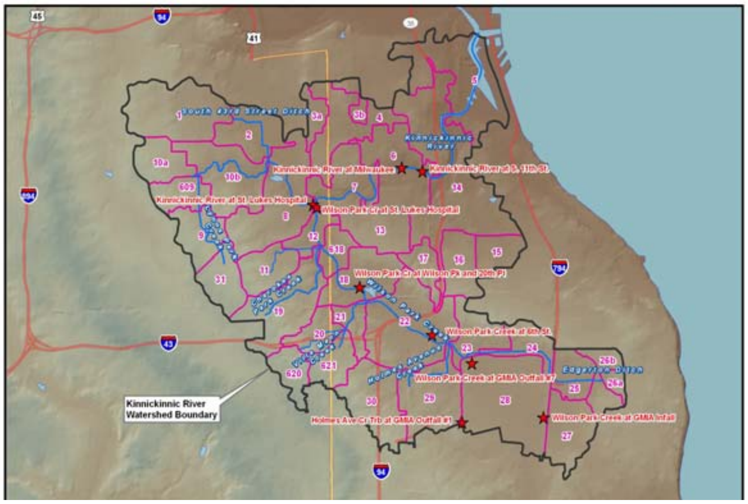
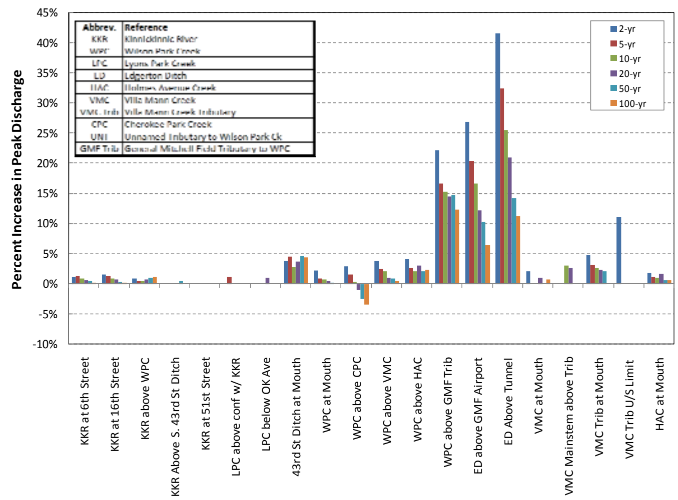
TECHNICAL MEMORANDUM

Hydrology

Kinnickinnic River Sediment-transport Planning Study
Contract No. W40004E01





Submitted to:

Milwaukee Metropolitan Sewerage District
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Preserving The Environment • Improving Water Quality

Submitted by:

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June 29, 2009

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1. INTRODUCTION AND BACKGROUND

An evaluation of the hydrologic characteristics of the Kinnickinnic River watershed was carried out by Tetra Tech, Inc. [formerly Mussetter Engineering, Inc. (TT-MEI)] to develop input for the hydraulic model and sediment-transport analysis for the Sediment Transport Planning Study that is being conducted for the Milwaukee Metropolitan Sewerage District (MMSD). The detailed study reach for which hydrologic data are required includes the Kinnickinnic River from South Chase Avenue to the upstream channel limit near South 60th Street, and major tributaries within this reach (**Figure 1**).

The watershed that was considered in this evaluation included all contributing basins to the mainstem Kinnickinnic River. The evaluation was conducted using available measured hydrologic data and an HSPF-based rainfall-runoff model of the study watershed that was originally developed by the District and recently modified by the Southeastern Wisconsin Regional Planning Commission (SEWRPC).

2. MEASURED HYDROLOGIC DATA

Measured mean daily flow and annual instantaneous peak flow data are available for numerous gages within the study reach with various periods of record (**Table 1**). The primary gage along the Kinnickinnic River (Kinnickinnic River at Milwaukee, WI; USGS Gage No. 04087160) was initially located at South 6th Street in Water Year (WY) 1977, and was replaced by the South 11th Street Gage (USGS Gage No. 04087160) in WY1983, resulting in a combined period of record of more than 30 years. The other gages in the study area have somewhat shorter periods of record, but in most cases, the available data are sufficient for mean daily flow and peak flood-frequency analyses. It should be noted that the USGS gages “Wilson Park Creek at GMIA Infall at Milwaukee, WI” (USGS Gage No. 040871473) and “Wilson Park Creek at GMIA Outfall #7 at Milwaukee, WI” are not located along the mainstem of Wilson Park Creek, but instead are located along the tributary to Wilson Park Creek that drains the General Mitchell Field Airport area to the south.

A duration analysis of the mean daily flows indicates that the median discharge at the Kinnickinnic River gages (Milwaukee and 11th Street gages) during the period of record was about 9.5 cubic feet per second (cfs), and the discharge exceeded 5.3 cfs about 90 percent of the time and 48 cfs about 10 percent of the time (**Figure 2**). The median discharge at the mouth of Wilson Park Creek (Saint Luke’s Hospital gage) was about 5.2 cfs, and the discharge exceeded 2.3 cfs 90 percent of the time and 29 cfs 10 percent of the time. As expected, flow-duration curves for the other gages located in the upstream portion of the Wilson Park Creek sub-basin show progressively decreasing discharge with decreasing drainage area.

Table 1. Summary of USGS Stream Gages located within the Kinnickinnic River Sediment Transport Planning Study Area.

Site Name	USGS Gage Number	D.A. (sq. mi.)	Datum (NGVD29)	Mean Daily Flow POR	Peak Flow POR
Kinnickinnic River at S. 11th St. at Milwaukee, WI	04087159	18.8	588.9	10/1/82-Present	WY77-WY08
Kinnickinnic River at Milwaukee, WI	04087160	20.4	590	7/1/76-12/31/82	WY77-WY82
Kinnickinnic River at St. Lukes Hospital at Milwaukee, WI	040871472	5.4	640	11/12/96-5/14/97	NA
Holmes Ave Cr Trb at GMIA Outfall #1 at Milwaukee, WI	040871476	0.03	695	11/12/96-Present	WY98-WY08
Wilson Park Creek at GMIA Infall at Milwaukee, WI	040871473	0.89	665	11/12/96-Present	WY98-WY08
Wilson Park Creek at GMIA Outfall #7 at Milwaukee, WI	040871475	2.25	670	11/13/96-Present	NA
Wilson Park Creek at 6th St. at Milwaukee, WI	040871478	6.1	670	11/12/96-5/15/97	NA
Wilson Park Cr at Wilson Pk and 20th Pl at Milwaukee, WI	040871482	NA	650	11/18/96-5/9/97	NA
Wilson Park Cr at St. Lukes Hosptl at Milwaukee, WI	040871488	11.34	640	11/18/96-Present	WY98-WY08

Annual instantaneous peak flow data at the Kinnickinnic River 11th Street gage include a continuous record from WY1977 to WY2008. (Apparently the peak flow data from the Milwaukee gage was adopted for the period between WY1977 and WY1982, although this is not expressly indicated in the USGS data files.) Using these flood peaks, a flood-frequency curve was developed using the U.S. Army Corps of Engineers HEC-FFA computer program (USACE, 1992), which is based on the procedures outlined in Water Resources Council (WRC) Bulletin 17B (WRC, 1981), with a generalized skew coefficient of -0.4. The resulting frequency curve indicates that the 2-year peak flow is about 3,410 cfs, the 10-year peak flow is about 5,960 cfs, and the 100-year peak flow is about 9,400 cfs (**Figure 3**). At the Wilson Park Creek gage at Saint Luke's Hospital, only 11 years of peak flow data are available (WY1998 to WY2008). A peak flood-frequency analysis of this relatively limited data set was carried out to evaluate peak flow recurrence intervals under recent climate and land-use conditions. This analysis indicates that the 2-year peak flow at the mouth of Wilson Park Creek is about 1,380 cfs, the 10-year peak flow is about 2,980 cfs, and the 100-year peak flow is about 5,440 cfs (**Figure 4**).

The large amount of concrete-lined channels and enclosed conduits in the Kinnickinnic River watershed, combined with relatively little overbank storage, significantly reduces flood attenuation and results in "flashy" hydrographs with short duration and high peaks relative to base level flows. Because the peak flood-frequency analyses only considered the annual peak flows, and because the magnitude of the flash floods are not reflected in the mean daily flow analyses, the relative magnitude of the flash floods were evaluated by comparing the published mean daily flow with the recorded maximum instantaneous flow for each day in the period of record at the South 11th Street gage (**Figure 5**). Results from the analysis indicate that the maximum instantaneous discharge is typically about 1.1 times higher than the mean daily flow when the mean daily flow is less than 10 cfs, about 3.1 times higher when the mean daily flow is between 10 and 100 cfs, and about 5.7 times higher when the mean daily flow exceeds 100 cfs.

3. HSPF RAINFALL-RUNOFF MODEL

The most recent HSPF rainfall-runoff models of the watershed were provided to TT-MEI by SEWRPC for use in this study. The models include a runoff component and separate surface flow routing components that represent existing and 2020 planned land-use (PLU) conditions.

3.1. Model Background

The original model was developed using the U.S. Environmental Protection Agency (EPA) Hydrologic Simulation Program—FORTRAN (HSPF) computer software. HSPF is a set of computer codes that can simulate the hydrologic processes on pervious and impervious land surfaces, and in streams and impoundments. Hydrologic processes simulated by HSPF include precipitation, interceptions, evaporation, evapotranspiration, infiltration, soil moisture, subsurface flow, and surface flows. HSPF uses time series of climatic inputs to calculate a corresponding time series of streamflow. The required climatic time series data include precipitation, potential evapotranspiration, wind speed, dew point temperature and solar radiation, and HSPF uses these data to compute surface and subsurface runoff rates by accounting for surpluses and deficits of soil moisture, groundwater storage, and snowpack.

3.2. Model Structure and Input

The HSPF model for the Kinnickinnic River watershed was developed for a historical sequence of representative climatology between 1940 and 2004. The historical climate data is input into HSPF as time series for warm weather hydrologic conditions (precipitation and potential evapotranspiration) and for winter hydrology (temperature, dew point temperature, wind speed, and solar radiation). The input time series were based on data collected at General Mitchell International Airport, and included 15-minute precipitation data, hourly temperature, wind speed, solar radiation, and potential evapotranspiration data, and daily dew point data.

The model was developed by subdividing the watershed into 38 subbasins to represent the main Kinnickinnic River tributary watersheds (Figure 1). The land use, soil types, and land cover of each of the basins was originally obtained from a Load Simulation Program C++ (LSPC) model of the watershed that was developed by Tetra Tech (2007). Each of the ground types were assigned appropriate hydrologic parameters that were adjusted as necessary for model calibration.

HSPF uses the input time series and hydrologic parameters to determine the amount of runoff from the overland portions of the individual basins that becomes concentrated flows in the primary drainage channels. The model routes the concentrated flows through the system of pools and channels using level-pool hydrologic routings. Tetra Tech (2007) developed storage-discharge relations as input to the model for areas where significant storage was identified.

3.3. Model Execution

TT-MEI evaluated the available models using WinHSPF, an interactive Windows interface to HSPF developed by Aqua Terra Consultants for the EPA (Aqua Terra Consultants, 2001). WinHSPF Version 2.3, which uses HSPF Version 11.0, provides a means of viewing or modifying the model representation of a watershed. Minor modifications to the input file were necessary to enable HSPF to open and execute the input files provided by SEWRPC that were developed for use with the MSDOS-based version of the program.

3.4. Model Results

Results from the HSPF model for existing conditions were compared to the measured data at the USGS gages to determine their suitability for use in the hydraulic and sediment-transport analysis. The comparison was made by developing mean daily flow-duration curves from the HSPF-computed flow record at the model nodes that are nearest the gages (Nodes 807 and 818), and comparing these curves with the flow-duration curves developed from the measured data for the equivalent time-period (WY1977-WY2004 for the Kinnickinnic River gage and November 1996 to December 2004 for the Wilson Park Creek Gage; **Figures 6 and 7**). It should be noted that the comparison was only made at these two locations, since the other gages do not have a corresponding modeling node or include a period of record that is not sufficient to develop flow-duration curves. The comparison at the Kinnickinnic River gage indicates that the HSPF model under-predicts the discharge over the full range of exceedence levels, with flows in the greater than 70-percent exceedence level under-predicted by a factor of more than two and less significant under-prediction at the higher discharges. For the Wilson Park Creek gage, the HSPF model under-predicts the discharge at flows less than the 50-percent exceedence level and over-predicts the discharge at higher flows. Since most flows at both locations are not accurately portrayed by the model results, direct use of the flow record from the HSPF model for sediment-transport modeling is not recommended.

An evaluation was also carried out to determine if the HSPF model results for existing conditions could be used to develop peak flow records for flood-frequency analysis at locations in the study reach other than the gage locations. Again, the evaluation was conducted by comparing annual peak flows developed from the model output at Node 807 with the observed annual peak flow at the Kinnickinnic River gage. The evaluation was not carried out for the Wilson Park Creek gage because the overlapping period of record is only seven years (WY1998 to WY2004) and is not sufficient to perform a flood-frequency analysis. The comparison indicates that the HSPF model significantly under-predicts the lower magnitude flows less than the 10-year event, but the frequency curve developed using the HSPF model tends to converge with the measured curve at flows approaching the 100-year event (**Figure 8**). Since high-flow events in the 1.5- to 5-year range of recurrence intervals typically transport the most sediment and, thus, do the most work in adjusting the channel, over the long-term, (Wolman and Miller, 1960; Andrews, 1980; Pickup and Werner, 1976), direct use of the maximum flows from the HSPF model for this study is not recommended.

4. FLOW-DURATION CURVES

The specific reasons for the HSPF model under-predicting most flows at the Kinnickinnic River and Wilson Park Creek gages are not known, and a relatively large effort that is well beyond the resources available for this project would be required to re-calibrate it. Since the model accounts for the important physical processes that control runoff, however, it should preserve the general distribution of flows and relative response of the system to the modeled climatological events.

4.1. Existing Conditions

Based on the above observations, it was determined that further analysis of the long-term hydraulic and sediment-transport conditions in the study reach can be conducted most effectively using mean daily flow-duration curves (i.e., curves representing the long-term, average percentage of time that discharges of a given magnitude are equaled or exceeded), rather than the detailed daily flows predicted by the model. The existing conditions flow-duration curves for each location of interest within the study area were developed from the model output and available data based on the following two key assumptions:

1. Flow-duration curves developed from the model output at any particular location in the Kinnickinnic River watershed, but excluding the Wilson Park Creek watershed, will deviate from the flow-duration curve that would result from measured data at that location (if it was available) by the same relative amount that the model-based flow-duration curve at the South 11th Street gage deviates from the gage-based flow-duration curve. Similarly, flow-duration curves developed from the model at any location in the Wilson Park Creek watershed will deviate from the flow-duration curve that would result from actual flows at that location by the same relative amount that the model-based flow-duration curve at the Wilson Park Creek at Saint Luke's Hospital gage deviates from the gage-based flow-duration curve. The relationships between the gage- and model-based flow-duration curves are shown in **Figure 9**. These relationships were developed by dividing the discharge from the gage-based flow-duration curve by the corresponding discharge from the model-based curve for each exceedence percentage in Figures 6 and 7.

2. The percentage of the annual precipitation that appears as runoff at each location in the study reach should be consistent with observed runoff percentages at similar gaged locations in the south Milwaukee area. Twelve stream gages in the south Milwaukee area with significantly urbanized drainage areas ranging in size from 0.03 to about 50 mi^2, and periods of record from 12 to 43 years, were identified for use in the evaluation. (Gages with shorter periods of record were not considered in the analysis since the records may not be representative of long-term hydrologic conditions.) Annual precipitation at the General Mitchell International Airport weather station, which is believed to be representative of precipitation in the Kinnickinnic River watershed, during the periods of record ranged from about 32 to 35 inches. The measured annual runoff at these 12 locations generally decreases with drainage area, ranging from about 63 percent in the very small drainage area (D.A. = 0.03 mi^2) of the Holmes Avenue Creek Tributary to about 37 percent at the Root River Franklin Gage (D.A. = 49.2 mi^2; **Figure 10**). In addition, the measured runoff in the Kinnickinnic River watershed defines the upper limit of these data. While the HSPF-computed runoff volumes align reasonably well with the measured volumes in the overall south Milwaukee area, the modeled volumes are somewhat less than the measured values in the Kinnickinnic River watershed (**Figure 11**). For example, the annual runoff at the South 11th Street gage on the Kinnickinnic River averaged about 18 inches during the period from WY1977 and WY2004 that overlaps with the HSPF flow record, or about 53 percent of the annual precipitation, while the average annual runoff from the modeled flows is only about 13 inches (42 percent). The measured annual runoff at the Saint Luke's Hospital gage on Wilson Park Creek for the concurrent period is about 16.7 inches, or 49 percent of the annual precipitation, while the annual volume predicted by the model is only about 14.5 inches (47 percent).

The above information clearly indicates that the flow-duration curves resulting from the HSPF model require adjustment to insure that they provide a realistic representation of the flow distribution in the study reaches. The following specific procedures were used to develop realistic flow-duration curves for the locations of interest along the Kinnickinnic River and its tributaries for which data could be extracted from the model:

1. A target annual runoff volume at each location in the Kinnickinnic River watershed except for those locations within the Wilson Park Creek subbasin was estimated by assuming that the runoff should vary between the observed value at the South 11th Street gage (17.8 inches or about 53 percent of annual precipitation) and the observed value at the Holmes Avenue Creek Tributary gage (21.4 inches or about 63 percent of annual precipitation; Figure 10). For locations in the Wilson Park Creek subbasin, the target annual runoff was assumed to vary between the observed value at the Saint Luke's Hospital gage (16.7 inches or 49 percent of the annual precipitation) and the observed value at the Holmes Avenue Creek Tributary gage (Figure 10). A summary of the estimated runoff volumes at all of the model nodes that were considered in the analysis is provided in **Table 2**.

2. A preliminary flow-duration curve was developed from the 65-year record of mean daily flows predicted by the model at each of the above nodes.

3. Using the relationships shown in Figure 9, the ordinates of the flow-duration curve from the model output were adjusted by the ratio of the measured to modeled discharge for the equivalent duration at the Saint Luke's Hospital gage for locations in the Wilson Park Creek subbasin or at the South 11th Street gage for the remainder of the locations.

4. The annual runoff volume indicated by the adjusted curves was compared to the target values in Table 2, and an additional, relatively minor adjustment was made to the curves to match the target values by applying a constant factor to all ordinates in the curve.

The resulting existing conditions flow-duration curves for the project area are shown in **Figures 12 through 14**. These curves indicate that the median flow in the Kinnickinnic River ranges from about 0.8 cfs below the confluence with Lyons Park Creek to about 9 cfs at South 6th Street near the downstream end of the study reach, and the flow that is exceeded 10 percent of the time on an annual basis ranges from about 4.4 to 48 cfs at these two locations, respectively. On Wilson Park Creek, the median flow ranges from 0.3 cfs in the upstream reach through Edgerton Ditch to about 5 cfs at the confluence with the Kinnickinnic River, and the 10 percent exceedence flow ranges from 1.1 cfs to about 26 cfs at these two locations, respectively.

4.2. 2020 Planned Land Use Conditions

The results from the HSPF model for 2020 PLU conditions indicate that future development in the basin will typically increase the annual runoff by an average of about 2 percent, with the largest increases occurring in the mostly undeveloped Edgerton Ditch watershed in the headwaters of the Wilson Park Creek subbasin (**Figure 15**). A decrease in runoff of nearly 4 percent is occurs in the basin that drains to South 43rd Street ditch, primarily due to a 50 percent increase in Forest and Wetland ground types under 2020 PLU conditions. Flow-duration curves for future conditions were developed by multiplying the ordinates of the existing conditions flow-duration curves by the predicted percent increase (or decrease) in runoff from the HSPF model. The resulting curves for the locations of interest are presented in **Figures 16 through 18**.

Table 2. Summary of HSPF-computed annual runoff volumes (as runoff depth and as percent of precipitation) under existing conditions. Also shown are the adopted runoff values as runoff depth and percent of precipitation.

Location	Model Node	Drainage Area (mi^2)	HSPF Runoff Volume (in.)	HSPF Runoff (Percent of Precip.)	Adopted Runoff Depth (in.)	Adopted Runoff (Percent of Precip.)	Adopted Volume (ac-ft)
Kinnickinnic River							
KKR at 6th Street	807	20.07	13.0	42	16.4	53	17549
KKR at 16th Street	808	19.08	12.8	41	16.4	53	16705
KKR above WPC	710	5.86	14.4	46	17.0	55	5313
KKR Above S. 43rd St Ditch	709	2.69	14.3	46	17.4	56	2498
KKR at 51st Street	809	1.77	14.5	47	17.6	57	1660
Lyons Park Creek							
LPC above conf w/ KKR	831	1.33	14.5	47	17.7	57	1261
LPC below OK Ave	31	0.96	14.5	47	17.9	58	913
S 43rd St Ditch							
43rd St Ditch at Mouth	801	1.71	15.9	51	17.6	57	1610
43rd St Ditch at U/S Outfall	1	1.13	15.7	51	17.8	58	1076
Wilson Park Creek							
WPC at Mouth	818	11.05	14.5	47	15.3	49	8993
WPC above CPC	722	9.34	14.2	46	15.4	50	7665
WPC above VMC	823	7.04	14.5	47	15.6	50	5854
WPC above HAC	828	3.90	13.6	44	16.0	52	3333
WPC above GMF Trib	824	1.22	13.9	45	16.9	55	1096
Edgerton Ditch							
ED above GMF Airport	826	0.85	14.3	46	17.2	55	776
ED Above Tunnel	26	0.46	2.8	9	17.6	57	428
Tributaries to Wilson Park Creek							
Unnamed Trib to WPC	11	0.47	12.9	42	17.6	57	446
CPC at Mouth	19	0.96	14.0	45	17.1	55	875
VMC at Mouth	820	1.32	15.2	49	16.8	54	1185
VMC Mainstem above Trib	521	0.39	16.2	52	17.7	57	372
VMC Trib at Mouth	20	0.68	14.7	48%	17.3	56	626
VMC Trib U/S Limit	520	0.34	13.1	42	17.8	58	326
HAC at Mouth	830	1.72	16.2	52	16.6	54	1524
HAC at Headwaters	30	0.84	14.8	48	17.2	55	769
GMF Trib at Mouth	827	2.21	13.2	43	16.5	53	1938

5. INSTANTANEOUS PEAK FLOWS

As discussed above, the peak flows estimated from the HSPF model output at the Kinnickinnic River at South 11th Street gage are significantly smaller than the measured peak flows over the range of more commonly occurring floods (Figure 8); thus, the predicted peak flows in this range of recurrence intervals at other locations in the reach are also most likely too low. The following sections describe the methods used to estimate peak floods at the locations of interest under existing and 2020 planned land use conditions.

5.1. Existing Conditions

Peak discharges at the 11th Street gage from the frequency analysis of the measured data discussed in Section 2 (Figure 3) are somewhat higher for the more commonly occurring flows, ranging from 1.2 times higher than the model-based frequency curve at the 2-year event to 1.1 times higher at the 20-year event (**Table 3**). The curves gradually converge at the less frequent flows, intersecting at about the 100-year recurrence interval. Existing conditions peak discharges at the locations of interest in the study area for which flow-duration curves were developed were estimated by assuming that the difference between the data-derived flood peaks and those from the HSPF model would be the same as those at the 11th Street gage. The specific values at each location, therefore, were estimated by performing a frequency analysis on the model results at each model node, and multiplying the ordinates of the resulting flood-frequency curves by the ratios listed in Table 3. A summary of the estimated flood-frequency curves is presented in **Table 4**.

Table 3. Computed peak flood-frequency analysis of the measured data and HSPF model results at the South 11th Street gage on the Kinnickinnic River for the overlapping period of record, and the shift factor used to adjust the HSPF-based curves at the other locations of interest.

Exceedence Percentage	Recurrence Interval (yrs)	Discharge based on Measured (cfs)	Discharge based on HSPF Results (cfs)	Adjustment Factor
0.2	500	11,300	12,300	0.92
0.5	200	9,970	10,300	0.97
1	100	8,960	8,930	1.00
2	50	7,980	7,670	1.04
5	20	6,700	6,150	1.09
10	10	5,740	5,080	1.13
20	5	4,760	4,070	1.17
50	2	3,320	2,730	1.22
80	1.25	2,320	1,890	1.23
90	1.111	1,920	1,580	1.22
95	1.053	1,650	1,370	1.20
99	1.01	1,230	1,070	1.15

Table 4. Summary of estimated flood-frequency characteristics under existing conditions.								
Location	Model Node	Peak Discharge (cfs)						
		1.5-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Kinnickinnic River								
KKR at 6th Street	807	2,570	3,190	4,630	5,540	6,590	7,350	8,050
KKR at 16th Street	808	2,480	3,080	4,480	5,360	6,380	7,120	7,800
KKR above WPC	710	930	1,150	1,720	2,090	2,560	2,910	3,260
KKR Above S. 43rd St Ditch	709	560	720	1,160	1,460	1,850	2,140	2,450
KKR at 51st Street	809	450	580	930	1,180	1,480	1,730	1,960
Lyon's Park Creek								
LPC above conf w/ KKR	831	430	550	850	1,060	1,330	1,520	1,720
LPC below OK Ave	31	310	400	620	770	960	1,100	1,240
S. 43rd Street Ditch								
43rd St Ditch at Mouth	801	460	520	650	730	810	860	910
Wilson Park Creek								
WPC at Mouth	818	1,730	2,150	3,100	3,640	4,230	4,620	4,960
WPC above CPC	722	1,360	1,690	2,440	2,890	3,400	3,760	4,070
WPC above VMC	823	1,040	1,300	1,960	2,380	2,890	3,260	3,600
WPC above HAC	828	570	730	1,130	1,380	1,670	1,880	2,080
WPC above GMF Trib	824	140	180	300	390	510	610	730
Edgerton Ditch								
ED above GMF Airport	826	210	260	390	480	590	680	770
ED Above Tunnel	26	20	24	37	47	60	70	80
Tributaries to Wilson Park Creek								
VMC at Mouth	820	380	480	730	880	1,050	1,180	1,280
VMC Mainstem above Trib	521	170	200	280	330	400	450	500
VMC Trib at Mouth	20	170	210	310	370	440	480	530
VMC Trib U/S Limit	520	74	90	150	180	220	250	270
HAC at Mouth	830	430	540	810	1,000	1,240	1,440	1,630

5.2. 2020 Planned Land-use Conditions

Flood-frequency characteristics under future conditions were estimated at locations of interest by performing a flood-frequency analysis of the peak flow results from the HSPF model for 2020 PLU conditions. Under the assumption that the HSPF model results for 2020 PLU conditions deviate from actual peak flow characteristics in a similar manner as identified for existing conditions, the HSPF model results were adjusted using the shift factors for existing conditions (Table 3). A comparison of the estimated peak discharges indicates that the peak values under 2020 PLU conditions will be only slightly larger than the existing conditions peaks throughout most of the basin, with increases of less than 5 percent over the range of peak flows at most locations (**Figure 19**). The exception to this general statement occurs in the upstream reaches of the Wilson Park Creek watershed (i.e., Edgerton Ditch), where the peak flow increases by about 42 percent at the 2-year recurrence interval and about 12 percent at the 100-year recurrence interval. The estimated future conditions peak discharges are summarized in **Table 5**.

Table 5. Summary of estimated flood-frequency characteristics under 2020 planned land use conditions.

Location	Model Node	Peak Discharge (cfs)						
		1.5-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Kinnickinnic River								
KKR at 6th Street	807	2,620	3,230	4,690	5,590	6,640	7,390	8,070
KKR at 16th Street	808	2,530	3,130	4,540	5,410	6,420	7,150	7,820
KKR above WPC	710	940	1,160	1,730	2,100	2,590	2,940	3,300
KKR Above S. 43rd St Ditch	709	560	720	1,160	1,460	1,850	2,150	2,450
KKR at 51st Street	809	450	580	930	1,180	1,480	1,730	1,960
Lyons Park Creek								
LPC above conf w/ KKR	831	430	550	860	1,060	1,330	1,520	1,720
LPC below OK Ave	31	310	400	620	770	960	1,100	1,240
S. 43rd Street Ditch								
43rd St Ditch at Mouth	801	470	540	680	750	840	900	950
Wilson Park Creek								
WPC at Mouth	818	1,780	2,200	3,130	3,670	4,240	4,630	4,960
WPC above CPC	722	1,410	1,740	2,480	2,900	3,360	3,670	3,930
WPC above VMC	823	1,080	1,350	2,010	2,430	2,930	3,290	3,620
WPC above HAC	828	590	760	1,160	1,410	1,710	1,920	2,130
WPC above GMF Trib	824	180	220	350	450	580	700	820
Edgerton Ditch								
ED above GMF Airport	826	270	330	470	560	670	750	820
ED Above Tunnel	26	28	34	49	59	71	80	89
Tributaries to Wilson Park Creek								
VMC at Mouth	820	390	490	730	880	1,060	1,180	1,290
VMC Mainstem above Trib	521	170	200	280	340	400	450	500
VMC Trib at Mouth	20	180	220	320	380	440	490	530
VMC Trib U/S Limit	520	76	100	150	180	220	250	270
HAC at Mouth	830	440	550	820	1,010	1,260	1,450	1,640

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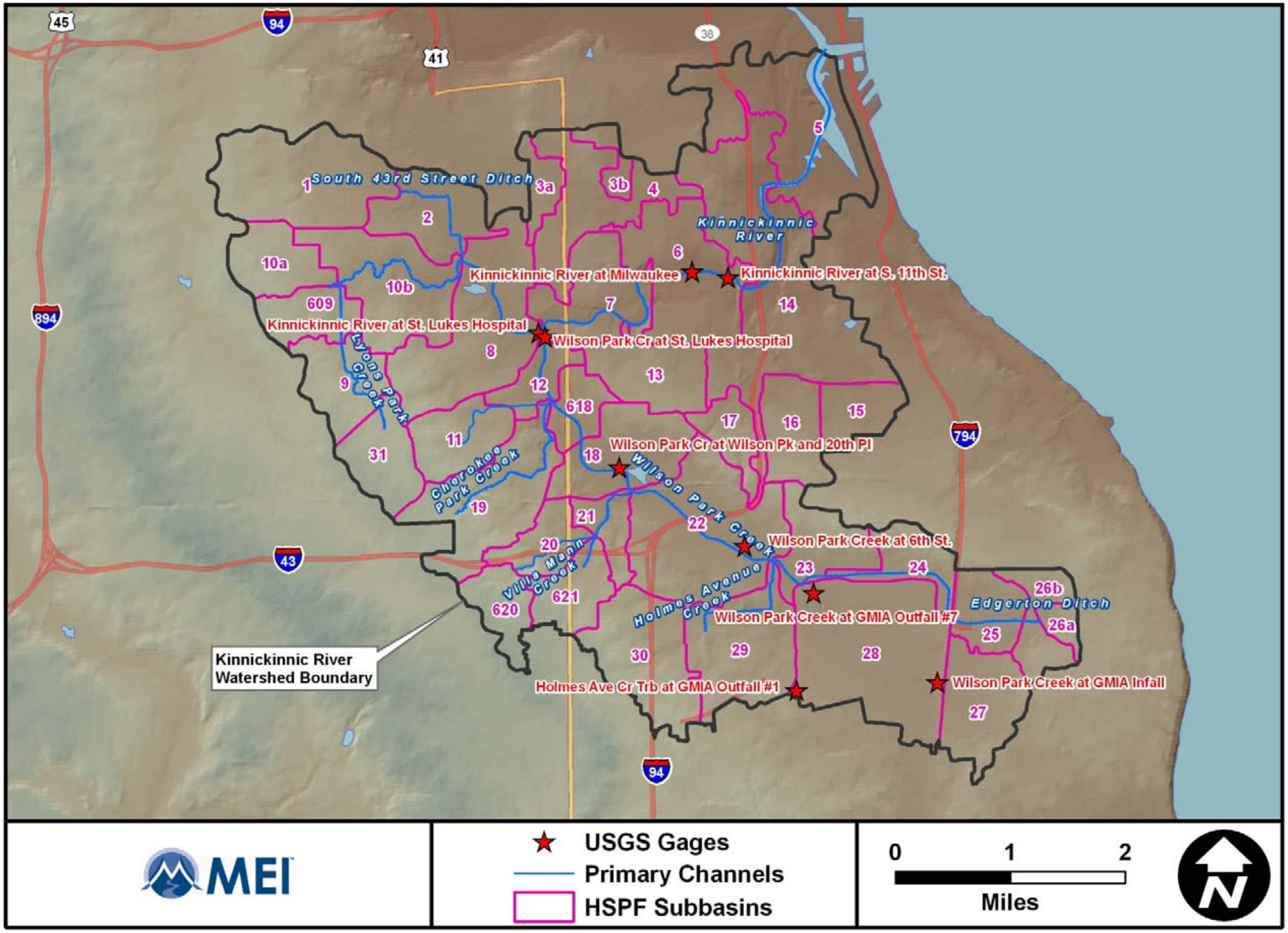

Figure 1. Map showing the overall Kinnickinnic River watershed, the HSPF model subbasins, and the location of the USGS stream gages within the study area.

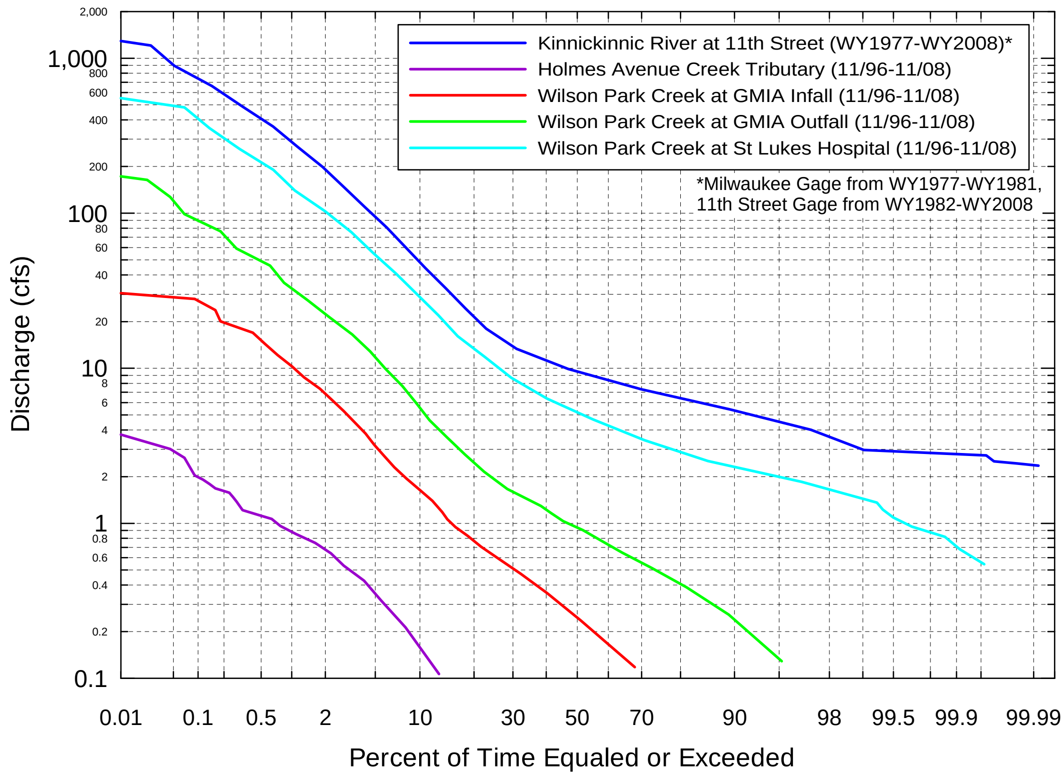

Figure 2. Computed flow-duration curves from the mean daily flow data at USGS gages with sufficient periods of record.

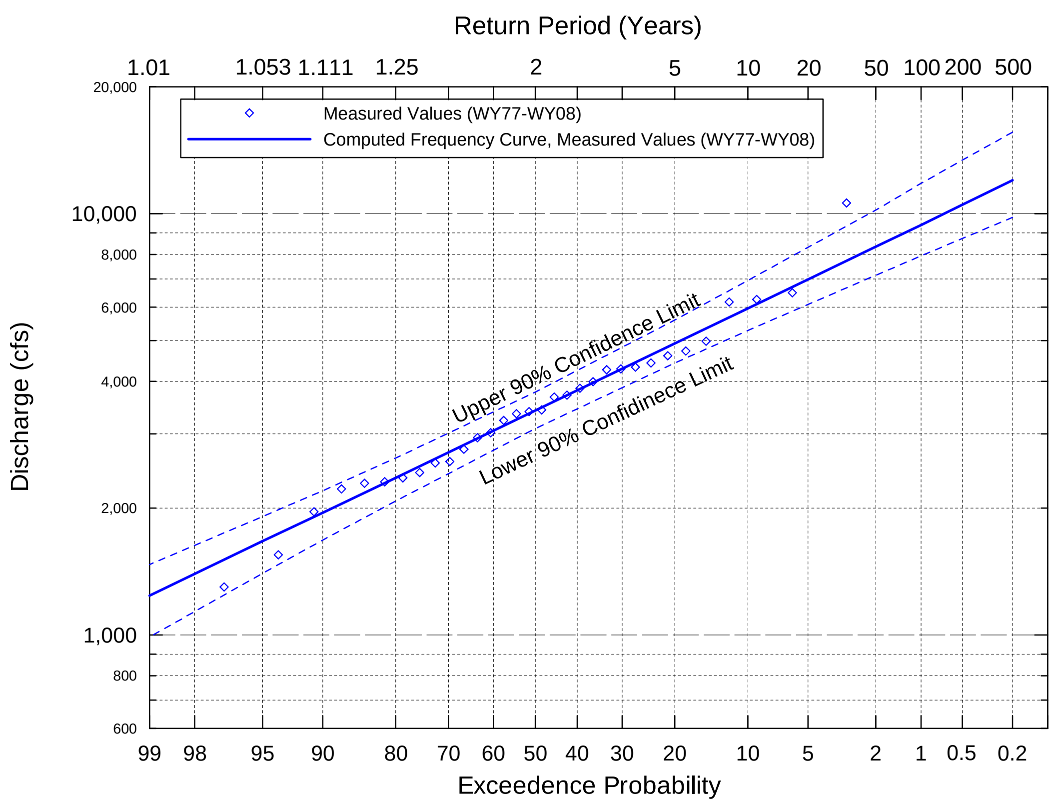

Figure 3. Computed flood-frequency curve for the Kinnickinnic River at S. 11th St. gage (USGS Gage No. 04087159) for the period between WY1977 and WY2008.

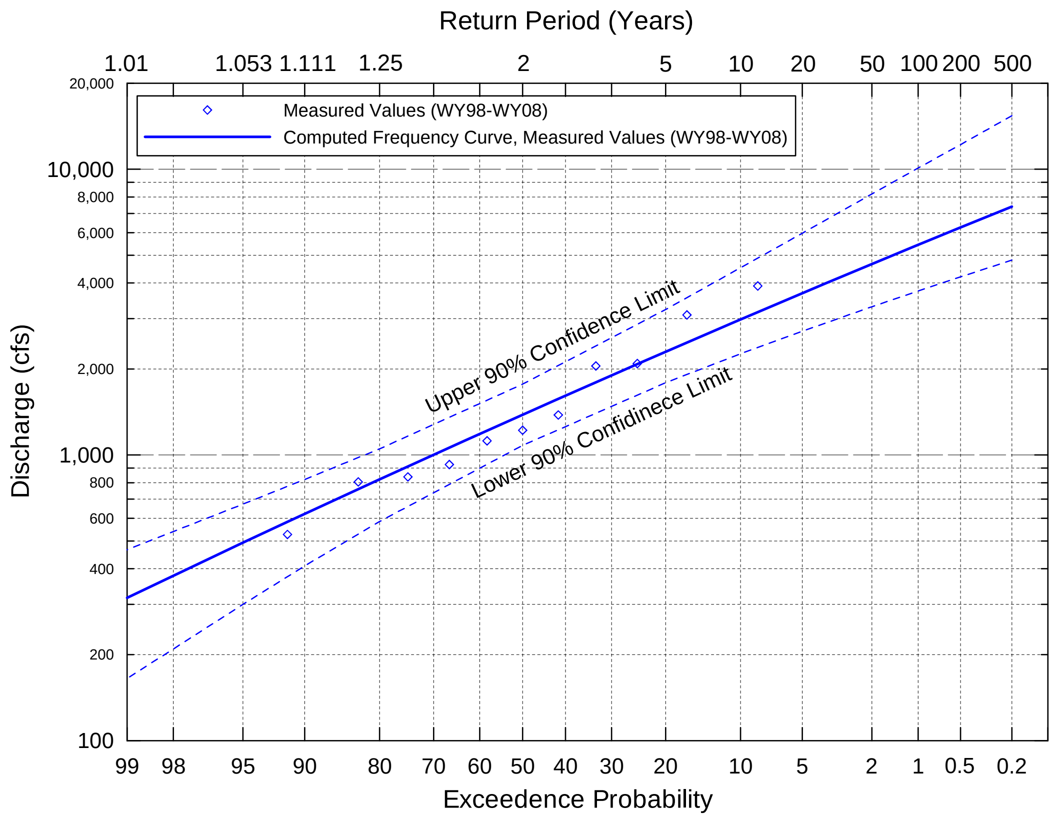

Figure 4. Computed flood-frequency curve for the Wilson Park Creek at St. Luke's Hospital gage (USGS Gage No. 040871488) for the period between WY1998 and WY2008.

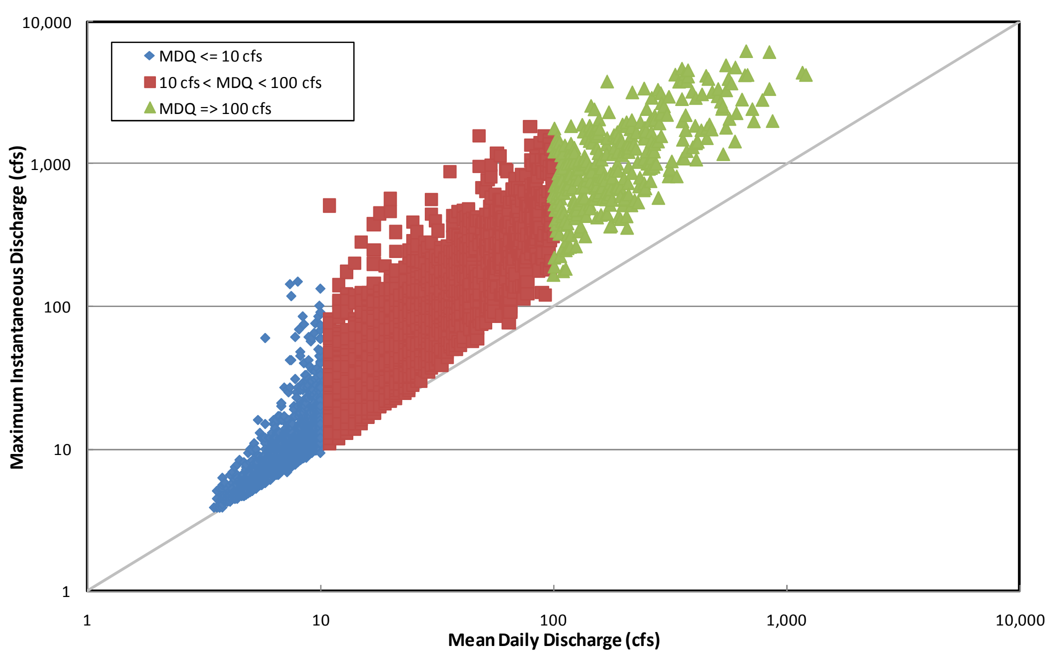

Figure 5. Comparison of measured mean daily discharge and the corresponding maximum instantaneous discharge for each day of record at the Kinnickinnic River South 11th Street gage.

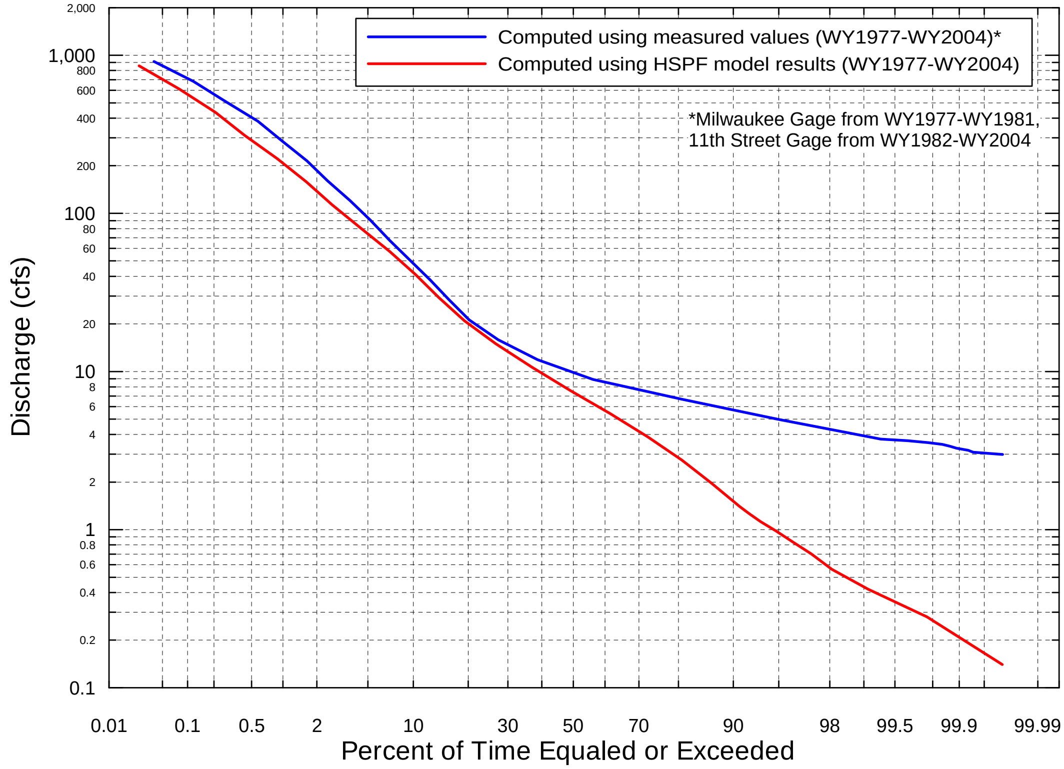

Figure 6. Mean daily flow-duration curves based on the measured data at the Kinnickinnic River at 11th Street/Milwaukee gages and the HSPF model output at Node 807 for the concurrent period of record between WY1977 and WY2004.

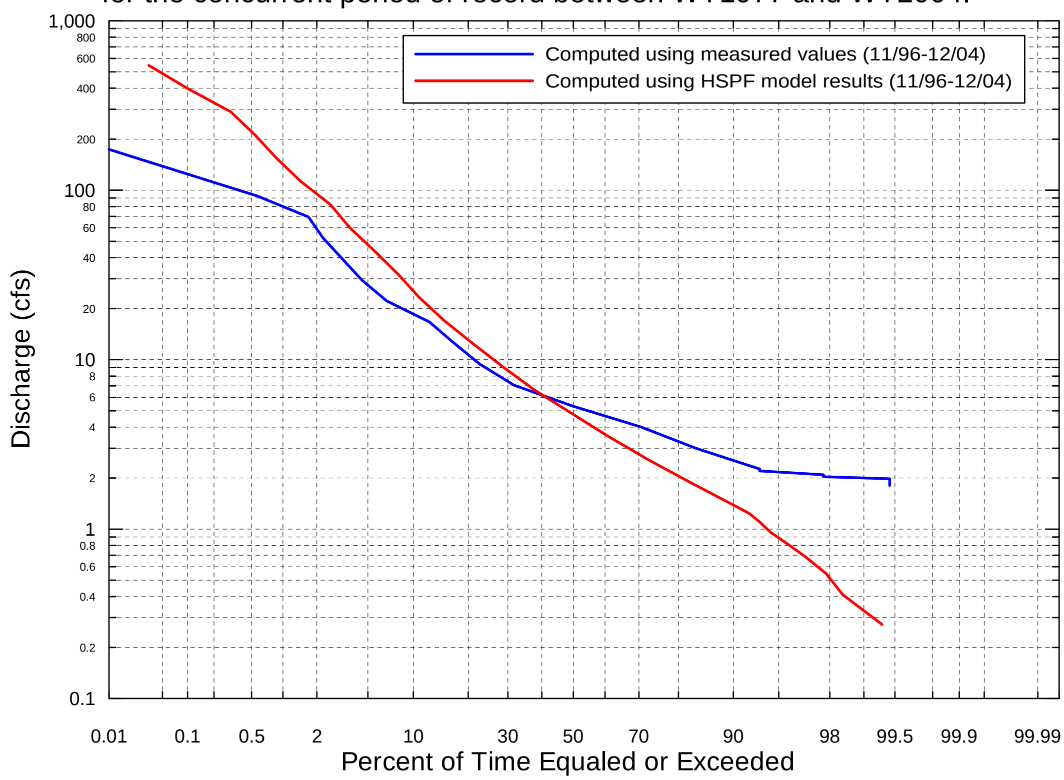

Figure 7. Mean daily flow-duration curves based on the measured data at the Wilson Park Creek at St. Luke's Hospital gage and the HSPF model output at Node 818 for the concurrent period of record between November 1996 and December 2004.

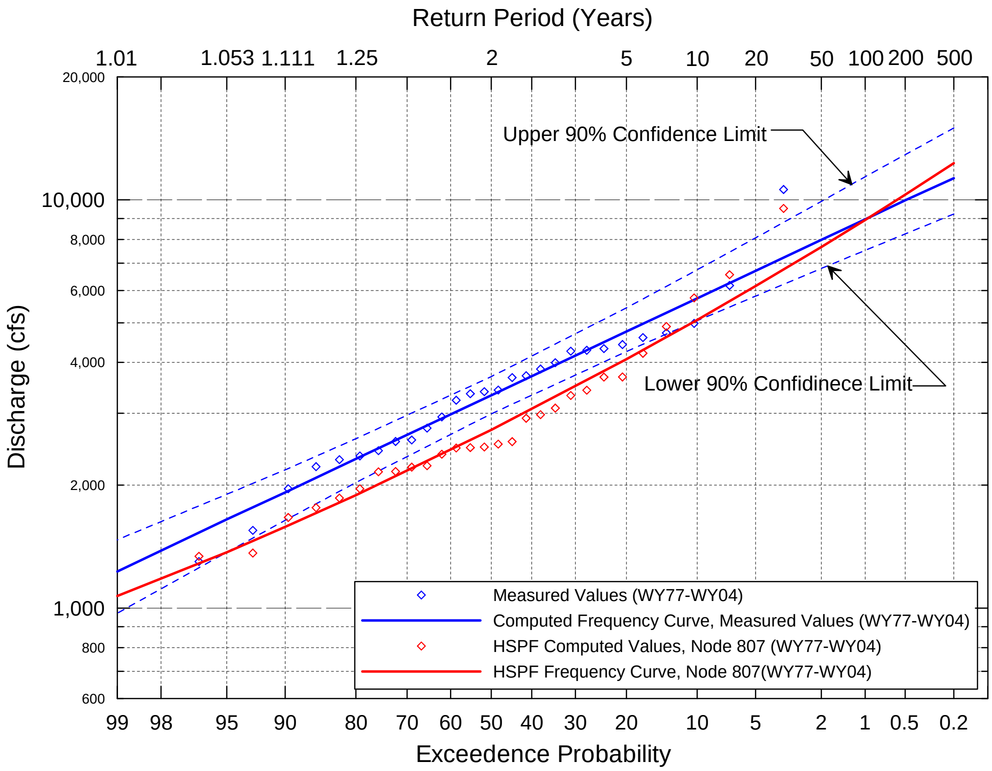

Figure 8. Computed flood-frequency curves based on measured peak flows at the Kinnickinnic River at South 11th Street gage and from the HSPF model at Node 807 for the coincident period of record between WY1977 and WY2004.

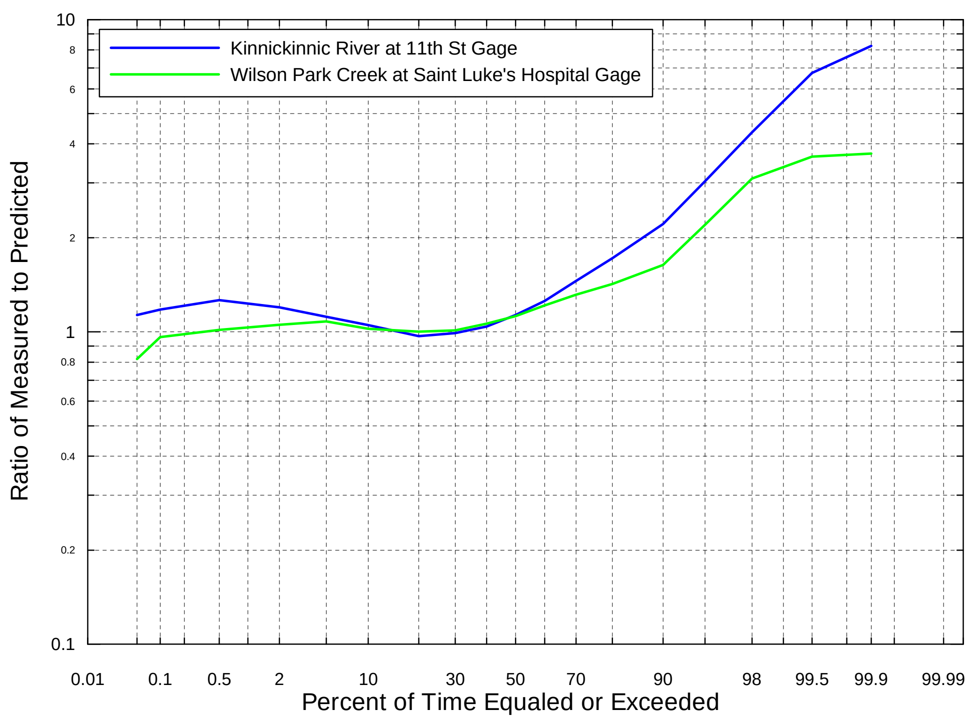

Figure 9. Computed shift factors for the mean daily flow-duration curves from the HSPF model results at the Kinnickinnic River at South 11th Street and Wilson Park Creek Saint Luke's Hospital gages.

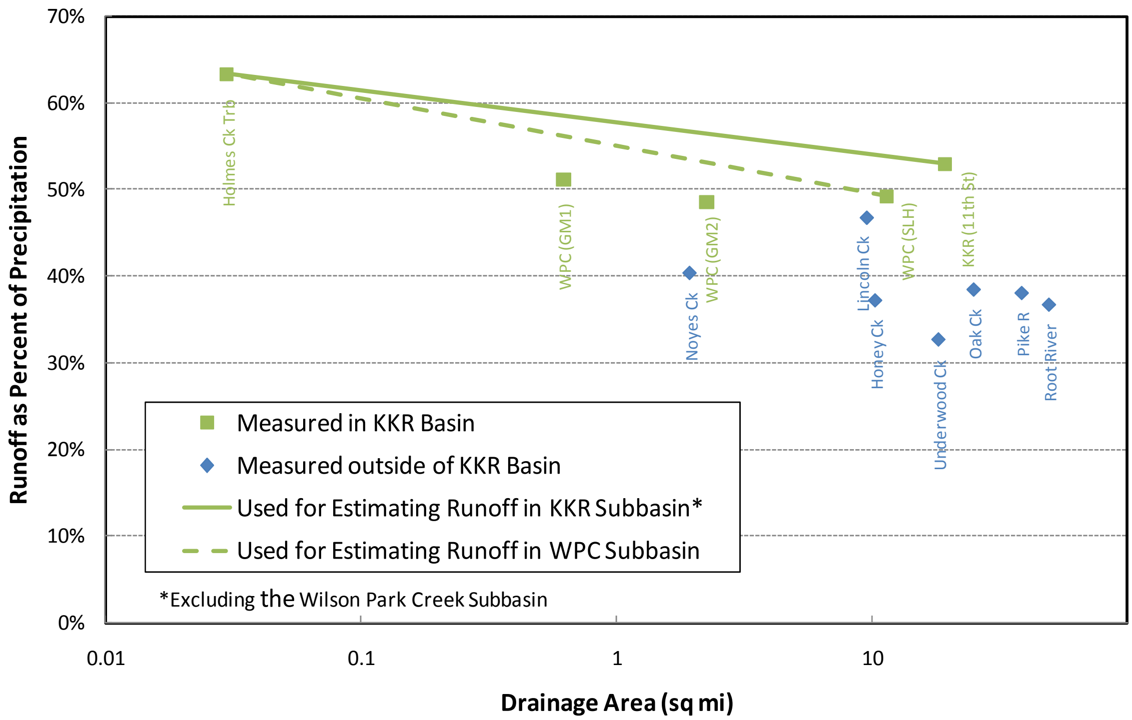

Figure 10. Measured runoff as percent of precipitation at gages on streams in urbanized areas in south Milwaukee.

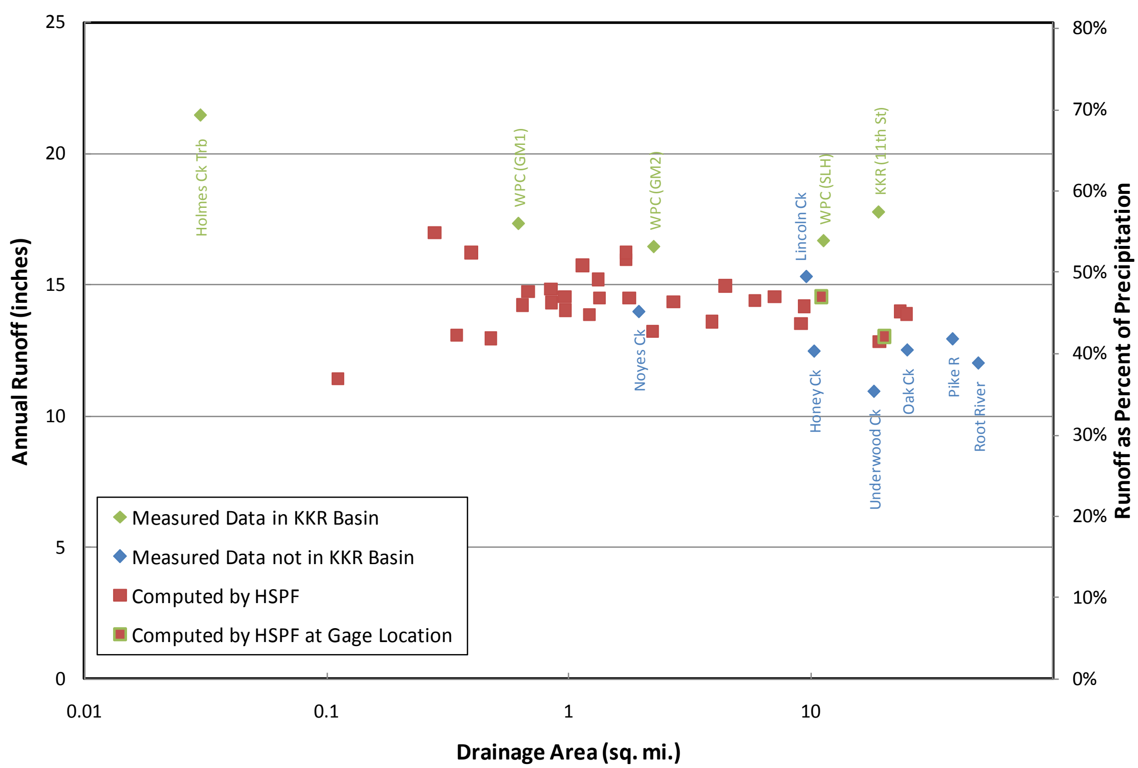

Figure 11. Average annual runoff predicted by the HSPF model for various locations in the study area, and measured runoff at stream gages in urbanized watersheds in the south Milwaukee area.

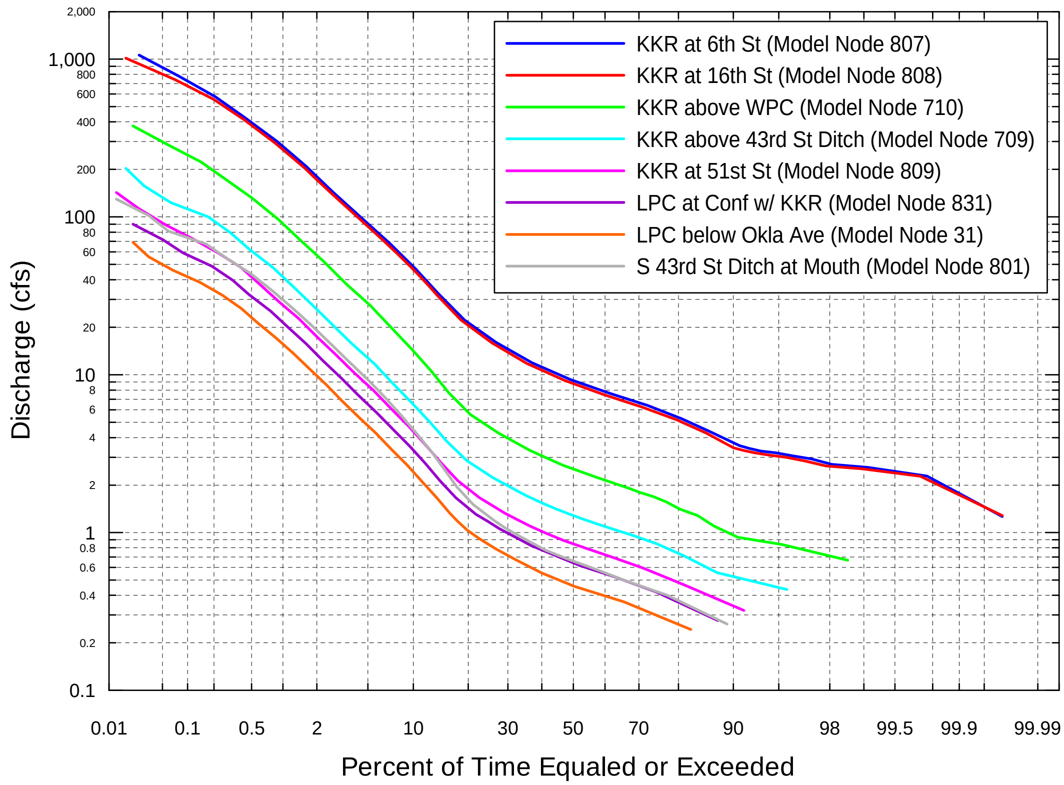

Figure 12. Flow-duration curves along the Kinnickinnic River, Lyons Park Creek, and South 43rd Street Ditch under existing conditions.

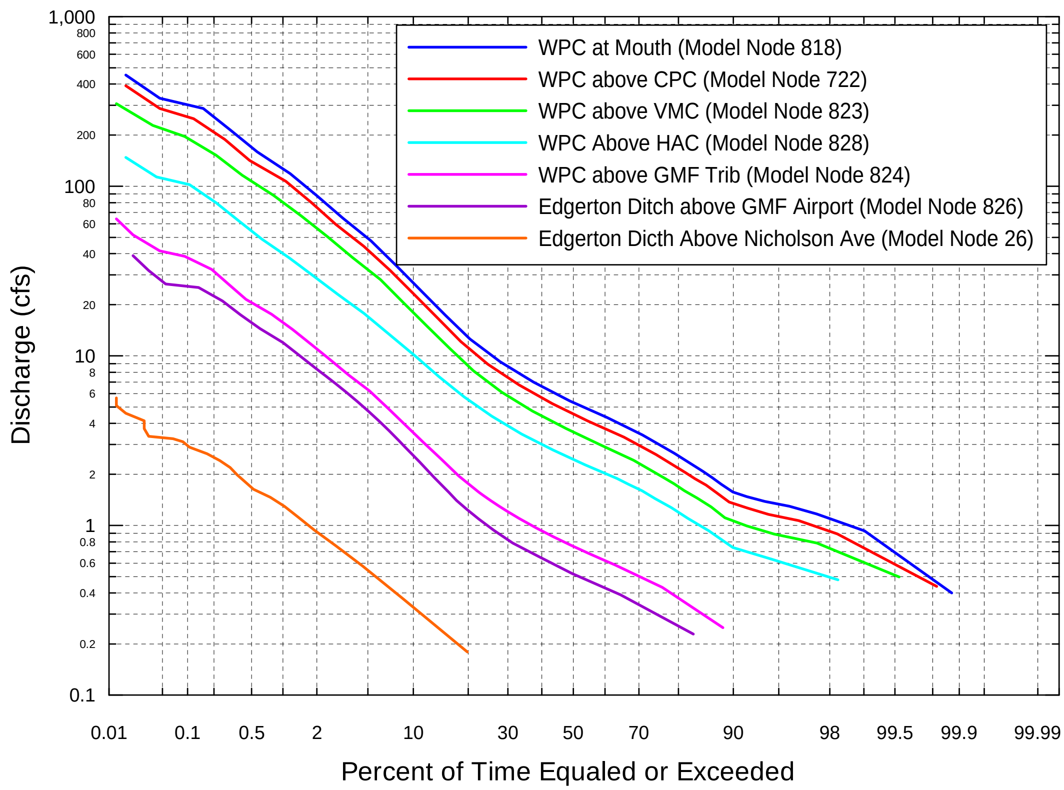

Figure 13. Flow-duration curves along Wilson Park Creek and Edgerton Ditch under existing conditions.

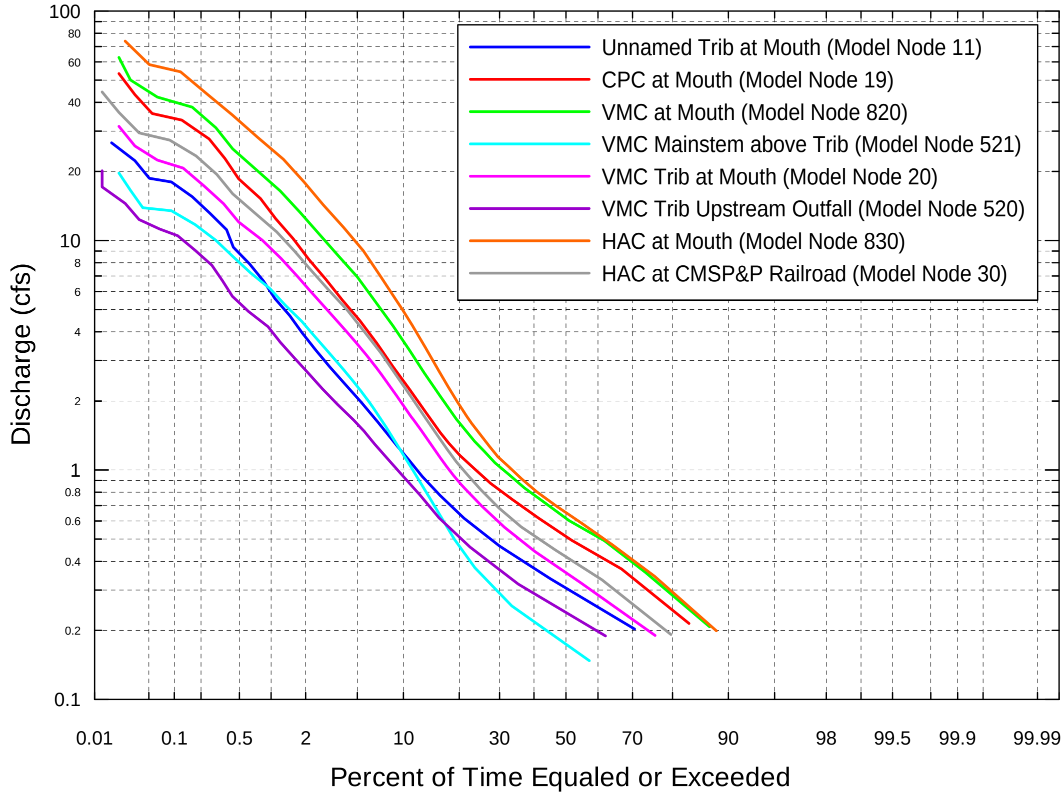

Figure 14. Flow-duration curves along the tributaries to Wilson Park Creek under existing conditions.

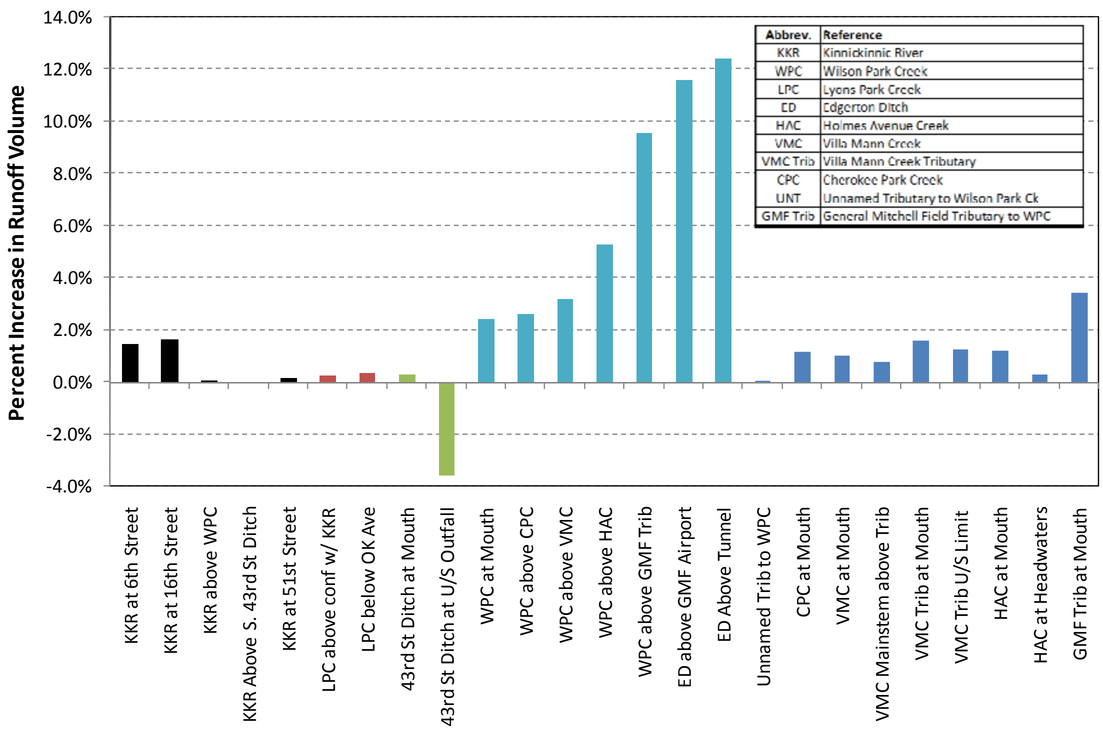

Figure 15. Percent increase in runoff volume from existing to future conditions predicted by the HSPF model.

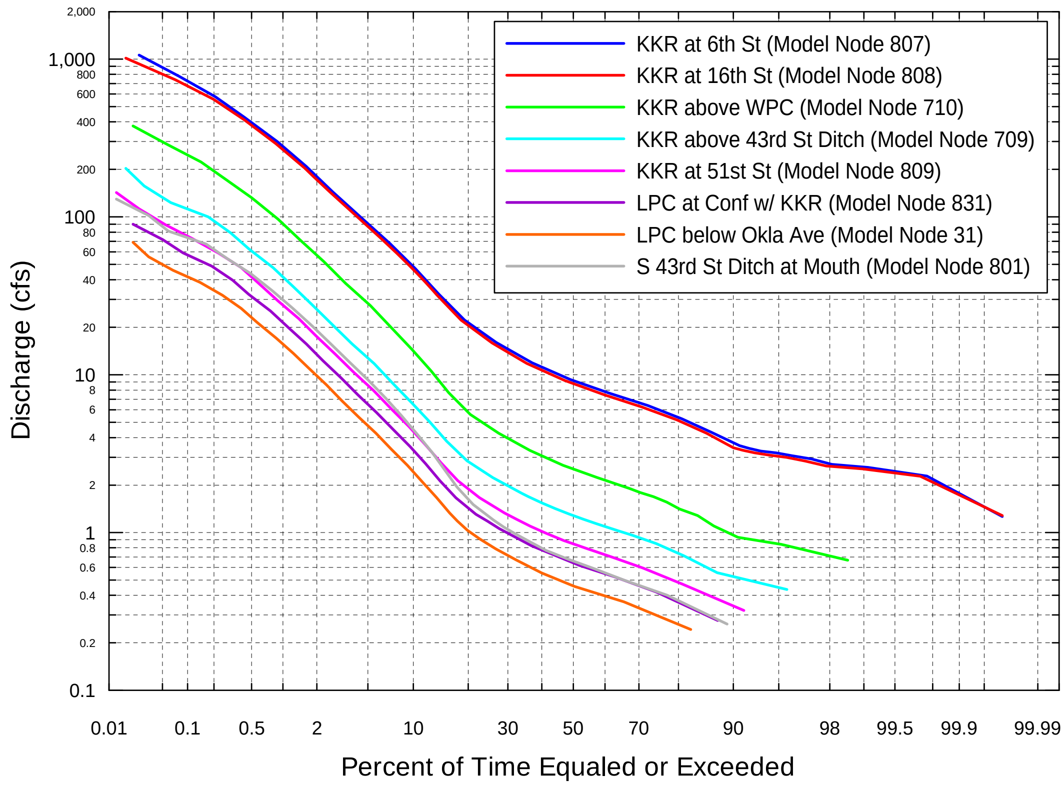

Figure 16. Flow-duration curves along the Kinnickinnic River, Lyons Park Creek, and South 43rd Street Ditch under 2020 planned land-use conditions.

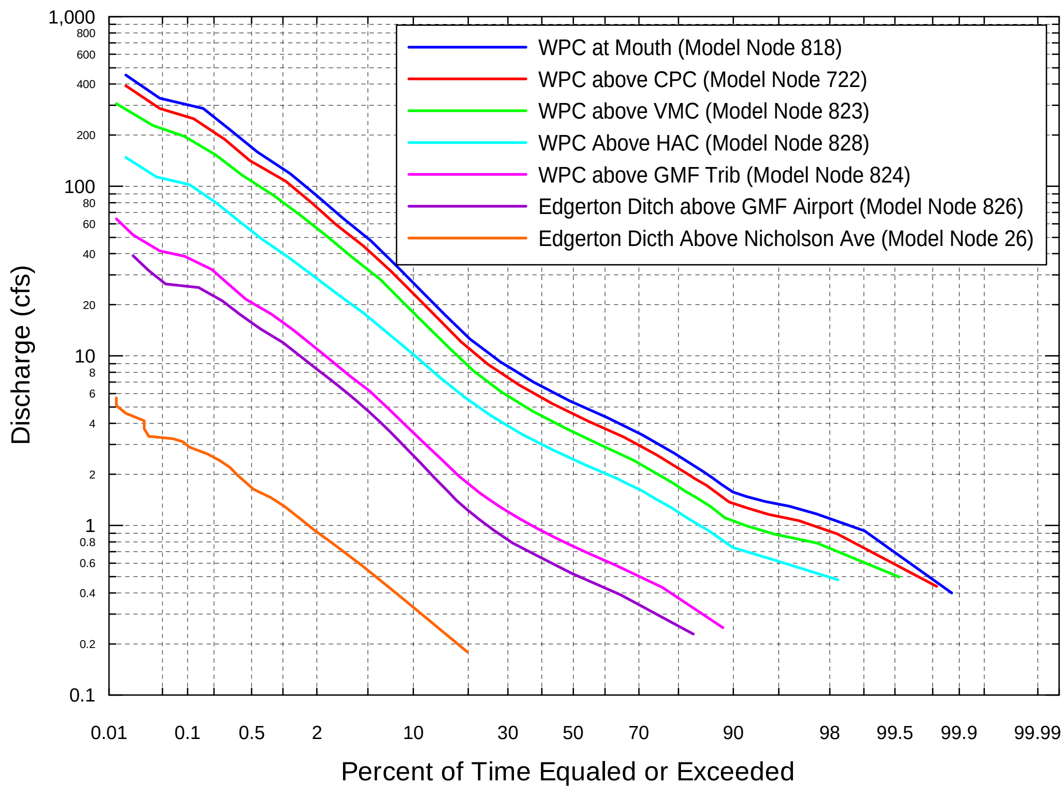

Figure 17. Flow-duration curves along Wilson Park Creek and Edgerton Ditch under 2020 planned land-use conditions.

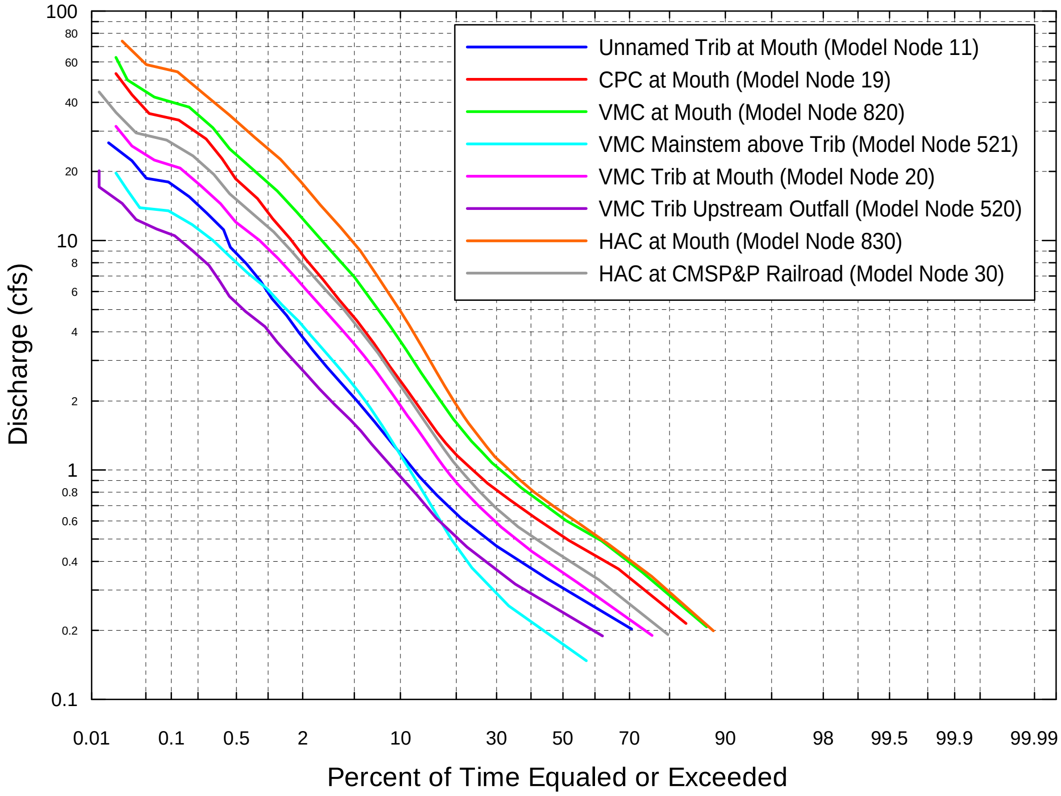

Figure 18. Flow-duration curves along the tributaries to Wilson Park Creek under 2020 planned land-use conditions.

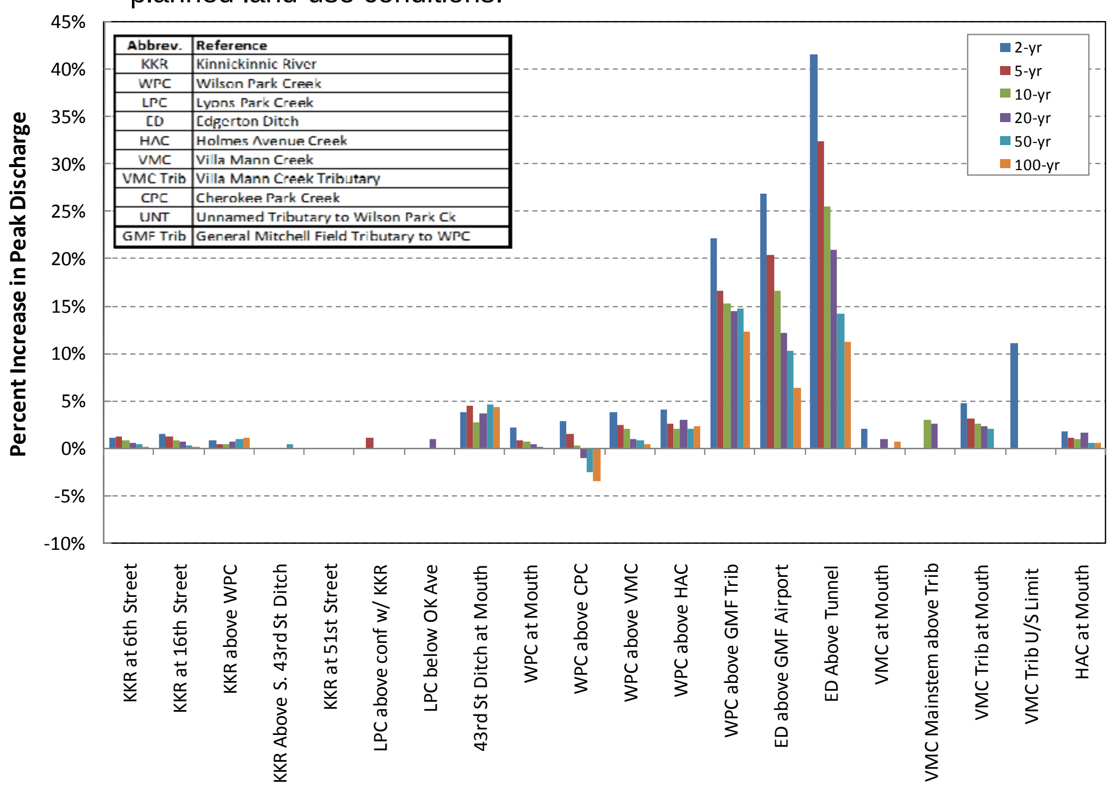

Figure 19. Estimated percent increase in peak discharge from existing to 2020 planned land-use conditions.