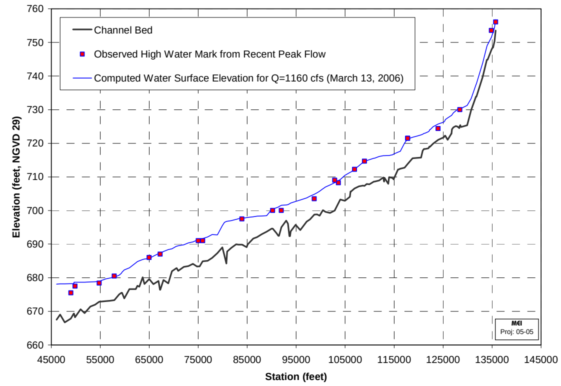
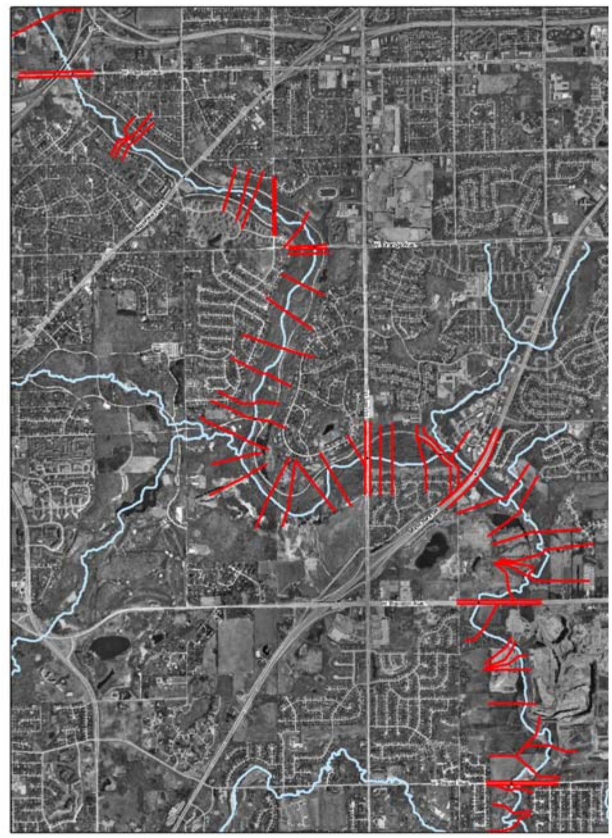
Project Code: P5000

TECHNICAL MEMORANDUM

Hydraulics

Root River Sediment-transport Planning Study
Contract No. W30003P01





Submitted to: **Milwaukee Metropolitan Sewerage District**
260 West Seeboth Street
Milwaukee, Wisconsin 53204

Preserving The Environment • Improving Water Quality

Submitted by: **Mussetter Engineering, Inc.**

Mainstream Restoration, Inc. DTM CONSULTING T N & Associates, Inc. Engineering and Science

September 7, 2007

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1. INTRODUCTION AND BACKGROUND

A hydraulic analysis of the approximately 17-mile long project reach of the North Branch Root River from the confluence with the Root River canal upstream to the Milwaukee-Waukesha County Line (South 124th Street) (**Figure 1**) was carried out by Mussetter Engineering, Inc. (MEI) to assess the channel capacity along the project reach and to develop input for the sediment-transport analysis. The work was performed as part of the Milwaukee Metropolitan Sewerage Districts (MMSD) Root River Sediment Transport Planning Study. In performing this analysis, an existing hydraulic model of the project reach was obtained and reviewed, and appropriate portions of this model were combined with new cross-section and bridge data to develop a new model that represents existing topographic and roughness conditions along the reach. Both the new and existing models were developed using the Corps of Engineers HEC-RAS computer software (USACE, 2005), a widely accepted one-dimensional (1-D) step-backwater program.

2. EXISTING HYDRAULIC MODEL

The existing HEC-RAS model was developed for the Phase 1 Watercourse Management Plan (WMP) (CDM, 2000) by combining several available HEC-2 files into a single, comprehensive HEC-RAS model of the reach. (HEC-2 is a DOS-based step-backwater program that was the precursor to the Windows-based HEC-RAS program.) The original HEC-2 files were developed by the Southeastern Wisconsin Regional Planning Commission (SEWRPC) using 1985 topography. The final HEC-RAS model included 151 cross sections and 22 duplicated cross sections at an average spacing of about 510 feet. The bridges that were included in the original HEC-2 files were also incorporated into the HEC-RAS model, and updated, as appropriate, using as-built plans to reflect to changes since 1985. Hydraulic roughness was modeled using Manning's n values that ranged from 0.035 to 0.058 in the main channel and 0.03 to 0.13 in the overbanks. The downstream boundary of the model was located near the Root River Canal confluence, and a constant water-surface elevation of 681.8 feet was used as the downstream boundary condition for the range of modeled flows. A more complete description of the model is presented in Appendix B of the Phase 1 WMP (CDM, 2000).

3. REVISED HYDRAULIC MODEL

A revised HEC-RAS model (**Appendix A**) was developed to incorporate new topographic and bathymetric data collected by MMSD surveyors specifically for this project. The revised model better represents the existing channel geometry, and therefore, should provide better estimates of existing hydraulic characteristics in the project reach than the previous model. The river stationing in the model is based on a mathematically computed station line that follows the river alignment from the confluence with Crayfish Creek (about 8.5 miles below the downstream limit of the project reach) to the upstream model limit at the Milwaukee-Waukesha County Line. The model, thus, extends from Sta 460+68 to Sta 1357+48, a river distance of 17 miles.

3.1. Model Development

3.1.1. Geometric Data

The geometry for the model is based on 118 cross sections that were collected by MMSD surveyors between September 2005 and April 2006, at an average cross-sectional spacing of about 760 feet (see Technical Memorandum: Surveying; Figure 1). To ensure the model can convey flows up to the 100-year event, the surveyed cross sections were extended into the overbanks using the available 2-foot contour interval mapping of Milwaukee County. This mapping was developed using stereo-photogrammetric methods with aerial photography taken on April 26, 1993, and March 26, 1999. Both the survey data and the topographic mapping are referenced to the Wisconsin State Plane, South Zone horizontal datum (NAD 19727) and the National Geodetic Vertical Datum of 1929 (NGVD29). In addition to the cross sections that were surveyed for this project, seven sections were developed for the reach between Oklahoma Avenue and South 116th Street using survey data that was collected by MMSD during early 2005 for the West Allis—National Avenue and Oklahoma Avenue Flood Abatement Alternatives Analysis (MMSD, 2004). These data were referenced to the City of Milwaukee vertical datum that was converted to NGVD29 by adding a constant value of 580.603 feet.

3.1.2. Bridge Geometries

Bridge geometries were incorporated into the HEC-RAS model based on data from in the previously developed HEC-RAS model (CDM, 2000), and updated with the most recent as-built bridge plans that were obtained by MEI from the Wisconsin Department of Transportation (**Table 1**). Since as-built plans were not available for the West Lincoln Avenue box culvert, the Root River Parkway Bridges, the West Drexel Avenue culverts, and the West Oakwood Road culverts, measurements taken during the field reconnaissance were used to validate the bridge geometries in the Phase 1 WMP model. Hydraulically significant bridges that were not included in the Phase 1 WMP HEC-RAS model were also added into the model. These bridges included the structures over the Root River at West Puetz Road, the pedestrian bridge near the mouth of the East Branch Root River and the West Lincoln Avenue box culverts. (The structures for South 124th Street and South 60th Street were not included in the Phase 1 WMP and MEI models since these structures are at the upstream and downstream model limits, respectively). The final version of the model contains 23 bridges and four culverts. The cross sectional geometries at the up- and downstream face of the bridges were based on the surveyed cross sections that were typically located within 10 feet of the upstream face of each bridge. When necessary, additional cross sections were added approximately one channel width up- and downstream from the bridge by interpolating between the adjacent surveyed cross sections to model flow contraction and expansion. Expansion and contraction coefficients of 0.5 and 0.3, respectively, were used in the vicinity of each of the bridges. Culvert crossings were modeled by determining the upstream energy grade for entrance and exit control conditions (with appropriate entrance and exit loss coefficients), and the solution with the highest energy grade was selected.

3.1.3. Other Input Data

Other input to the model included hydraulic roughness coefficients, overbank obstructions, and the downstream boundary condition. The hydraulic roughness was incorporated into the model using Manning's *n*-values. Main channel Manning's *n*-values ranged from 0.028 to 0.06, based on field observations and bed-material characteristics, past experience with similar streams, and published values for similar streams (Barnes, 1967; Hicks and Mason, 1991; Arcement and Schneider, 1989). Manning's *n*-values in the lower portion of the range (0.028 to 0.034) were used for low-energy, silt- and sand-bed reaches with minimal form roughness from the bed and banks. Moderate *n*-values ranging from 0.035 to 0.045 were used for reaches where the bed material is composed of coarse sand and gravel, where the bed and lower banks are moderately irregular, or where debris affects the flow. Higher n-values (0.046 to 0.060) were used for areas where the channel bed is coarse-grained, where the bed and banks are highly irregular, or where debris blocks the flow. Overbank roughness was modeled using a horizontal variation in Manning's *n*-value that allows for adjustment of the computed flow conveyance based on the type and density of vegetation and irregularity in the topography. Overbank *n*-values ranged from 0.030 in relatively flat, grassy areas with little or no woody vegetation to 0.150 in areas with dense forest composed of thick brush and/or significant woody debris.

Because many of the cross sections extend into overbank areas where structures could affect the flow at peak discharges, the structures were coded into the model using the HEC-RAS *blocked obstruction* option. In many cases, structures that could affect the flow were located a short distance up- or downstream from the cross-section line. At these locations, the obstruction associated with the structure was projected onto the affected cross section.

The downstream boundary condition for flow less than the 1 percent exceedence discharge on the mean daily flow duration curve (480 cfs below Ryan Creek) was established by assuming normal depth with an energy slope of 0.0006 (3.2 ft/m), based on the average bed slope in the vicinity of the downstream model limit. This slope is consistent with the water-surface slope at the time of the field surveys, and, as will be discussed in the following section, the resulting water-surface profiles match the measured high-water marks in the downstream portion of the reach reasonably well. The Stormwater Drainage and Flood Control System Plan (SEWRPC, 1990) indicates that significant backwater occurs at the downstream boundary of the model at flows above the 10-year event due to the constriction caused by the South 60th Street Bridge and other topographic features discussed in the Geomorphology Technical Memorandum. The starting water-surface elevations for higher flows were, therefore, taken from Figure 34 in SEWRPC (1990). The backwater effects indicated in SEWRPC (1990) were observed at much lower flows during a field reconnaissance on March 22, 2007, when the provisional discharge at the USGS gage near Franklin (USGS gage number 04087220) indicated the discharge was 440 cfs (about the 1-percent exceedence discharge on the mean daily flow-duration curve). Based on the measured water surface elevation at the South 60th Street bridge, the starting water-surface elevations for flows between the 1 percent exceedence discharge and the 10-year event were estimated using a logarithmic-linear interpolation scheme.

3.2. Model Calibration

The model was calibrated, to the extent possible, by adjusting the Manning's *n*-values so that the predicted water-surface elevations matched observed high-water marks along the reach and measured water-surface elevations at the two stream gages that are located in the reach.

The USGS has taken 197 stream-gaging measurements at the Franklin gage, which is located about 400 feet upstream from the State Highway 100 (Ryan Road) Bridge, since 1987 at discharges ranging from less than 1 to 1,420 cfs for purposes of calibrating the gage-rating curve. The water-surface elevations for these measurements were determined by adding the gage datum (674.5 feet NGVD) to the reported outside gage height. Comparison of model results indicates very good agreement over the range of flows encompassed by the measurements (**Figure 2**).

A total of 24 stream-gaging measurements have also been taken since 2004 at the Grange Avenue gage. The datum for this gage is reported by the USGS to be 705 feet above NGVD of 1929 "from topographic map". A comparison of the predicted water-surface elevations with the rating curve (developed by adding 705 feet to the gage height measurements) indicates a discrepancy of about 9 feet, suggesting that the reported elevation is approximate. If the gage datum is shifted downward by 9 feet, the resulting rating curve matches the predicted water-surface elevations very well over the range of flows encompassed by the measurements (**Figure 3**). While this is not an ideal validation of the model results at this location, it does indicate the model accurately reproduces the correct trend of increasing water-surface elevation with increasing discharge. The validity of this comparison could be improved by obtaining an accurate elevation on the gage plate.

During the field mapping that was conducted in August 2005 and August 2006, high-water marks associated with recent flooding, such as grass, mud, or debris that is typically suspended above the ground surface by woody vegetation, were located, and their elevations determined by measuring their height above the channel bed. Since most of these high-water marks were identified during the 2006 field reconnaissance, they were assumed to be associated with the March 13, 2006, provisional peak discharge of 1,160 cfs at the Franklin gage. (Provisional peak discharges have not been reviewed by the USGS and are subject to change.) The corresponding peak discharges at other locations along the reach during this event were estimated by adjusting the Franklin gage peak to reflect inflows from the tributaries, based on the flow distribution that was developed for the hydrologic analysis (see Technical Memorandum: Hydrology; **Table 2**). Normal depth at a slope of 0.0006 was used as the downstream boundary condition for the 1,160 cfs calibration profile. With minor adjustments to the initially selected *n*-values, the computed water-surface profile for the March 2006 peak flow matches the recent high-water marks very well (**Figure 4**). The model results also indicate significant amounts of overbank flooding that would be consistent with a flood of this magnitude (approximately the 2.6-year event).

3.3. Model Results

The model was executed over a range of discharges from baseflows through the future conditions 100-year flood peak. The flow distribution along the project reach was established by adjusting the discharge to reflect tributary inflows based on the relationship developed in the hydrology analysis (see Technical Memorandum: Hydrology; **Tables 2 and 3**). The modeled water-surface profiles indicate that significant backwater occurs at discharges greater than the existing conditions 2-year peak flow in the reaches upstream from the Root River Canal (Subreach 10), West Rawson Avenue (Subreach 6), South 76th Street (Subreach 5), West College Avenue (Subreach 4), and West Forest Home Avenue (Subreach 3; **Figure 5**).

As expected, the model results indicate that hydraulic conditions (i.e., velocities, depths, main channel topwidths) vary considerably through the project reach based on the local slope, channel geometry, hydraulic roughness conditions, and downstream hydraulic controls (**Figures**

6 through 8). In spite of the large variability along the reach, channel capacity is relatively consistent, ranging from the 1 percent exceedence flow on the mean daily flow-duration curve to the 2-year peak discharge.

To facilitate the sediment-transport analysis, subreach-averaged hydraulic conditions were developed from the model output using the 10 subreaches that were identified in the geomorphic analysis (see Technical Memorandum: Geomorphology; **Table 4**). The results indicate that the average main channel hydraulic depth at the 2-year flood peak ranges from about 2.1 feet in Subreach 1 at the upstream end of the study reach to about 6.5 feet in Subreach 10, with a general trend of increasing depth with increasing drainage area in the upstream portion of the reach, and no consistent trend in the downstream portion of the reach (**Figure 9**). Main channel velocities are highest in Subreaches 1, 4 and 7. The high main channel velocities in Subreach 1 are due to the steep channel gradient, with average velocities ranging from 2.5 fps at the 2-year peak discharge to about 3.5 fps at the 100-year peak (**Figure 10**). Similar main channel velocities are indicated in the relatively steep reach in Subreach 4. In Subreach 7, high main channel velocities result from constricted terraces and range from 2.5 fps at the 2-year peak discharge to 4.8 fps at the 100-year peak, also occur in Subreach 7 that has been affected by gravel-mining. Relatively low main channel velocities (less than 1.2 fps for each of the peak discharges) occur in Subreach 6, due to the relatively low channel gradient and high hydraulic roughness that results from the presence of significant woody debris in this swamp-bound portion of the reach. Average main channel topwidth increase dramatically in the downstream direction from Subreach 1 to Subreach 3 due to the increase in contributing drainage area. At the 2-year peak discharge, for example, the average topwidth in Subreach 1 is only about 19 feet, increasing to about 35 feet in Subreach 2 and 51 feet in Subreach 3 (**Figure 11**). Topwidths are quite narrow in Subreach 4 (about 31 feet at the modeled flood peak flows) due to observed bank accretion in this area (see Technical Memorandum: Geomorphology). Similar to the velocity profiles, there is no significant trend in the average topwidths through the remainder of the reach, although Subreach 10 (the most downstream subreach) is somewhat narrower. The reasons for the variability in channel widths are discussed in more detail in the Geomorphology Technical Memorandum.

Under future conditions, the computed hydraulics demonstrate similar general trends to existing conditions, except the magnitudes of the hydraulic variables are typically slightly greater due to the higher discharges (**Figures 12 through 14**). Except for the large increase indicated at the 10-year event in Subreach 6, subreach-averaged hydraulic depths in the main channel increase by less than 7 percent throughout the project reach. Maximum increases in hydraulic depth occur in the backwater zones upstream, South 76th Street (Subreach 5) and West Rawson Avenue (Subreach 6), as well as the portion of Subreach 8 that is affected by the increased flows from the East Branch Root River (**Figure 15**). The largest increase in hydraulic depth typically occurs at the 2-year event because the relative increase in discharge from existing to future conditions is greatest at this recurrence interval. Main channel velocities generally increase by less than 5 percent from existing to future conditions (**Figure 16**). Decreases in main channel velocities occur at locations where the increased discharge results in greater backwater (Subreaches 3, 5 and 10).   Since the existing conditions 2- through 100-year peak flows are either close to or exceed the main channel capacity (and since the overall topwidth of the main channel is a geometric constant), the increase in main channel topwidth under future conditions is relatively insignificant (**Figure 17**).

3.4. Upstream Model

To assist in estimating the sediment supply to the upstream limit of the project reach, a less-detailed hydraulic model was developed for the portion of the North Branch upstream from the Milwaukee-Waukesha County Line. The model extended from about 800 feet upstream from South 124th Street to Sta 135487.8 (the second-most upstream cross section in the primary North Branch Model) with a total distance of about 1100 feet. A total of 12 cross sections were used in the model (including the two bridge sections for the S 124th Street culvert), resulting in an average cross section spacing of about 92. The downstream two cross sections in the model were based on the surveyed sections in the primary North Branch model (Sta 135487.8 and Sta 135748.1). For the upstream cross sections, since no surveyed cross sections were available in this area, the upstream surveyed cross section (Sta 135748.1) was duplicated and the elevations were adjusted to match the average bed slope of 0.0055 based on the available USGS quadrangle topography. The downstream boundary condition was obtained from the primary North Branch model, and similar expansion and contraction (loss) coefficients and roughness values were used. The South 124th Street culvert was modeled using geometry based on bridge plans obtained from the Wisconsin Department of Transportation and appropriate bridge modeling procedures. The model was executed over the range of future conditions discharges upstream from Hale Creek. Results from the hydraulic model are summarized in **Table 5**.

4. SUMMARY AND CONCLUSIONS

A hydraulic model was developed for the project reach using the Corps of Engineers HEC-RAS computer software (USACE, 2005), a one-dimensional (1-D) step-backwater program. The model geometry was based on survey data collected by MMSD surveyors and 2-foot contour mapping of Milwaukee County based on aerial photography collected between April 1993 and March 1999. Hydrologic input to the model was based on results from the hydrologic analysis (see Technical Memorandum: Hydrology). Other model input (hydraulic roughness and downstream boundary conditions) were based on field observations and engineering judgment. The model results indicate the hydraulic characteristics vary significantly along the project reach, and are affected in several locations by infrastructure. The results also indicate that the bankfull channel capacity along the reach is between the 1-percent exceedence flow on the mean daily flow-duration curve and the 2-year peak flow. This issue is discussed in more detail in the geomorphic analysis.

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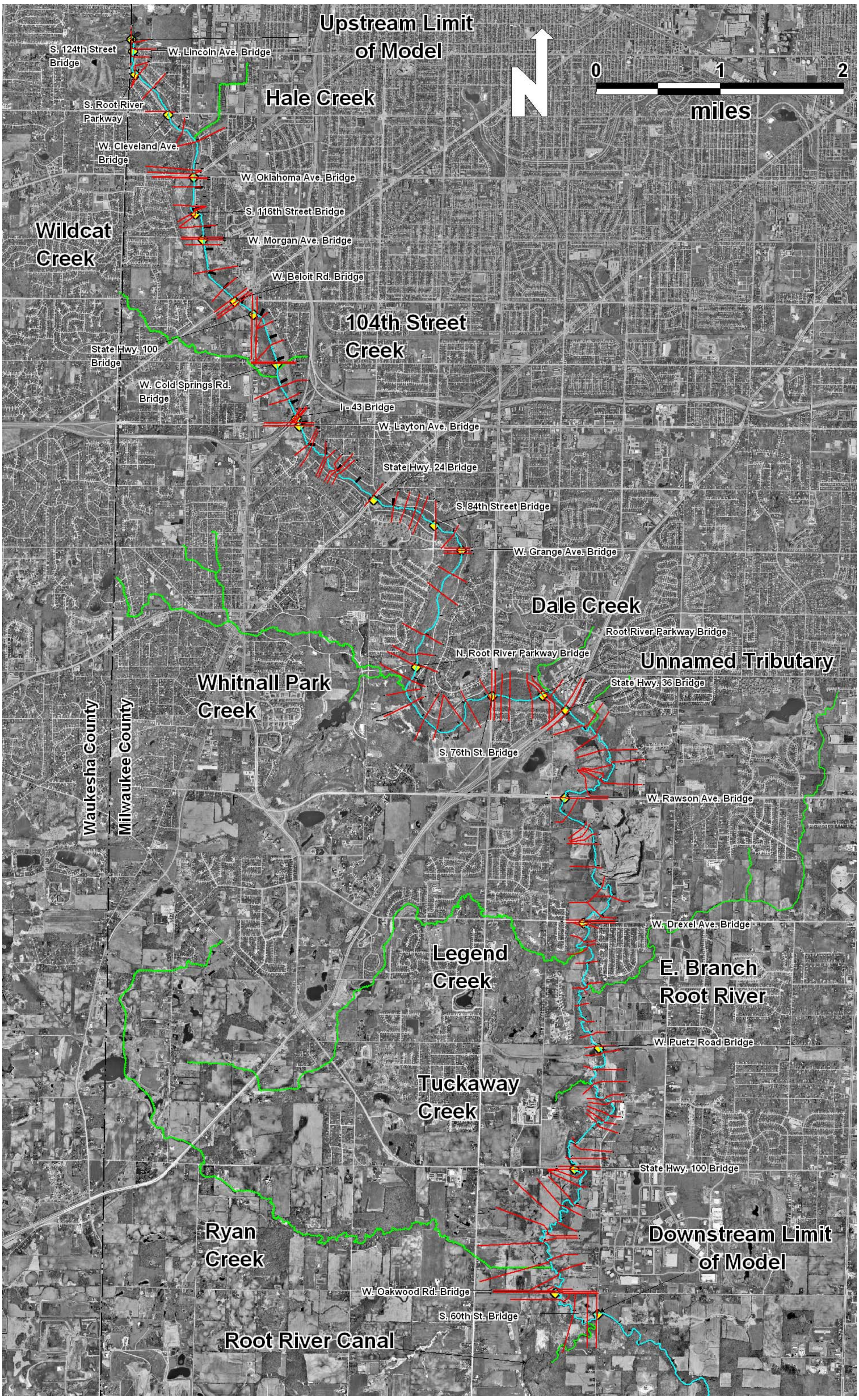


Figure 1. Aerial photograph showing the limits of the study reach, and the location of the cross sections that were used in the analysis.

Table 1. Summary of modeled bridge geometries.

Bridge or Culvert	Bridge Plans		Field Measurements Used?	Comment
	Available?	Date		
South 124th Street	No	-	No	Not included in MEI model because at upstream limit of model; not included in Phase 1 WMP model
W Lincoln Ave (box culvert)	No	-	Yes	Model geometry based on survey data and field measurements; not included in Phase 1 WMP model
Root River Parkway Bridge	No	-	Yes	Model geometry from MMSD model and field observations
W Cleveland Ave (culverts)	Yes	1995	No	Model geometry from bridge plans
National Ave/Oklahoma Ave	Yes	1968	Yes	Model geometry from bridge plans
South 116th St Brg	Yes	1995	No	Model geometry from bridge plans
W Morgan Ave	Yes	1995	No	Model geometry from bridge plans
W Beloit Road	Yes	2003	No	Model geometry from bridge plans
South 108th St (State HWY 100)	Yes	1968	No	Model geometry from bridge plans
W Cold Spring Rd	Yes	1994	No	Model geometry from bridge plans
I-43 SB, from Bridge Plans	Yes	1992	No	Model geometry from bridge plans
I-43 NB, From Bridge Plans	Yes	1992	No	Model geometry from bridge plans
W Layton Avenue	Yes	1999	No	Model geometry from MMSD model, updated with bridge plans
Abandoned RR Bridge	No	-	Yes	Model geometry from MMSD model and field observations
West Forest Home Ave (State HWY 100)	Yes	1963	No	Model geometry from MMSD model, updated with bridge plans
Parkway Bridge (DOT Whitnall Park Road)	No	-	Yes	Model geometry from MMSD model, bridge inspection and field observations
S. 84th Street	Yes	1993	No	Model geometry from bridge plans
W Grange Ave	Yes	1979	No	Model geometry from bridge plans
W College Avenue	Yes	2005	No	Model geometry from bridge plans
South 76th Street	Yes	1992	No	Model geometry from Phase 1 WMP model (same as bridge plans)
Root River Parkway	No	-	Yes	Model geometry from field measurements and Phase 1 WMP model
West Loomis Road (State HWY 36)	Yes	1968, 198[illegible]	No	Model geometry from Phase 1 WMP model (same as bridge plans)
West Rawson Street	Yes	1998	No	Model geometry from bridge plans
West Drexel Avenue (culverts)	No	-	Yes	Phase 1 WMP model included bridge and not five culverts
Pedestrian near confluence with East Branch Root River	No	-	Yes	Model geometry based on field measurements; not included in Phase 1 WMP model
W Puetz Road	Yes	1997	Yes	Not included in Phase 1 WMP model
W Ryan Rd (State HWY 100)	Yes	1987	No	Updated Phase 1 WMP model geometry with bridge plans
W Oakwood Road (culverts)	No	-	Yes	Model geometry based on bridge inpsection report, survey data and field measurements
South 60th Street	Yes	1983	No	Not included in MEI model because at upstream limit of model; not included in Phase 1 WMP model

Table 2. Summary of existing conditions flows at various locations in the model based on the flow distribution developed for the hydrologic analysis.

Location	Mean Daily Exceedence Discharge (cfs)									Instantaneous Peak Discharge (cfs)						Calibration Discharge (cfs)	
	99% Exc	90% Exc	70% Exc	50% Exc	30% Exc	10% Exc	5% Exc	2% Exc	1% Exc	2-yr Pk	5-yr Pk	10-yr Pk	25-yr Pk	50-yr Pk	100-yr Pk	Recent Peak	Historical Peak
Above Hale Creek	0.0	0.1	0.1	0.2	0.7	3	6	16	36	96	220	310	440	560	700	130	770
Below Hale Creek	0.0	0.1	0.2	0.4	1	9	18	37	66	200	450	630	900	1,140	1,420	260	1,570
Below Wildcat Creek	0.2	0.5	1	2	6	24	41	74	110	400	820	1,120	1,600	2,020	2,490	510	2,750
Below Whitnall Park Creek	0.9	2	4	8	20	66	110	190	270	760	1,350	1,820	2,540	3,170	3,890	920	4,280
Below Dale Creek	1	2	5	9	23	70	110	210	270	760	1,340	1,810	2,510	3,130	3,840	920	4,230
Below Unnamed Tributary	1	3	6	11	25	76	120	210	280	790	1,390	1,860	2,590	3,220	3,940	960	4,340
Below Legend	2	4	8	14	33	95	150	260	360	850	1,460	1,950	2,710	3,370	4,120	1,020	4,530
Below East Branch Root Rive	2	4	8	14	33	95	150	260	360	950	1,620	2,170	3,020	3,750	4,590	1,130	5,060
Below Tuckaway Creek	2	4	9	16	31	92	160	300	440	970	1,650	2,210	3,060	3,810	4,660	1,160	5,130
Below Ryan Creek	2	5	10	18	40	110	190	330	480	1,030	1,720	2,300	3,180	3,940	4,820	1,230	5,300

Table 3. Summary of future conditions flows at various locations in the model based on the flow distribution developed for the hydrologic analysis.

Location	Mean Daily Exceedence Discharge (cfs)									Instantaneous Peak Discharge (cfs)					
	99% Exc	90% Exc	70% Exc	50% Exc	30% Exc	10% Exc	5% Exc	2% Exc	1% Exc	2-yr Pk	5-yr Pk	10-yr Pk	25-yr Pk	50-yr Pk	100-yr Pk
Above Hale Creek	0.0	0.1	0.1	0.2	0.7	3	7	17	38	100	240	330	460	580	720
Below Hale Creek	0.0	0.1	0.2	0.4	1	9	18	39	68	210	470	650	930	1,170	1,450
Below Wildcat Creek	0.3	0.6	1	3	7	28	47	84	130	440	880	1,200	1,680	2,110	2,590
Below Whitnall Park Creek	1	2	5	9	22	73	120	210	300	900	1,550	2,040	2,790	3,440	4,150
Below Dale Creek	1	3	5	10	25	77	120	230	300	870	1,490	1,970	2,700	3,330	4,030
Below Unnamed Tributary	1	3	6	12	28	83	130	230	300	890	1,520	2,010	2,750	3,390	4,120
Below Legend	2	4	9	16	37	110	180	290	410	950	1,600	2,110	2,880	3,550	4,300
Below East Branch Root River	2	4	9	16	37	110	180	290	410	1,120	1,850	2,420	3,300	4,050	4,890
Below Tuckaway Creek	2	5	10	18	35	100	180	340	490	1,120	1,850	2,430	3,310	4,070	4,910
Below Ryan Creek	2	5	11	20	44	130	210	360	530	1,180	1,920	2,510	3,420	4,190	5,060

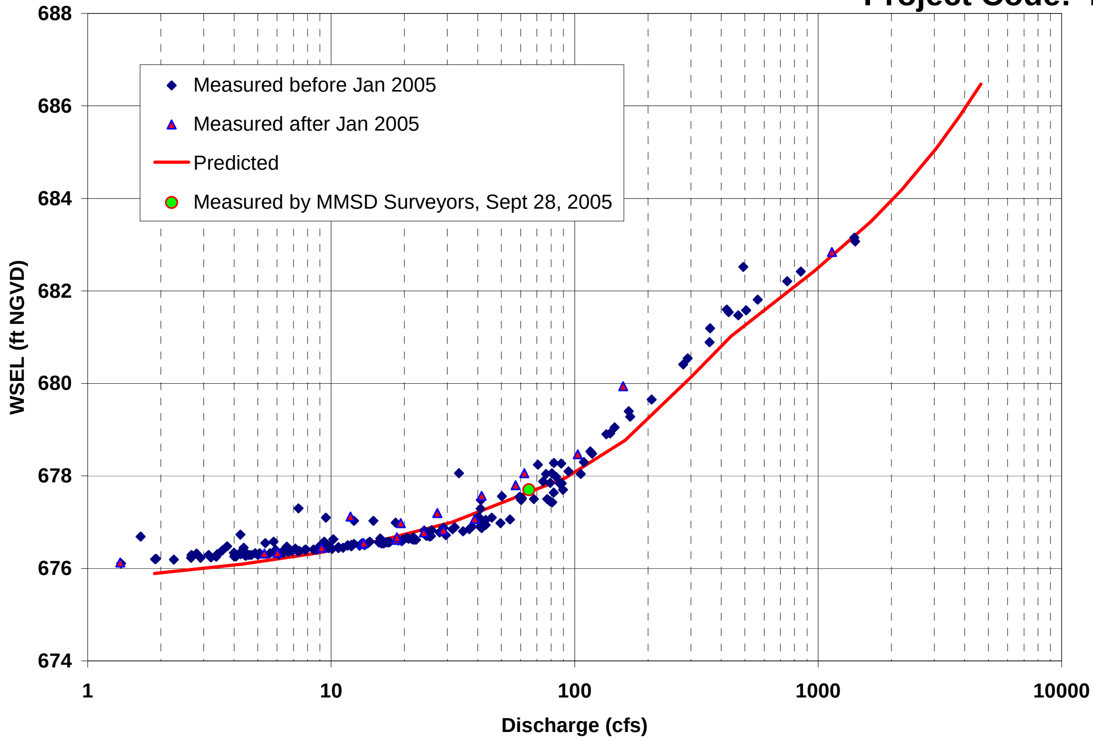


Figure 2. Measured and predicted water-surface elevations at the Root River near Franklin, WI gage (USGS Gage No. 04087220). Also shown is the water-surface elevation measured by the MMSD surveyors on September 28, 2006 when the reported mean daily discharge was 65 cfs.

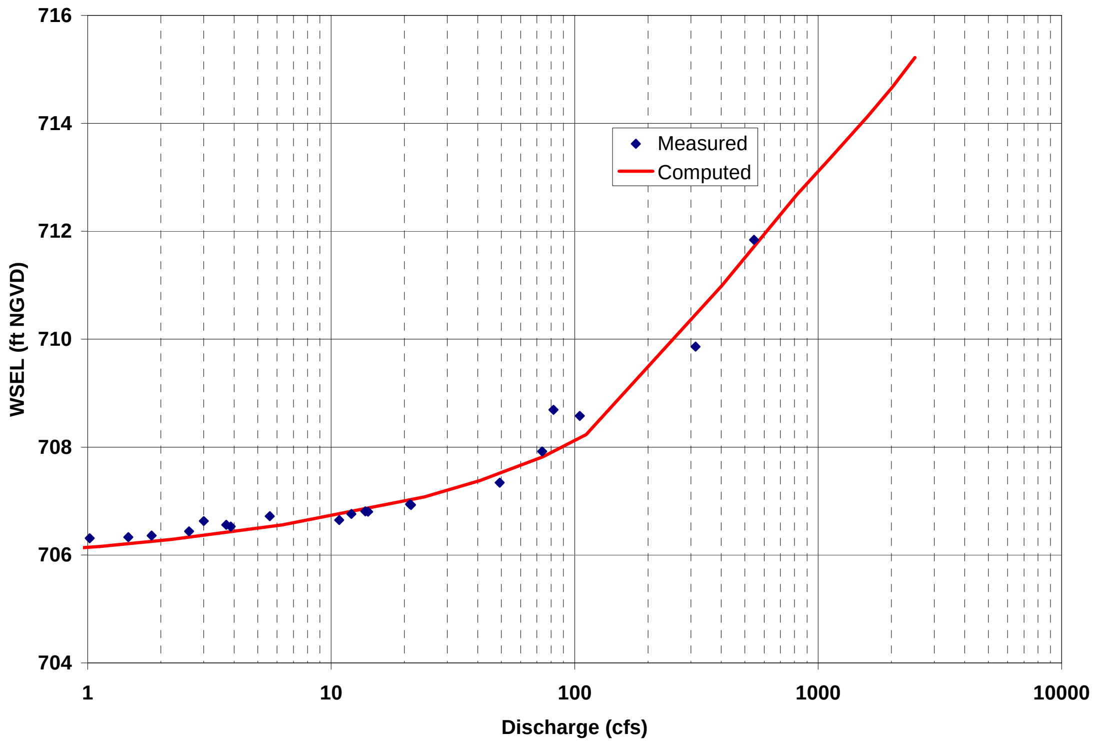


Figure 3. Measured and predicted water-surface elevations at the Root River at Grange Avenue at Greenfield, WI gage (USGS Gage No. 04087214).

Table 4. Location of the geomorphic subreaches used to compute subreach-averaged hydraulics.					
Subreach	U/S Limit	D/S Limit	U/S Station (ft)	D/S Station (ft)	Subreach Length (mi)
1	S 124th St (Milwaukee-Waukesha Cnty Line)	Hale Creek	135,748	130,000	1.1
2	Hale Creek	Wildcat Creek	130,000	118,200	2.2
3	Wildcat Creek	West Grange Ave	118,200	106,158	2.3
4	West Grange Ave	Whitnall Park Creek	106,158	99,300	1.3
5	Whitnall Park Creek	West Loomis Rd (Hwy 36)	99,300	90,104	1.7
6	West Loomis Rd (Hwy 36)	West Rawson Ave	90,104	82,785	1.4
7	West Rawson Ave	Below Gravel Pits	82,785	77,000	1.1
8	Below Gravel Pits	West Puetz Rd	77,000	67,226	1.9
9	West Puetz Rd	West Ryan Rd (Hwy 100)	67,226	59,369	1.5
10	West Ryan Rd (Hwy 100)	South 60th St	59,369	46,068	2.5

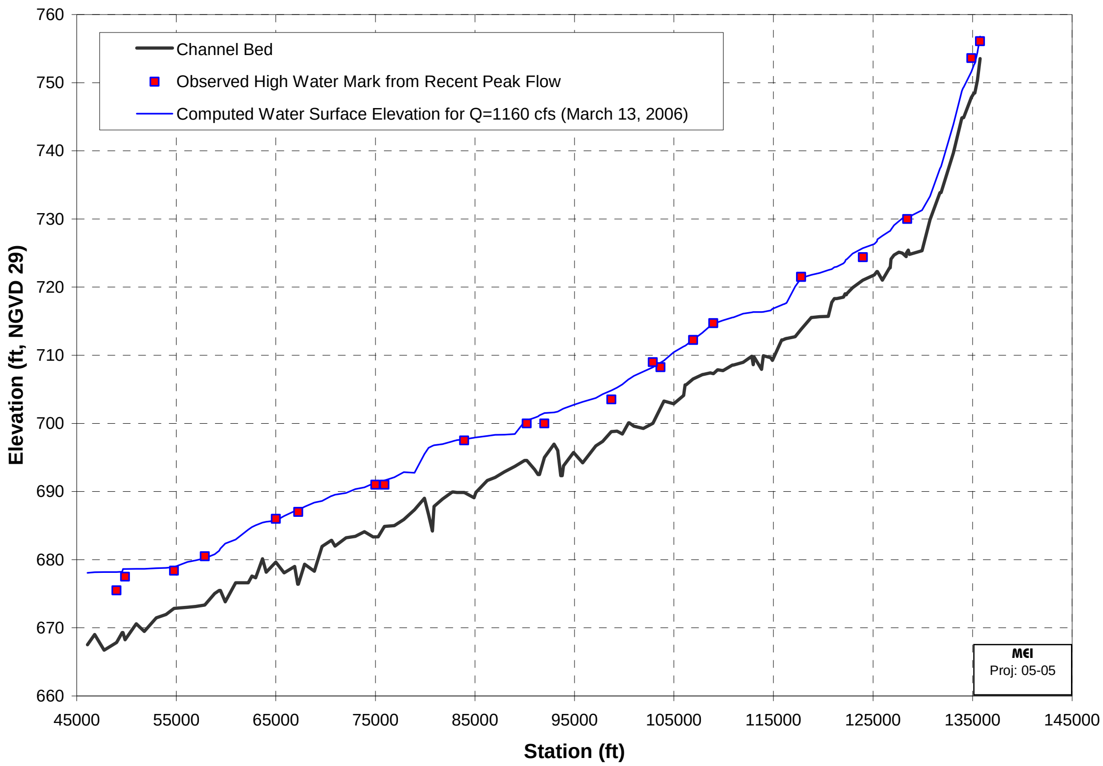
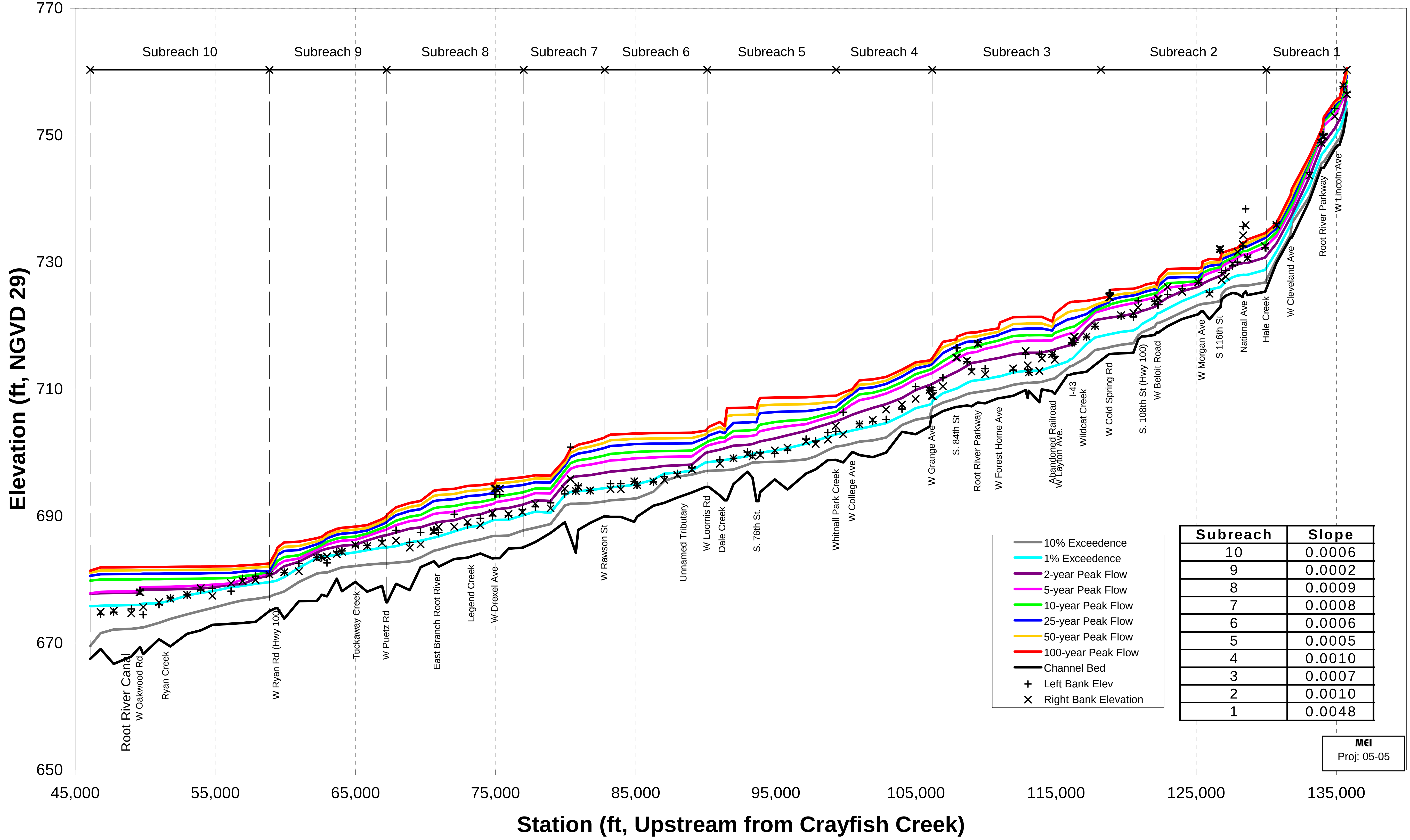


Figure 4. Observed high-water marks and the computed water surface profile for the recent peak flow of 1,160 cfs that occurred on March 13, 2006, at the Franklin gage (based on USGS provisional data).



Subreach	Slope
10	0.0006
9	0.0002
8	0.0009
7	0.0008
6	0.0006
5	0.0005
4	0.0010
3	0.0007
2	0.0010
1	0.0048

Figure 5. Computed water-surface profiles under existing conditions.

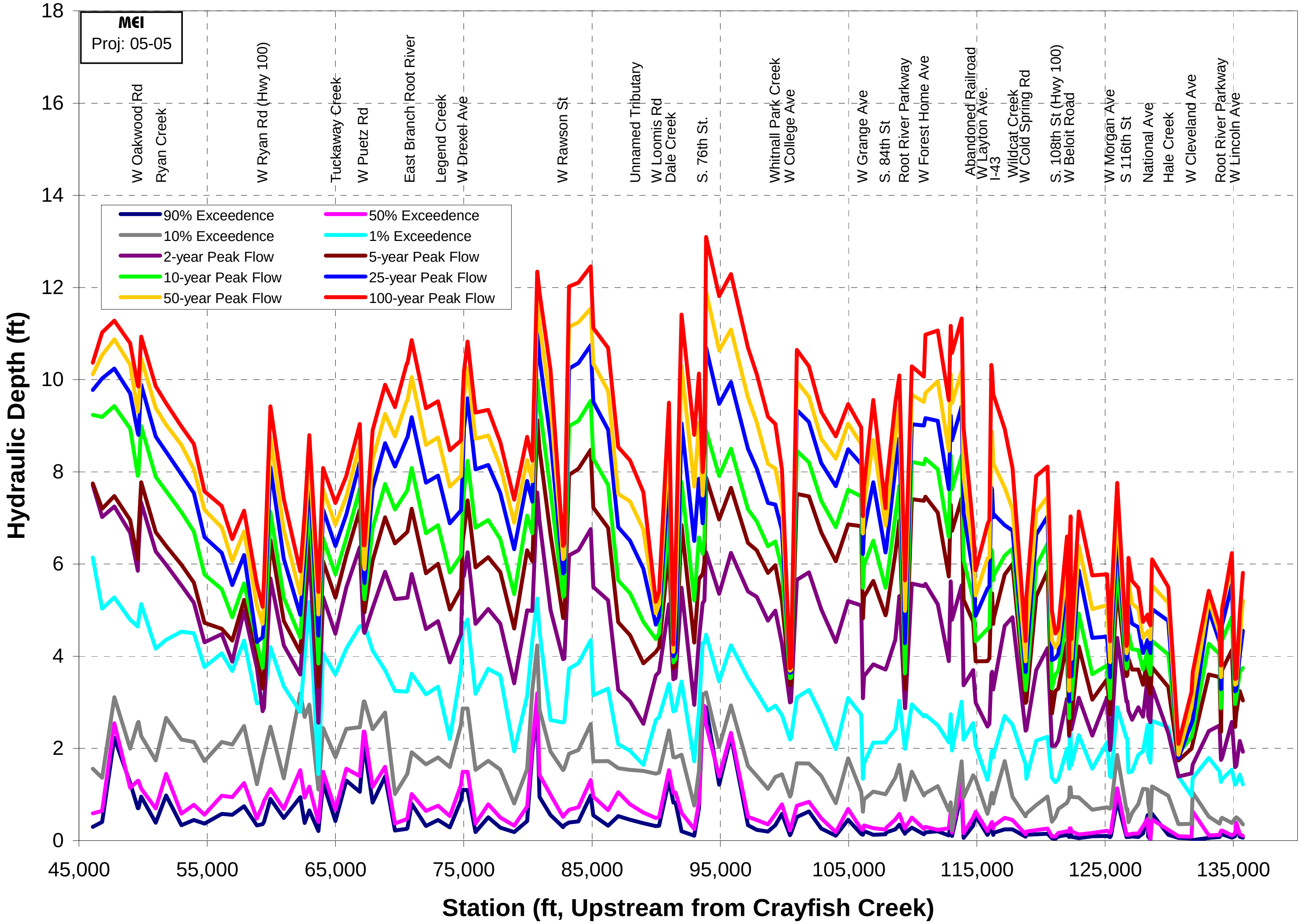


Figure 6. Computed hydraulic depth profiles under existing conditions.

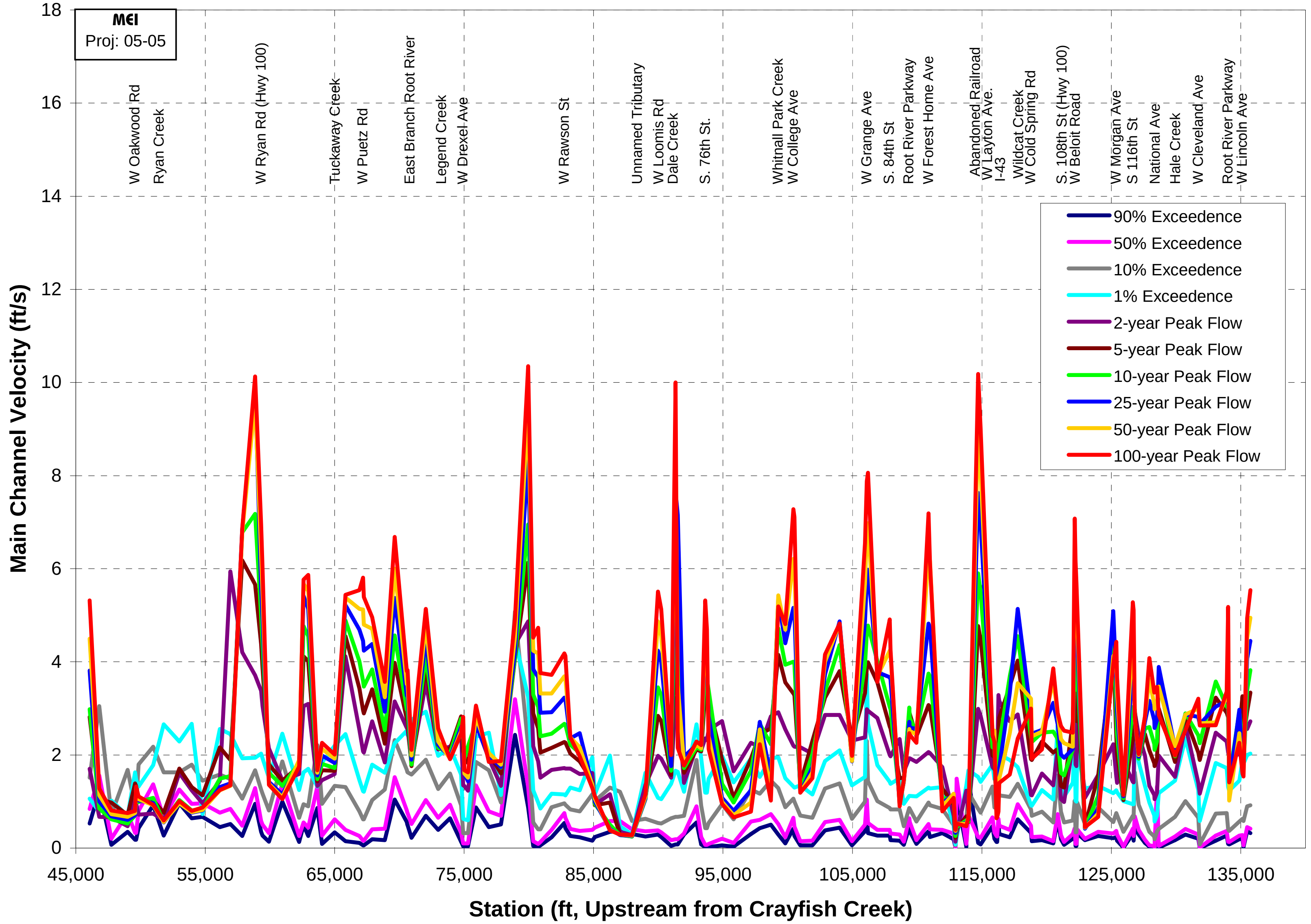


Figure 7. Computed velocity profiles in the main channel under existing conditions.

Project Code: P5000

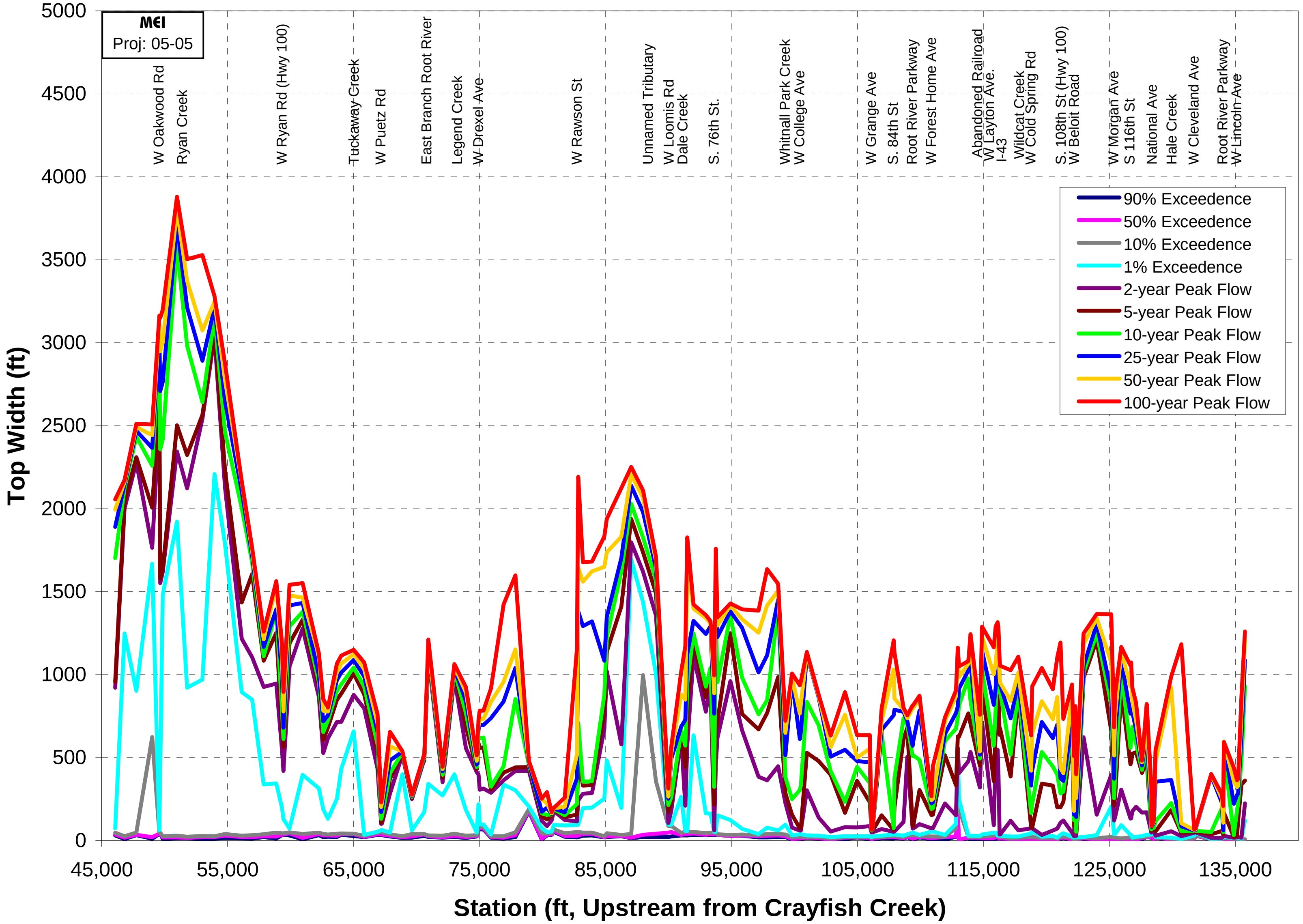


Figure 8. Computed total top width profiles under existing conditions.

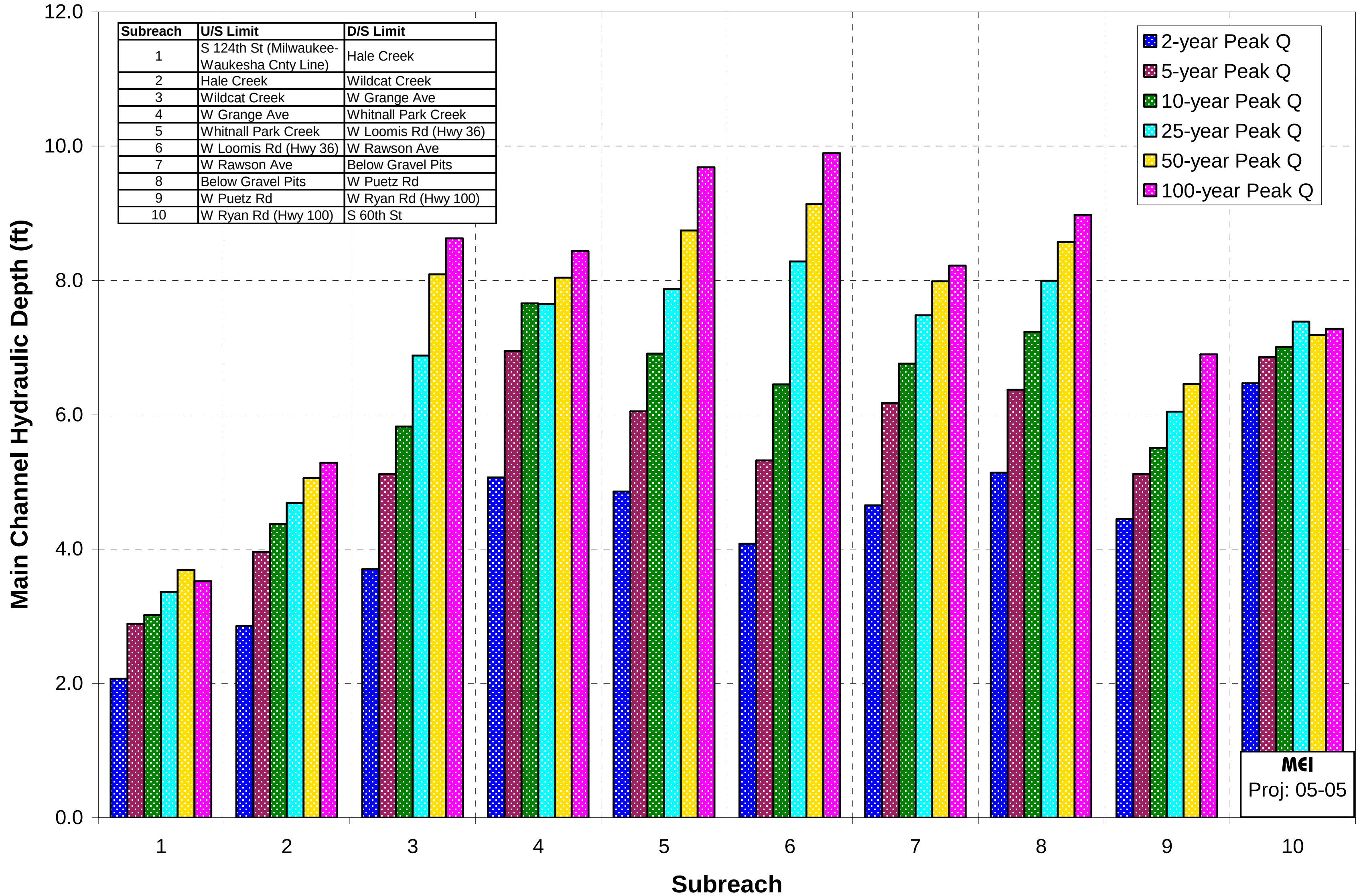


Figure 9. Subreach-averaged main channel hydraulic depth under existing conditions.

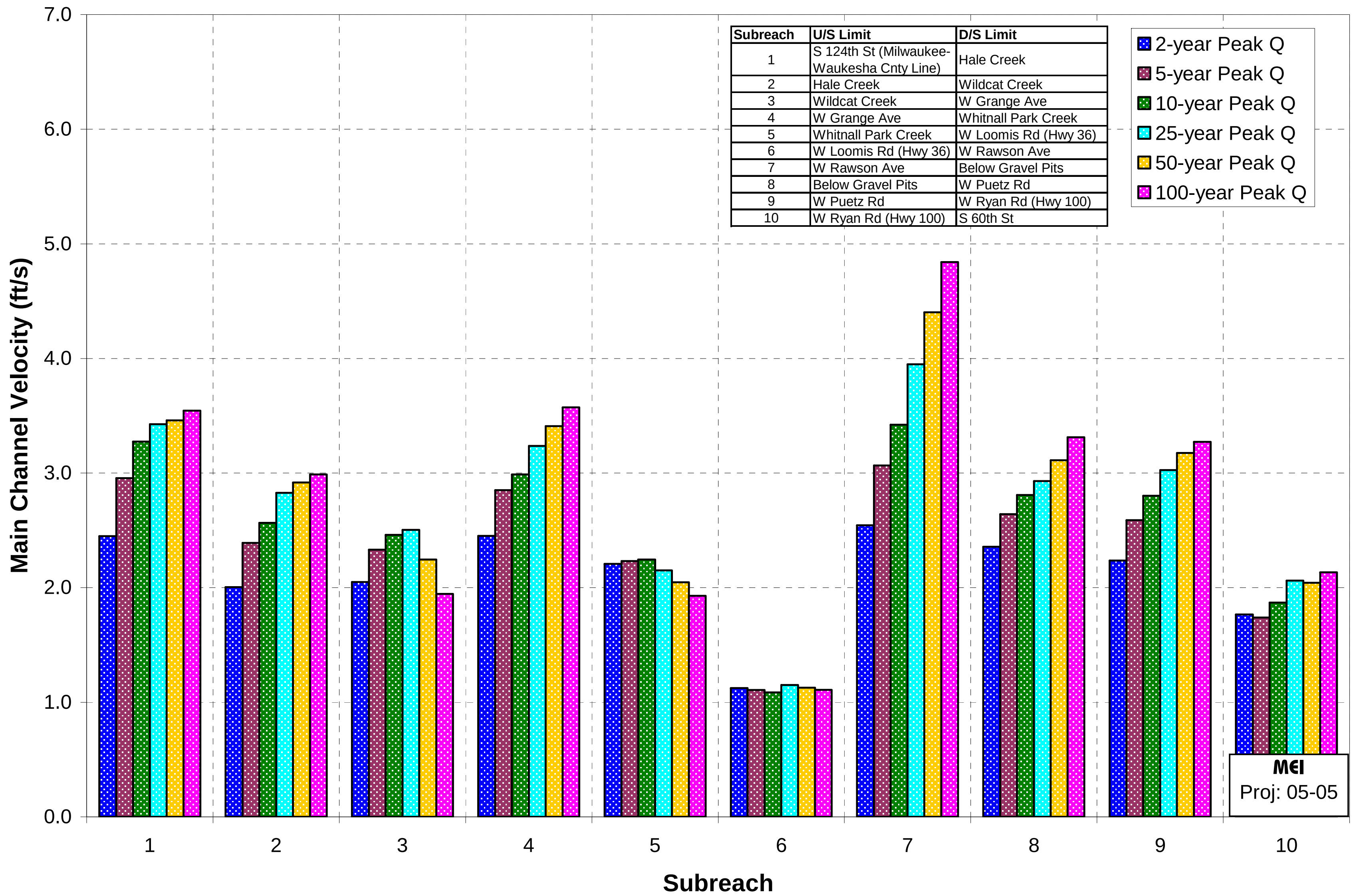


Figure 10. Subreach-averaged main channel velocity under existing conditions.

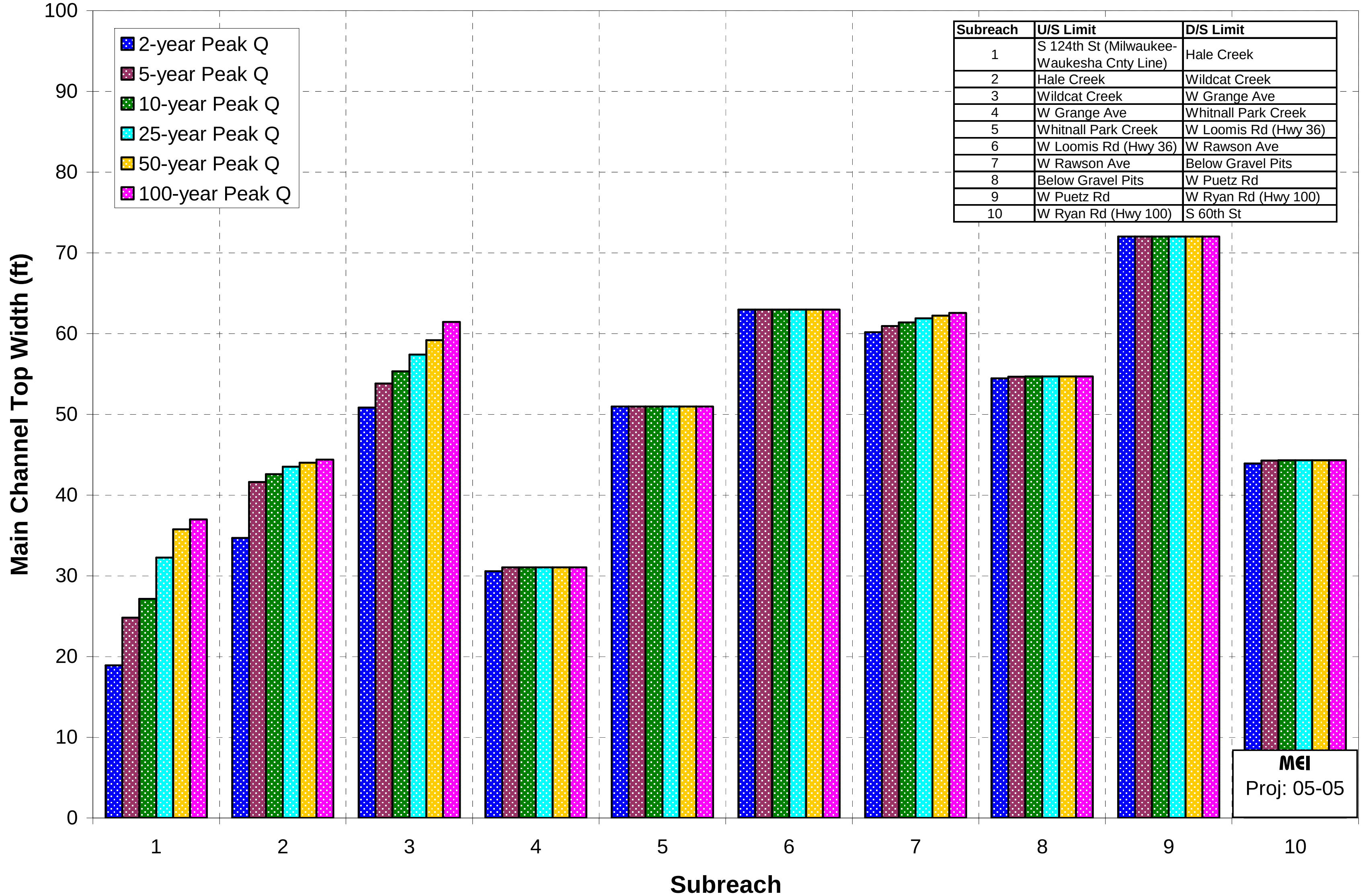


Figure 11. Subreach-averaged main channel top width under existing conditions.

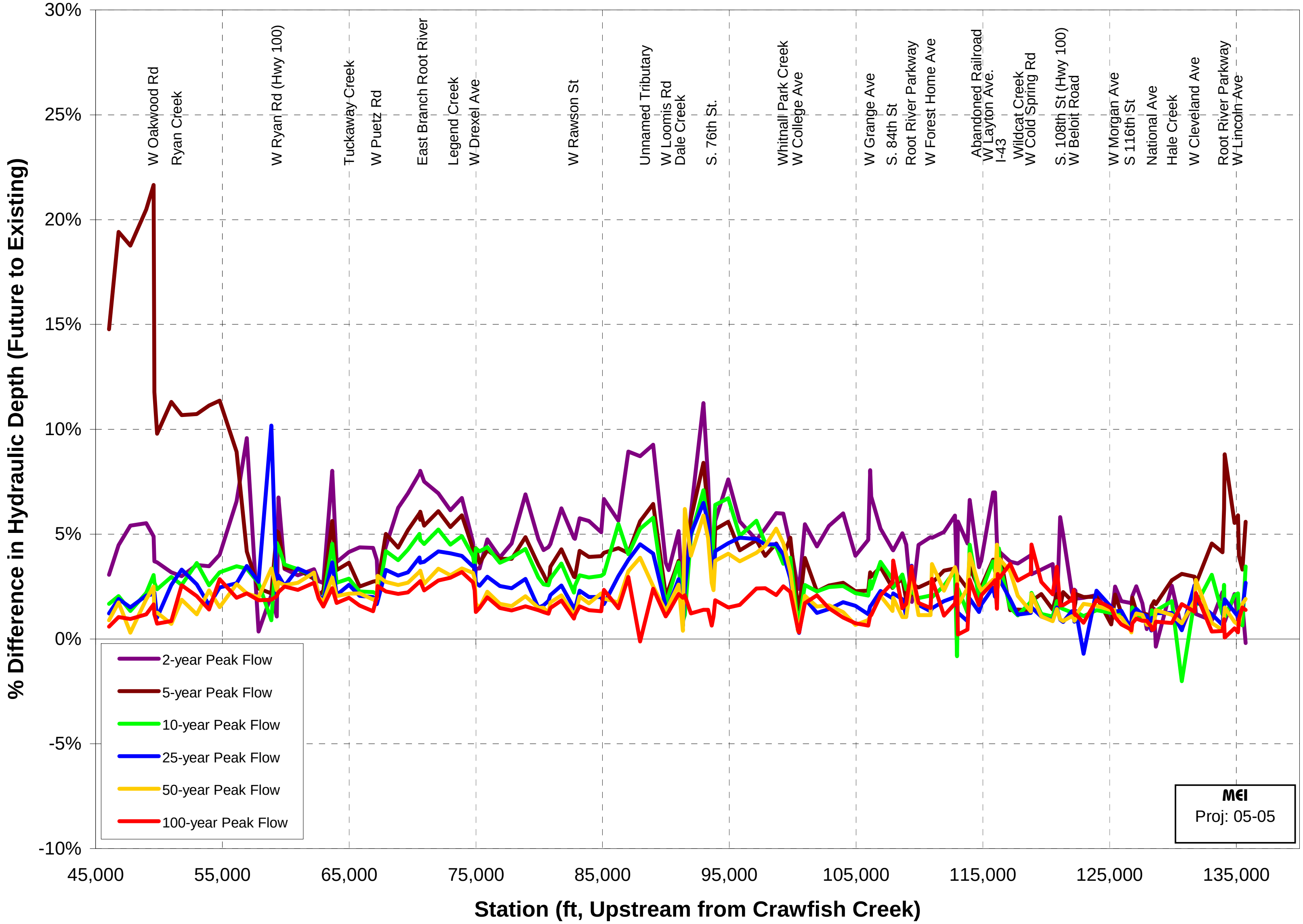


Figure 12. Percent difference in computed main channel hydraulic depth for future conditions compared to existing conditions.

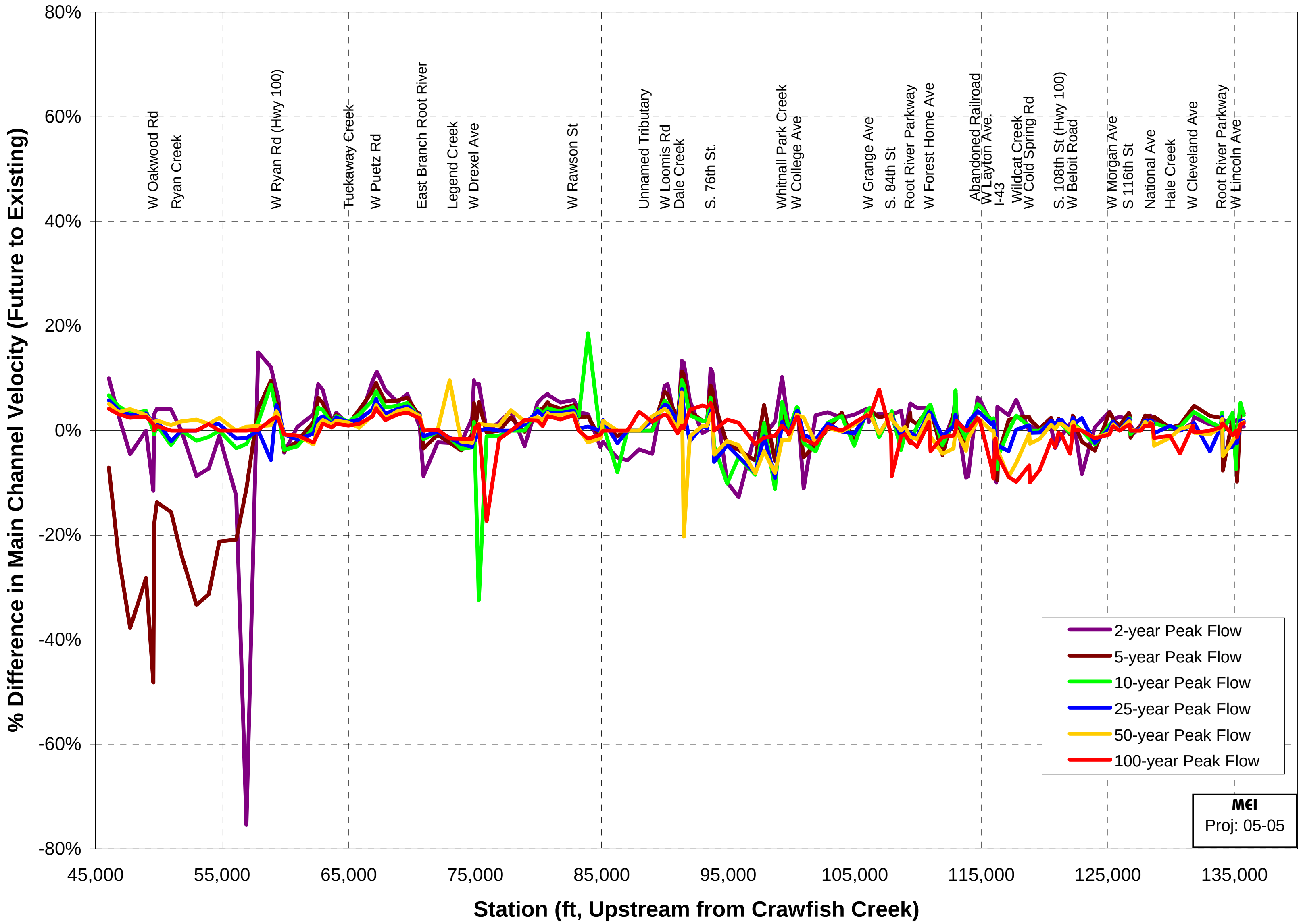


Figure 13. Percent difference in computed main channel velocity for future conditions compared to existing conditions.

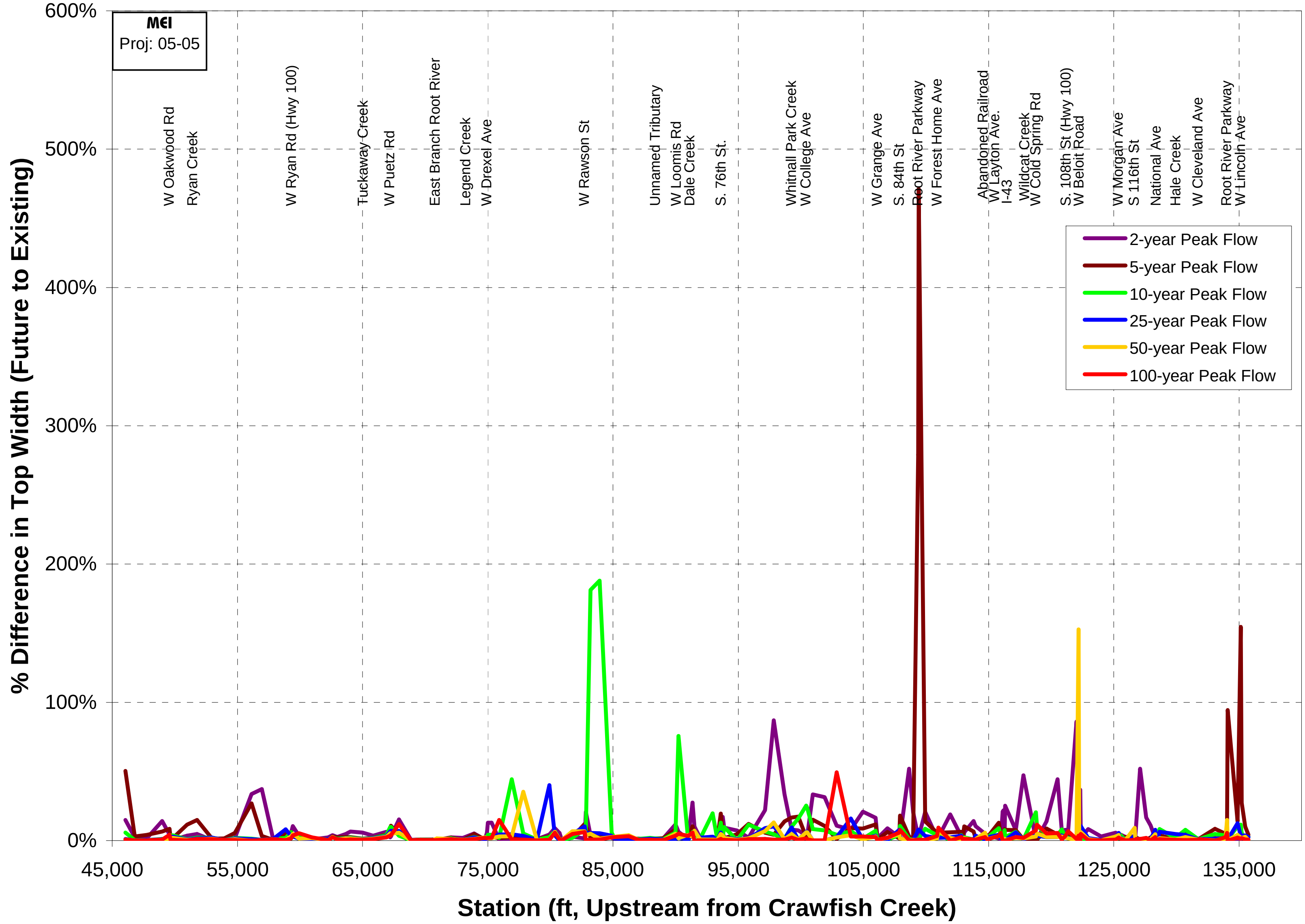
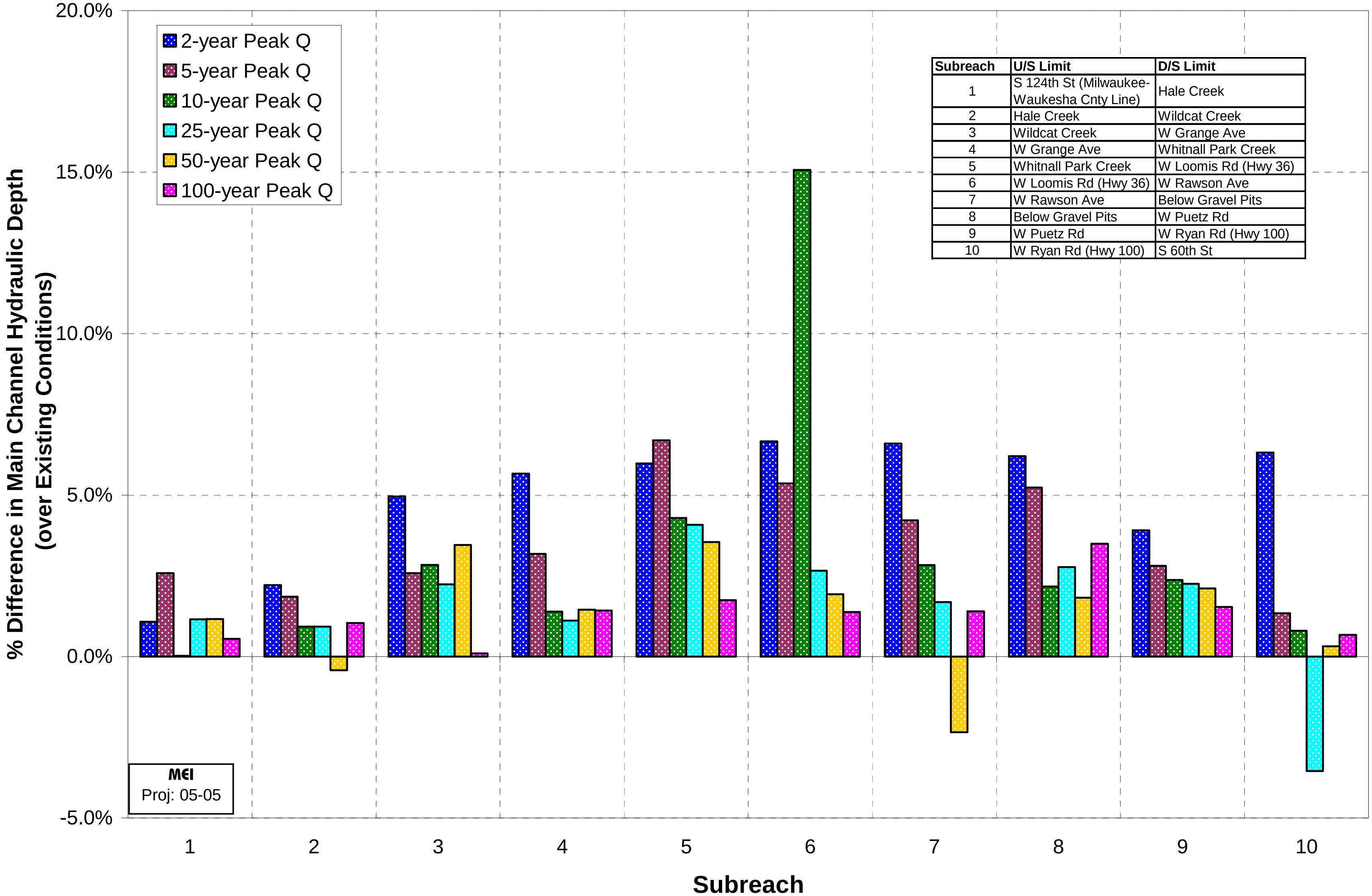
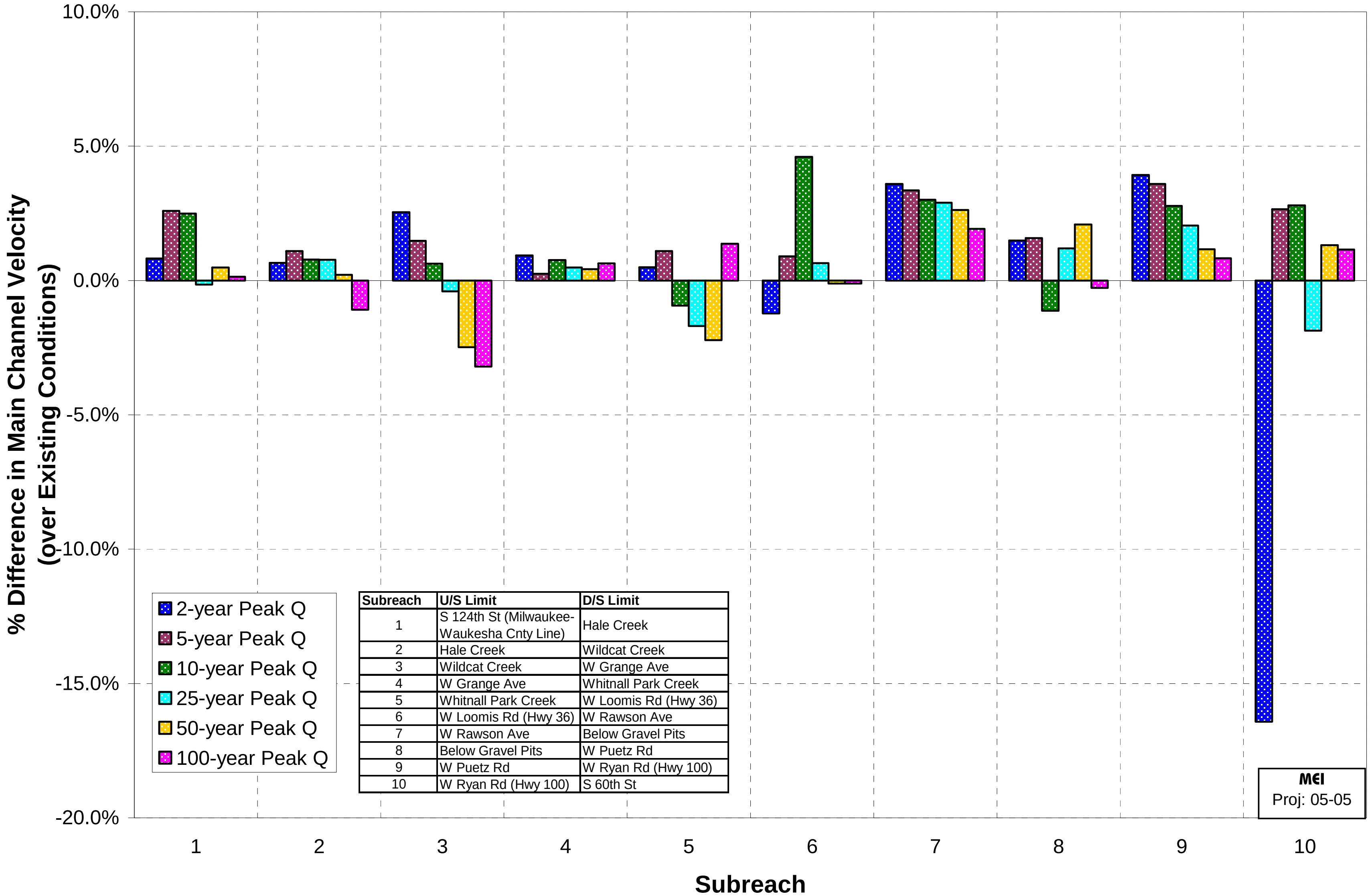


Figure 14. Percent difference in computed top width for future conditions compared to existing conditions.



Subreach	U/S Limit	D/S Limit
1	S 124th St (Milwaukee-Waukesha Cnty Line)	Hale Creek
2	Hale Creek	Wildcat Creek
3	Wildcat Creek	W Grange Ave
4	W Grange Ave	Whitnall Park Creek
5	Whitnall Park Creek	W Loomis Rd (Hwy 36)
6	W Loomis Rd (Hwy 36)	W Rawson Ave
7	W Rawson Ave	Below Gravel Pits
8	Below Gravel Pits	W Puetz Rd
9	W Puetz Rd	W Ryan Rd (Hwy 100)
10	W Ryan Rd (Hwy 100)	S 60th St

Figure 15. Percent increase in the future conditions subreach-averaged hydraulic depth in the main channel (over existing conditions).



Subreach	U/S Limit	D/S Limit
1	S 124th St (Milwaukee-Waukesha Cnty Line)	Hale Creek
2	Hale Creek	Wildcat Creek
3	Wildcat Creek	W Grange Ave
4	W Grange Ave	Whitnall Park Creek
5	Whitnall Park Creek	W Loomis Rd (Hwy 36)
6	W Loomis Rd (Hwy 36)	W Rawson Ave
7	W Rawson Ave	Below Gravel Pits
8	Below Gravel Pits	W Puetz Rd
9	W Puetz Rd	W Ryan Rd (Hwy 100)
10	W Ryan Rd (Hwy 100)	S 60th St

Figure 16. Percent increase in the future conditions subreach-averaged velocity in the main channel (over existing conditions).

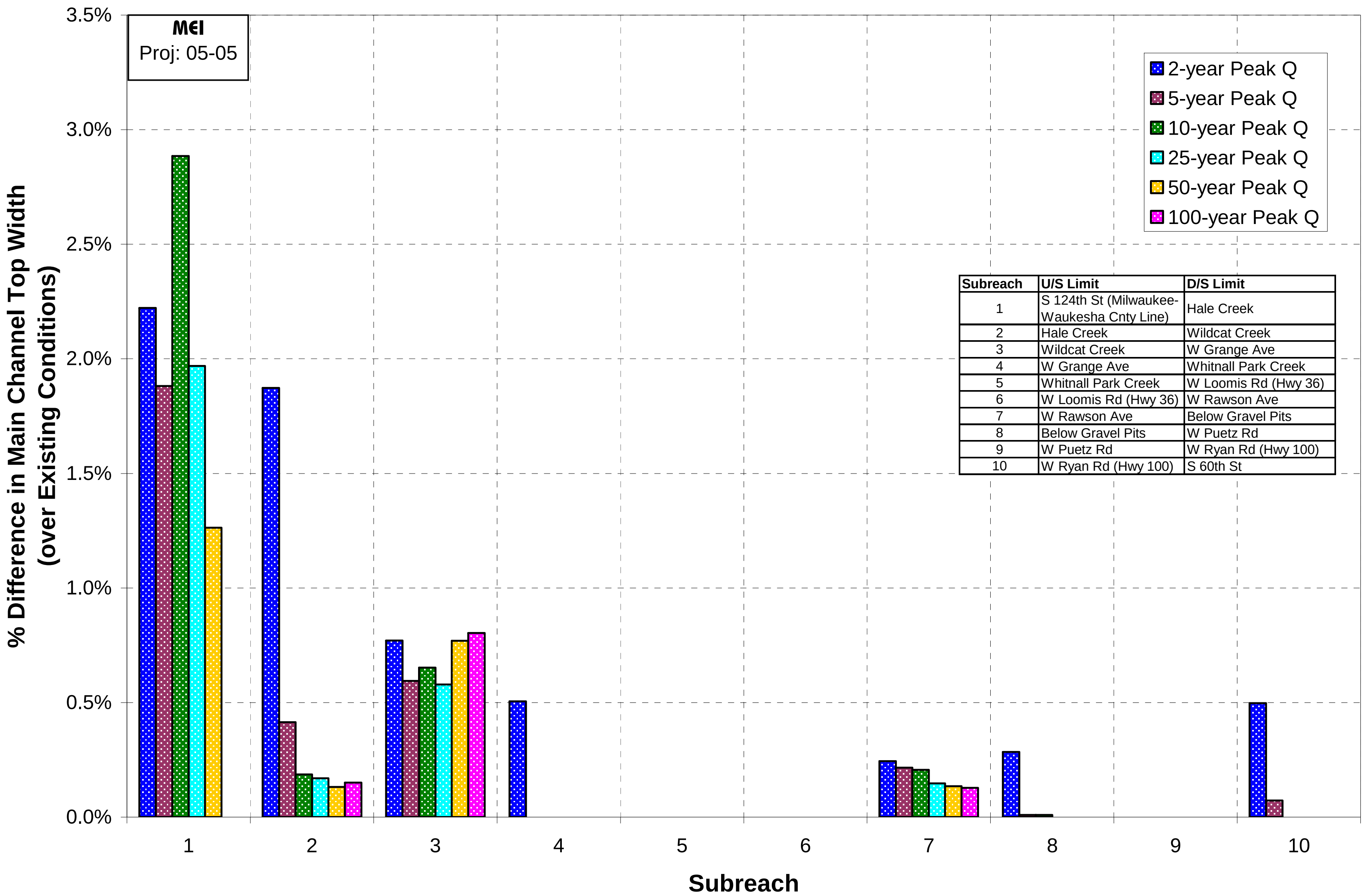


Figure 17. Percent increase in the future conditions subreach-averaged top width in the main channel (over existing conditions).

Table 5. Summary of subreach-averaged hydraulic parameters under future conditions in the portion of the North Branch upstream from the Milwaukee Waukesha County Line (based on the short model developed to estimate hydraulic conditions upstream from South 124th Street).

Profile	Discharge (cfs)	Main Channel Discharge (cfs)	Main Channel Velocity (fps)	Main Channel Hydraulic Depth (ft)	Main Channel Top Width (ft)	Energy Slope (ft/ft)
50% Exceedence	0.2	0.2	0.50	0.1	4.3	0.00548
10% Exceedence	3.0	3.0	1.10	0.3	8.9	0.00535
1% Exceedence	38.0	38.0	2.46	1.1	14.2	0.00493
2-year	100.0	100.0	2.74	1.9	19.2	0.00291
5-year	240.0	239.8	3.14	3.5	21.7	0.00167
10-year	330.0	329.7	3.96	3.8	21.7	0.00238
25-year	460.0	459.5	5.06	4.2	21.7	0.00346
50-year	580.0	579.2	5.97	4.5	21.7	0.00440
100-year	720.0	718.6	6.91	4.8	21.7	0.00538

APPENDIX A

Hydraulic Model for North Branch Root River (existing and future conditions hydrology plans) (on CD)