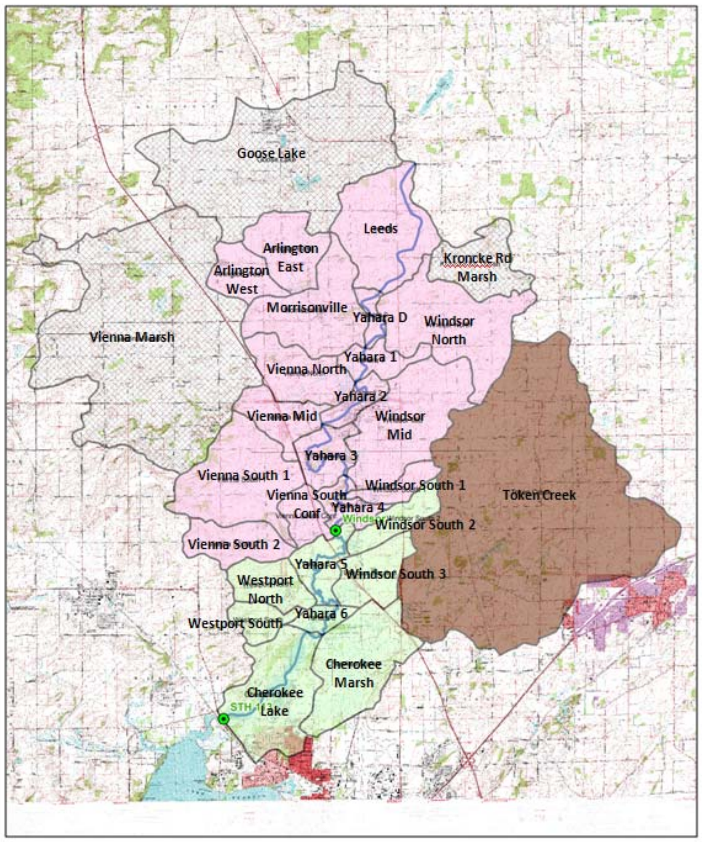
HYDROLOGIC & HYDRAULIC MODELING YAHARA RIVER INPUTS TO LAKE MENDOTA



DNR Lake Planning Grant Final Report

Submitted By:

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Introduction

The Yahara River receives approximately two-thirds of its water from the Mendota Watershed. Additionally, much of this water entering Lake Mendota is attributed to dry weather and wet weather flow from the Yahara River Watershed. In order to understand the changing conditions of water level for Lake Mendota it is necessary to evaluate hydrologic and hydraulic conditions in upstream waters (Cherokee Marsh).

Construction of a hydrologic and hydraulic model was contracted to be completed by MSA Professional Services. Originally, the work was initiated to evaluate the restriction of flow that may be caused by three bridges along the Yahara River including the Westport Road Bridge, Railroad Bridge, and Highway 113 Bridge. After further analysis, there were several issues with the original hydrology and hydraulic models for this area. The work provides more detailed analysis and an update to the existing models. The work and analysis is further explained and presented in this report.

An online model is available at www.infosyahara.org.

HEC-HMS Hydrology Model

The hydrologic computer model HEC-HMS was used to produce simulated hydrograph output from the Yahara River watershed. In this application of HEC-HMS, the TR-55, 'Urban Hydrology for Small Watersheds' (USDA, 1986) methodology was used. TR-55 requires three primary input parameters which can be used to determine peak discharge rates and runoff volumes from a particular rainfall event. These parameters include drainage area, runoff curve number, and time of concentration (although when applied in HEC-HMS, time of concentration is converted to a similar parameter called lag time). A rainfall event (time-series record) is subsequently selected and applied to the model watershed.

At the start of this study, there were two HEC-HMS models of the Yahara River watershed that were publicly available. One was completed by Black & Veatch in 2003 and was used in the Flood Insurance Study for Dane County. However, the model was used primarily for determining likely peak flood elevations in Lake Mendota, and did not contain sufficient detail of subwatersheds or river reach data in the watershed above STH 113. Therefore, none of the data from that model was utilized in the calibration discussed here. The other available HEC-HMS model covers only the watershed of Token Creek (a main tributary to the Yahara River) and was completed by the Wisconsin DNR in 2005 and was used in the Flood Insurance Study for Dane County. This model contained detailed subwatershed delineations, reach routing, and a calibration of the watershed to a 1993 flood event. The model covered the watershed of the creek from its source to the confluence with the Yahara (27.1 square miles). However, the main calibration point was USGS stream gage # 5427800 which was located somewhat upstream of the confluence and had a contributing area of 24.1 square miles. The calibration discussed here uses the data from the WDNR's 2005 HEC-HMS model for the area above the Token Creek gage in its entirety and unchanged from how it was developed by the WDNR. MSA constructed a HEC-HMS model for the entire watershed above STH 113. The model contains twenty-five subwatersheds in addition to those taken from the WDNR's Token Creek model. These subwatersheds have an average size of about 2 square miles and a maximum size of about 5 square miles. The model connects the subwatersheds with one another by means of stream reaches. These stream reaches use the Muskingum-Cunge method of hydrograph routing, which adds hydrographs produced by the watersheds and translates them downstream. The reach routing parameters were estimated from USGS 10-foot interval contour mapping and augmented with 4-foot interval contour data where available. The model was structured to include junction points at the stream gages, so that model results at these points can be compared to actual recorded data.

HEC-HMS Calibration

Available Data

Storm events in August 2007 and June 2008 produced significant rain amounts and flooding in the Yahara River watershed. The following data was available:

- Daily rainfall data published by NOAA for recording stations statewide, including those at Arlington, Lodi, Truax Field in Madison, and Portage.

• Hourly rainfall amounts provided by the Wisconsin state climatologist for the entire months of August 2007 and June 2008 at the recording station at Truax Field in Madison.

• Hourly rainfall data purchased from NOAA for the entire months of August 2007 and June 2008 at the recording station at Arlington University Farms.

• Five-minute interval stream flow and stage data at the USGS gage at STH 113 (#05427850). As part of the hydraulic model calibration previously presented to the City, this data was normalized to fifteen-minute intervals.

• Fifteen-minute interval stream flow data at the USGS gage at the Windsor Golf Course (#05427718).

• Fifteen-minute interval stage data at the USGS gage at the dam on Lake Mendota (#05428000).

Storm Event Dates

The storm event dates that were used in the model included August 17 through 22, 2007, and June 7 through 12, 2008. Each six-day "event" consisted of two days of heavy rain, followed by two to three days of little to no rain, and finally another period of heavy rain. The model time frame was established at 120 hours (five days) which allowed runoff from the first days of heavy rain to work its way through the watershed, producing a measureable hydrograph at the gage locations. This duration is not long enough to include the complete hydrograph from the final period of rainfall. However, the model computation of the complete second runoff hydrograph is not necessary in this case, for two reasons. First, the SCS method does not include a "curve number regeneration" parameter for the watershed. This generally results in the model computing higher runoff volumes as compared to gage-measured runoff volumes for rain events that follow an initial event. Second, for the particular case of August 22, 2007, rainfall amounts at the Arlington rain gage were significantly lower than the amounts at Truax Field, and did not produce a large enough hydrograph at the Windsor stream gage to be able to do a justifiable calibration.

Hydrograph Separation/Storm Runoff Computation

The overall hydrograph was separated into three components. A runoff hydrograph for the first two days of rainfall for each event (the primary hydrograph) was determined by subtracting a constant base flow amount from each stream gage (estimated from the average flow in the stream over the seven days prior to the main storm) and extrapolating an estimated receding limb for the primary hydrograph. This produced three separate hydrographs: a base flow hydrograph (constant line), a primary hydrograph (the product of the first days of rain) and a secondary hydrograph (the product of the last day of rainfall). Figure 1 shows the June 2008 Hydrograph separation. The total area under the primary hydrograph was computed to determine a total storm runoff volume. This runoff volume was used in the determination of runoff curve number discussed in the following section.

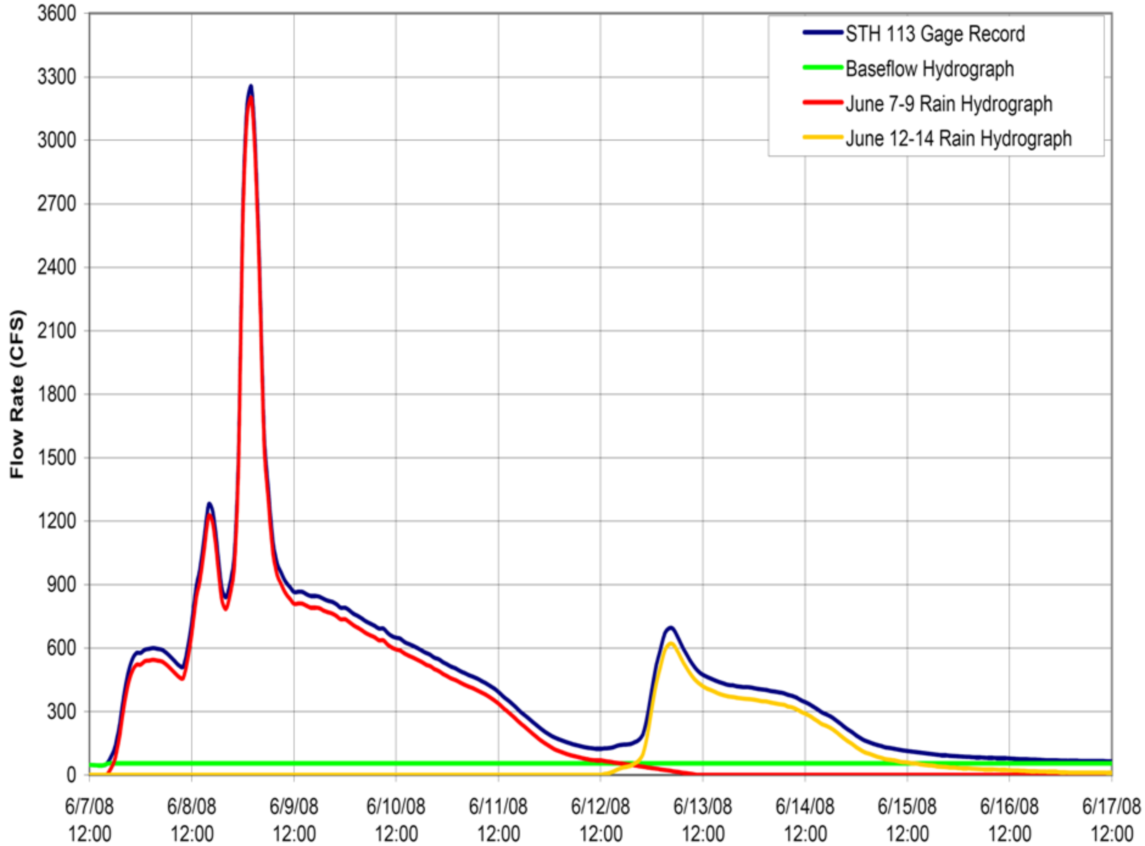

Figure 1: June 2008 Hydrograph Seperation

Hourly Rainfall Data Adjustment

As mentioned in the Available Data section above, two locations with hourly rainfall data were available. At the Truax Field station, the sum of hourly rainfall amounts from the state climatologist matched the daily amounts reported in the NOAA monthly publication. However, the Arlington station data exhibited a discrepancy between hourly amounts and daily totals for June 7-9, 2008. The sum of the daily totals from the NOAA publication was 7.4 inches, but the sum of the hourly amounts was 7.1 inches. Since this difference is less than 5%, the hourly amounts were used as published without adjustment. Additionally, the Arlington station data was missing hourly rainfall amounts from August 18-22, 2007 altogether. To create useable hourly rainfall data at Arlington, it was assumed that the rainfall intensity pattern at Arlington matched that experienced at Truax, but with a few minor adjustments:

- The daily data published for Arlington is logged daily at 4:00 PM, so first the Truax data was rearranged to match this time cycle.
- The hourly rainfall amounts for August 18 to 20 were scaled by a factor of 0.86 to reflect the ratio of the total rainfall at Arlington (4.94 inches) to the total rainfall at Truax (5.73 inches) as reported by NOAA for this three-day time cycle.
- The hourly data for this time frame was shifted by -8 hours (i.e. the 8 AM value was backed up to midnight, 9 AM was backed up to 1 AM, etc.) so that the sum of the hourly rainfall matched the published daily data for each day from the 18th to the 20th.

• The hourly data for the 21st and 22nd was inserted entirely on the assumption that rainfall occurred at a constant intensity of 0.1 inches per hour for five hours, since the rainfall at Truax had no detectable correlation to the daily totals at Arlington for this time period. These adjustments resulted in an hourly rainfall data set that, when summed, matched the published daily amounts to within 0.2 inches on any given day.

HEC-HMS Model Characteristics

As mentioned above, three primary watershed characteristics are used to determine peak discharge rates and runoff volumes from a particular rainfall event: drainage area, runoff curve number (RCN), and time of concentration or lag time.

Drainage Areas

The watershed above STH 113 was broken down into thirty-six subwatersheds. Eleven of these subwatersheds are taken directly from the DNR's HEC-HMS model above the Token Creek gage and are not individually mapped. Of the twenty-five remaining, seventeen are above the Windsor stream gage and eight are above STH 113 but below the Windsor and Token Creek gages. When summed, the watershed areas match the "contributing area" given by the USGS at the gaging stations to within 1 square mile (a difference of 2.7% at the Windsor gage and 0.6% at the STH 113 gage).

Figure 2 shows the delineation of each of the watersheds. Note that the figure includes three "non-contributing" areas that are crosshatched. These areas are counted by the USGS and other agencies as being within the Yahara River watershed, but do not discharge surface runoff to the river due to low areas within the terrain. The rain gage nearest to each watershed was used to determine which rain gage should be assigned to which watershed in the model.

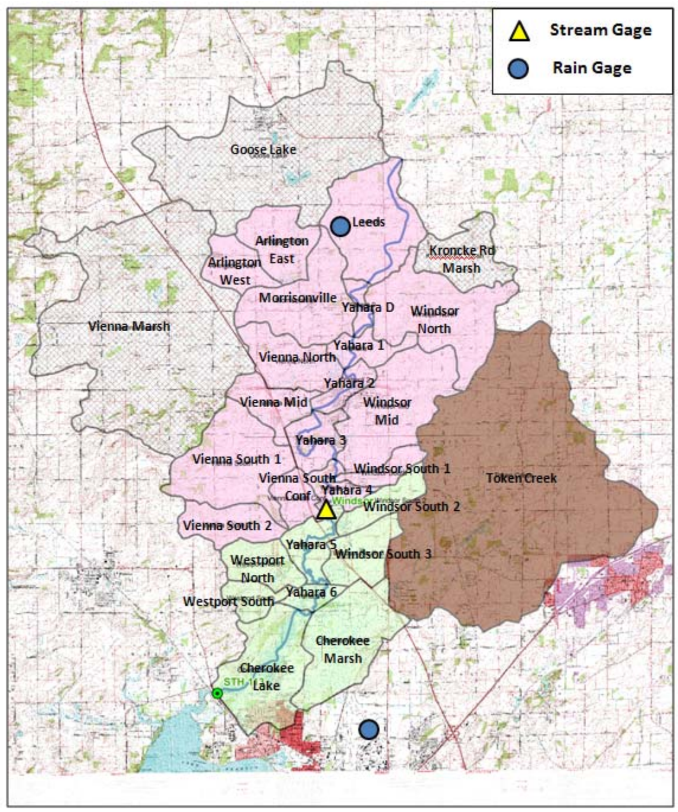

Figure 2: HMS Model Framework

Runoff Curve Number

The runoff curve number (RCN) is a concept created by the Soil Conservation Service in 1972 and is widely used today to estimate expected runoff depths from watersheds for single rainfall event depths. The RCN is usually estimated according to land use and soil information describing a watershed, and then is applied with rainfall information to determine the runoff. In this calibration exercise of the Yahara River, however, rainfall amounts and total runoff volumes are known measured values from gage information. From this known data, an estimate for the value of RCN in the HMS model can be extracted for the watershed.

The runoff from the gaged watersheds can be determined for a given storm event by performing a water budget computation. The total volume of runoff from a watershed is equal to the total hydrograph through a gage, minus any volume accounted for by an upstream gage, plus the increase in storage volume upstream of the gage. When the primary hydrographs for the two storms at the two flow gages discussed in the "Hydrograph Separation" section above are considered, the following table summarizes this water budget.

Table 1: Watershed Water Budget

Description	Windsor Gage		STH 113 Gage	
	August 2007	June 2008	August 2007	June 2008
Flow Volume through Gage	2,066 acre-feet	6,230 acre-feet	6,136 acre-feet	8,687 acre-feet
Minus (-) Volume through upstream Gage	None	None	2,066 acre-feet	6,230 acre-feet
Plus (+) Increase in upstream storage volume	None	None	955 acre-feet	1,435 acre feet
Equals (=) Excess Runoff Volume from Watershed	2,066 acre-feet	6,230 acre-feet	5,025 acre-feet	3,892 acre-feet

This excess runoff amount can be expressed as the depth (in inches) over the entire watershed area. Then, this depth can be related to total storm rainfall and to the potential maximum retention of rainfall on the watershed. This retention parameter is labeled S in Equation 2-3 of TR-55. Finally, S can be related to RCN using Equation 2-4 from TR-55. Table 2 summarizes the computation of the RCN for the watersheds above the two gages.

Table 2: Watershed RCN Estimate

Description	Windsor Gage		STH 113 Gage	
	August 2007	June 2008	August 2007	June 2008
Excess Runoff Volume from Watershed	2,066 acre-feet	6,230 acre-feet	5,025 acre-feet	3,892 acre-feet
Runoff Depth, Q[1]	1.02 inches	3.08 inches	2.36 inches	1.75 inches
Total Rainfall (Weighted by Location), P[2]	5.02 inches	7.03 inches	5.48 inches	6.59 inches
Potential Maximum Retention, S[3]	8.01 inches	5.49 inches	4.39 inches	8.65 inches
Runoff Curve Number[4]	56	64	69	54

[1] Q, runoff depth, is equal to the runoff volume from the watershed divided by the watershed area.
[2] P, total rainfall or precipitation, is measured for the timeframe producing the hydrograph volume.
[3] Equation 2-3 states runoff depth Q equals (P-0.2S)^2/(P+0.8S); equation is solved for S.
[4] Equation 2-4 states that S is equal to (1000/RCN)-10; equation is solved for RCN.

It should be noted that in these watersheds, the RCN will not be entirely uniform. Most of the area is agricultural or open space, but there are areas of urbanization around Deforest and the northerly edge of Madison. While a detailed land use analysis was not performed, a simple delineation of urbanized area was estimated from aerial photographs. It was determined that approximately 6 square miles above the Windsor gage is urbanized, and approximately 2 square miles above the STH 113 gage is urbanized. In general, urban areas have an RCN of between 70 and 80, and agricultural/open space areas will have an RCN of between 60 and 70. For this study it was assumed that urban area RCNs are 10 points higher than nonurban area RCNs. Accordingly, the following RCNs were applied to land uses in the initial uncalibrated model such that they provided the required weighted average RCN of 62:

- Urban areas: 71
- Non-urban areas: 61

Lag Time

Lag time is defined as the time elapsed between the centroid of a rainfall hyetograph plot and the centroid of the resulting hydrograph. While the lag time for the entire watershed above each gage could be directly extracted from the measured data (similar to the RCN derivation above) it could not be easily proportionally divided among the several smaller subwatersheds. Therefore, the initial parameters for insertion into the HMS model were estimated using the upland equation, as described in the NRCS's National Engineering Handbook Chapter 4. The methodology states that the lag time for a given flow path is directly proportional to the travel distance, inversely proportional to the square root of the travel slope, and inversely proportional to an empirical velocity factor (Kv). The flow path length for each watershed was measured using available topographical data, and the slope of the flow path was estimated by dividing the difference in elevation at each end of the flow path by the flow path length. The initial value for Kv was assumed based on guidance found in NEH-4, and was set at 18 ft/sec (note that the velocity factor differs from flow velocity) for urban watershed segments and 9 ft/sec for non-urban watershed segments.

Boundary Conditions

The USGS operates a stream flow gage at the Windsor Golf Course (#05427718) just upstream of Interstate 39/90/94. At this location, the river is within a relatively narrow valley and storage doesn't appear to be significant. Additionally, at and below the bridge near the gage, there do not appear to be any hydraulic restrictions which would significantly backup flow in the stream. Therefore, the flow boundary condition is a Qin = Qout situation and the flow at the gage will not be influenced by downstream tailwater conditions. The USGS also operates a stream flow gage at STH 113 (#05427850). At this location the Cherokee Marsh provides a great deal of storage upstream of the gage. Additionally, as discussed in the previous memo to the City regarding the hydraulic model of the reach between the STH 113 bridges and Lake Mendota, the flow rate out of the marsh is greatly influenced by the tailwater stage of Lake Mendota. Therefore, the boundary conditions at this calibration point need to include both storage on the upstream side and a Tailwater elevation on the downstream side. The HEC-HMS model can include storage areas, but cannot simulate conditions where tailwater varies with time. At this calibration point, all of the flows generated from the watershed were computed using HEC-HMS, but an unsteadystate HEC-RAS model was used to compute the effect of the marsh storage and the hydraulic influence of the tailwater. To accomplish this, a storage node was inserted on the upstream end of the hydraulic model and USGS water stage data from the gage at the dam on Lake Mendota (#05428000) was used to establish a time-series boundary condition for the lake for both the August 2007 and June 2008 events.

Calibration Process

As discussed above, the three main variables in the hydrologic model are land area, RCN, and lag time. Land area is a measured value and should not be adjusted to attempt to match model results to gage data; therefore, RCN and lag time need to be varied in successive model runs until a satisfactory match is achieved. The calibration process consisted of the following steps:

1. Construction of the model as discussed above, using measured land areas and estimated initial RCN and lag time values.

2. Execution of the model and comparison of:
   - the peak flow and total runoff volumes from the HMS model to the gage record at Windsor
   - the peak flow, total runoff volume, and the elevation at the upstream side of the STH 113 bridge from the RAS model to the gage record at STH 113

3. Modification of the initially-assumed RCNs for both urban and non-urban areas with the intent of matching the modeled storm runoff volume to the gage record.

4. Modification of the initially-assumed velocity factors (Kvs) with the primary intent of matching the modeled storm peak flow rate to the gage record and the secondary intent of matching the modeled storm hydrograph shape to the gage record.

5. Reiteration of steps 2 through 4 until the model results substantially matched the gage record. It was found that the model run based on initial assumptions of RCN and lag time produced significantly higher peaks and volumes at both gages as compared to recorded data. Therefore, the RCN values and the velocity factors were iteratively reduced. During the final calibration steps, it was found that to better separate the distinct peaks that appear in the Windsor gage record, the urban RCN and velocity factors needed to be differentiated from their non-urban counterparts by a somewhat greater factor. Urban RCN values were increased to a number 15 higher than the non-urban values. The urban velocity factor values were increased to a number 2.5 times that of the non-urban areas. Lastly, the RCNs in marshy areas between STH 113 and STH 19 were reduced by 5. Summary tables of initial and final values of RCN and lag time are shown in Table 3.

Table 3: Model RCN Summary

Subwatershed	Area (square miles)	Runoff Curve Number	
		Initial	Final
Subwatersheds Delineated by MSA			
Arlington East	2.191	61	56
Arlington West	1.417	61	56
Cherokee Lake	4.200	63	59
Cherokee Marsh	4.144	61	54
Leeds	4.314	61	56
Morrisonville	3.727	61	56
Vienna Mid	1.583	65	62
Vienna North	2.273	62	58
Vienna South 1	4.556	61	56
Vienna South 2	1.767	61	56
Vienna South Confluence	1.005	62	57
Westport North	1.541	61	56
Westport South	0.923	61	56
Windsor Mid	4.691	65	61
Windsor North	5.003	61	56
Windsor South 1	0.831	63	59
Windsor South 2	1.263	64	61
Windsor South 3	1.813	66	64
Yahara 0	0.494	61	56
Yahara 1	1.009	63	58
Yahara 2	0.785	71	71
Yahara 3	1.369	71	71
Yahara 4	0.956	69	68
Yahara 5	1.322	62	52
Yahara 6	0.577	61	51
Subwatersheds Taken from WDNR Token Creek Model			
R100W80	2.176	51	51
R170W160	3.806	50	50
R200W180	2.135	55	55
R20W20	1.898	54	54
R280W280	1.198	51	51
R290W290	3.039	55	55
R30W30	0.624	60	60
R40W40	2.443	51	51
R60W10	3.298	51	51
R70W70	2.394	50	50
R90W90	1.134	53	53

HEC-HMS Model Results

Graphical results of the model runs are attached to this memo, showing time-series plots of the following data:

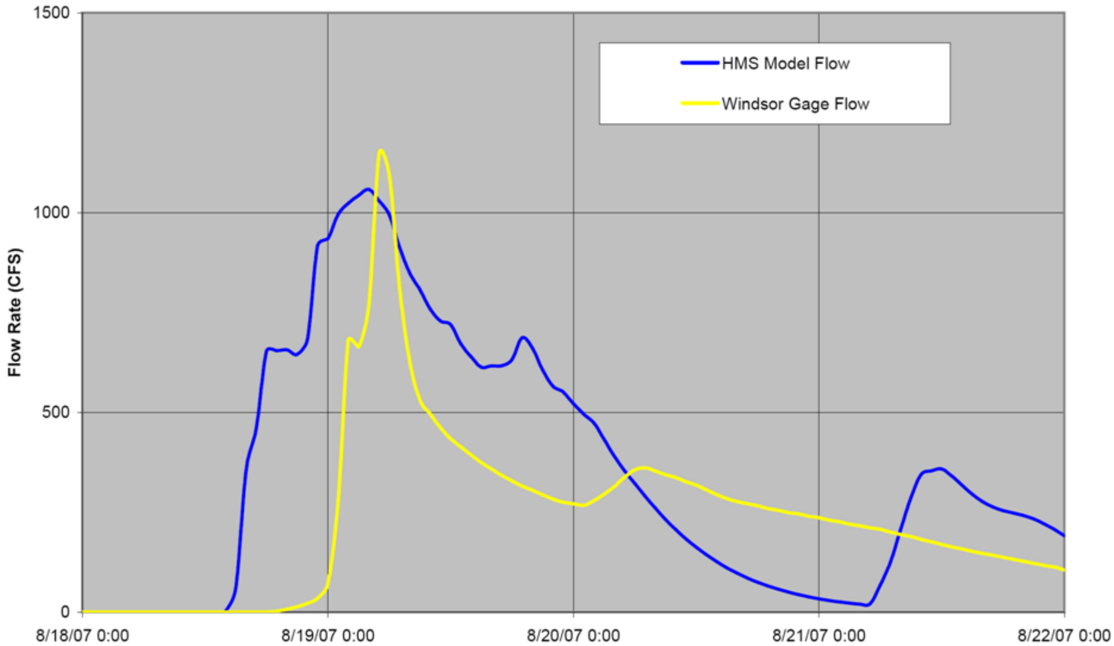

Figure 3: HEC-HMS Model vs. gage flow rate for August 2007 at the Windsor Gage

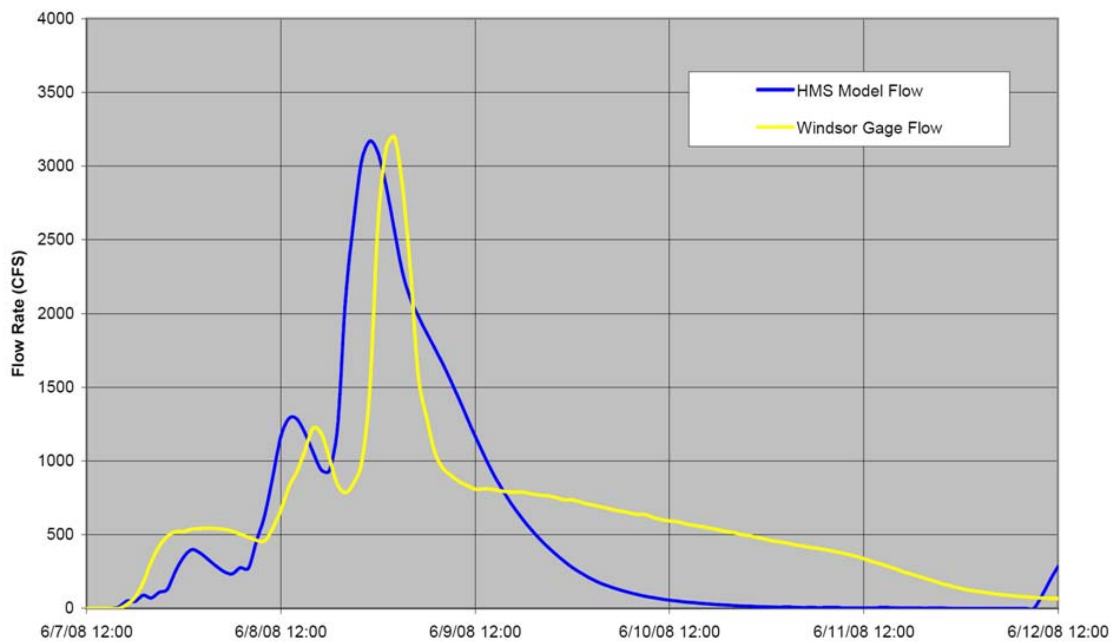

Figure 4: -HMS Model vs. gage flow rate for June 2008 at the Windsor Gage

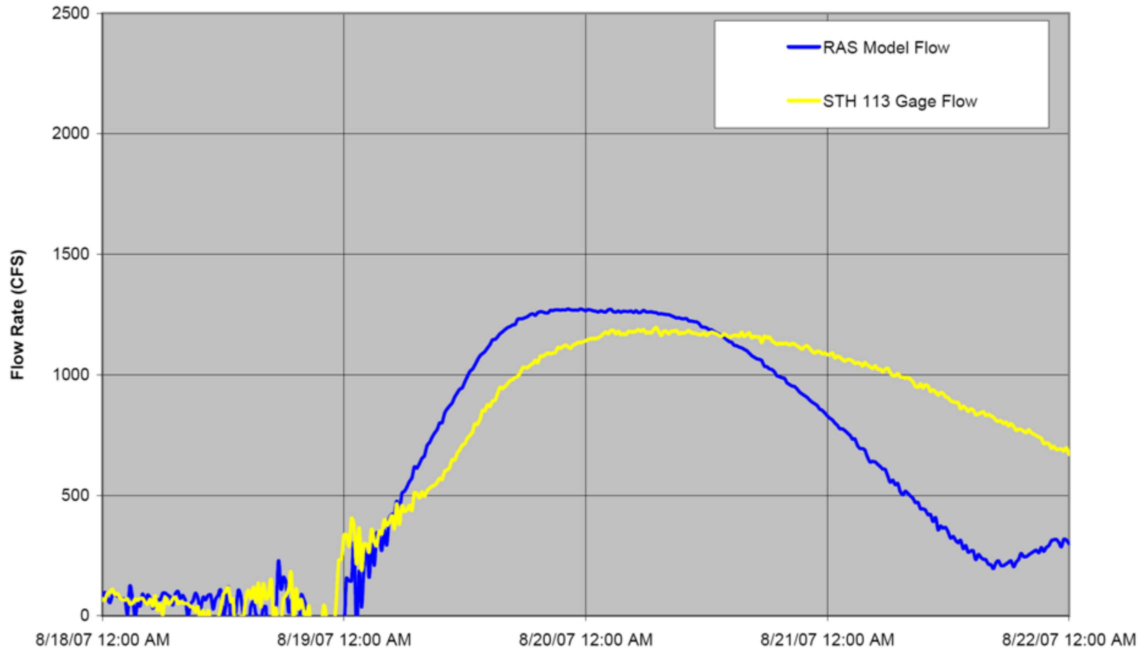

Figure 5: HEC-RAS Model vs. gage flow rate for August 2007 at the STH 113 Gage

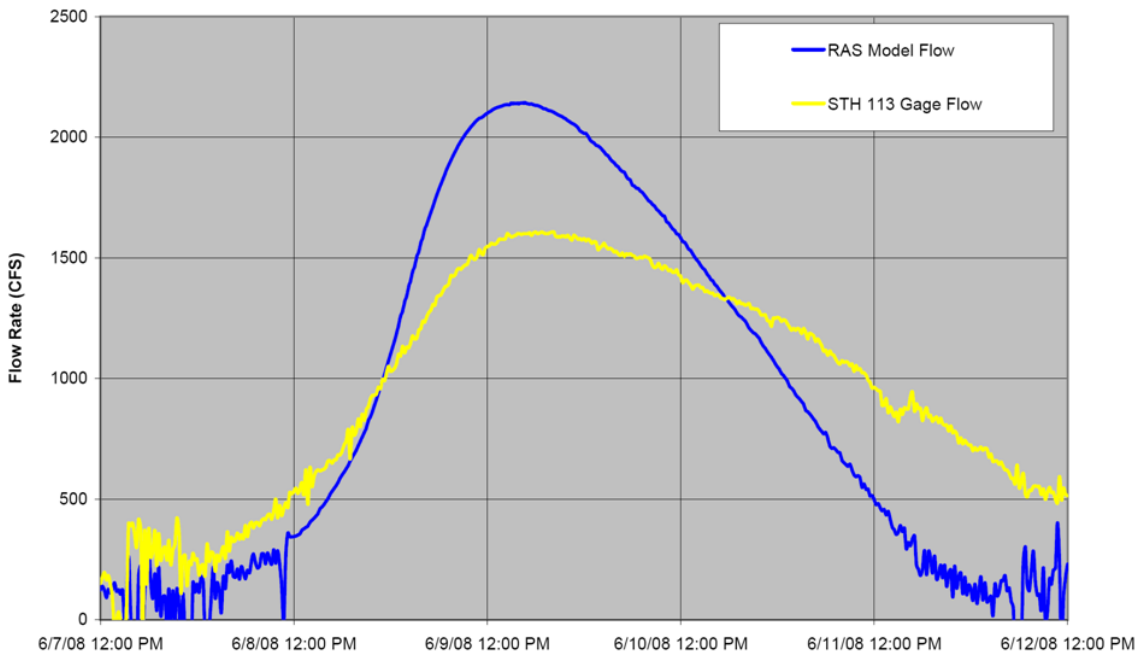

Figure 6: HEC-RAS Model vs. gage flow rate for June 2008 at the STH 113 Gage

Table 4 and Table 5 compare the modeled flow rates and volumes to the gaged flow rates and volumes for each of the events.

Table 4: Gage vs. Model Volume Comparison

Description	Windsor Gage		STH 113 Gage	
	August 2007	June 2008	August 2007	June 2008
Flow Volume through Gage	2,066 acre-feet	6,230 acre-feet	6,136 acre-feet	8,687 acre-feet
Flow Volume from Model	2,443 acre-feet	4,986 acre-feet	5,379 acre-feet	9,130 acre-feet
Ratio of model:gage (%)	118%	80%	88%	105%

Table 5: Gage vs. Model Peak Flow Comparison

Description	Windsor Gage		STH 113 Gage	
	August 2007	June 2008	August 2007	June 2008
Peak Flow Rate through Gage	1148 CFS	3198 CFS	1197 CFS	1607 CFS
Peak Flow Rate from Model	1058 CFS	3170 CFS	1273 CFS	2142 CFS
Ratio of model:gage (%)	92%	99%	106%	133%

It can be seen from above that there is good agreement between the model and gage for both volume and peak flow. The exception is the peak flow from the 2008 event at the STH 113 gage, where the modeled peak flow is 33% higher than the gaged peak flow. Additionally, the shape of the modeled time-series plots (hydrographs) are generally wider than the gage hydrographs; modeled hydrographs also have more rapidly descending trailing limbs than the gage hydrographs. While runoff volumes for both the 2007 and 2008 floods differed by up to 20%, for the 2007 event the model over predicted the volume at Windsor and under predicted at STH 113 while the reverse was true for the 2008 event. This cannot be rectified through further calibration of RCNs in the model, and can only be attributed to the degree of natural variability in runoff generation that any given watershed has from one storm to the next. Modeled peak flow rates for both flood events at the Windsor gage slightly under predicted the gage values, but were within 10%. However, modeled results for the 2007 flood event at the STH 113 gage slightly overpredicted the gage values by about 6%. These results are satisfactory. It is speculated that there is some floodplain storage in areas of the watershed upstream from the Village of Deforest that are not accounted for in the model. These storage areas attenuate the hydrograph shape beyond the ability of the model's reach-routing methods to reproduce. The sharp peaks in the Windsor gage record are likely from the urban watershed of Deforest. These peaks would not appear if the storage areas were between Deforest and the gage. So, it is most likely these storage areas are above the urbanized area of Deforest.

Additionally, under very high rainfall events such as June 2008, other storage areas may become active that do not exist in more moderate events such as August 2007. These areas (perhaps in the reaches of Token Creek on either side of STH 19, or the reaches of the Yahara River upstream of STH 19) would serve to dampen out the peak flow into the Cherokee Marsh and therefore also reduce the rate of flow at STH 113.

HEC-RAS Hydraulic Model

The HEC-RAS model was originally created for the 2008 Dane County floodplain mapping project. Under the current MSA study, substantial additional detail was added to the model using a combination of LIDAR, bathymetry, and river channel cross section survey data. The analysis of this date revealed that the river has three different segments of the stream that likely exhibit similar channel roughness and transition losses within them:

• Segment 1: Lake Mendota upstream to 3,000 feet downstream of Westport Road. In this reach, the river is wide, non-meandering, and the banks generally have marshy areas.

• Segment 2: 3,000 feet downstream of Westport Road to just upstream of STH 113. In this reach, the river is narrow and meandering, and contains four bridge crossings in close proximity. Also, the banks generally contain boat slips and dense urban development.

• Segment 3: Upstream of STH 113 to the confluence with Token Creek. This is within Cherokee Marsh. In this reach, the river is wide, shallow, non-meandering, and the banks generally have marshy areas. This reach lies outside of the model area being calibrated. Since the characteristics are similar to Segment 1 the final model for the river will apply the calibrated Segment 1 variables to Segment 3.

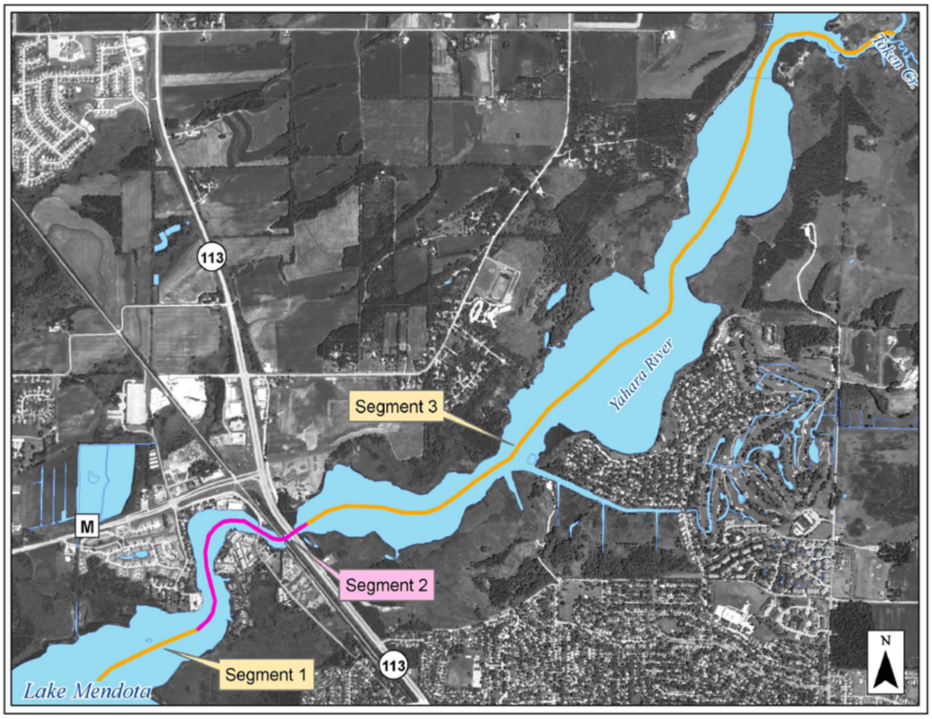

Figure 7: Model Stream Segments

The majority of input data describing the physical channel in an HEC-RAS model are directly measured and not subject to much user interpretation. Other values, such as the location of ineffective flow areas are dictated by well-established modeling protocols; however, there are two parameters which fall to the discretion of the model builder. These include Manning's roughness coefficient (n) and the expansion/contraction loss coefficients. Calibration of the model began by assigning typical values for n and expansion/contraction losses were inserted into the model. The first and third stream segments listed above were very similar in nature, and so identical parameters were used for each. The second stream segment was assigned separate parameters. Table 6 lists the parameter ranges and typical values for the described stream segments.

Table 6 Typical Stream Model Parameter Ranges

Parameter	Segment 1			Segment 2		
	Description	Typical	Range	Description	Typical	Range
Channel n[1]	Sluggish reaches, weedy	0.070	0.060-0.080	Clean and winding, some pools and shoals	0.040	0.033-0.045
Overbank n	Marsh grass and brush	0.100	0.070-0.160	Grass lawns with some trees	0.060	0.050-0.080
Contraction Loss[2]	Gradual Transitions	0.1	0.1 – 0.6	Bridge Transitions	0.3	0.1 – 0.6
Expansion Loss	Gradual Transitions	0.1	0.1 – 0.8	Bridge Transitions	0.5	0.1 – 0.8

HEC-RAS Boundary Conditions

The USGS operates stream flow gages at STH 113 (#05427850) and at the dam on Lake Mendota (#05428000). Information is logged every five minutes at the STH 113 gage, and includes water surface elevation and flow rate. The Lake Mendota gage records only water surface elevation, only at fifteen minute increments. Note: both gages are tied to NGVD 29 datum, which for the purposes of this study has been converted to NAVD 88 datum to match topographical data using the Countywide conversion of -0.2 feet. Figures 8 and 9 show time-series plots of the gage elevations for August 2007 and June 2008. Under normal conditions the difference in elevation between STH 113 and the Lake Mendota dam is approximately 0.2 feet.

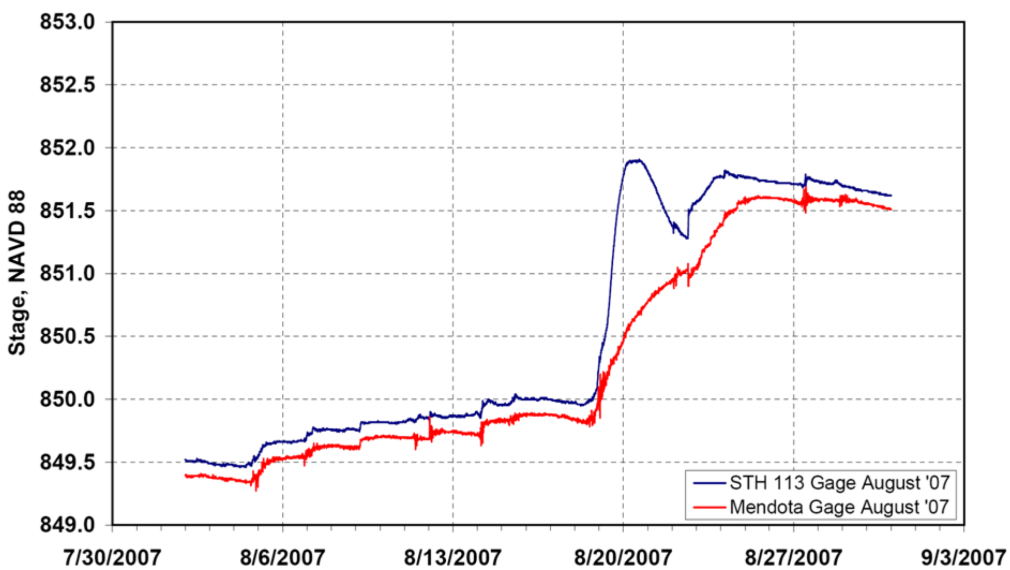

Figure 8: August 2007 Gage Data

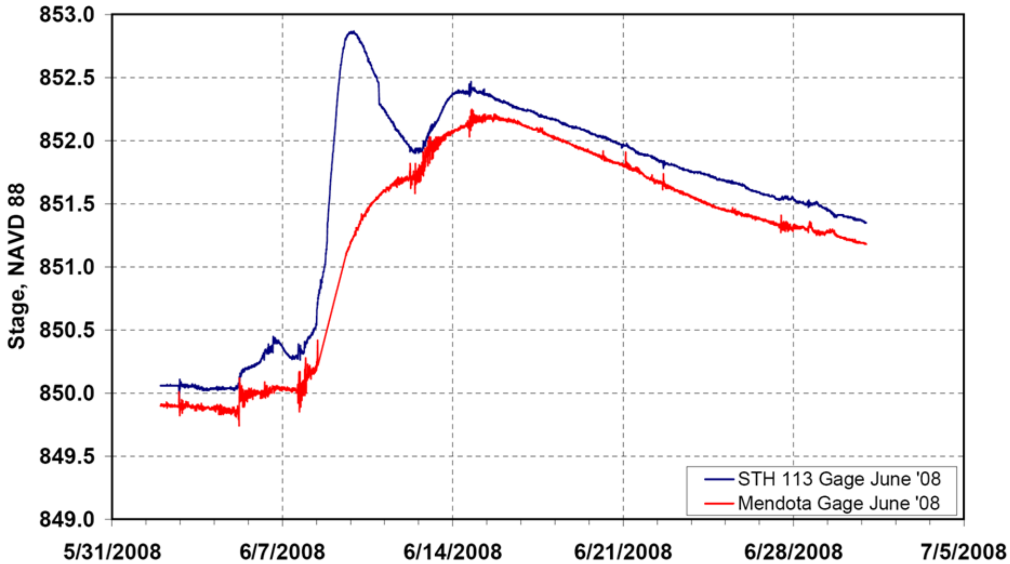

Figure 9: June 2008 Gage Data

Under wet weather conditions the difference in elevation is much greater. For the purposes of the HEC-RAS model calibration it was assumed that all of the losses occur between STH 113 and the upper end of the lake, and that the lake itself has a constant elevation. Figure 10 and Figure 11 show the recorded flow rate versus water surface elevation at STH 113 for August 19 to 29, 2007 and June 5 to 15, 2008, respectively. Several portions of the plot associated with lower flow rates (approximately 600CFS or less, with a few exceptions) show a non-linear relationship between flow rate and elevation. This occurs when the lake and marsh have very little elevation difference. Portions of the plot with higher flow rates are associated with storm events and exhibit a more linear relationship between flow rate and elevation. These linear portions of the plot represent relatively stable positive flow conditions, where a single flow rate can be assocated with a single water surface elevation with reasonable certainty. In these ranges, fifteen separate time points with a recorded flow rate, recorded water elevation at STH 113, and recorded water elevation in Lake Mendota were chosen as calibration check points. These points are highlighted on Figures 10 and 11 and in Table 7.

Table 7: Calibrated Boundary Conditions

	Date/Time	Mendota Gage Elev [1]	Flow Rate (CFS)	STH 113 Gage Elev [1]
1	8/19/07 09:00	850.20	542	850.66
2	8/19/07 12:30	850.27	749	851.02
3	8/19/07 18:00	850.35	1030	851.48
4	8/20/07 08:00	850.61	1180	851.89
5	8/21/07 08:00	850.84	989	851.68
6	8/21/07 19:00	850.92	774	851.46
7	8/22/07 01:00	850.96	675	851.36
8	6/8/08 17:00	850.41	662	850.98
9	6/8/08 22:00	850.50	931	851.20
10	6/9/08 03:30	850.74	1200	852.05
11	6/9/08 08:00	850.88	1430	852.48
12	6/9/08 19:30	851.20	1600	852.86
13	6/10/08 13:30	851.47	1370	852.62
14	6/11/08 08:30	851.63	1070	852.18
15	6/11/08 19:30	851.65	834	852.05

1 Gage data is collected as NGVD 29 datum but is shown here converted to NAVD 88 datum.

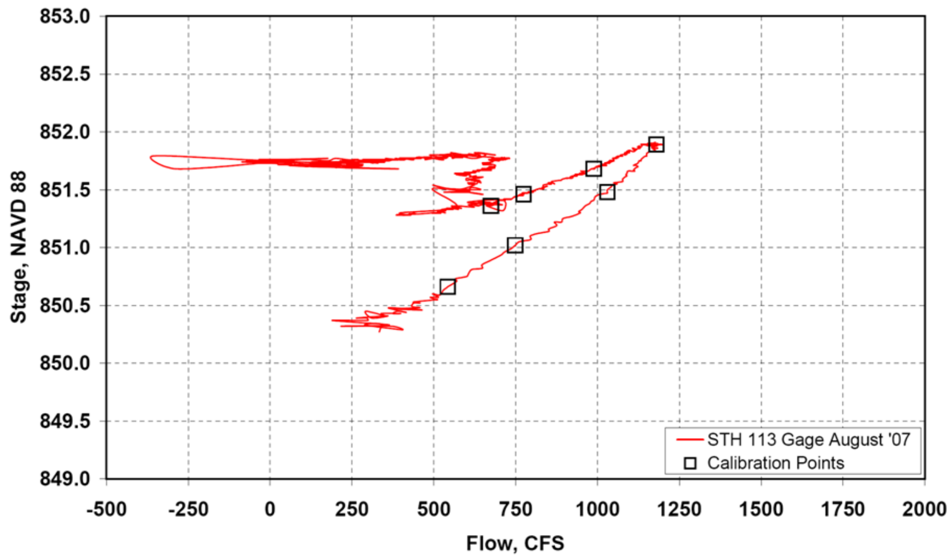

Figure 10: August 19-29, 2007 Stage-Discharge Plot

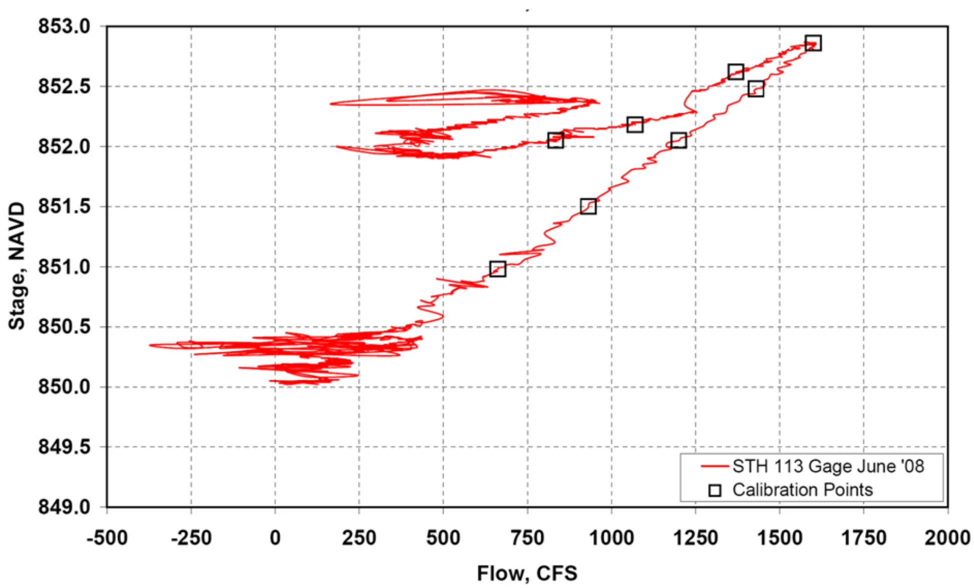

Figure 11: June 5-15, 2008 Stage-Discharge Plot

The Mendota gage elevation was assigned as the downstream boundary condition for each particular model simulation. The corresponding flow rate at STH 113 was assigned as an upstream boundary condition. The elevation at STH 113 was used as a comparison for model output to verify the accuracy of the calibration.

HEC-RAS Bridge Modeling Approach

Four main modeling approaches are available in HEC-RAS for modeling bridge losses. They include the energy method, the momentum method, the Yarnell method, and the WSPRO method. Each approach uses different input variables and components of potential energy losses (piers, abutment types, etc.) The use of any given method could produce valid results that differ from another method's valid results. While these differences are usually small, they can be several tenths of a foot, which can be enough to affect a model calibration. Therefore, as part of the calibration of this model, four different versions of the base geometry were created – one for each bridge modeling methodology.

After solving each model it was discovered that:

- The energy method produced valid results for each bridge in the model;
- The momentum method produced valid results for two of the bridges, but defaulted to the energy method for the other two as a valid solution to the momentum equations could not be found;
- The Yarnell method produced valid results for two of the bridges, but defaulted to the energy method for the other two as a valid solution to the momentum equations could not be found; and
- The WSPRO method produced no valid results. It did not default to any other method and gave no water surface elevations as output.

Each of the runs (excluding the WSPRO run) generated results within a few hundredths of a foot of one another. Since no particular method appeared to be any more or less applicable than another, and due to the energy method's consistently valid results, the energy method was used throughout the remainder of the calibration process.

HEC-RAS Calibration

Several sensitivity analyses were performed to determine to what extent the two variable parameters influenced the model results at STH 113. These tests consisted of the following steps:

1. Solving the model where typical values for Manning's n and transition losses were applied.

2. Comparing the model results to the gage record at STH 113.

3. Modifying the parameters to different values in an attempt to more closely match the gage record and repeating the run.

4. Reiterating the second and third steps until the model results substantially matched the gage record (within ~0.1').

5. This process was repeated for each of the 15 boundary condition data sets.

It was found that the model run based on initial assumptions of Manning's-n and expansion/contraction losses produced significantly lower water elevations than the STH 113 gage record for each of the 15 boundary condition data sets, indicating that one or both of the variables needed to be increased. Even after both parameters were increased to the upper limits of the typically-accepted range, it was found that resulting water elevations were still consistently lower than recorded gage values. The final calibrated runs required application of some values outside the typically-accepted range. Table 8 lists the parameter values used in each of the runs discussed above.

Table 8: Calibrated Model Variable Values

Parameter	Segment 1			Segment 2		
	Typical Value	Upper Value in Range (see Table 1)	Calibrated Value	Typical Value	Upper Value in Range (see Table 1)	Calibrated Value
Channel n	0.050	0.080	0.080	0.040	0.045	0.070[1]
Overbank n	0.100	0.160	0.120	0.040	0.080	0.085[1]
Contraction Loss	0.1	0.6	0.1	0.3	0.6	0.6
Expansion Loss	0.3	0.8	0.3	0.5	0.8	0.8

1 Calibrated value outside the limits of typically-accepted values.

Table 10 in the following section provides a comparative analysis of the calibration of the HEC-RAS model. Note that this calibration is not the preferred method. The preferred method is discussed in the following section. While the parameter values shown in the "Calibrated Value" column are not unrealistic, most are higher than the upper value in the ranges given in Table 6. Therefore, it is possible that other influencing factors are present. Two such possible factors were explored and are discussed below.

Factor 1: Bridge Model vs. Culvert Model

The railroad bridge is a clear span with vertical abutments and a transitioned wingwall – not unlike a culvert with a very large vertical dimension. To check this as a possible source of model variance, the base geometry was modified to include this as a culvert instead. The culvert model results varied by only a few hundredths of a foot compared to the bridge model results, so this was rejected as an influencing factor.

Factor 2: Boat Slips

The reach between the southbound STH 113 bridge and Lake Mendota has a significant number of boat slips serving residences and marinas. Aerial photographs show that in many locations, these structures span more than half of the river width. To account for these structures, the following geometry modifications were considered:

1. Modeling the areas where slips were present as 'ineffective,' essentially assuming that no flow was able to occur in these areas. The use of 'ineffective' areas was rejected almost immediately, as it is reasonable to assume that some amount of flow occurs below and between the slip and docked boats.

2. Modeling the areas where slips were present with a Manning's n of 0.15, similar to the effects of a very dense stand of timber. The use of a high Manning's n was rejected as it was determined by several model tests that the values would need to be several times higher than the originally-assumed 0.15 value and that the use of this method would only account for friction along the bottom of the bed and not against the structure or boats.

3. Modeling the areas where slips were present as having 'lids,' or areas of blocked flow of some thickness at a point somewhere above the channel bottom, similar to a bridge deck. While still somewhat inexact (for instance, the thickness was based on an assumption about the depth of draft of the moored boats, and there is no way of accounting for the flow between boats, or for the rising and falling of the structure as the water level fluctuates), this appeared to be the most viable due to its ability to seemingly reasonably accommodate measureable input parameters.
Ultimately, it was decided that the slips would be modeled as 'lids' having four feet of thickness extending from approximately one foot above the top of bank elevation to three feet below the top of bank elevation. Table 9 lists the parameter values used in the model run containing the 'lids' concept.

Table 9: Calibrated Model Variable Values

Parameter	Segment 1			Segment 2		
	Typical Value	Upper Value in Range (see Table 1)	Calibrated Value	Typical Value	Upper Value in Range (see Table 1)	Calibrated Value
Channel n	0.070	0.080	0.070	0.040	0.045	0.045
Overbank n	0.100	0.160	0.100	0.060	0.080	0.070
Contraction Loss	0.1	0.6	0.1	0.3	0.5	0.6
Expansion Loss	0.3	0.8	0.3	0.5	0.7	0.8

Using the concept of 'lids' for the boat slips, and varying both 'n' and transition loss coefficients to somewhat higher values than used in the original base geometry but within generally accepted ranges of values, a favorable match between modeled and gage water surface elevations at STH 113 was found.

HEC-RAS Final Discussion

Two possible model conditions were found which produced favorable results when compared to gage data at STH 113:

• Matching Condition #1: Use of model n-values and expansion/contraction coefficients outside of generally-published ranges for observed channel conditions
(Table 8).

• Matching Condition #2: Use of model n-values and expansion/contraction coefficients within generally-published ranges for observed channel conditions, plus model considerations for presence of boat slips (Table 9).

Table 10 compares model results with gage data for both matching model runs.

Table 10: Model Results

Calibration Point	STH 113 Gage Elev	Matching Condition #1		Matching Condition #2	
		WS Elev	Difference	WS Elev	Difference
8/19/07 09:00	850.66	850.59	-0.07	850.54	-0.12
8/19/07 12:30	851.02	850.95	-0.07	850.87	-0.15
8/19/07 18:00	851.48	851.48	0.00	851.36	-0.10
8/20/07 08:00	851.89	851.88	-0.01	851.84	-0.05
8/21/07 08:00	851.68	851.70	+0.02	851.72	+0.04
8/21/07 19:00	851.46	851.46	0.00	851.48	+0.02
8/22/07 01:00	851.36	851.37	+0.01	851.39	+0.03
6/8/08 17:00	850.98	850.92	-0.06	850.86	-0.12
6/8/08 22:00	851.20	851.39	+0.19	851.31	+0.11
6/9/08 03:30	852.05	851.98	-0.07	852.00	-0.05
6/9/08 08:00	852.48	852.44	-0.04	852.47	-0.01
6/9/08 19:30	852.86	852.87	+0.01	852.88	+0.02
6/10/08 13:30	852.62	852.65	+0.03	852.65	+0.03
6/11/08 08:30	852.18	852.36	+0.18	852.36	+0.18
6/11/08 19:30	852.05	852.11	+0.06	852.11	+0.06
AVERAGE DIFFERENCE		-----	+0.012	-----	-0.007
STD DEVIATION OF DIFFERENCE		-----	+/- 0.081	-----	+/- 0.092

While both conditions produce good results, the use of out-of-range coefficients required by Matching Condition #1 does not appear to be justifiable in the presence of another measureable and observeable condition such as the boat slips. It is MSA's position that the model which contains parameters describing the presence of boat slips (Matching Condition #2) is the most accurate hydraulic representation of the Yahara River between STH 113 and Lake Mendota.

Project Summary

The development of a hydrology model (HEC-HMS) to predict discharges and a hydraulic model (HEC-RAS) to predict in stream water elevations is critical in order to evaluate flood conditions and for scenario testing.

Scenario Modeling

The differences between calibrated flow rates and regulatory flow rates, combined with the possible differences between lake levels at the time of peak flow in the river yield eight different possible steady state flow conditions. These eight conditions are represented by the chart in Figure 12.

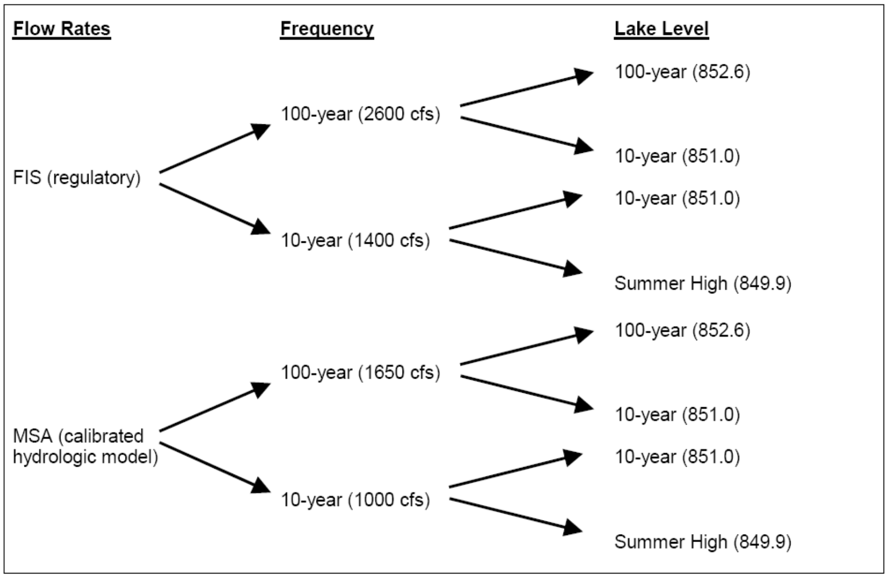

Figure 12: Scenario Simulations

Scenario Simulations

The hydraulic model HEC-RAS pairs a single geometry with a set of one or more flow conditions to produce a model run. The eight geometry scenarios described above were each paired with the eight possible flow conditions described above. This yielded sixty-four separate results. The basis of comparison for these models was to examine the water surface elevation (WSEL) at the cross section upstream of STH 113 (RS 183638.3) for every geometry scenario at a given flow rate. Table 2, below, summarizes the results for the current regulatory FEMA flow conditions (100-year and 10-year flows, matched with 100-year and 10-year lake levels) and for the flow conditions determined by MSA's hydrologic study (100-year and 10-year flows, matched with 10-year and summer maximum lake levels) for the geometries where all bridges are present, and where all are removed. The elevation differences between the pairs of bridge scenarios represent the maximum possible reductions in marsh flood elevations achievable through structure modifications.

Table 11: Summary of Maximum Possible Flood Elevation Reduction

Flood Condition	Flow Rate	Lake Level Recurrence Interval	Bridge Scenario	Cherokee Marsh WSEL
FEMA 100-year	2,600 CFS	100-year	All Bridges In[1]	854.69[1]
			All Bridges Out	853.48
FEMA 10-year	1,400 CFS	10-year	All Bridges In	852.39
			All Bridges Out	851.87
MSA 100-year	1,650 CFS	10-year	All Bridges In[2]	852.77[2]
			All Bridges Out	852.10
MSA 10-year	1,000 CFS	Summer Maximum	All Bridges In	850.97
			All Bridges Out	850.60

1Represents 100-year flood elevation using current FEMA flow conditions and calibrated hydraulic model of river.
2Represents 100-year flood elevation if MSA hydrologic model is adopted and used with calibrated hydraulic model of river.

Table 12 summarizes the results for the same flood conditions as in Table 11, but for each of the geometries where a single bridge is removed. For each case where a single bridge was removed, the removal of the STH 113 bridges presented the greatest reduction in water surface elevation. Therefore, it appears that the STH 113 bridges are the greatest impediment to flow. Additionally, the removal of these bridges represented 50% or more of the maximum possible reduction achievable; therefore, an improvement to STH 113 has the most potential for reducing flood elevations in Cherokee Marsh.

Table 12: Summary of Flood Elevation Reductions for Single Bridge Improvements

Flood Condition	Bridge Scenario	Cherokee Marsh WSEL	Marsh WSEL Reduction[1]	% of Max Potential Reduction[2]
FEMA 100-year	STH 113 Out	853.99	0.70 feet	58%
	Railroad Out	854.53	0.16 feet	13%
	Westport Out	854.48	0.21 feet	17%
FEMA 10-year	STH 113 Out	852.00	0.39 feet	75%
	Railroad Out	852.33	0.06 feet	12%
	Westport Out	852.35	0.04 feet	8%
MSA 100-year	STH 113 Out	852.29	0.48 feet	72%
	Railroad Out	852.69	0.08 feet	12%
	Westport Out	852.70	0.07 feet	10%
MSA 10-year	STH 113 Out	850.63	0.34 feet	92%
	Railroad Out	850.95	0.02 feet	5%
	Westport Out	850.97	0.00 feet	0%

1Reduction as compared to “All Bridges In” scenario from Table 2.
2Reduction as compared to difference in WSELs between scenarios from Table 2.

While not summarized here, similar results occur for the scenarios where two bridges are removed – the scenarios where only the STH 113 bridges remain exhibit the least reduction in marsh WSEL.

Findings

To explain the likely reason for the obstruction that STH 113 presents as compared to the other bridges, MSA conducted an examination of the bridge geometries. The bridge data shows that although the STH 113 bridges have the widest deck span (about 90 feet long), they are also the only bridges that have a trapezoidal river cross section below them and have a narrow river bottom (about 30 feet wide). The railroad and Westport Road bridges are not as long (spans of about 50 and 60 feet, respectively) but have vertical abutments which make the river bed just as wide at the bottom of the structure as it is at the top. If the channel through the STH 113 bridges were modified so that it had a more rectangular cross section approximately 60 feet wide, it is likely that the structure would behave similarly to the railroad and Westport Road bridges.