North Branch of the Ecorse Creek Drain Flood Control Study

Task 2

Hydrologic & Hydraulic Model Development

City of Allen Park

City of Dearborn Heights

City of Ecorse

City of Inkster

City of Lincoln Park

City of Melvindale

City of Taylor

City of Romulus

City of Westland

Prepared by:

Spicer group

Applied Science, Inc.

NTH Consultants, Ltd.
Professional Engineering and Environmental Services

ECT
Environmental Consulting & Technology, Inc.

Hennessey Engineers, Inc.

Robert A. Ficano
Wayne County Executive

Department of Environment
Wayne County

Kurt L. Heise
Wayne County Drain Commissioner

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TASK 2 – HYDROLOGIC AND HYDRAULIC MODEL DEVELOPMENT

Overview

The objective of Task 2 is to develop a computer model to simulate and accurately estimate the extent of flooding along the North Branch of the Ecorse Creek Drain (NBECD) for the May 2004 storm event and for various design storm events. The computer model consists of a hydrologic model and a hydraulic model. The development and results of these models are presented in this report under Task 2.1-Hydrologic Model Development and Task 2.2-Hydraulic Model Development.

TASK 2.1 - HYDROLOGIC MODEL DEVELOPMENT

Introduction

A hydrologic computer model was prepared and used to simulate storm water runoff for a range of storm events in the NBECD watershed. The NBECD watershed comprises portions of nine communities in southeastern Michigan: Ecorse, Lincoln Park, Melvindale, Allen Park, Dearborn Heights, Taylor, Inkster, Westland and Romulus.

The hydrologic model used for this analysis was the computer program HEC-HMS, Version 2.2.2. HEC-HMS, which stands for Hydrologic Engineering Center – Hydrologic Modeling System, was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers (USACE). The hydrologic model was used to calculate runoff volumes and peak flow rates within the watershed under existing and future conditions. The existing condition model run represented the current (2000) land use breakdown of the NBECD watershed. The future condition model was developed assuming "build-out" conditions, in which the currently undeveloped land in the watershed was assumed to be entirely developed by the year 2030. The "build-out" conditions model run was developed based on the future land use conditions maps presented in the "Ecorse Creek Watershed Management Plan," dated August 8, 2005.

In addition, the future conditions model was generated with storm water detention in-place for those areas that were assumed to be developed in the future. The storm water detention for each subdistrict was sized according to the Wayne County Storm Water

Management Program dated October 19, 2000 and amended February 19, 2002 and February 2007.

The HEC-HMS hydrologic model was run for the May 2004 storm and for a range of design storms. The May 2004 storm was used as the calibration period.

The hydrographs generated with HEC-HMS were loaded into the HEC-RAS hydraulic model of the NBECD to calculate water surface elevations and peak flow rates for a range of storm events. The HEC-RAS model is discussed in detail in Task 2.2–Hydraulic Model Development of this report.

The HEC-HMS model required Soil Conservation Service (SCS) runoff curves numbers (RCNs), drainage areas, times of concentration (t_c), baseflows and incremental rainfall depths. These input parameters for the HEC-HMS model were developed using the following information:

- Existing land use maps for year 2000 as compiled by the Ecorse Creek Inter-Municipality Committee (ECIC) and presented in the "Ecorse Creek Watershed Management Plan," dated August 8, 2005
- Future land use maps assuming "build-out" conditions for the year 2030 as compiled by the ECIC and presented in the "Ecorse Creek Watershed Management Plan," dated August 8, 2005
- Natural Resources Conservation Service (NRCS) soil inventory maps for Wayne County (2000)
- Municipal storm sewer section maps
- Wayne County drainage district maps
- USGS topographic maps
- Wayne County aerial photographs (May 2005)
- USGS stream gauge at Beech-Daly Road in Dearborn Heights
- SEMCOG, Detroit Metro Airport and Rouge Program Office (RPO) rainfall data for May 2004

- Bulletin 71: Rainfall Frequency Atlas of the Midwest, Huff and Angel (1992)
- Computing Flood Discharges for Small Ungaged Watersheds, Sorrell, MDEQ (2001)

Hydrologic Parameters

Drainage Areas

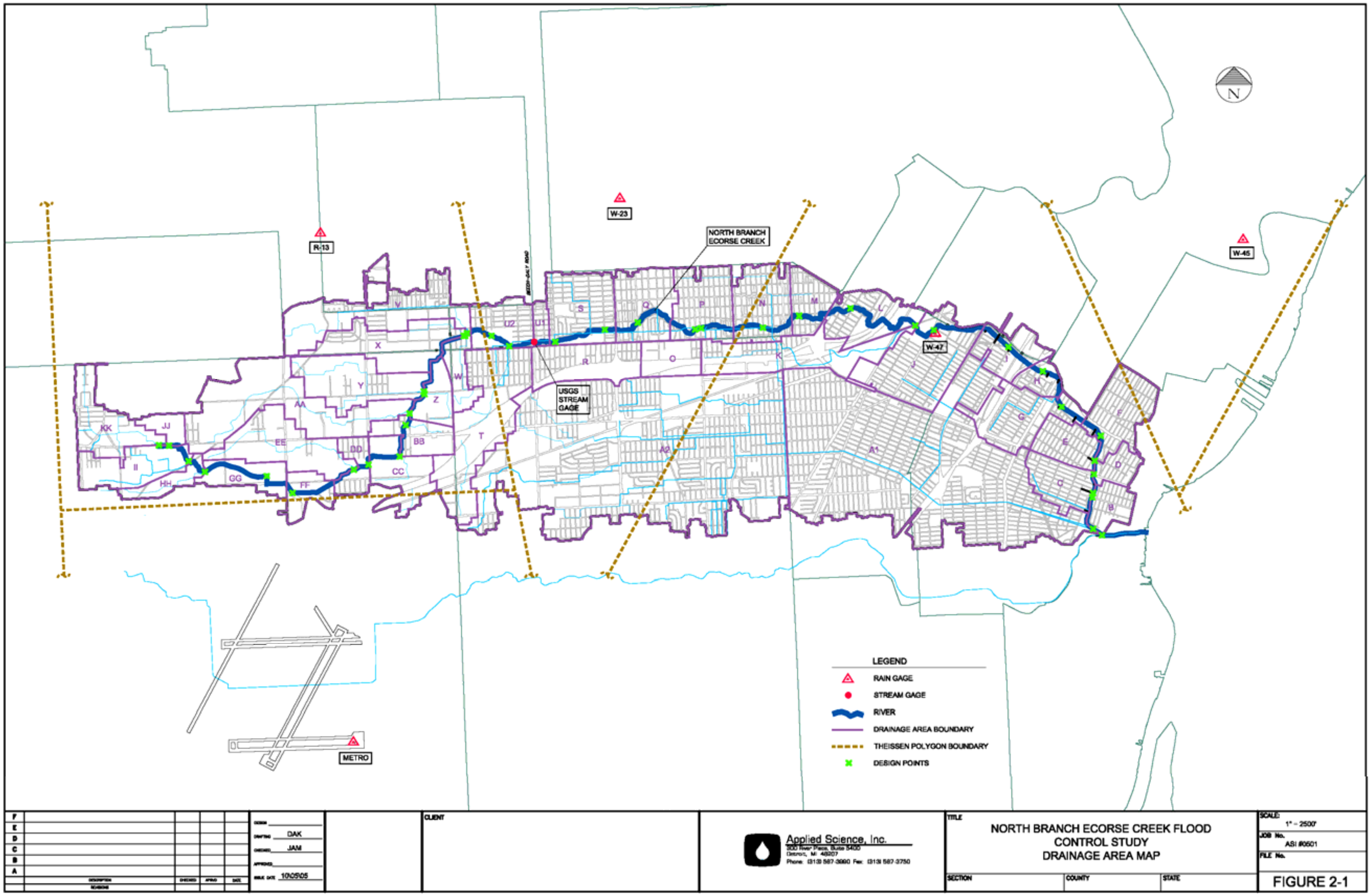
The total drainage area of the NBECD watershed at the confluence with Ecorse Creek is about 19,200 acres. To develop an accurate hydrologic model, the overall watershed was divided into small subdistricts. Fifty-eight (58) subdistricts were delineated based on existing drainage patterns. The existing drainage patterns were identified based upon review of existing municipal storm sewer section maps, Wayne County drainage district maps, USGS topographic maps and aerial photography. A map of the subdistricts in the NBECD watershed is shown on Figure 2-1.

Soil Type

Soil types were classified by Hydrologic Soil Group based on soil characteristics. Soil type affects the rate and volume of runoff in a drainage area. Heavy soils, such as clay, have low infiltration rates compared to light soils, such as sand, which allows higher rates of infiltration. Higher runoff rates and volumes would be expected in drainage areas that contain heavier soils.

For each subdistrict, the area of soils for each Hydrologic Soil Group was calculated. Soil types and Hydrologic Soil Groups were obtained from NRCS soil survey maps where available. The NRCS did not complete soil surveys throughout the entire NBECD watershed. Specifically, the soil survey was not completed for the portion of the watershed located north of Van Born Road and east of Pelham Road. The predominant classification of soil types in the watershed is Hydrologic Soil Group B and this classification was assumed in areas where the soil survey was not completed.

Figure 2-1
NBECD Subbasin and Rain Gage Location Map



Time of Concentration

The time of concentration represents the time required for the peak rate of storm water runoff to travel through a subdistrict and discharge to the NBECD. The time of concentration for each subdistrict was calculated by estimating the velocity of storm water runoff as it traveled through the subdistrict drainage system. The time of concentration was based on the slope and type of the drainage flow paths within the subdistrict. The types of drainage flow paths considered were sheet (overland) flow, concentrated shallow (small tributary or gutter) flow and open channel (pipe) flow.

The rainfall, land use, time of concentration and soil data were input into spreadsheets developed by Ric Sorrell of the Michigan Department of Environmental Quality (MDEQ) to calculate peak flow rates for a range of 24-hour, SCS Type II design storms. The RCNs were calculated within the spreadsheets and the initial peak runoff rates and hydrographs for each subdistrict were calculated using this method. Table 2-1 presents the drainage area, RCN and time of concentration used for each subdistrict in the NBECD watershed for the May 2004 calibration period.

Baseflow

Baseflows for each subdistrict were determined using measured stream flow data from a USGS stream gage on the NBECD at Beech-Daly Road. The average baseflow recorded at the stream gage was determined by inspection of the flow data for the period of May 20 through 27, 2004. The baseflow recorded at the end of the day on May 27, 2004, about four days after the last significant rainfall, was 7.68 cfs. Since the drainage area tributary to the Beech-Daly stream gage was determined to be 9.81 square miles, the baseflow per square mile was 7.68 cfs / 9.81 mi^2 = 0.78 cfs/mi^2. This ratio was applied to each of the subdistricts to derive an incremental baseflow. These baseflows were inputted into the HEC-RAS model of the NBECD and were assumed constant for the simulation periods. The baseflows are significantly below the flood flow rates being evaluated and do not impact the results. The incremental and cumulative baseflows for each of the subdistricts along the NBECD are also presented on Table 2-1.

Table 2-1
North Branch Ecorse Creek May 2004 Calibration Period Model Hydrologic Parameters

Subbasin ID	Drainage Area		Runoff Curve Number	Baseflow (cfs)		Subbasin Time of Concentration		Rain Gage Assignment	Total Rainfall for May 2004 Calibration Period
	acres	mi^2		Incremental	Cumulative	(minutes)	(hours)		
A1	3,205	5.01	86.5	3.91	8.52	271	4.5	W-47	4.50
A2	2,757	4.28	84.1	3.31	4.61	145	2.4	R-13, METRO, W-23, W-47	3.95
A3	1,125	1.68	84.1	1.30	1.30	57	1.0	R-13, METRO, W-23, W-47	3.95
B	162	0.25	87.4	0.20	14.80	70	1.2	W-47	4.50
C	260	0.41	85.2	0.32	14.60	52	0.9	W-47	4.50
D	123	0.19	84.8	0.15	14.28	40	0.7	W-47	4.50
E	217	0.34	85.4	0.26	14.13	49	0.8	W-47	4.50
F	379	0.59	84.2	0.46	13.87	56	0.9	W-45, W-47	4.24
G	420	0.66	87.7	0.51	13.41	123	2.1	W-47	4.50
H	116	0.18	85.9	0.14	12.90	37	0.6	W-47	4.50
I	255	0.40	86.3	0.31	12.76	34	0.6	W-47	4.50
J	571	0.89	83.9	0.69	12.45	104	1.7	W-47	4.50
O1	30	0.05	83.5	0.04	0.04	40	0.7	W-23	3.63
O2	135	0.21	90.2	0.16	0.20	35	0.6	W-23	3.63
O3	12	0.02	58.0	0.01	0.21	29	0.5	W-23	3.63
O4	9	0.01	91.2	0.01	0.22	64	1.1	W-23	3.63
O5	24	0.04	85.0	0.03	0.25	40	0.7	W-23	3.63
K1a	18	0.03	86.2	0.04	0.29	41	0.7	W-23, W-47	4.39
K1b	30	0.05	86.2	0.03	0.32	86	1.4	W-23, W-47	4.39
K1c	41	0.06	86.2	0.13	0.45	20	0.3	W-23, W-47	4.39
K2a	16	0.03	79.5	0.02	0.47	40	0.7	W-23, W-47	4.39
K2b	22	0.03	79.5	0.03	0.50	28	0.5	W-23, W-47	4.39
K2c	15	0.02	79.5	0.02	0.52	34	0.6	W-23, W-47	4.39
K2d	12	0.02	79.5	0.01	0.53	17	0.3	W-23, W-47	4.39
K3a	30	0.05	90.2	0.04	0.57	51	0.9	W-23, W-47	4.39
K3b	98	0.15	90.2	0.12	0.69	16	0.3	W-23, W-47	4.39
K3c	9	0.01	90.2	0.01	0.70	27	0.5	W-23, W-47	4.39
K3d	18	0.03	90.2	0.02	0.72	15	0.3	W-23, W-47	4.39
K3e	19	0.03	90.2	0.02	0.74	46	0.8	W-23, W-47	4.39
K3f	96	0.15	90.2	0.12	0.86	15	0.3	W-23, W-47	4.39
K4a	76	0.12	64.2	0.09	0.95	107	1.8	W-23, W-47	4.39
K4b	50	0.08	64.2	0.06	1.01	82	1.4	W-23, W-47	4.39

Table 2-1 continued
North Branch Ecorse Creek May 2004 Calibration Period Model Hydrologic Parameters

Subbasin ID	Subbasin Area		Runoff Curve Number	Baseflow (cfs)		Time of Concentration		Rain Gage Assignment	Total Rainfall for May 2004 Calibration Period
	acres	mi^2		Incremental	Cumulative	(minutes)	(hours)		
L	343	0.54	81.5	0.42	10.75	31	0.5	W-47	4.50
M	211	0.33	84.8	0.26	10.33	40	0.7	W-47	4.50
N	373	0.60	85.6	0.45	10.07	63	1.0	W-23, W-47	4.17
P	396	0.61	85.6	0.48	9.63	68	1.1	W-23	3.63
Q	369	0.60	85.7	0.44	9.15	64	1.1	W-23	3.63
R	364	0.57	86.2	0.44	8.71	32	0.5	W-23	3.63
S	435	0.68	85.5	0.52	8.27	72	1.2	W-23	3.63
U1	71	0.11	86.1	0.09	7.75	38	0.6	W-23	3.63
T	768	1.20	80.3	0.94	7.66	239	4.0	R-13, METRO, W-23	3.35
U2	257	0.40	86.1	0.31	6.73	52	0.9	R-13, W-23	3.56
V	674	1.05	75.8	0.82	6.41	231	3.8	R-13	3.34
W	155	0.24	77.4	0.19	5.59	178	3.0	R-13	3.34
X	466	0.73	73.8	0.57	5.40	231	3.9	R-13	3.34
Y	474	0.74	75.7	0.58	4.83	225	3.7	R-13	3.34
Z	170	0.27	78.5	0.21	4.25	120	2.0	R-13	3.34
AA	867	1.35	78.6	1.06	4.05	331	5.5	R-13	3.34
BB	208	0.32	79.8	0.25	2.99	124	2.1	R-13	3.34
CC	314	0.49	81.6	0.38	2.74	109	1.8	R-13, METRO	3.21
DD	142	0.22	78.5	0.17	2.35	48	0.8	R-13	3.34
EE	487	0.76	77.6	0.59	2.18	209	3.5	R-13	3.34
FF	162	0.25	87.5	0.20	1.59	40	0.7	R-13, METRO	3.11
GG	218	0.34	82.1	0.27	1.39	100	1.7	R-13	3.34
HH	129	0.20	77.3	0.16	1.12	116	1.9	R-13	3.34
II	334	0.52	70.9	0.41	0.96	202	3.4	R-13	3.34
JJ	97	0.15	67.3	0.12	0.56	101	1.7	R-13	3.34
KK	361	0.56	73.3	0.44	0.44	179	3.0	R-13	3.34

An interflow option in the HEC-HMS program was tested in which the baseflow was assumed to rise and to recede at an exponential rate following storms. Interflow parameters were selected by reviewing the USGS streamflow record for May 2004. The hydrographs generated with the interflow option were put into the HEC-RAS model and the calculated stage versus time was compared to the data recorded at the USGS stream gage at Beech-Daly Road. After several iterations, it was determined that the interflow option in HEC-HMS produced reasonable hydrographs for the USGS stream gage hydrograph during the initial flood peak on May 21, 2004. However, the hydrograph volume (runoff plus interflow) for subsequent peaks of the calibration storm were significantly overestimated. The interflow option was determined to work well for only single peak storm events. Therefore, the interflow option was not used for the calibration storm or any design storms for this project.

Runoff Curve Numbers (RCN)

For each subdistrict, a composite RCN was developed based on a breakdown of land use categories and soil types. The RCN is higher for more impervious or paved areas and lower for pervious or vegetated areas. The composite RCN for each subdistrict was developed by area-weighting the RCN assigned to each land use category. For the future land use conditions, the RCNs were higher for areas that were assumed to be developed in the future. The existing and future land use condition RCNs used in the HEC-HMS model are presented in Table 2-2.

Rainfall Data

Rainfall data for May 2004 were obtained from rain gages that are owned and operated by the Southeastern Michigan Council of Governments (SEMCOG), the Rouge River National Wet Weather Demonstration Project (RRDP) and the Detroit Metropolitan Airport (METRO). The May 2004 rainfall data were used for the calibration runs of the HEC-HMS model for the calibration period. The May 2004 calibration period was defined as the period from 9:00 pm on May 20, 2004 through 9:00 am on May 22, 2004. The rain gages used for this analysis are listed in Table 2-1 and are shown on Figure 2-1.

Table 2-2
Summary of NBECD Subbasin Hydrologic Parameters
Existing and Future Land Use Conditions

Subbasin ID	Incremental Baseflow (cfs)	Existing Land Use Conditions		Future Land Use Conditions			
				Existing Developed Land		Future Developed Land*	
		Subbasin Area (acres)	Runoff Curve Number	Subbasin Area (acres)	Runoff Curve Number	Subbasin Area (acres)	Runoff Curve Number
A1	3.91	3,205	86.5	3,205	86.5	0	NA
A2	3.31	2,757	84.1	2,757	84.1	0	NA
A3	1.30	1,125	84.1	1,125	84.1	0	NA
O1	0.04	30	83.5	30	83.5	0	NA
O2	0.16	135	90.2	135	90.2	0	NA
O3	0.01	12	58.0	12	58.0	0	NA
O4	0.01	9	91.2	9	91.2	0	NA
O5	0.03	24	85.0	24	85.0	0	NA
B	0.20	162	87.4	162	87.4	0	NA
C	0.32	260	85.2	260	85.2	0	NA
D	0.15	123	84.8	123	84.9	0	NA
E	0.26	217	85.4	217	85.5	0	NA
F	0.46	379	84.2	379	84.4	0	NA
G	0.51	420	87.7	420	88.0	0	NA
H	0.14	116	85.9	104	88.2	12	92.0
I	0.31	255	86.3	234	88.9	21	92.0
J	0.69	571	83.9	545	83.9	26	92.0
K1a	0.04	18	86.2	18	86.2	0	NA
K1b	0.03	30	86.2	30	86.2	0	NA
K1c	0.13	41	86.2	41	86.2	0	NA
K2a	0.02	16	79.5	16	79.5	0	NA
K2b	0.03	22	79.5	22	79.5	0	NA
K2c	0.02	15	79.5	15	79.5	0	NA
K2d	0.01	12	79.5	12	79.5	0	NA
K3a	0.04	30	90.2	30	90.2	0	NA
K3b	0.12	98	90.2	98	90.2	0	NA
K3c	0.01	9	90.2	9	90.2	0	NA

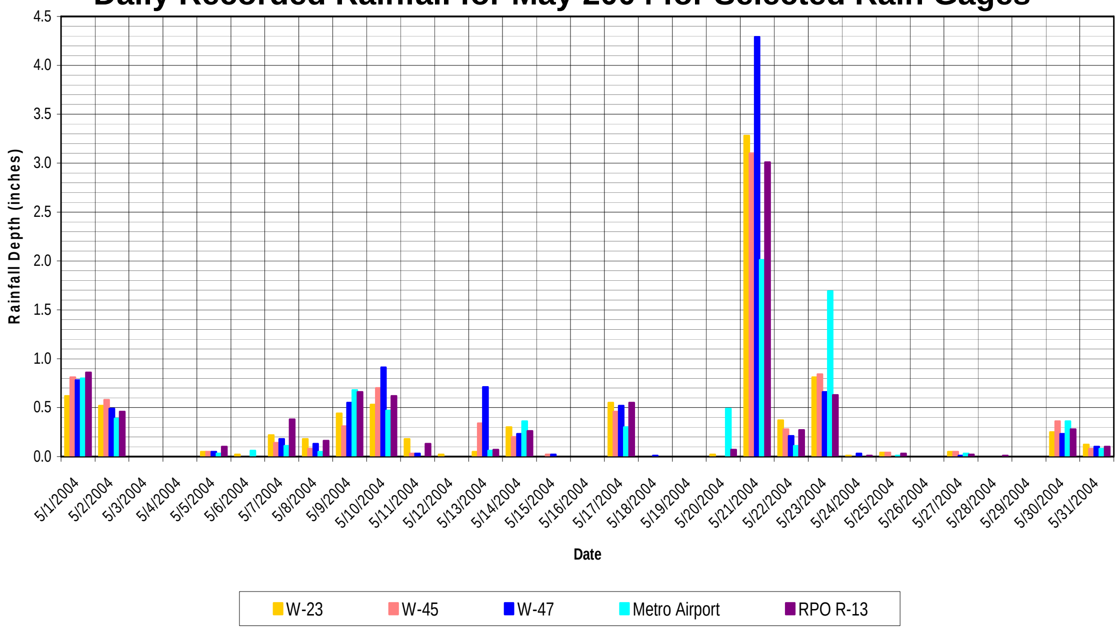
Table 2-2 continued
Summary of NBECD Subbasin Hydrologic Parameters
Existing and Future Land Use Conditions

Subbasin ID	Incremental Baseflow (cfs)	Existing Land Use Conditions		Future Land Use Conditions			
				Existing Developed Land		Future Developed Land*	
		Subbasin Area (acres)	Runoff Curve Number	Subbasin Area (acres)	Runoff Curve Number	Subbasin Area (acres)	Runoff Curve Number
K3d	0.02	18	90.2	18	90.2	0	NA
K3e	0.02	19	90.2	19	90.2	0	NA
K3f	0.12	96	90.2	96	90.2	0	NA
K4a	0.09	76	64.2	21	92.0	55	92.0
K4b	0.06	50	64.2	50	64.2	0	NA
L	0.42	343	81.5	297	85.1	46	92.0
M	0.26	211	84.8	211	84.8	0	NA
N	0.45	373	85.6	373	85.6	0	NA
P	0.48	396	85.6	396	85.6	0	NA
Q	0.44	369	85.7	369	85.7	0	NA
R	0.44	364	86.2	364	86.2	0	NA
S	0.52	435	85.5	435	85.5	0	NA
U1	0.09	71	86.1	71	86.1	0	NA
T	0.94	768	80.3	576	84.9	192	92.2
U2	0.31	257	86.1	257	86.9	0	NA
V	0.82	674	75.8	619	82.2	55	85.0
W	0.19	155	77.4	121	86.1	34	77.3
X	0.57	466	73.8	328	86.4	139	85.2
Y	0.58	474	75.7	213	88.6	261	92.4
Z	0.21	170	78.5	98	92.2	72	92.3
AA	1.06	867	78.6	413	83.1	454	82.0
BB	0.25	208	79.8	133	92.4	75	92.4
CC	0.38	314	81.6	144	93.0	170	93.0
DD	0.17	142	78.5	103	86.4	39	93.1
EE	0.59	487	77.6	187	85.7	300	92.4
FF	0.20	162	87.5	137	92.2	24	92.2
GG	0.27	218	82.1	95	93.1	123	93.1

Table 2-2 continued
Summary of Subbasin Hydrologic Parameters
Existing and Future Land Use Conditions

Subbasin ID	Incremental Baseflow (cfs)	Existing Land Use Conditions		Future Land Use Conditions			
				Existing Developed Land		Future Developed Land*	
		Subbasin Area (acres)	Runoff Curve Number	Subbasin Area (acres)	Runoff Curve Number	Subbasin Area (acres)	Runoff Curve Number
HH	0.16	129	77.3	53	92.6	76	92.8
II	0.41	334	70.9	42	88.5	292	90.0
JJ	0.12	97	67.3	13	76.5	84	77.3
KK	0.44	361	73.3	161	81.6	200	82.2

Figure 2-2
Daily Recorded Rainfall for May 2004 for Selected Rain Gages



Plots of the daily and hourly recorded rainfall for the selected rain gages used for this analysis for May 2004 are shown on Figures 2-2 and 2-3, respectively. The tabulated daily rainfall data for May 2004 are presented in Table 2-3. Tabulated hourly rainfall data for May 2004 are presented in Appendix A.

Figure 2-3
Hourly Recorded Rainfall for the May 2004 Calibration Period for Selected Rain Gages

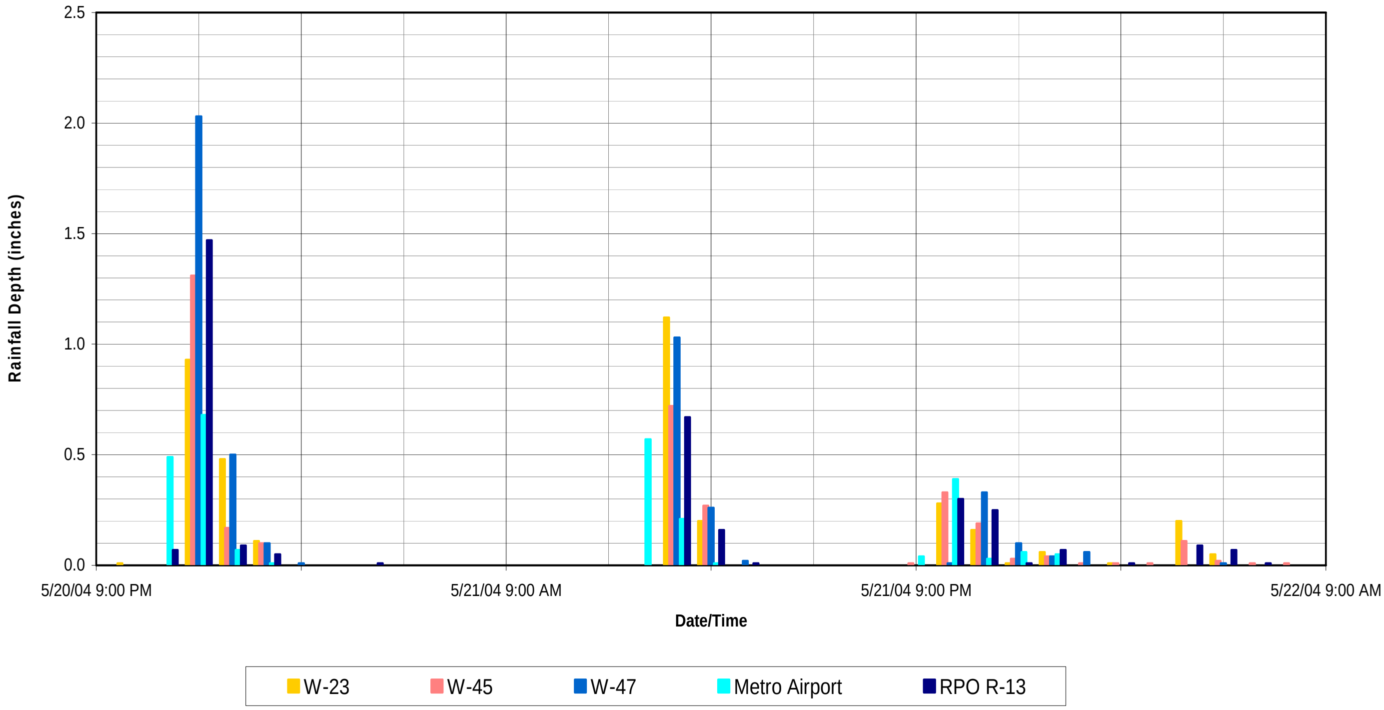


Table 2-3
Daily Recorded Rainfall for the May 2004 Calibration Period

Date	SEMCOG Rain Gages			Detroit Metro Airport Rain Gage	RRDP Rain Gage R-13
	W-23	W-45	W-47		
May 1, 2004	0.62	0.81	0.78	0.80	0.86
May 2, 2004	0.52	0.58	0.49	0.39	0.46
May 3, 2004	0.00	0.00	0.00	0.00	0.00
May 4, 2004	0.00	0.00	0.00	0.00	0.00
May 5, 2004	0.05	0.05	0.05	0.03	0.10
May 6, 2004	0.02	0.00	0.00	0.06	0.00

Table 2-3 continued
Daily Recorded Rainfall for the May 2004 Calibration Period

Date	SEMCOG Rain Gages			Detroit Metro Airport Rain Gage	RRDP Rain Gage R-13
	W-23	W-45	W-47		
May 7, 2004	0.22	0.14	0.18	0.11	0.38
May 8, 2004	0.18	0.08	0.13	0.05	0.16
May 9, 2004	0.44	0.31	0.55	0.68	0.66
May 10, 2004	0.53	0.70	0.91	0.47	0.62
May 11, 2004	0.18	0.03	0.03	0.00	0.13
May 12, 2004	0.02	0.00	0.00	0.00	0.00
May 13, 2004	0.05	0.34	0.71	0.06	0.07
May 14, 2004	0.30	0.20	0.23	0.36	0.26
May 15, 2004	0.00	0.02	0.02	0.00	0.00
May 16, 2004	0.00	0.00	0.00	0.00	0.00
May 17, 2004	0.55	0.46	0.52	0.30	0.55
May 18, 2004	0.00	0.00	0.01	0.00	0.00
May 19, 2004	0.00	0.00	0.00	0.00	0.00
May 20, 2004	0.02	0.00	0.00	0.49	0.07
May 21, 2004	3.28	3.10	4.29	2.01	3.01
May 22, 2004	0.37	0.28	0.21	0.11	0.27
May 23, 2004	0.81	0.84	0.66	1.69	0.63
May 24, 2004	0.01	0.00	0.03	0.00	0.01
May 25, 2004	0.04	0.04	0.00	0.01	0.03
May 26, 2004	0.00	0.00	0.00	0.00	0.00
May 27, 2004	0.05	0.05	0.01	0.03	0.02
May 28, 2004	0.00	0.00	0.00	0.00	0.01
May 29, 2004	0.00	0.00	0.00	0.00	0.00
May 30, 2004	0.25	0.36	0.23	0.36	0.28
May 31, 2004	0.12	0.08	0.10	0.08	0.10

The rainfall data were area-weighted by using the Theissen Polygon Method. The Theissen polygons are shown on Figure 2-1. The majority of the NBECD watershed area was assigned to two rain gages: RPO rain gage R-13; and SEMCOG rain gage W-47. Note that rain gage W-47 is the only rain gage within the NBECD watershed.

The total rainfall depths recorded for the May 2004 calibration period ranged from 2.61 inches (Metro) to 4.50 inches (W-47). The maximum rainfall depths in a 24-hour period during the May 2004 calibration period ranged from 2.47 inches (Metro) to 4.29 inches (W-47). The return frequency of the lowest recorded maximum rainfall depth in a 24-hour period of 2.47 inches is between a 2-year and a 5-year storm event. In one case, 2.63 inches fell in a matter of only 3 hours. The return frequency of the highest recorded maximum rainfall depth in a 24-hour period of 4.29 inches is between a 50-year and a 100-year storm event. Table 2-4 presents a summary of the total rainfall depths for the May 2004 calibration period.

Table 2-4
Rainfall Totals for May 2004 Calibration Period and Maximum 24-Hour Rainfall Totals within Calibration Period

Rain Gage	Owner	Location	Maximum Rainfall Depth in 24-Hour Period (inches)	Total Rainfall Depth for Calibration Period May 20-22, 2004 (inches)
W-23	SEMCOG	Telegraph Road and Michigan Avenue	3.28	3.63
W-45	SEMCOG	Jefferson Avenue near Dearborn Street	3.10	3.35
W-47	SEMCOG	University Street and Hope Street	4.29	4.50
Metro	Detroit Metro Airport	Smith Terminal at Detroit Metro Airport	2.47	2.61
R-13	RRDP	M-13, east of Merriman Road	3.01	3.34

Table 2-5
Design Storm for Various Return Frequencies
Type II Distribution, 24-Hour Duration

Recurrence Interval (Year)	Rainfall Depth* (inches)
2	2.26
5	2.75
10	3.13
25	3.60
50	3.98
100	4.36

***NOTE:** The reported rainfall depths correspond to Michigan Climatic Zone 10 from Bulletin 71: Rainfall Frequency Atlas of the Midwest, Huff and Angel (1992).

Rainfall depths for a range of design storms were also obtained and are shown in Table 2-5. The rainfall depths for the 100-year, 50-year, 25-year, 10-year, 5-year and 2-year design storms were taken from, “Bulletin 71: Rainfall Frequency Atlas of the Midwest.” The rainfall depths assume a Type II distribution over 24 hours for Midwest Region 10, which includes southeastern Michigan. Table 2-5 presents a summary of the total rainfall depths for the range of design storms.

Model Assumptions

The HEC-HMS model requires several hydrologic parameters to predict runoff volumes and flow rates for a given rainfall event. One of these hydrologic parameters is the initial abstraction, which is defined as the amount of precipitation that is lost due to depression storage, antecedent soil moisture conditions and vegetation present in the watershed. The percent imperviousness of the watershed is another hydrologic parameter that defines the amount of impervious land in the watershed that prevents rainfall from infiltrating the soil due to rooftops, driveways, sidewalks and streets and roadways. A SCS lag time, which is dependent on the time of concentration, is also required in HEC-HMS. The time of concentration is the time required for a drop of water to travel from the farthest boundary of the watershed to the watershed outlet.

For the calibration runs, the initial abstraction was set to zero for the entire watershed. An initial abstraction of zero was a reasonable assumption for the May 2004 calibration period since wet antecedent soil moisture conditions were present and much of the existing depression storage was filled from previous storm events. May 2004 was one of the wettest months of record in southeastern Michigan.

For the design storm runs, the HEC-HMS program calculated the initial abstraction for each subdistrict in the NBECD watershed assuming normal, dry antecedent moisture conditions and some depression storage. The program calculated the initial abstraction using the following formula.

$$I_a = 0.2*S \quad \{\text{where } S = (1{,}000 - 10*RCN)/RCN\}$$

It was also assumed that the percent imperviousness for each subdistrict was the default value of zero. As previously discussed, the RCNs were developed based on a land use breakdown that included some impervious areas. Therefore, it was unnecessary to include a nonzero percent imperviousness.

The HEC-HMS model required a lag time for each subdistrict to use in the SCS Method. It was assumed that the SCS lag time for each subdistrict was calculated using the following equation.

$$t_{lag} = 0.6 * t_c$$

In addition, the baseflow for each subdistrict was set to a constant rate and did not change through the calibration and design storm model simulations.

Model Development

The HEC-HMS model was set up to calculate expected runoff hydrographs for a range of storm events under existing and future land use conditions. The HEC-HMS model was initially run for the May 2004 calibration period under existing land use conditions. Once the model was calibrated, hydrographs were generated for 24-hour design storms having return frequencies of 100 years, 50 years, 25 years, 10 years, five years and two years.

The HEC-HMS model consisted of three components that were used to calculate runoff volumes and peak flow rates: a “loss rate” method, a “transform” method and a “baseflow” method.

The HEC-HMS loss rate method was used to predict the portion of total rainfall that was lost to ground depression storage and soil infiltration. For this analysis, the “SCS Curve Number Method” was selected. This method required an SCS RCN for each subdistrict, the initial abstraction and a percent imperviousness, which was set to zero for all runs.

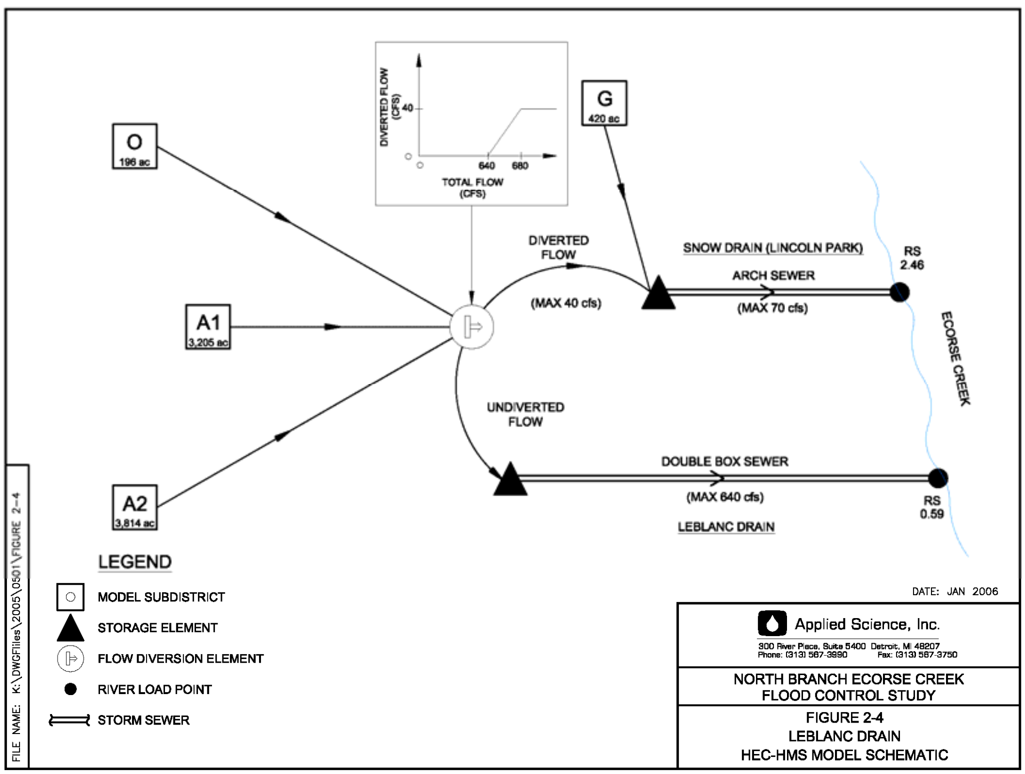
The HEC-HMS transform method predicted the amount of excess precipitation that was converted to direct runoff. For this analysis, the “SCS Transform Method” was selected. This method required an SCS lag time, which was related to the subdistrict time of concentration as previously discussed.

The baseflow method predicted the portion of the total runoff hydrograph that was baseflow and the portion that was direct runoff. This method determined the shape of the runoff hydrograph, in particular how the hydrograph receded from direct runoff to baseflow. The “Constant Monthly Baseflow Method” was selected for this analysis. This method required an incremental baseflow for each subdistrict that was assumed to be constant for the entire simulation period.

LeBlanc and Snow Drain Representation

The LeBlanc and Snow Drains were modeled as part of the NBECD watershed hydrologic model. A HEC-HMS model schematic of the LeBlanc and Snow Drain model set-up is presented on Figure 2-4. The subdistricts’ tributary to the LeBlanc and Snow Drains (Subbasins A1 and A2) is connected to a storage element that represents the in-system storage and storage in the Goddard and Monroe storm water detention basins. A maximum of 71 cfs is discharged to a diversion element. Subbasin A1 is connected to the diversion element, which diverts a maximum of 40 cfs to the Snow Drain and the remaining undiverted flow discharges to the LeBlanc Drain. The capacity of the 4-foot diameter interconnection between the LeBlanc Drain and the Snow Drain along Dix Highway was determined to be about 40 cfs. Zero flow is diverted to the Snow Drain until the total flow rate reaches 640 cfs in the LeBlanc Drain. The diverted flow in the Snow Drain combines with runoff from the Snow Drain subdistrict in Lincoln Park (Subbasin G). A maximum flow rate of 70 cfs discharges through an existing arch storm sewer to the NBECD through the Snow Drain. For the LeBlanc Drain, a maximum of 640 cfs discharges through an existing double box storm sewer to the NBECD downstream of the Snow Drain. The hydrographs to the NBECD from the Snow and LeBlanc Drains are flat-topped in shape and reflect the restrictive capacity of these drains.

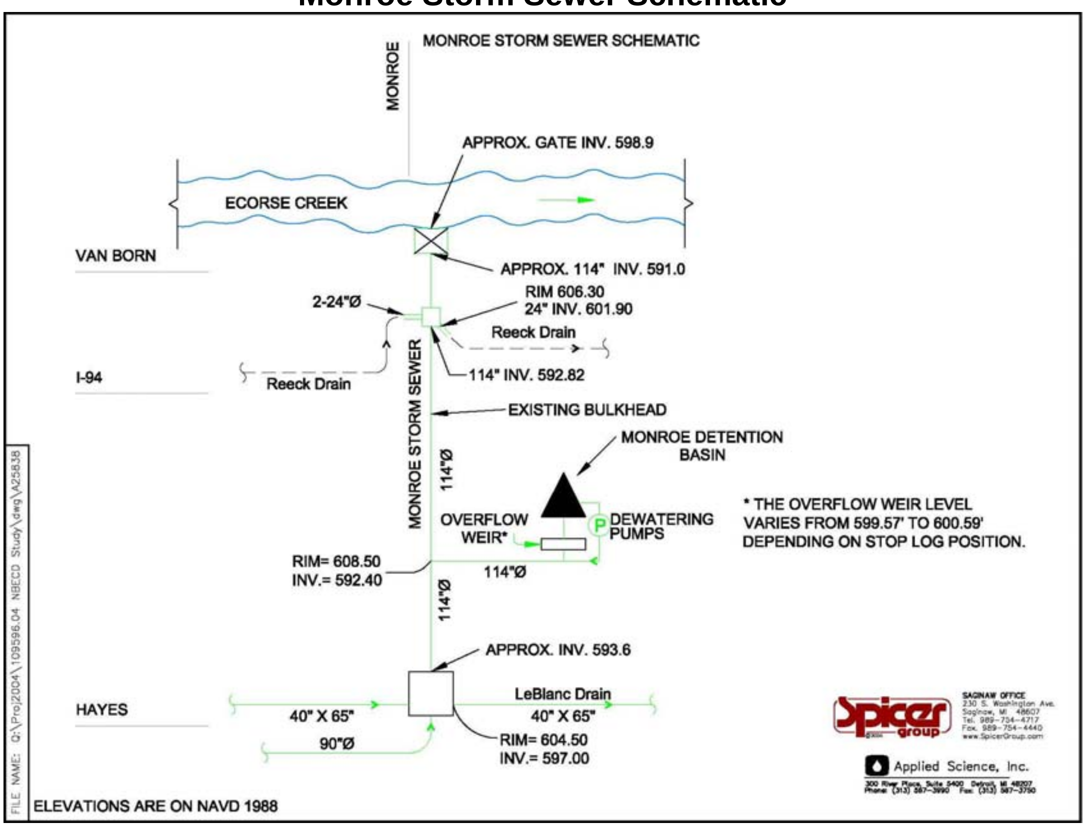
Figure 2-4
Leblanc Drain
HEC-HMS Model Schematic



Monroe Road Interconnection

The Monroe Road storm sewer is 114 inches in diameter and runs along Monroe Road from a junction chamber at Monroe and Hayes in the City of Taylor northerly to the NBECD in the City of Dearborn Heights. The section of storm sewer, when originally installed, connected to the Leblanc Drain and NBECD. Currently, this section of storm sewer was found to be bulkheaded near I-94 with an 18-inch diameter shear gate in the CLOSED position. Therefore, the section of storm sewer does not interconnect the Leblanc Drain and NBECD. The storm sewer does convey storm water from the City of Taylor south of I-94 into the Monroe Road storm water detention basin. North of I-94, the storm sewer conveys storm water to the NBECD. At the NBECD, a 30-inch wide by 24-inch high sluice gate exists on the Monroe Road storm sewer with an invert elevation about one foot above the NBECD. Currently, this sluice gate is partially open. The Monroe Road storm sewer is shown on Figure 2-5.

Figure 2-5
Leblanc Drain
Monroe Storm Sewer Schematic



During low flow conditions, the section of the Monroe Road storm sewer located north of I-94 outlets to the NBECD. The Reeck Drain interconnects with the Monroe Road storm sewer in this section. The Reeck Drain, west of Monroe Road, outlets into the Monroe Road storm sewer and flows to the NBECD.

In wet weather conditions, as water levels in NBECD are high, the NBECD can back water into the Monroe Road storm sewer and result in the storm sewer to overflowing into the downstream section of the Reeck Drain. This occurs at or about elevation 601.9 feet.

The Monroe Road storm sewer cannot completely drain and the storm sewer always has a water level about four to six feet deep throughout the entire length. Under wet weather flow conditions, flow can occur in either direction through the partially open sluice gate depending on the differential head between the NBECD and the Reeck Drain. The flow rate that occurs is small relative to the flood flow rates in the NBECD. For model accuracy, the transfer of drainage occurring north of I-94 between the Monroe Road Street storm sewer, the Reeck Drain and the NBECD was included in the model.

The neighborhood in the City of Taylor north of the I-94 expressway and east of Monroe Road along the Reeck Drain reports frequent flooding. The Monroe Road storm sewer interconnection along with the NBECD overflowing Van Born Road into Taylor is contributing to this flooding. Also, a lower section of the Reeck Drain east of Pelham Street in the City of Allen Park was observed to flood the streets and overflow Van Born Road during the May 2004 flood.

Existing and Future Land Use Conditions

The watershed was divided into subdistricts and each subdistrict was modeled with a HEC-HMS subbasin. Hydrographs from each subbasin are loaded at a river junction, or load point, that corresponds to a HEC-RAS station along the NBECD.

The existing and future land use conditions and RCNs are presented on Table 2-2. The future land use conditions model was set up by splitting each of the subdistricts into two separate parts: the first representing the existing developed land; and the second representing the undeveloped land to be developed in the future. Future condition RCNs were developed for both parts representing built-out conditions.

Most of the currently undeveloped areas are located in the upper reaches of the NBECD, upstream of Beech-Daly Road. Currently, the majority of the subdistricts in the lower portions of the NBECD watershed are nearly fully developed. The future hydrologic model of the subdistricts in the lower portions of the NBECD watershed remained unchanged from the existing land use conditions hydrologic model. While there may be some redevelopment in these areas that reduce peak flows rates in the NBECD, the "worst-case scenario" was determined to be the existing conditions and existing conditions were used for the future built-out conditions.

A HEC-HMS model schematic showing a typical layout of a subdistrict split into existing and future developed parts is presented on Figure 2-6. The part of each subdistrict that will be developed in the future was assumed to have storm water controls (i.e. detention basins). "Lumped" detention basins for the future developed parts were sized for a 100-year design storm based on the Wayne County Storm Water Management Program requirements. The "lumped" detention basin was a stage-storage-outflow based on an allowable release rate of 0.15 cfs per acre. Figure 2-7 shows a hydrograph for the future developed part of Subdistrict KK (KK2) with the allowable release rate (Q_a) and the required storage volume. Table 2-6 presents the "lumped" detention calculations for all of the future developed parts of the subdistricts.

Figure 2-6
Future Conditions
Typical HEC-HMS Model Set-Up

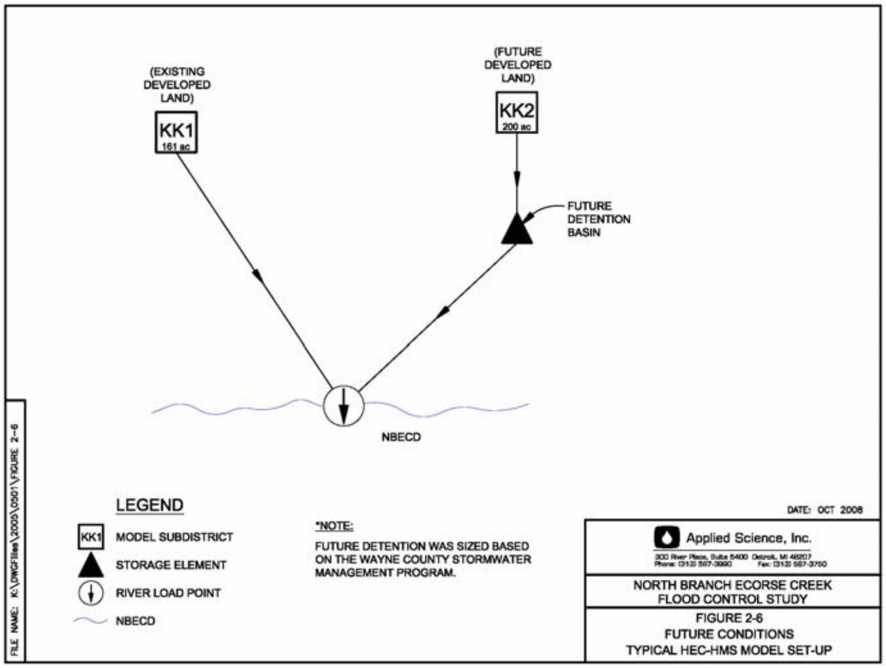


Figure 2-7
Future Land Use Conditions Hydrographs
Future Developed Land With Storm Water Controls (Basin)
Subdistrict KK2

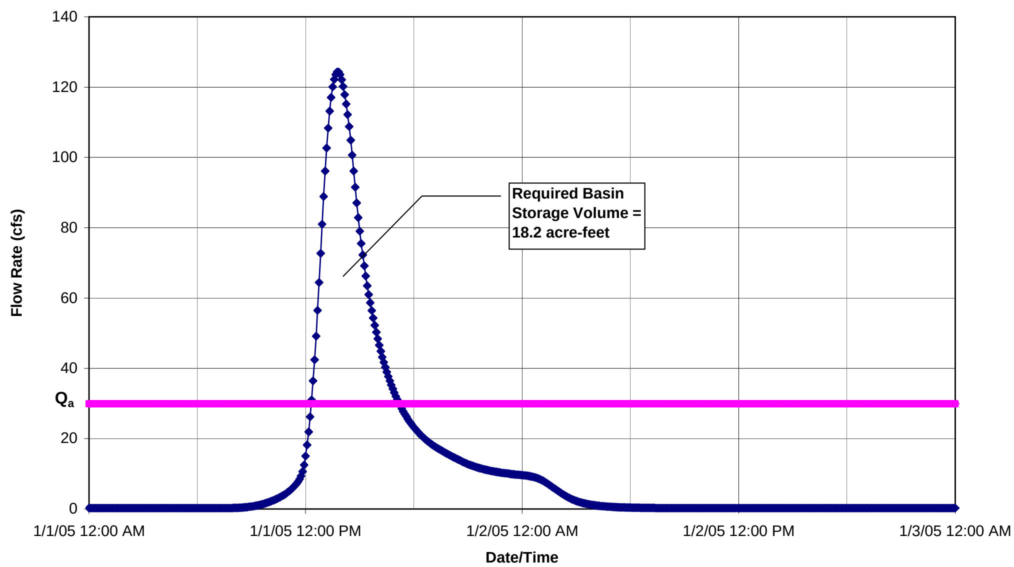


Table 2-6
Future Conditions Hydrologic Parameters and Storm Water Controls for HEC-HMS Model
100-Year, 24-Hour Design Storm

HEC-HMS Element	Tributary Drainage Area (mi^2)	Future Build-Out RCN	Required Storage Volume (acre-feet)
KK2	0.31	82.2	18.1
JJ2	0.13	77.3	6.5
II2	0.46	90.0	39.0
HH2	0.12	92.8	12.0
GG2	0.19	93.1	19.2
FF2	0.04	92.2	4.1
EE2	0.47	92.4	43.3
DD2	0.06	93.1	6.2
CC2	0.27	93.0	27.4
BB2	0.12	92.4	11.9
AA2	0.71	82.0	31.1
Z2	0.11	92.3	10.7
Y2	0.41	92.4	37.3
X2	0.22	85.2	14.3
W2	0.05	77.3	2.1
V2	0.09	85.0	6.0
T2	0.30	92.2	26.7
L2	0.07	92.0	7.0
K4a2	0.10	92.0	10.2
J2	0.04	92.0	3.9
I2	0.03	92.0	2.9
H2	0.02	92.0	2.0

Model Calibration Runs

The May 2004 calibration period was selected because several intense bursts of rainfall occurred during this time period that produced significant flooding along the NBECD. In addition, reliable stream gage and high water mark data were available for this period. The HEC-HMS model was run for this calibration period under existing land use conditions.

The HEC-HMS model was calibrated by comparing two parameters at Beech-Daly Road: total runoff volume and peak stage for the May 2004 calibration period. The hydrographs from HEC-HMS model for each subdistrict were put into the HEC-RAS model and routed through the NBECD to determine the predicted peak stage at Beech-Day Road and other locations with high water marks. High water marks and the HEC-RAS model development are presented in Task 2.2 of this report.

The total runoff volume and the peak stage predicted by the model were compared to the volume the USGS stream gage data at Beech-Daly Road for the calibration period. The USGS gage at Beech-Daly Road has only been installed since 2001 and the rating curve was developed in October 2004. Therefore, more weight was given to matching peak stage than volume measured at the station.

Three HEC-HMS model runs were generated for the calibration analysis. Runs 1 and 2 used the recorded rainfall from rain gages R-13, Metro, W-23 and W-47. Run 3 tested the impact of the spatial rainfall variability on the results and used rainfall from only rain gage W-47. Run 1 utilized "textbook" RCNs that were based on the existing conditions land use and soil types. For Run 2, the RCNs for the subdistricts' tributary to the USGS stream gage at Beech-Daly Road was increased in order to better predict both volume and stage. Run 3 was a sensitivity run that used the "textbook" RCNs with rainfall from only rain gage W-47.

Table 2-7 presents the results for the three calibration runs and a comparison to the total volume estimated at the USGS stream gage. The total runoff volume recorded at the USGS stream gage at Beech-Daly Road for the May 2004 calibration period was 1,357 acre-feet (2.59 inches).

Run 1 slightly under-predicted the peak stage and predicted a significant shortfall in predicted runoff volume.

In Run 2, the RCNs were increased by nine percent to obtain better agreement with the recorded stage and volume. The peak stage for Run 2 closely matched the recorded peak stage within 0.1 feet at Beech-Daly Road. Run 2 provided very good agreement in peak stages and an acceptably close agreement in runoff volume given the accuracy of the stage-discharge relationship for the stream gage. Therefore, a nine percent increase in the textbook RCNs was used as the final HEC-HMS calibration run for the May 2004 flood event.

Run 3 produced significantly higher predicted stage and runoff volume compared to the Beech-Daly stream gage. The Run 3 results indicated that the rainfall variability was a likely cause for the discrepancy between modeled and estimated runoff volumes.

Table 2-7
Calibration to USGS Stream Gage at Beech-Daly Road
May 2004 Calibration Period

Calibration Run	RCNs	Area-Weighted RCN	Initial Abstraction (inches)	Rain Gages Used	Area-Weighted Rainfall (inches)	Runoff Volume		Peak Stage (feet)
						(inches)	(acre-feet)	
1	Land Use/Soils Based	77.8	0.0	R-13, METRO, W-23	3.34	1.92	1,004	614.0
2	Land Use/Soils Based * 1.09	85.2	0.0	R-13, METRO, W-23	3.34	2.30	1,202	614.4
3	Land Use/Soils Based	77.8	0.0	W-47	4.50	2.86	1,499	615.6
Measured USGS Stream Gage Volume =						**2.59**	**1,357**	**614.3**

The HEC-RAS calibration run results are discussed further under Task 2.2-Hydraulic Model Development. The predicted stage data were compared to the stage data recorded at the USGS stream gage at Beech-Daly Road for the May 2004 calibration period. In addition, the predicted stage data at other key locations along the NBECD were compared to recorded high water marks for the May 2004 calibration period.

Design Storm Runs

Existing and Future Land Use Conditions

The model calibration indicates that the difference in modeled and measured stage and volume at the USGS stream gage at Beech-Daly Road was most likely due to spatial rainfall variability, not lower than textbook RCNs. Therefore, textbook RCNs were utilized for the design storm model runs.

The predicted peak flood flow rates calculated with HEC-HMS were compared to the flood flow rates provided by MDEQ and the flood flow rates used in by the FEMA in the Flood Insurance Study (FIS) reports.

Table 2-8 presents a comparison of the unrouted peak flow rates computed for existing land use conditions with the HEC-HMS model and the peak flow rates calculated with the MDEQ version of the SCS Method (the "MDEQ Method") for a range of design storms. The peak flow rates calculated in the HEC-HMS model are unrouted peak hydrograph flow rates are significantly higher than the peak flow rates and volumes calculated with the MDEQ Method.

A comparison of the unrouted hydrographs at Beech-Daly Road for the 100-year, 24-hour design storm under existing and future land use conditions with and without projected storm water controls (detention) is presented on Figure 2-8. Under future land use conditions, the runoff volumes for each subdistrict will be significantly higher than the runoff volumes calculated with existing land use conditions. The total runoff volume at Beech-Daly Road is predicted to increase by about 35 percent with future built-out conditions.

The existing and future land use condition hydrographs at Beech-Daly Road are tabulated and presented in Appendix B.

Table 2-8
Existing Land Use Condition Peak Flow Rates for a Range of Design Storms
Unrouted HEC-HMS Peak Flow Rates vs. MDEQ Method Peak Flood Flow Rates

Subbasin ID	Subbasin Area		Runoff Curve Number	Peak Flow Rates (cfs)											
				HEC-HMS Version 2.2.2						MDEQ Method					
	acres	mi^3		100-Year	50-Year	25-Year	10-Year	5-Year	2-Year	100-Year	50-Year	25-Year	10-Year	5-Year	2-Year
A1	3,205	5.01	86.5	1,703	1,499	1,297	1,052	858	617	1,014	893	780	630	515	355
A2/A3	3,882	6.07	84.1	2,348	2,050	1,757	1,403	1,125	785	1,417	1,240	1,073	856	693	467
O	210	0.33	87.2	275	243	212	173	142	103	180	159	139	113	93	64
B	162	0.25	87.4	241	213	186	152	124	90	158	139	122	99	81	56
C	260	0.41	85.2	442	389	335	270	219	155	307	269	234	188	153	104
D	123	0.19	84.8	243	213	183	147	119	83	174	153	133	106	86	58
E	217	0.34	85.4	389	341	294	237	192	136	272	239	208	167	136	93
F	379	0.59	84.2	600	525	450	360	289	201	405	354	307	245	198	134
G	420	0.66	87.7	421	372	324	265	218	159	265	234	205	167	137	95
H	116	0.18	85.9	252	222	192	156	127	91	183	161	140	113	92	63
I	255	0.40	86.3	572	505	438	356	291	210	440	387	338	272	223	153
J	571	0.89	83.9	589	514	440	351	281	195	373	326	282	225	182	122
K	550	0.86	88.6	340	301	263	216	179	132	203	180	158	129	106	75
L	343	0.54	81.5	693	600	508	399	313	212	540	469	403	317	253	166
M	211	0.33	84.8	418	366	315	253	204	143	301	264	229	183	149	101

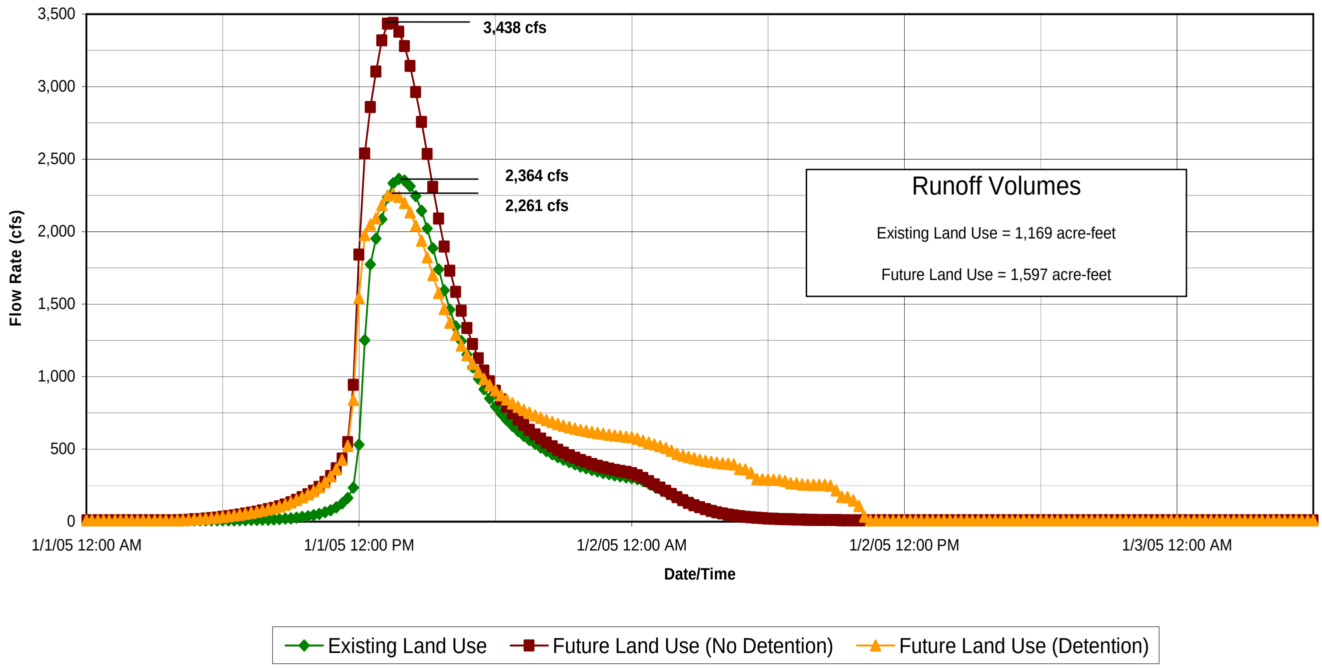
Table 2-8 continued
Existing Land Use Condition Peak Flow Rates for a Range of Design Storms
Unrouted HEC-HMS Peak Flow Rates vs. MDEQ Method Peak Flood Flow Rates

Subbasin ID	Subbasin Area		Runoff Curve Number	Peak Flow Rates (cfs)											
				HEC-HMS Version 2.2.2						MDEQ Method					
	acres	mi^3		100-Year	50-Year	25-Year	10-Year	5-Year	2-Year	100-Year	50-Year	**25-Year**	**10-Year**	**5-Year**	**2-Year**
N	373	0.58	85.6	579	509	439	354	287	204	391	343	299	240	196	134
P	396	0.62	85.6	562	494	426	344	279	199	372	327	285	229	187	127
Q	369	0.58	85.7	577	507	438	354	287	204	389	342	297	239	195	133
R	364	0.57	86.2	896	789	683	554	451	323	674	593	518	417	341	234
S	435	0.68	85.5	601	527	455	367	297	211	394	346	301	242	197	134
U1	71	0.11	86.1	149	132	114	92	75	54	111	98	85	69	56	38
T	768	1.20	80.3	361	311	262	204	159	105	218	189	161	126	100	65
U2	257	0.40	86.1	448	395	342	277	226	162	309	272	237	191	156	107
V	674	1.05	75.8	271	229	189	141	106	65	166	142	119	91	70	44
W	155	0.24	77.4	81	69	57	43	33	21	50	43	36	28	22	14
X	466	0.73	73.8	170	143	116	85	63	37	107	90	75	56	43	26
Y	474	0.74	75.7	193	163	134	100	75	46	119	102	85	65	50	31
Z	170	0.27	78.5	125	107	89	68	52	33	81	70	59	46	36	23
AA	867	1.35	78.6	296	253	212	163	125	81	177	152	129	100	79	50
BB	208	0.32	79.8	158	136	114	88	68	45	98	85	72	56	45	29
CC	314	0.49	81.6	278	240	204	160	126	85	177	154	132	104	83	55

Table 2-8 continued
Existing Land Use Condition Peak Flow Rates for a Range of Design Storms
Unrouted HEC-HMS Peak Flow Rates vs. MDEQ Method Peak Flood Flow Rates

Subbasin ID	Subbasin Area		Runoff Curve Number	Peak Flow Rates (cfs)											
				HEC-HMS Version 2.2.2						MDEQ Method					
	acres	mi^3		100-Year	50-Year	25-Year	10-Year	5-Year	2-Year	100-Year	50-Year	**25-Year**	**10-Year**	**5-Year**	**2-Year**
DD	142	0.22	78.5	198	169	141	109	83	53	139	120	102	79	62	39
EE	487	0.76	77.6	227	193	160	122	93	59	140	120	101	78	61	39
FF	162	0.25	87.5	349	308	268	219	180	131	250	221	194	157	129	90
GG	218	0.34	82.1	211	183	156	122	97	66	134	117	101	79	64	42
HH	129	0.20	77.3	93	79	66	50	38	24	59	51	43	33	26	16
II	334	0.52	70.9	117	97	77	55	39	22	75	63	52	38	29	17
JJ	97	0.15	67.3	46	37	29	19	13	6	32	27	22	15	11	6
KK	361	0.56	73.3	156	130	105	77	56	33	99	83	69	52	40	24
Subtotal to Beech-Daly	**6,281**	**9.81**	**77.8**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**
Total to Detroit River	**19,192**	**29.91**	**82.9**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**	**NA**

Figure 2-8
Comparison of Unrouted Hydrographs at Beech-Daly Road
100-Year, 24-Hour Type II Design Storm



Without storm water controls implemented in future developed areas, the peak flow rates will be significantly higher in the NBECD than under existing conditions. However, with storm water controls in place, the peak flow rates expected are close to those expected under existing conditions and are predicted to drop about four percent. It is our opinion that the predicted drop in peak flow rate under future conditions is not reasonable to expect and is due to limitations of the modeling method.

This comparison shows the importance of the Wayne County Stormwater Management Program. If future development occurs without storm water controls, significantly higher peak flow rates will occur in the NBECD. Consequently, it is imperative that all future land be developed according to the Wayne County Stormwater Management Program to ensure that peak flood flow rates, flooding levels, the extent of flooding and flooding frequency are not increased.

TASK 2.2 - HYDRAULIC MODEL DEVELOPMENT

Introduction

Task 2.2 consists of developing and calibrating a hydraulic computer model to simulate flood flow rates and flood levels along the North Branch of the Ecorse Creek Drain (NBECD) for existing conditions. The NBECD consists of nearly 17 miles of drain traversing from its headwaters in the City of Romulus to the outlet located at the confluence with the Sexton-Kilfoil Drain in Lincoln Park. The reach of drain from the confluence with the Sexton-Kilfoil Drain to the Detroit River is commonly known as the Ecorse River. The Ecorse River is included in the hydraulic computer model and is considered by reference to be part of the NBECD in this report. The NBECD runs through six communities: Romulus, Westland, Dearborn Heights, Allen Park, Lincoln Park and Ecorse.

In addition to the NBECD, a major tributary to the NBECD, called the Reeck Drain, was modeled. The Reeck Drain and the NBECD are hydraulically interconnected via over-land flooding and incorporating the Reeck Drain into the NBECD model was critical for modeling. The Reeck Drain begins at its headwaters near the intersection of the I-94 Expressway and Telegraph Road in Taylor and runs in an easterly direction parallel to the NBECD until it discharges to the NBECD at the confluence near RS 4.91 in Allen Park between I-94 and Allen Road. The Reeck Drain is located on the south side of Van Born Road. Approximately 3.5 miles of the Reeck Drain were modeled for this analysis.

The development and results of the existing conditions hydraulic model are presented in this report.

Survey and Mapping

A survey of the hydraulic characteristics and geometry of the NBECD and the Reeck Drain was completed. Along the NBECD, the survey started at the NBECD outlet to the Detroit River and ended at the headwaters of the drain at Ecorse Road in the City of Romulus. River mile stationing (RS) was established beginning at the confluence with the Detroit River (RS 0.00) and ending at Ecorse Road (RS 16.91).

Along the Reeck Drain, the survey started at the confluence with the NBECD (RS 4.91 on NBECD; RS 0+00 on Reeck) in the City of Allen Park and ended at the headwaters (RS 3.39) near I-94 and Telegraph Road in the City of Taylor.

Survey Control

A survey control network was established. The control network provided reference points to ensure that survey work along the NBECD was completed using the same datum and coordinate system. Control points were referenced with Michigan Department of Transportation reference stations (MDOT CORS) and National Geodetic Survey (NGS) control points and analyzed with NGS On-line Positioning User Service (OPUS).

Correlation between National Geodetic Vertical Datum of 1929 (NGVD 29) and North American Vertical Datum of 1988 (NAVD 88) was computed at primary control points.

High Water Marks

High water marks along the NBECD and Reeck Drain for the May 2004 storm event were obtained from landowners, photographs and field inspections during the May storm event. Elevations of high water marks were obtained. Table 2-9 summarizes the high water marks along the NBECD and Reeck Drain.

Table 2-9
High Water Marks for the May 2004 Flood

Point	River MI Station	State Plane Coordinates		Location	HEC-RAS Model May 2004 Calibration Storm Run 1B (ft)	May 2004 Storm High Water Mark (ft)	Source
		Northing	Easting				
NBECD							
1	5.35	283,208.640	13,437,373.240	Downstream of I-94	593.6	593.3	Bridge High Water Mark
2	6.07	284,219.725	13,434,922.601	Larme	596.3	596.6	Sanitary Sewer Manhole
3	6.83	284,064.720	13,431,612.250	Edgewood & Powers	599.7	599.9	Resident's Front Steps
4	7.22	283,000.263	13,430,009.902	Jackson	601.7	602.2	High Water on Bridge Sidewalk
5	8.15	283,108.200	13,425,428.470	Monroe	605.9	605.5	Bridge High Water Mark (Downstream)
6	8.51	283,318.850	13,423,943.660	Pardee	607.6	607.1	Bridge Low Chord (Upstream)
7	8.81	284,187.260	13,422,691.400	Parker	608.8	607.4	Footbridge Low Chord (Upstream)
8	9.10	283,459.630	13,421,395.180	Madison	609.6	610.0	Bridge High Water Mark (Upstream)
9	9.43	283,171.717	13,419,853.234	Telegraph	610.4	610.7	Resident's Porch
10	9.57	282,896.842	13,419,371.535	Banner	611.0	610.8	Level on Hydrant
11	9.96	282,654.138	13,417,278.099	Powers & Hanover	612.5	612.9	Resident's Front Yard
12	10.20	282,404.602	13,415,728.089	Gulley	613.5	614.6	Resident's Front Yard
13	10.46	282,272.546	13,414,813.766	Beech-Daly	614.4	614.3	* USGS Gage
Reeck Drain							
14	16+88			Quandt & Euclid	592.8	593.1	Neighborhood Street Flooding
15	23+96			Quandt & Hanfor	592.9	593.1	Neighborhood Street Flooding
16	99+58			Buckingham near Van Born	600.4	600.0	Resident's Front Yard

*** Indicates a high level of confidence in high water mark level.**

Contour Mapping

Aerial photography and survey ground control were obtained in the spring of 2005. Based upon aerial photography, contour maps were prepared for the land adjacent to the NBECD and Reeck Drain and within the limits of the currently established FEMA 100-year flood plain. The contour maps were used for flood mapping and development of the hydraulic model.

A digital terrain model (DTM) in the form of a triangular irregular network (TIN) was developed based upon the contour mapping. The aerial terrain map is presented in Appendix C.

Survey of Drain Crossings

There are 81 drain crossings over the NBECD within the study limits, including 50 public roads, eight railroads, 13 footbridges, nine private drives and one long enclosure of the NBECD. There are 18 drain crossings over the Reeck Drain, including five public roads, two railroads, one footbridge, eight parking lots, one barrier wall and one long enclosure of the Reeck Drain. All of these drain crossings were surveyed.

Table 2-10 shows the survey results for each drain crossing and the river stationing (RS) assigned to each crossing. The drain crossing data were entered into the hydraulic model along with cross section data at the face of each crossing. A minimum of one cross section was obtained at each crossing.

Table 2-10
Stationing and Description of River Crossings

No.	River Mile	Crossing Name	Crossing Type	Span (ft)	Rise (ft)	Single Opening Area (ft^2)	No. of Openings	Total Available Open Waterway Area (ft^2)	Approximate Existing Crossing Waterway Area (ft^2)
1	0.09	W. Jefferson Ave.	Concrete Bridge	110	14	0	3	0	1300
2a	0.22	Railroad Tracks	Steel Truss	111	9	0	4	0	1015
2b	0.23	Railroad Tracks	Steel Truss	120	9	0	3	0	1140
2c	0.24	Railroad Tracks	Steel Truss	109	9	0	3	0	1035
2d	0.26	Railroad Tracks	Concrete Bridge	81	9	0	7	0	623
3	1.40	Southfield Rd.	Concrete Bridge	35	4	0	2	0	329
4	2.44	Austin Ave.	Concrete Bridge	16	6	0	3	0	286
5	2.74	Victoria Ave	Concrete Bridge	13	4	71	2	143	107
6a	2.83	Fort St. (85)	Concrete Bridge	24	9	220	2	441	182
6b	2.85	Fort St. (85)	Concrete Bridge	24	9	220	2	441	182
7	3.12	Lafayette Blvd.	Concrete Bridge	28	6	140	1	140	160
8	3.20	Fisher Fwy. (I-75)	Concrete Bridge	23	14	0	1	0	330
9	3.30	John Papalas Dr.	CMP Low Profile Arch	31	10	0	1	0	255
10	3.41	Railroad Tracks	Concrete Bridge	11	7	74	2	149	164
11	3.43	Private Drive	Concrete Bridge	32	9	0	1	0	175
12	3.54	Porter Ave.	Concrete Bridge	28	7	0	1	0	182
13	3.74	Dix Hwy.	Concrete Bridge	12	9	108	2	216	110
14	3.84	Footbridge	Steel Truss	76		0	1	0	131
15	3.92	Frank Ave.	Concrete Bridge	26	6	0	1	0	144
16	4.20	Stanley Ave.	Concrete Bridge	14	6.5	0	1	0	90
17	4.61	Allen Rd.	Concrete Bridge	10	10	100	2	200	222
18	4.83	City Park	Steel Truss Footbridge	40	Irregular	0	1	0	366
19	4.92	Railroad	Concrete Bridge	14	6	98	2	196	211
20	4.97	Railroad	Footbridge	39	Irregular	0	1	0	131
21	5.07	Railroad	Concrete Bridge	11	7	80	2	161	136
22	5.35	Baker College	Wood Plank Bridge	35	6	0	2	0	143
23a	5.43	E. I-94	Concrete Bridge	12	Irregular	0	2	0	202
23b	5.47	W. I-94	Concrete Bridge	12	Irregular	0	2	0	207
24	5.92	Shenandoah Ave	Concrete footbridge	36	Irregular	0	1	0	163
25	6.07	Larme Ave./ Keppen Ave.	Double CMPA	7	5	37	2	74	74
26	6.16	Russell Ave.	Double CMPA	7	5	37	2	74	74
27	6.24	Watson Ave.	Double CMPA	7	5	37	2	74	74
28	6.31	Euclid Ave.	Double CMPA	7	5	37	2	74	74
29	6.37	Southfield Rd. (M-39)	Bridge & Double Pipes	20' x 8' Bridge and 2 - 5.5' Dia RCP		0	3	0	180
30	6.70	Bedford St.	Concrete footbridge	39	6	0	1	0	168
31	6.82	Edgewood St.	Concrete Bridge	30	6	180	1	180	84
32	6.89	Kingston St.	Concrete Bridge	30	6	180	1	180	61
33	7.12	Pelham St.	Concrete Bridge	14	5	77	1	77	82
34	7.22	Jackson St.	Concrete Bridge	30	5	165	1	165	105
35	7.42	Hanover St.	Concrete Bridge	26	4	119	1	119	61
36	7.48	Hipp St.	Double CMPA	6	4	23	2	46	46
37	7.55	Polk St.	Double CMPA	6	4	23	2	46	46
38	7.65	Hanover St.	Concrete Bridge	29	6	177	1	177	115
39	7.70	Harding Ave.	Wood Plank Bridge	40	-	0	1	0	185
40	7.75	Gertrude Ave.	Steel Truss	40	-	0	1	0	188
41	7.83	Campbell St.	Concrete Bridge	29	5	147	1	147	100

Table 2-10 (continued)
Stationing and Description of River Crossings

No.	River Mile	Crossing Name	Crossing Type	Span (ft)	Rise (ft)	Single Opening Area (ft^2)	No. of Openings	Total Available Open Waterway Area (ft^2)	Approximate Existing Crossing Waterway Area (ft^2)
42	7.84	Hanover St.	Concrete Bridge	31	6	204	1	204	141
43	7.97	Williams St.	Double CMPA	8	5	42	2	84	85
45	8.16	Monroe St.	Concrete Bridge	31	6	204	1	204	127
46	8.51	Pardee Ave.	Concrete Bridge	22	8	180	1	180	144
47	8.81	Parker Ave.	Steel Truss Footbridge	50	-	0	1	0	208
48	9.10	Madison Ave.	Concrete Bridge	31	6	204	1	204	152
49a	9.41	Telegraph Rd. North	Concrete Bridge	27	8	216	1	216	222
49b	9.43	Telegraph Rd. South	Concrete Bridge	29	6	174	1	174	190
50	9.57	Banner Ave.	Steel Truss Footbridge	36	7	252	1	252	182
51	10.20	Gulley St.	Concrete Bridge	26	9	238	1	238	102
52	10.46	Beech Daly Rd.	Concrete Bridge	35	6	210	1	210	143
53	10.74	Old Driveway	CMPA	11	7	74	1	74	75
54	10.80	Private Footbridge	Concrete I-Beam	36	-	0	1	0	182
55	10.84	Bayham St.	CMPA	9	6	54	1	54	54
56	11.57	Inkster Rd.	Concrete Bridge	14	5	72	1	72	76
57	11.75	Van Born Rd.	Concrete Bridge	20	6	123	1	123	63
58	12.46	Beverly Rd.	Concrete Bridge	25	4	100	1	100	68
58b	12.77	Private Drive	CMPA	8	5	42	1	42	42
59	13.10	Ecorse Rd.	Concrete Bridge	12	5	60	1	60	61
60	13.78	Middlebelt Rd.	Concrete Bridge	14	5	70	1	70	55
61	13.84	Private Drive	RCP	-	5	23	1	23	24
62	14.34	Smith Rd.	CMPA	8	5	42	1	42	42
63	15.30	Merriman Rd.	-	-	-	-	-	-	-
64	15.91	Venoy Rd.	Concrete Bridge	8	5	44	1	44	38
65	16.41	Henry Ruff Rd.	CMP	-	4	15	1	15	16
66	16.45	Sargent Rd.	CMPA	5	3	15	1	15	16
67	16.54	Private Drive	CMPA	7	5	32	1	32	32
68	16.55	Private Footbridge	I-Beam	21	-	0	1	0	38
69	16.62	Private Drive	CMPA	7	5	32	1	32	32
70	16.63	Private Footbridge	I-Beam	22	-	0	1	0	48
71	16.65	Private Drive	I-Beam	15	-	0	1	0	37
72	16.70	Private Drive	RCP	-	5	19	1	19	20
73	16.71	Private Drive	RCP	-	5	19	1	19	20
74	16.75	Private Drive	RCP	-	5	19	1	19	20
75	16.91	Ecorse Rd.	Concrete Box	8	5	44	1	44	30

Reeck Drain

No.	River Mile	Crossing Name	Crossing Type	Span (ft)	Rise (ft)	Single Opening Area (ft^2)	No. of Openings	Total Available Open Waterway Area (ft^2)	Approximate Existing Crossing Waterway Area (ft^2)
1	0.47	Railroad	Circular RCP	4	4	13	1	13	13
2	0.66	Enterprise Drive	Circular RCP	5	5	20	1	20	20
3	0.78	Parking Lot	CMP	6	6	24	1	24	24
4	0.84	Parking Lot	Concrete Box Culvert	6	3	18	1	18	16
5	0.9	Parking Lot	Concrete Box Culvert	8	2	14	1	14	14

Table 2-10 (continued)
Stationing and Description of River Crossings

No.	River Mile	Crossing Name	Crossing Type	Span (ft)	Rise (ft)	Single Opening Area (ft^2)	No. of Openings	Total Available Open Waterway Area (ft^2)	Approximate Existing Crossing Waterway Area (ft^2)
6	0.96	Parking Lot	Triple RCP	3	3	7	3	21	18
7	1.09	Parking Lot	Triple RCP	2 Culverts - 3 1 Culvert - 2	2 Culverts - 3 1 Culvert - 2	6	3	17	15
8	1.15	Parking Lot	Triple RCP	3	3	7	3	21	16
9	1.19	Parking Lot	Triple RCP	3	3	7	3	21	14
10	1.27	Southfield Road	Triple CMP on D/S Triple Concrete on U/S	(us)Culvert 1 - 3 (us)Culvert 2 - 2 (us)Culvert 3 - 3	(us)Culvert 1 - 3 (us)Culvert 2 - 2 (us)Culvert 3 - 3	6	3	17	13
11	1.37	Parking Lot	Double RCP	3	3	7	2	14	14
12	1.46	Footbridge	Steel I-Beam Footbridge	18	3	48	1	48	48
13	1.64	Railroad	RCP	3	3	7	1	7	7
14	1.77	I-94	Elliptical RCP	5	3	11	2	22	11
15	2.33	Storm sewer Enclosure **	Concrete storm sewer	Upstream - 4	Upstream - 2	10	1	10	10
				Downstream - 2	Downstream - 2	3	4	13	13
16	3.16	Monroe Road and Avalon Road	Double RCP	2	2	3	2	6	6
17	3.27	Barrier Wall	Double RCP	3	3	5	2	10	10
18	3.39	I-94	RCP	3	3	7	1	7	7

A section of the NBECD is enclosed with a series of storm sewers that range in size and shape and extends about 4,900 feet across Smith Road and Merriman Road in Romulus. Available as-built drawings for the series of storm sewers were reviewed. Some sections of these sewers were surveyed. The series of storm sewers range in size from twin 34-inch by 53-inch horizontal elliptical concrete sewers to an 84-inch diameter circular concrete sewer. The series of storm sewers was converted to a hydraulically equivalent single storm sewer for modeling simplicity. The hydraulically equivalent pipe was a 6.2-foot diameter circular concrete sewer.

A section of the Reeck Drain is also enclosed. The enclosure consists of a 2.83-foot rise by 4.42-foot span horizontally-laid elliptical concrete sewer that extends about 3,120 feet from Merrick Road to Pelham Road in Taylor. Available as-built drawings for this enclosure were reviewed. The elliptical storm sewer discharges to a chamber at Pelham

Road. The chamber allows flow to an open channel portion of the Reeck Drain downstream (east) of Pelham Road through four 24-inch diameter culverts. This outlet has a larger total waterway opening area than the elliptical sewer. The elliptical sewer was conservatively modeled as a series of three single-sized long culverts without the four 24-inch diameter culverts at Pelham Road.

Survey of Cross Sections

The majority of the NBECD and Reeck Drain consists of trapezoidal cross sections typical of open channel construction. Obstructions due to sediment bars and debris in the drain are common. In-stream cross sections were surveyed at 185 locations along the NBECD. Each cross section was mapped and assigned a reference line and river stationing based on its stream mile location. Beginning at the confluence with the Detroit River, stream mile distance was measured in the upstream direction. A typical cross section of the NBECD is shown in Figure 2-9.

Along the Reeck Drain, each cross section was assigned a reference line based on its stream location in feet. Beginning at the confluence with the NBECD, stream feet distance was also measured in the upstream direction. A typical cross section of the Reeck Drain is shown on Figure 2-10.

Figure 2-9
Typical Cross Section for NBECD

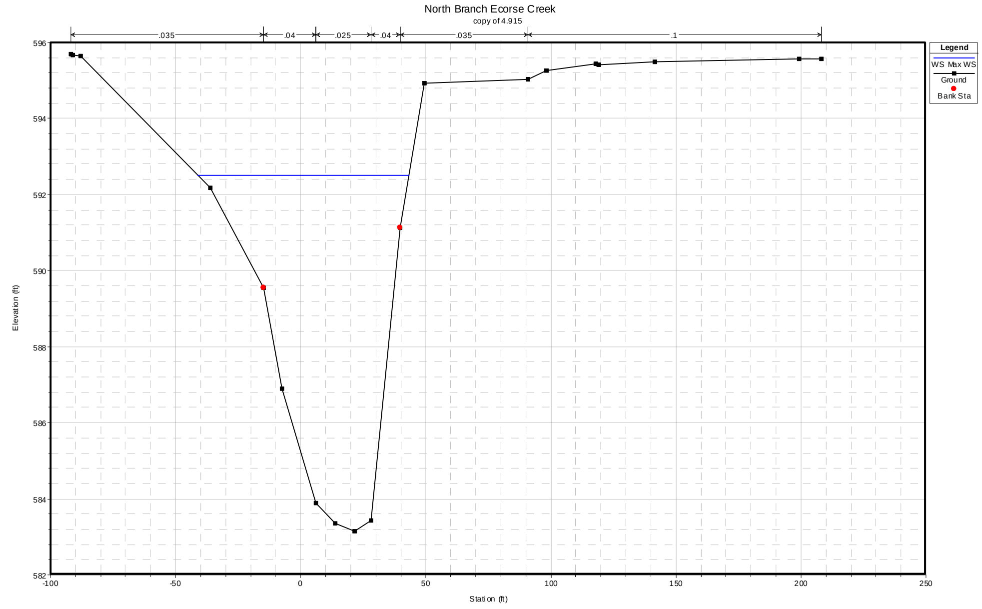
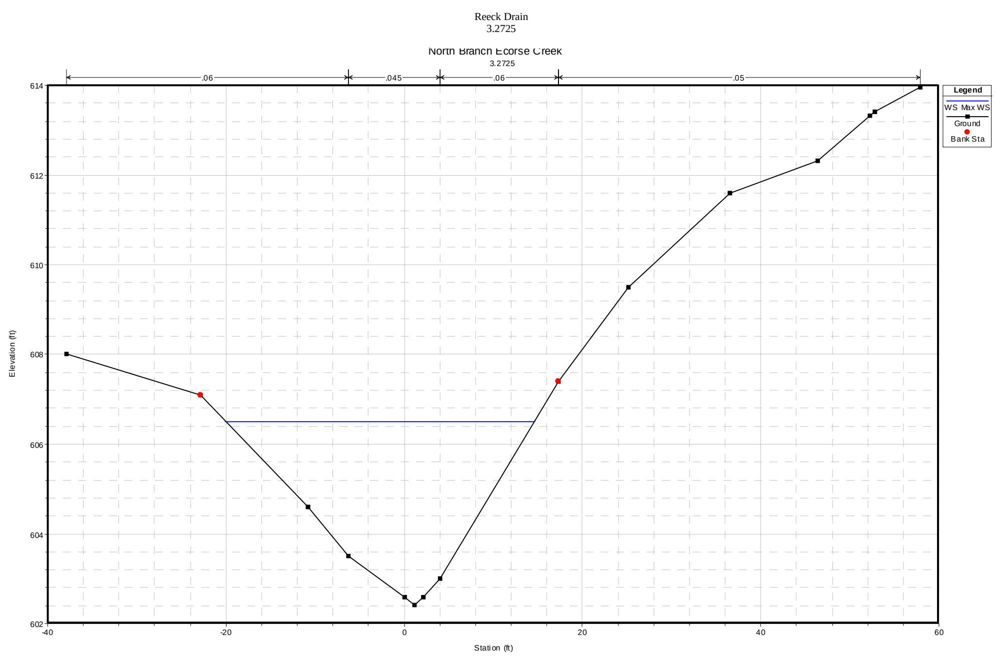


Figure 2-10
Typical Cross Section for Reeck Drain



Cross sections were extended to the limits of the known floodplain. In-stream cross sections were merged with the DTM of the known floodplain. The extended cross sections were coded into the model along with corresponding reach lengths and Manning's roughness coefficients.

A map of the cross section locations can be found in Appendix C.

Hydrologic Model

Runoff hydrographs were calculated using the HEC-HMS hydrologic model. The results of the hydrologic modeling are presented in the Hydrologic Model Development. The runoff hydrographs were entered into the hydraulic model at the load points along the NBECD as described in Task 2.1. The hydrographs were routed through the channel reaches and flooding levels and peak flood flow rates along the NBECD were predicted.

Hydraulic Model

A hydraulic model of the existing stream channel conditions was developed to represent the current stream channel geometry with the existing drain crossings. This configuration was used to calibrate the model. The hydraulic model was calibrated to a storm event in May 2004 as previously discussed in the Hydrologic Model Development. The hydraulic model was also run for a range of 24-hour, SCS Type II design storms: the 100-year, 50-year, 25-year, 10-year, 5-year and 2-year design storms.

The hydraulic computer model was prepared using the U.S. Army Corps of Engineers (USACE) HEC-RAS (Hydrologic Engineering Center River Analysis System) computer program, Version 3.1.3 (May 2005). HEC-RAS is the standard hydraulic model for the Federal Emergency Management Agency (FEMA) and the Michigan Department of Environmental Quality (MDEQ) flood studies.

Water surface profiles were computed using the HEC-RAS model. HEC-RAS supports steady-state and unsteady-state flow analyses. The NBECD and Reeck Drain were primarily modeled under unsteady-state conditions. Previous FEMA flood studies suggest that the extensive flooding along the NBECD and Reeck Drain results in significant attenuation of peak flood flow rates. Therefore, it was concluded that unsteady-state conditions would best simulate the existing conditions.

Model Assumptions

Manning's Roughness Coefficients

The Manning's roughness coefficients along the NBECD and Reeck Drain were based on the values estimated during the Condition Survey and are outlined on the maps provided in the Task 1 report. Vegetation and soil conditions of the stream channel and overbanks were noted during the survey. The observations were compared to text book values to determine the Manning's roughness coefficients to use in the model.

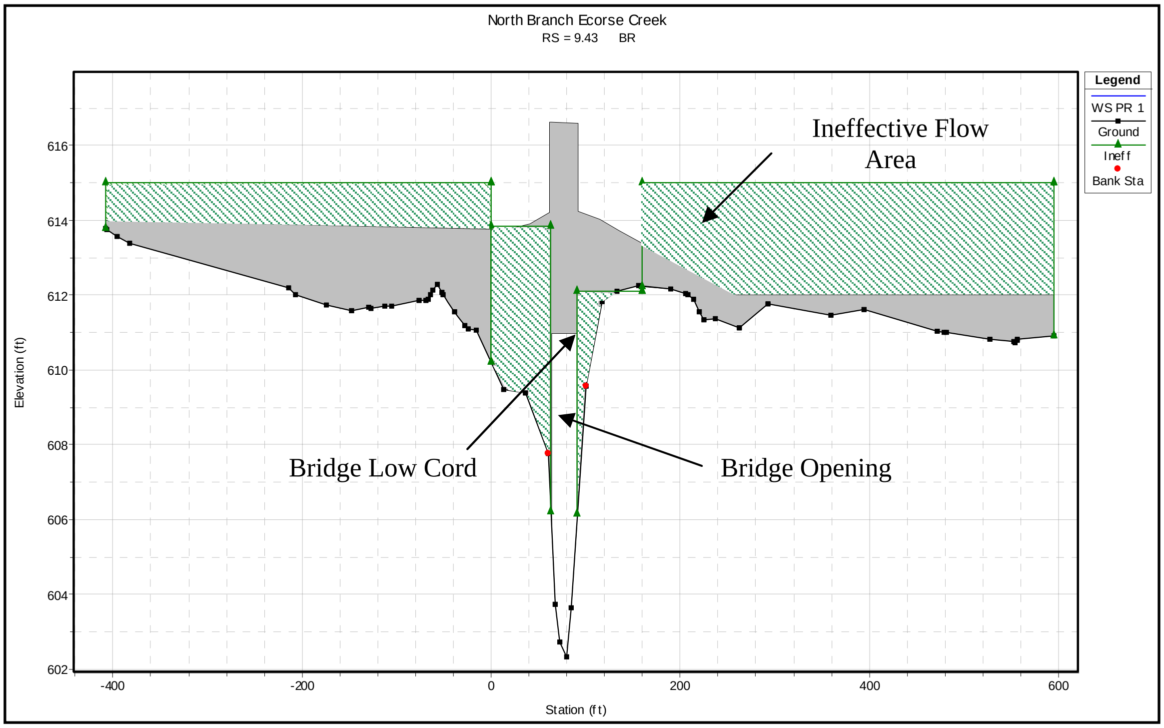
Along the NBECD, Manning's roughness coefficients typically ranged from 0.030 to 0.045 for the main channel to represent sandy to gravel channels, respectively. For the overbanks, Manning's roughness coefficients typically ranged from 0.030 to 0.10 to represent light vegetation and short grass to heavy vegetation and trees, respectively.

Along the Reeck Drain, Manning's roughness coefficients typically ranged from 0.035 to 0.06 for the main channel to represent sandy to gravel channels with medium vegetation, respectively. For the overbanks, Manning's roughness coefficients typically ranged from 0.030 to 0.06 to represent light vegetation and short grass to medium vegetation and light brush, respectively.

Ineffective Flow Areas

Ineffective flow areas in a flood plain are typically defined as areas with insignificant conveyance capacity and where velocities are very small. Determining and modeling these areas is important to accurately define the flow area of each cross section. The available flow area, in part, determines the predicted flooding levels along a reach. Areas where flooding occurs but conveyance is restricted by the overbank topography were determined to be ineffective. The area just upstream and downstream of bridge abutments were determined to be ineffective flow areas since flooding would occur in these areas but conveyance is limited to within the bridge openings. Figure 2-11 presents a typical cross section with ineffective flow areas on the overbanks and around the bridge opening.

Figure 2-11
Typical Cross Section with Ineffective Flow Areas



Crossing Hydraulics

The Energy Equation (Standard-Step Method) was used to calculate the head loss across all bridge and culvert crossings in the hydraulic model. This method produced the most stable flood profiles across the crossings and therefore increased the level of confidence in the results. Estimates of the weir and orifice flow coefficients were based on a review of published values and engineering judgment.

Dartmouth Road Lateral Overflow

For some storms, the NBECD floods residential areas in the City of Dearborn Heights and the flooding extends to the northern boundary with the City of Dearborn along Dartmouth Road. This floodwater can flow over the top of Dartmouth Road and enter Dearborn's combined sewerage system. This loss of storm water over Dartmouth Road was simulated as a series of lateral overflows in HEC-RAS to predict the overflow hydrographs and flood stage along the NBECD for a range of storms. Figure 2-12 presents the HEC-RAS lateral overflows along Dartmouth Road.

Van Born Road Lateral Overflow

During the May 2004 storm, flow over portions of Van Born Road was observed from the Reeck Drain in Taylor to the NBECD in Dearborn Heights and vice versa. This overtopping of Van Born Road was simulated as a series of lateral overflows along the Reeck Drain in HEC-RAS to more accurately predict the overflow hydrographs and flood stage along the NBECD and Reeck Drain for a range of storms. Figure 2-13 presents a profile of the HEC-RAS lateral overflows along Van Born Road.

Figure 2-12
Lateral Outflow Along Dartmouth Road

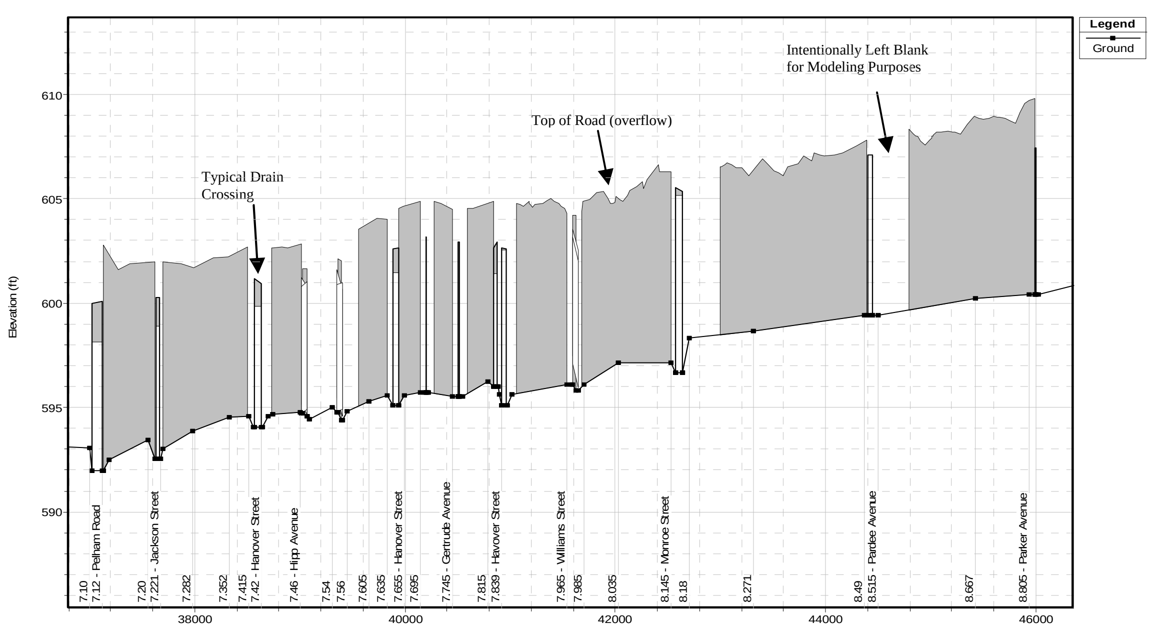
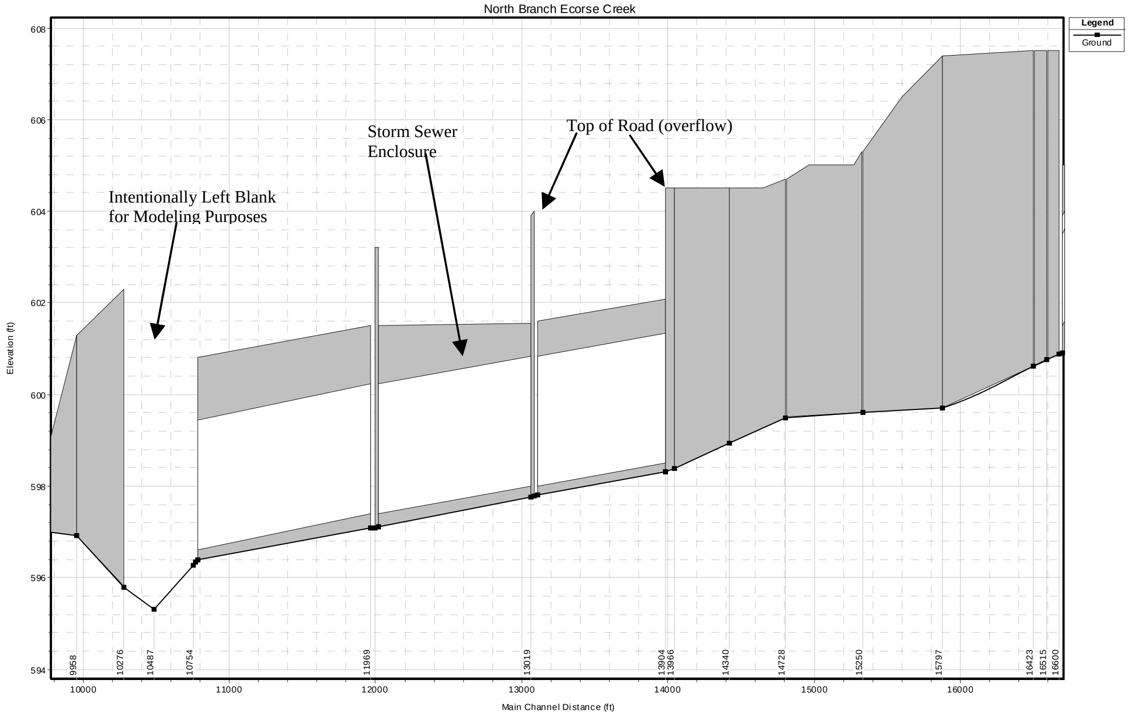


Figure 2-13
Lateral Overflow Along Van Born Road



NBECD Drain Outlet

The NBECD commences at its outlet into the Sexton-Kilfoil Drain near Council Point in the City of Lincoln Park. From that point, the Sexton-Kilfoil Drain commences at its outlet in the Detroit River near Jefferson Avenue in the City of Ecorse. The section of the Sexton-Kilfoil Drain located between the Detroit River and the NBECD is also referred to as the Ecorse River and is about one half of a mile long. The hydraulic model of the NBECD extends to the Detroit River.

A sensitivity analysis was performed to determine the impact of the Sexton-Kilfoil Drain flood flow rates and Detroit River levels on the NBECD.

Detroit River Analysis

The river levels in the downstream reaches of the NBECD from Austin Avenue, Lincoln Park, to its outlet are backwater affected by the Detroit River. The expected monthly minimum, mean and maximum Detroit River levels at the mouth of the NBECD were determined and were put into the hydraulic model as starting water surface elevations.

The expected river levels were interpolated using data from USACE river gages on the Detroit River at the Fort Wayne and the Gibraltar stations.

The monthly mean data for the period of record for each gage was obtained from the National Oceanic and Atmospheric Administration (NOAA) National Oceanographic Service (NOS) Center for Operational Oceanographic Products and Services Web site (http://www.co-ops.nos.noaa.gov/data_res.html). The period of record included data from 1901 through 2004 for the Fort Wayne gage, and 1909 through 2004 for the Gibraltar gage.

The monthly river level data obtained from NOAA are based on the International Great Lakes Datum (IGLD) of 1985. The river levels were converted to the NAVD 88 datum using the following relationships:

IGLD 85 + 0.75 feet = NGVD 29

NGVD 29 – 0.47 feet = NAVD 88

The monthly mean at the mouth of Ecorse River was linearly interpolated between the Fort Wayne and Gibraltar gages. The distance between the Fort Wayne and Gibraltar gages is about 84,000 feet. The mouth of Ecorse River is about 56,000 feet upstream of the Gibraltar gage.

Months that data was not available for both gages were excluded from this analysis. This resulted in an analysis period from 1939 through 2004, with a total of 64 years of data due to some data gaps during this period. The interpolated river levels at the mouth were sorted and ranked in descending order. The exceedance probability of each monthly river level was determined by dividing the monthly rank by the total number of monthly river levels. The calculated percentage was the exceedance probability percentage at the gage. Table 2-11 shows the monthly mean river levels at the two gages and at the mouth of Ecorse Creek that were exceeded one percent, four percent, 10 percent, 50 percent and 99 percent of the time.

Table 2-11
Monthly Mean River Levels by Exceedance Probability

Location	Monthly Mean River Levels by Exceedance Probability (feet – NAVD 88 Datum)				
	1%	4%	10%	50%	99%
Fort Wayne Gage	576.1	576.0	575.0	574.0	571.0
Interpolated to Ecorse Creek	575.6	575.1	574.7	573.4	570.6
Gibraltar Gage	574.5	574.1	573.6	572.3	569.6

The historical maximum and minimum monthly mean river levels were also determined from the data for the two gage locations and for the interpolated data at the mouth of Ecorse Creek. These river levels are provided in Table 2-12.

Table 2-12
Monthly Mean River Levels

Location	Monthly Mean River Levels (feet – NAVD 88 Datum)	
	Maximum Level (occurrence date)	Minimum Level (occurrence date)
Fort Wayne Gage	576.4 (July 1986)	570.8 (January 1936)
Interpolated to Ecorse Creek	575.9 (July 1986)	570.4 (February 1942)
Gibraltar Gage	574.8 (June 1986)	569.8 (November 1964)

The available daily mean data at the Fort Wayne gage and Gibraltar gage also were reviewed for the calibration period of May 20 through 27, 2004. The highest interpolated level was 573.6 feet NAVD88 on May 25, and the lowest level was 573.0 feet NAVD88 on May 20. The mean river level for this period was determined to be 573.4 feet in NAVD88. Table 2-13 shows river levels used in HEC RAS Model.

Table 2-13
Expected River Levels at the Mouth of Ecorse Creek

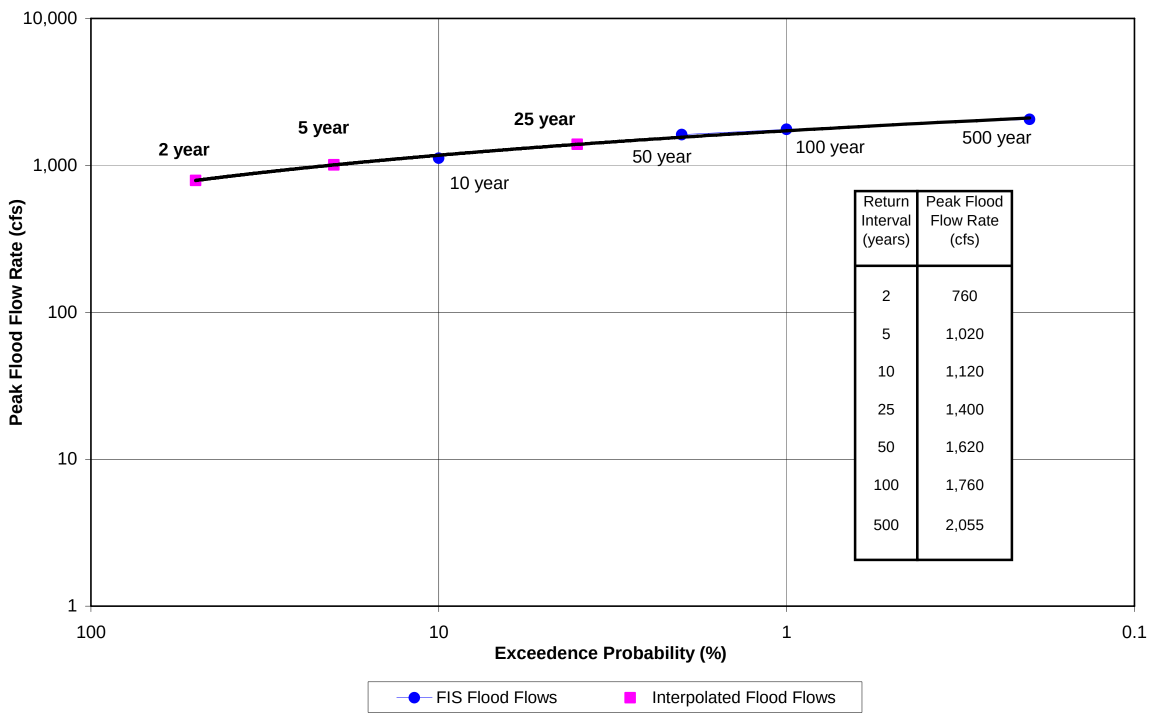
Event	Interpolated Detroit River Levels at Ecorse Creek (feet - NAVD 88 Datum)
Maximum Monthly Mean (July 1986)	575.9
Average Monthly Mean	573.4
Daily Mean During Calibration Period (May 2004)	573.4
Minimum Monthly Mean (February 1942)	570.4

Sexton-Kilfoil Drain

The flood flow rates for the Sexton-Kilfoil drain found in the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) reports were used for the 100-year, the 50-year and the 10-year storms. The flood flow rates for the 25-year, the 5-year and the 2-year storms were interpolated on a probability-log scale using the FIS flood flow rates. Figure 2-14 presents a plot of the FIS and interpolated flood flow rates for the range of storms.

The flood flow rates for the Sexton-Kilfoil Drain were put into the HEC-RAS model as constant flow rates for the design storm runs (Runs 4-9).

Figure 2-14
Exceedence Probability Versus Peak Flood Flow Rate
Sexton-Kilfoil Drain (South Branch Ecorse Creek)
Peak Flood Discharges at Mouth



Model Development

Sexton-Kilfoil Sensitivity Analysis

Two (2) model runs (Runs 3 and 3A) were completed to determine the impact of the Sexton-Kilfoil Drain flood flow rates and the Detroit River levels on the NBECD. One model was created assuming an average dry weather level of 573.4 feet on the Detroit River and no flow from the Sexton-Kilfoil Drain. Another model was created assuming a maximum monthly mean river level of 575.9 feet on the Detroit River and the 100-year flood flow rate of 1,760 cfs from the Sexton-Kilfoil Drain.

Reeck Drain Sensitivity Analysis

Three model runs (Runs 1B, 4, and 7) were completed that integrated the Reeck Drain with the NBECD. The runs were generated to determine the impact of the Reeck Drain on flood levels along the NBECD for the May 2004 calibration period and the 10-year and 100-year storms. Existing stream channel geometry (with sediment present at several drain crossings) and existing land use hydrologic conditions were assumed. For these three model runs, the Reeck Drain and NBECD were hydraulically connected via the following three mechanisms.

1. The Reeck Drain confluence with the NBECD
2. Lateral overflow over Van Born Road
3. The Monroe Road storm sewer (with partially open outlet)

Model Calibration

Hydrologic parameters assuming existing land use conditions were used for the May 2004 calibration run. In addition, existing stream channel geometry was assumed.

Stage data from a United States Geological Survey (USGS) stream gage on the NBECD at Beech-Daly Road in Dearborn Heights were obtained for the May 2004 calibration period. The USGS gage at Beech-Daly Road has only been installed since 2001. The USGS stream gage records river level. Flow rates are determined by using a rating curve developed for the gage. The rating curve is a plot of measured river levels versus measured flow rates. A regression analysis is performed on the data points to come up with a rating curve. There was a lower level of confidence for the USGS gage at Beech-Daly Road due to the short period of operation. Therefore, the recorded level data at Beech-Daly Road was used for the model calibration.

The observed high water marks along the NBECD for the May 2004 calibration period were also obtained and entered into the model for comparison purposes.

Design Storm Runs

Six design storm runs were also generated with HEC-RAS: the 100-year, 50-year, 25-year, 10-year, 5-year and the 2-year, 24-hour design storm runs. The design storm runs were created to determine the NBECD response to different storm events.

HEC-RAS was run in steady-state and unsteady-state modes. The unsteady-state mode was run to simulate the attenuation of flood hydrographs through the floodplain.

The purpose of the steady-state analysis was to compare the flood levels predicted with the HEC-RAS model to FIS flood levels on the NBECD for the 100-year storm. The 100-year flood flow rates were obtained from the FIS reports and entered into the calibrated HEC-RAS model and run to steady-state conditions. The hydraulic profile was compared to the FIS profile of the NBECD for the 100 year flood to verify the HEC-RAS model.

Model Results

Each model run consisted of a combination of land use (hydrologic) and stream channel (hydraulic) conditions. A matrix was developed that summarizes the HEC-RAS model runs generated to evaluate the existing channel. The matrix of model runs is presented on Table 2-14. The selected HEC-RAS model runs that are discussed in this chapter are the sensitivity runs (Runs 3 and 3A), calibration runs (Runs 1A through 2C) and design storm runs (Runs 4 through 9).

Table 2-14
North Branch Ecorse Creek Drain Flood Control Study
Matrix of Unsteady Flow HEC-RAS Model Runs

HEC-RAS Plan Short ID	HEC-RAS Plan Name	Historical or Design Storm	Land Use Condition	Detroit River Level (feet)	Sexton-Kilfoil Drain Flow Rate (cfs)	NBECD Stream Channel Condition	New Storm Water Detention Bains	Modeling Alternative No.	Computaional Time Step (seconds)	Continuity Error
NBECD										
Run 1A	Calibration Run 1A	May 2004 (HEC-HMS Run 1)	Existing	573.4	0	Existing	None	N/A	10	0.4%
Run 1B	Calibration Run 1B	May 2004 (HEC-HMS Run 2)	Existing	573.4	0	Existing	None	N/A	10	-0.6%
Run 1C	Calibration Run 1C	May 2004 (HEC-HMS Run 3)	Existing	573.4	0	Existing	None	N/A	5	-2.1%
Run 2A	Calibration Run 2A	May 2004 (HEC-HMS Run 1)	Existing	573.4	1,760	Existing	None	N/A	30	0.0%
Run 2B	Calibration Run 2B	May 2004 (HEC-HMS Run 2)	Existing	573.4	1,760	Existing	None	N/A	30	-0.3%
Run 2C	Calibration Run 2C	May 2004 (HEC-HMS Run 3)	Existing	573.4	1,760	Existing	None	N/A	20	-1.3%
Run 3	Run 3	100-year, 24-hour	Existing	573.4	0	Existing	None	N/A	30	-6.9%
Run 3A	Run 3A	100-year, 24-hour	Existing	575.9	1,760	Existing	None	N/A	30	-1.9%
Run 4	Run 4	100-year, 24-hour	Existing	573.4	1,760	Existing	None	N/A	20	-1.7%
Run 5	Run 5	50-year, 24-hour	Existing	573.4	1,620	Existing	None	N/A	10	-1.0%
Run 6	Run 6	25-year, 24-hour	Existing	573.4	1,400	Existing	None	N/A	12	-0.5%
Run 7	Run 7	10-year, 24-hour	Existing	573.4	1,120	Existing	None	N/A	5	0.2%
Run 8	Run 8	5-year, 24-hour	Existing	573.4	1,020	Existing	None	N/A	10	0.6%
Run 9	Run 9	2-year, 24-hour	Existing	573.4	760	Existing	None	N/A	3	16.0%
Reeck Drain										
Run 1B Reeck VB	Calibration Run 1B (Reeck) VB	May 2004 (HEC-HMS Run 2)	Existing	573.4	0	Existing	None	N/A	12	-10.1%
Run 4 Reeck	Run 4 (Reeck)	100-year, 24-hour	Existing	573.4	1,760	Existing	None	N/A	15	-8.9%
Run 7 Reeck	Run 7 (Reeck)	10-year, 24-hour	Existing	573.4	1,120	Existing	None	N/A	10	-5.1%

Sexton-Kilfoil Sensitivity Analysis

Based on the model results, a higher level on the Detroit River backwater affects the NBECD to Austin Avenue. Therefore, the flood levels in the NBECD are dependent on the Detroit River levels and are not significantly affected by the flood flow rates in the Sexton-Kilfoil Drain. It was concluded that the Detroit River and the Sexton-Kilfoil Drain do not significantly impact the flood levels on the NBECD upstream of Austin Avenue. Results from the sensitivity analysis are presented in Appendix C.

Reeck Drain Sensitivity Analysis

The HEC-RAS model predicted that incorporating the Reeck Drain caused a negligible impact to flood levels along the NBECD for the May 2004 calibration period and the 10-year and 100-year storms. For all three storms, the peak flood levels on the NBECD near the confluence with the Reeck Drain were slightly lower (about 0.5 feet) with the Reeck Drain in the model than without. The reduction in peak flood levels was due to attenuation of the inflow hydrographs generated with HEC-HMS when they were put into the HEC-RAS model and routed through the Reeck Drain. A comparison of flood levels along the NBECD with and without the Reeck Drain are presented in Appendix C.

Model Calibration

The model runs presented in this section consist of the existing stream channel geometry for the May 2004 calibration period and for the range of design storms with the Reeck Drain integrated with the NBECD.

A summary of the May 2004 calibration runs is presented on Table 2-15. The calculated peak flood stage data were compared to the Beech-Daly Road stream gage data and the observed high water marks along the NBECD for the May 2004 calibration period.

The peak recorded flood stage at Beech-Daly Road stream gage was 614.3 feet. Run 1B generated a peak flood stage of 614.4 feet that was the closest to the Beech-Daly stream gage data and observed high water marks. These conditions were determined to be the best fit and chosen as the final calibration. The parameters for Run 1B were used in the design storm runs.

The peak flood levels calculated for the six calibration runs are also presented on Table 2-15. A comparison of the predicted flood stage for Run 1B and the recorded stage data at the Beech-Daly stream gage for the May 2004 calibration period is shown on Figure 2-15. In addition, a comparison of the flood stage data predicted with HEC-RAS and the observed high water marks by river mile for the NBECD and Reeck Drain is shown on Figures 2-16 and 2-17, respectively. A summary of the flood stage data can also be found in Table 2-15. Refer to Hydrologic Model Development for a full description of the parameters in the table.

Figure 2-15
North Branch Ecorse Creek Drain
HEC-RAS vs. USGS Gage Stage at Beech-Daly Road
For May 2004 Storm

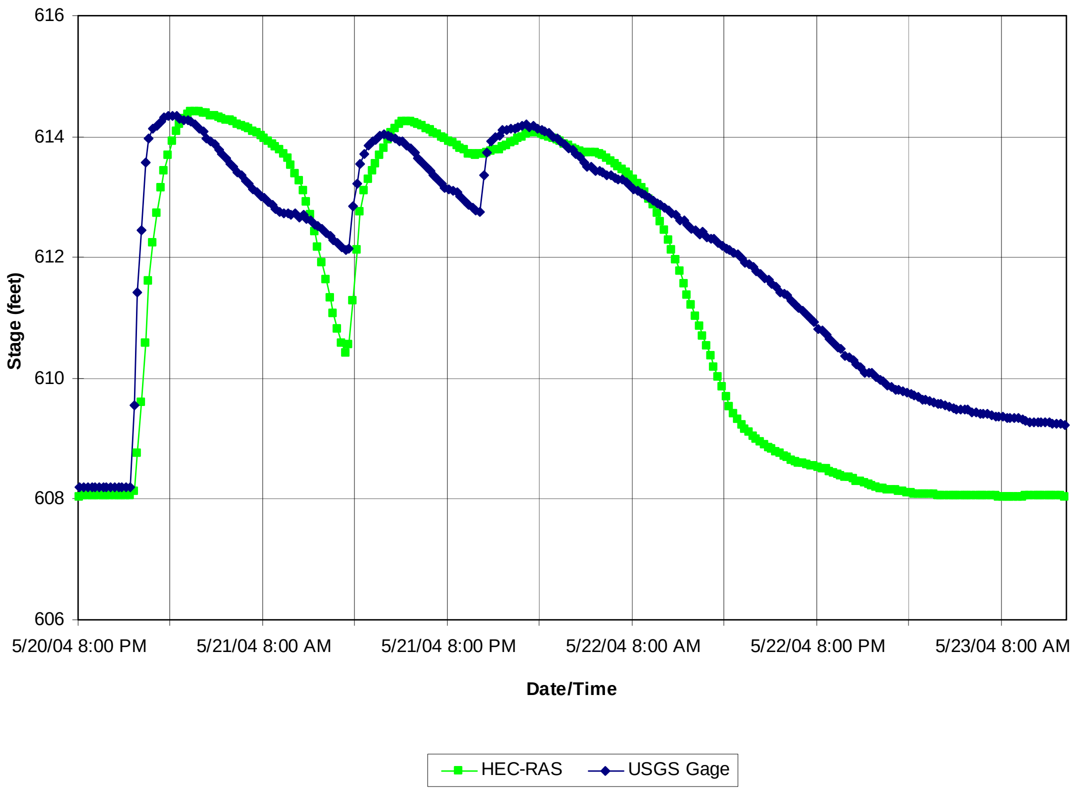


Table 2-15
Summary of May 2004 Calibration Runs

HEC-RAS Plan Name	Historical or Design Storm	RCNs	Area-Weighted RCN	Detroit River Level (feet)	Sexton-Kilfoil Drain Flow Rate (cfs)	Runoff Volume at Beech-Daly Rd		Peak Stage at Beech-Daly Rd (feet)
						(inches)	(acre-feet)	
Run 1A	May 2004 (HEC-HMS Run 1)	Land Use/Soils Based	77.8	573.4	0	1.92	1,004	614.0
Run 1B	May 2004 (HEC-HMS Run 2)	Land Use/Soils Based * 1.09	85.2	573.4	0	2.30	1,202	614.4
Run 1C	May 2004 (HEC-HMS Run 3)	Land Use/Soils Based	77.8	573.4	0	2.86	1,499	615.6
Run 2A	May 2004 (HEC-HMS Run 1)	Land Use/Soils Based	77.8	573.4	1,760	1.92	1,004	614.0
Run 2B	May 2004 (HEC-HMS Run 2)	Land Use/Soils Based * 1.09	85.2	573.4	1,760	2.30	1,202	614.4
Run 2C	May 2004 (HEC-HMS Run 3)	Land Use/Soils Based	77.8	573.4	1,760	2.86	1,499	615.6
Run 1B Reeck VB	May 2004 (HEC-HMS Run 2)	Land Use/Soils Based * 1.09	85.2	573.4	0	2.30	1,202	614.4

Note: Existing conditions soil parameters were used for all HEC-RAS model runs.

Figure 2-16
High Water Marks and HEC-RAS Flood Levels
Versus River Mile Along NBECD
May 2004 Storm

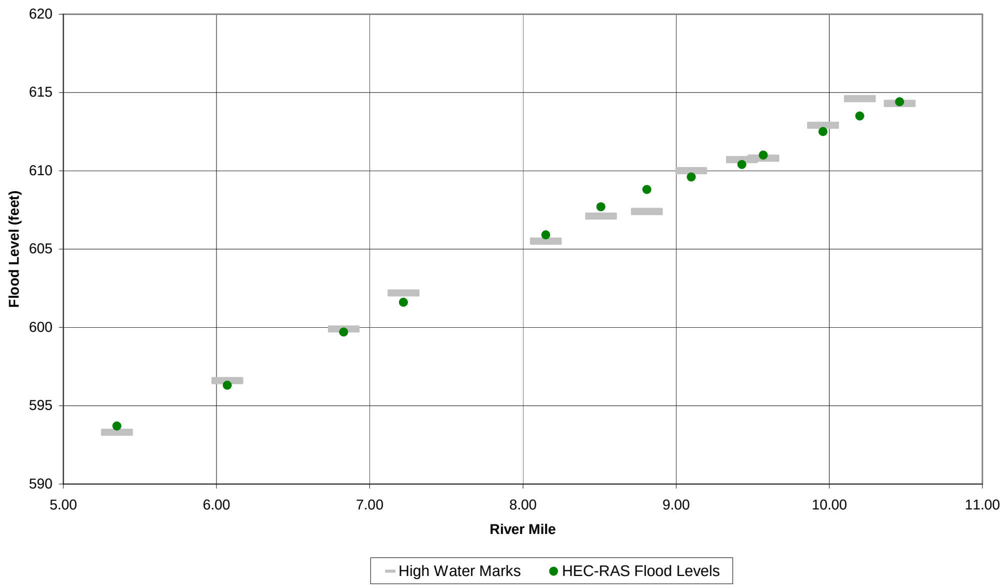
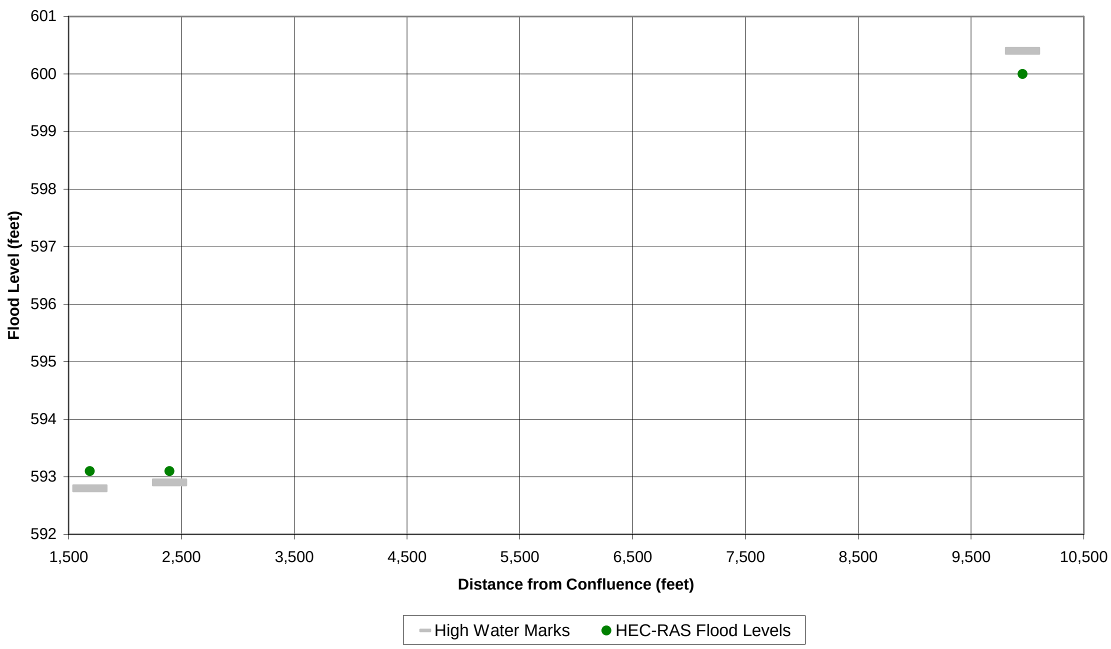


Figure 2-17
High Water Marks and HEC-RAS Flood Levels
Versus River Mile Along Reeck Drain
May 2004 Storm



Flow over Dartmouth Road was predicted to occur for the May 2004 calibration period (Run 1B). The overflow hydrograph at Dartmouth Road for the May 2004 calibration period is shown on Figure 2-18. The predicted peak flow rate and total overflow volume at Dartmouth Road is presented in Table 2-16 and used to compare to the design storm results.

Figure 2-18
Dartmouth Road Lateral Structure
Existing Conditions Model
May 2004 Calibration Period

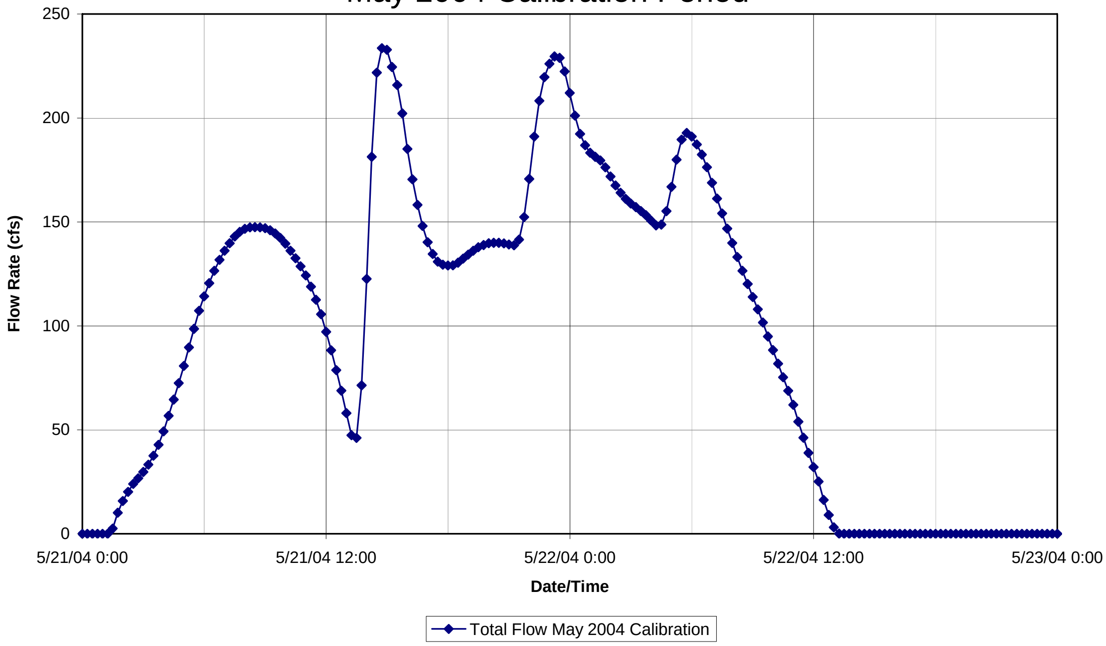


Table 2-16
Dartmouth Road Lateral Overflow Structure
Overflow Peak Flow Rates and Volumes

HEC-RAS Plan Short ID	HEC-RAS Plan Name	Historical or Design Storm	Peak Flow Rate (cfs)	Overflow Volume (acre-ft)
Run 1B	Calibration Run 1B	May 2004 (HEC-HMS Run 2)	234	380
Run 4	Run 4	100-year, 24-hour	669	813
Run 5	Run 5	50-year, 24-hour	572	635
Run 6	Run 6	25-year, 24-hour	450	472
Run 7	Run 7	10-year, 24-hour	283	283
Run 8	Run 8	5-year, 24-hour	178	154
Run 9	Run 9	2-year, 24-hour	55	30

Flow over Van Born Road was also predicted for the May 2004 calibration period. From Merrick Road to Pelham Road, flow over Van Born from the Reeck to the NBECD was predicted. Directly downstream of Pelham Road, a low point in the top of the road at elevation 599 feet allows significant flow over the road. For the May 2004 calibration period model run, flow over Van Born Road from the Reeck Drain to the NBECD and vice versa was predicted in this area. Two overflow hydrographs along Van Born Road for the May 2004 calibration period are shown on Figures 2-19 and 2-20. These two predicted lateral overflows, flow from the Reeck Drain to the NBECD between Merrick and Pelham Roads and "sloshing" of flow downstream of Pelham Road were observed during the May 2004 flood.

Figure 2-19
Van Born Road Lateral Structure
Overflow Between Monroe and Pelham Roads
May 2004 Calibration Period

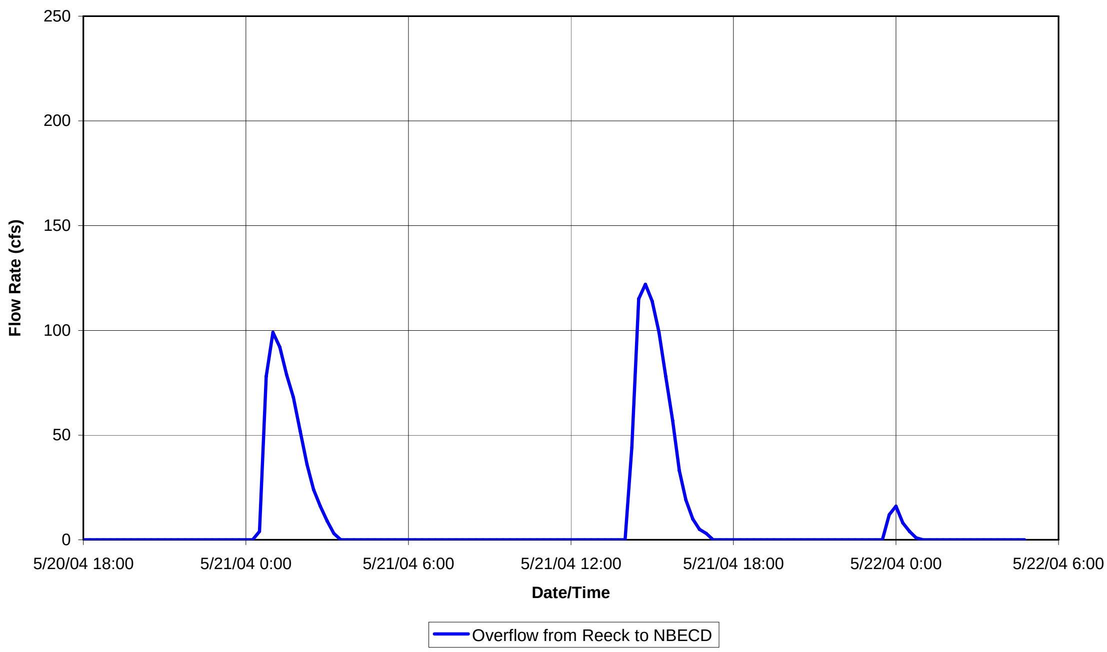
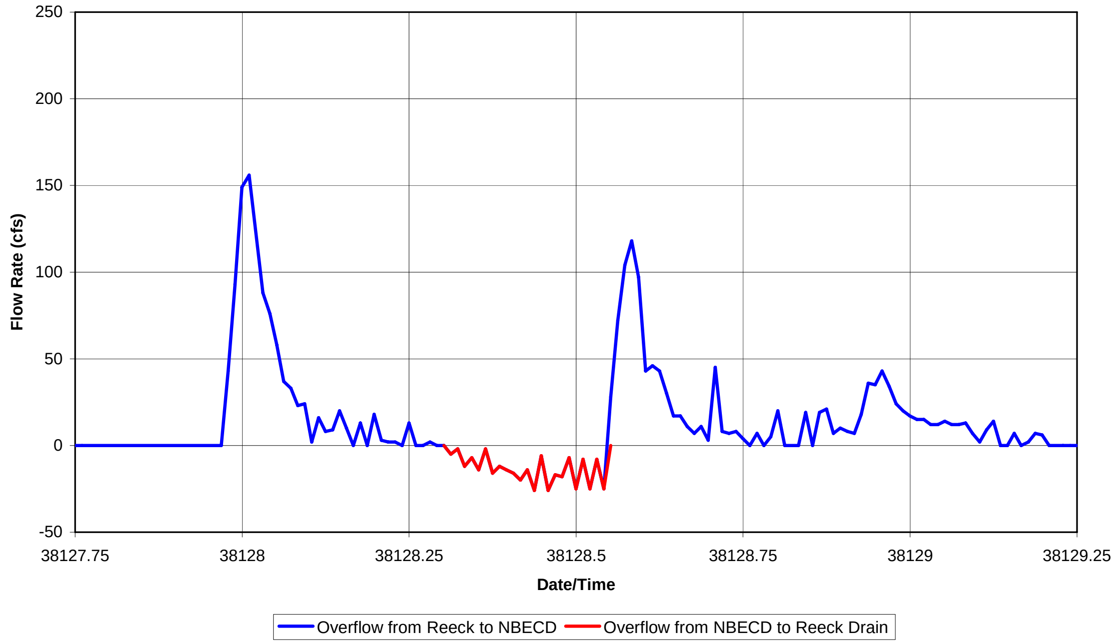


Figure 2-20
Van Born Road Lateral Structure
Overflow Downstream of Pelham Road
May 2004 Calibration Period



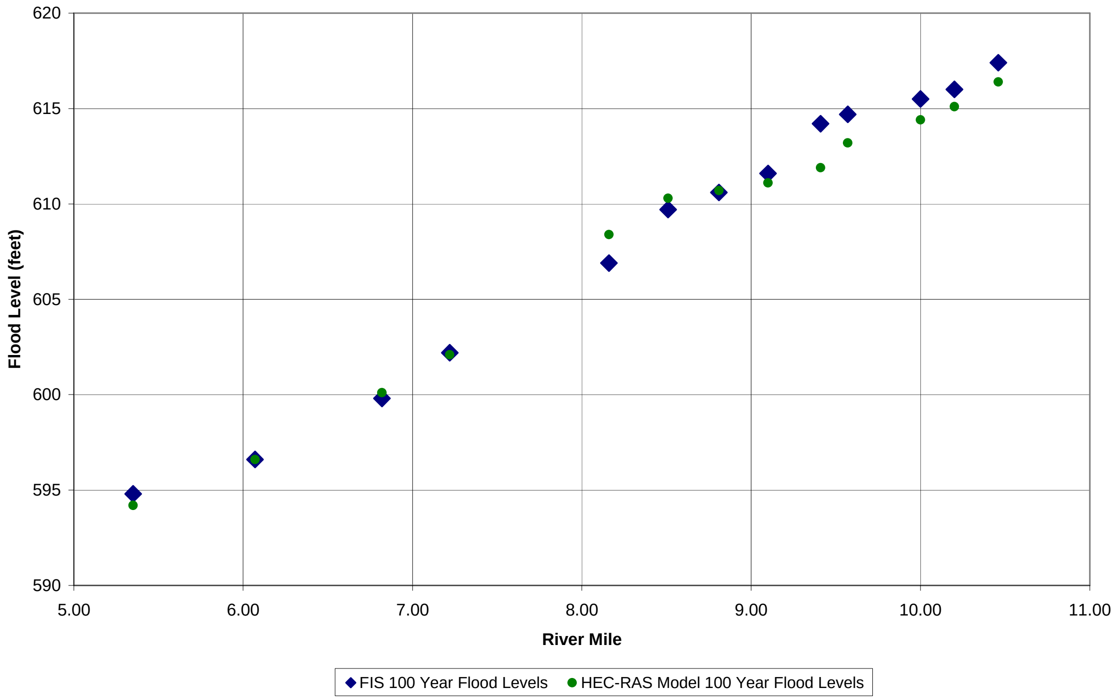
Results of the May 2004 calibration are identified in the following figures found in Appendix C.

- HGL Profile for the May 2004 Calibration Period
- Flood Map of the Modeled May 2004 Floodplain

Steady-State Design Storm Runs

The FIS 100-year flood flow rates were entered into the HEC-RAS model and run to steady-state conditions. The 100-year flood levels predicted with HEC-RAS were compared to the FIS 100-year flood levels along the NBECD for verification of the HEC-RAS model. Figure 2-21 presents the 100-year flood levels predicted with the HEC-RAS model and the FIS 100-year flood levels. In general, the HEC-RAS model predicted 100-year flood levels within 0.2 feet of the FIS 100-year flood levels, with the maximum difference of 2.3 feet.

Figure 2-21
FIS and HEC-RAS (Steady-State Conditions) Flood Levels Versus River Mile
100-Year, 24-Hour Storm



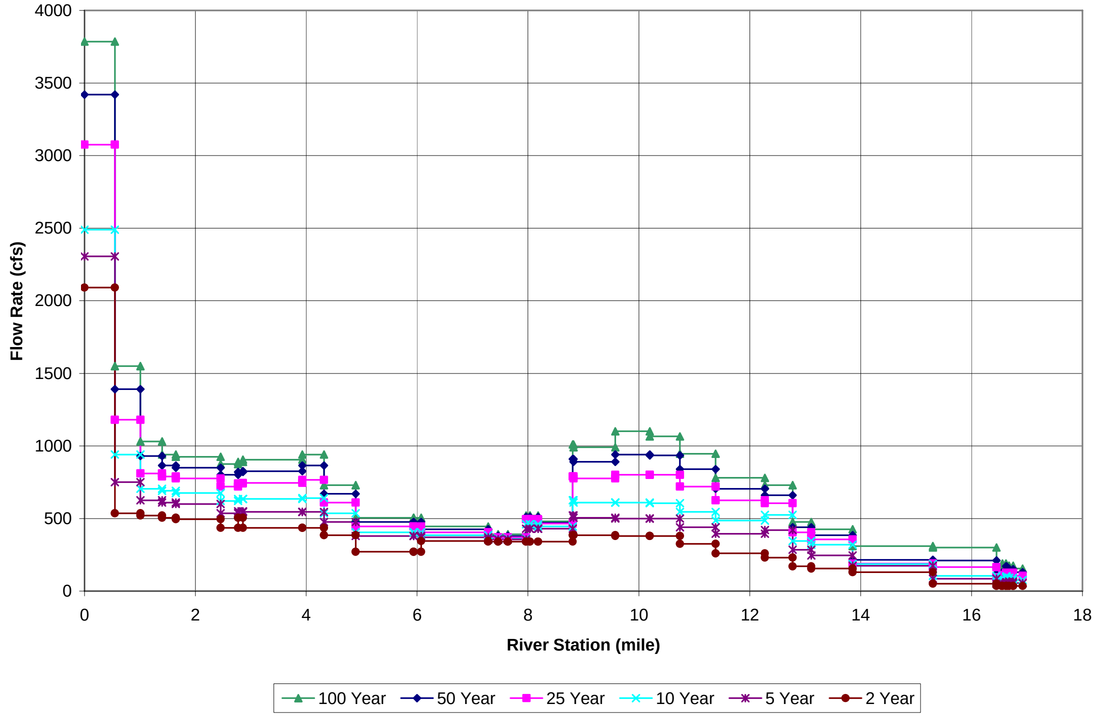
Unsteady-State Design Storm Runs

The unsteady-state mode better simulates the NBECD response to storm events than the steady-state mode. The unsteady-state mode simulates the restrictions at drain crossings and the attenuation of flood hydrographs through the floodplain.

Drain restrictions and areas prone to flooding were identified based on the unsteady-state analysis of the range of design storms. This was important to identify these areas for future mitigation work.

Minor instabilities were removed from the HEC-RAS 100-year flood flow profile. The instabilities artificially increased flood flow rates in several reaches. The smoothed flow profiles for the 2-year through the 100-year design storms are presented on Figure 2-22.

Figure 2-22
Unsteady-State HEC-RAS Peak Flow Rates vs. River Reach along the NBECD
Existing Stream Channel and Existing Land Use Conditions



The 100-year flood flow rates as routed and predicted with HEC-RAS were compared to the 100-year flood flow rates provided by the MDEQ and presented in the FIS reports. A comparison of the 100-year flood flow rates by reach are presented on Figure 2-23. The flood flow rates by reach for the range of design storms and the MDEQ and FIS 100-year flood flow rates are presented on Table 2-17.

In general, the flood flow rates generated in the HEC-RAS model were lower than the MDEQ and FIS steady-state 100-year flood flow rates. This is in contrast with the unrouted HEC-HMS results as outlined in Task 2.1, which were higher than MDEQ flow rates. The HEC-RAS flood flow rates by reach as shown on Figure 2-23 and presented on Table 2-17 were attenuated flood flow rates generated by routing the input hydrographs from the HEC-HMS hydrologic model through each NBECD channel reach in the unsteady state HEC-RAS model. The unsteady state model considers flood plain storage created from DTM, the lateral overflow to the City of Dearborn and the volume of storm water.

Figure 2-23
Predicted Flood Flow Rate Comparison
MDEQ, FEMA and HEC-RAS (Unsteady-State) Flood Flow Rates
100-Year, 24-Hour Storm

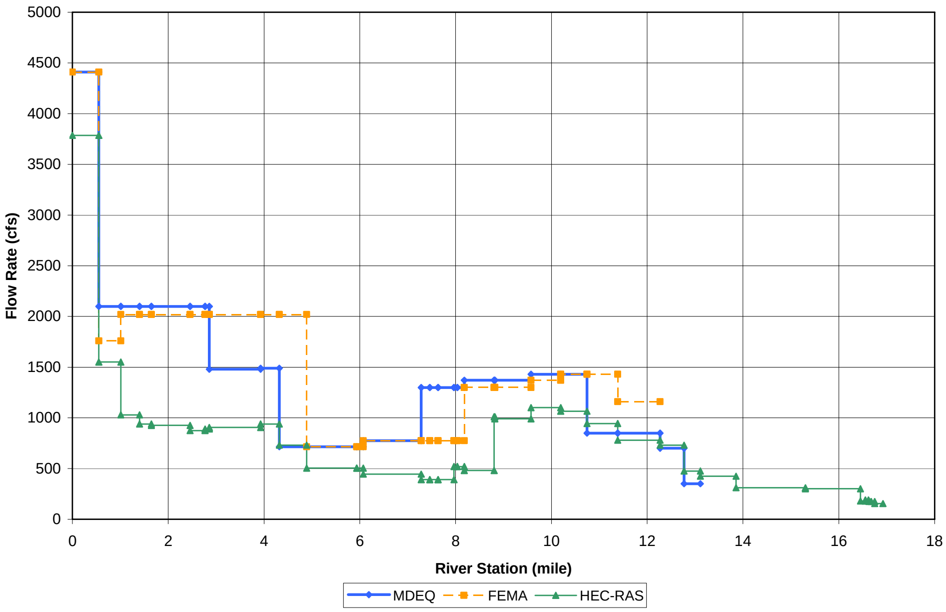


Table 2-17
Predicted Flood Flow Rate Comparison
MDEQ, FEMA and HEC-RAS (Unsteady-State) Flood Flow Rates
100-Year, 24-Hour Storm

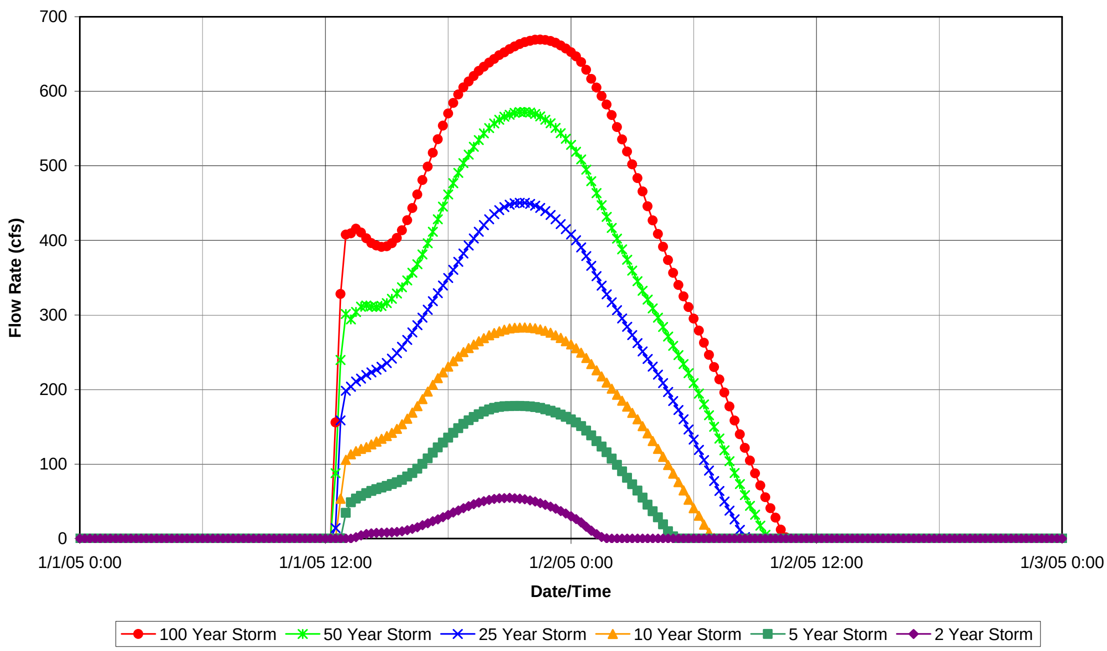
River Location	100-Year Flood Flow Rate (cfs)		
	MDEQ	FEMA FIS	HEC-RAS
Mouth	4,410	4,410	3,785
Southfield Rd	2,100	2,020	1,030
Fort St (M-85)	2,100	2,020	890
Stanley Ave	1,490	2,020	940
Allen Road	715	2,020	730
Pelham Road	715	715	505
Jackson Street	1,300	775	390
Williams Street	1,300	775	390
Monroe Street	1,300	775	520
Gulley Street	1,430	1,370	1,100
Inkster Road	850	1,430	945
Beverly Road	700	NA	730

Results of design storm runs are identified in the following figures found in Appendix C.

- HGL Profiles of the Range of Design Storms Under Existing Conditions
- HGL Profiles of the Sexton-Kilfoil and Detroit River Level Sensitivity Analysis

Flow over Dartmouth Road was predicted for the 2 through the 100-year, 24-hour design storms. The overflow profiles at Dartmouth Road for the range of design storms are shown on Figure 2-24. The predicted peak flow rates and total overflow volumes for the 2- through 100-year, 24-hour design storms are presented in Table 2-16.

Figure 2-24
Dartmouth Road Lateral Structure
Existing Conditions Model
Total Overflow for 2 through 100-Year, 24-Hour Storms



Flow over Van Born Road was predicted for the 10-year and 100-year, 24-hour design storms. The overflow profiles at Van Born Road for these design storms are shown on Figure 2-25. The predicted peak flow rates and total overflow volumes for the 10-year and 100-year, 24-hour design storms are presented in Table 2-18.

Figure 2-25
Van Born Road Lateral Structure
Existing Conditions Model
Total Overflow for 10-Year and 100-Year, 24-Hour Storms

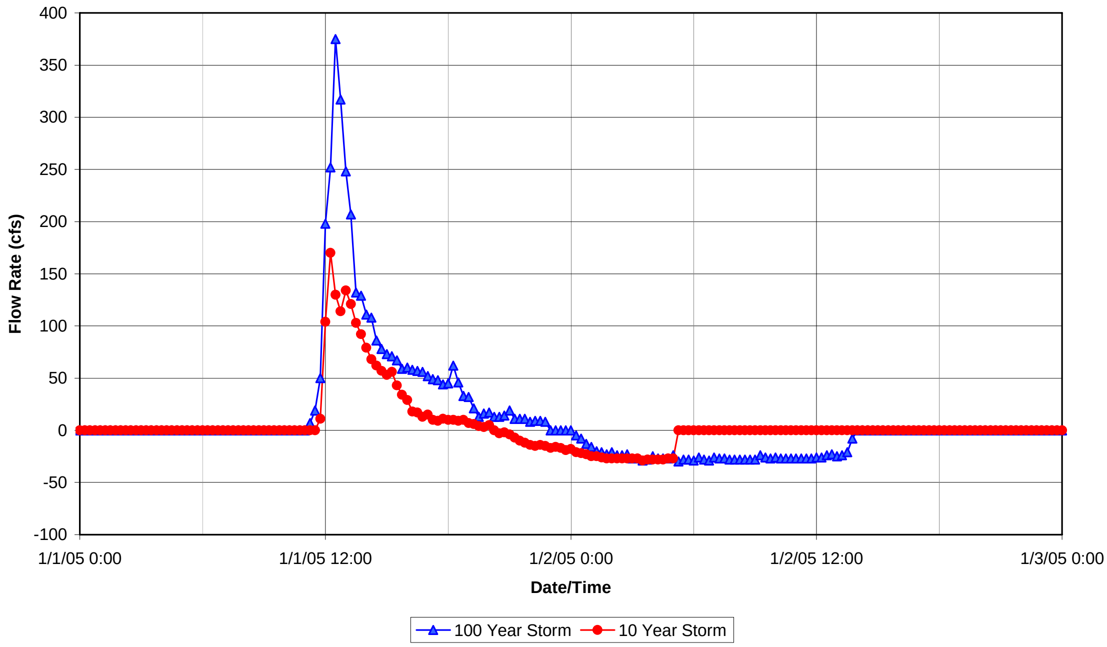


Table 2-18
Van Born Road Lateral Overflow Structure
Overflow Peak Flow Rates and Volumes

HEC-RAS Plan Short ID	HEC-RAS Plan Name	Historical or Design Storm	Peak Flow Rate (cfs)	Overflow Volume (acre-ft)
Reeck Drain to NBECD				
Run 1B Reeck VB	Calibration Run 1B (Reeck) VB	May 2004 (HEC-HMS Run 2)	175	73
Run 4 Reeck	Run 4 (Reeck)	100-year, 24-hour	375	707
Run 7 Reeck	Run 7 (Reeck)	10-year, 24-hour	170	33
NBECD to Reeck Drain				
Run 1B Reeck VB	Calibration Run 1B (Reeck) VB	May 2004 (HEC-HMS Run 2)	26	6.7
Run 4 Reeck	Run 4 (Reeck)	100-year, 24-hour	30	28
Run 7 Reeck	Run 7 (Reeck)	10-year, 24-hour	29	14.6