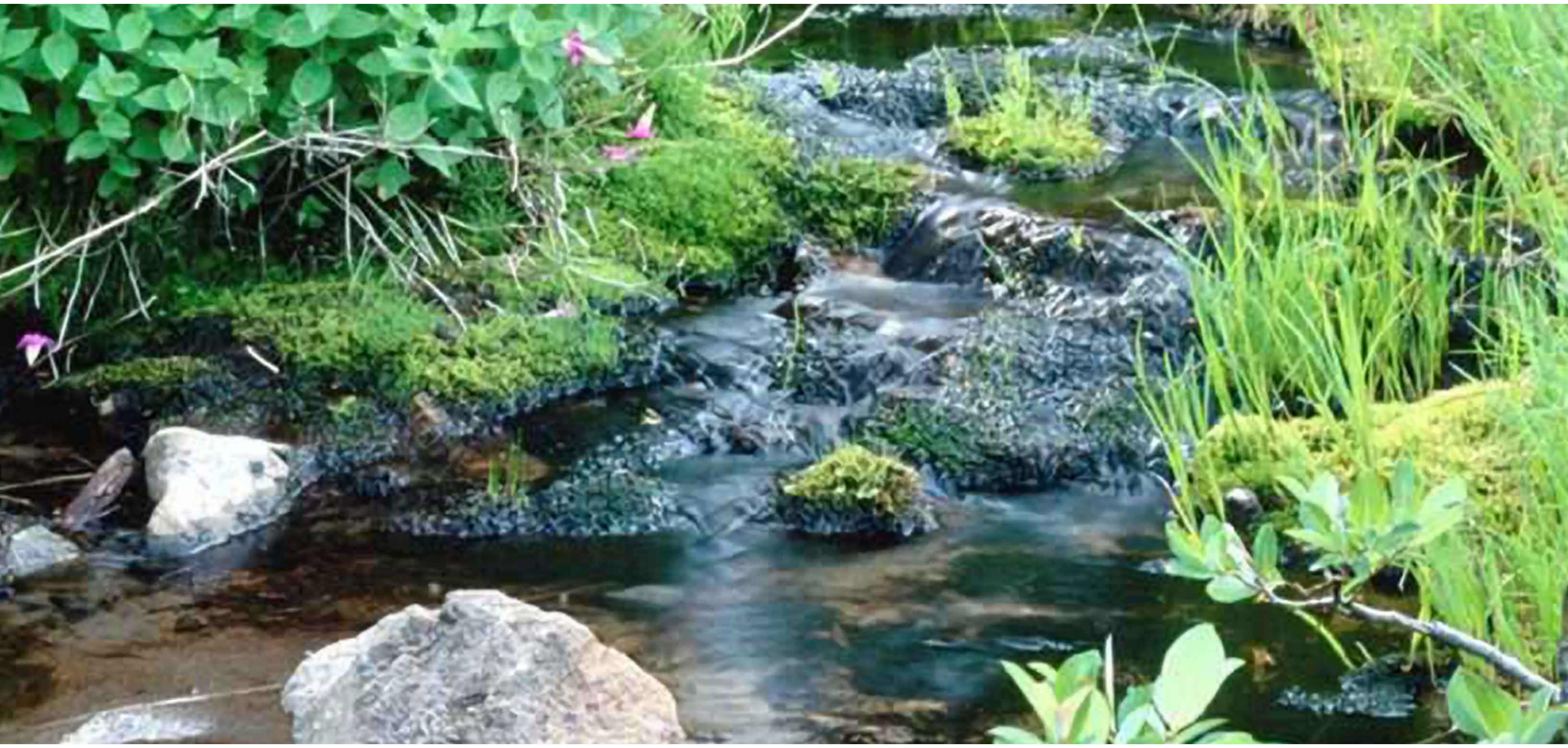

Floodplain Report

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Executive Summary

The Illinois Department of Transportation Division of Aeronautics (IDOT), as part of the South Suburban Airport (SSA) Master Plan, retained the services of AECOM Transportation and Hanson Professional Services Inc. (Hanson) to perform floodplain modeling for a group of streams within the Inaugural Airport boundary limits. The purpose of this report is to prepare existing condition floodplain mapping and to analyze the potential impacts of infrastructure associated with the airport on those floodplains. This study does not evaluate drainage or stormwater management features that will be required as part of the airport development.

As part of this study, existing conditions 100-year floodplain boundary maps within the proposed inaugural airport boundary were mapped. The streams within the Inaugural Airport boundary (Rock Creek, Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek) are currently categorized by the Federal Emergency Management Agency (FEMA) as having Unnumbered Zone “A” floodplains. This designation means that the flood boundaries for the 1 percent chance (100-year) flood event are approximate and that detailed flood studies to determine the Base Flood Elevations have not been performed.

This document summarizes the processes and procedures that were used to develop floodplain boundaries and flood profiles for five watersheds (Rock Creek, Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek) within the area of the Inaugural Airport boundary. The floodplain boundaries were developed following a “limited detailed study” approach. A limited detailed floodplain study provides a 100-year flood profile and floodplain boundary and map with less survey information collected than for a detailed study. A floodway is determined in the model but not shown on the map. The results contained in this report provide the level of detail needed for a Federal environmental review of floodplain impacts.

The floodplain boundaries mapped by this study compare well to the previously drawn approximate boundaries and represent an improvement in detail and confidence with this effort. Floodplain boundaries are now quantified in stage, flow and location based on current topography and conditions in the watershed. Differences between the boundaries are attributable to the care and standards followed to develop representative hydrologic and hydraulic models and the high level of detail associated with the Light Detection And Ranging (LiDAR) topographic data used for this study compared to the coarser data used in the original FEMA Will County flood zone mapping.

The newly delineated floodplains impact on the structures and facilities associated with the proposed Airport Layout Plan (ALP), dated September 20, 2012, was analyzed. A total of eight new hydraulic structures have been identified within the proposed inaugural airport facilities. Preliminary structure parameters were determined such that the new structures would create no net increase in water surface elevation beyond the project property boundaries. Additionally, any lost floodplain storage volume was calculated to determine approximate compensatory storage volumes that may be required.

The results of this analysis indicate that floodplain impacts associated with the proposed ALP facilities can be managed and mitigated within the current planned airport property boundaries.

This study does not include a formal submittal of the floodplain modeling and mapping to FEMA for incorporation into the National Flood Insurance Program (NFIP). The boundaries and flood profiles presented in this report were performed in accordance with accepted industry practices. Their development should reduce the future level of effort necessary to complete a detailed study, when the need arises to submit detailed modeling and mapping to FEMA for formal incorporation into the Flood Insurance Rate Maps (FIRM) for Will County.

Section 1 – Hydrology

The drainage areas and hydrologic basin parameters for each watershed were delineated with topographic data compiled in a Geographical Information System (GIS). The topographic data was compiled using LiDAR topographic data acquired from Will County which was collected in and distributed between 2007 and 2008. Basin parameters for the watersheds (with the exception of Rock Creek) were prepared using AECOM's Watershed Information System (WISE) software. WISE preprocesses geospatial data, such as topography and land use, to speed the development of model input data. The hydrologic and hydraulic modeling methods used are not specific to WISE and similar geospatial data processing can be done using add-ons to ESRI's ArcView[1] software and other software packages as was done for Rock Creek.

The hydrologic model used for this study was the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's-Hydrologic Modeling System (HEC-HMS) version 3.5.[2] HEC-HMS is designed to compute the precipitation-runoff processes of dendritic watershed systems.

1.1 Watershed Description

1.1.1 – Rock Creek

The Rock Creek study area is located in Will County, Illinois just north of Peotone and is bordered by Black Walnut Creek watershed to the east and South Branch Forked Creek to the west. Rock Creek converges with South Branch Rock Creek and eventually discharges to the Kankakee River northwest of Bourbonnais, Illinois. The portion of Rock Creek that was included in this study covers an area of 12.7 square miles from Section 22 of Township 34N, Range 13E to Section 13 of Township 33N, Range 12E of the 3rd Principal Meridian. For purposes of this study, the watershed was divided into 45 sub-basins. **Exhibit 1-1 – Rock Creek Study Area** is an illustration of the Rock Creek study area associated with this project and includes the limits of the study, the names of the streams and FEMA's approximate Zone "A" floodplain boundaries. **Exhibit 1-2 – Rock Creek Sub-Basin Layout** depicts the location and layout of the sub-basins developed for Rock Creek.

1.1.2 – Black Walnut Creek

The Black Walnut Creek study area is located in Will County, Illinois and is bordered by the South Branch Rock Creek watershed to the east and Rock Creek watershed to the west. Black Walnut Creek converges with South Branch Rock Creek and eventually discharges to the Kankakee River northwest of Bourbonnais, Illinois. The portion of Black Walnut Creek that was included in this study covers an area of 13.4 square miles from Section 18 of Township 34N, Range 14E to Section 20 of Township 33N, Range 13E of the 3rd Principal Meridian. For purposes of this study, the watershed was divided into 27 sub-basins. **Exhibit 1-3 - Black Walnut Creek Study Area** depicts the Black Walnut Creek study area associated with this project and includes the limits of the study, the names of the streams and FEMA's approximate Zone "A" floodplain boundaries. **Exhibit 1-4 – Black Walnut Creek Sub-Basin Layout** shows the location and layout of the sub-basins developed for Black Walnut Creek.

1.1.3 – South Branch Rock Creek

The South Branch Rock Creek study area is located in Will County, Illinois and is encompassed by the Exline Slough watershed to the east and Black Walnut Creek to the west. The portion of South Branch Rock Creek that was included in this study covers an area of 7.2 square miles from Section 1 of Township 33N, Range 13E to Section 28 of Township 33N, Range 13E of the 3rd Principal Meridian. For purposes of this study, the watershed was divided into 18 sub-basins. **Exhibit 1-5 – South Branch Rock Creek Study Area** illustrates the South Branch Rock Creek study area associated with this project and includes the limits of the study, the names of the streams and FEMA's approximate Zone "A" floodplain boundaries. **Exhibit 1-6 – South Branch**

[1] http://www.esri.com/
[2] http://www.hec.usace.army.mil/software/hec-hms/

Rock Creek Sub-Basin Layout depicts the location and layout of the sub-basins developed for South Branch Rock Creek.

1.1.4 – Exline Slough

The Exline Slough study area is located in Will County, Illinois and is bordered by the South Branch Rock Creek watershed to the west and Plum Creek to the east. Exline Slough flows to the south and discharge to the Kankakee River at Kankakee, Illinois. The portion of Exline Slough that was included in this study covers an area of 0.9 square miles from Section 12 of Township 33N, Range 13E to Section 13 of Township 33N, Range 13E of the 3rd Principal Meridian. For purposes of this study, the watershed was divided into 3 sub-basins. **Exhibit 1-7 – Exline Slough Study Area** depicts the Exline Slough study area associated with this project and includes the limits of the study, the names of the streams and FEMA's approximate Zone "A" floodplain boundaries. **Exhibit 1-8 – Exline Slough Sub-Basin Layout** illustrates the location and layout of the sub-basins developed for Exline Slough.

1.1.5 – Plum Creek

The Plum Creek study area is located in Will County, Illinois and is bordered by Village of Crete to the north, Village of Beecher to the east and South Branch Rock Creek to the west. Plum Creek flows to the north and eventually discharges to Hart Ditch at Dyer, Indiana. The portion of Plum Creek that was included in this study covers an area of 6.7 square miles from Section 33 of Township 34N, Range 14E to Section 18 of Township 33N, Range 14E of the 3rd Principal Meridian. For purposes of this study, the watershed was divided into 21 sub-basins. **Exhibit 1-9 – Plum Creek Study Area** shows the Plum Creek study area associated with this project and includes the limits of the study, the names of the streams and FEMA's approximate Zone "A" floodplain boundaries. **Exhibit 1-10 – Plum Creek Sub-Basin Layout** highlights the location and layout of the sub-basins developed for Plum Creek.

1.2 Precipitation Losses

The Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), runoff Curve Number method was used to define the precipitation runoff losses in the HEC-HMS model. Curve numbers for each sub-basin were estimated by overlaying digital mapping of land uses and soil conditions and calculating a weighted curve number based on the relative areas of all land use/soil group combinations.

1.2.1 – Rock Creek

Digital soils data provided by the NRCS Soil Survey Geographic database for Will County, Illinois was used to determine the hydrologic soil group classifications present in each sub-basin. Land cover and usage data for Rock Creek was based on the Land Cover of Illinois 1999-2000[3] inventory and associated database. The U.S. Department of Agriculture (USDA) National Agricultural Statistics Service, the Illinois Department of Agriculture and the Illinois Department of Natural Resources began an interagency initiative to produce statewide land cover information as part of the Illinois Interagency Landscape Classification Project.[4] This project was completed in the summer of 2002 which resulted in the Land Cover of Illinois 1999-2000 inventory. This dataset groups land cover into five main categories including agricultural land, forested land, urban land, wetland and other. The land uses were compared to 2008 aerial photos and field trip data for each basin. Based on this comparison, no adjustments to the land usage data were required as the land cover dataset appeared to be an appropriate estimate of the existing land use.

There were dual hydraulic soil classifications for the Rock Creek watershed. Dual classifications are assigned when the soil is either classified as well-drained (Class A, B or C) or poorly-drained (Class D). For Rock Creek, the dual hydrologic soil classifications were limited to Class C/D soils. Based on site topography, field inspections and land usage data, it was determined that dual class soils classified as C/D would use the curve number associated with well-drained or Class C soils. This was a slightly less conservative assumption of

[3] http://www.agr.state.il.us/gis/landcover99-00.html
[4] http://www.dnr.state.il.us/orep/pfc/landcover/

potential runoff. However, the areas designated as C/D soils were in areas under agricultural land use practices, many of which contain field tile drainage systems. Lands which are continually used for agricultural practices are often considered well-drained. Therefore, this assumption was considered appropriate for this site.

Appendix C-1 – Rock Creek Supporting Hydrologic Information contains a summary of the basin parameters and a summary of the final curve numbers used in this study based on hydrologic soil classification and land usage. **Appendix C-1 – Rock Creek Supporting Hydrologic Information** also contains two maps illustrating the locations of the hydrologic soil classifications and land usages used in this study. The final hydrologic parameters for the Rock Creek watershed used in this study are summarized in **Table 1-1 - Rock Creek Hydrologic Parameter Summary**.

Table 1-1 – Rock Creek Hydrologic Parameter Summary

Basin	Drainage Area (mi^2)	Longest Flow Path Length (mi)	Longest Flow Path Slope (ft/mi)	Curve Number	Time of Conc. – Tc (hr)	Storage Coefficient - R (hr)
A1-1	0.254	0.789	42.23	81	0.636	0.983
A1-2	0.440	1.495	38.34	79	1.132	1.320
A1-3	0.188	0.575	60.30	79	0.452	0.665
A-2	0.105	0.584	52.51	81	0.470	0.746
B1-1	0.253	1.161	48.23	76	0.870	1.010
B1-2	0.099	0.408	94.74	76	0.308	0.414
B1-3	0.156	0.812	42.71	81	0.650	0.983
B1-4	0.550	1.366	41.00	75	1.033	1.213
B1-5	0.268	0.775	36.12	75	0.644	1.105
B1-6	0.153	0.546	29.30	76	0.492	1.156
B-2	0.097	0.631	54.98	72	0.498	0.739
B-3	0.484	1.445	28.60	79	1.158	1.644
C1-1	0.063	0.621	49.35	77	0.502	0.801
C1-1-2	0.137	0.599	40.07	78	0.504	0.932
C1-2	0.137	0.747	82.07	78	0.538	0.571
C1-3	0.154	0.631	52.80	80	0.502	0.763
C1-4	0.094	0.511	91.32	72	0.378	0.461
C-2	0.115	0.569	21.09	81	0.542	1.521
D-1	0.102	0.969	59.18	78	0.716	0.807
E-1	0.202	1.375	40.72	74	1.041	1.223
E-2	0.399	1.163	27.52	73	0.965	1.574
E-3	0.399	0.550	55.81	81	0.440	0.697
E-4	0.113	1.410	29.31	72	1.129	1.599
M1-1	0.497	1.125	39.12	77	0.879	1.178
M1-2	0.469	0.835	15.97	76	0.796	2.159
M1-3	0.236	0.391	3.41	75	0.781	5.636
M1-4	0.030	1.434	29.75	73	1.143	1.590
M1-5	0.526	0.996	57.56	72	0.737	0.833
ME-1	0.374	0.802	56.49	79	0.612	0.785
ME-2	0.231	0.192	27.83	87	0.199	0.842
ME-3	0.105	0.657	77.10	76	0.486	0.574
ME-4	0.288	1.127	52.07	75	0.836	0.941
ME-5	0.626	0.591	63.22	78	0.459	0.647
NE-1	0.144	1.423	24.36	78	1.176	1.856
NE-2	0.668	0.842	28.52	79	0.722	1.370
NW-1	0.497	1.411	46.79	76	1.037	1.105
NW-2	0.373	1.488	19.72	80	1.271	2.228
R-1	0.643	0.399	13.35	82	0.431	1.933

Table 1-1 – Rock Creek Hydrologic Parameter Summary						
Basin	**Drainage Area (mi^2)**	**Longest Flow Path Length (mi)**	**Longest Flow Path Slope (ft/mi)**	**Curve Number**	**Time of Conc. – Tc (hr)**	**Storage Coefficient - R (hr)**
R-2	0.021	0.659	54.60	71	0.519	0.754
R-3	0.019	0.202	13.21	73	0.238	1.544
R-4	0.003	1.496	12.48	77	1.387	3.202
R-5	0.074	1.366	11.72	81	1.296	3.264
S-1	0.042	1.422	29.06	78	1.139	1.615
SE-1	0.631	0.866	33.87	80	0.718	1.207
W-1	0.408	0.647	82.40	72	0.474	0.542
W-2	0.053	1.829	41.55	79	1.331	1.327
W-3	0.998	0.637	54.40	77	0.504	0.748

1.2.2 – Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek

For these four watersheds, the land use mapping represents 2009 conditions. Land cover and usage data was based on a 2005 land use coverage provided by Will County. 2009 aerial photography was used to update the land use mapping to that year. The soils mapping is the standard digital USDA county soil survey mapping, with each soil unit assigned to one of the hydrologic soil groups used in curve number calculation.

Each land use/soil combination was assigned a curve number, based on the standard reference tables of SCS-recommended curve numbers for typical land uses and soil groups. A weighted curve number for each sub-basin was then calculated. **Appendix C-2 - Black Walnut Creek / South Branch Rock Creek / Exline Slough / Plum Creek Supporting Hydrologic Information** contains a summary of the basin parameters used for this study and includes a summary of the final curve numbers used for each watershed based on hydrologic soil classification and land usage. **Appendix C-2 - Black Walnut Creek / South Branch Rock Creek / Exline Slough / Plum Creek Supporting Hydrologic Information** also contains four sets of maps illustrating the locations of the hydrologic soil classifications and land usages for each watershed. The final hydrologic parameters for the four watersheds used in this study are summarized in **Table 1-2 - Black Walnut Creek Hydrologic Parameter Summary, Table 1-3 - South Branch Rock Creek Hydrologic Parameter Summary, Table 1-4 - Exline Slough Hydrologic Parameter Summary** and **Table 1-5 - Plum Creek Hydrologic Parameter Summary.**

Table 1-2 – Black Walnut Creek Hydrologic Parameter Summary						
Basin	**Drainage Area (mi^2)**	**Longest Flow Path Length (mi)**	**Longest Flow Path Slope (ft/mi)**	**Curve Number**	**Time of Conc. – Tc (hr)**	**Storage Coefficient - R (hr)**
1	0.112	0.580	55.97	79.45	0.493	0.925
2	0.057	0.572	38.02	80.77	0.663	1.243
3	0.273	0.606	79.73	79.63	0.381	0.714
4	0.081	0.549	45.94	79.60	0.563	1.056
5	0.230	1.214	44.88	78.60	0.781	1.465
6	0.056	0.504	51.22	78.88	0.500	0.938
7	0.139	0.729	36.43	78.83	0.754	1.413
8	0.492	1.481	36.43	80.15	0.993	1.863
10	0.365	1.255	50.16	78.94	0.726	1.361
11	2.046	2.284	26.40	77.80	1.512	2.836
12	0.391	0.956	53.86	77.79	0.617	1.158
13	0.933	1.873	26.40	79.13	1.400	2.624
14	0.839	1.522	34.85	78.78	1.039	1.949
15	0.213	1.070	58.08	78.92	0.608	1.140
16	0.163	0.934	69.17	79.67	0.503	0.944
17	0.302	1.073	64.94	78.01	0.558	1.046
18	0.648	1.531	42.77	77.63	0.888	1.665

Table 1-2 – Black Walnut Creek Hydrologic Parameter Summary

Basin	Drainage Area (mi^2)	Longest Flow Path Length (mi)	Longest Flow Path Slope (ft/mi)	Curve Number	Time of Conc. – Tc (hr)	Storage Coefficient - R (hr)
19	0.649	1.426	31.15	76.45	1.106	2.074
20	0.459	1.431	45.94	76.44	0.818	1.534
21	0.836	1.985	28.51	78.08	1.348	2.528
22	0.366	1.152	61.25	78.83	0.601	1.126
23	0.051	0.594	35.90	75.28	0.704	1.320
24	0.828	1.995	29.04	78.51	1.332	2.497
25	0.815	1.666	32.74	78.64	1.131	2.120
28	0.490	1.792	32.21	78.24	1.178	2.209
29	0.762	2.175	26.40	77.69	1.484	2.782
30	0.976	2.190	26.40	77.73	1.488	2.790

Table 1-3 – South Branch Rock Creek Hydrologic Parameter Summary

Basin	Drainage Area (mi^2)	Longest Flow Path Length (mi)	Longest Flow Path Slope (ft/mi)	Curve Number	Time of Conc. – Tc (hr)	Storage Coefficient - R (hr)
1	0.272	0.996	41.13	78.77	0.774	1.452
2	0.504	1.651	30.29	77.21	1.197	2.244
3	0.500	0.757	26.70	79.84	0.974	1.827
4	0.117	1.162	26.40	76.49	1.162	2.179
5	0.291	1.694	26.40	79.20	1.346	2.523
6	0.201	1.545	26.40	79.78	1.299	2.435
7	0.593	1.139	26.40	78.06	1.153	2.162
8	0.328	1.334	26.40	78.02	1.226	2.299
9	0.492	1.767	36.65	77.80	1.059	1.986
10	0.300	1.299	26.40	77.98	1.214	2.276
11	0.700	1.446	26.40	79.41	1.265	2.372
12	0.272	0.845	26.40	76.58	1.026	1.924
13	0.367	1.242	26.40	79.76	1.192	2.236
14	0.233	1.044	26.40	79.78	1.115	2.090
15	0.484	1.554	27.85	79.61	1.248	2.341
16	0.314	1.331	26.40	79.35	1.225	2.297
17	0.651	1.694	26.40	79.72	1.346	2.524
18	0.492	1.102	26.40	79.01	1.138	2.134

Table 1-4 – Exline Slough Hydrologic Parameter Summary

Basin	Drainage Area (mi^2)	Longest Flow Path Length (mi)	Longest Flow Path Slope (ft/mi)	Curve Number	Time of Conc. – Tc (hr)	Storage Coefficient - R (hr)
1	0.193	1.195	44.64	84.66	0.780	1.170
1-2	0.494	1.370	38.93	84.40	0.915	1.373
1-3	0.230	1.080	27.09	83.36	1.108	1.662

Table 1-5 – Plum Creek Hydrologic Parameter Summary

Basin	Drainage Area (mi^2)	Longest Flow Path Length (mi)	Longest Flow Path Slope (ft/mi)	Curve Number	Time of Conc. – Tc (hr)	Storage Coefficient - R (hr)
1	0.075	0.510	62.89	79	0.428	0.835
2	0.079	0.548	50.91	79	0.519	1.031

Table 1-5 – Plum Creek Hydrologic Parameter Summary						
Basin	Drainage Area (mi^2)	Longest Flow Path Length (mi)	Longest Flow Path Slope (ft/mi)	Curve Number	Time of Conc. – Tc (hr)	Storage Coefficient - R (hr)
3	0.107	0.716	58.61	77	0.516	1.007
4	0.025	0.250	96.10	62	0.233	0.454
5	0.089	0.376	58.08	70	0.405	0.789
6	0.236	0.945	39.10	79	0.789	1.539
7	0.222	0.926	23.15	76	1.178	2.298
8	0.120	0.826	17.64	69	1.393	2.716
9	0.072	0.503	29.77	80	0.763	1.489
10	0.166	0.906	28.56	73	0.992	1.934
11	1.154	3.213	22.31	77	1.970	3.842
12	0.116	0.665	24.30	79	0.997	1.943
13	0.588	1.951	36.21	75	1.112	2.168
14	0.214	1.072	32.63	79	0.955	1.861
15	0.346	1.489	10.32	79	2.663	5.192
16	0.442	1.328	24.41	80	1.301	2.537
17	0.302	1.275	10.13	80	2.544	4.960
18	0.058	0.530	26.02	75	0.865	1.687
19	0.164	0.889	31.45	77	0.913	1.781
20	1.324	2.745	21.00	78	1.942	3.787
21	0.799	2.385	18.10	78	2.064	4.025

1.3 Unit Hydrograph Methodology

The Clark unit hydrograph methodology for small watersheds in Illinois was used to characterize the watershed runoff potential. The unit hydrograph parameters for time of concentration (T_c) and Storage Coefficient (R) were computed using the methods published in two U.S. Geological Survey's (USGS) Water-Resources Investigations Reports: 82-22, "A Technique for Estimating Time of Concentration and Storage Coefficient Values for Illinois Streams",[5] and 00-4184 "Equations for Estimating Clark Unit-Hydrograph Parameters for Small Rural Watersheds in Illinois".[6] Clark storage coefficients and time of concentrations were calculated for each basin utilizing topographic data to determine the longest flow path. **Exhibit 1-2 - Rock Creek Sub-Basin Layout, Exhibit 1-4- Black Walnut Creek Sub-Basin Layout, Exhibit 1-6- South Branch Rock Creek Sub-Basin Layout, Exhibit 1-8 - Exline Slough Sub-Basin Layout** and **Exhibit 1-10 - Plum Creek Sub-Basin Layout** contain maps of the boundaries and the associated longest flow paths for the time of concentration estimates for sub-basin in each of the five watersheds. Hydrologic parameters including estimated time of concentration and storage coefficients are summarized in **Table 1-1 - Rock Creek Hydrologic Parameter Summary, Table 1-2 - Black Walnut Creek Hydrologic Parameter Summary, Table 1-3 - South Branch Rock Creek Hydrologic Parameter Summary, Table 1-4 - Exline Slough Hydrologic Parameter Summary** and **Table 1-5 - Plum Creek Hydrologic Parameter Summary**. Detailed time of concentration and storage coefficients calculations are included in **Appendix C-1 – Rock Creek Supporting Hydrologic Information** for Rock Creek and **Appendix C-2 - Black Walnut Creek / South Branch Rock Creek / Exline Slough / Plum Creek Supporting Hydrologic Information** for the remaining streams.

1.4 Channel Routing

Once the sub-basin runoff hydrograph enters the channel network, it is routed downstream using a defined channel routing algorithm which serves to account for travel time and channel storage in the hydrology model. For Rock Creek, the Muskingum-Cunge Routing Method[7] was utilized to represent the characteristics of these

[5] http://il.water.usgs.gov/pubs/wri82_22.pdf
[6] http://smig.usgs.gov/SMIG/features_0301/clark.pdf
[7] http://www.hec.usace.army.mil/publications/TechnicalPapers/TP-135.pdf

channels. There were a total of 30 reaches defined across the 12.7-square miles of sub-basins modeled. The Muskingum-Cunge routing method requires the input of cross section data, the slope of the reach and the Manning's N-values for the channel and the overbanks. The cross sections are limited to eight points and were derived using available site topographic data. The slopes of the reaches were also determined by using available site topographic data. The Manning's N-values were chosen based on aerial photography, site visits and Cowan's Method.[8] **Appendix C-1 – Rock Creek Supporting Hydrologic Information** includes documentation of the channel routings used for this stream.

For the rest of the watersheds in the study area (Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek), two of the routing options available in HEC-HMS were initially compared: Muskingum-Cunge and Modified Puls. The Muskingum-Cunge routing method resulted in very little or no attenuation of peak flows, which appeared unrealistic, given the watershed's flat topography and numerous wetland areas. Hence, the Modified Puls method was ultimately selected for channel routing. The Modified Puls method uses conservation of mass and a relationship between storage and discharge to route flow through the stream reach. The storage-discharge curve for each reach was automatically processed within the WISE software. WISE first processes the routing cross sections using digital terrain mapping from LiDAR and uses Manning's equation to calculate a range of discharges based on a range of water depths in the routing cross section. Each water surface elevation is then used to calculate the equivalent storage within that particular reach, which then is used to develop an outflow/storage rating curve for that reach in the model. **Appendix C-2 - Black Walnut Creek / South Branch Rock Creek / Exline Slough / Plum Creek Supporting Hydrologic Information** includes storage-discharge curve tables of the channel routings used for the four watersheds.

1.5 Monee Reservoir in the Rock Creek Watershed

Monee Reservoir is a recreational lake which is owned and operated by the Forest Preserve District of Will County. The reservoir is located west of Illinois Route 50 (IL-50) and south of Pauling Road approximately two miles south of Monee, Illinois. The reservoir is comprised of the primary lake, which is approximately 44 acres and two additional sedimentation ponds located on the north end of the main reservoir. **Exhibit 1-11 – Monee Reservoir and Pond Layout** illustrates the main lake and the sedimentation pond locations.

The reservoir's primary outlet structure is a Morning Glory structure with a trash rack which drains to the east through the reservoir's dam embankment where it joins with Rock Creek before discharging under the IL-50 bridge. The reservoir's dam embankment is also the embankment for the rail line which parallels IL-50. There are also two overflow locations associated with Monee Reservoir. One of the overflow points is located along the west ditch adjacent to the rail embankment. The other overflow drains in a southwesterly direction. Both overflows eventually discharge to Rock Creek. The map shown in **Exhibit 1-11 – Monee Reservoir and Pond Layout** illustrates overflow locations and their proximity to the main reservoir. The sub-basin layout map in **Exhibit 1-2 – Rock Creek Sub Basin Layout** shows Monee Reservoir and its three ponds located in Sub-Basins ME-1, ME-2 and ME-3.

1.5.1 – Monee Reservoir Inflows

A concrete weir structure located northeast of the main reservoir controls the inflows which enter Monee Reservoir through the sedimentations ponds. The weir structure is located upstream of Sediment Basin B as shown in **Exhibit 1-11 – Monee Reservoir and Pond Layout**. The weir structure is constructed so that a set of stop logs can be used to manually control the inflows. Water flowing from the northeast can either go into the sediment pond through a set of twin 2-foot by 6-foot concrete box culverts or flow over the weir structure, bypassing the sediment ponds and flowing directly into Monee Reservoir.

The upstream invert for the twin culverts is 747.3 feet. The notch in the weir is 5.2 feet wide and 1.3 feet deep and has a base elevation of 748.0 feet. If the stop logs are installed, the weir length changes to 13.3 feet

[8] http://www.fhwa.dot.gov/bridge/wsp2339.pdf

with an elevation of 749.3 feet. **Appendix D-2 – Morning Glory Outlet Pictures at Monee Reservoir** contains pictures of the culverts and diversion weir.

The weir and culverts structures were modeled as an inflow-diversion in the HEC-HMS model. The external calculations used to model this diversion are included in **Appendix D-1 – Inflow Diversion Calculations for Monee Reservoir**. If the flow enters Sediment Basin B through the culverts, it will also flow through Sediment Basin A and the area called Upstream North Monee, as shown in **Exhibit 1-11 – Monee Reservoir and Pond Layout**, before reaching the main reservoir. Each sediment pond was modeled as separate storage areas with independent stage-volume-discharge curves utilizing project topographic data.

1.5.2 – Primary Spillway Structure for Monee Reservoir

Monee Reservoir has a normal pool elevation of 743.8 feet which is controlled by the Morning Glory outlet structure with an inverted-bell inlet and a trash rack as its primary outlet structure. This outlet structure is depicted in the plan and profile drawing in **Exhibit 1-12 – Monee Reservoir Primary Spillway Structure from 1989 As-Built Plans**. The Morning Glory structure drops vertically down, curves and drains east through the reservoir's dam embankment under the rail line. The outlet connects to a 42-inch steel pipe which drains through an elongated headwall and dissipation structure between the rail embankment and IL-50 at an elevation of 725.4 feet. **Appendix D-2 – Morning Glory Outlet Pictures at Monee Reservoir** contains a set of pictures showing the Morning Glory and headwall outlet structures.

An elevation-discharge rating curve was developed to model the discharge through the trash rack and Morning Glory structure. Hanson developed this rating curve based on guidance from the Bureau of Reclamation's publication titled, *Design of Small Dams*.[9] It was assumed that the trash rack was 40 percent open (60 percent clogged). This assumption was based on documentation in the as-built plans which are included in **Appendix D-4 – As-Built/Record Drawings for Monee Reservoir**. Hanson's rating also varied the weir coefficient for the Morning Glory structure with depth as recommended in the *Design of Small Dams*. This adjustment was not included in the as-built plan rating. As a result, the rating developed for this study was utilized in the HEC-HMS model of the reservoir.

Rock Creek tailwater elevations at the outlet for Monee Reservoir also have the ability to affect the discharges from the reservoir. Tailwater refers to waters located immediately downstream from a hydraulic structure, such as a dam (excluding minimum release such as for fish water), bridge or culvert. The outlet itself discharges just west of the IL-50 bridge after joining the main stem of Rock Creek. A wide range of tailwater elevations were examined to determine the sensitivity of this variable. For this study, it was assumed that the tailwater would be set to elevation 728.9 feet which is the elevation of the crown of the 42-inch diameter discharge pipe, i.e., the full flow discharge condition. This assumption was chosen in part because it is a conservative (higher flow) estimate of the discharge out of Monee Reservoir which is an appropriate assumption for the floodplain mapping purposes of this study. **Appendix D-1 – Inflow Diversion Calculations for Monee Reservoir** contains the calculations documenting the outlet rating curve developed for Monee Reservoir. The HEC-HMS stage-frequency results compared well with as-built plans from the reservoir which can be seen in **Exhibit 1-13 – Stage Frequency Curve for Monee Reservoir From 1989 As-Built Plans**.

1.5.3 – Overland Flow Outlets

The two aforementioned overland flow outlets from the reservoir were both modeled as dam overtopping points in HEC-HMS utilizing topographic data for the site. One of the overflow points is located along the west ditch adjacent to the rail embankment and has an overflow elevation of 746.8 feet. The other overflow drains in a southwesterly direction and has an overflow elevation of 748.8 feet. Both overflows eventually discharge to the main stem of Rock Creek. The critical duration for Monee Reservoir is the 24-hour event. For the reservoir, the critical duration is defined as the duration which results in the highest stage. **Exhibit 1-13 – Stage Frequency Curve for Monee Reservoir from 1989 As-Built Plans** contains a stage-frequency curve from

[9] http://www.usbr.gov/pmts/hydraulics_lab/pubs/manuals/SmallDams.pdf

the Monee Reservoir as-built plans which compares well with the estimated stages from HEC-HMS. The results of the modeling effort for Monee Reservoir can be found in the stage-discharge summary table located in **Table 1-6 – Monee Reservoir Stage Discharge Summary** for the modeled storm frequencies.

Table 1-6 – Monee Reservoir Stage Discharge Summary

Storm Event	Reservoir Stage (feet)	Reservoir Outflow (cfs)
5-yr – 24-hr	746.1	149.2
10-yr – 24-hr	746.9	174.4
25-yr – 24-hr	748.0	180.0
100-yr – 24-hr	749.6	187.1

1.6 – Design Frequency Events

A "design storm" rainfall simulation approach was used for the hydrologic modeling. The rainfall depths for the modeled combinations of recurrence interval and rainfall duration are the Will County, Illinois regulatory rainfall depths, which are to be used in all floodplain modeling projects being submitted to FEMA in the county *(Will County Technical Guidance Manual, Table 3: ISWS Bulletin 70 Rainfall Depths Averaged for Will County)*.[10] This report uses the county's regulatory rainfall depths to be consistent with similar hydrologic/hydraulic modeling projects being performed in the area.

Rainfall distributions (hyetographs) from the *Rainfall Frequency Atlas of the Midwest* (Bulletin 71, Illinois State Water Survey)[11] were used. These are also known as the "Huff" or "Huff/Angel" distributions. They are the most widely accepted rainfall distributions used in Illinois for hydrologic modeling. Several different rainfall distributions are recommended by the rainfall frequency atlas, depending upon the duration of rainfall to be modeled. A critical duration analysis was performed to determine the appropriate length of rainfall to be used as the design storm and the rainfall distributions were varied according to the distribution to be used for the particular rainfall length being simulated.

The critical storm duration analysis was performed for the 100-year, or 1 percent annual chance, storm event. The 1-, 3-, 6-, 12-, 24- 48-, 72-, 120- and 240-hour storm durations were simulated. HEC-HMS model results for all these durations were compared. The critical duration event, which generated the highest discharge for the Rock Creek watershed, was the 12-hour storm event and the 3-hour storm and was determined to be the critical duration for the other four watersheds, as it produced the highest peak flow at most of the hydrological elements. **Table 1-7 – Rock Creek Critical Duration Summary, Table 1-8 – Black Walnut Creek Critical Duration Summary, Table 1-9 – South Branch Rock Creek Critical Duration Summary, Table 1-10 - Exline Slough Critical Duration Summary and Table 1-11 - Plum Creek Critical Duration Summary** are summaries of the 100-year storm critical duration analysis for all watersheds.

Table 1-7 – Rock Creek Critical Duration Summary

Furthest Downstream Point of Rock Creek (HEC-HMS Model Output for Junc 10)	
Storm	**Discharge (cfs)**
100-yr – 01-hr	2,415.6
100-yr – 03-hr	3,174.2
100-yr – 06-hr	3,150.1
100-yr – 12-hr	**3,322.7**
100-yr – 24-hr	3,142.7
100-yr – 48-hr	2,404.2

[10] http://willcountylanduse.com/sites/default/files/documents/Approved_TGM_August_25_2010.pdf
[11] http://www.isws.illinois.edu/pubdoc/b/iswsb-71.pdf

Table 1-8 – Black Walnut Creek Critical Duration Summary	
Furthest Downstream Point of Black Walnut Creek (HEC-HMS Model Output for 30C)	
Storm	**Discharge (cfs)**
100-yr – 1-hr	1,549.0
100-yr – 3-hr	**2,191.8**
100-yr – 6-hr	2,349.2
100-yr – 12-hr	2,907.0
100-yr – 24-hr	2,973.4
100-yr – 48-hr	2,531.3
100-yr – 72-hr	2,101.7
100-yr – 120-hr	1,423.4
100-yr – 240-hr	837.9

Table 1-9 – South Branch Rock Creek Critical Duration Summary	
Furthest Downstream Point of South Branch Rock Creek (HEC-HMS Model Output for 18C)	
Storm	**Discharge (cfs)**
100-yr – 1-hr	891.4
100-yr – 3-hr	**1,393.9**
100-yr – 6-hr	1,736.7
100-yr – 12-hr	2,056.4
100-yr – 24-hr	1,938.0
100-yr – 48-hr	1,475.6
100-yr – 72-hr	1,183.1
100-yr – 120-hr	775.5
100-yr – 240-hr	450.5

Table 1-10 – Exline Slough Critical Duration Summary	
Furthest Downstream Point of Exline Slough (HEC-HMS Model Output for Junc-3)	
Storm	**Discharge (cfs)**
100-yr – 1-hr	483.2
100-yr – 3-hr	**543.8**
100-yr – 6-hr	507.8
100-yr – 12- hr	479.2
100-yr – 24-hr	391.9

Table 1-11 – Plum Creek Critical Duration Summary	
Furthest Downstream Point of Plum Creek (HEC-HMS Model Output for 21C)	
Storm	**Discharge (cfs)**
100-yr – 1-hr	554.1
100-yr – 3-hr	**849.6**
100-yr – 6-hr	971.5
100-yr – 12-hr	1,065.0
100-yr – 24-hr	1,070.2
100-yr – 48-hr	984.9
100-yr – 72-hr	891.7
100-yr – 120-hr	659.4
100-yr – 240-hr	429.2

1.7 – Model Refinements

The most common methodology for estimating peak discharges for ungauged rural watersheds in Illinois is flood flow regional regression equations. In Illinois, the regression equations are programmed into the web-based StreamStats USGS interface.[12] A StreamStats analysis was made for a location corresponding to the downstream outlet of each watershed and compared to the model results of the uncalibrated HEC-HMS model. Since the initial HEC-HMS peak flow results seemed unreasonably high (for detailed documentation of model refinements, see the modeling logs in **Appendix G**), a number of refinements were made because the flows were generally higher than anticipated based on regional regression equation data. The refinements included an assessment of the basin parameters and available floodplain storage.

1.7.1 – Clark Storage Coefficients

Historical stream flow data was available for Plum Creek at a National Weather Service gage near Crete, Illinois at Burville Road. The stream gage is located just east of IL Route 394 approximately 12 miles northeast of Monee Reservoir. The Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) recently performed a watershed study of the Little Calumet River, which Plum Creek is ultimately a tributary to. Preliminary comparisons to the historical gage at Crete have resulted in a recommendation to increase the Clark unit hydrograph storage coefficient by 25 percent above the standard calculated values. This adjustment would increase the assumed floodplain storage and decrease estimated basin discharges. Based on this recommendation, the storage coefficient for all watersheds except Plum Creek was similarly increased. The storage coefficients for Plum Creek were increased to 30 percent or 50 percent to lower peak flow values comparable to the StreamStats calculation.

1.7.2 – Curve Numbers

The dominant land use/soil complex in the Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek study areas is row crops with hydrologic soil group C. These lands were initially assigned a curve number of 85, because the row crops were assumed to have no cropping management (contoured, terraced, crop residue, etc.). But the MWRDGC Little Calumet River study found that a calibrated curve number of 79 was appropriate for these lands. Hence, the curve number of 79 was assigned to row cropped areas within the four watersheds.

1.7.3 – Number of Subreaches

Given the flat topography of the watershed, numerous wetland and low overbank areas and storage behind culverts and road crossings that occurs in the study area, more flow attenuation was expected than what resulted in the initial model simulations. It was then found that the peak flows were sensitive to one of the HEC-HMS parameters for the Modified Puls channel routing – the number of subreaches that models reach is divided into for numerical routing calculations. For the four watersheds using Modified Puls for channel routing, initially the number of subreaches had been calculated using an automated procedure in the WISE software. Further information on this Modified Puls routing model parameter indicated a different calculation method would be appropriate for the SSA watersheds. The subreach values were then recalculated, which resulted in lower number of subreaches for the numerical calculations, which in turn lowered peak flow values.

1.7.4 – Modeling of Floodplain Storage in Rock Creek Watershed

Floodplain storage and storage as a result of restrictive culverts were modeled using results from the USACE's Hydrologic Engineering Centers-River Analysis System (HEC-RAS) modeling. Using a range of flows as key locations in HEC-RAS, stage-volume relationships were developed and added to the HEC-HMS model as storage ponds. **Exhibit 1-14 – Rock Creek Storage Area Locations** shows the locations of the storage areas which were added to the HEC-HMS model to simulate floodplain storage. This process reduced the flows downstream of the modeled locations and helped bring the HEC-HMS flow estimates closer to the estimates

[12] http://streamstats.usgs.gov/illinois.html

from the USGS's StreamStats program. More detailed discussion of how the HEC-HMS flow estimates compared to StreamStats can be found in the calibration section of this report are contained in Section 3.

A storage pond was also incorporated into the HEC-HMS model at the headwaters of Tributary B. It is called the Kuersten Road Pond and is modeled based on site pictures from a landowner and available site topography. The location of the pond is shown in **Exhibit 1-14 – Rock Creek Storage Area Locations**.

According to field survey data, a 36-inch reinforced concrete pipe labeled as R9 in **Exhibit 2-6 – Rock Creek Culvert & Bridge Locations** was the only structure that conveys flows under the railroad embankment on Tributary C. Initial modeling efforts showed that flows would overtop the rail embankment at this location which did not appear to be a valid estimate of flood conditions. A field visit performed in September of 2011 concluded that no other structures were present along the embankment to help convey flood flows under the rail embankment. The field visit did reveal that the area between the railroad embankment and IL-50 contained a fair amount of available surface storage. It was also noted by IDOT staff at the SSA field office that this area would pond during relatively small rain events. As a result, this area was modeled as a storage pond in HEC-HMS to limit flows to the culvert. Accordingly, a stage-discharge rating for the channel between the railroad tracks and IL-50 was developed using data HEC-RAS. Site topography also revealed that if flows were elevated, they would overtop a ditch summit south of the R9 culvert and rejoin Rock Creek further downstream. The addition of this storage area lowered peak flow estimates from 345 cfs to 114 cfs for the 100-year event. Using this approach, a more accurate stream discharge estimate was made which ultimately showed that the railroad tracks would not be overtopped at this location. **Table 1-12 - Summary of Discharges from HEC-HMS Model of Rock Creek** is a summary of the final peak 12 hour discharges from the HEC-HMS model for Rock Creek.

Table 1-12 – Summary of Discharges from HEC-HMS Model of Rock Creek

Hourly Discharge	Outlet at Monee Flow (cfs)	Main US Flow (cfs)	Main DS Flow (cfs)	Trib A Flow (cfs)	Trib B Flow (cfs)	Trib C Flow (cfs)	Trib D Flow (cfs)
5-yr – 12-hr	141.1	755.0	1,230.7	158.2	288.1	99.1	39.9
10-yr – 12-hr	167.6	963.5	1,596.1	218.3	408.7	106.9	55.7
25-yr – 12-hr	178.5	1,244.6	2,035.3	293.4	565.2	118.0	75.6
100-yr – 12-hr	186.9	2,057.9	3,080.1	475.4	946.7	139.4	124.1
Component in HMS	Monee Reservoir	Junc. 38	Junc. 20	Junc. 32	Junc. 31	Junc. 28	Sub-Basin D1

Note: Flows taken at stream outlet for the designated location.

1.7.5 – Modeling of Floodplain Storage in Plum Creek Watershed

A floodplain storage approach similar to what was described in the prior section at a railway embankment in Rock Creek was taken at a railroad culvert in Plum Creek. The culvert under the Union Pacific Railroad crossing is undersized at the downstream end of the watershed south of Offner Road. During a 100-year event, the high embankment under the crossing blocks the Plum Creek drainage and creates backwater upstream up to 1,400 feet north of Church Road (total about 2.6 miles). It appeared that the Modified Puls channel routing used in the previous hydrological analysis is not appropriate to simulate the floodplain storage at certain locations of major storage along the channel. Reservoir routing for several reaches along the main stem was used with stage storage curves generated from the HEC-RAS model.

Table 1-13 – Summary of Discharges from HEC-HMS Model of Black Walnut Creek, Table 1-14 – Summary of Discharges from HEC-HMS Model of South Branch of Rock Creek, Table 1-15 – Summary of Discharges from HEC-HMS Model of Exline Slough and Table 1-16 – Summary of Discharges from HEC-HMS Model of Plum Creek summarize the final peak 3-hour discharges from the HEC-HMS model for Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek, respectively.

Table 1-13 – Summary of Discharges from HEC-HMS Model of Black Walnut Creek

Hourly Discharge	Outlet at Monee Flow (cfs)	Trib 1 Flow (cfs)	Trib 1-1 Flow (cfs)	Trib 2 Flow (cfs)	Trib 3 Flow (cfs)
5-yr – 3-hr	807.5	75.8	19.3	132.2	46.2
10-yr – 3-hr	1,029.1	108.7	27.6	189.3	67.4
25-yr – 3-hr	1,393.7	152.3	38.7	264.6	95.5
100-yr – 3-hr	2,191.8	260.9	66.3	452.7	166.2

Note: Flows taken at stream outlet for the designated location.

Table 1-14 – Summary of Discharges from HEC-HMS Model of South Branch of Rock Creek

Hourly Discharge	Main Flow (cfs)	Trib 1 Flow (cfs)	Trib 2 Flow (cfs)
5-yr – 3-hr	423.2	141.1	88.1
10-yr – 3-hr	604.3	189.8	125.6
25-yr – 3-hr	835.0	256.4	175.3
100-yr – 3-hr	1,393.9	441.8	299.5

Note: Flows taken at stream outlet for the designated location.

Table 1-15 – Summary of Discharges from HEC-HMS Model of Exline Slough

Hourly Discharge	Main Flow (cfs)
5-yr – 3-hr	177.6
10-yr – 3-hr	245.0
25-yr – 3-hr	332.2
100-yr – 3-hr	543.8

Note: Flows taken at stream outlet for the designated location.

Table 1-16 – Summary of Discharges from HEC-HMS Model of Plum Creek

Hourly Discharge	Main Flow (cfs)	Trib 2 Flow (cfs)	Trib 3 Flow (cfs)	Trib 4 Flow (cfs)	Trib 5 Flow (cfs)	Trib 6 Flow (cfs)
5-yr – 3-hr	275.6	103.6	109.1	27.4	60.0	11.9
10-yr – 3-hr	379.4	147.9	157.6	40.4	87.3	17.3
25-yr – 3-hr	518.4	205.2	221.9	58.0	122.6	24.5
100-yr – 3-hr	849.6	339.3	381.9	103.6	207.2	46.6

Note: Flows taken at stream outlet for the designated location.

Section 2 – Hydraulics

Once hydrologic modeling has produced an estimate of rainfall runoff for a particular area, hydraulic modeling is performed to convert those discharge estimates into water surface elevations. Peak water surface elevations for the 100-year design flood event and other events were calculated using HEC-RAS version 4.1.0.[13]

2.1 – Datum Correlation for the Rock Creek Watershed

All of the HEC-RAS models developed for this study reference the North American Vertical Datum of 1988 (NAVD88). Existing bridge, culvert and as-built plans used in this study reference the National Geodetic Vertical Datum of 1929 (NGVD29). The datum adjustment from NGVD29 to NAVD88 is -0.47 feet. Further documentation of this adjustment is provided in **Appendix F – Field Survey Data & Notes**. Survey data collected as part of this study utilized NAVD88 and was used to set the elevations of existing structures in the models. All elevations in this report are given in NAVD88 unless otherwise noted.

2.2 – Modeled Stream Reaches

2.2.1 – Rock Creek

Hydraulic modeling for Rock Creek involved the modeling of nine miles of stream that flows in a southwesterly direction and included 179 cross sections, four bridges and nine culverts. Rock Creek was divided into separate reaches called Rock Creek Upstream and Rock Creek Downstream. For this study, the four unnamed tributaries to Rock Creek were named Tributary A, B, C and D. Tributaries A, B, C and D flow into Rock Creek at different locations as illustrated in **Exhibit 2-1 – Rock Creek Cross Section Locations**.

Rock Creek Upstream. Rock Creek Upstream (Main US on **Exhibit 2-1 – Rock Creek Cross Section Locations**) is just over a mile in length and begins at the outlet of Monee Reservoir and extends downstream through the bridge at Offner Road before ending just northeast of the IL-50 bridge. It is comprised of 35 cross sections ranging from stations 15588 to 9967 with one bridge and one culvert.

Rock Creek Downstream. Rock Creek Downstream (Main DS on **Exhibit 2-1 – Rock Creek Cross Section Locations**) is comprised of 18 cross sections ranging from station 9795 to 35 and is a continuation of Rock Creek Upstream. This reach is almost two miles in length and has two bridges and one culvert along it.

Tributary A. Tributary A is approximately 1.5 miles in length and is comprised of one culvert and 32 cross sections starting at station 8294 and ending at station 80. Tributary A has a smaller tributary (labeled Tributary A2 in the model) draining into it just below cross section 5858.

Tributary B. Tributary B is the longest reach in the study with a length of 2.5 miles. Tributary B begins on the west side of South Kuersten Road and runs west before turning south and running east along Offner Road. There are 46 cross sections along this reach ranging from cross sections 119 to 12176. Tributary B is broken into two reaches at the junction where Tributary B2 enters, which is just below cross section 4019. Tributary B Upstream has two culverts and one bridge. Tributary B Downstream has five culverts. Tributary B2 has one culvert.

Tributary C. Tributary C is one individual reach with a length of 1.3 miles and consists of 50 cross sections. The cross sections begin at station 7034 and end at station 421. There are a total of seven culverts along this reach.

Tributary D. Tributary D is the shortest reach in the study with a total length of 0.5 mile. It contains 16 cross sections and one culvert.

[13] http://www.hec.usace.army.mil/software/hec-ras/

2.2.2 – Black Walnut Creek

Hydraulic modeling for Black Walnut Creek involved the modeling of 11.3 miles of stream that flow in a southwesterly direction and included 126 cross sections from station 45239 to 417, five bridges and two culverts. All the structures are on the mainstream of the creek. The locations and layouts of structures and cross sections are illustrated in **Exhibit 2-2 – Black Walnut Creek Cross Section Locations**. For this study, the three unnamed tributaries to Black Walnut Creek were named Tributary 1, 2 and 3.

Tributary 1. Tributary 1 is the longest tributary in the watershed with a length of 0.9 mile. Tributary 1 begins on the east side of Central Avenue and south of Church Road, runs west and passes under Central Avenue before flowing into the mainstream below cross section 6680. There are 10 cross sections along this tributary ranging from cross section 4617 to 550. Tributary 1 is broken into two reaches at the junction where Tributary 1-1 enters, which is just below cross section 2266.

Tributary 2. Tributary 2 is the shortest tributary in the watershed with a length of 0.6 mile. Tributary 2 begins south of Pauling Road and runs southeast into the mainstream below cross section 32277. There are six cross sections along this tributary ranging from cross sections 3000 to 538.

Tributary 3. Tributary 3 is 0.8 mile long and begins on the north side of Pauling Road and runs south into the mainstream below cross section 33542. There are eight cross sections along this tributary ranging from cross sections 4000 to 451.

2.2.3 – South Branch Rock Creek

Hydraulic modeling for South Branch Rock Creek involved the modeling of eight miles of stream that flows in a southwesterly direction and included 86 cross sections from station 28500 to 353 and seven culverts. The locations and layouts of structures and cross sections are illustrated in **Exhibit 2-3 – South Branch Rock Creek Cross Section Locations**. For this study, the two unnamed tributaries to South Branch Rock Creek were named Tributary 1 and 2.

Tributary 1. Tributary 1 is approximately two miles in length and is comprised of two culverts and 22 cross sections starting at station 10500 and ending at station 733. Tributary 1 begins on the west side of Kedzie Avenue and runs southwest, passing under Church Road and Crawford Avenue before flowing into the mainstream below cross section 15000.

Tributary 2. Tributary 2 is the shortest tributary in the watershed with a length of 0.5 mile. Tributary 2 flows north to south along Crawford Avenue and flows into the mainstream below cross section 2000. There are five cross sections along this tributary ranging from cross sections 2500 to 1000.

2.2.4 – Exline Slough

Hydraulic modeling for Exline Slough involved the modeling of 2.6 miles of stream that flow in a southerly direction and included 17 cross sections from station 8250 to 4500 and one culvert. The locations and layouts of structures and cross sections are illustrated in **Exhibit 2-4 – Exline Slough Cross Section Locations**.

2.2.5 – Plum Creek

Hydraulic modeling for Plum Creek involved the modeling of eight miles of stream which included 96 cross sections from station 21250 to 250, two bridges and six culverts. This is the only waterway that generally flows in a northeasterly direction in the study area. The locations and layouts of structures and cross sections are illustrated in **Exhibit 2-5 – Plum Creek Cross Section Locations**. For this study, the five unnamed tributaries to Plum Creek were named Tributary 2, 3, 4, 5 and 6.

Tributary 2. Tributary 2 is the longest tributary in the watershed with a length of 1.5 miles. Tributary 2 begins on the south side of south of Eagle Lake Road and runs north into the mainstream below cross section 6000. There is one culvert and 16 cross sections along this tributary ranging from cross sections 7499 to 250.

Tributary 3. Tributary 3 is 0.7 mile long and flows north to south east of Ashland Road to the mainstream below cross section 8250. There are seven cross sections along this tributary ranging from cross sections 3500 to 500. There are no structures along this reach.

Tributary 4. Tributary 4 is 0.6 mile long and begins south of Eagle Lake Road and runs southwest into the mainstream below cross section 15250. There is one culvert and six cross sections along this tributary ranging from cross sections 3000 to 750.

Tributary 5. Tributary 5 is 0.9 mile long and begins near Church Road and runs west into the mainstream below cross section 17250. There are 11 cross sections along this tributary ranging from cross sections 4500 to 250. There are no structures along this reach.

Tributary 6. Tributary 6 is the shortest tributary in the watershed and is 0.3 mile long and begins on the south side of Church Road and runs northwest into the mainstream below cross section 18750. There is one culvert and five cross sections along this tributary ranging from cross sections 1428 to 242.

2.3 – Cross Sections

Cross sections for the study area were collected based on a limited hydraulic survey performed by DB Sterlin Consultants, Inc. between August 2010 and March 2011. Field survey of the streams was limited to the channel and supplemented with LiDAR data to develop full floodplain cross sections. Surveys of hydraulic structures, (i.e., culverts), were limited to only the upstream face of the structure. Additional cross sections were added upstream and downstream of the surveyed cross sections to provide additional definition along the model reach. **Exhibit 2-1 – Rock Creek Cross Section Locations, Exhibit 2-2 – Black Walnut Creek Cross Section Locations, Exhibit 2-3 – South Branch Rock Creek Cross Section Locations, Exhibit 2-4 – Exline Slough Cross Section Locations** and **Exhibit 2-5 – Plum Creek Cross Section Locations** show the cross section locations along the modeled reaches. The cross sections shaded in orange represent the stations where channels were surveyed.

2.3.1 – Rock Creek

Two sets of cross section shapefiles were created, one of which was based on field survey data while the other was based on LiDAR-based topographic data. The cross section survey data was imported into a GIS project and ultimately compiled into stream features using the USACE HEC-GeoRAS.[14] HEC-GeoRAS is a computer program which interfaces with GIS and includes a set of utilities for processing topographic data for import into the USACE's one-dimensional steady flow hydraulics model called HEC-RAS. Once the relevant data is in HEC-RAS, it can be manipulated and edited to represent the desired site conditions.

Due to the limited detail survey, two different surface datasets in Triangular Irregular Network (TIN) format were developed using GIS. One TIN was based on the LiDAR-based topographic data and the second TIN was based on field survey data. The LiDAR based TIN was used to provide floodplain data for all of the cross sections. The survey TIN was used to obtain the cross sections with channel survey data into the model. Both sets of cross sections were exported to HEC-RAS and combined using the graphical cross section editor. The channel slope between the surveyed cross sections was carried through to the cross sections without a surveyed channel. In areas along the reach in the spreadsheet where survey data was not obtained for a distance greater than 500 feet, the channel invert was often incorrect. In these locations, the channel invert was adjusted based on the LiDAR channel slope or a separate evaluation of the survey and the LiDAR slopes. Spreadsheets documenting these calculations are included in **Appendix E - Rock Creek Channel Slope Calculations**.

2.3.2 – Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek

For these four watersheds, cross section locations were initially set 500 feet apart throughout the reaches to be modeled. In some areas where the channel slopes and cross section geometry appeared quite consistent

[14] http://www.hec.usace.army.mil/software/hec-ras/hec-georas.html

for a longer distance, the cross section spacing was increased to 1,000 feet. In addition, cross section locations were set upstream and downstream of specific bridge and culvert crossings. Cross sections were also occasionally added to capture apparent changes in channel characteristics.

Cross section geometry was composited from several sources. Digital terrain mapping from LiDAR was used to generate the geometry of the cross sections above the channel area though WISE. However, the LiDAR data usually did not capture the actual channel cross section in enough detail for model use. One reason for this is LiDAR's inability to capture the channel boundaries below any water surfaces.

Channel cross sections were field-surveyed upstream and downstream of bridge and culvert crossings. For this limited detailed study, it was therefore assumed that similar channel geometry extended between the ground-surveyed areas. The channel shapes from the ground survey were then merged with the overbank geometry from LiDAR to create composite cross section geometry.

Near the bridge and culvert crossings, efforts were made to survey channel cross sections that didn't reflect the immediate influence of the crossing (artificial changes to the channel as it approached the constrained waterway opening for the structure).

2.4 – Bridges

Field survey data, record drawings and site photos were used to model the structures in the study area. Due to right-of-way access issues, the culvert under I-57 on Tributary A of Rock Creek did not have a surveyed upstream invert, so the invert was modeled based on site topographic data and the channel profile where survey was provided. **Exhibit 2-6 – Rock Creek Culvert & Bridge Locations, Exhibit 2-7 – Black Walnut Creek Culvert & Bridge Locations, Exhibit 2-8 – South Branch Rock Creek Culvert & Bridge Locations, Exhibit 2-9 – Exline Slough Culvert & Bridge Locations** and **Exhibit 2-10 – Plum Creek Culvert & Bridge Locations** show the tributary locations and the locations of the culvert and bridge structures for the study area that were field surveyed. **Appendix F – Field Survey Data & Notes** contains the field survey notes and photographs for all five watersheds.

2.5 – Manning's N-Values

Manning's N-values were selected based on site visit photographs, aerial photography and Cowan's Method. These values represent channel roughness and are assigned at different locations for each cross section. The values vary throughout the model based on factors such as channel and overbank vegetation.

In the Rock Creek watershed, a typical value of 0.035 was used for the channel N-values. In the south part of the model, another channel is present that conveys flow through the IL-50 culvert, but it does not always have flow in it. Therefore, a slightly higher value of 0.04 was used. For open or cultivated fields, values ranging from 0.05 to 0.06 were used. For areas with heavy tree cover, values ranged from 0.10 to 0.12. **Appendix G-1 – Rock Creek Modeling Logs** contains detailed descriptions of the Manning's roughness throughout the reaches and tributaries.

For the other watersheds, Manning's N roughness coefficients were estimated from standard reference tables and review of watershed conditions. Typically, 0.04 was used for the channel N-value. For Manning's N-values for left and right overbanks, a value of 0.04 was used for working farmland and 0.1 for wooded land.

2.6 – Expansion/Contraction Coefficients

In accordance with guidelines in Appendix B of the *HEC-RAS Hydraulic Reference Manual,*[15] contraction and expansion coefficients of 0.3 and 0.5 were used at all cross sections where structures, such as bridges or culverts, influenced the characteristics of overbank flow. The default contraction and expansion coefficients of 0.1 and 0.3 were used at all other cross sections.

[15] http://www.hec.usace.army.mil/software/hec-ras/documents/hydref/append_b.pdf

2.7 – Ineffective Flow Areas

Ineffective flow areas are used in HEC-RAS to define areas where flow is stagnant but still functional as floodplain storage. Ineffective flow areas were incorporated into the HEC-RAS model upstream and downstream of the bridges in order to account for the contraction and expansion of flood flows as they pass through the restrictive openings. In accordance with the modeling guidelines in Chapter 5 of the *HEC-RAS Hydraulic Reference Manual*, a contraction ratio of 1:1 was used upstream of each structure and an expansion ratio of 2:1 (parallel: perpendicular to flow) was used downstream of each structure.

2.8 – Blocked Obstructions

Blocked obstructions are used to model areas where there is no flow or floodplain storage. Homes and other buildings located within the floodplain were modeled as blocked obstructions. The locations of the building(s) were determined using GIS based on digital orthographic photography of the site.

2.9 – Levees

Marking a point in a cross section as a levee is a method commonly used to prevent water from flowing into areas that are not necessarily available for storage until a certain elevation is reached. In the study area, physical (topographic) levees exist along the channel at some locations. These are not regulatory levees but appear to have been constructed by local property owners. Although at some locations these levees appear to contain flow when the peak 100-year water surface at just one cross section is considered, in most cases there is wide overbank flow upstream and/or downstream. Therefore, in general, levees did not appear to be effective at fully restricting the 100-year flow and in most cases the levee option was not used in HEC-RAS to confine the 100-year flow to the channel, or the levee elevation was set low enough to be overtopped during the 100-year event. For some locations at Rock Creek, levee points were placed on cross sections that showed the floodplain being lower than the channel to improve the accuracy and stability of the hydraulics modeling. This modeling tool is not intended to indicate that an actual levee is present at that location.

2.10 – HEC-RAS Model Output & Results

The HEC-RAS models included with this report reflect the culmination of all the modeling efforts for the SSA Floodplain Report. **Table 2-1 – Rock Creek Final 100-Year Flows for HEC-RAS Model, Table 2-2 – Black Walnut Creek Final 100-Year Flows for HEC-RAS Model, Table 2-3 – South Branch Rock Creek Final 100-Year Flows for HEC-RAS Model, Table 2-4 – Exline Slough Final 100-Year Flows for HEC-RAS Model** and **Table 2-5 – Plum Creek Final 100-Year Flows for HEC-RAS Model** document the final 100-year flow rates used for this study and include the location of the HEC-HMS element where the flows were used and HEC-RAS cross sections where the flows were applied. A set of standard summary tables from HEC-RAS are included in **Appendix H-1 – Rock Creek HEC-RAS Model Data, Appendix H-2 – Black Walnut Creek HEC-RAS Model Data, Appendix H-3 – South Branch Rock Creek HEC-RAS Model Data, Appendix H-4 – Exline Slough HEC-RAS Model Data** and **Appendix H-5 – Plum Creek HEC-RAS Model Data**.

Table 2-1 – Rock Creek Final 100-Year Flows for HEC-RAS Model

Cross Section	Flow (cfs)	HMS Component
Main Branch		
15588	1,415.5	Junction-24
13335	1,478.2	Junction-4
12609	2,333.3	Junction-33
10901	1,902.2	Junction-6
8697	2,050.9	Junction-26
7200	2,057.9	Junction-38

Table 2-1 – Rock Creek Final 100-Year Flows for HEC-RAS Model		
Cross Section	**Flow (cfs)**	**HMS Component**
7009	2,100.2	Junction-19
5980	2,300.7	Junction-25
3897	2,352.6	Junction-30
2622	2,789.0	Junction-7
1054	3,080.1	Junction-20
Tributary A		
8294	137.3	Sub-Basin A1-02
5665	201.5	Junction-8
4863	317.5	Junction-9
3170	475.4	Junction-32
Tributary A2		
1159	69.6	Sub-Basin A2
Tributary B		
12176	81.6	Junction-1
11618	142.8	Junction-36
9573	382.9	Junction-2
6082	647.9	Junction-16
4557	734.5	Junction-37
3571	781.0	Junction-3
Tributary B2		
895	52.5	Sub-Basin 8
Tributary C		
7034	67.1	Sub-Basin C1-04
6191	125.0	Junction-21
5418	180.3	Junction-35
4261	266.6	Junction-34
2997	345.3	Junction-22
2375	110.2	Trib C RR Tracks
730	139.4	Junction-28
Tributary D		
2936	124.1	Sub-Basin D1

Table 2-2 – Black Walnut Creek Final 100-Year Flows for HEC-RAS Model		
Cross Section	**Flow (cfs)**	**HMS Component**
Main Branch		
45239	777.0	Sub-Basin 11
43239	875.3	Junction-12C
40239	969.1	Junction-13C
36739	1,084.6	Junction-14C
35239	1,092.3	Junction-15C
32777	1,192.3	Junction-15CC
31602	1,649.2	Junction-16CC
29170	1,799.3	Junction-18C
25788	1,829.8	Junction-19C
21622	1,822.0	Junction-20C
17530	1,890.0	Junction-21C
15662	1,880.2	Junction-22C
14256	1,877.2	Junction-23C
12662	1,892.7	Junction-24C
9162	1,907.6	Junction-25C
6000	1,909.4	Junction-25CC
5500	1,896.9	Junction-28C

Table 2-2 – Black Walnut Creek Final 100-Year Flows for HEC-RAS Model

Cross Section	Flow (cfs)	HMS Component
2956	2,053.7	Junction-29C
1000	2,191.8	Junction-30C
Tributary 1		
4617	86.4	Sub-Basin 1
3519	139.6	Junction-4C
1913	260.9	Junction-7C
Tributary 1-1		
1494	66.3	Junction 3C
Tributary 2		
3000	261.4	Sub-Basin 8
2000	452.7	Junction 10C
Tributary 3		
4000	134.2	Sub-Basin 5
1500	166.2	Junction 6C

Table 2-3 – South Branch Rock Creek Final 100-Year Flows for HEC-RAS Model

Cross Section	Flow (cfs)	HMS Components
Main Branch		
28500	161.0	Sub-Basin 1
27500	156.8	Reach 3R
26389	286.7	Junction 3C
23000	428.6	Junction 4C
19322	625.8	Junction 14C
17500	692.8	Junction 9C
14500	1,103.3	Junction 7CC
9519	1,284.9	Junction 10CC
7485	1,359.2	Junction 15CC
4500	1,356.2	Junction 16CC
3000	1,383.7	Junction 17C
1993	1,393.9	Junction 18C
Tributary 1		
10500	323.8	Sub-Basin 2
7000	373.8	Junction 5C
5017	426.3	Junction 6C
2500	441.8	Junction 7C
Tributary 2		
2500	299.5	Sub-Basin 8

Table 2-4 – Exline Slough Final 100-Year Flows for HEC-RAS Model

Cross Section	Flow (cfs)	HMS Components
Main Branch		
8250	155.5	Junction 1
6376	500.1	Junction 2
5750	543.8	Junction 3

Table 2-5 – Plum Creek Final 100-Year Flows for HEC-RAS Model

Cross Section	Flow (cfs)	HMS Components
Main Branch		
21250	60.0	Sub-Basin 1

Table 2-5 – Plum Creek Final 100-Year Flows for HEC-RAS Model

Cross Section	Flow (cfs)	HMS Components
19781	125.3	Junction 3C
18250	161.4	Junction 3CC
16750	231	Reservoir 2
15000	481	Reservoir 3
13581	530	Junction 12C
11500	742.7	Junction 13C
9750	840.2	Junction 14C
8000	750.7	Reservoir 4
5641	1061.6	Reservoir 5
4250	1186.1	Junction 21C
Tributary 2		
7499	192.9	Sub-Basin 15
4730	74.1	Sub-Basin 16C
1750	27.6	Sub-Basin 18C
Tributary 3		
3500	433.8	Sub-Basin 20
Tributary 4		
3000	69.3	Sub-Basin 9
2027	36.1	Junction 10C
Tributary 5		
4500	136.0	Sub-Basin 8
2382	207.2	Junction 10C
Tributary 6		
1428	57.4	Sub-Basin 2
745	46.6	Junction 4C

Section 3 – Calibration

In order to verify that hydrologic and hydraulic models are producing a reasonable approximation for frequency-based flood events, the models are often calibrated to known storm events. The process of calibration requires not only precipitation data for known flood events but also a set of known high water marks for that flood event. As part of the preparation of the SSA Floodplain Report, questionnaires were distributed to landowners throughout the study area. The response rate was low and several responses received did not provide enough information to allow a detailed model calibration to a historical event to be performed. **Exhibit 3-1 – Floodplain Survey Response Map** depicts responses to this questionnaire in a graphic form.

3.1 – August 2007 Storm

For the Rock Creek Watershed study, an attempt was made to calibrate the HEC-HMS model with the collected high water marks at Monee Reservoir. Flood information was provided by Will County staff for flooding that occurred at the reservoir. Site staff provided pictures which depicted the flooding at the reservoir for the 3-day rain event in 2007. Surveyors were sent to Monee Reservoir to meet with site staff to determine elevations of these high water marks for the storm based on the photos from August of 2007. The elevations ranged from 745.87 feet to 746.86 feet with an average elevation of 746.5 feet. Copies of the letter and photographs from Will County, as well as elevation information from the field survey, can be found in **Appendix F**.

Since the pictures only depict the extent of flooding and not a specific high water mark, the uncertainty of the high water mark elevations is rather high. However, the results from the HEC-HMS model were used to validate model results against the best available high water marks taken at Monee Reservoir. Precipitation information from a rainfall gage at Monee and a rainfall gage at Chicago Midway International Airport (MDW), located approximately 28 miles north, was acquired. The gage at Monee recorded 9.88 inches of rain from August 19-23, 2007 and the gage at MDW recorded 5.82 inches of rain for the same period. Gridded rainfall data obtained from the National Oceanic and Atmospheric Administration (NOAA) for the August 19-23, 2007 storm was also compared to the gage data. The NOAA data reported a precipitation amount of 5.9 inches over the five day period. The NOAA data likely included the precipitation data from the MDW gage; therefore, it was used primarily to assess the spatial distribution of the storm event.

Since the Monee rain gage, only provided daily rainfall summaries for this event, the MDW gage, which supplied hourly precipitation data for the calibration event, was used to distribute the total precipitation amount for the Monee gage (9.88 inches). This combined dataset was input into HEC-HMS to determine an estimated peak stage in Monee Reservoir for the August 2007 event. This estimate of the peak stage was compared to a high water mark collected and surveyed at the Monee Reservoir site. Using this approach, a peak reservoir elevation of 747.5 feet was computed in the HEC-HMS model, which is 1.0 foot above the average of the surveyed high water marks (746.5 feet) and 0.36 feet below the highest of the surveyed high water marks (746.86 feet).

Typically it is a goal to calibrate the model to within 0.5 feet of the high water marks on a project. Having one high water mark location, the opportunities to adequately evaluate the model performance are poor. The uncertainty of the elevation of the lone high water mark somewhat invalidates the calibration effort. Generally speaking, reasons for the discrepancy between the modeled peak stage at Monee Reservoir for the August 2007 storm and the surveyed high water mark range from discrepancies in the actual versus modeled conditions for the reservoir's trash rack, the assumed tailwater elevation at the discharge point and the validity of the high water mark elevation.

It is worth noting that the model used in this study assumed that the trash rack on the primary spillway on Monee Reservoir was 40 percent clogged which would reduce the discharge capability of the reservoir thus increasing the stage. The complex rating of the reservoir's Morning Glory structure was refined to better simulate the August 2007 storm. However, based on the record drawings for the reservoir, the discharge estimates for the reservoir assumed that the trash rack on the primary spillway on Monee Reservoir was 40 percent open (60 percent clogged). This condition was used for the overall study and was maintained for the calibration of the August 2007 storm.

3.2 – Comparison to USGS Regression Equations

Due to the limited response for flood information at the site, the project team used other approaches to assess the reasonableness of the HEC-HMS results. The most common methodology for estimating peak discharges for ungauged rural watersheds in Illinois is flood flow regional regression equations. In Illinois, the regression equations are programmed into the web-based StreamStats USGS interface. StreamStats estimates runoff utilizing a combination of basin parameters like total drainage area and basin slope. These parameters are determined using a combination of topographic data and statistical data for the subject stream, if available. The basin parameters and statistical numbers are input into regional regression equations published by the USGS in the 2004 report titled, "Estimating Flood-Peak Discharge Magnitudes and Frequencies for Rural Streams in Illinois."[16]

The StreamStats was used to generate flow estimates in similar locations along Rock Creek to provide a comparison of the discharge estimates developed using the more detailed HEC-HMS model. For the other watersheds, StreamStats analysis was performed for a location corresponding to the downstream outlet of the HEC-HMS model. The regression equations yield peak discharge estimates for a full range of frequency events from the 2-year to the 100-year storm. Flow from the HEC-HMS model was compared to the results of the StreamStats flow estimates. Initially, the unadjusted versions of the HEC-HMS model for all the watersheds predicted higher discharges than StreamStats. Through use of storage areas, well-drained soils, lower curve numbers and increased storage coefficients from the Clark Unit Hydrograph Method, flows were reduced in the final HEC-HMS model.

Ultimately, it was determined that the final HEC-HMS model results provided reasonable peak flood flows and were therefore utilized for this study. A comparison of the final HEC-HMS and StreamStats flows at select locations along Rock Creek for the 100-year, 12-hour event are shown in **Table 3-1 – HEC-HMS & StreamStats Flow Comparison Table (Rock Creek)**. **Table 3-2 - HEC HMS & StreamStats Flow Comparison Table (Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek** depicts comparisons for the other SSA watershed reaches. It is worth noting that StreamStats lists the prediction error for all of the locations shown in **Table 3-1 – HEC-HMS & StreamStats Flow Comparison Table (Rock Creek)** as 49 percent and the equivalent years of record is only 5.6-years which is extremely low.

Table 3-1 – HEC-HMS & StreamStats Flow Comparison Table (Rock Creek)

Element	StreamStats Est. Flow (cfs)	90 Percent Prediction Interval		HEC-HMS Estimated Flow (cfs)	Percent Higher Than Stream Stats Est. Flow	Percent Higher Than Stream Stats Max Est. Flow
		StreamStats Est. Minimum Flow (cfs)	StreamStats Est. Maximum Flow (cfs)			
Junc-02 (Trib B only	140	65	303	383	174%	26%
Junc-22	222	103	479	345	56%	-28%
Junc-16	339	158	728	648	91%	-11%
Junc-31 (Trib B only)	415	193	892	947	128%	6%
Junc-24	660	309	1,410	1,416	114%	0.4%
Structure R5 (Junction 4)	704	330	1,510	1,478	110%	-2%
Junc-04	714	334	1,530	1,478	107%	-3%
Junc-06	982	460	2,100	1,902	94%	-9%
Structure R4 (Junction 6)	983	460	2,100	1,902	94%	-9%

[16] http://il.water.usgs.gov/pubs/sir2004-5103.pdf

Table 3-1 – HEC-HMS & StreamStats Flow Comparison Table (Rock Creek)

Element	StreamStats Est. Flow (cfs)	90 Percent Prediction Interval		HEC-HMS Estimated Flow (cfs)	Percent Higher Than Stream Stats Est. Flow	Percent Higher Than Stream Stats Max Est. Flow
		StreamStats Est. Minimum Flow (cfs)	StreamStats Est. Maximum Flow (cfs)			
Junc-30	1,040	489	2,230	2,353	126%	6%
Structure R1 (Junction 30)	1,040	489	2,230	2,353	126%	6%
Junc-20	1,220	573	2,610	3,080	152%	18%

Table 3-2 – HEC HMS & StreamStats Flow Comparison Table (Black Walnut Creek, South Branch Rock Creek, Exline Slough and Plum Creek

River	StreamStats Est. Flow (cps)	90 Percent Prediction Interval		HEC-HMS Estimated Flow (cfs)	Percent Higher Than Stream Stats Est. Flow	Percent Higher Than Stream Stats Max Est. Flow
		Minimum Flow (cfs)	Maximum Flow (cfs)			
Black Walnut Creek	976	456	2,090	2,190	124%	5%
South Branch Rock Creek	811	379	1,740	1,550	91%	-11%
Exline Slough	326	152	703	550	69%	-22%
Plum Creek	885	414	1,890	1,190	34%	-37%

Section 4 – Floodway and Floodplain Mapping

The existing conditions 100-year floodplain map was created based on the results of this study for all five streams and their tributaries. Maps depicting the floodplain boundary are included in **Exhibit 4–1 – Rock Creek Floodplain Mapping Key Sheet**; **Exhibit 4–1–A – Rock Creek Floodplain Mapping – Sheet 1; Exhibit 4–1–B – Rock Creek Floodplain Mapping – Sheet 2; Exhibit 4–1–C- -Rock Creek Floodplain Mapping – Sheet 3; Exhibit 4–1–D - Rock Creek Floodplain Mapping – Sheet 4; Exhibit 4–1–E – Rock Creek Floodplain Mapping – Sheet 5; Exhibit 4–2–A – Black Walnut Creek Floodplain Mapping – Sheet 1; Exhibit 4–2–B - Black Walnut Creek Floodplain Mapping – Sheet 2; Exhibit 4–2–C - Black Walnut Creek Floodplain Mapping – Sheet 3; Exhibit 4–3–A – South Branch Rock Creek Floodplain Mapping – Sheet 1; Exhibit 4–3–B – South Branch Rock Creek Floodplain Mapping – Sheet 2; Exhibit 4-4 – Exline Slough Floodplain Mapping**; and **Exhibit 4-5 – Plum Creek Floodplain Mapping**. The floodway was also determined using encroachment methods in HEC-RAS which is an iterative process. However, these boundaries are not presented on the floodplain maps.

The floodway encroachment boundaries were developed initially using encroachment Method 4 in HEC-RAS which uses the program to iteratively determine the proper encroachment stations based on equal conveyance reductions for each cross section which would surcharge the water surfaces the desired amount. In Illinois, the maximum surcharge for the floodway is 0.10 feet. The outcome of Method 4 is typically an initial determination of the proper encroachment stations. Often further refinement of the floodway boundaries to meet the surcharge requirement is needed. This refinement is performed using encroachment Method 1 which allows the user to enter the exact locations for the encroachment stations for each section. For this study, Method 4 was used as a starting point for the encroachment stations and Method 1 was used to refine the encroachment locations.

There are several locations in the floodway run where encroachments result in 0.00 feet of increase. This does not mean that further encroachment would be allowable. For this model, there are two situations when this condition takes place. The first situation is when the 100-year flow is contained within the channel banks which occur at the headwaters of some of the tributaries which drain the agricultural fields. When defining the floodway, the encroachments are not allowed to enter within the channel. Therefore, the encroachments are set as the channel banks and considered to be the maximum allowable encroachment. The other situation which takes place is when encroaching any further at a particular cross section causes an unacceptable increase at a cross section located further upstream. This typically happens in areas where the encroachments are near higher velocity flow, especially at or near culvert or bridge structures and their ineffective flow boundaries. In some instances, the reverse effect would occur and the surcharge upstream would invert and produce a negative increase (surcharge < 0.00 ft). This means that the velocities increased due to the encroachment and, therefore, caused negative impacts upstream. This situation was also avoided, by restricting the encroachments.

In the end, a floodway model was created with encroachment impacts ranging from 0.00 feet to 0.10 feet throughout. This meets the Illinois Department of Natural Resources, Office of Water Resources requirements outlined in Part 3708.60 Delineation of the Regulatory Floodway[17] which states that encroachment of the 100-year profile cannot result singularly or cumulatively in more than 0.10 feet of increases in flood stage or a 10 percent increase in velocity.

[17] http://www.dnr.illinois.gov/adrules/documents/17-3708.pdf

Section 5 – Impact of Proposed ALP Facilities

The impact on the delineated floodplains of the structures and facilities associated with the proposed Airport Layout Plan (ALP), dated September 20, 2012, was analyzed. The purpose of this task was to estimate the size, shape, material, length and slope of the proposed drainage structures to be constructed over/along the five waterways analyzed in this study and identify anticipated impacts. After reviewing the layout plan, eight new hydraulic structures were identified with the proposed inaugural airport facilities. Six structures are proposed on the Rock Creek system, one structure on Black Walnut Creek and one structure on the South Branch of Rock Creek.

This evaluation was made using the hydrologic and hydraulic models developed as part of the overall floodplain study. The criteria for sizing these structures involved determining the proper structure opening that would create no net increase in water surface elevation beyond the project property boundary. Also, if proposed roadway profile data is available for the stream crossing location, there should also be at least three feet of freeboard above the 100-year headwater elevation to the proposed edge of pavement.

Since no proposed contour or profile grade line information was provided, the arbitrary deck elevations were input into the model high enough that no overtopping would occur. All culverts were assumed to be reinforced concrete and the culvert lengths were estimated slightly longer than the embankment width to account for the embankment slope. Culvert inverts were projected based on the longitudinal slope between the bounding cross-sections in the model. Initial estimates of the culvert diameters were made by looking at nearby culverts in the model that had already been sized.

Once all the information for the new structures was input into the model, the model was run and the results were compared to the existing conditions model. The proposed sizes were increased until the model results showed no increases in water surface at the airport property boundaries. Since the freeboard criteria could not be checked, suggestions for the minimum ramp elevations were made based on the estimated structure sizes. If the ramp elevations are lower than the suggested elevations, it is possible the size of the culvert may need to increase in order to reduce the headwater.

This task also includes an estimate of the floodplain fill volumes associated with each crossing. The preliminary structure data presented as part of this effort is not considered adequate for design and permitting purposes and should only be used for planning purposes. Final sizes will be determined as the facilities design process moves forward and model parameters could be updated at that time to provide additional guidance for design and mitigation of impacts.

5.1 – Rock Creek

Six new hydraulic structures on Rock Creek and its tributaries were identified as part of the proposed passenger access road and the I-57 interchange for SSA. **Exhibit 5-1 – Rock Creek New Hydraulic Structures** is a map showing the locations (numbered as 1 through 6) of these structures for Rock Creek. The roadways (decks) were located in the model by measuring distances from the decks to the model cross sections. Cross sections located at the proposed structure locations were either deleted or moved so that the structure could be defined in the model. Structures 3 and 5 were incorporated into the model as culvert extensions of the existing 6-foot diameter culvert under I-57 due to the close proximity of the ramps to the interstate at this location. A distance of 20 feet was added to both ends of the culvert to account for the width of the ramps.

Table 5-1 - Rock Creek Preliminary Drainage Structure Summary lists the results of the modeling process including the structure types, estimated structure sizes, and suggested minimum pavement elevations (to obtain 3-foot of freeboard). As shown above, the structures along Tributary A (1, 2, 3 and 5) were sized as 6-foot diameter culverts. This matches the size of the existing culvert under I-57. Structures 3 and 5 are extensions of the existing culvert under I-57. No increases in water surface are present in the model at the property line approximately 4,500 feet upstream of the interstate. Structure 4 was sized as a 4-foot culvert which resulted in no increases in water surface at the property line approximately 1,400 feet upstream. Structure 6 was sized as a 49-foot bridge, which matches the length of the bridge immediately downstream on Governor's Highway. Increases in water surface at this location were unavoidable since this structure is located in the backwater of the downstream structures. Therefore, this structure was modeled separately with no other structures included in the model. Water surfaces were then compared to a natural conditions model to ensure that the bridge caused no more than a 1-foot increase above natural conditions.

Based on the preliminary structure sizing, the floodplain fill volumes associated with these structures were estimated. These volumes were based on an assumed floodplain length, width and depth for each crossing. The volumes have been included in **Table 5-1 – Rock Creek Preliminary Drainage Structure Summary**. Estimates of floodplain fill volumes presented in this table do not include fill for areas not associated with these proposed drainage structures.

Table 5-1 – Rock Creek Preliminary Drainage Structure Summary

Structure Location	Type	Shape	Material	Dia. (ft)	Length (ft)	U/S Invert (ft)	D/S Invert (ft)	Slope (ft/ft)	Bridge Length (ft)	Bridge Width (ft)	Minimum Pavement Elev. (ft)	Estimated Floodplain Fill (yd^3)
1	Culvert	Circular	Concrete	6	18	747.3	747.1	0.0111	NA	NA	756.7	90
2	Culvert	Circular	Concrete	6	107	740.9	738.9	0.0180	NA	NA	752.8	450
3	Culvert	Circular	Concrete	6	360	729.4	727.8	0.0044	NA	NA	741.5	300
4	Culvert	Circular	Concrete	4	98	721.3	720.0	0.0133	NA	NA	738.6	400
5	Culvert	Circular	Concrete	6	360	729.4	727.8	0.0044	NA	NA	736.2	90
6	Bridge	Bridge	Concrete	NA	NA	NA	NA	NA	49	51	726.9	21,000

Note: Structures 3 and 5 were modeled as extensions of the existing 6-foot diameter culvert under I-57. Structure 6 is a bridge that does not meet the "no-rise" criteria over existing conditions. It does not cause more than 1 foot of an increase over natural conditions at the bridge.

5.2 – Black Walnut Creek

The proposed structure to be constructed on Black Walnut Creek is a bridge at the Airport Access Road north of Eagle Lake Road between Will Center Road and Crawford Avenue. This bridge is depicted as Structure Number 7 on **Exhibit 5–2 – Black Walnut Creek New Hydraulic Structures**. The impacts of the structure to the 100-year flood elevations inside and outside the airport inaugural boundary were evaluated. **Table 5-2 – Black Walnut Creek Preliminary Drainage Structure Summary** lists the results of the modeling process including the bridge length and width, suggested minimum pavement elevations and estimated floodplain fill. The required compensatory storage due to fill associated with the proposed structure was calculated by comparing the floodplain storage volume changes before and after the structure is incorporated into the model. A bridge at this location would be required to have an opening span of approximately 67 feet to not increase the 100-year flood elevations outside the Inaugural Airport boundary (the first upstream and downstream river cross sections beyond the boundary are 25662 and 11162, respectively). The associated compensatory storage with this structure was estimated around 10 acre-ft. The new structure does not increase the 100-year flood elevations of river segment beyond the proposed airport boundary. The elevations increase by 0.3 to 0.76 feet and the floodplain widths increase by 26 to 44 feet within the airport boundary.

Table 5-2 – Black Walnut Creek Preliminary Drainage Structure Summary

Structure Location	Type	Shape	Material	Bridge Length (ft)	Bridge Width (ft)	Minimum Pavement Elevation (ft)	Estimated Floodplain Fill (yd^3)
7	Bridge	Bridge	Concrete	178	67	722.3	16,100

5.3 – South Branch Rock Creek

The one structure to be constructed on South Branch Rock Creek is a new 1,600 foot culvert extending under the new airport Runway 9R-27L. The culvert is depicted as Structure Number 8 on **Exhibit 5-3 – South Branch Rock Creek New Hydraulic Structures**. In order to limit the 100-year flood elevation increases within the airport boundary to 1 foot and zero outside the airport boundary (the first downstream river cross section outside the boundary is 23000), the proposed 1,600 foot culvert under the new Airport Runway 9R-27L would be a 10 foot by 12 foot rectangle box culvert. **Table 5-3 – South Branch Rock Creek Preliminary Drainage Structure Summary** lists the results of the modeling process including the culvert span length, upstream and downstream invert elevations, slope, suggested minimum pavement elevation and estimated floodplain fill. The compensatory storage associated with this structure is 1.3 acre-ft. The new structure increases the 100-year flow elevations of river segment within the airport boundary by 0.02 to 0.78 feet. The floodplain widths increase by 2.6 to 287.5 feet. There are no increases in the floodplain beyond the proposed airport boundary.

Table 5-3 – South Branch Rock Creek Preliminary Drainage Structure Summary

Structure Location	Type	Shape	Material	Span (ft)	Rise (ft)	U/S Invert (ft)	D/S Invert (ft)	Slope (ft/ft)	Minimum Pavement Elev. (ft)	Estimated Floodplain Fill (yd^3)
8	Culvert	Rectangle	Concrete	12	10	736.9	732.8	0.003	741.3	2,000

Section 6 - Conclusions

This report summarizes the procedures that were used to develop floodplain boundaries and flood profiles for the group of streams within the limits of the South Suburban Airport project using the "limited detailed study" approach. The impact on the newly delineated floodplains of the structures and facilities associated with the proposed Airport Layout Plan (ALP), dated September 20, 2012, was then analyzed. This study does not evaluate drainage or stormwater management features that will be required as part of the airport development.

The existing condition floodplain boundaries mapped by this study compare well to the previously drawn approximate boundaries and represent an improvement in detail and confidence with this effort. Floodplain boundaries are now quantified in stage, flow, and location based on current topography and conditions in the watershed. Differences between the boundaries are attributable to the care and standards followed to develop representative hydrologic and hydraulic models and the high level of detail associated with the LiDAR topographic data used for this study compared to the coarser data used in the original FEMA Will County flood zone mapping.

A total of eight new hydraulic structures were identified with the proposed inaugural airport facilities. Six roadway crossing structures are proposed on the Rock Creek system associated with a new airport access road. One roadway crossing structure is proposed on Black Walnut Creek to accommodate the new airport access road. One long culvert structure is proposed on South Branch Rock Creek beneath the proposed taxiway and runway.

The impacts were evaluated using the hydrologic and hydraulic models developed as part of the overall floodplain study. Preliminary structure parameters (shape, material, size, length, inverts, and slope) were determined such that the new structures would create no net increase in water surface elevation beyond the project property boundaries. Additionally, any lost floodplain storage volume (identified as floodplain fill) was calculated to determine approximate compensatory storage volumes that will be required.

The results of this analysis indicate that floodplain impacts associated with the proposed ALP can be managed and mitigated within the current planned airport property boundaries.

A formal submittal to FEMA of the floodplain modeling and mapping for incorporation into the NFIP will not be made at this time. The boundaries and flood profiles presented in this report have been performed in accordance with accepted industry practices and have been developed to prepare existing condition floodplain mapping and analyze the potential impacts of infrastructure associated with the airport on those floodplains. Their development should reduce the future level of effort necessary to complete a detailed study, should a need arise to submit detailed modeling and mapping to FEMA for formal incorporation into the Flood Insurance Rate Maps (FIRM) for Will County.