operations as model runoff characteristics. The modeled falling limb has the wrong shape with water coming out of the basin too quickly. Investigation into storm volume indicated that the HMS modeled storm using initial hydrologic and meteorological parameters exceeded the pumping volume significantly. During the course of calibration, this discrepancy in storm volume was addressed by decreasing precipitation to the district by 10% and increasing hydrologic loss (decreasing curve number by 5). Flood wave timing was also addressed during calibration by increasing hydrologic lag time (a product of increasing loss) and lowering lateral structure and storage area weir coefficients. These measures did not bring the model into the level of calibration achieved in the Algiers district. Improvements in the model could reasonably be expected by acquiring and integrating survey data for the major canals and structures in the system into the model. Operations in both districts began with pumping the system to the lowest possible stage in order to create storage volume in the primary and secondary drainage features. In English Turn, these stages are well below the water level shown in the LiDAR data that provided the channel geometry. The modeler provided a reasonable estimate of channel bathymetry, but the actual channel shape and available storage volume in the channels is unknown. Comparison between observed and computed high water elevations is shown in Table 8-8.

Table 8-8
Computed Elevation Versus Observed Elevation

Pump Intake Stages	HEC-RAS Computed Elevation	Observed Elevation
Orleans Pump Station #11 Intake, English Turn	-2.7	-3.9
Orleans Pump Station #13 Intake, Algiers	-3.7	-3.5

Model Results and Floodplain Mapping

The modeling effort reproduced the Hurricane Katrina event within reasonable tolerances. Localized flooding due to storm intensities overwhelming the secondary storm system is indicated in Algiers. Shallow flooding reported in the lowest areas of Algiers adjacent to the pumping station is also shown. The wetland areas of English Turn, as shown in the land use Figure 8-2, show shallow flooding. Shallow flooding is also shown in English Turn behind road grades and artificially at some storage area boundaries. Model results showing the extent and depth of flooding for Katrina are mapped in Figure 8-7.

Figures 8-8 and 8-9 show the computed stage and flow hydrograph for English Turn and Algiers districts respectively for the Katrina scenario. Note the discrepancy between modeled and observed stage occurring on August 30 in Figure 8-9. This rapid decrease in the observed hydrograph is attributed to screen plugging with the figure showing the observed stage at the pump side of the screen. For the English Turn and Algiers areas, only the Katrina event and Hypothetical 2 were modeled. Results for Hypothetical 1, 3 and 4 are the same as for the Katrina event since there were no levee or floodwall overtopping or breaching in these areas.

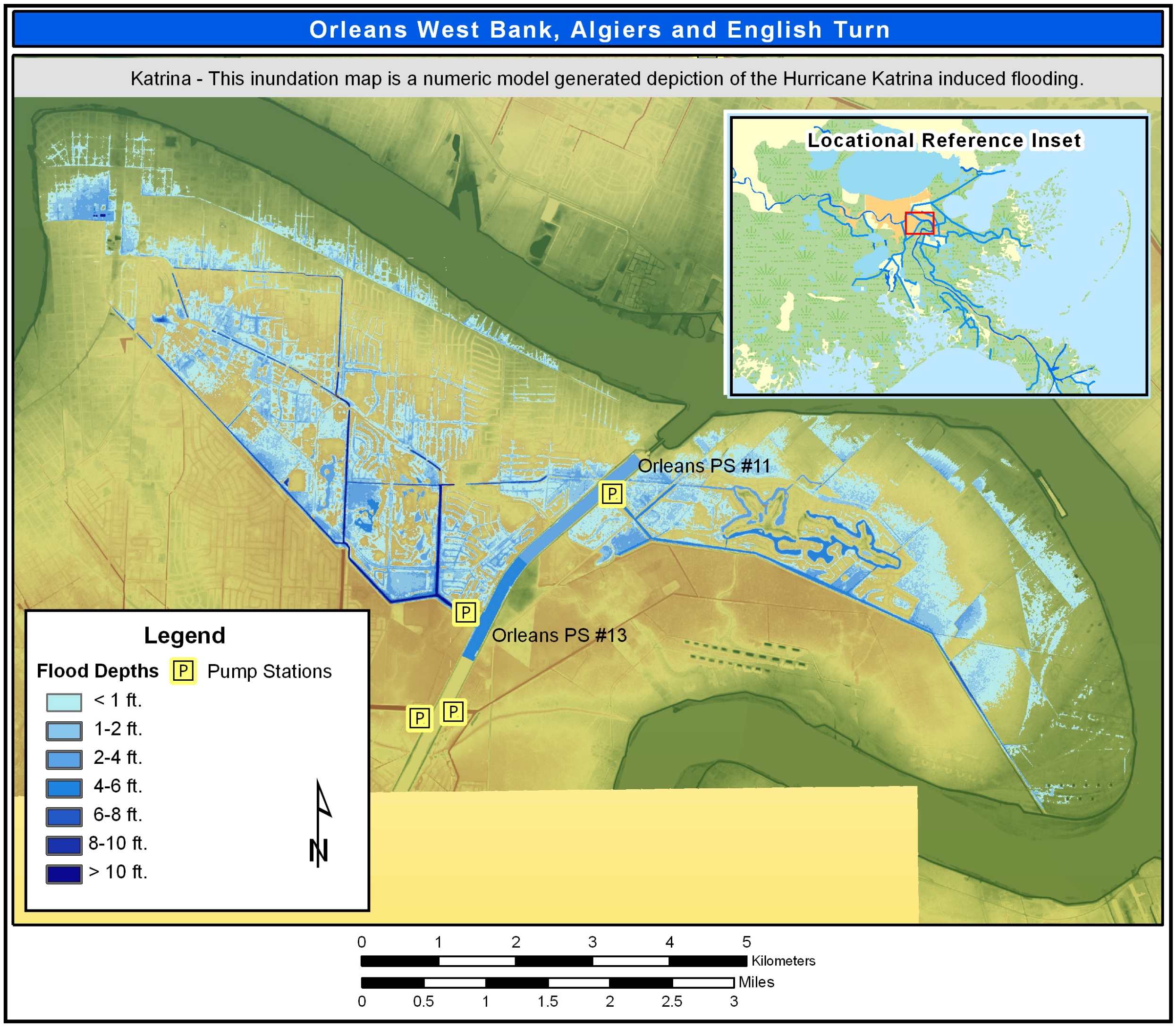

Figure 8-7. Maximum Flood Depths from Katrina Event

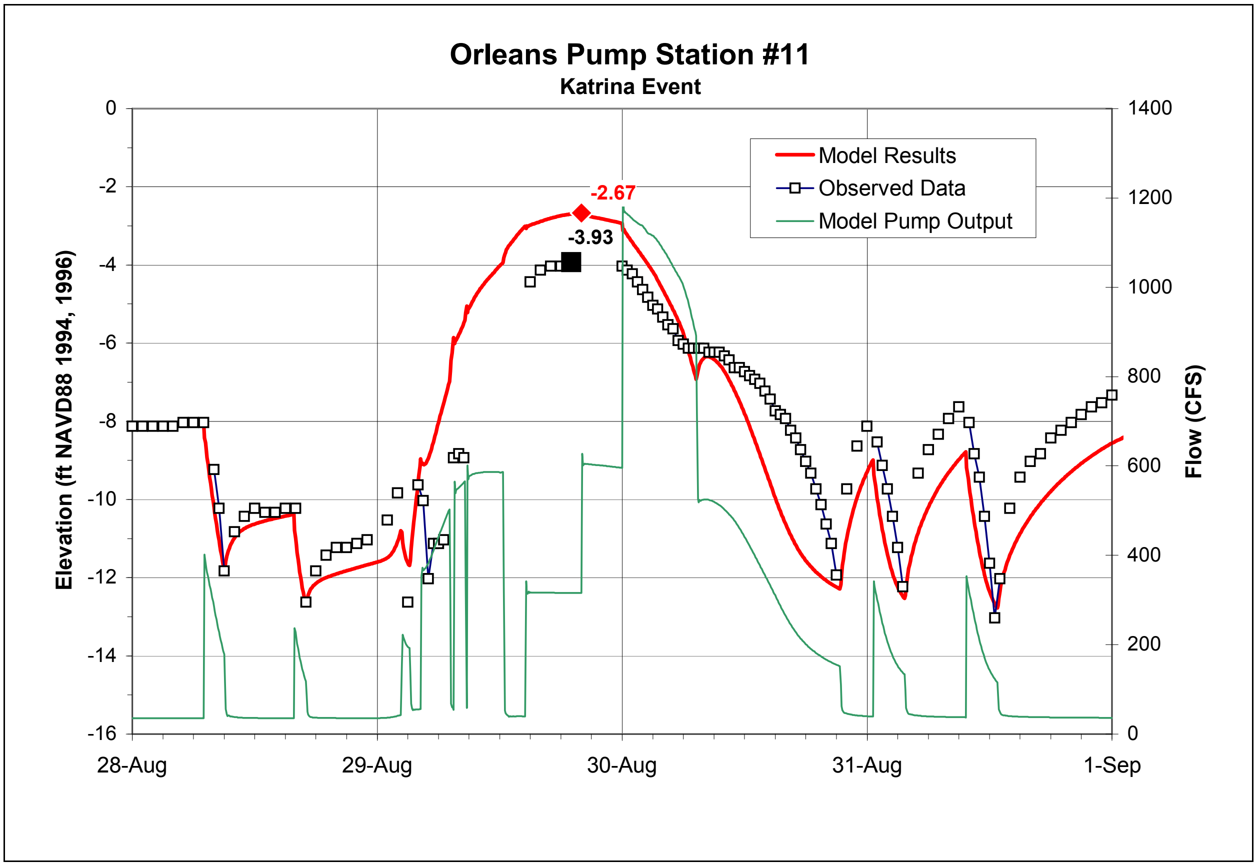

Figure 8-8. Computed Stage Hydrograph at Orleans Pump Station #11 (English Turn) pump intake for Katrina Simulation

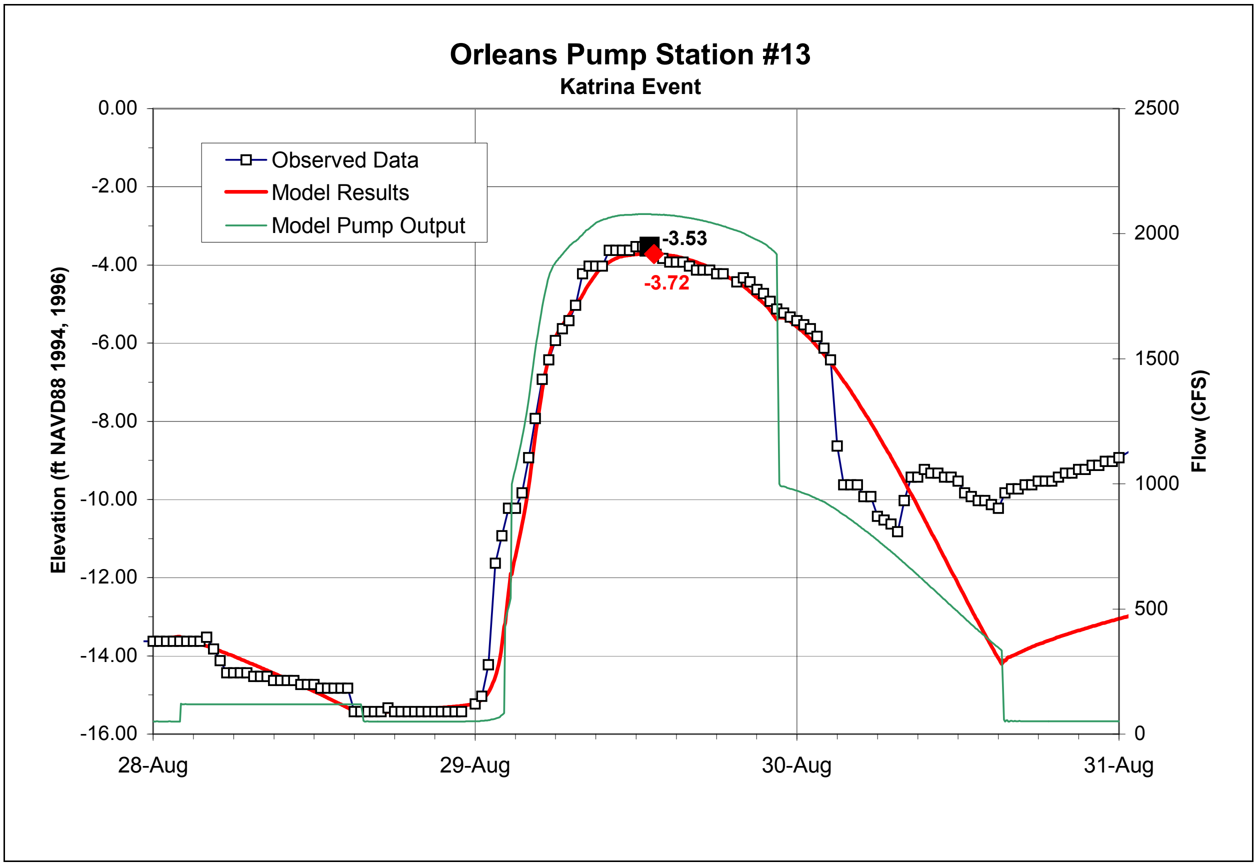

Figure 8-9. Computed Stage Hydrograph at Orleans Pump Station #13 (Algiers) pump intake for Katrina Simulation

Figure 8-10 maps the extents and depths of flooding determined by the Hypothetical 2 modeling scenario with resilient pump. Table 8-9 shows a comparison of stages for the two scenarios for Orleans West Bank. It should be noted that English Turn did not report any flooding due to inadequate pump capacity during the event. The inundation in the uplands and lower wetlands of English Turn is similar to the Katrina event simulation. The shallow flooding in the lowest portions of Algiers district, adjacent to pumping station #13, is alleviated in this hypothetical. The loss of line power and subsequent reduction in pumping capacity did cause flooding of the lowest parts of Algiers during the Katrina Event. Figure 8-10 does show that much of the shallow surface flooding experienced throughout the higher areas of Algiers would have occurred with full pumping capacity.

Table 8-9
Computed Stages for Katrina and Hypothetical 2 (see Figures 8-11 and 8-12)

HEC-RAS Storage Area	Katrina	Hypothetical 2
Orleans Pump Station #11 Intake, English Turn	-3.9	-7.8
Orleans Pump Station #13 Intake, Algiers	-3.5	-6.2

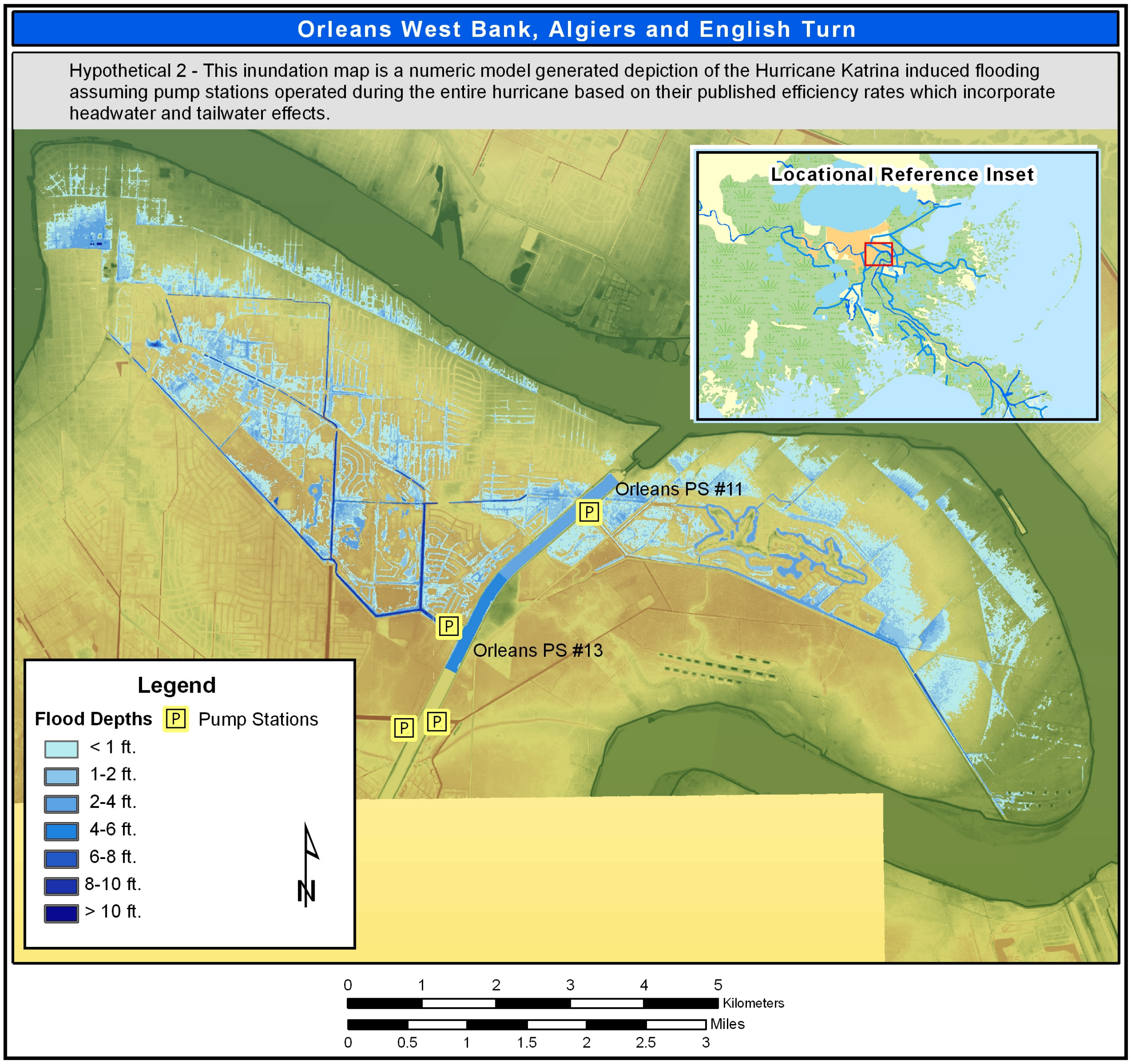

Figure 8-10. Depth of Flooding from Hypothetical 2 Scenario

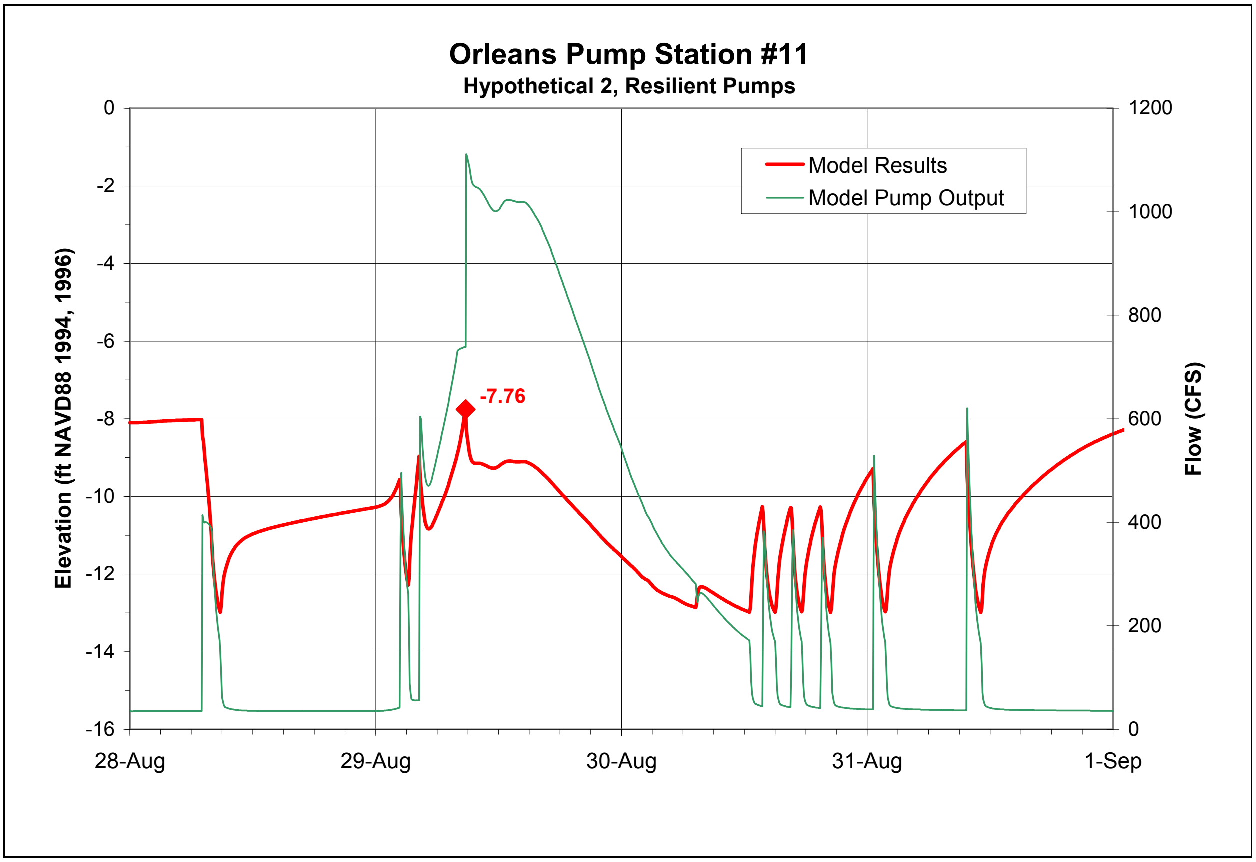

Figure 8-11. Computed Stage Hydrograph at Orleans Pump Station #11 (English Turn) pump intake for Hypothetical 2, Resielient Pumps.

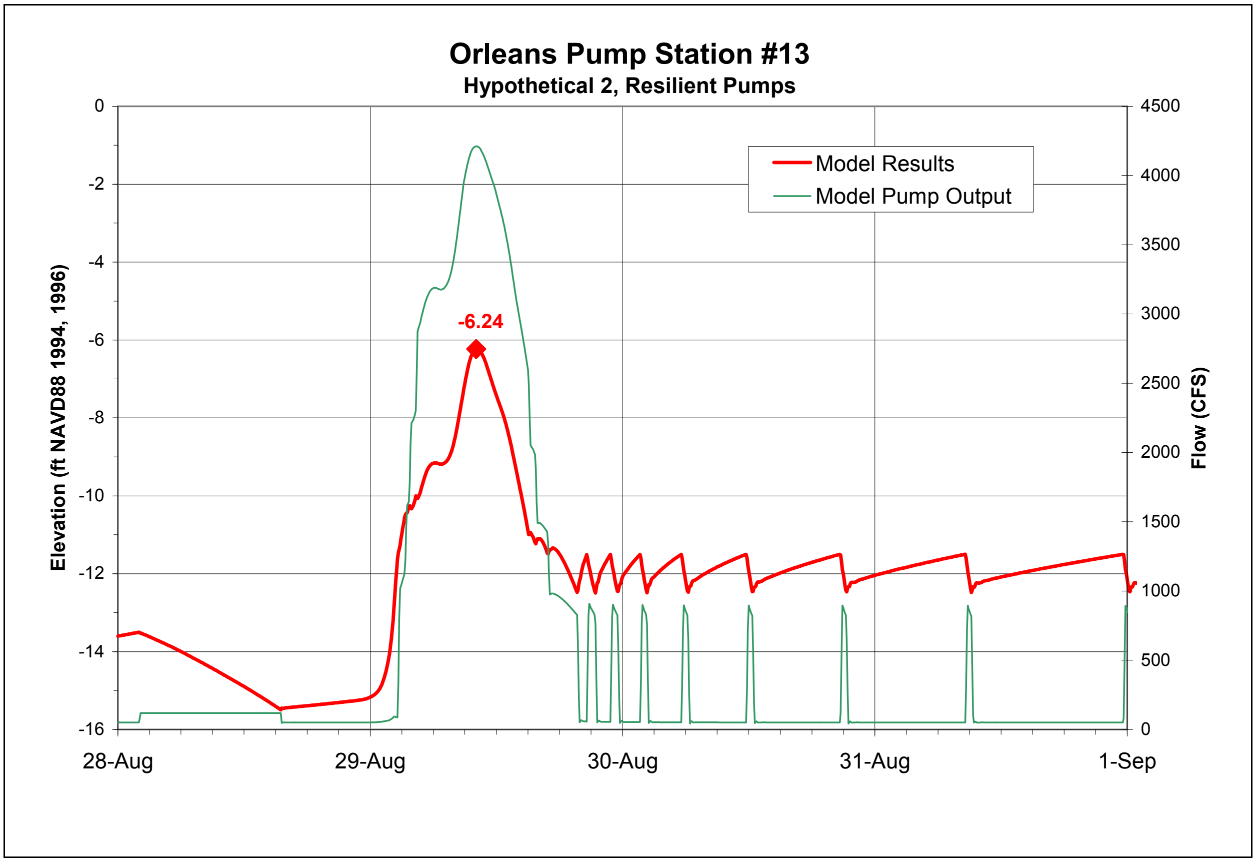

Figure 8-12. Computed Stage Hydrograph at Orleans Pump Station #13 (English Turn) pump intake for Hypothetical 2, Resielient Pumps.

Appendix 9
Interior Drainage Analysis – St. Charles Parish East Bank

Introduction

Study Purpose

To answer the questions regarding the performance of the hurricane protection system, the interior drainage analysis focused on the filling and unwatering of the separate areas protected by levees and pump stations, referred to as basins. Interior drainage models were developed for Jefferson, Orleans, St. Bernard and Plaquemines Parishes to simulate water levels for what happened during Hurricane Katrina and what would have happened had all the hurricane protection facilities remained intact and functioned as intended.

The primary components of the hurricane protection system are the levees and floodwalls designed and constructed by the Corps of Engineers. Other drainage and flood control features (land topography, streets, culverts, bridges, storm sewers, roadside ditches, canals, and pump stations) work in concert with the Corps of Engineers levees and floodwalls as an integral part of the overall drainage and flood damage reduction system and are included in the models.

Interior drainage models are needed for estimating water elevations inside leveed areas, or basins, for a catastrophic condition such as Hurricane Katrina and for understanding the relationship between HPS components. Results from the interior drainage models can be used to determine the extent, depth and duration of flooding for multiple failure and non-failure scenarios. The models can also be used to:

- Support the Risk modeling effort
- Estimate time needed to unwater an area
- Support evacuation planning
- Evaluate design options of the HPS to include multiple interior drainage scenarios.

This appendix will provide details of the development of the HEC-HMS and HEC-RAS models for the East bank of St Charles Parish. In summary, an HEC-HMS model was developed to transform the Katrina precipitation into runoff for input to the HEC-RAS models. HEC-RAS models were developed to simulate the five conditions discussed below

This model was developed to help answer questions 3 and 4 listed on page 1 of Volume VI. Question 3 is answered by the Katrina simulation listed below. Question 4 is a more difficult one to answer. This is mainly due to the variety of possible combinations of system features, especially pumps. It was decided to bracket these combinations with the four hypothetical combinations listed below. Not all of the hypothetical scenarios apply to St. Charles. If the scenario is not modeled it is noted in the description below.

One of the major difficulties is determining what pumps may have continuing operating. There are many potential factors that can cause pump stations to not operate during a hurricane event. Some of these are power failures, pump equipment failures, clogged pump intakes, flooding of the pump equipment, loss of municipal water supply used to cool pump equipment and no safe housing for operators at the pump stations resulting in pump abandonment. Because there is such a wide range of possible pumping scenarios that could occur during a hurricane event, it is difficult to establish a pumping scenario for what could have happened. At best, a variety of possible scenarios could be run to evaluate the potential range of possible consequences. For the purposes of the IPET analysis, it was decided to operate the pumps two ways. (1) As they actually operated during hurricane Katrina and (2) the pumps operated throughout the hurricane.

Described below are the 5 scenarios generally applied in the Interior Drainage and Pumping Technical Appendices. St. Charles Parish experienced very little flooding during Hurricane Katrina, no levees were overtopped and no floodwalls or pumps failed. Therefore, the only simulation run for St Charles was the Katrina simulation

Katrina

Simulate what happened during Hurricane Katrina with the hurricane protection facilities and pump stations performing as actually occurred. Compare results to observed and measured high water marks. Pre-Katrina elevations are used for top of floodwalls and levees.

Hypothetical 1 – Resilient Levees and Floodwalls

Simulate what would have happened during Hurricane Katrina had all levees and floodwalls remained intact. There are no levee or floodwall breaches or failures for this scenario even where overtopping occurs. Pump stations operate as they did in the Katrina event. Pre-Katrina elevations are used for top of floodwalls and levees. This scenario is meant to simulate what could have happened if all levees and floodwalls had protection that would allow them to be overtop but not breach. St Charles Parish did not experience any levee or floodwall breaches or failures therefore this scenario was not modeded since the results would be the same as for the Katrina simulation.

Hypothetical 2 – Resilient Floodwalls, Levees and Pump Stations

Simulate what would have happened during Hurricane Katrina had all levees, floodwalls and pump stations remained intact and operating. There are no levee or floodwall breaches or failures for this scenario even where overtopping occurs. Pump stations operate continuously throughout the hurricane. Pump operations are based on the pump efficiency curves which reflect tail water impacts. Pre-Katrina elevations are used for top of floodwalls and levees. It is understood, that in their present state, most pump stations would not have been able to stay in operation during Katrina. However, this scenario was simulated to provide an upper limit on what could have been the best possible scenario had no failures occurred.

Hypothetical 3 – Resilient Floodwalls

Simulate what would have happened during Hurricane Katrina had all floodwalls, which failed from foundation failures, remained intact. All other areas are modeled as they actually functioned. Pump stations operate as they did in the Katrina event. Pre-Katrina elevations are used for top of floodwalls and levees. Since there were no floodwall failures, this scenario was not simulated because the results would be the same as the Katrina simulation.

Hypothetical 4 – Resilient Floodwalls and Levees at Authorized Design Grade

Simulate what would have happened during Hurricane Katrina had all levees and floodwalls remained intact and the crest of all levees and floodwalls were at the authorized design grade elevation. There are no levee or floodwall breaches or failures for this scenario even where overtopping occurs. Pump stations operate as they did during the Katrina event. Since there were no floodwall failures and levees are at the Authorized Design Grade, this scenario was not simulated because the results would be the same as the Katrina simulation.

Table 9-1 lists the simulation scenarios in matrix format. Since there was no flooding, overtopping, levee failure, or pump failure, and the levee was recently constructed and is at its design grade, all scenarios are the same.

Table 9-1
Katrina Scenarios

	Katrina	Hypothetical 1	Hypothetical 2	Hypothetical 3	Hypothetical 4
Pumps operate as during Katrina	X	X		X	X
Pumps operate throughout Katrina	N/A	N/A	N/A	N/A	N/A
Levee and floodwall breaches occur everywhere as during Katrina	N/A	N/A	N/A	N/A	N/A
Levee and floodwall breaches occur on West wall of IHNC and in, St Bernard, New Orleans East and Plaquemines as during Katrina	N/A	N/A	N/A	N/A	N/A
Levee and floodwalls overtop but do not breach	N/A	N/A	N/A	N/A	N/A
No failures on 17th Street and London Ave	N/A	N/A	N/A	N/A	N/A
Levee and floodwall elevations based on pre-Katrina elevations	X	X	X	X	X
Levees and Floodwalls elevations are at authorized elevation	N/A	N/A	N/A	N/A	N/A

General Modeling Approach

HEC-HMS, version 3.0, was used to simulate the rainfall hydrograph from Hurricane Katrina using the SCS Curve method described in TR-55, *Urban Hydrology for Small Watersheds*. The one-dimensional unsteady flow method available in HEC-RAS, was used to calculate water surface elevations in channels and storage areas.

Hydrologic Model Development

Background

HEC-HMS version 3.0.0 was used to model the rainfall-runoff response for the Hurricane Katrina event for subbasins in St Charles Parish. Subbasin boundaries in the HEC-HMS model correspond to storage areas defined in the HEC-RAS model. Rainfall for each subbasin was determined using radar-rainfall estimates from the National Weather Service. The SCS curve number and the SCS dimensionless unit hydrograph methods were used to compute runoff hydrographs given basin average precipitation. GIS data, like landuse and soil data, were used to estimate SCS curve numbers and lag times.

Development of GIS Watershed Model

Subbasin boundaries for the St Charles Parish HEC-HMS model are shown in Figure 9-1 and Figure 9-2. Basin boundaries correspond to storage areas defined in the HEC-RAS model for this area. A shapefile of subbasin boundaries was used for estimating HEC-HMS model parameters, subbasin area, curve numbers and lag times, and determining subbasin average precipitation from the radar-rainfall data. The shapefile was also used as the background map in the HEC-HMS basin model.

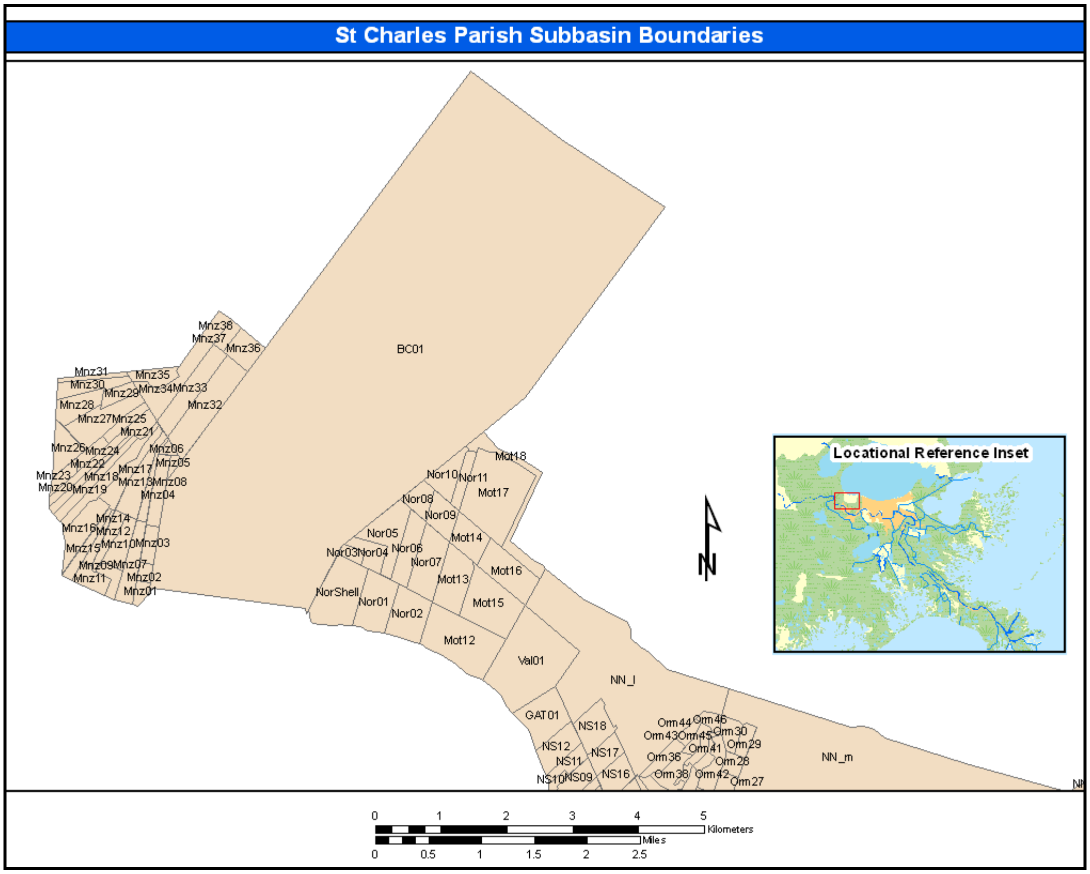

Figure 9-1. Subbasin boundaries western half of parish

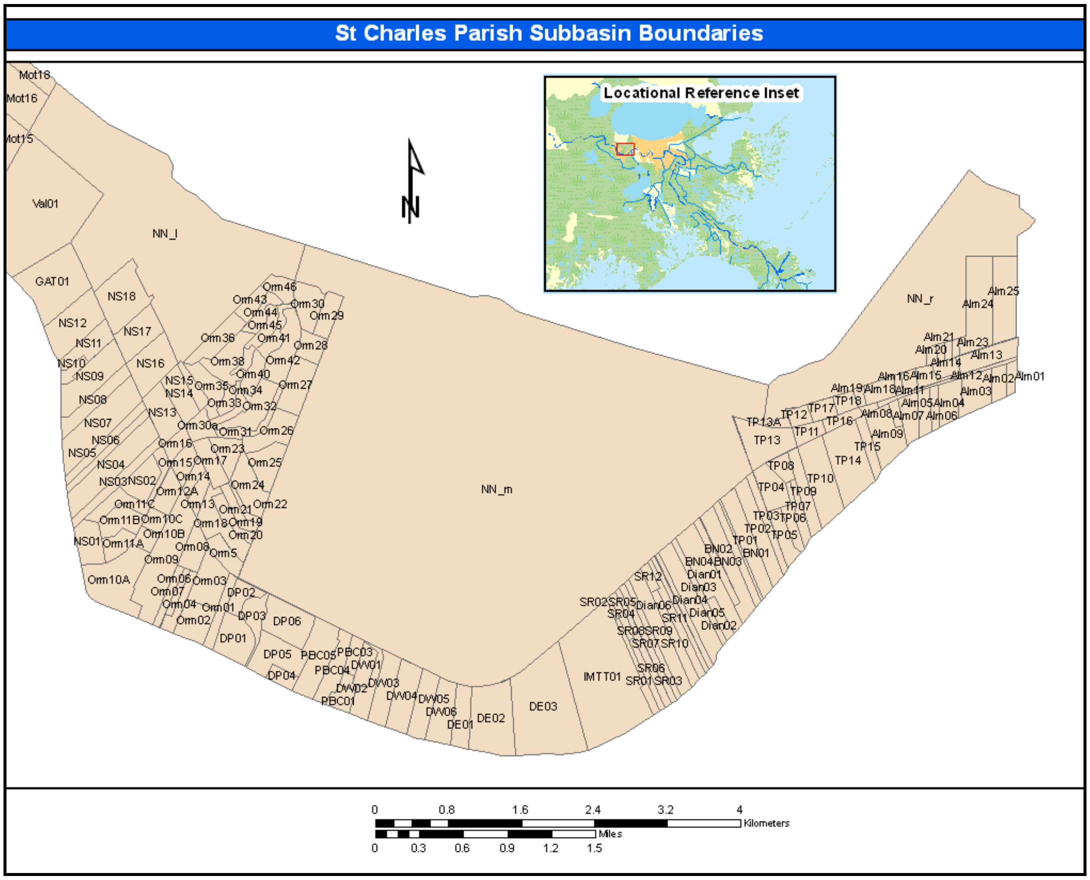

Figure 9-1. Subbasin boundaries eastern half of parish

Landuse and Soil Data

Landuse and soil data were used to estimate SCS curve numbers. Landuse data was obtained from the New Orleans District (MVN). The landuse data was a raster coverage of 24 different landuse types (Table 9-2, Landuse Categories). Soil data, contained in the Soil Survey Geographic (SSURGO) Database, was downloaded from the following National Resources Conservation Service (NRCS) website: http://www.ncgc.nrcs.usda.gov/products/datasets/ssurgo/. The SSURGO dataset is a digital copy of county soil survey maps and provides the most level of detailed for digital soil maps from the NRCS.

Table 9-2. Landuse categories

LANDUSE	A	B	C	D
Fresh Marsh	39	61	74	80
Intermediate Marsh	39	61	74	80
Brackish Marsh	39	61	74	80
Saline Marsh	39	61	74	80
Wetland Forest-Deciduous	43	65	76	82
Wetland Forest- Evergreen	49	69	79	84
Wetland Forest- Mixed	39	61	74	80
Upland Forest- Deciduous	32	58	72	79
Upland Forest- Evergreen	43	65	76	82
Upland Forest- Mixed	39	61	74	80
Dense Pine Thicket	32	58	72	79
Wetland Scrub/shrub - deciduous	30	48	65	73
Wetland Scrub/Shrub - evergreen	35	56	70	77
Wetland Scrub/Shrub - Mixed	30	55	68	75
Upland Scrub/Shrub - Deciduous	30	48	65	73
Upland Scrub/Shrub - Evergreen	35	56	70	77
Upland Scrub/Shrub - Mixed	30	55	68	75
Agriculture-Cropland-Grassland	49	69	79	84
Vegetated Urban	49	69	79	84
Non-Vegetated Urban	71	80	87	91
Upland Barren	77	86	91	94
Wetland Barren	68	79	86	89
Wetland Complex	85	85	85	85
Water	100	100	100	100

Loss Rates

Loss rates are used to account for the amount of precipitation intercepted by the canopy and depressions on the land surface and the amount of precipitation that infiltrates into the soil. Precipitation that is not lost to interception or infiltration is called "excess precipitation" and becomes direct runoff. The Soil Conservation Service (SCS) Curve Number (CN) method was used to model interception and infiltration. The SCS CN method estimates precipitation loss and excess as a function of cumulative precipitation, soil cover, landuse, and antecedent moisture. This method uses a single parameter, a curve number, to estimate the amount of precipitation excess\loss from a storm event. Studies have been carried out to determine appropriate curve number values for combinations of landuse type and condition, soil type, and the moisture state of the watershed.

Table 9-2 was used to estimate a curve number value for each combination of landuse and soil type in the study area. The hydrologic soil group (A, B, C, or D) is one of the soil properties contained in the SSURGO database. The percent impervious cover is already included in the curve number value in Table 9-2. More information about the background and use in the SCS curve number method can be found in Soil Conservation Service (1971, 1986). Figure 9-3 and Figure 9-4 show landuse types and hydrologic soil groups, respectively, in St Charles Parish. The ArcGIS map calculator was used to create a raster coverage of curve numbers, Figure 9-5, from these two data sets and the curve number lookup table. Subbasin average curve numbers were computed for each subbasin using the subbasin boundary shapefile and the curve number raster coverage, Table 9-3.

Table 9-3 Subbasin average curve numbers

Subbasin Name	Subbasin Average Curve Number	Subbasin Name	Subbasin Average Curve Number
Alm01	84.5	IMTT01	83.5
Alm02	84.9	Mnz01	77.8
Alm03	84.0	Mnz02	77.3
Alm04	81.9	Mnz03	75.6
Alm05	80.4	Mnz04	77.8
Alm06	79.4	Mnz05	76.2
Alm07	79.7	Mnz06	77.8
Alm08	77.3	Mnz07	74.7
Alm09	78.7	Mnz08	76.4
Alm10	81.2	Mnz09	84.7
Alm11	81.4	Mnz10	74.8
Alm12	81.1	Mnz11	83.5
Alm13	89.1	Mnz12	81.6
Alm14	89.7	Mnz13	66.8
Alm15	89.1	Mnz14	83.4
Alm16	90.2	Mnz15	82.8
Alm17	84.0	Mnz16	75.3
Alm18	82.7	Mnz17	74.6
Alm19	89.3	Mnz18	78.7
Alm20	88.1	Mnz19	77.8
Alm21	88.3	Mnz20	76.1
Alm22	88.4	Mnz21	76.2
Alm23	85.8	Mnz22	78.7
Alm24	88.5	Mnz23	75.8
Alm25	87.8	Mnz24	78.2
BC01	76.2	Mnz25	76.0
BN01	79.1	Mnz26	76.6
BN02	79.0	Mnz27	76.5
BN03	79.0	Mnz28	76.0
BN04	79.0	Mnz29	81.7
DE01	79.7	Mnz30	78.2
DE02	82.2	Mnz31	78.2
DE03	79.0	Mnz32	76.2
Dian01	79.0	Mnz33	67.2
Dian02	80.4	Mnz34	76.2
Dian03	79.8	Mnz35	78.5
Dian04	81.9	Mnz36	76.2
Dian05	81.6	Mnz37	69.0
Dian06	81.0	Mnz38	80.6
DP01	78.6	Mot12	89.7
DP02	76.9	Mot13	91.0
DP03	78.6	Mot14	91.0
DP04	78.7	Mot15	91.0
DP05	76.6	Mot16	91.1
DP06	76.1	Mot17	91.6
DW01	81.4	Mot18	84.6
DW02	83.7	Nor01	78.3
DW03	82.4	Nor02	79.3
DW04	82.5	Nor03	78.3
DW05	81.8	Nor04	76.8
DW06	82.9	Nor05	79.0
GAT01	89.7	Nor06	79.0

Table 9-3 Continued

Subbasin Name	Subbasin Average Curve Number	Subbasin Name	Subbasin Average Curve Number
Nor07	79.8	Orm24	83.9
Nor08	88.5	Orm25	83.8
Nor09	79.8	Orm26	83.8
Nor10	83.3	Orm27	84.1
Nor11	85.3	Orm28	84.5
NorShell	89.2	Orm29	85.3
NS01	81.3	Orm30	84.7
NS02	84.0	Orm30A	82.8
NS03	83.4	Orm31	84.5
NS04	83.2	Orm32	84.9
NS05	83.4	Orm33	84.8
NS06	83.3	Orm34	86.1
NS07	82.1	Orm35	83.2
NS08	82.2	Orm36	83.9
NS09	82.1	Orm38	84.5
NS10	80.0	Orm39	85.5
NS11	80.7	Orm40	84.1
NS12	82.8	Orm41	85.7
NS13	81.8	Orm42	84.8
NS14	81.3	Orm43	84.0
NS15	80.7	Orm44	84.0
NS16	82.5	Orm45	84.0
NS17	83.1	Orm46	84.0
NS18	81.8	PBC01	78.4
Orm01	81.0	PBC02	81.0
Orm02	81.2	PBC03	78.5
Orm03	83.6	PBC04	78.9
Orm04	81.8	PBC05	76.9
Orm05	82.5	SR01	79.6
Orm06	83.3	SR02	78.0
Orm07	82.6	SR03	79.8
Orm08	84.0	SR04	77.5
Orm09	83.3	SR05	76.6
Orm10A	81.4	SR06	79.9
Orm10B	84.0	SR07	79.6
Orm10C	84.0	SR08	79.0
Orm11A	84.0	SR09	76.3
Orm11B	84.0	SR10	78.9
Orm11C	84.0	SR11	80.2
Orm12A	84.0	SR12	76.0
Orm12B	84.0	TP01	80.4
Orm13	83.1	TP02	79.8
Orm14	84.0	TP03	82.0
Orm15	84.0	TP04	77.1
Orm16	84.0	TP05	81.5
Orm17	83.8	TP06	81.5
Orm18	82.3	TP07	81.3
Orm19	82.1	TP08	76.4
Orm20	82.1	TP09	79.9
Orm21	83.8	TP10	81.0
Orm22	83.9	TP11	85.6
Orm23	84.0	TP12	87.9

Table 9-3 Continued

Subbasin Name	Subbasin Average Curve Number
TP13	86.3
TP13A	88.4
TP14	80.1
TP15	78.6
TP16	80.8
TP17	89.1
TP18	88.6
Val01	90.9

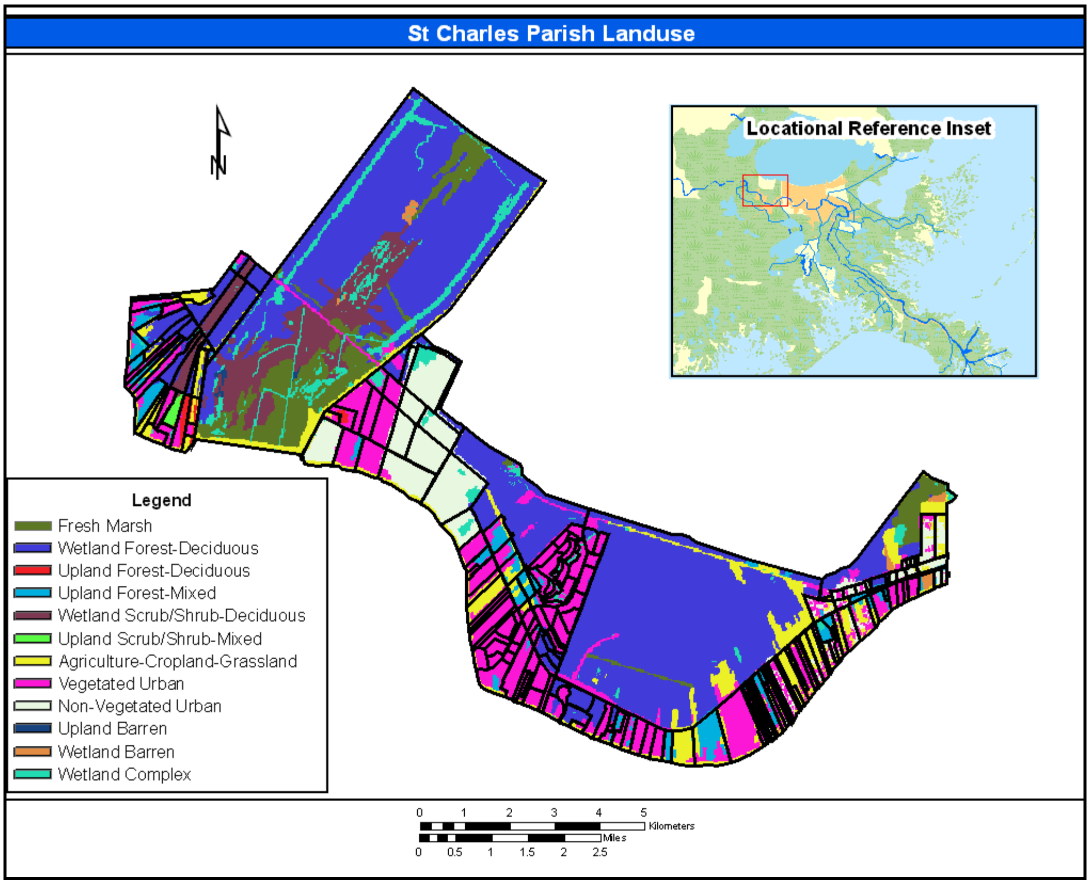

Figure 9-2. Landuse types in St Charles Parish

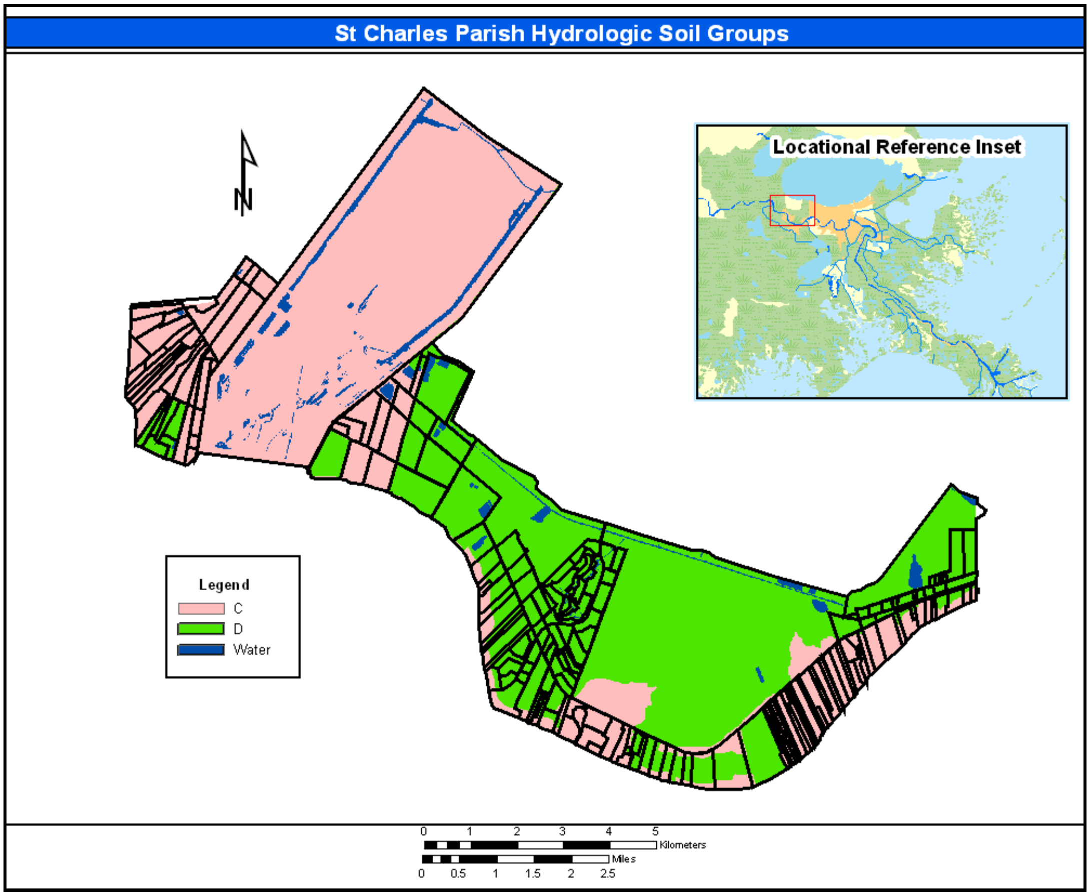

Figure 9-3. Hydrologic soil groups in St Charles Parish

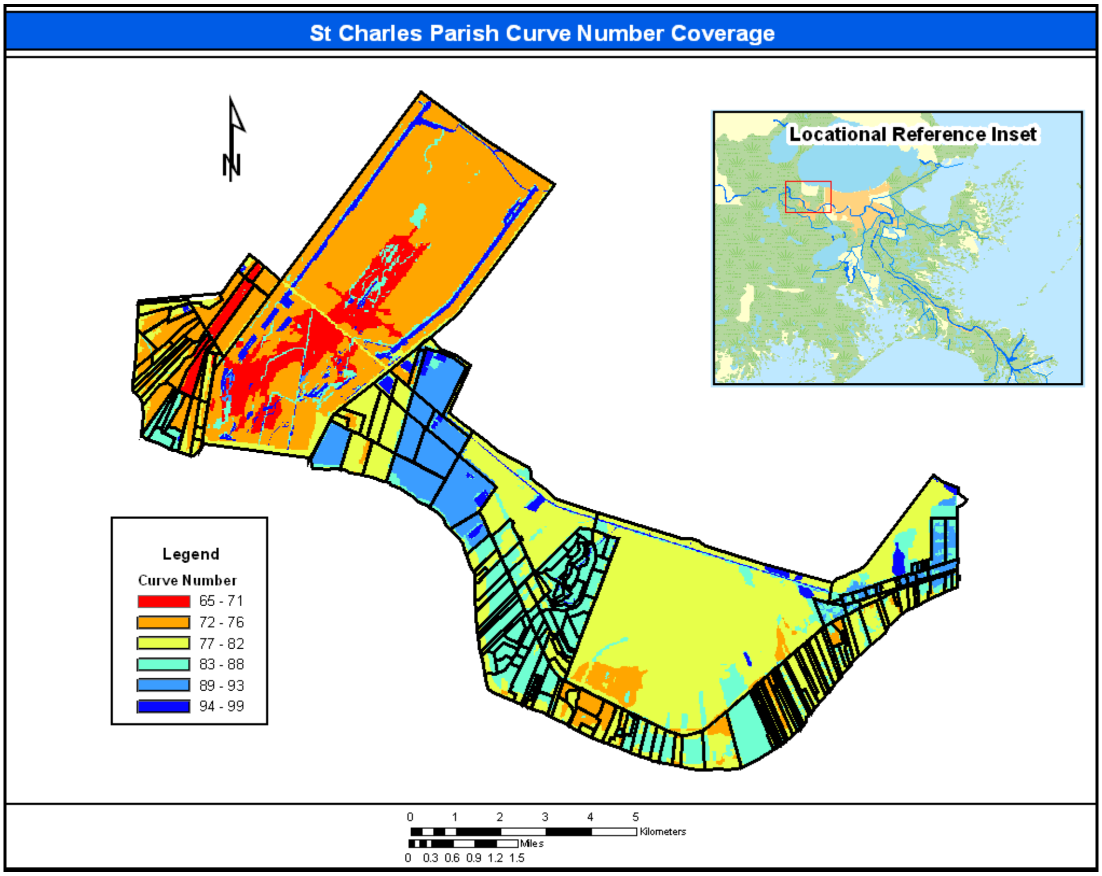

Figure 9-4. Curve number coverage

Transform

Excess precipitation was transformed to a runoff hydrograph using the SCS unit hydrograph method. The SCS developed a dimensionless unit hydrograph after analyzing unit hydrographs from a number of small, gaged watersheds. The dimensionless unit hydrograph is used to develop a unit hydrograph given drainage area and lag time. A detailed description of the SCS dimensionless unit hydrograph can be found in SCS Technical Report 55 (1986) and the National Engineering Handbook (1971).

Lag times for the SCS unit hydrograph method were estimated using the following equation:

$$t_l = \frac{L^{0.8} * (1000 - 9CN)^{0.7}}{1900 * CN^{0.7} * Y^{0.5}}$$

where t_l is the subbasin lag (hr), L is the hydraulic length (ft), CN is the subbasin average curve number, and Y is the average subbasin land slope (percent). The hydraulic length was determined visually using topographic maps of St Charles Parish. Terrain Data, 30 meter DEMs, were used to compute the average land slope for each subbasin. Computed lag times are shown in Table 9-4.

Table 9-4. Basin lag time

Subbasin Name	Hydraulic Length (ft)	Average Subbasin Land Slope %	Lag Time (minutes)
Alm01	1097	0.016	139
Alm02	1057	0.024	109
Alm03	1215	0.056	83
Alm04	1404	0.044	112
Alm05	1449	0.017	194
Alm06	1528	0.028	164
Alm07	1579	0.040	140
Alm08	790	0.011	162
Alm09	1634	0.074	108
Alm10	526	0.007	135
Alm11	2669	0.014	342
Alm12	2257	0.010	357
Alm13	604	0.057	39
Alm14	764	0.019	79
Alm15	801	0.021	81
Alm16	590	0.015	71
Alm17	175	0.007	50
Alm18	922	0.030	94
Alm19	529	0.010	85
Alm20	642	0.009	107
Alm21	562	0.007	109
Alm22	604	0.007	118
Alm23	572	0.006	134
Alm24	3463	0.126	108
Alm25	3133	0.101	115
BC01	28000	10.944	93
BN01	3842	0.011	540
BN02	3846	0.041	285
BN03	3861	0.083	201
BN04	3232	0.034	273
DE01	1942	0.049	147
DE02	2527	0.145	98
DE03	3596	0.294	101
Dian01	3964	0.102	185
Dian02	1491	0.019	187
Dian03	4112	0.032	330
Dian04	4135	0.099	178
Dian05	4470	0.034	328
Dian06	4515	0.028	369
DP01	1188	0.072	85
DP02	920	0.035	106
DP03	1221	0.039	118
DP04	590	0.022	88
DP05	1792	0.079	120
DP06	1781	0.086	116
DW01	1098	0.030	113
DW02	2071	0.046	142
DW03	1602	0.064	102
DW04	1723	0.075	99
DW05	1470	0.063	98
DW06	1783	0.068	106

Table 9-4. Continued

Subbasin Name	Hydraulic Length (ft)	Average Subbasin Land Slope %	Lag Time (minutes)
GAT01	2149	0.165	62
IMTT01	4972	0.390	98
Mnz01	931	0.026	118
Mnz02	1015	0.023	139
Mnz03	1783	0.047	159
Mnz04	2512	0.086	146
Mnz05	912	0.015	163
Mnz06	706	0.014	130
Mnz07	995	0.109	67
Mnz08	415	0.044	50
Mnz09	1632	0.042	119
Mnz10	2471	0.079	163
Mnz11	1027	0.059	72
Mnz12	698	0.055	58
Mnz13	3452	0.117	218
Mnz14	420	0.034	46
Mnz15	536	0.053	46
Mnz16	3255	0.124	160
Mnz17	895	0.091	68
Mnz18	2563	0.115	125
Mnz19	2088	0.072	137
Mnz20	2016	0.042	183
Mnz21	847	0.063	75
Mnz22	2472	0.065	160
Mnz23	1489	0.038	152
Mnz24	1858	0.035	176
Mnz25	2605	0.046	216
Mnz26	2037	0.081	131
Mnz27	3894	0.157	159
Mnz28	2213	0.092	134
Mnz29	666	0.044	62
Mnz30	3620	0.076	204
Mnz31	3419	0.031	307
Mnz32	6273	0.247	187
Mnz33	4795	0.158	242
Mnz34	1661	0.148	83
Mnz35	957	0.060	78
Mnz36	2985	0.083	178
Mnz37	534	0.034	86
Mnz38	656	0.039	68
Mot12	2199	0.310	46
Mot13	1423	0.157	43
Mot14	1613	0.106	58
Mot15	1803	0.236	42
Mot16	1322	0.168	39
Mot17	3815	0.391	58
Mot18	987	0.129	45
Nor01	2269	0.166	95
Nor02	1978	0.155	85
Nor03	976	0.027	119
Nor04	857	0.067	72

Table 9-4. Continued

Subbasin Name	Hydraulic Length (ft)	Average Subbasin Land Slope %	Lag Time (minutes)
Nor05	2685	0.164	107
Nor06	760	0.110	48
Nor07	1301	0.165	58
Nor08	1276	0.065	68
Nor09	1378	0.085	85
Nor10	2540	0.091	120
Nor11	2709	0.053	154
NorShell	3172	0.245	70
NS01	1501	0.052	111
NS02	1294	0.052	91
NS03	4274	0.048	248
NS04	4079	0.124	150
NS05	3887	0.039	257
NS06	3825	0.041	247
NS07	3499	0.136	132
NS08	3087	0.058	182
NS09	2593	0.102	120
NS10	325	0.017	60
NS11	954	0.052	79
NS12	1785	0.104	86
NS13	800	0.065	59
NS14	654	0.054	56
NS15	808	0.023	103
NS16	1562	0.088	84
NS17	1535	0.070	91
NS18	2149	0.114	98
Orm01	2675	0.034	220
Orm02	1393	0.058	99
Orm03	1941	0.072	108
Orm04	2206	0.027	208
Orm05	1663	0.019	186
Orm06	2213	0.022	222
Orm07	1366	0.042	105
Orm08	3941	0.167	126
Orm09	2456	0.136	101
Orm10A	823	0.021	98
Orm10B	731	0.021	90
Orm10C	1059	0.033	97
Orm11A	1501	0.049	104
Orm11B	1348	0.041	105
Orm11C	890	0.039	77
Orm12A	329	0.022	46
Orm12B	907	0.023	105
Orm13	880	0.019	111
Orm14	807	0.031	80
Orm15	823	0.024	93
Orm16	391	0.043	39
Orm17	1238	0.069	80
Orm18	922	0.020	119
Orm19	585	0.010	118
Orm20	2685	0.164	107

Table 9-4. Continued

Subbasin Name	Hydraulic Length (ft)	Average Subbasin Land Slope %	Lag Time (minutes)
Orm21	1532	0.028	143
Orm22	1383	0.012	197
Orm23	1291	0.039	105
Orm24	2209	0.065	124
Orm25	1850	0.055	118
Orm26	2248	0.061	131
Orm27	2180	0.067	120
Orm28	1383	0.035	114
Orm29	1267	0.038	99
Orm30	938	0.053	67
Orm30A	598	0.030	67
Orm31	808	0.032	78
Orm32	1794	0.065	102
Orm33	902	0.025	95
Orm34	583	0.018	76
Orm35	1181	0.048	90
Orm36	2240	0.054	138
Orm38	2923	0.066	151
Orm39	803	0.014	114
Orm40	1238	0.031	113
Orm41	889	0.030	83
Orm42	2485	0.072	126
Orm43	1247	0.027	121
Orm44	1697	0.028	152
Orm45	1581	0.028	143
Orm46	1065	0.030	101
PBC01	1433	0.036	124
PBC02	757	0.025	101
PBC03	339	0.009	82
PBC04	942	0.022	127
PBC05	1690	0.057	126
SR01	2212	0.095	128
SR02	3649	0.026	338
SR03	1620	0.004	461
SR04	740	0.023	99
SR05	2675	0.018	335
SR06	1531	0.011	281
SR07	2718	0.026	263
SR08	4484	0.040	320
SR09	4601	0.038	341
SR10	4608	0.086	247
SR11	3523	0.043	260
SR12	4150	0.045	278
TP01	884	0.014	167
TP02	3793	0.081	193
TP03	3759	0.074	203
TP04	3710	0.040	256
TP05	1639	0.051	137
TP06	1858	0.037	155
TP07	1506	0.030	145
TP08	2070	0.025	206

Table 9-4. Concluded

Subbasin Name	Hydraulic Length (ft)	Average Subbasin Land Slope %	Lag Time (minutes)
TP09	2950	0.050	204
TP10	2613	0.107	122
TP11	1005	0.026	99
TP12	806	0.020	87
TP13	1984	0.090	89
TP13A	364	0.009	67
TP14	2385	0.127	107
TP15	2017	0.029	204
TP16	1321	0.022	156
TP17	975	0.024	88
TP18	863	0.020	89
Val01	4217	0.485	58

Rainfall Data

Radar rainfall data, referred to as Multisensor Precipitation Estimator (MPE), was used as a boundary condition in the HEC-HMS model to determine runoff hydrographs produced by the Hurricane Katrina event. MPE data from the Lower Mississippi River Forecast Center (LMRFC) was downloaded from the following website: http://dipper.nws.noaa.gov/hdsb/data/nexrad/lmrfc_mpe.php. Raw radar data is adjusted using rain gage measurements and possibly satellite data to produce the MPE product.

The radar-rainfall data was imported into a GIS program. The GIS program was used to compute subbasin average precipitation; the downloaded radar-rainfall data was a raster or gridded coverage of precipitation. Also, the downloaded radar-rainfall data provides hourly estimates of precipitation. A precipitation hyetograph was computed for each subbasin in the St Charles Parish basin model. The individual hyetographs were imported into an HEC-DSS file where they were read by HEC-HMS. Total rainfall from Hurricane Katrina varied from 7 to 9 inches across subbasin in St Charles Parish, Figure 9-6. As an example, the precipitation hyetograph for the "ALM07" subbasin is shown in Figure 9-7. This figure shows the time distribution of rainfall from Hurricane Katrina.

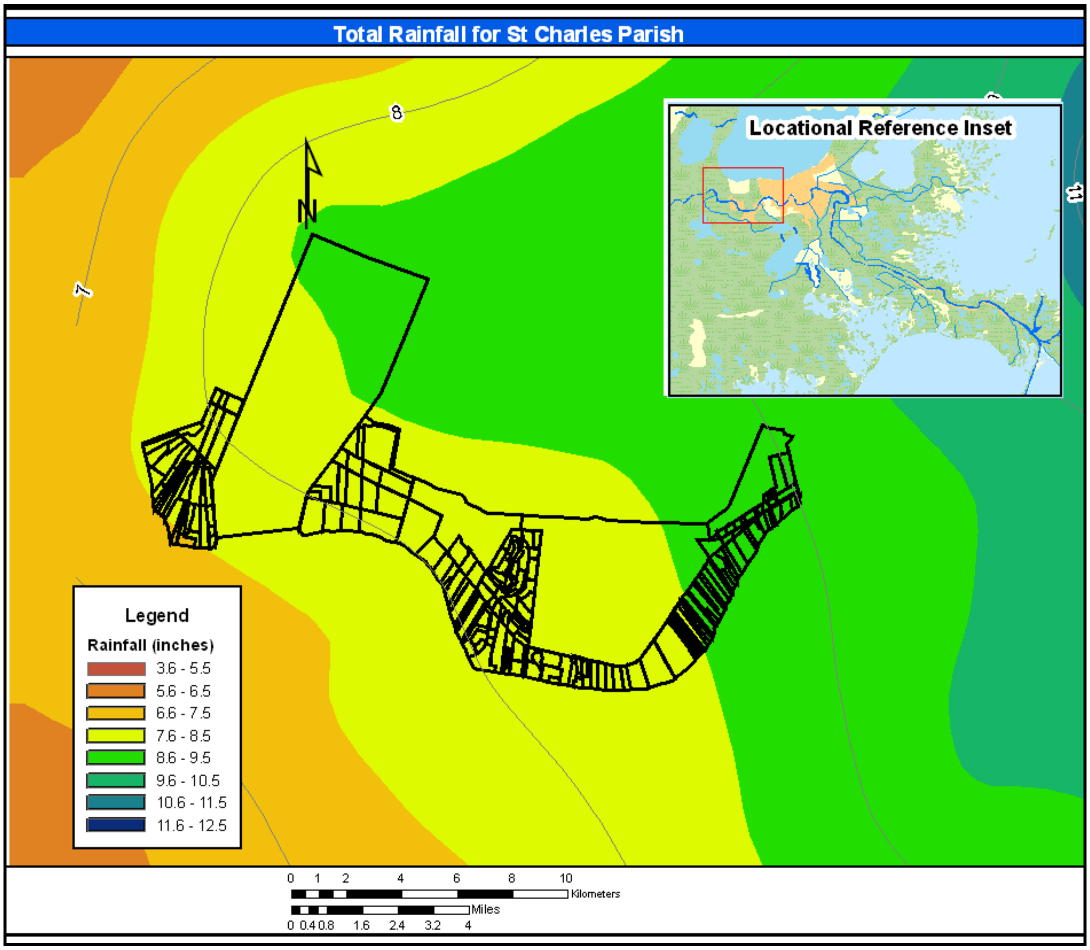

Figure 9-5. Total storm precipitation

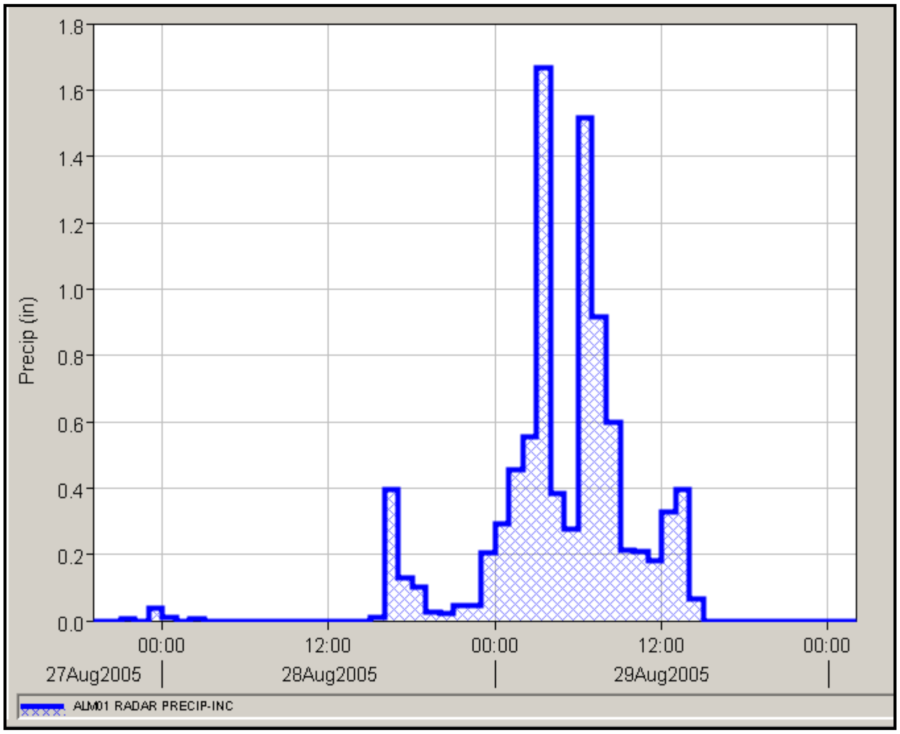

Figure 9-6. Average rainfall for ALM01 subbasin

Model Results

Summary output from the HEC-HMS model is available in Table 9-5. A complete runoff hydrograph was also computed by the program. This information was stored in an HEC-DSS file and provided as inflows to storage areas for the HEC-RAS model of St Charles Parish.

Table 9-5. Summary output from HEC-HMS model

Subbasin Name	Drainage Area (mi2)	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
Alm01	0.0164	9	29Aug2005, 09:19	7.2
Alm02	0.0243	14	29Aug2005, 08:48	7.2
Alm03	0.0556	35	29Aug2005, 08:16	7.1
Alm04	0.0439	25	29Aug2005, 08:52	6.8
Alm05	0.0171	8	29Aug2005, 10:10	6.7
Alm06	0.0276	13	29Aug2005, 09:45	6.5
Alm07	0.0395	20	29Aug2005, 09:22	6.6
Alm08	0.0111	5	29Aug2005, 09:44	6.3
Alm09	0.0742	41	29Aug2005, 08:48	6.4
Alm10	0.0066	3	29Aug2005, 09:15	6.4
Alm11	0.0136	5	29Aug2005, 12:03	6.8
Alm12	0.0098	4	29Aug2005, 12:19	6.7
Alm13	0.0566	48	29Aug2005, 04:16	7.7
Alm14	0.0191	13	29Aug2005, 08:10	7.8
Alm15	0.0205	14	29Aug2005, 08:12	7.7
Alm16	0.0152	11	29Aug2005, 07:59	7.9
Alm17	0.0069	5	29Aug2005, 07:33	7.1
Alm18	0.0302	17	29Aug2005, 08:29	6.8
Alm19	0.0096	6	29Aug2005, 04:59	7.1
Alm20	0.0089	5	29Aug2005, 08:45	7.6
Alm21	0.0069	4	29Aug2005, 08:47	7.6
Alm22	0.0066	4	29Aug2005, 08:57	7.6
Alm23	0.0056	3	29Aug2005, 09:14	7.3
Alm24	0.1257	77	29Aug2005, 08:46	7.7
Alm25	0.1008	60	29Aug2005, 08:54	7.6
BC01	10.9440	3953	29Aug2005, 08:38	5.7
BN01	0.0113	3	29Aug2005, 15:57	5.8
BN02	0.0412	14	29Aug2005, 11:23	5.8
BN03	0.0828	32	29Aug2005, 10:20	5.8
BN04	0.0338	12	29Aug2005, 11:15	5.8
DE01	0.0495	21	29Aug2005, 09:32	5.8
DE02	0.1451	74	29Aug2005, 08:35	6.1
DE03	0.2938	143	29Aug2005, 08:40	5.8
Dian01	0.1021	41	29Aug2005, 10:06	5.8
Dian02	0.0192	8	29Aug2005, 10:07	6.0
Dian03	0.0325	10	29Aug2005, 12:00	5.9
Dian04	0.0986	42	29Aug2005, 09:58	6.1
Dian05	0.0336	11	29Aug2005, 11:54	6.1
Dian06	0.0279	9	29Aug2005, 12:37	6.0
DP01	0.0725	35	29Aug2005, 08:13	5.9
DP02	0.0346	15	29Aug2005, 08:40	5.6
DP03	0.0392	17	29Aug2005, 08:54	5.9
DP04	0.0218	10	29Aug2005, 08:16	5.9
DP05	0.0792	33	29Aug2005, 08:57	5.6
DP06	0.0863	37	29Aug2005, 08:53	5.6
DW01	0.0304	14	29Aug2005, 08:47	6.2
DW02	0.0456	20	29Aug2005, 09:18	6.5
DW03	0.0638	31	29Aug2005, 08:33	6.3
DW04	0.0753	36	29Aug2005, 08:29	6.3
DW05	0.0626	30	29Aug2005, 08:28	6.2
DW06	0.0683	34	29Aug2005, 08:43	6.2

Table 9-5. Continued

Subbasin Name	Drainage Area (mi2)	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
GAT01	0.1648	84	29Aug2005, 07:47	6.2
IMTT01	0.3904	203	29Aug2005, 08:35	6.3
Mnz01	0.0264	7	29Aug2005, 09:20	4.9
Mnz02	0.0229	6	29Aug2005, 09:49	5.1
Mnz03	0.0471	12	29Aug2005, 10:23	4.9
Mnz04	0.0859	24	29Aug2005, 10:01	5.1
Mnz05	0.0147	4	29Aug2005, 10:29	4.9
Mnz06	0.0140	4	29Aug2005, 09:35	5.1
Mnz07	0.1093	33	29Aug2005, 08:31	4.7
Mnz08	0.0436	14	29Aug2005, 08:17	4.9
Mnz09	0.0415	13	29Aug2005, 09:18	5.8
Mnz10	0.0792	20	29Aug2005, 10:31	4.8
Mnz11	0.0585	20	29Aug2005, 08:33	5.6
Mnz12	0.0546	19	29Aug2005, 08:22	5.5
Mnz13	0.1172	24	29Aug2005, 11:39	3.9
Mnz14	0.0341	12	29Aug2005, 08:13	5.8
Mnz15	0.0535	19	29Aug2005, 08:13	5.7
Mnz16	0.1237	32	29Aug2005, 10:25	4.8
Mnz17	0.0909	28	29Aug2005, 08:31	4.7
Mnz18	0.1147	33	29Aug2005, 09:27	5.2
Mnz19	0.0721	20	29Aug2005, 09:45	5.1
Mnz20	0.0422	11	29Aug2005, 10:55	4.9
Mnz21	0.0627	19	29Aug2005, 08:37	4.9
Mnz22	0.0650	18	29Aug2005, 10:21	5.2
Mnz23	0.0382	10	29Aug2005, 10:13	4.9
Mnz24	0.0354	9	29Aug2005, 10:43	5.2
Mnz25	0.0459	11	29Aug2005, 11:29	4.9
Mnz26	0.0809	22	29Aug2005, 09:37	5.0
Mnz27	0.1565	42	29Aug2005, 10:22	5.0
Mnz28	0.0915	25	29Aug2005, 09:42	4.9
Mnz29	0.0440	15	29Aug2005, 08:25	5.6
Mnz30	0.0759	20	29Aug2005, 11:15	5.2
Mnz31	0.0307	7	29Aug2005, 12:45	5.2
Mnz32	0.2469	68	29Aug2005, 10:48	5.3
Mnz33	0.1579	32	29Aug2005, 11:59	3.9
Mnz34	0.1479	44	29Aug2005, 08:43	4.9
Mnz35	0.0603	19	29Aug2005, 08:39	5.2
Mnz36	0.0833	25	29Aug2005, 10:05	5.6
Mnz37	0.0338	10	29Aug2005, 14:38	4.4
Mnz38	0.0386	13	29Aug2005, 14:23	5.1
Mot12	0.3105	188	29Aug2005, 04:23	7.2
Mot13	0.1572	99	29Aug2005, 04:20	7.3
Mot14	0.1056	60	29Aug2005, 07:41	7.3
Mot15	0.2360	150	29Aug2005, 04:19	7.3
Mot16	0.1681	110	29Aug2005, 04:17	7.4
Mot17	0.3906	223	29Aug2005, 07:41	7.4
Mot18	0.1294	75	29Aug2005, 07:27	6.6
Nor01	0.1662	61	29Aug2005, 08:42	6.0
Nor02	0.1550	63	29Aug2005, 08:26	6.1
Nor03	0.0272	9	29Aug2005, 09:08	6.0
Nor04	0.0675	26	29Aug2005, 08:19	5.8

Table 9-5. Continued

Subbasin Name	Drainage Area (mi2)	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
Nor05	0.1637	59	29Aug2005, 08:55	6.1
Nor06	0.1101	47	29Aug2005, 07:42	6.1
Nor07	0.1649	77	29Aug2005, 07:46	6.1
Nor08	0.0648	29	29Aug2005, 08:12	7.3
Nor09	0.0849	36	29Aug2005, 08:22	6.1
Nor10	0.0907	36	29Aug2005, 09:03	6.5
Nor11	0.0533	22	29Aug2005, 09:33	6.7
NorShell	0.2446	108	29Aug2005, 08:14	7.3
NS01	0.0517	20	29Aug2005, 08:50	5.2
NS02	0.0515	24	29Aug2005, 08:22	6.0
NS03	0.0483	15	29Aug2005, 11:16	5.5
NS04	0.1240	44	29Aug2005, 09:33	5.4
NS05	0.0387	12	29Aug2005, 11:25	5.4
NS06	0.0413	13	29Aug2005, 11:17	5.4
NS07	0.1360	50	29Aug2005, 09:13	5.3
NS08	0.0579	19	29Aug2005, 10:14	5.3
NS09	0.1020	39	29Aug2005, 09:00	5.3
NS10	0.0166	7	29Aug2005, 07:46	5.0
NS11	0.0516	22	29Aug2005, 08:11	5.1
NS12	0.1042	44	29Aug2005, 08:20	5.3
NS13	0.0652	34	29Aug2005, 07:42	5.8
NS14	0.0537	31	29Aug2005, 07:37	6.1
NS15	0.0231	11	29Aug2005, 08:35	6.1
NS16	0.0884	38	29Aug2005, 08:17	5.3
NS17	0.0705	29	29Aug2005, 08:26	5.4
NS18	0.1144	46	29Aug2005, 08:34	5.3
Orm01	0.0341	13	29Aug2005, 10:39	6.1
Orm02	0.0582	28	29Aug2005, 08:30	6.2
Orm03	0.0718	35	29Aug2005, 08:41	6.4
Orm04	0.0266	10	29Aug2005, 10:27	6.2
Orm05	0.0357	14	29Aug2005, 10:05	6.3
Orm06	0.0191	7	29Aug2005, 10:37	6.4
Orm07	0.0223	11	29Aug2005, 08:37	6.3
Orm08	0.0420	19	29Aug2005, 09:01	6.5
Orm09	0.1669	82	29Aug2005, 08:32	6.4
Orm10A	0.1360	60	29Aug2005, 08:32	5.7
Orm10B	0.0213	11	29Aug2005, 08:19	6.5
Orm10C	0.0211	11	29Aug2005, 08:27	6.5
Orm11A	0.0327	14	29Aug2005, 08:41	5.5
Orm11B	0.0493	21	29Aug2005, 08:41	5.7
Orm11C	0.0412	19	29Aug2005, 08:07	5.6
Orm12A	0.0389	25	29Aug2005, 07:26	6.5
Orm12B	0.0223	11	29Aug2005, 08:37	6.5
Orm13	0.0232	11	29Aug2005, 08:44	6.4
Orm14	0.0186	10	29Aug2005, 08:06	6.5
Orm15	0.0312	16	29Aug2005, 08:22	6.5
Orm16	0.0239	16	29Aug2005, 07:20	6.5
Orm17	0.0426	23	29Aug2005, 08:06	6.5
Orm18	0.0690	32	29Aug2005, 08:54	6.3
Orm19	0.0199	9	29Aug2005, 08:53	6.3
Orm20	0.0097	4	29Aug2005, 09:21	6.3

Table 9-5. Continued

Subbasin Name	Drainage Area (mi2)	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
Orm21	0.0276	11	29Aug2005, 10:15	6.5
Orm22	0.0122	6	29Aug2005, 08:37	6.5
Orm23	0.0386	18	29Aug2005, 08:59	6.5
Orm24	0.0653	31	29Aug2005, 08:52	6.5
Orm25	0.0549	25	29Aug2005, 09:07	6.5
Orm26	0.0606	28	29Aug2005, 08:55	6.5
Orm27	0.0673	32	29Aug2005, 08:47	6.5
Orm28	0.0346	17	29Aug2005, 08:29	6.6
Orm29	0.0380	22	29Aug2005, 07:52	6.5
Orm30	0.0531	30	29Aug2005, 07:51	6.5
Orm30A	0.0300	16	29Aug2005, 08:06	6.2
Orm31	0.0316	16	29Aug2005, 08:33	6.6
Orm32	0.0648	33	29Aug2005, 08:24	6.6
Orm33	0.0250	14	29Aug2005, 08:01	6.6
Orm34	0.0176	9	29Aug2005, 08:18	6.7
Orm35	0.0481	21	29Aug2005, 09:15	6.4
Orm36	0.0538	23	29Aug2005, 09:28	6.5
Orm38	0.0664	32	29Aug2005, 08:47	6.6
Orm39	0.0136	7	29Aug2005, 08:46	6.7
Orm40	0.0306	16	29Aug2005, 08:10	6.5
Orm41	0.0301	14	29Aug2005, 09:01	6.7
Orm42	0.0722	34	29Aug2005, 08:56	6.6
Orm43	0.0273	12	29Aug2005, 09:29	6.4
Orm44	0.0281	13	29Aug2005, 09:20	6.4
Orm45	0.0284	14	29Aug2005, 08:33	6.5
Orm46	0.0303	14	29Aug2005, 09:02	6.3
PBC01	0.0254	11	29Aug2005, 08:32	5.9
PBC02	0.0090	5	29Aug2005, 08:09	6.2
PBC03	0.0224	10	29Aug2005, 09:05	5.8
PBC04	0.0565	24	29Aug2005, 09:03	5.9
PBC05	0.0946	39	29Aug2005, 09:06	5.7
SR01	0.0258	8	29Aug2005, 12:08	5.9
SR02	0.0042	1	29Aug2005, 14:31	5.7
SR03	0.0229	11	29Aug2005, 08:37	5.9
SR04	0.0183	6	29Aug2005, 12:10	5.6
SR05	0.0111	4	29Aug2005, 11:25	5.5
SR06	0.0262	9	29Aug2005, 11:06	5.9
SR07	0.0403	13	29Aug2005, 11:51	5.9
SR08	0.0381	12	29Aug2005, 12:13	5.8
SR09	0.0856	29	29Aug2005, 11:00	5.5
SR10	0.0431	15	29Aug2005, 11:05	5.8
SR11	0.0452	16	29Aug2005, 11:16	5.9
SR12	0.0135	5	29Aug2005, 09:52	5.4
TP01	0.0805	32	29Aug2005, 10:12	6.0
TP02	0.0745	29	29Aug2005, 10:21	5.9
TP03	0.0399	15	29Aug2005, 10:58	6.2
TP04	0.0511	22	29Aug2005, 09:21	5.6
TP05	0.0370	16	29Aug2005, 09:37	6.1
TP06	0.0301	14	29Aug2005, 09:27	6.1
TP07	0.0253	10	29Aug2005, 10:22	6.1
TP08	0.0233	10	29Aug2005, 09:10	5.5

Table 9-5. Continued

Subbasin Name	Drainage Area (mi2)	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
TP09	0.0496	19	29Aug2005, 10:22	5.9
TP10	0.1071	51	29Aug2005, 09:04	6.0
TP11	0.0257	13	29Aug2005, 05:15	6.7
TP12	0.0200	11	29Aug2005, 05:02	7.0
TP13	0.0904	49	29Aug2005, 08:21	6.7
TP13A	0.0090	6	29Aug2005, 04:42	7.0
TP14	0.1273	64	29Aug2005, 08:44	6.2
TP15	0.0292	12	29Aug2005, 10:18	6.3
TP16	0.0222	9	29Aug2005, 09:31	6.1
TP17	0.0240	14	29Aug2005, 05:03	7.1
TP18	0.0198	11	29Aug2005, 05:04	7.0
Val01	0.4855	273	29Aug2005, 07:41	7.2

RAS Interior Modeling

Background

Land in the St. Charles Parish is highest near the river and slopes away to the north and south. Figure 9-8 shows the east bank of St. Charles Parish in colored relief. The ribbon of land above sea level is plainly visible in brown. The marginal land close to sea level is shown in light brown, the marsh is light blue and Lake Pontchartrain is blue-green. The city of Kenner, in neighboring Jefferson Parish, is shown in dark blue indicating the area is substantially below sea level. Drainage is from the river, northward to the lake. The thin line of the Canadian National Railroad (CNNRR) forms a boundary between the traditionally dry land to the south (in brown) and the marsh to the north (light blue). The hurricane protection levee (HPL), Airline Highway, and the Kansas City Southern Railroad, cross the middle of the wetlands as a set of narrow brown lines. The Mississippi River is seen to be about the same elevation as the wetland but is often much higher. Lake Pontchartrain has a natural outlet to the Gulf of Mexico and is normally about one foot above mean sea level.

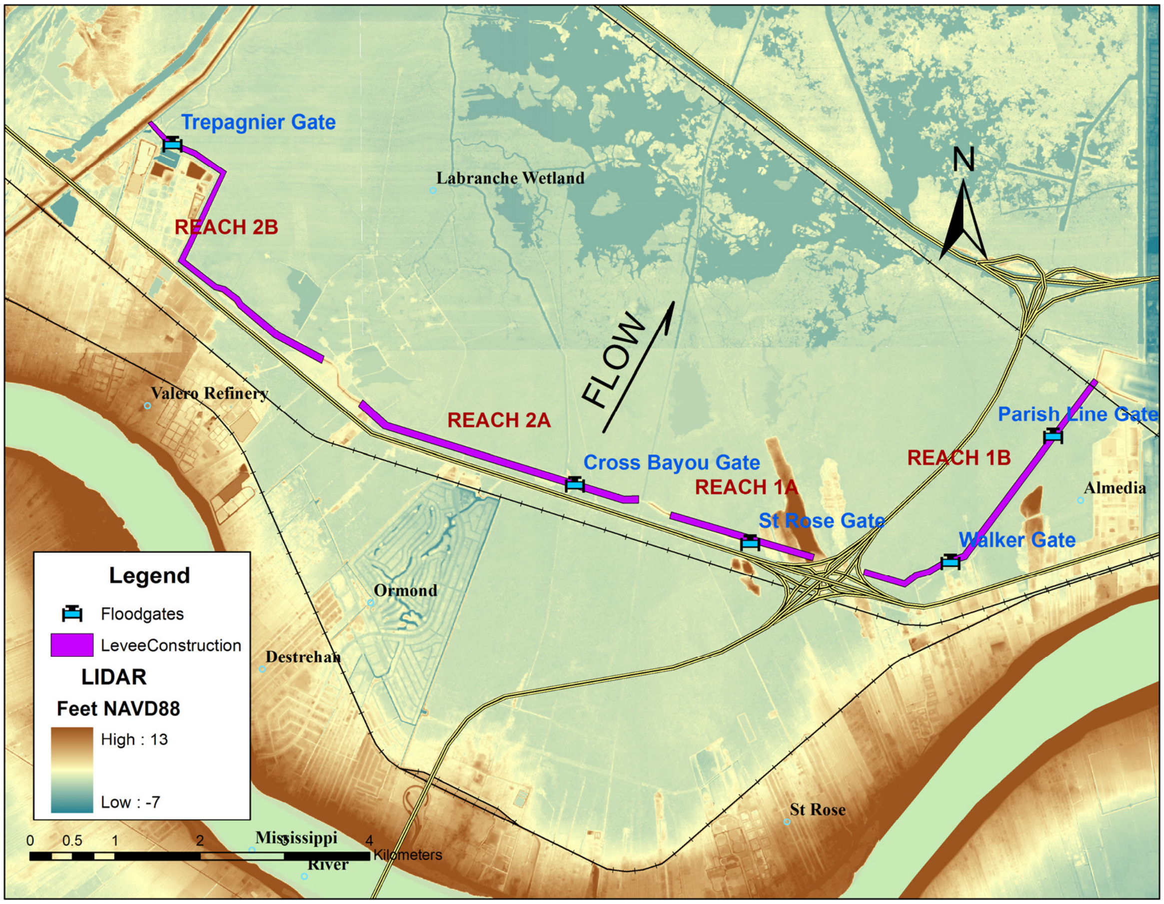

Figure 9-8. St Charles Parish, East Bank Levee Construction

Previous Studies

Master Drainage Plan

St. Charles Parish contracted Brown, Cunningham and Gannuch (BCG) to develop a master drainage plan for the east bank of the parish in 1990 and recommend drainage improvements to reduce frequent flooding. BCG produced a set of 1-ft contour maps of the basin, divided them into community-size sub-basins, and delineated hydrologic sub-units within each sub-basin based on land use to compute precipitation and runoff. They then made comparative runs of the model using the Corps' HEC-1 program to evaluate various improvement options. The resulting plan was published in December 1994 and is the basis for the hydrologic sub-unit delineations used in this study.

Destrehan Pump Station Model

A few years later, the parish contracted Burk-Kleinpeter, Inc. to do a follow-up study of the Ormond area using the detailed modeling capabilities of the Corps' UNET program, which models flow in rivers and canals. Although one-dimensional in design, UNET is capable of model the movement of water through a network of lakes, streams, and culverts and is capable of modeling reverse flow and duration of flooding. Their report is dated January 1998.

Hurricane Protection Levee

Congress first authorized construction of the Lake Pontchartrain and Vicinity, Louisiana Hurricane Protection Project in the Flood Control Act of 1965 to provide hurricane protection to areas around the lake in the parishes of Orleans, Jefferson, St. Bernard, and St. Charles. In 2002, the Federal Government, via the Corps of Engineers, began construction of a hurricane protection levee at an elevation of 13.0 feet NAVD from the Bonnet Carré Spillway to the west Jefferson back levee as shown in Figure 9-8 above. The levee is 15,200 meters long (9.4 miles) and contains five sets of flood gates to allow drainage to Lake Pontchartrain. The supporting feasibility study, however, found no federal interest in construction of pump stations along the levee to provide interior drainage when the flood gates are closed. The natural lowlands inside the levee are large enough to hold the 1%-chance rainfall with only minor flooding of surrounding homes and businesses. Table 9-6 is reprinted from the IPET report volume III, page 224.

Table 9-6

Summary of St. Charles Parish East Bank Hurricane Protection Features

Exterior levee and floodwall (I-wall and T-wall)	10 miles
Drainage structures	5
Highway closure structures	1
Railroad closure structures	1

The hurricane protection levee was completed prior to the storm but reaches 1A, 2A, and 2B, shown in Figure 9-8 and completed in the 1990's, had settled 3 to 4 feet by 2005 and reach 1B, completed in 2002, had settled about 2.5 feet. All floodgates were in place and at grade. Reach 1A was returned to design grade in 2003 but plans to raise the rest of the levee were awaiting Congressional funding at the time of the storm. The railroad gate at the east end of the levee (next to Louis Armstrong International Airport) was built by the FAA and completed prior to August 2005.

Flood Damage Reduction Study

Not satisfied without pump stations, the parish asked the Congress to re-evaluate the basin using new data and the newest modeling tools in 1999. A reconnaissance study was completed in 2003 and a feasibility study was begun in 2005 using new channel, culvert, and bridge surveys. Because the Labranche wetlands have often been considered an extension of Lake Pontchartrain, sixteen separate community models with the wetlands as their downstream starting point were developed first and then merged to create one super-model for the basin. The wetland was modeled in three zones connected to Lake Pontchartrain by a short reach of channel.

Datum Reconciliation

The IPET report on datums (Vol II, pg 57, Tbl 11) states local mean sea level (LMSL) at the west end of Lake Pontchartrain is 0.580 NAVD88 2004.65 using data from 2001 to 2005. Since LMSL was previously equal to 1.6 NGVD29, the conversion from NGVD to NAVD is -1.02 ft. Field surveys and aerial LIDAR are in or were converted to NAVD88 2004.65. The work of Brown, Cunningham and Gannuch in 1992 was in NGVD29 but was used primarily for general organization of the model and for estimating rainfall. Many canals and ditches had no field surveys and were too small to get reliable estimates from LIDAR so they were merely estimated. They exist to convey water from upstream storage areas to downstream main canals and should be considered only engineering estimates of actual dimensions.

Terrain Model

LIDAR surveys of South Louisiana were taken for the Federal Emergency Management Agency in 2004 using NAVD88 2004.65. The vertical accuracy for this data is +/- 0.7 feet. The horizontal projection is Louisiana State Plan South 1983 (1702). The basin boundaries for the HMS models are in the same projection. The data collected during these LIDAR surveys were processed using ESRI Arc software to develop other information needed for the modeling of this basin.

Storage-elevation curves were computed for each storage area and extended downward manually to prevent drying during computation. Elevations of drainage divides such as levees, roads, and railroad grades were also taken from LIDAR and compared to field surveys when possible. The accuracy of elevations is relatively unimportant for this model because flooding from Hurricane Katrina did not overtop any levees or railroads however it did calibrate well and is usable for other work.

Brown, Cunningham and Gannuch carefully laid out their rainfall basins on 7.5-min quadrangles and were easily traced into ESRI's ArcMap for use in the current study and saved in Louisiana State Plane South (1702) coordinates. Shape files were created and used in HEC-HMS to georeference the rainfall models. State plane coordinates were imported to HEC-RAS to georeference the model.

Manning's *n*-Values

Most Manning's *n*-values were 0.035 for earthen channels and 0.045 for overbanks. For culverts, the Manning's value varied from 0.015 for smooth steel pipe used in retro-fitted jack-and-bore culverts to about 0.030 for corrugated steel.

Channels

The study area was represented by 15 major channels as shown in Figure 9-9, each containing multiple reaches for a total of 108 channel segments as follows:

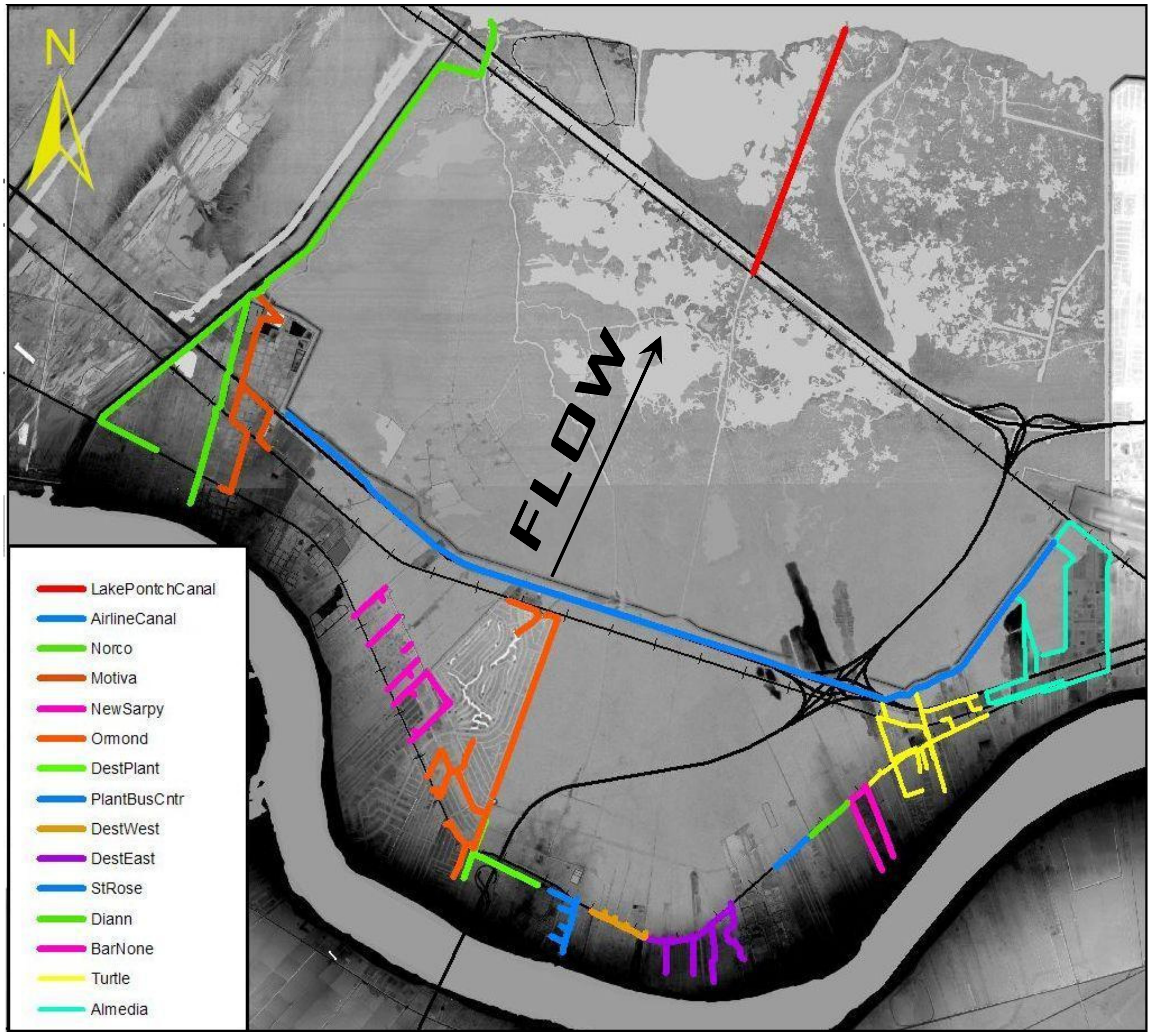

Figure 9-9. St Charles RAS Model Channels

Developed Communities and Industrial Parks

Norco	2
Shell Refinery	0
Motiva	3
Valero Refinery	0
GATX Storage	0
New Sarpy	16
Ormond	12
Ormond Plaza	0
Destrehan Plantation	3
Plantation Business Center	9
Destrehan West	8
Destrehan East	11
IMTT Storage	1
St Rose	1
Dianne Place	1
Bar None	6
Turtle Pond	23
Almedia	7

Undeveloped Wetlands and Special-Purpose Areas

Airline Canal	5
Lake Pontchartrain Canal	1

Many reaches are simple continuations of a single channel broken by a connection with a tributary. Others are short secondary and tertiary tributaries added to the RAS model to emulate the drainage pattern of the 1992 HEC-1 model. Free-draining channels are laid out like traditional stream systems with all branches flowing into a central main stem that drains to the Labranche Wetland. Pumped areas like New Sarpy and Ormond drain to storage areas that are pumped to the wetland.

The four areas with zero reaches were modeled as simple weir and culvert connections. No profile was attempted due to the shortness of the connection. In some cases, no measurements or photographs were provided so typical culvert sizes were used and may be a source of error but being industrial, they may not be as prone to flood damage as other areas.

One or two ground surveys were available in most channels. For modeling purposes, they were replicated as necessary to compute water surface profiles. However, these too may be a source of error since the suitability of one or two cross sections to represent an entire reach was not verified. In most cases, the assumption is not a problem and in other cases the error is small because stream velocities are low. Only in a few instances will velocities be high and the channel too poorly defined to give reliable results.

Bridges

Entrance, exit, and friction losses at bridges and culverts were computed using coefficients appropriate for each structure. Most bridges have approximately square abutments as do culverts with concrete headwalls. Most other culverts extend from the bank a short distance and incur higher head loss do to adverse flow conditions near the entrance. Exit losses were universally set to $\mathbf{v^2/2g}$. In most cases, additional cross sections were placed above and below bridges and culverts to facilitate additional calculations in the area of rapidly changing geometry from open channel to closed conduit.

Inline Weirs

Management of the water table is a delicate issue in south Louisiana. Drainage ditches need sufficient capacity to store runoff and convey water to the pumping stations but low water levels in the canals tends to lower the water table, which exposes more organic soil to oxidation and aggravates subsidence. Consequently, low weirs are often placed in drainage ditches to help maintain the water table. Since energy loss is proportional to the square of velocity, the low weirs have a great impact on conveyance during small storms when water is low in the canals and much less impact during major storms when water is deep in the canals, pump stations are running at capacity, and velocities are moderate to low. If deep water is running fast, there is also a high likelihood that the weir will be severely eroded and it's impact will be reduced. Water surface profiles for high frequency storms like the 50% and 25%-chance will be moderately affected by the absence of these weirs from the hydraulic model but it is assumed that their absence has little effect on the low frequency storms that cause the most out-of-bank flooding.

Ineffective Flow Areas

In all bodies of water, it is not uncommon for various portions to be moving at different speeds. At the shore, the velocity is often near zero while in mid stream the velocity may be approaching critical depth. One dimensional models like HEC-RAS don't account for this variation in velocity so the modeler often estimates the effective width of significant flow by either limiting the width of the cross section or using artificial constraints within the cross section to reduce the zone of active flow. This is particularly useful near channel transitions but may also appear elsewhere when considered expedient by the modeler. Approaches to most bridges, culverts, and unconnected channel include zones of ineffective flow.

Initial Conditions

HEC-RAS requires a reasonable balance of hydraulic conditions at all times including when the model first starts. One way to estimate this condition for the 165 separate storage areas and 103 reaches of canals is to assume the basin is completely flooded to the lowest practical elevation. With all sub-basins flooded to 3.3 ft, the corresponding stream-to-stream flow will be near zero due to lack of slope in the water surface. As calculations begin, Lake Pontchartrain is quickly lowered to 1.0 NAVD88 and six virtual pump stations help draw the artificially flooded basin down to nominal antecedent conditions prior to the start of the rainfall being modeled. For Hurricane Katrina, the storm hydrograph begins on 29 Aug 2005 at 0030 and ends at 1700. The model was therefore started on 28 Aug 2005 at 1900 to allow 5 hours for initial equalizing of the model although the last equalizing pump shuts off 3 hours later at 0330. This small overlap is considered insignificant due to the low rainfall in the early hours of Hurricane Katrina and due to the time required for flood water to travel from the developed portions of the basin to the last equalizing pump station.

Storage Areas

The study area was divided into 165 storage areas as follows and shown in Figure 9-10:

Developed Communities and Industrial Parks	
Norco	11
Shell Refinery	1
Motiva	7
Valero Refinery	1
GATX Storage	1
New Sarpy	18
Ormond	27
Ormond Plaza	1
Destrehan Plantation	6
Plantation Business Center	5
Destrehan West	6
Destrehan East	7
IMTT Storage	1
St Rose	12
Dianne Place	7
Bar None	6
Turtle Pond	14
Almedia	25
Undeveloped Wetlands and Special-Purpose Areas	
Labranche Wetlands	4
Pump Station sumps	5
Channel connections	2

FEMA's 2004 LIDAR data was also used to determine the stage-volume relationship for each storage area, which was usually connected to an adjacent canal over a lateral weir taken from LIDAR. Some storage areas were also connected to the network via the uppermost cross section of a reach by terminating the reach at the storage area. Many storage areas were also connected to adjacent storage areas by weirs or linear routing.

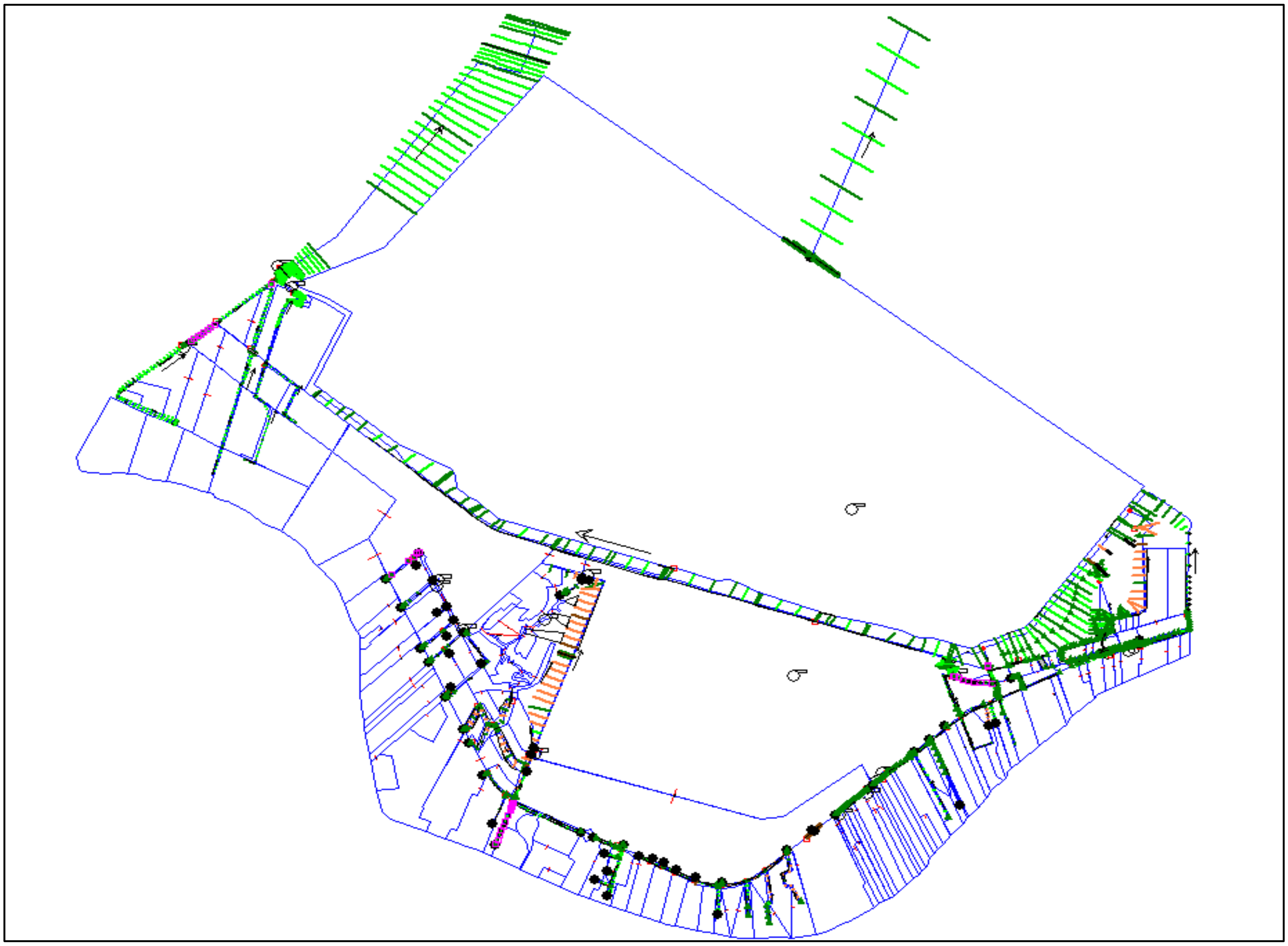

Figure 9-10. RAS Model Storage Areas

Weir and Storage Area Connections

Weirs connecting storage areas to canals were given a discharge coefficient of around 1.0 when the weir length was the full length of the interface however most weirs were drawn shorter than their full length for expediency and the discharge coefficient was set to 2.0. When the lateral weir represented an elevated berm such as a railroad track our highway the discharge coefficient was increased to 2.6.

When linear routing was used, the flow coefficient *k* was set to values ranging from 0.1 to 0.2, in the equation $Q = k \ \Delta S \ Hour^{-1}$, where ΔS is the change in storage (i.e. the net head difference x the area of the receiving storage area).

Levees

Many of the river communities in St Charles Parish have small levees. As shown in Figure 9-11, the Canadian National Railroad was built approximately on the border between marsh and upland and provides constitutes an informal levee between developed areas and Lake Pontchartrain. Recent communities that have built north of the railroad have also constructed more substantial levee and pumping systems to defend against storm surge. In 2005, the government built a formal hurricane protection levee that provides consolidated protection from the 1%-chance surge and includes both the existing developed communities and a substantial portion of low-lying wetlands. It has five gate structures as shown in Figure 9-11, each with several gates as shown in Table 9-7.

Table 9-7
Gate Structures

Structure	No. of Gates	Width	Height
Trepagnier PS & Gate	3	5	5
Cross Bayou Gate	6	6	6
St. Rose Gate	2	6	6
Walker Gate	1	4	4
Parish Line Gate	1	4	4

Pump Stations

St. Charles Parish has only recently been protected by a hurricane protection levee. Most of the development is therefore on high ground and reasonably well drained except during tropical storms and other occasions of high water elevations in Lake Pontchartrain such as when strong winds blow from the east. The notable exception is Ormond, which like much of New Orleans, is very close to sea level.

Although rarely causing flooding, high lake elevations have often caused very poor drainage in the past so small pump stations were built as needed to provide relief. Since most simply discharge to the north side of the railroad embankment, it is assumed that local officials place sandbags on normal drainage culverts to prevent return flow. There are 10 pump stations containing a total of 27 pumps as shown in Figure 9-11 and Table 9-8. All are operated by the parish public works department and, with the exception of Engineer's Canal and Trepagnier Pump Stations, discharge into the Lebranche Wetlands *inside* of the hurricane protection levee and are of limited value. The Engineer's Canal station discharges into the Bonnet Carré Spillway. Trepagnier Pump Station discharges through the hurricane protection levee to a canal that leads to Lake Pontchartrain.

In this model, starting and stopping elevations were assigned to all pump stations and they were allowed to run without supervision. Nearby culverts that might normally be sandbagged were left open so some circulation occurs.

Figure 9-11. Levees, Pump Stations and Flood Gates

In the RAS model, pump operation was set as closely as possible to the Parish's published operating plan as required by IPET Scenario 2. However, since many of the pumps turn on and off at similar elevations and can cause stability problems in the RAS model, minor changes were made to separate pump sequencing as shown in the table. Further information on the pump stations is described in Volume VI, Appendix 7.

Table 9-8
Pump Station Information and Water Surface On/Off Elevations, NAVD88 2004.65

Pump Station Name	Location	Capacity (cuffs)	No. of Pumps	Mean WSEL On	Mean WSEL OFF	Mean RAS WSEL On	Mean RAS WSEL OFF
Pumps to Lake Pontchartrain (outside hurricane protection levee)							
Engineer's Canal PS	Norco	64	2			3.25	2.60
Trepagnier PS	Norco	800	4			2.13	-0.13
Pumps to Labranche Wetland (inside hurricane protection levee)							
New Sarpy PS	New Sarpy	150	2	-2.00	-2.50	-2.87	-3.95
Schexnaydre PS	New Sarpy	200	3	-2.60	-2.9	-2.51	-4.03
Ormond I PS	Ormond	750	6	-3.5	-3.9	-3.19	-5.20
Ormond II PS	Ormond	930	5	-3.5	-3.9	-3.41	-4.27
4th Street PS	St Rose	64	2			3.05	2.30
Oak Street PS	St Rose	73	2			3.03	1.97
Dianne Place PS	Dianne Place	100	3	0.5	0	-0.78	-1.16
Turtle Pond PS	Turtle Pond	42	1	-1.25	-1.5	0.13	-1.87
Walker PS	Almedia	85	2	-1.5	-1.75	0.50	-0.75
Oakland PS	Almedia	67	2		-2	0.21	-0.82
Fairfield PS	Almedia	25	1	-2	-2.5	-1.51	-3.51

Storm Drain System

The drainage system in St. Charles Parish consists of ditches and culverts. Newly developed areas have catch basins and drop-inlets while older areas rely mostly on open ditches with dozens of driveway crossings. These connect to open ditches along the south side of the Canadian National Railroad that leads to culverts under the railroad at odd intervals. Many of the culverts lead to larger ditches that carry water north to the hurricane protection levee and then to Lake Pontchartrain but many simply terminate in the wetlands with no clear drainage path to the lake. The RAS model was built in community-size blocks by several engineers. Some include only the major canals while others include secondary and tertiary ditches. Although the level of detail varies, the results are uniform because the smaller ditch systems have little effect on the results.

Levee Overtopping and Breaching

No levees were overtopped or breached in St. Charles Parish during Hurricane Katrina. The floodplain for the Katrina event predicted by the numerical model is shown in Figure 9-12. Some shallow flooding can be seen along the railroad and in industrial areas but most residential and retail areas were flood free.

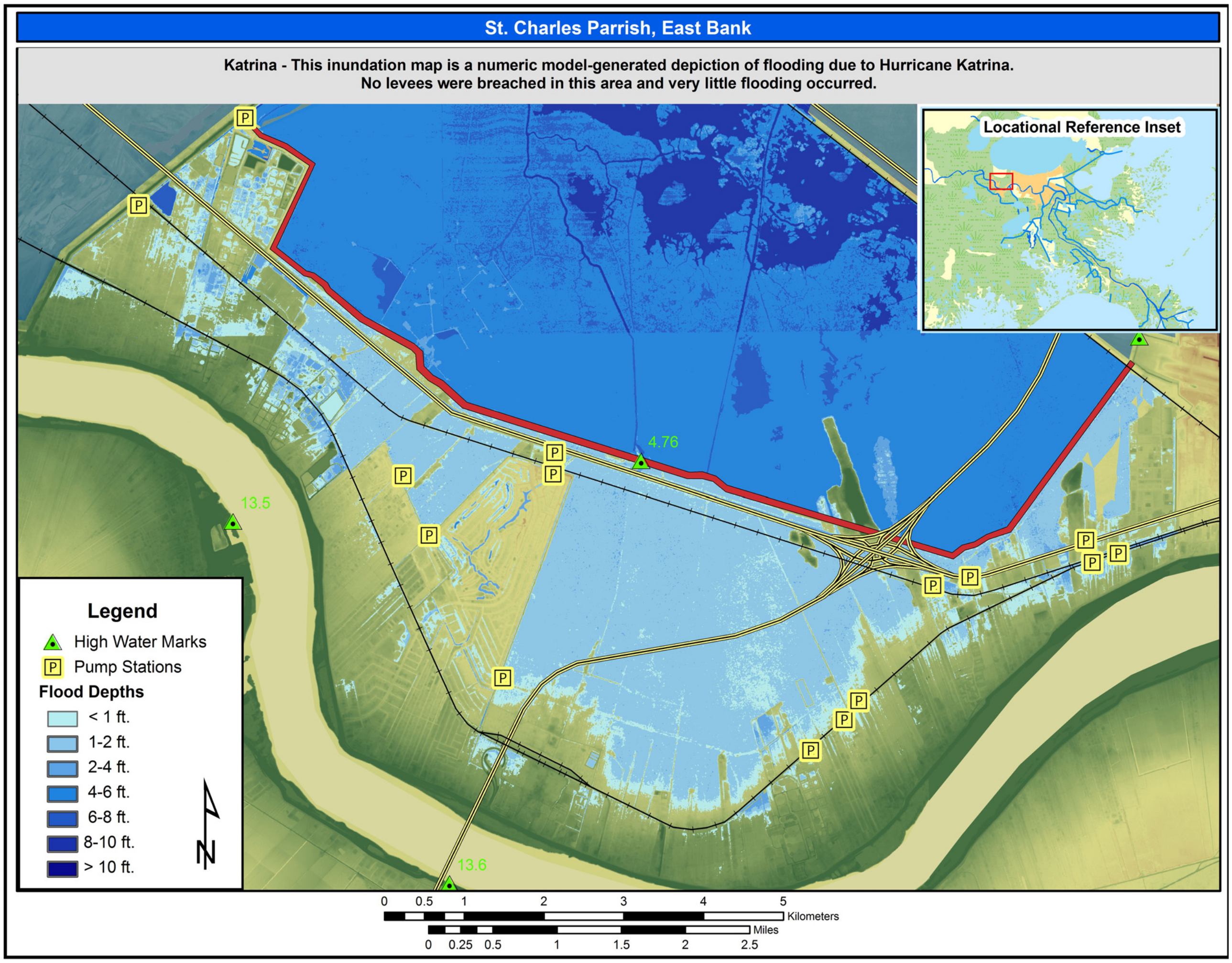

Figure 9-12. Flooding due to Hurricane Katrina

Model Calibration

No calibration was performed because there is no reliable data for any specific-frequency event and no reliable separation of rainfall flooding and surge flooding for known tropical events.