Appendix G
Modeling Logs

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. Typically, a 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 Route 15 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. These values are consistent with recommendations in the HEC-RAS Hydraulic Reference Manual.

1 – Rock Creek Upstream
A Manning's n-value of 0.045 was used for the channel and a 0.045 to 0.10 value was used for the overbanks. Areas that were heavily wooded were assigned a value of 0.10. The 0.045 value was chosen for the channel to represent a winding channel with some stones and overgrowth. The 0.045 value in the overbanks was chosen to represent cultivated areas such as farm fields.

2 – Rock Creek Downstream

Similar to Rock Creek Upstream, Manning's n-values of 0.035 to 0.10 were used for Rock Creek Downstream. Rock Creek Downstream has very many of the same features as Rock Creek Upstream. The channel meanders and there is some overgrowth. The overbanks consist of farm fields and woody areas.

3 – Tributary A

Tributary A has Manning's n-values ranging from 0.035 to 0.010. On the upstream end of Tributary A and the downstream end of Tributary A, Manning's n-values of 0.035 were chosen for the channel to represent a non-winding channel with some weeds and stones. The overbanks were represented with Manning's n-values from 0.045-0.10. The Manning's n-value of 0.045 represents farm fields, while 0.05 represents the grassy waterways within the farm fields. The 0.10 Manning's n-value represents dense brush and timber.

4 –Tributary B

The Manning's n-values for this tributary were set from 0.035 to 0.040 for the channel and 0.050 to 0.10 for the overbanks. The Manning's n-values of 0.035 to 0.040 were used to represent a winding channel with some stones and minimal overgrowth. The values of 0.050 to 0.10 for the overbanks represent row crops and heavy timber areas with dense brush.

5 –Tributary C

The Manning' n-values for Tributary C were set between 0.035 and 0.10. Like the other reaches, the 0.035 Manning's n-value in the channel represents a winding channel, with some stones and minimal overgrowth. The 0.050 value in the overbank represents row crops. A value of 0.10 was chosen to represent dense brush and heavy timber areas.

02S2021 Model Log
AJM

8/31/2010

1) The HEC-HMS model was originally started in version 3.1 two years ago. It was saved out and opened using version 3.3. It was then opened in ***Version 4.3 (This is the version to be used based on project scope).*** Meteorologic models and precipitation gages were added based on ISWS Bulletin 70 (Project scope called for the storm event precipitation amounts to be based on Table 13). Quartile distributions were used based on ISWS Circular 173.

9/1/2010

1) ~~Monee Reservoir was added as a Reservoir to the model. Elevation-Storage and Elevation-Discharge curves were added based on data from the plans that was entered into a spreadsheet.~~
    a) ~~The elevation-discharge table is based on the morning glory outlet structure.~~
    b) ~~The elevation-storage table is based on numbers read from the diagrams on the plans and entered into an excel table.~~
2) Geo-RAS project was started. It is called "GeoRAS_RockCreek".

9/3/2010

1) Centerline, banks, cross sections and bridge shapefiles were all created in the GeoRAS database. The LiDAR terrain (G:\GIS\State\Illinois\County\Will\LiDAR\3D_Data.mdb) was brought in to make sure cross sections were made long enough.
2) LiDAR terrain was compared to the Will County Contours (G:\GIS\State\Illinois\County\Will\Contours\WillCountyContoursshp.shp) at 2-ft intervals to make sure data is all correct. They matched so terrain is being used to extend out cross sections.
3) Preliminary firm flood maps were found on the illinoisfloodplainmaps.org (for Will County) and were downloaded into the references directory.

9/8/2010

1) Manning's n-value shapes were drawn in GIS using Geo-RAS. The n values were assigned based on Chapter 3 of the HEC-RAS Hydraulic Reference Manual. (THESE WERE NOT USED LATER ON)
    a) Residential structures are not listed in the table of manning's n values, so a value of 0.04 was used.
        i) .035-pasture-high grass (floodplain)
        ii) .03-cultivated, no crop areas
        iii) .025-pavement
        iv) .035-channel w/none to very few trees
        v) .045-channel with thick brush on either side
        vi) .05-dense trees

9/10/2010

1) Landuse shapefile provided by AECOM was compared to ortho photo for accuracy.
    a) The landuse shapefile date is **2009**-08-19

b) The ortho photo date is **2007**-11. A more recent ortho photo dated 2008 was compared the 2007 one and is the same, so the ortho used for the project is the 2008 ortho from the ArcGIS Server.

c) The missing CN's were replaced in the "LandUse_Soils_Union" based on an email from Jaren dated 2010-09-08. A spreadsheet included in the email gave CN based on Landuse and soil types.

10/13/2010

1) Subbasins were edited based on QA by Brian Wozniak.

10/14/2010

1) Subbasins were finalized and Time of Concentrations were sent for QA/QC. (TimeofConcentration_02.mxd)
2) Curve numbers were established for finalized subbasins using LandCover_Soils.mxd project.
   a) Curve numbers were developed based on land use. The "LandUse_Soils_Combo" was intersected with "PrelimSubbasins20100929" to create "Subbasin_LandUse_Soils_Intersect". The CN's were multiplied by the areas (in acres) to produce the CN_Area.
   b) A summary table was produced based on subbasin name and CN_Area (acres) and subbasin areas (in acres)
   c) An excel spreadsheet was developed to calculate the CN for each subbasin and is called "CN_02.xlsx".
3) Subbasin shapefile was brought into HMS and all the subbasins were named and began to be linked together.

10/18/2010

1) Subbasins were edited according to the 2-ft Will County Contours. Time of concentrations were delineated based on the Will County 2-ft Contours also. The project is called "TimeofConcentration02.mxd".

10/20/2010

1) The same steps were redone as completed on 10/14. Subbasins were edited to match 2-ft Will County Contours. The shapefile that now has the subbasins with the CNs is called "Subbasins_CN" and is located in the hydrology section of the geodatabase.
   a) The excel spreadsheet with the CNs is called "CN_00.xls" or .xlsx. it has to be in the .xls format to read in GIS. Once the CN's were computed in excel, the spreadsheet was joined with the subbasin shapefile to attach the CN's to the subbasin according to subbasin name.

10/26/2010

1) Complex rating curve was developed using a spreadsheet called "Spillway Rating Curve" to represent the complexity of the outlet structure of Monee Reservoir. The elevation on the plans was stated as NGVD. All elevations taken from the plans were converted into NAVD.
   a) The morning glory part of the outlet structure was input as a weir, with the length representing the outer circumference of the structure.

b) The information entered under orifice control was used as the inside diameter of the pipe (running underneath the railroad embankment) and the upstream inlet invert (taken from the base of the 90 degree bend).

c) The length of the culvert used for outlet control is from the base of the 90 degree bend to the 90 degree bend on the exit of the pipe. An n value of 0.015 was chosen based on the Fundamentals of Hydraulic Engineering pg 140 stating pipe material of wrought iron/ commercial steel is from 0.012-0.017. An entrance loss of 0.04 was chosen based on the Handbook of Hydraulics reference (pg 6-21).

   i) The 90 degrees short radius elbow was addressed as a minor loss and entered into the spreadsheet under K_{b1-2} (90 deg short radius elbow), having a coefficient of 0.9 (from the Fundamentals of Hydraulic Engineering pg 144).

   ii) The 90 degrees long radius elbow was addressed as a minor loss and entered into the spreadsheet under K_{b2-3}(90 deg long radius elbow), having a coefficient of 0.6 (from the Fundamentals of Hydraulic Engineering pg 144).

2) The datum conversion was calculated using the normal pool elevation called out on the plans of 743.7-ft. The latitude and longitude used was given in Bing Maps as the location of Monee Reservoir. The difference between NGVD 29 and NAVD 88 is NAVD 88 is 0.472-ft higher. This is documented in the references directory and is called "Datum Conversion".

10/27/2010

1) Different TW scenarios were calculated in the Spillway Rating Curve spreadsheet. A TW elevation of 728, a TW elevation of 740 and inlet control were the three scenarios computed in the Spillway Rating Curve spreadsheet.

2) The length of the pipe without the 90 degree bend near the outlet was calculated from drawing 198D-012 showing 9-12.25-ft and 3-8-ft sections of pipe computing to 134.25-ft of pipe. Including the 90 degree bend (just before the outlet), the total pipe length is 144.5-ft

11/01/2010

1) Survey data was imported into GIS for structures that were not surveyed the first time around. The data was edited in the excel spreadsheet to make it readable for GIS. A descrip1, 2, and 3 columns were added along with a culvert size column. Many of the point codes had multiple point codes instead of just one and they included a dash. These were the ones that had to be separated, along with the ones that included double quotes for culvert sizes (to mean inches).

11/02/2010

1) Spillway rating curve was modified to have a different weir coefficient for the orifice flow based on the created head (H_o). This calculation showed the discharges were higher than for the as-built plans.

2) Edits were made to the subbasins which will result in changes in the Time of Concentrations and the CN calculations.

~~3) For the Regression Equation calcs a shapefile was created that contained only the large subbasin and one each for the wetlands and the water within the large subbasin. This was used to calculated the percentage of water. Will County is considered region 2 based off the Hydro Region & ID figure. The drainage area based on the large subbasin is 12 square miles. This excludes the basin at the very south end because the discharge of interest is the one at the outlet of Subbasin M-1.~~

1/11/2011

1) CN's were developed for each subbasin based on the edits made to subbasins from the field visit performed in November 2010. The CN's were then entered into HMS. The file including the subbasin areas and the CN's is called "LandUse_Soils_Subbasins" and is located in the geodatabase under hydrology in the features directory of the project.
2) The final time of concentration shapefile is called "TimeofConcentration" and is located in the geodatabase under hydrology in the features directory of the project.

1/12/2011

1) A mxd file was created called "MoneeReservoirHydrologyModeling.mxd". This mxd has shapefiles in it for documenting the normal pool elevations area for the reservoir and the two sediment basins.
2) The HMS model was updated to contain a diversion tool downstream of watershed W-01.

1/13/2011

1) The mxd file "GeoRAS_RockCreek" was broken out into six (6) mxd projects based on the layout of the streams. The terrain had to be broken into 6 pieces because one piece was too large for GIS to process. The six pieces are Trib A, Trib B, Trib C, Trib D, Main US and Main DS.
    a. After the mxd files were created a Geo-RAS project was started for each trib. Survey bounding polygons were drawn around the ground survey (does not include any structural points besides CL of pavement, edge of pavement, and top of rail shots). An elevation was given to the polygon so that when the tin was created and the information was exported to RAS it was clear on the XS which points were survey.

1/14/2011

1) Survey tin was created for "GeoRAS_RockCreek_MainUS". It is called survey_tin and is located I:\02jobs\02S2021\CADD\GIS\Features\RockCreek_MainUS. The Geo-RAS files were created and the processes were run. The only shapefile not created was the ineffective areas shapefile.
2) A survey breaklines file was created for MainDS and added to the "GeoRAS_RockCreek_MainDS" mxd file.

1/24/2011

1) The HEC-HMS model is now a Version 3.4 model. The diversion flow tool was added into the model and a rating curve was developed for it. The culvert was modeled as a rectangular orifice followed by a weir and the downstream channel was modeled as a weir. The spreadsheet is called RatingDiversion_HMS.xls and is located in the calcs directory of the project. In the model the diversion tool asks for the max volume and max flow. These values were not entered because if entered, the diversion flow will be computed first without any limitations then reduced as necessary to meet the option requirements.

1/28/2011

1) Lidar and survey geometries were imported into RAS for the Main US and Main DS reaches. On the Main DS reach the channel centerline was edited between cross sections 8805 and 8714 to accommodate the survey data points showing the centerline of the stream.

2/04/2011

1) HMS MODEL: Monee Reservoir was modeled using an outflow curve to represent the morning glory outlet structure and the culvert outlet structure. The rating curve was developed based on elevation. The chart entered into HMS was the storage-discharge curve. The storage information, in ac-ft, came from the "Monee Reservoir Conditions" spreadsheet. The storage was developed based on the existing and proposed plan sheets for Monee. The values past this curve were then interpolated using the elevations that were developed for the discharge curve. The 'Morning Glory Structure Rating Curve" spreadsheet on the NoTW tab has two columns for the HEC-HMS Input and this is what was input into the Storage-Discharge_Monee table in the HMS model.
   a. At this point the other tables in HMS for Monee are not needed. They have not been deleted in case they will be needed in the future.
   b. Cross sections were cut for each of the routing reach locations using the tin in GIS. The data was used to develop 8-pt sections for the Muskingum-Cunge routing method. The cross section data is located in the "XS_Reaches_HMS" spreadsheet.

2/10-2/16/2011

1) HEC-RAS MODEL: Lidar and survey geometries were merged for the Main US and DS reaches. A spreadsheet was created for each of the reaches to determine the slope of the reach based on the surveyed cross sections along the reaches. The spreadsheets are located in the calcs directory and are labeled "***Main-DS_SlopeEstimations.xlsx***" and "***Main-US_SlopeEstimations.xlsx***".
   a. In the Main US reach model, XS 5684 was deleted and XS 5755 was renamed to 5780. The two cross sections that were altered where immediately located on the upstream and downstream face of the double box culvert.
      i. For the box culvert an entrance loss of 0.5 was chosen to represent headwall or headwall and wingwalls square edge (pg 6-26 reference manual).
      ii. The manning's n value was chose as 0.11 for concrete culvert, straight and free of debris (pg 6-24 reference manual).
   b. XS 1868 was renamed to 1803 so it would be moved down from the DS face of the culvert.
   c. XS 2463 was added downstream of the pedestrian bridge so that there would be 2 XS downstream of the bridge. Another XS is going to need to be added upstream of the bridge so that there is 2 XS upstream. The latest geometry is ***Existing_MainUS_02.***

2/17/2011

1) HEC-RAS MODEL – MAIN DS: Manning's n-values were chosen as such:
   a. Farm land/farm field: 0.04
   b. Wooded/Trees: 0.06
   c. Channel: 0.035
   d. Residential/Farm: 0.04

2) The bridges at stations 9003 & 8859 (on the US end of Main DS reach) only contain 3 cross sections inbetween. According to the HEC-RAS manual, each structure needs two cross sections upstream and downstream of it. Per this, an additional cross section was added inbetween these two structures.
   i. This export was called "LidarExport_USXS_20110118"
3) HEC-RAS MODEL – MAIN US:
   a. At the double box culverts location on Main-02 Reach there is only one XS on the upstream side and the DS XS are located approximately 350 feet away.
      i. Another cross section was added on the upstream side of the structure and another cross section was added on the downstream side of the structure approximately 25 feet away.
      ii. The channel of XS 5780 was copied from XS 5832 (See GeoRAS_MainUS.mxd file) and the bottom elevation was adjusted to match the survey elevation at XS 5780. XS 5684 was deleted because it was too close to the downstream face of the box culverts.
      iii. **XS 600** goes through the sediment basin and was coded manually based off the Lidar data and the normal pool elevation from the plans for the sediment basin. The normal pool elevation was adjusted on the plans from NGVD 29 datum to NAVD 88 datum.
   b. Manning's n values were chosen as such:
      i. Farm land/farm field: 0.04
      ii. Wooded/Trees: 0.06
      iii. Channel: 0.035 or 0.045 (if it has some weeds and stones)
      iv. Grassy areas with no brush: 0.35 (used around Monee Reservoir mostly)
         1. For XS 3537-169 this manning's n-value was used for the channel because based on the pictures and site visit there were little to no rocks and/or weeds within the banks.
   c. The pedestrian bridge on Main-01 was coded using the bridge design feature. It was not coded shown as an arc as the survey drawing shows. The survey information provided did not allow for the arc to be coded properly. Additional XS were added upstream and downstream of the bridge so that it would be modeled properly.

<u>2/21/2011</u>

1) Ineffective Flow Areas were developed upstream and downstream of the bridges at 1:1 and 2:1 ratios respectively. For the DS Main Reach, a shapefile called "MainDS_IFConstructLines" was created and contains the 1:1 and 2:1 construction lines, used to develop the IF areas.
   a. At structure R3 and R4 the ineffectives were drawn along the channel showing no expansion and contraction inbetween structures because the distance between is only approximately 60-feet.

<u>3/01/2011</u>

1) The monee reservoir outlet was not modeled in RAS. Instead, it was modeled in HMS with a rating curve that incorporates the weir and orifice flow at that location.

a. A Model was developed just showing the reach downstream of the outlet at Monee. It is called "MoneeOutletRating". It was developed in HMS and includes 8 XS. 4 of the XS represent the rectangular channel immediately downstream of the culvert. The next 4 XS represent the channel after the rectangular concrete channel before the confluence of the stream from the north.

2) Ineffective Flow Areas were developed on the Main US Reaches as well. 1:1 and 2:1 contraction and expansion ratios were used.

3/02/2011: Summary

1) Sediment Basin A, Sediment Basin B and the ponding area just north of the pedestrian bridge on Monee were added to the model. The elevation area curve entered for Monee had to be altered to not include the three areas just added to the model. The three areas were then given their own elevation-area curve.
   a. The sediment basins needed to be added to accommodate the diversion in the model. The sediment basins were modeled to have a weir as their outflow (dam top).
   b. The storage area just north of the pedestrian bridge was named US North Monee and was modeled to have a spillway and dam top as the outflow. The actual channel invert on the upstream side of the pedestrian bridge is 742.79. Due to where the rating curve started and the way HMS calculates the outflow, the discharge elevation was set at 744 for the spillway outflow.
      i. Since the pedestrian bridge is not square to make a correct shape weir, a rating had to be developed for the US North Monee Reservoir. HEC-RAS was used with a range of flows to develop a rating curve for the US North Reservoir.
2) The rating curve is being developed for Monee Reservoir. The discharge that is output from HMS is inputed into the RAS model and ran. The elevation from that discharge at XS 136 (located immediately downstream of pipe) is then input into the Morning Glory rating spreadsheet. The curve is input into the HMS model under the Monee Reservoir table for Elevation-Discharge and the HMS model is run. When the discharge at Junction 28 and the elevation at XS 136 are equivalent, the models are considered balanced.

3/04/2011

1) XS 5780 and 5790 (us face of culvert on reach Main_04) were extended out to see if that would help balance out the model at the upstream end of the culverts. It did not make a difference in the model stability or the water surface elevations for the storm event frequencies, so the model was reverted back to the original geometry. The geometry file that contains the extended XS is called "ExistCond_MoneOutRating_ExtendXS.g20".

3/16/2011

HMS Model:

Due to the hydrologic complexity of Monee Reservoir, extra modeling efforts were needed when including this in the HMS model. Upstream of Monee Reservoir there are two sediment basins followed by a storage area located just upstream of the pedestrian bridge. These three areas were broken out into individual reservoirs (storage areas) and an elevation-area curve was developed for each one. GIS was used to develop the rating curve along with information from the Patrick Engineering Plans. Sediment Basin A and B were each modeled to have a dam overtopping and a spillway. The dam top elevation was based off the contours in GIS and the spillway elevation was based off the Patrick Engineering Plans and adjusted +0.43-ft. The length of each of these was measured in GIS.

Monee Reservoir was modeled as a reservoir with a storage curve (representing the elevation-area) and also has two dam overtoppings, a spillway and a pump. One of the dam overtops represents flow that could possibly go out the south end of the reservoir. It is represented by a height of 749-ft with a length of 153-ft The other dam overtop represents the embankment along the railroad tracks and is set at elevation 750-ft with a length of 215-ft. Any length longer than that and the model produces a storage area error. The spillway is modeled as an auxillary spillway that shows there is more than one outlet for the Reservoir. The auxillary spillway represents the water draining south to a ditch along the railroad tracks and then eventually meeting up with the main channel of Rock Creek. The last element modeled under Monee Reservoir is the pump apparatus. It is used to control the elevation of when water will start flowing out of the reservoir.

A tailwater curve was set for the outlet at the reservoir based on the HEC-HMS User's Manual. Within the model there are five different options for the tailwater condition. The one chosen for this case is the "downstream of main discharge". This method is chosen when reservoirs represent an interior pond or pump station, and the outflow from the reservoir will be a significant impact on the downstream stage. In this instance there is a junction just downstream of the reservoir outlet that combines the outflow from the reservoir with the flow that is traveling down the channel from the north. The elevation-discharge curve for this instance was based off of a range of flows input in the HEC-RAS model. Flows and elevations at a cross section just upstream of the junction (double box culverts) were used to build a curve, which was then input into the HMS model. The curve is labeled "DSChannelMoneeReservoir".

The significance of having this curve in the model is that it outputs what the tailwater elevation is at the outlet of the reservoir. This can then be used to balance the model between the outflow curve developed in the excel spreadsheet and the HMS model. The RAS model is no longer needed for this step.

3/17/2011

~~The auxiliary spillway was changed to a dam top so that it could be modeled as non-level overflow with a representative cross section.~~ There is a total of 3 dam tops for Monee Reservoir. Two represent the reservoir drainage to the south and have a cross section, and the third represents the railroad embankment along the east side of the reservoir.

3/23/2011: SUMMARY

After several modeling techniques were tried for Monee Reservoir, the final one was that the outflow structure was modeled as a specified spillway and there were three dam tops (as described above). Using the TW method described in the 3/16 entry, an outlet curve for the spillway was developed for each duration and frequency. The spreadsheet is called: Monee Reservoir Morning Glory rating curve_QAQC_Final.xls and is located in the calcs directory under the project. It also has a tab that identifies the critical duration for each frequency.

3/29/2011: HMS Model

Since the flows being computed in HMS are relatively higher than StreamStats, an optimization trial was run in HMS based on CN's. Previous discussions have brought up the fact that the CN's are too high. In order to do this, a discharge gage has to be in the model for observed flow. Since no gage data is available for discharges, the outflow discharge from the Monee Storm Event simulation run was used

4/04/2011: HMS Model

After the optimization trial was run, it concluded an unreasonable answer for lowering the curve numbers. Based on an email from AECOM on March 30, it was recommended to increase the Clark storage coefficients by 25%. The original computed storage coefficients were increased by 25% and input into the model. The optimization trial was re-run to optimize the curve number values, but with 25% higher Clark storage coefficients. An unreasonable answer was produced once again. The optimization trial solution was considered to be a non-'vaild' method for the solution to getting lower flows, so it was ruled out.

Precipitation data was obtained from three different sources to document the Aug 2007 storm and to calibrate the HMS model. It was obtained from a gage at Monee, a gage at Midway and the NEXRAD data for Aug 19-23, 2007. The Monee gage reported 9.88-inches over the 4 day period, while the Midway gage reported 5.82-inches and the NEXRAD data reported 5.9-inches. The rainfall data based on the Monee gage is located here: I:\02jobs\02S2021\Admin\3160-H&H\10-ReferenceData\13-FloodplainStudy\Precip_Data\MoneeReservoirPrecipDataFromMCC.xlsx. The data for the Midway gage is located here: I:\02jobs\02S2021\Admin\3160-H&H\10-ReferenceData\13-FloodplainStudy\Precip_Data\Chicago Midway Airport August 2007 hourly data.xlsx.

The NEXRAD data was merged together to create a shapefile for the Aug 19-23 storm and the Aug 19-26 storm. The 5 day storm shapefile is called: NEXRAD_5daystorm_Aug19_23 and is located in the project directory under features in the NEXRAD folder. The 8-day storm shapefile is called: NEXRAD_8daystorm_Aug19_26 and is located in the project directory under features.

4/06-07/2011: HMS Model

The curve numbers were lowered based on all the soils assumed to be well drained. If a soil type of A/C was in the dataset, the letter that represented the well drained soil was chosen.

Since the storm event from the Midway gage yielded a precip total of 0-inches on one of the days (Aug 21) a hunt was on to get an additional data set to compare the precip data to. Hourly nexrad data was found on the NOAA website, but it was very laborious to get the precip total for each day from that data.

The solution was to not use that data and that each storm event will have the same distribution as the Midway gage, but just a different total rainfall amount. The rainfall amount of 9.88 inches was input into the HMS model using the hourly distribution from the Midway gage.

The results showed that the rainfall from the Midway gage resulted in a 1.5-ft lower high water mark in the reservoir and the Monee rainfall amount resulted in 1-ft higher than the high water mark in the reservoir.

Manipulations to the model were done to see what it took to hit the high water mark of 746.5-ft. When the 25%, 50%, 75% and 100% greater clark storage coefficient numbers were input into the model with the original curve numbers and ran for the Midway gage, this did not reach our high water mark. Such an extreme storage coefficient was needed for each subbasin in order to reach the HWM, effort was stopped and the 25% greater clark storage coefficient was input into the model and ran. These in reality will be the flows that we go forward with and the ones that are used to map the floodplain as deliverables to the client for the project.

At this point the TW is not calibrated to match the downstream condition and needs to be done based on the 25% greater clark coefficients. At this point the peak elevation is 747.5 which is 1-ft higher than the high water mark at Monee for the August 2007 storm. The HWM is 746.5-ft.

4/12/2011: HEC-RAS -- TRIB A

Imported the LiDAR and the survey data into HEC-RAS. The two were merged together. A spreadsheet was used to determine the channel slope and what elevation was to be used when merging the surveyed channel section with the LiDAR data. The spreadsheet is located in the RAS project directory. For XS 4344 on Trib A, an interpolated LiDAR elevation was used to base the channel slope and the elevation to be met for the survey data because the LiDAR original elevation was much higher and did not follow the trend of the rest of the LiDAR data on the other cross sections.

XS 3170 contains a detention pond. A manning's n-value of 0.03 was used for the detention pond limits on the XS. Everywhere else, a manning's n-value of 0.035 was used for the channel and 0.04 was used for field/row crops.

The upstream side of the culvert was not surveyed under I-57 due to private property issues so an estimated upstream invert was chosen, based on channel profile and terrain.

4/13/2011: HEC-RAS -- TRIB A

Based on surrounding terrain and the model profile an upstream invert of 729.4-ft was chosen (set at 0.5% slope). When running the RAS model as subcritical flow, a warning message appears in the culvert table that says the flow in the culvert is entirely supercritical so mixed flow should be run to check if the cross section downstream of the culvert has supercritical flow. The model was then run as mixed flow and the cross section downstream of the culvert could not converge to a supercritical answer. For this reason the flow was left as subcritical.

No survey data was provided for the culvert under Offner Rd (due to private property issues), so this was not modeled. This may need to be modeled in the future if it is decided it is a restrictive element in the flow to the culvert under I-57. From the pictures it looks like a fairly large concrete culvert.

4/27/2011: HYDROLOGY

Trib A1-02 was divided where Trib A2 ties in with Trib A1. The same curve number was used for the new subbasin, A1-3 as is for A1-2, since it is the same subbasin just divided. Subbasin A1-3 has a different Tc than subbasin A1-2. A couple junctions were added: One at the downstream end of Trib C and one on Main DS (in the HMS model). The new junctions are called Junction 28 and Junction 30.

The 100-yr 01-hr storm event, with a balance TW, produces an elevation at Monee Reservoir of 746.7-ft, which is 0.2-ft higher than the HWM of 746.5-ft.

5/10/2011: HYDRAULICS

Most of the tribs are defaulting to critical depth in the computations. Main DS is defaulting to critical flow (FR = 1) at XS 8822. When putting in interpolated sections the problem was not fixed.

In Trib A there are several locations where it defaults to critical depth during the computation. The current geometry is Existing_Revised_03 (g14). Geometry was saved as Existing_Revised_04. In this geometry XS 4344 was removed because the lidar section was picking up the roadway elevation with no channel causing the channel to be very deep. This did not solve the critical depth issues.

Main US was going to critical depth at XS 4302. By increasing the ineffective flow area on the DS XS and changing the contraction and expansion coefficients to 0.3 and 0.5 on XS 4302 the critical depth issues subsided.

Trib D critical depth issues were corrected by reanalyzing the merged cross sections with the geometry data and by making the n-values in the channel 0.040 instead of 0.035.

Main DS was defaulting to critical depth just downstream of the railroad bridge. When the manning's n-values were raised from 0.035 to 0.045 the critical depth issues were resolved. It takes into account the channel roughness including rocks and other debris, along with the severity of the bends in the channel.

May 24, 2011

Trib A critical depth issues were resolved. After carefully assessing the channel geometry and how the survey channels were merged with the lidar sections, the critical depth issues subsided. A channel n-value of 0.045 was used in areas where it was more wooded (and to help get rid of the critical depth answer). The reach lengths at the junction were edited to represent the distance between the cross sections entering into the main reach where flow occurs. The final geometry is g17, final plan is p12 and f02 (Exist_05_100yr-03hr). The reach boundary conditions were entered as normal depth with the slope being measured from the most downstream cross section to the most upstream cross section on the profile plot.
the flows from hms have been entered in the flow data and used in the most recent run

Trib B has critical depth issues on Trib B2. Trib B2 is not within the mapping limits though, so the critical depth issues are not going to be addressed. On the main Trib B, the manning's n values were set to 0.045 in the channel and the overbanks. The channel consists of heavy brush and overgrown, so 0.045 was selected. The overbanks consist of farm fields (crops), so the n-value chosen for that was 0.045 as well. The main trib seems to be sensitive to the normal depth boundary condition. It is currently set at 0.00292 which is the measurement from the lowest cross section to the highest cross section. The greater the slope gets, obviously the steeper the WS profile becomes on the most downstream end.

Trib C had all the critical depth issues fixed. A new model was built and is called "TribC_RockCreek_May2011". Blocked obstructions were used in the overbanks where no flow was present until a certain elevation. The manning's n-values were set to 0.045 in the channel and 0.045 where there was crops in the overbanks. Originally cross section 6827 had points coded into it that represent the channel to the east of the culvert with the culvert invert elevations. The same thing with the cross section just downstream (6739). In review, this cross section was not a correct representation of the channel at the upstream and downstream face, so cross section 6839 was copied to replace these two cross sections (6827 & 6739). The inverts were then adjusted to account for the channel slope
The final geometry is "ExistingConditions_04" (g09), the plan is "Exist_04_100yr" (p04) and "100YR-03HR_00" (f01). The normal depth boundary condition was used in the flow file and was based on the most downstream xs to the most upstream xs slope which is 0.00461. When looking at the slope just downstream of the most downstream culvert, it is 0.00472. When that is

used as the normal depth slope, the water surface elevations are not affected upstream of that and the water surface downstream of the culvert is steeper and just follows the slope of the channel more.
the flows from hms have been entered in the flow data and used in the most recent run

Trib D had all of the critical depth issues fixed. Since the slope is steep, the flow runs close to critical depth the whole time, but never reaches it. The manning's n-values in the banks were switched from 0.04 to 0.045 to be consistent with the other models. The channel n-value is set to 0.04 to account for some bending along the reach and to account for the steep slope. The normal depth boundary condition slope was set based on the most downstream cross section to XS 2346.
the flows from hms have been entered in the flow data and used in the most recent run

Main DS had a couple minute critical depth issues that were resolved. Blocked obstructions were used on the cross sections that had divided flow either going to the left or the right overbank. The normal depth boundary condition was defined using the most upstream and downstream point on the geometry profile.
the flows from hms have been entered in the flow data and used in the most recent run

Main US did not have any critical depth issues. At XS 4302 the water surface nears critical depth but does not reach it. For some reason the water surface elevation drops a significant amount at this cross section. This reason has yet to be determined.
the flows from hms have been entered in the flow data and used in the most recent run

May 27, 2011

1.) All of the tribs were joined together in a new project and a new geometry. The new project is located in the project directory under CADD/HYD/RAS and is called SouthSuburbanAirportFloodplainStudy. The first geometry where there were imported is called Merged_01. When the junctions were added the geometry is called Merged_01_Junctions.

The geometry locations and names that were used in the final join geometry

Trib A: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\Trib_A\Existing_Revised_05.g17

Trib B: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\Trib_B\ExistingConditions_03.g08

Trib C: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\Trib_C/ExistingConditions_04.g09
(Project: TribC_RockCreek_May2011)

Trib D: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\Trib_D\Existing_Merged_Revised_03.g09

Main US: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\MainUS\Existing_MainUS_04.g24

Main DS: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\MainDS\Existing_MainDS_00.g19

2.) Junctions were joined where each one of the tribs was combined. Trib B joined Main US reach where a bridge was located. The bridge needed to be located on the north side of Trib B so the bridge was moved to Main_04 reach. Cross section 2811 on the MainUS_02 reach was copied over to the Main_04 reach and made station 3007.

May 31, 2011

After all the tribs were merged together, the XS numbering on Main US reach had to be renumbered because a Junction could not be put in to combine the two together. The numbering is documented in the excel spreadsheet called "Merged_RennamedRiverStations.xlsx". After the 100 year flow was run, Trib D and Trib B2 were having critical depth issues. Trib D critical depth was solved by increasing the channel n value to 0.045 throughout the entire reach and adding interpolated XS between the 3 XS on the downstream end.

June 01, 2011

Preliminary mapping results were created in HEC-RAS using the Mapper tool. Areas to look at were defined in GIS using a polygon shapefile once the ras Mapper shapefile was brought in. Upstream in Trib B it was showing conveyed flow in the left overbank. Blocked obstructions and levees were used in the geometry 08 to show no conveyance. Cross sections with blocked obstructions already to show houses had levees placed because the program would otherwise error out.

June 02, 2011

Edits were made to the reaches with the blocked obstructions. Main US, DS and Trib B were modified.

Trib C was edited around the railroad embankment. The cross sections upstream and downstream of the railroad bridge (culvert) were a misrepresentation of the bridge. XS 1821 was deleted and re-cut to be a bit further away from the front of the bridge. As it stands now, the cross section is picking up the railroad embankment. Cross section 1720 was no longer used as the downstream face and a new cross section was cut to be the downstream face of the bridge. XS 1764 was the new downstream face and XS 1866 was the new upstream face for the railroad culvert. Cross section 2260 was not used as the downstream face of the roadway bridge. Cross section 2309 was.

Cross section 2997 was straightened out on the north end (Trib C).

June 06, 2011

XS comparison pdf's were made to compare the lidar geometry with the surveyed channel and the final cross section. In the main ds reach, cross section 8956 was cut in the lidar after the original geometry data was imported into HEC-RAS. This cross section was added so more cross sections occurred between 9003 and 8859. The "Existing_MainDS_02.g12" was used for cross section 8956. This was the first geometry that this cross section appeared in.

Main US final and Main DS final were both imported back to the individual HEC-RAS models from the final HEC-RAS project: SouthSuburbanAirport. This project is located here: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\SouthSuburbanAirport.prj.

Discharges were compared between the HEC-HMS model and the StreamStats reports. For the entire basin StreamStats calculated a drainage area of 12.9 sq miles with a report 100-year discharge of 1300-cfs. For the same square miles, HEC-HMS reports a 100-year discharge of 4600-cfs.

June 23, 2011

To reduce the overly high discharges a stage-storage method was tested on Subbasin M1-04 to see if it would reduce the discharges. Instead of using the Muskingcum Cunge reach method, this method was used and it reduced the flows by about 1000 cfs. The new flow was then within the confidence interval for the stream stats method, making it an acceptable flow. This method will now be used in other reaches where there the floodplain allows for an increased amount of storage.

July 08, 2011

The model was qced and comments are being addressed. One of the comments was to extend some of the cross sections further in their right overbank on Main US and Main DS. Some of the cross sections could be extended further to tie into high ground, but others could not (specifically XS 12082 and 12609). To address this issue a "wall" was used to extend the XS. The low point of the vertical wall was the ground and the high point was the elevation chosen to grasp the water surface elevation. The most current project is called "SSA Duplicate" in the project directory. The most current geometry is g04 (QA_QC_Geometry_Revised_03).

July 12, 2011

GeometryRevised_04 had the downstream cross section of culvert 3429 on Trib C merged in the channel with the upstream cross section to make the geometry more consistent across the culvert and to try and fix the errors at the culvert. This solution did not work because there are still errors saying the culvert and weir could not get to a balance.

July 18, 2011

Corrections are still being made based on the QC comments. For the Main US XS 10901, it was extended using the LIDAR and the geo-ras process. The geometry was imported into the Main US HEC-RAS project and then the overbanks of the XS were merged with the US Final geometry. This XS was then imported into the SSA Duplicate HEC-RAS project – g05 geometry. For the very upstream of Main US_01 XS extended them using the LiDAR did not produce high enough elevations so a wall was used on the LOB to make the elevations high enough.

July 21, 2011

Process of Flow Reduction in HMS:

Areas that were reach routed in HMS were replaced with a stage storage curve using HEC-RAS. A range of flows was modeled in RAS and the resulting storage area for each XS was used. The difference in storage areas between the XS's were computed in excel ("Elevation_Volume_HMSModeling.xlsx"). This table was then input into HMS as a stage-storage relationship replacing the specified reach.

Different curves were looked at for the spillway discharge in Monee Reservoir. When comparing the Hanson rating curve to the as-built plans developed by Patrick engineering there were some differences. Hanson used Table 3 precipitation from the Will County Guidance Manual and Patrick used the SCS Method for rainfall amounts. Patrick assumed a 40% clogged trash rack as well. When the Hanson developed rating curve was input into HMS with no tailwater elevation the stage-volume relationship appeared to match the Patrick as-built plans.

A storage area was developed off of Kuersten Rd and is called the Kuersten Rd Pond. Maximum elevations were determined by contours and pictures provided by the property owner. The shapefile with the polygons for the storage capacity is located here:
I:\02jobs\02S2021\CADD\GIS\Features\KuerstenRd_Pond.shp

1) Hanson developed curve -- 40% clogged grate w/no tailwater

Using the Hanson developed curve with a 40% blocked trashrack and no tailwater, the peak elevation is 748.3-ft and the peak outflow is 197.1-cfs. The outflow matches within approx 10 cfs of the Patrick plans showing a 40% clogged grate. The peak inflow Q (no matter what) is 1417.9-cfs (compared to Patrick at 2044-cfs). The discharge at Junction 04 is still not within the StreamStats interval for the 100-year (2317.4-cfs).

2) Patrick developed curve – from as builts w/TW (and 40% clogged)

Discharges and elevations were scaled off of the as builts to develop an outflow curve for Monee Reservoir. The curve follows the proposed morning glory inlet curve with the max Q at 232-cfs. This curve still does not reduce the discharges enough for the discharge at Junction 04 to be within the StreamStats limits. The discharge is 2339.8-cfs. At Monee Reservoir the peak elevation is 748.5 (compare to 748) and the peak discharge is 231.9 (compared to 232)

3) Hanson developed curve – 40% clogged grate w/TW at 740

The peak outflow is 121.1-cfs (compared to 232) and the peak elevation is 748.7-cfs (compared to 748). The discharge at Junction-04 is 2257.4-cfs.

4) Hanson developed curve –TW at 740 & 0% clogged

The peak elevation was 748.6-ft and the peak outflow was 128.8-cfs. The peak storage was 434.2-ft. The peak discharge at Junction 04 was 2263.8-cfs.

5) Hanson developed curve –TW at 732 & 0% clogged

The peak elevation was 748.4-ft and the peak discharge was 177.7-cfs. The discharge at Junction 04 was 2308.9-cfs.

6) Hanson developed curve –TW at 727.15 (assume pipe ½ full) & 0% clogged

The peak elevation was 748.3-ft and the peak discharge was 201.9-cfs. The discharge at Junction 04 was 2317.6-cfs.

August 18, 2011

Continuing on with the flow reduction analysis….

If the outlet curve for Monee includes a 40% clogged trash rack and a tailwater elevation at 740-ft most of the discharges will be within the 90% prediction interval for SS. The very most downstream outlet is still showing higher than the prediction interval.

August 25, 2011

A final decision was made for the tailwater rating curve for Monee Reservoir. The conclusion was to assume the 3.5-ft diameter outlet pipe (out of the reservoir) would flow full. The other condition is that the trashrack would be 40% clogged. This makes the tailwater elevation 728.9-ft. This is going to be the curve used for all frequencies of the model.

August 26, 2011

The issue with Trib C and the railroad bridge overtopping is being addressed. The reach lengths were adjusted because some of them were not correct. Geometry Revised 04 reflects these changes. Geometry Revised 05 reflects the above changes along with the 4 "short" cross sections along the channel deleted. The upstream and downstream cross sections around the railroad bridge culvert were altered within the channel to have points on either side of the culvert at approximately the culvert invert elevation. None of these prevented the flow from not overtopping the railroad.

G06 – "QA_QC_Geometry_Revised_05" has the 4 short cross sections deleted on Trib C and the upstream and downstream inverts of the cross sections on the railroad culvert have been adjusted to be just below the culvert invert.

The things tried did not work. When the model was run in supercritical flow it did not have any overtopping. It was decided to take the 4 short cross sections out and leave them out for good (Final). The reach length between the two cross sections before and after the 4 cross sections was adjusted to a distance of 30-ft.

September 6-8, 2011

Floodplain encroachments were modeled first by using method 4 and setting a target water surface elevation. When this method was no longer valid for the cross sections, method 1 was used. The encroachment stations were adjusted to account for the difference in water surface elevation to maintain a 0.10-ft difference. In some areas of the reaches, such as Trib B US, Trib A DS, and Trib D the WSEL difference is around 0.00. In most of these areas the water is contained within the channel and is not affected by the encroachments.

All tribs had encroachment runs performed besides Trib C. Encroachment runs were done on the downstream end of Trib C up to the railroad bridge. We are supposed to be getting plans from the railroad to see if there are any more culverts/structures through the embankment.

September 9, 2011

Using RAS mapper the floodway encroachments and floodplain were overlayed. The most current geometry is g11. p14 is the most recent plan.

Left to do:

Get railroad plans for Trib C
Have floodway encroachments QAed.

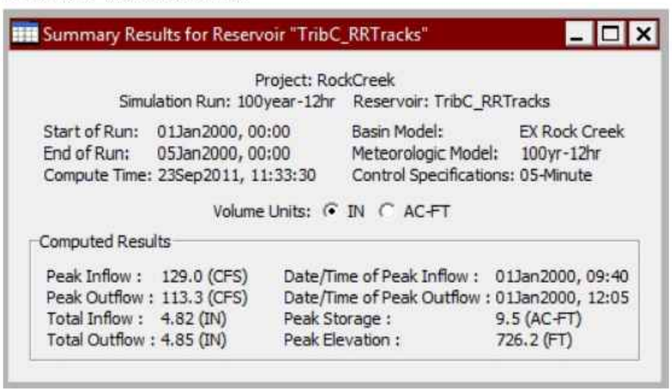
September 19, 2011

Due to the railroad embankment still overtopping (even after flow reductions have been made) a channel rating for a diversion was created. Further field observation showed no additional structures along the railroad embankment, so in order to compensate for the amount of flow needed to go through the railroad culvert, the flow that cannot be handled by the culvert will go south and through a culvert returning it to the main branch river system.

HEC-RAS was used to develop a rating curve with a range of flows and elevations to give the channel rating for the diversion table. The inflow discharge was based off the range of flows in Trib C and the divert discharge was based off the range of flows from the channel rating RAS model. The diversion outflow was then entered into the RAS model on Trib C at XS 1968. This showed the railroad embankment not overtopping.

September 23, 2011

Instead of using a diversion for the channel rating between the road and the railroad tracks on Trib C, an elevation-storage reservoir was modeled. Using the Will County 2-ft contours the area based on the contour size could be determined. The contours used for the area are located in GeoRAS_RockCreek_TribC1.mxd. By using the conic method, a volume for each of the elevations was determined.

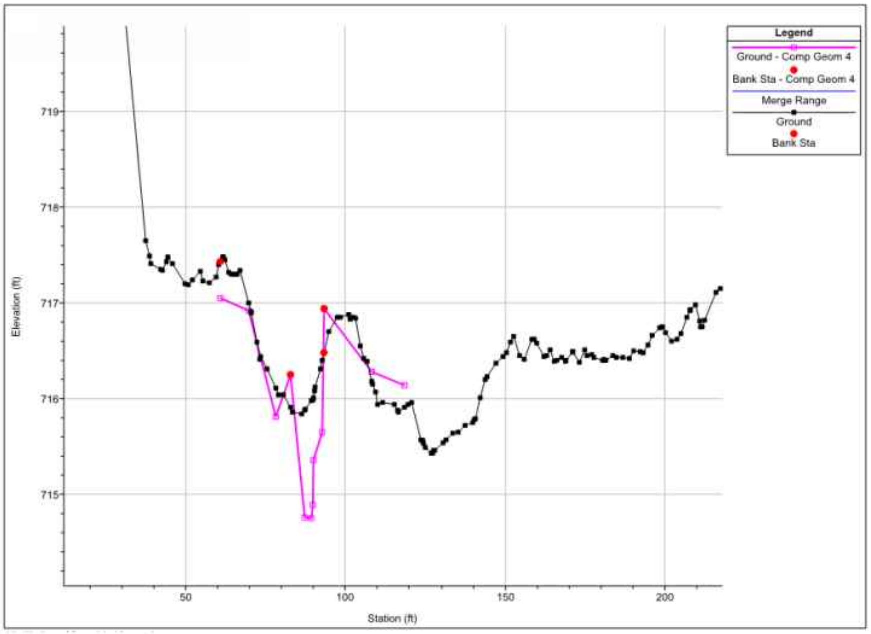


These are the results from HMS with the new storage pond. Before the discharge was 228-cfs going into the railroad culvert. This discharge is handled by the culvert without overtopping. The roadway overtops still but is within a foot of the peak elevation of this pond, therefore showing the hydrology and hydraulics coincides.

September 27, 2011

Trib C XS 1720 – to show merging of lidar and survey is correct

I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\Trib_C\XS1720_LidarvsSurvey.pdf



XS PDF locations (These may have been updated since the pdfs were made)

I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\Trib_A\XS PDFs
I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\Trib_C\XS PDFs
I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\MainDS\XS PDFs
I:\02jobs\02S2021\CADD\Hyd\Model\HEC-RAS\MainUS\XS PDFs

September 28, 2011

Tweaking was done to the storage on Trib C involving the railroad tracks overtopping. A new storage area was defined that included the storage on the cross sections upstream of the rt 50 bridge to the next upstream structure. The reservoir was modeled to have an outflow structure (the culvert underneath the tracks) and an auxillary spillway (for the flow draining to the south). The amount of flow draining to the south increased the discharge at Junction 20 from 3179.5 cfs to 3290.1 cfs. This was not a big enough increased to be concerned about.

02S2021 Model Log
AJM

September 30, 2011

The elevations and lengths of the weirs had been messed with at Monee Reservoir. The elevation was saying 800-ft and the weir length was not correct. This was fixed to show the correct elevation and weir length. This increased some of the flows along the main reach because at elevation 800 no flow was discharging into the auxillary overflow.

A file was zipped and is located here: I:\02jobs\02S2021\CADD\Hyd\Model\HEC-HMS\Ver3_5.

November 15, 2011

All the changes were made in g17.

The overlapping cross sections in the HEC-RAS model were addressed as follows:

XS that overlap
12609 – Main US 02 - right overbank location was recut on tin. The channel data was used from the old geometry (N2).
12082 – Main US 02 – right overbank location was recut on tin
8697 – Main US 02 – not going to make a difference in mapping
8220 – Main US 02 – not going to make a difference in mapping
7826 – Main US 02 – not going to make a difference in mapping
7200 – Main US 02 – could be trimmed
    Trimmed the right side of the cross section after station 1225.86
7009 – Main US 02 – could be trimmed
    Trimmed the right side of the cross section after station 1255.5
6250 – Main DS 01
    Trimmed the right side of the cross section after station 1377.97
457 – Trib A DS – could be trimmed on the right hand side so it did not cross XS 5980
    Trimmed the right side of the cross section after sta 1140.77
421 – Trib C – could be trimmed
    Trimmed the right side of the cross section after sta 744.47
119 – Trib B DS – recut using the tin. The channel data was used from the old geometry (N2).

XS that need to be extended
1632 – Main DS 03 – extended using the tin
616 – Trib B DS
    Added levee
328 – Trib B DS
    Same as XS 616
119 – Trib B DS
    Left overbank recut using tin and added levee on right side for road

November 17, 2011

Elevations and lengths of the dam overtops on Monee Reservoir were edited because they were not correct. A spreadsheet called HMSMoneeModelChanges_Documentation.xlsx documents all the changes.

SSA 02S2021 – Rock Creek Floodplain Analysis
Modeling Log (BJW)

1/25/11 ....Trib B
Created Survey TIN and LiDar TIN. Used GeoRAS to import x-sections into HEC-RAS. Filtered LiDar sections down to 500 points or less. Noted difference in mapping elevation data between LiDar generated TIN and Will County 2-ft contour mapping. Differences are 2'-4' in some locations near 7964-8477 with LiDar being the higher elev. Continue to use LiDar mapping and survey for HEC-RAS analysis, note where Will County 2-ft mapping is significantly different.

1/26/11 ....Trib B
Merged survey and LiDar x-sections where channel surveys were taken. Survey data consistently lower by 1 to 3-ft on channel bottom (as expected), and compares well elsewhere. X-sect comparisons were saved to PDF for documentation. At 814 the survey at left bank is 2-ft higher than LiDar. Photo inspections indicate that a levee/berm may exist there that the LiDar missed. The higher survey elevation (726.06) was projected outward for 15+/- ft then quickly brought down to tie back in (this is comparable to the right bank). This location should be visually inspected during next field visit. At 12042 there are actually two separate channels through this x-section (something to keep in mind during floodplain mapping). And there is a new house located near sta. 520 on the x-section that likely has considerable fill beneath it (fill in floodplain). This "new" fill does not show on the LiDar. Something to inspect during next field visit. Two existing houses are located at the two humps in the x-section. All structures located near channel were coded into RAS as obstructions and labeled either house or outbuilding.

1/27/11 ....Trib B
Set invert slope to be used in RAS based on survey data and interpolation in between. Outside of survey data (above x-section 9261) the LiDar slopes were used to project channel invert upstream from section 9261, using survey elev at 9261 (3.05' lower than LiDar. Excel spreadsheet was created to calc inverts and profiles were charted for graphical comparisons.
Started coding bridges into Existing HEC-RAS.

1/31/11 ....Trib B
Added additional survey invert data at culverts and bridges to set channel profile. Noted areas for additional survey (noted in red above). Spliced interpolated channel definition into all LiDar sections.
Resume coding bridges/culverts into Existing HEC-RAS.

2/15/11 ....HEC-HMS
Reviewed routing reach definitions. Need to extend many 8-pt sections to encompass full 100-yr floodplain width. Use HY8 or other program to estimate defined channel capacity and compare preliminary HEC-HMS peak flow rates.

2/16/11 ....Trib B
Returned to HEC-RAS definition of crossings. Found survey of each Trib B structure but searching for actual point data that covers Trib B. Found it.

2/18/11 ....Trib B
Coding crossing structures. Coded all buildings as blocked obstructions.

2/21/11 ....HEC-HMS
HEC-HMS QA/QC. Monee reservoir outlet.

2/23/11 ....HEC-HMS and Trib B
Tailwater analysis for reservoir outflow. Need to look at various scenarios to determine worse case condition. Amanda created HEC-RAS model from highway bridge to reservoir outlet. Use it to determine tailwater w/ (1) normal depth start, (2) road culvert backwater start. Also look at normal depth on concrete rectangular channel.
Added ineffective areas to model. Rotated ineffective "construction lines" with channel centerline to avoid it crossing the channel where meanders occur. Analyzed the driveway culvert ineffectives and overlap. Decided to scrap the driveway ineffectives because the major conveyance element in this reach is the left overbank and the right overbank is limited by a levee at the top of road elevation.

2/25/11 ....Trib B
Finalized model by adding structure on Trib B2, added junction, and ineffective areas. Just need final flows and starting water condition. Plan to eliminate XS48 from model. It is too far downstream and inundated by the Rock Creek floodplain. The high ground required to get above Q100 is same as high ground in XS119. Use XS119 to start Trib B. Use Rock Creek floodplain elevation to start with Known WS Elevation.

2/25/11 ....Trib B (AJM)
Additional survey for the most upstream cross section was incorporated into the model. Also the roadway data for the bridge at station 769 was coded into the model.

Monday, January 24, 2011 through Friday, January 28, 2011

1) Export of ArcGIS data into HEC-RAS format through GeoRAS for Tribs C & D. Setup work for all tributaries was completed in ArcGIS for all tributaries to show centerlines, cross sections and bridge locations on top of aerial topography. No changes to these locations were necessary. Amanda did the initial trial exports for Trib D, but these were subsequently redone for the final usage. Trib C was done entirely by me.
2) A spreadsheet was set up to estimate the slope for the HEC-RAS model based upon the LIDAR data. Using the nearest downstream surveyed cross section, upstream sections without survey data. It was discovered in Trib C that the survey crew picked up a roadway ditch instead of the assumed stream centerline cross section.
3) GIS data from Tribs C & D were imported into HEC-RAS using the standard import routine of GeoRAS data. The number of points was filtered to be no more than 500. Manning's "n" coefficients were given for the overbanks and main channel, using values of 0.045 and 0.035, respectively.
4) Using the field survey notes, the culverts were entered in the appropriate locations. Invert elevations were taken directly from the field survey. Pipe lengths were taken from measured distances between the surveyed inverts. The FHWA HDS-5 chart number & entrance loss coefficient were estimated based upon the survey notes and from photographs.
5) Sample flows were input to test the stability of the model. Final flows would be input once the HEC-HMS model is complete. There is a concern that the cross sections could be overtopped by the design flows in Trib C Section 491, 716, 1045 and 1320. In Trib D, no overtopping is anticipated.

Sunday, February 13, 2011 through Monday, February 14, 2011

1) Attempt to repeat the process of previous GIS work for Trib A. Trib A has minimal surveyed cross sections, so additional survey was requested in addition to the portion of Trib C where the wrong location was surveyed. Additional work on Trib A was put on hold, pending this survey.

Friday, March 25, 2011

1) Export of ArcGIS data into HEC-RAS format through GeoRAS for Trib C. Additional survey was picked up to be incorporated into the upstream two cross sections (6811 & 6984), as was mentioned in Comment #2 from the January 24-28 portion of the model log. Attempted automatic import did not appear to give the correct results, so the cross section data was entered manually given the procedure spelled out in the Trib C spreadsheet (Trib-C_SlopeEstimations.xls). Survey is still outstanding for Trib A, so no addition work was done there.

AJM Modeling Notes – 4/22/2011:

Merged revised channel slopes using survey data and created a new geometry in Trib C project called "Merged_survey-and-LIDAR_data_Revised". Upon inspection, it was noticed that some XS with survey were not calling out the correct bottom of channel elevation. These were revised via survey elevations in GIS project (GeoRAS_RockCreek_TribC). A new geometry for Lidar and survey was imported into the project. A new XS downstream (XS 50) was brought into the model because it has survey data and XS 6790 was renamed from XS 6811, which incorporated additional survey data.
After remerging with the new survey elevations, the most upstream culvert is reporting lower inverts that than the channel. Something needs to be decided to be done about this. The channel centerline elevation is coming in around 737 on the upstream side and the culvert invert on the U/S side is approximately 734. With no survey on the downstream side of that culvert, the channel survey slopes are used to create a channel centerline elevation on the downstream side. That elevation is reported 736 and the D/S invert of the culvert is approximately 733.

AJM Modeling Notes – 4/25/2011:
Even though RAS will allow flow to go through the culvert that is lower than the channel stream elevation, an additional cross section was added at the upstream face of the culvert using the surveyed cross section data that was taken to the east of the culvert.

Model Logs

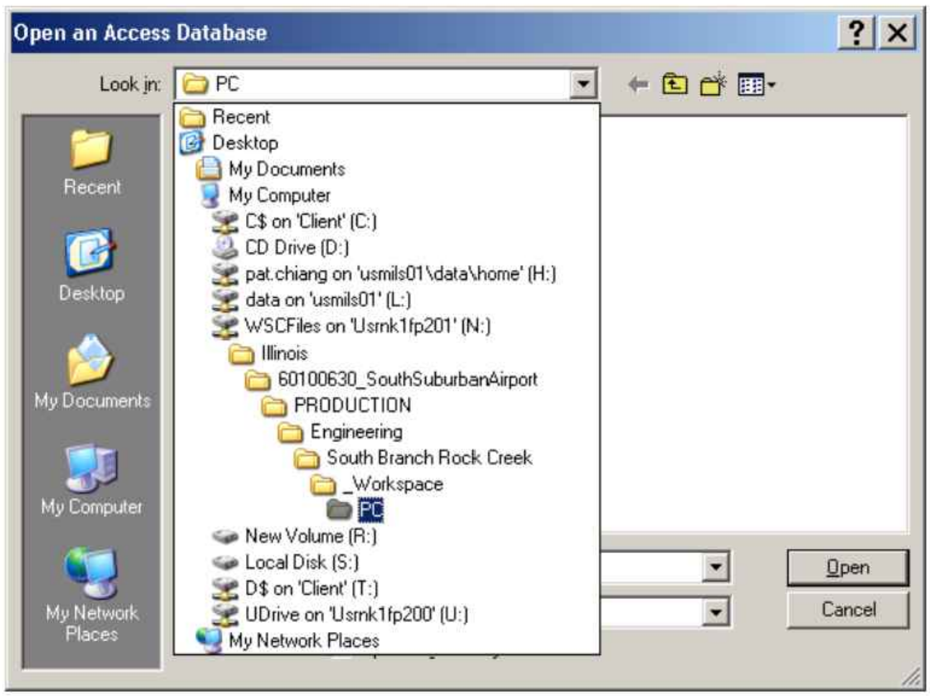
July 2, 2009

WISE training workshop exercise:

N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\Engineering\South Branch Rock Creek_Workspace\PC

- File location/working folder:

Wise project: SBR.cse



August 14, 2009

- **Contour Information**

The LiDAR version of the Will, IL South Suburban Airport:
N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\Terrain\WillCoIL_SSA_LiDAR\WillCoIL_SSA_LiDAR.ter

1. NAD_1983_StatePlane_Illinois_East_FIPS_1201_Feet
2. 20ft hydrocorrect DEMs

2ft contours were created from the LiDAR based TIN:

N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\Terrain\WISE_Contours\SSA_LiDAR

WISE format model streams were created to match the LiDAR:

N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\Terrain\Streams\SSA_Model_Streams_LiDAR.shp

August 20, 2009

- **Landuse file (2005 Will County LU reviewed and edited to 2009 Aerial Photo)**

N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\GIS\SSA_LU081909.shp

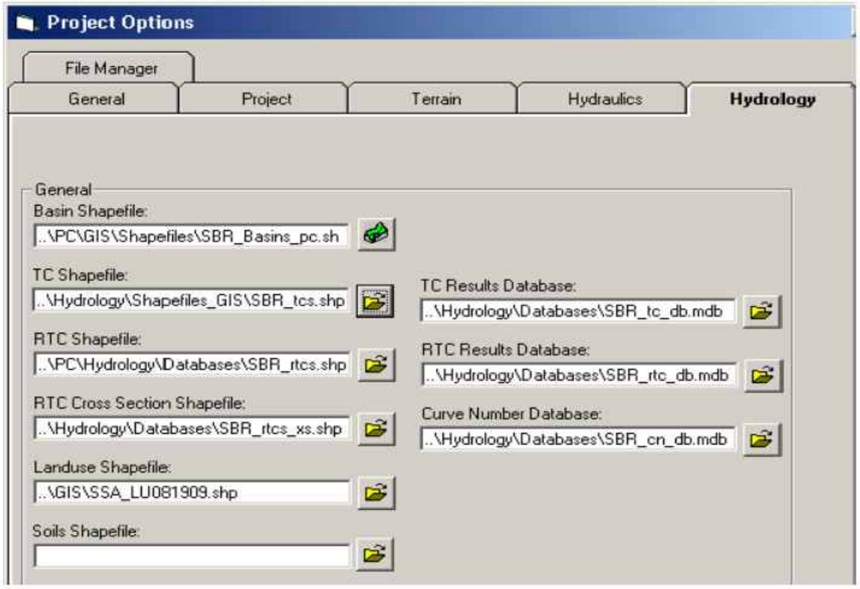
- **Soil file**

N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\GIS\Soils_Project.shp
(NAD_1983_StatePlane_Illinois_East_FIPS_1201_Feet

All the field data and Will County GIS data is on NAD_1983_StatePlane_Illinois_West_FIPS_1202 Coordinate. Need to project to East Coordinate. (8/4/2010)

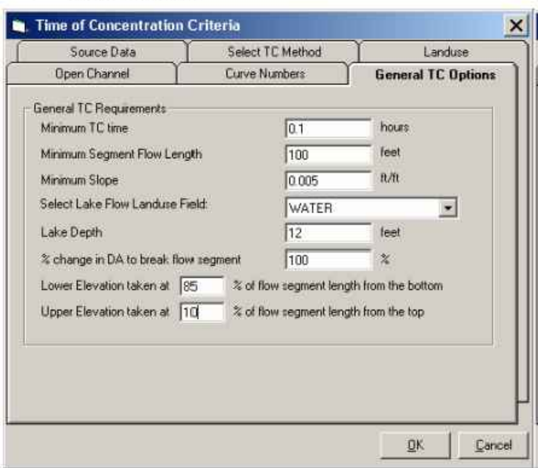
- **Hydrology:**

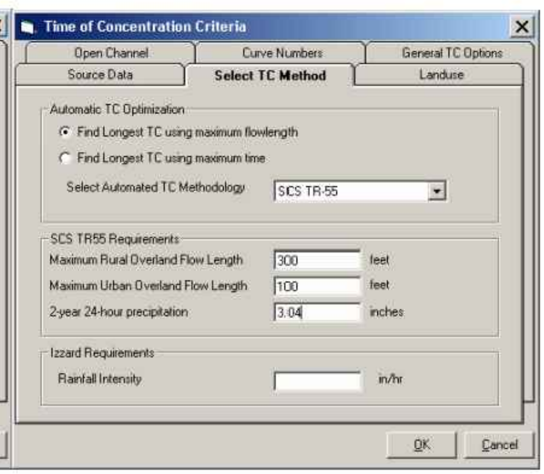
Shapefiles are under N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\Engineering\South Branch Rock Creek_Workspace\PC



Basin delineation criteria: 320 acres.

- **Tc Calculation:**





October 27, 2009

- **82-22 Method** (Melching &Marquardt, 1996):

$(TC+R) = 35.2L^{0.39} S^{-0.78}$

L is the stream length measured along the main channel from the watershed outlet to the watershed divide, in mile, and S is the main-channel slope determined from elevations at points that represent 10 and 85 percent of the distance along the channel from the watershed outlet to the watershed divide, in ft/mi.
Regional values of $R/(TC+R)$ were determined for various areas of the State (fig. 2). The regional value for Will County is 0.6.

A Technique for Estimating Time of Concentration and Storage Coefficient Values for Illinois Streams, USGS Water Resources Investigation Report, 82-22, Graf and others,1982a,b;

According to Bulletin 70 Zone 2 (Northeast) 2-year 24-hour rainfall depth is 3.04 in.

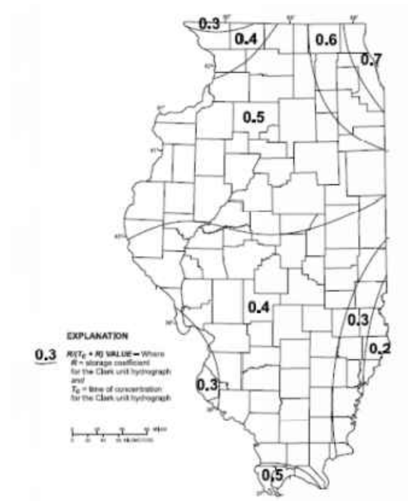


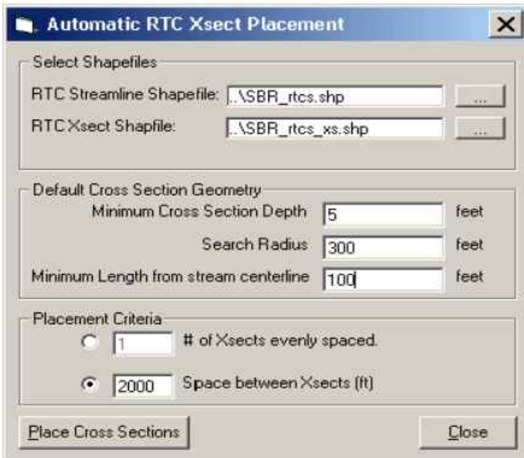
Figure 2. Regional values of $R/(T_c + R)$ determined by Graf and others (1982b) for Illinois.

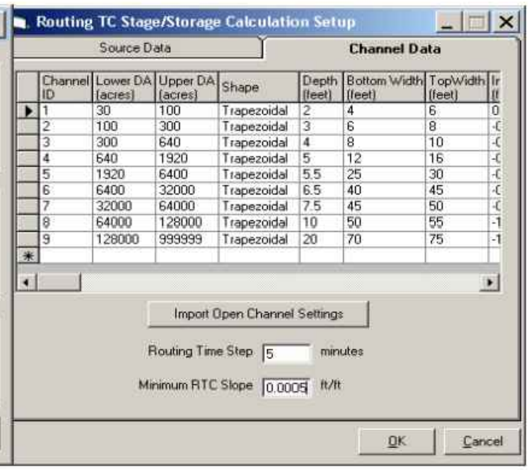
December 3, 2009

An AECOM personnel spoke to Perry Masouridis(IDOT) about applying 82-22 method on the small subbasins instead of large watershed. He could think of numerous projects where the Illinois TC+R equations had been applied to small subbasins. He was aware of the USGS work involving only large watersheds and agrees that a purist would only apply methods under conditions similar to those under which they were derived.

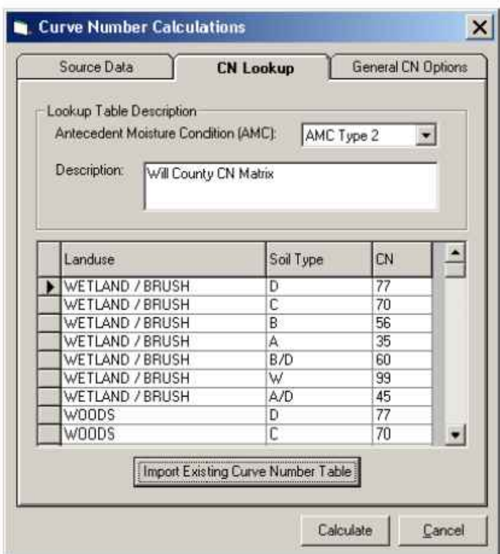
In this study, an agreement has reached to calculate L as the length of overland flow path instead of main channel length. Once L and S are calculated, $(TC+R)$ is solved using 82-22 equation (calculate outside of WISE). Using Regional values of $R/(TC+R)$ of 0.6 to calculate R.

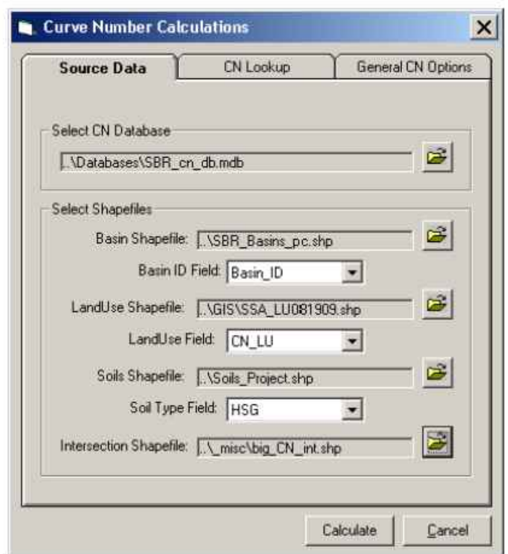
- **How to calculate 10 and 85 percent of the distance channel slope using WISE:**
  SCS simple method is not working, use TR-55 method. However, no matter what flow segment percent combinations were put in the lower and upper elevation boxes, the calculated slope length is only 5% of the total flow length. Also, in order to trick WISE to calculate the whole overland flow path as only sheet flow instead of breaking into three types of flow path, an attempt was made to set the maximum overland flow length and first several tier open channel drainage areas as large as possible. It was unsuccessful, the program still break overland flow path into three types.





It was determined to calculate the flow slope outside WISE.
Have not calculated RTC time and will use channel x-secs updated with survey data.





November 10, 2009

- **Precipitation:**

AECOM personnel contacted IDNR for the acceptable hydrologic analysis methods. The answer was below:

(from Bill Boyd) We accept any commonly accepted method within HECHMS to study watersheds which includes the SCS method. If you intend to go through the mapping process HEC1 and HEC2 should be avoided for new watershed studies. HECHMS and HECRAS should be used. The analysis should include a critical duration storm analysis and should use Bulletin 70 rainfall amounts and Huff distributions. Please stick with the 1st, 2nd, 3rd and 4th Huff distributions in Bulletin 70 and the appropriate rainfall amounts.

August 6, 2010

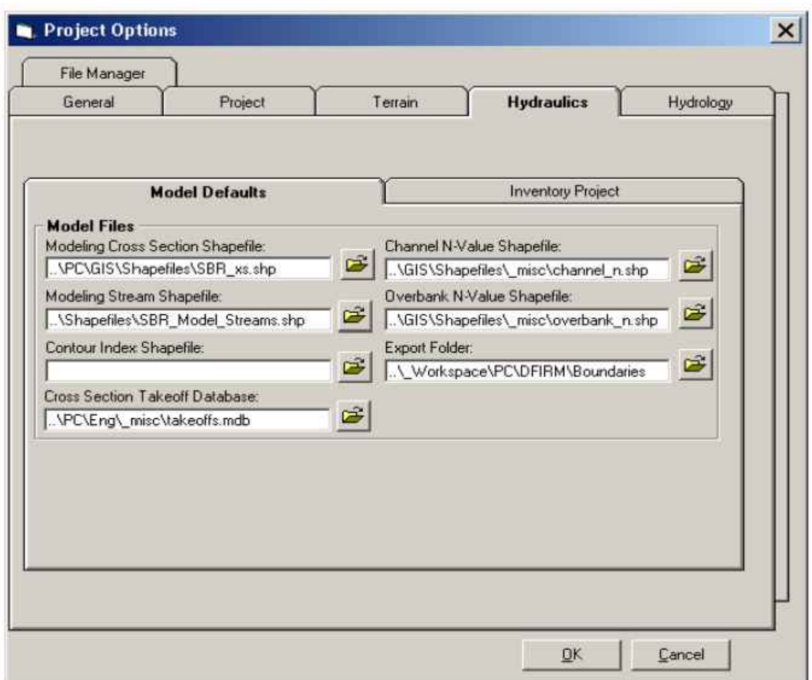
Design rainfall in this study was determined to follow the following two manuals:

Stormwater Technical Guidance Manual, page 32
http://www.willcountylanduse.com/SWComm/ordinances.html#stormord

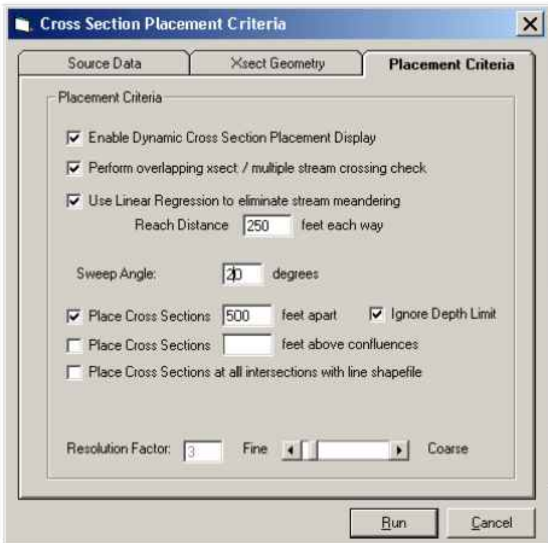
Water Resources Ordinance for Unincorporated Will County, page 22

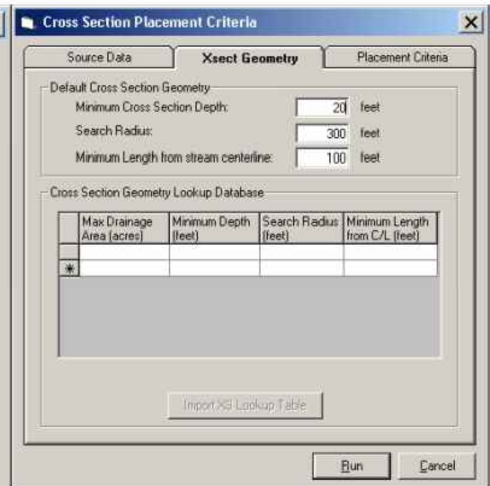
http://www.willcountylanduse.com

- **Hydraulic:**



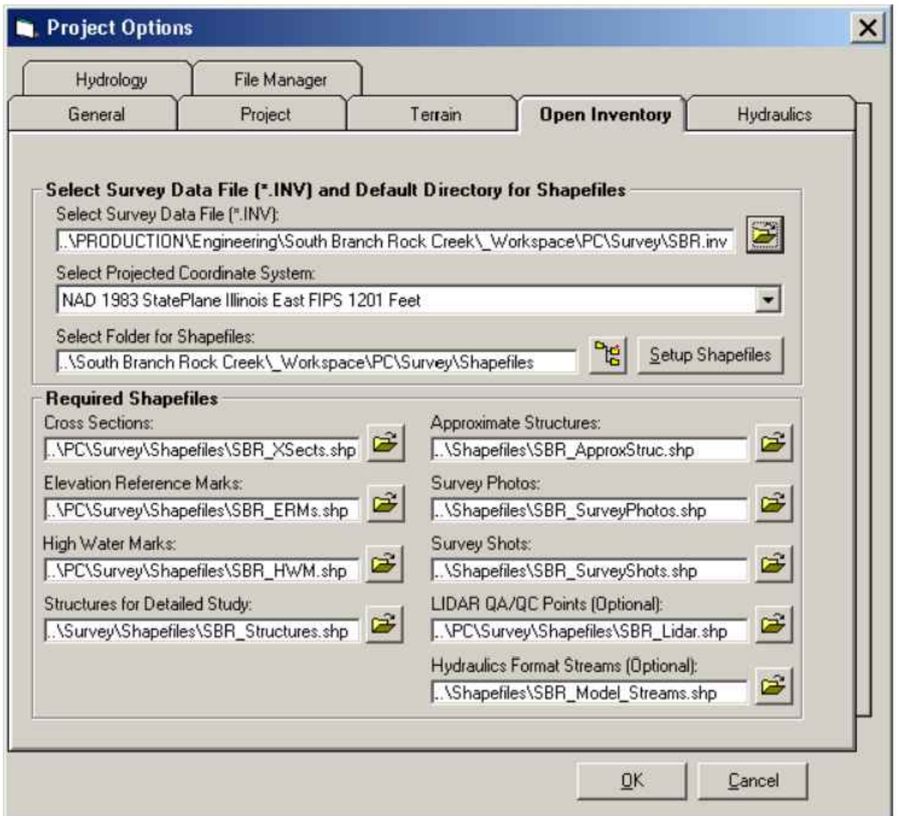
X-sec placement criteria:



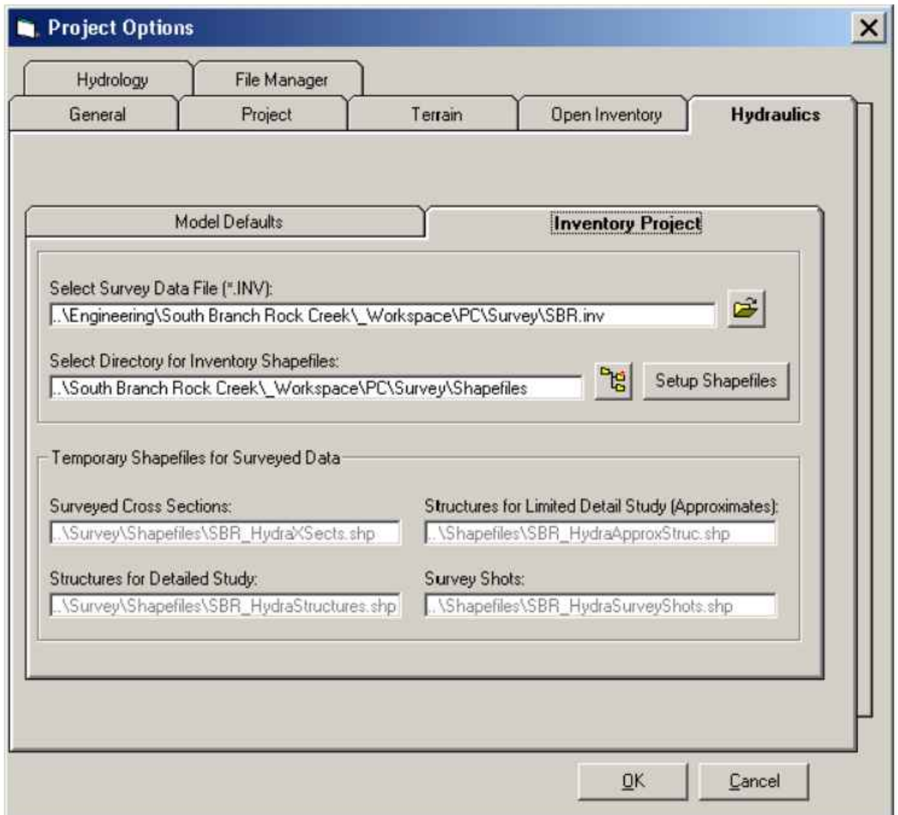


According to North Carolina Cooperating Technical State Mapping Program, for LDS study, the cross sections shall be placed in the model every 500 feet.

HydrauMAX models N:\Illinois\60100630_SouthSuburbanAirport\PRODUCTION\Engineering\South Branch Rock Creek_Workspace\PC\Eng\South Branch Rock Creek Trib 1



Project Options
Hydrology
File Manager
General
Project
Terrain
Open Inventory
Hydraulics
Select Survey Data File (*.INV) and Default Directory for Shapefiles
Select Survey Data File (*.INV):
..\PRODUCTION\Engineering\South Branch Rock Creek_Workspace\PC\Survey\SBR.inv
Select Projected Coordinate System:
NAD 1983 StatePlane Illinois East FIPS 1201 Feet
Select Folder for Shapefiles:
..\South Branch Rock Creek_Workspace\PC\Survey\Shapefiles
Setup Shapefiles
Required Shapefiles
Cross Sections:
..\PC\Survey\Shapefiles\SBR_XSects.shp
Elevation Reference Marks:
..\PC\Survey\Shapefiles\SBR_ERMs.shp
High Water Marks:
..\PC\Survey\Shapefiles\SBR_HWM.shp
Structures for Detailed Study:
..\Survey\Shapefiles\SBR_Structures.shp
Approximate Structures:
..\Shapefiles\SBR_ApproxStruc.shp
Survey Photos:
..\Shapefiles\SBR_SurveyPhotos.shp
Survey Shots:
..\Shapefiles\SBR_SurveyShots.shp
LIDAR QA/QC Points (Optional):
..\PC\Survey\Shapefiles\SBR_Lidar.shp
Hydraulics Format Streams (Optional):
..\Shapefiles\SBR_Model_Streams.shp
OK
Cancel



August 5, 2010

The coordinate system is the Illinois State Plane Coordinate System, 1201 East Zone and NAVD88 vertical datum

October 4, 2010

Asking about what the typical routing methods to use in Illinois projects are. Since it doesn't look like a specific routing methodology is required or suggested in the Will County Stormwater Technical Guidance Manual (http://www.willcountylanduse.com/DevReviewDiv/SubEng/SubEngDocs/TGM_20100825.pdf). It was determined to use the two methods for routing flows through the watershed: Muskingum-Cunge flood routing and Level-Pool Reservoir routing.

February 4, 2011

Question about what duration to simulate for the smaller storm event. For the Exline Sough watershed is to run the critical duration analysis in the 100-year and apply that duration to the other storm events. This approach was applied to the rest of the watersheds.

March 15, 2011

In response to Aaron's comments regarding flow attenuation:

It looks like little to no attenuation of flow is occurring in the channel system with the watershed as being a pretty flat area with significant wetlands. Since it have not high water marks in this watershed or computed peak flows from other study to compare with, a different routine method was used (Muskigum-Cunge) for comparison (L:\work\103576\Wat_Res\HydrologyModels\SBR\MuskingumCunge). The flow at the watershed discharge point is 3690 cfs for the 3-hr, 100-yr event, which is 70% higher than the flow calculated using Modified Puls method.

Already sent out the request to Watershed Concept regarding WISE method for calculating subreach parameters (the numbers WISE calculated are very high, hence little attenuation is occurring).

As we discussed in our South Suburban Airport floodplain modeling phone call last week, the 100-year peak discharges being calculated by current HEC-HMS models seem high when compared to other estimates of 100-year flows. It appears the correct model-building procedures are being followed, so there are not obvious fixes to modeling procedures to implement. And we were not able to obtain enough project-area-specific high water data that will allow us to do detailed calibration to surveyed events. Therefore, we need to further consider what reasonable 100-year peak discharges for these areas may be, and what HEC-HMS model adjustments should be done to achieve reasonable discharges.

I've summarized my thoughts on this matter in this email. It's not polished yet, but I wanted to get this out to you for your consideration. Feel free to question, comment on or suggest changes to any part of this. If we agree on a course of action, then we could send out a writeup to the rest of the modeling team.

Supporting information shows that HEC-HMS 100-year peak flows seem high:

Consider Plum Creek.

A StreamStats (Illinois USGS rural regression equations; one of the preferred methods in Illinois for calculating rural peak discharges) analysis was done by Hannah for a location near the downstream point of the HEC-HMS model. Drainage area = 6.39 square miles

StreamStats (regression equation) peak flow, 100-year recurrence interval = 885 cfs

90% prediction interval for Q100 = 414 cfs (low end) to 1,890 (high end)

Compare to HEC-HMS discharge at comparable location:

Q100, HEC-HMS Muskingum-Cunge routing = 2,500 cfs

Q100, HEC-HMS Modified Puls routing using WISE relations= 3,250 cfs

Q100, HEC-HMS Modified Puls routing using HEC-RAS-derived discharge/storage: 1,900 to 2,000 cfs

Conclusion: Even the lowest HEC-HMS-calculated peak flow is more than double the flow calculated from StreamStats. Calculated HEC-HMS flow is outside of the 90% confidence interval for Q100.

Another point of comparison: The Plum Creek / Hart Ditch system has been studied by others, and some documentation is available online

From a 2009 CBBEL presentation: at the Illinois/Indiana State Line (Drainage area = 36 square miles), Illinois calculated a regulatory Q100 using HEC-1 of 2,700 cfs

Indiana calculated a regulatory Q100 using Coordinated Discharges of 1,900 cfs

For a drainage area more than 5 times the size of our study area.

Note that in September 2008 a peak discharge of 3,100 cfs occurred at this location during a major flood, so regulatory Qs may be low, but our Q100 discharge for a 6 square mile area still seems quite high.

Yet another point of comparison for Plum Creek:

From the MWRDGC watershed plan for the Little Calumet Basin (published in 2009 or 2010):

Plum Creek was one of the subwatersheds modeled and studied in this plan.

They developed and calibrated an HEC-HMS model of the overall watershed.

Their most upstream Plum Creek basin, PC-01 has an area of 10.3 square miles, so it apparently encompasses all of our Plum Creek modeled area and more.

The calculated 100-year, 24-hr rain duration peak discharge for this 10.3 square mile basin (provided by John Morgan) is about 1,600 cfs.

MWRDGC Little Calumet plan is available online at:

http://www.mwrd.org/irj/go/km/docs/documents/MWRD/internet/protecting%20the%20environment/Stormwater%20Management/html/Little%20Calumet%20River%20Watershed/Little_Calumet_River_DWP.htm

Suggested HEC-HMS adjustments:

First of all, I investigated why the calculated curve numbers seem quite high. It looks like the dominant land use / soil complex in the project area is row crops, hydrologic soil group C. This was assigned a curve number of 85 in the WISE lookup table, apparently because the row crops were assumed to be in have no cropping management (contoured, terraced, crop residue, etc.).

According to the MWRDGC Little Calumet study Appendix C, a curve number of 79 was assigned for C soils in the NIPC land use code 2100 crops/grain/grazing (which we used as equivalent to the row crop category).

The curve numbers assigned to row crops will dominate the watershed curve number calculations. In light of the fact that our calculated curve numbers and peak flows seem high, I think we are justified in lowering the row crop curve numbers to a line reflecting some management measures such as contour cropping and conservation tillage, or even to the MWRDGC Little Calumet values for land use category 2100.

We could update the standard curve number lookup table, recalculate subbasin curve numbers, and rerun the HEC-HMS models. Peak flows may still be quite high compared to other sources of data.

As John Morgan has pointed out, the MWRDGC study of the Little Calumet River (which Plum Creek is a part of) is a source of useful information.

From Section 3.6 of the MWRDGC Little Calumet plan, on Plum Creek:

"A detailed calibration was performed for the Plum Creek subwatershed using historic gage records under the guidelines of the Cook County Stormwater Management Plan. Three historical storms, April 2006, April 2007 and September 2008 were evaluated…

"During calibration of the Plum Creek subwatershed model, the curve number, directly connected impervious area percentage and lag times were adjusted…during calibration, the Clark's storage coefficient R was increased by 25 percent."

Appendix F of this report gives the calibrated subbasin parameters for their HEC-HMS model. Their most upstream Plum Creek basin, PC-01 has an area of 10.3 square miles, so it apparently encompasses all of our Plum Creek modeled area and more. Their calculated model input parameters are:

Clark time of concentration: 6.05

Clark R (storage coefficient): 11.35

Curve number: 78.7

% directly connected impervious area: 0.7%

Therefore, I think we could lower our curve numbers for Plum Creek so they average about 79 or even lower.

Consider how we could use these calibrated Clark parameters also.

And considering the similar land use, topography and soils – if we come up with more reasonable hydrologic parameters for the Plum Creek basin, I think it would be reasonable to also use these for other basins.

Another model change that may lower peak flows is to more explicitly account for major storage areas behind restrictive culverts. I'm not sure how much this situation comes up in the various watersheds – I'd leave that to individual modelers to take another look at – but the example that comes to mind is the Chicago&Eastern Illinois / Union Pacific railroad embankment and culvert on Plum Creek. According to our HEC-RAS model, this railroad embankment dominates the flood profile for Plum Creek, causing a nearly level backwater for a long distance upstream. Water depths in the vicinity of the culvert are 15 to 20 feet, so there would be a lot of area inundated with floodwater storage. But in HEC-HMS, there is very little flood peak attenuation in this reach. This may be a place where we should add more detail to the HEC-HMS model, with a more explicit storage node and a stage/storage/discharge curve based on the culvert hydraulics and upstream terrain.

We could also do some initial rough floodplain mapping and see if our floodplain limits seem realistic. If a large percentage of the project area is shown as within the floodplain limits, including many structures that haven't reported historical flood damage, that may indicate that our floodplain elevations are unrealistically high. And since the HEC-RAS input data I've seen so far seems reasonable, the most likely cause would be high discharge rates.

We could also compare our initial mapping output to the effective FEMA Zone A boundaries, to see if the two are at least in the same ballpark.

Conclusion:

I suggest the following steps to assess the reasonable of the 100-year peak discharges we're calculating from our HEC-HMS modeling, and make adjustments to the HEC-HMS models where appropriate. Feel free to disagree or suggest anything else you see fit.

- Determine a reasonable range of Q100s for each watershed, based on StreamStats and/or published studies such as the MWRDGC Little Calumet Studies (not sure if any similar studies exist for Rock Creek or Black Walnut Creek).
- Adjust curve numbers in WISE lookup table downward, especially for agricultural land which dominates the project area. One appropriate source may be the published curve numbers from the MWRDGC Little Calumet study.
- If Q100s from HEC-HMS still are outside of reasonable range, make further adjustments to hydrologic model input parameters. Consider using reported parameters from Little Calumet Study for guidance.
- We probably have the most sources of other data for comparison/calibration for Plum Creek, since it was part of the MWRDGC Little Calumet study, but because of the similar land use / topography / soil conditions throughout the project area, trends in input parameters for Plum Creek could be applied to other modeled basins.
- Review water surface profiles and watershed characteristics to determine if any culvert/bridge restrictions should be modeled more explicitly in HEC-HMS as a major flow attenuation.
- Do some quick, rough floodplain mapping based on our preliminary HEC-HMS and HEC-RAS results to see if results seem reasonable, compared to FEMA Zone A boundaries, engineering judgment and historical flooding damage/observations (or lack thereof).

April 5, 2011

The Union Pacific railroad embankment culvert (river station 5571) is modeled as Conspan Arch with the arch span/arch rise ratio 2:1. I lowered the upstream velocity for the 100-year event to 16 ft/s and the downstream velocity is 18 ft/s.

Regarding Hydrology comment, few HEC-HMS adjustments were made:

1. HEC-HMS model: L:\work\103576\Wat_Res\HydrologyModels\Plum\Plum_MP_HEC-RAS-original model
2. HEC-HMS model: L:\work\103576\Wat_Res\HydrologyModels\Plum\Plum_MP_HEC-RAS_CN-The Curve Numbers Adjustments

Land Use Description	A	B	C	D	A/D	B/D	C/D
2100 CROP/GRAIN/GRAZ	64	73	79	83	73	78	81

3. HEC-HMS model: L:\work\103576\Wat_Res\HydrologyModels\Plum\Plum_MP_HEC-RAS_CN_R_25- same as Point 2 plus the Clark's storage coefficient R was increased by 25 percent.
4. HEC-HMS model: L:\work\103576\Wat_Res\HydrologyModels\Plum\Plum_MP_HEC-RAS_CN_R_30_C- same as Point 2 plus the Clark's storage coefficient R was increased by 30 percent and few by 50 percent. The CN for subbasin 20 was lowerd from 81 to 78.
5. HEC-HMS model: L:\work\103576\Wat_Res\HydrologyModels\Plum\Plum_MP_HEC-RAS_CN_R_30_C1- same as Point 2 plus the Clark's storage coefficient R was increased by 50 percent. The CN for subbasin 20 was lowerd from 81 to 75 and subbasin 21 was lowerd from 78 to 75.

Model output summary: L:\work\103576\Wat_Res\HydrologyModels\Plum_Results_HEC-RAS_CN.xlsx

L:\work\103576\Wat_Res\HydrologyModels\Plum_Results_HEC-RAS_CN_R_25.xlsx

L:\work\103576\Wat_Res\HydrologyModels\Plum_Results_HEC-RAS_CN_R_30_C.xlsx

L:\work\103576\Wat_Res\HydrologyModels\Plum_Results_HEC-RAS_CN_R_30_C1.xlsx

HEC-RAS model: L:\work\103576\Wat_Res\HydraulicModels\Plum\ Plum Creekm.prj

Plans: 100year existing and routing. All cross-sections in HEC-RAS were extended based on 2 foot contours topo for routing.

Streamstats for structures: L:\work\103576\Wat_Res\HydrologyModels\Plum\Streamstats\

In HEC-RAS single 100 year I used Flows from L:\work\103576\Wat_Res\HydrologyModels\Plum_Results_HEC-RAS_CN_R_30_C.xlsx but I ran also few flows under multirun plan. I still think flow should be reduced at structure P3 (HEC-HMS -13C) based on streamstats. The HEC-HMS flow is 787 cfs and streamstats is 564 cfs.

April 11, 2011

Here are the file locations of the HH analysis for the South Branch Rock Creek:

- HEC-HMS models L:\work\103576\Wat_Res\HydrologyModels\SBR
  There are three models generating flows from different hydrological parameters and routine methods:
  - L:\work\103576\Wat_Res\HydrologyModels\SBR\ModifiedPuls - Modified Puls routine, WISE curve numbers, Clark storage coefficients
  - L:\work\103576\Wat_Res\HydrologyModels\SBR\MuskingumCunge - Muskingum-Cunge routine, WISE curve numbers, Clark storage coefficients
  - L:\work\103576\Wat_Res\HydrologyModels\SBR\NewCN_R25_MP - Modified Puls routine, MWRDGC Little Calumet plan curve numbers, 25 percent increased Clark storage coefficients
- HEC-RAS models L:\work\103576\Wat_Res\HydraulicModels\SBR
  There are two models corresponding to the above HEC-HMS flows (since the Muskingum-Cunge flows are even higher than the Modified Puls flows, the Muskingum-Cunge flows were not used to create HEC-RAS model)
  - South Branch Rock Creek.prj - Modified Puls routine, WISE curve numbers, Clark storage coefficients
  - SBR_NewCN_R25_MP.prj - Modified Puls routine, MWRDGC Little Calumet plan curve numbers, 25 percent increased Clark storage coefficients

I did a quick floodplain comparison with the Will County flood zone. Instead of doing an elevation comparison (I checked few locations. It seems like the topos has gone a significant change, the elevations read from the Will County right and left flood boundaries of one single location sometimes have couple feet of difference), I did a flood plain width comparison at channel cross sections. The results named "ModelCheck_MP.xlsx" and "ModelCheck_NewCN_R25_MP.xlsx" are located at L:\work\103576\Wat_Res\HydraulicModels\SBR.

In summary, out of 86 cross sections (57 at South Branch Rock Creek main stem, 22 at Tributary 1 and 4 at Tributary 2), there are 16 cross sections having flood plain widths wider than the historical ones, using WISE CN and Clark storage coefficients. After adjusting the hydrological parameters, 14 out of 86 cross sections have flood plain widths wider than the historical ones. Both models show some significant flood plain width increases, the percent increases range from 0.58 % to 1600%.

September 8, 2011

(from Hanson) The floodway boundaries in Illinois are set using a 0.10-ft surcharge. So we start with Method 4 or Method 5 (which is the same as 4 but includes an optimization scheme) and work our way upstream eventually locking-in the boundaries with Method 1. Illinois is also an equal conveyance reduction state so we look at that as well.