August 2012

Stormwater Master Plan 2012

CITY OF ST. PETERS, MISSOURI

BLACK AND VEATCH
PATTI BANKS ASSOCIATES

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Report Summary

This Stormwater Master Plan will be used for planning of future capital improvements, improving water quality, and continuing compliance with Missouri Department of Natural Resources Municipal Separate Storm Sewer Systems (MDNR MS4) regulations. Preliminary investigations have evaluated the existing conditions of the City's open-channel stormwater infrastructure based on hydrologic and hydraulic modeling, field reconnaissance, water quality modeling, and review of relevant policies at the City, State, and Federal levels.

The City of St. Peters has a population of approximately 57,000 and encompasses about 22 square miles. The City owns, operates and maintains over 165 miles of storm sewer pipe and associated structures, as well as 3 pump stations. The City contains more than 47 miles of waterway, 55 stream/roadway crossings, 43 wet retention basins and 207 dry detention basins. The majority of the City is situated in the Dardenne Creek Watershed, a 29 mile long basin that covers approximately the middle third of St. Charles County and drains to the Mississippi River. The primary subwatershed in St. Peters is Spencer Creek. Other subwatersheds identified by the US Army Corps of Engineers (USACE) include East Dardenne, Sandfort Creek, and un-named Tributaries No. 1 and No. 2. A small portion in the southeast of St. Peters drains to the Missouri River.

Hydrologic and Hydraulic Modeling

The City's current stormwater master plan, completed in 2002, included computer models developed at a level of detail suitable for master planning. The 2002 study identified flooding and erosion problems within the City's watersheds, and provided conceptual improvements to alleviate the defined problems. In 2007, the USACE St. Louis District completed a study of the entire Dardenne Creek watershed. Both the 2002 and 2007 studies were referenced during the analysis of existing conditions.

The HEC-HMS model developed by the USACE St. Louis District was used as the basis for hydrologic modeling of the watersheds within and around St. Peters. The detention basins selected through a screening process were incorporated into the USACE hydrology model to allow the opportunity to gage effectiveness, model detention basin improvements, and evaluate downstream erosion effects. The methodology developed by the USACE St. Louis District was maintained for the hydrologic analysis of this watershed study and modifications to the model were limited to the addition of detention basins.

To evaluate open channel hydraulics, the HEC-RAS model developed by the USACE St. Louis District in 2007 was refined to represent the most current topographic information within the City limits. The USACE 2007 was based on a 3D terrain model created with digital orthophotos. The 2007 study model was based on additional data including limited bridge and culvert drawings, previous hydraulic models, rainfall and streamflow data, land use and soil maps, and other GIS data. Black & Veatch evaluated the existing model to establish continuity of peak flows, determine extents, and identify modeling constraints within the city limits. The review of the 2007 hydraulic model produced several concerns described in this report. For this study, Black & Veatch refined the 2007 HEC-RAS hydraulic model to

represent the most current topographic information within the City limits, based on LiDAR data collected in 2008.

Flooding

Based on the updated hydraulic modeling, the area of inundation was established for the revised existing conditions model during the 100-year event. The results of the 2007 USACE hydraulic model were compared to the results of the updated model. On average, there was less than one foot of change in 100-year water surface elevation between the revised existing conditions model and the original 2007 USACE model. Typically, the water surface elevation decreased. There were, however, significant differences between the areas of inundation and the updated FEMA floodplain maps.

Stream Stability and Habitat

In April and May of 2011, Black & Veatch and PBA staff joined the City to evaluate stream stability and habitat within the City limits. As a team, Black & Veatch and PBA conducted a geomorphology and stream health field survey of the 47 miles of streams in the City of St. Peters. The team applied standards developed by the Kansas City Metropolitan Chapter of American Public Works Association and the United States Department of Agriculture to evaluate stream stability throughout the City.

A majority of the stream network in St. Peters was documented to be incised, as described in the 2002 watershed study. Channel incision is a response to changes in the hydrology of the contributing drainage area as well as to changes in channel bed materials and downstream conditions. The stream network within the City limits has varying levels of stability and incision. The downstream ends of the main tributaries to Spencer Creek, West Spencer and East Spencer, have already experienced changes in structure and are now very incised. In these streams, further incision is likely to occur due to a small low flow channel cutting into the hard clay bed that was documented in the field. Reaches higher up in the watershed have very steep profiles and debris jams of roots or other material were often found to provide temporary vertical grade control.

Water Quality

The P8 model was used to predict pollutant removal efficiencies for each of the selected stormwater detention basins. The model simulated the generation and transport of stormwater runoff pollutants in the watersheds contributing to the detention basins. The simulations were driven by continuous hourly rainfall based on ten years of data recorded at the Lambert International Airport provided by the National Climatic Data Center.

Policies

This study provides a summary existing policies, ordinances, and design criteria. It also identifies regulations and other factors that may influence future policies and identifies preliminary recommendations for updating and adopting polices to meet regulations and achieve the goals for storm water management in St. Peters. The recommendations to the City's stormwater policies are

suggested in the following areas: 1) Best Management Practices, 2) Low Impact Development, 3) Stream Setbacks, 4) FEMA Community Rating System, 5) Street Sweeping, 6) Sediment and Erosion Control, 7) Homeowner Drainage Issues, and 8) Education and Awareness.

CIP

This study resulted in the development of over 100 projects with a total cost of $125,000,000. These projects are located throughout the City and are classified as flooding, stream stability, detention, or preservation projects. Some projects are multi-functional and are associated with two or more of these categories. A comprehensive approach to solving stormwater-related concerns will include projects that are rated with a high priority score, projects that have a low cost benefit ratio and projects that preserve the City's existing resources. An ESRI geodatabase accompanies this report and provides a tool for the City to use in managing CIP projects.

Nine Critical Element Plan

The Nine Critical Element plan is provided in Appendix A.

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1. Introduction

This master plan will be used for planning of future stormwater capital improvements, improving water quality, and continuing compliance with Missouri Department of Natural Resources Municipal Separate Storm Sewer Systems (MDNR MS4) regulations. The Nine Critical Element Plan, presented in Appendix A, may be used to help secure funding for water quality related improvements.

The City of St. Peters, Missouri has a population of approximately 57,000 and encompasses approximately 22 square miles. The City owns, operates and maintains over 166 miles of storm sewer pipe and associated structures, as well as three pump stations. The City contains more than 47 miles of waterway, 55 stream/roadway crossings, 43 wet retention basins and 207 dry detention basins. The majority of the City is situated in the Dardenne Creek Watershed, a 29 mile long basin that covers approximately the middle third of St. Charles County and drains to the Mississippi River. The primary subwatershed in St. Peters is Spencer Creek. Other subwatersheds identified by the Corps of Engineers include East Dardenne, Sandfort Creek, and un-named Tributaries No. 1 and No. 2. A small portion in the southeast of St. Peters drains to the Missouri River.

2.0 Review of Existing Data and Conditions

2.1 Existing Data

At the beginning of the study, the City provided a copy of the existing stormwater master plan, completed in 2002. This master plan included computer models developed at a level of detail suitable for master planning, identified flooding and erosion problems within the City's watersheds, and provided conceptual improvements to alleviate the defined problems.

The City also provided hydrologic and hydraulic models developed by the USACE St. Louis District for a 2007 study of the Dardenne Creek Watershed. GIS data was provided by the City and USACE. After Black & Veatch conducted a detention basin screening analysis, described later in the report, the City compiled available documentation for each detention basin of interest. Additional information provided by the City through the study included CAD drawings of the Highway 94 improvements and pipe information related to the drainage east of Old Town.

2.1.1 Hydrologic Model

The hydrologic model was provided to Black & Veatch by the USACE St. Louis District. This model, developed by the USACE St. Louis District in 2006 - 2007, encompasses nearly the entire City limits and was calibrated to a series of rainfall events using two gauges along Dardenne Creek. Existing condition and future condition peak flows were developed for use in the hydraulic modeling.

2.1.2 Hydraulic Model

The steady state HEC-RAS model, developed by the USACE in 2007, uses the output flows from the HEC-HMS model and routes these flows through a stream network that includes approximately 20 miles of stream within the City limits.

2.1.3 GIS Data

Data received from the City and the Corps of Engineers (District) is presented in Table 1.

TABLE 1. GIS DATA AND SOURCES

GIS Data Received	Source
Aerials	*City of St. Peters*
2-foot contours	*City of St. Peters*
Geodatabase Layers	*City of St. Peters*
City Boundary	
Floodplain and Floodway	
Building Footprints	
Parcels and Easements	
Streams and Water Bodies	
Watersheds	
Land Use and Zoning Areas	
Transportation Corridors	
Stormwater Infrastructure	
Sewer Infrastructure	
Water Infrastructure	
GeoHMS Data	*USACE*
Gages	
Survey Points	
Subwatersheds	
GeoRAS Data	*USACE*
Cross sections	
Flowpaths	
Stream Centerline	

2.1.4 Detention Basin Documentation

The City compiled available documentation for each detention basin of interest. The level of detail in each document was variable and Black & Veatch was able to confirm existing conditions as documented on most detention basins. For each detention basin, Black & Veatch conducted a more thorough field survey to document existing conditions and to better determine outfall and storage characteristics. A standard documentation form, provided in Appendix B, was completed for each detention basin, and multiple pictures were taken for the modeling and development of conceptual recommendations.

Detention Basin Screening Process

Black & Veatch developed a screening process with the City to prioritize existing detention facilities for inclusion in the hydrologic model. A total of 248 detention basins were included on the geodatabase provided by the City of St. Peters. The scope of the project included modeling of up to 75 detention basins. The screening process outlined below resulted in the selection of the most significant detention in the City.

Of the 248 detention basins, 82 had a surface area greater than ½ acre. The 166 detention basins with a smaller surface area were considered to have a less significant impact on the hydrology and were excluded. There were 29 basins with a surface area greater than 1 acre and these facilities were automatically included. The 53 basins that were between ½ acre and 1 acre in surface area were further analyzed to determine their significance. An additional four selection criteria were developed to screen the remaining basins. Basins that are located on public or vacant land are more easily retrofitted or altered because of ownership issues. Detention facilities that are properly sized for the associated contributing drainage area have increased functionality for affecting peak flows. Contributing drainage areas were established automatically using the ESRI ArcHydro tools. Similarly, detention basins in the floodplain are adjacent to streams and typically show improved function. Of the 53 intermediate sized basins, 5 basins were located on public land, 7 basins were situated on vacant land, 15 basins had a surface area that was greater than 5% of the contributing drainage area, 4 basins were located within a new subdivision, and 3 basins were located within a floodplain. One of the basins located within a floodplain, DB-7270-05 in Carrington Estates, was selected by the screening process and then excluded due to lack of available data. An additional 12 basins were selected by the City to be evaluated, bringing the total to 74 basins. The detention basins included in the study are presented in Figure 1. Each of these basins was visited in the field. A list of selected basins is presented in Table 2.

TABLE 2. SELECTED DETENTION BASINS

Detention ID	Location	CDA (acres)	% Impervious
DB-6869-02	FAWN LAKE	31	26%
DB-6870-15	CHURCH	5	34%
DB-6870-16	MANLIN HOMES AT BELLEAU CREEK	17	9%
DB-6871-01	I-70 TRADE CENTER	28	65%
DB-6871-03	INDUSTRIAL PARK WEST	27	66%
DB-6965-01	MONTECITO	7	13%
DB-6968-02	COUNTRY CROSSING MANOR	33	21%
DB-6969-02	WOODLANDS SPORK PARK	10	5%
DB-6969-03	LAKES OF DEVONDALE	41	10%
DB-6969-04	WOODLANDS SPORT PARK	35	1%
DB-6969-06	OHMES FARM	1	21%
DB-6969-07	OHMES FARM	10	5%
DB-6969-08	OHMES FARM	33	13%
DB-6969-09	OHMES FARM	5	8%
DB-6969-10	OHMES FARM	22	13%
DB-6970-03	LAKES OF DEVONDALE	15	17%
DB-6971-08	RICHMOND	24	35%
DB-7065-01	HIGHLAND ESTATES	1	22%
DB-7065-02	THE HIGHLANDS	2	17%
DB-7065-03	THE HIGHLANDS	5	20%
DB-7065-04	THE HIGHLANDS	197	11%

Detention ID	Location	CDA (acres)	% Impervious
DB-7065-05	MONTECITO	19	15%
DB-7067-01	MID RIVERS TOWN CENTER	44	54%
DB-7067-02	MID RIVERS PLACE	56	0%
DB-7069-06	CRYSTAL LAKE ESTATES	116	7%
DB-7069-07	CRYSTAL LAKE ESTATES	15	24%
DB-7069-08	LAKES OF DEVONDALE	11	24%
DB-7069-10	SAVANNAH	43	17%
DB-7069-12	BELLEMEADE	2	5%
DB-7069-13	OHMES FARM	16	13%
DB-7070-03	ST. PETERS VILLAS	38	52%
DB-7070-10	BELLEMEADE	10	1%
DB-7071-04	MID RIVERS MALL	32	78%
DB-7169-01	CARRINGTON PLACE	3	22%
DB-7169-02	DUBRAY MIDDLE SCHOOL	49	37%
DB-7169-04	PEGASUS FARMS	58	30%
DB-7169-08	BELLEMEADE	5	7%
DB-7169-09	BELLEMEADE	5	21%
DB-7169-10	BELLEMEADE	10	3%
DB-7169-11	BELLEMEADE	6	7%
DB-7267-04	PARK RIDGE ESTATES	32	29%
DB-7267-14	WALMART FIRST ADDITION	68	36%
DB-7268-04	COUNTRY CREEK	13	24%
DB-7268-06	ENWOOD	16	29%
DB-7268-07	HIDDEN LAKE ESTATES	21	12%
DB-7269-01	PENNY LANE	65	27%
DB-7270-01	ST. PETERS EXECUTIVE CENTRE	12	35%
DB-7270-09	I-70 EXECUTIVE CENTRE	37	26%
DB-7270-10	EXECUTIVE CENTRE PARKWAY	12	52%
DB-7270-15	CITY OF ST PETERS	27	43%
DB-7366-01	CANYON CREEK	72	37%
DB-7367-04	MC CLAY JUNGERMANN COMMERCIAL	6	41%
DB-7367-07	ASHLEIGH COMMERCIAL	7	42%
DB-7367-10	ASHLEIGH ESTATES	13	26%
DB-7367-19	QUEENSBROOKE VILLAGE	18	22%
DB-7367-20	BELLA VISTA	60	13%
DB-7368-08	COUNTRY LAKE ESTATES	13	25%
DB-7368-09	LAUREL PARK	7	10%
DB-7369-04	APPLEWOOD SUBDIVISION	13	18%
DB-7369-06	HI-POINT ACRES	93	16%
DB-7369-07	ST. PETERS CONDOS	29	28%

Detention ID	Location	CDA (acres)	% Impervious
DB-7369-11	OAK TREE VILLAGE RETIREMENT	40	37%
DB-7370-14	ST. PETERS CENTRE	51	24%
DB-7370-16	FORT ZUMWALT EAST	29	62%
DB-7467-04	SUN RIVER VILLAGE	46	35%
DB-7467-06	HERITAGE MANOR	21	28%
DB-7467-07	REGENCY ESTATES	19	50%
DB-7467-12	VILLAGES OF WINDWOOD	27	35%
DB-7467-13	FIRST CHRISTIAN CHURCH OF HARV	7	51%
DB-7467-14	BELLA VISTA	6	4%
DB-7467-15	BELLA VISTA	15	4%
DB-7467-16	BELLA VISTA	23	26%
DB-7468-01	HUNTLEIGH ESTATES	4	15%
DB-7468-06	LAUREL VILLAGE	161	21%

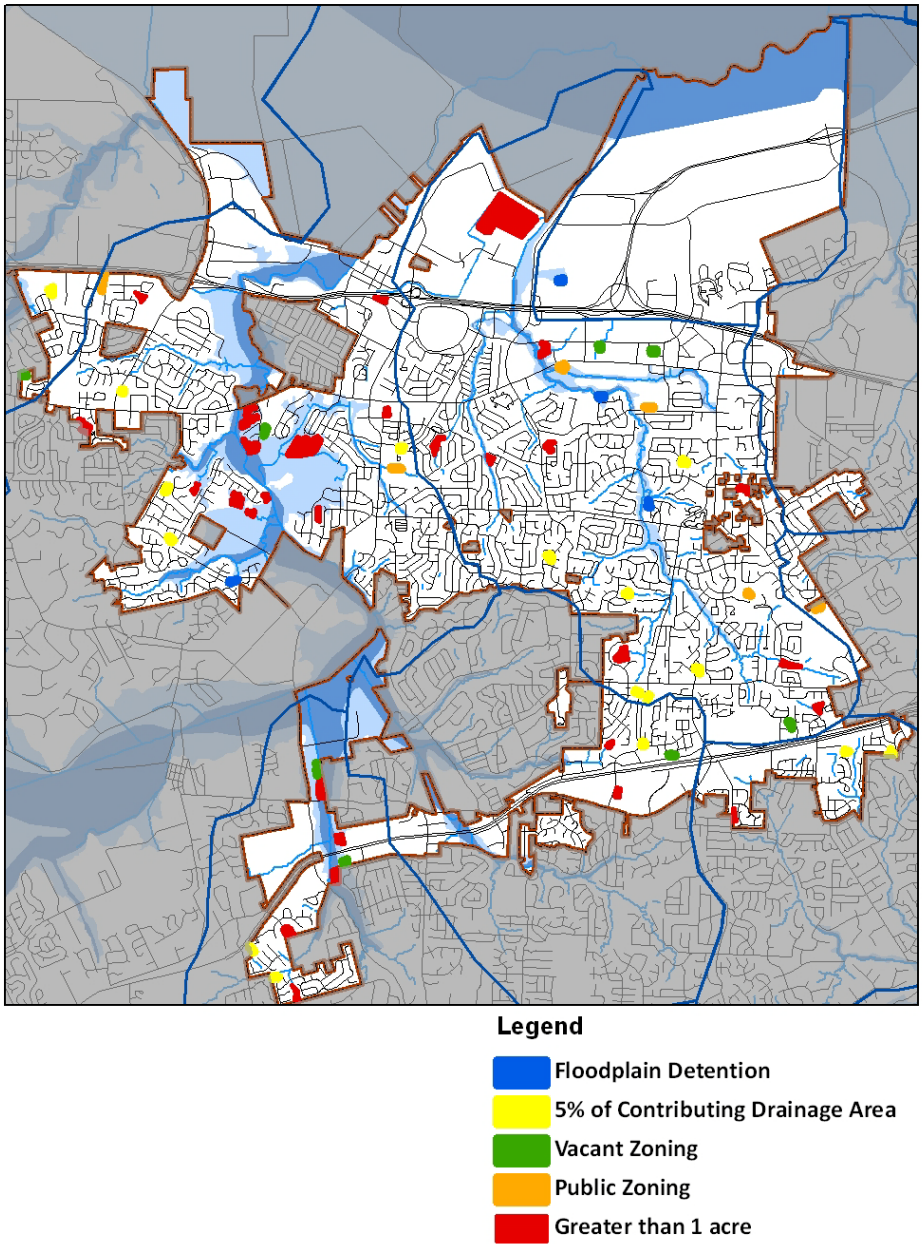

FIGURE 1. DETENTION BASINS INCLUDED IN STUDY

A larger version of Figure 1 is provided in Appendix J.

2.2 Hydrologic Model of Existing Conditions

The HEC-HMS model developed by the USACE St. Louis District was used as the basis for hydrologic modeling of the watersheds within and around St. Peters. The detention basins selected through the detention basin screening process were input into the USACE hydrology model to allow the opportunity to gage their effectiveness, model detention basin improvements, and evaluate downstream erosion effects.

2.2.1 HEC HMS Methodology

The methodology developed by the USACE St. Louis District was used for the hydrologic analysis of this watershed study. Modifications to the model were limited to the addition of detention basins, as described in detail in the following section. For a more complete description of the USACE's hydrologic methods, see the document, "Dardenne Creek Watershed Study," prepared by the USACE St. Louis District in 2007.

In order to input the selected detention basins, it was necessary to delineate sub-watersheds contributing drainage to these detention basins. It was also necessary to gather information regarding the storage and outfall characteristics of each detention basin. Storage and outflow characteristics were input as provided by the City. In some instances however, the information available from the City was not sufficient to approximate the hydrologic performance of the detention basin. In these cases, detention basin characteristics were gathered from the topography and field investigations in order to provide a more complete model. This secondary information is not optimal, and for this reason it is suggested that field topography and structure survey be gathered before design decisions are made using the model information.

Loss Method

The SCS Curve Number method was used to calculate losses, in conjunction with a percent imperviousness. That is to say that the curve numbers used were not composite curve numbers, but rather the raw curve numbers representing soil types. When drainage areas were spit in order to include a detention basin, the curve number for the larger basin was used for both basins.

Transform Method

The Clark Transform Method was used to create a unit hydrograph based on characteristics of the sub-watershed. This method uses two inputs: time of concentration and a storage coefficient. The time of concentration is calculated as the travel time for a drop of rain along the longest flowpath within the sub-watershed. The storage coefficient used by the USACE was calculated as 2/3 of the time of concentration. For detention basins, the TR-55 method was used to calculate new times of concentration. A minimum of 0.25 hours was used for model stability.

Baseflow Method

The USACE model used the Recession Baseflow method to calculate baseflow from each sub-watershed. The calibrated value of baseflow was based upon the size of the sub-watershed, and was found to be 0.6 cfs/square mile.

Modified Puls Routing Method

Routing in reaches was achieved using the Modified Puls Routing method, which requires a storage-discharge relationship for each reach. This method lags and attenuates the hydrographs produced by the sub-watersheds as flow travels down reaches.

METEOROLOGICAL DATA

This study used the hypothetical storms developed by the USACE to produce streamflows. These flows were calibrated as described in the USACE report.

2.2.2 Detention Basin Entry Methodology

Black & Veatch refined the USACE HMS model by adding detention basins selected in the screening process (Section 2.1.4). To incorporate the selected detention basins, it was necessary to break the USACE sub-watersheds into at least two portions. One portion is the part of the sub-watershed that does not drain to one of the selected detention basins. The other portion(s) drain to a detention basin, then out of the sub-basin. It should be noted that in many instances, several detention basins are located within a single USACE sub-watershed, requiring that the sub-basin be split into several different portions. An example is shown in Figures 2 and 3. Figure 2 shows the sub-watershed as depicted in the GIS and HMS model before being split. Figure 3 shows the same sub-watershed with an additional sub-area draining to a detention basin.

FIGURE 2. WATERSHED IN ORIGINAL USACE MODEL

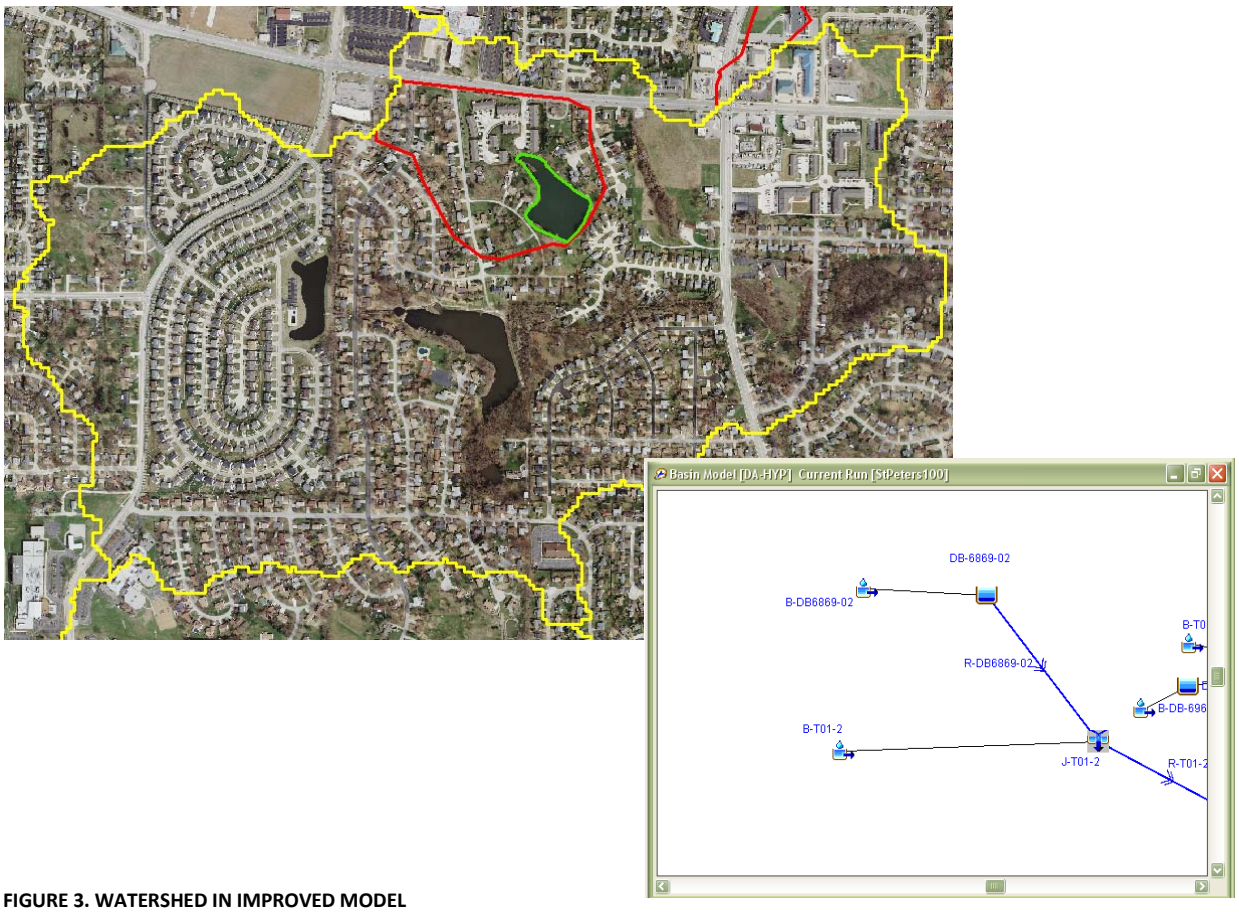

FIGURE 3. WATERSHED IN IMPROVED MODEL

As the detention basins and their contributing drainage areas are entered into the model, it was necessary to make the following changes:

Drainage Area Corrections

The drainage areas for the respective portions were updated to accurately reflect the size of the portion draining to the detention basin and the corresponding decrease in area of the un-detained portion.

Loss Characteristics

The loss characteristics (Initial Abstraction, Curve Number, and Percent Imperviousness) of the detained portion were assumed to be the same as the un-detained portion.

Clark Transform Parameters

The parameters associated with the Clark Transform (Time of Concentration and Storage Coefficient) were determined for each portion on a case-by-case basis. In general, the un-detained portion kept the same parameters, unless the longest flowpath within the sub-watershed was interrupted by the newly input detention basin. In these cases, a new time of concentration was determined.

For the detained portion(s), new times of concentration were determined using the TR-55 methodology. Storage coefficients for the detained portions were assumed to be 0.5, the smallest value that would allow the model to run. This prevents the model from over-estimating detention in the detained portion.

Baseflow Characteristics

No baseflow was assumed for the detained portion. Baseflow information was left unchanged for the un-detained portion.

The detention basins were input using stage/storage and storage/outflow tables. When information was available, these tables were developed using detention reports supplied by the City. When detention reports were not available, best efforts were made to develop these tables using available GIS information and field reports.

2.2.3 HEC-HMS Calibration Notes

The model supplied by the USACE was calibrated using two USGS gages along Dardenne Creek. The addition of detention basins into the USACE model resulted in a general reduction in flow across the sub-watersheds. To address this reduction in flow, the storage coefficients of the un-detained portions were reduced to match the 100-year (1% exceedence probability) hypothetical event.

By reducing the storage coefficients, the model has been transformed from using implied detention to explicit detention. In other words, instead of providing the sub-watershed with a large storage coefficient to imply detention within the sub-watershed, the improved model explicitly models the storage effects by including a detention basin.

This calibration effort produces peak flows that are generally within +/- 1% of the USACE HEC-HMS model for the 100-year event.

2.2.4 Suggestions for Improvement of the HEC-HMS Model

There are several opportunities for improvement of the HEC-HMS model for future preliminary engineering work. This model was initially developed by the USACE for the purpose of a generalized flood study of the Dardenne Creek watershed. This original purpose does not make the model an optimal one for preliminary engineering study. Some opportunities for improvement include:

- Re-delineation of sub-watersheds. The sub-watersheds were originally delineated using the USACE DEM and automated GeoHMS software. This produces coarsely-defined sub-watersheds that only roughly follow the contours developed through the more recent LiDAR mapping.
- Additionally, some of the sub-watersheds are delineated to points that are not optimal within the watershed.
- Topographic survey of detention basins and outfall structures. Many of the detention basins are input into the model using GIS-level contour information and field notes regarding the outlet structure. In some cases, these two information sources offer conflicting information about the detention basin characteristics.

- Times of Concentration. In general, times of concentration for the detained portions were developed primarily using overland flow. Local, in-depth analyses of times of concentration may be appropriate.
- Time step. The HMS model time step was set by the USACE as 15 minutes. This means that flows are only calculated at 15 minute intervals. In order to match peaks, this convention was retained. However, smaller time steps may provide more accurate flow estimates. Additionally, the smallest time of concentration allowed by the model is equal to a time step. This means that if a time of concentration for a small detained portion is below 15 minutes, the model automatically increases the time of concentration to 15 minutes.

2.3 Hydraulic Modeling of Existing Conditions

2.3.1 Existing Conditions Model

For this study, the Dardenne Creek hydraulic model developed by the USACE St. Louis District in 2007 was refined to represent the most current topographic information within the City limits. The extents of the hydraulic model are presented in Figure 4. A larger version of Figure 4 is presented in Appendix J. The USACE 2007 model was based on a 3D terrain model created with digital orthophotos. The 2007 study model also utilized additional data including bridge and culvert drawings, previous hydraulic models, rainfall and streamflow data, land use and soil maps, and other GIS data. The 2007 HEC-RAS model was developed using a GeoRAS extension of Arcview GIS and previous hydraulic models were used to describe the channel sections for streams that were not surveyed completely. Some of the cross sections in the 2007 HEC-RAS models were developed from field surveys conducted as part of the study. The cross sections that are based on survey information are noted within the model.

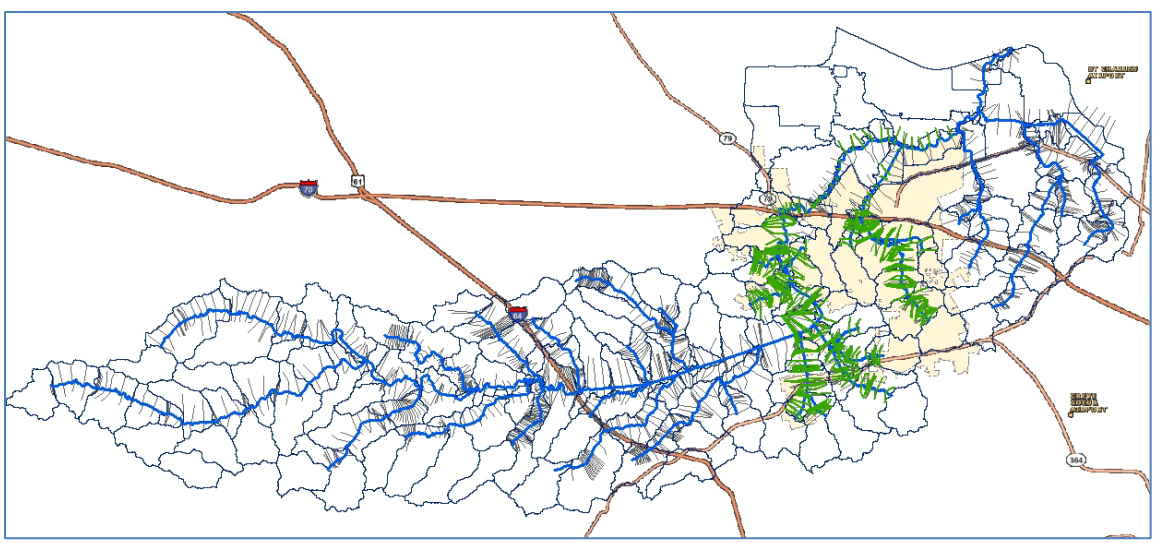

FIGURE 4. DARDENNE CREEK WATERSHED, HEC-RAS MODEL EXTENTS

The base condition modeling effort of the 2007 USACE hydraulic model analyzed eight different storm frequencies over the entire Dardenne Creek basin. The USACE model did not include a floodway

calculation. The forecasted condition modeling used the same flood frequencies with a forecasted condition that included changes to the hydrology based on anticipated developments in the watershed.

2.3.2 Evaluation of Existing Conditions Model

The existing model was evaluated to establish continuity of peak flows, determine extents, and identify modeling constraints within the city limits. The use of following model components was evaluated: bounded cross sections, ineffective flow areas, blocked obstructions, Manning's n values, lateral and storage structures, and bridge and culvert routines.

The review of the 2007 hydraulic model produced several concerns. First, the GIS data provided by the USACE was incomplete because USACE cross sections did not correspond to the 2007 USACE HEC-RAS model and were spaced infrequently, as shown in Figure 5. To develop the refined model, Black & Veatch used the 2007 HEC-RAS model to develop a GIS database that included the correct cross sections and stream centerlines. GeoRAS was utilized to export the complete set of cross sections and stream networks.

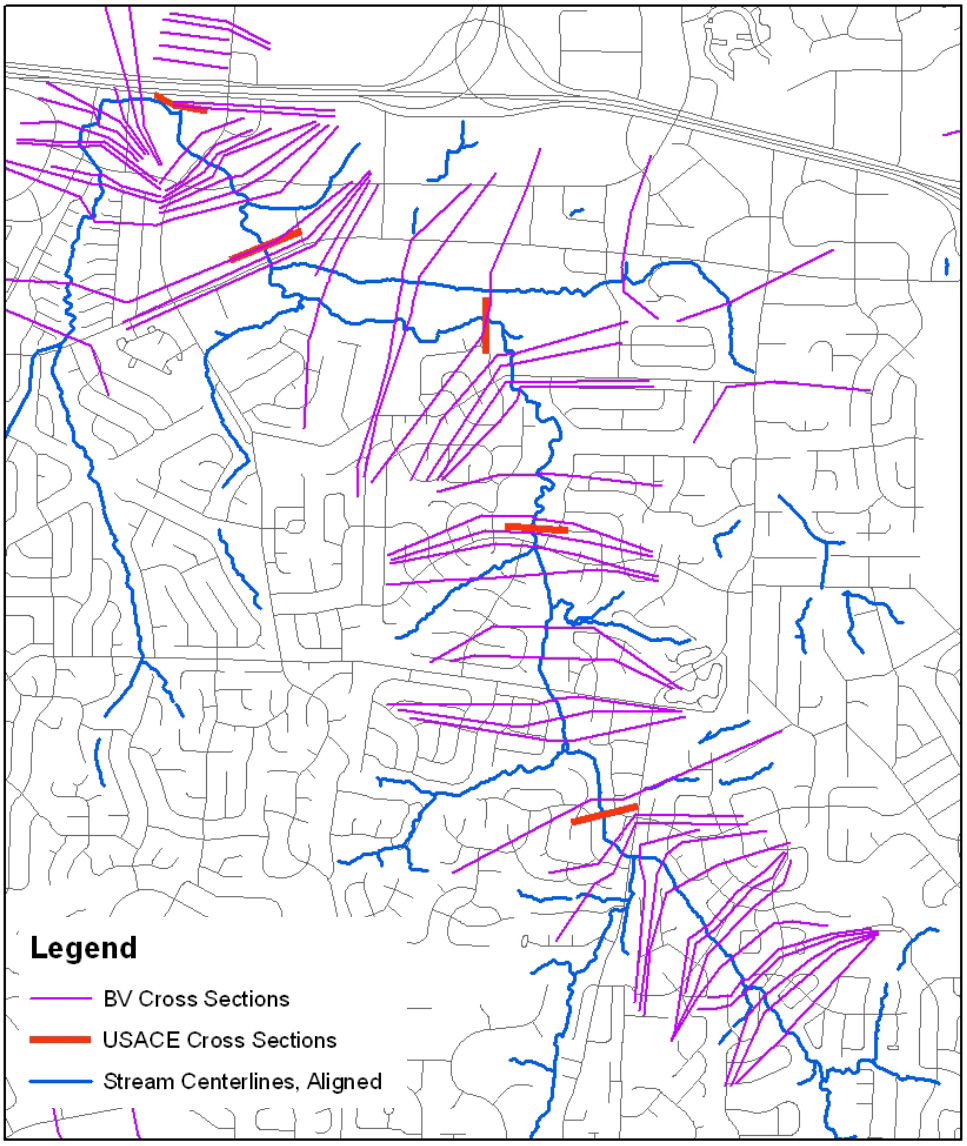

FIGURE 5. HEC-RAS CROSS SECTIONS, BV AND USACE MODELS

Additionally, the 2007 HEC-RAS model provided by the COE was read only and incomplete. Data missing from this model included the following: upstream boundary conditions for the flow file, storage area elevations, and ineffective flow area designation on several cross sections. The 2007 HEC-RAS model presented a steady state model that included storage areas and lateral structures. Storage areas and lateral structures are typically used in unsteady models to determine how much flow moves across a weir over a duration of time. As noted in the 2007 report, there is a limitation in the ability of HEC-RAS to perform an iterative optimization calculation for several levees at the same time in a steady flow analysis. Therefore, the 2007 study accounted for the flow leaving the channel by way of the levees with another method applied within the hydrologic analysis: the Modified Puls method of routing. The lateral structures and storage areas were left in the 2007 HEC-RAS models to be used in later modeling efforts when the steady flow calculations in the RAS program are improved.

The Manning's n values used in the model are presented in Table 3. Manning's n values depend on several factors including surface roughness, channel alignment, scour and deposition of materials, obstructions, and seasonal changes. For the 2007 USACE study, the Manning's n value was used to calibrate the model and therefore was not modified in the revised hydraulic model.

TABLE 3. HEC-RAS MANNING'S N VALUES

Creek	Manning's n value		Typical Overbank Land Cover
	Channel	Overbanks	
Dardenne	0.04-0.055	0.06-0.07	Agricultural and Levee, Forested, Golf Course, Residential Development
Spencer	0.045-0.05	0.05-0.065	Parkland, Wooded Areas, Residential Development, Commercial Development

Finally, the downstream boundary condition for the USACE hydraulic model is based on backwater conditions of the Mississippi River for each hypothetical storm event. In a steady flow hydraulic analysis, the downstream boundary condition must be defined and the standard step backwater calculations begin at this most downstream point of the model. In the 2007 HEC-RAS model, the mouth of Dardenne Creek at the Mississippi River is the most downstream point and the elevation of the Mississippi River therefore provides the starting point. The 2007 USACE study applied a conservative estimate for this starting water surface elevation. For each hypothetical event in the study, the corresponding flood elevation of the same frequency on the Mississippi River was used. These were based on Mississippi River Flood Frequency profiles from the USACE Water Control Office. The downstream boundary conditions of the existing conditions model are presented in Table 4.

TABLE 4. 2007 HEC-RAS MODEL DOWNSTREAM BOUNDARY CONDITIONS

Recurrence Interval	Discharge *(cfs)*	Water Surface Elevation *(ft)*
2-year	5,558	429
5-year	8,635	433.2
10-year	10,304	434.6
15-year	10,926	435.7
25-year	12,044	437.4
50-year	14,222	439.5
100-year	17,423	441.4
200-year	25,943	444.3

2.3.4 Revisions to Existing Model

For this study, the 2007 HEC-RAS hydraulic model was refined to represent the most current topographic information within the City limits, based on LiDAR data collected in 2009. The extents of the hydraulic model within the City limits are presented in Figure 6. The dark green lines represent cross sections that were updated based on current topography. The dark blue line represents the stream centerline for the model.

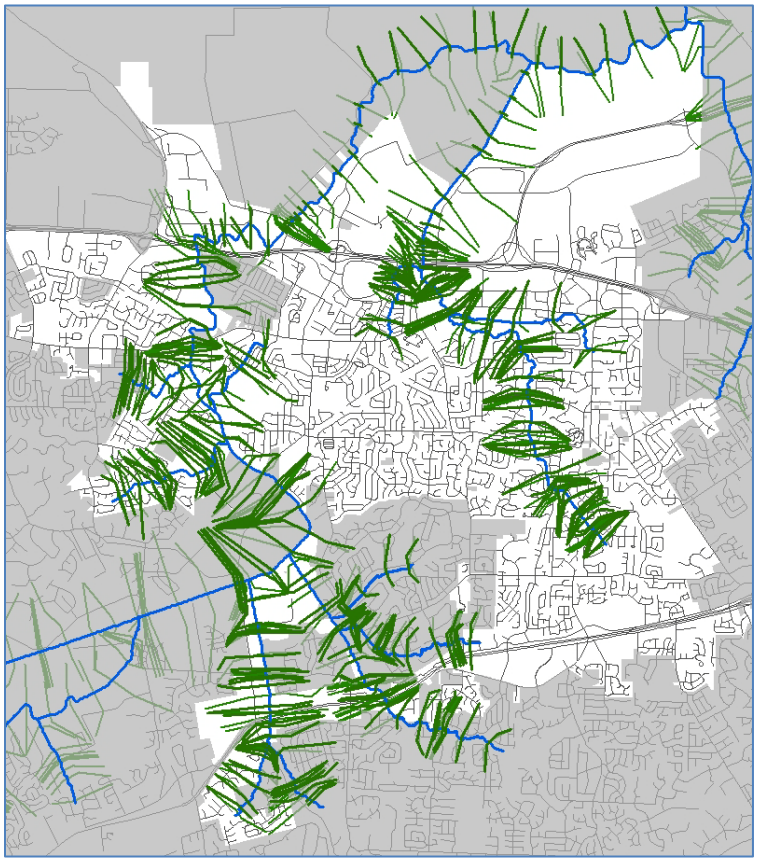

FIGURE 6. HEC-RAS MODEL EXTENTS, CITY OF ST. PETERS

The GeoRAS extension to ArcView GIS was utilized to update cross section geometry. A digital elevation model with 1 meter resolution, provided by the City, was used to define the new terrain. Cross sections and stream centerlines developed in the 2007 hydraulic model were overlaid on this new terrain. Several stream centerlines and the majority of the cross sections were updated to correctly represent the new terrain. Each cross section that was revised to correctly represent the new terrain is compared with the closest original cross section but the majority of revised cross sections do not overlap original sections at the exact location.

In most instances, surveyed cross sections from the 2007 study were evaluated but not included in the revised area of interest. These cross sections defined the topography in the channel however they were not consistently higher or lower than the GIS terrain. The average difference in elevation was 0.2 feet with a standard deviation of 2.5 feet. In most cases, using only the LiDAR digital elevation model to define the new terrain provided a more conservative approach because any conveyance area under the water surface elevation is not included in the hydraulic model. The downstream reaches of Dardenne Creek (between Mexico Road and the confluence), however, were represented with 2007 information because the LIDAR information did not accurately represent the channel conveyance.

The revised cross sections and stream centerline were imported into a revised HEC-RAS model watershed using the GeoRAS program. The cross sections highlighted in dark green in Figure 6 were replaced in the existing conditions hydraulic model. As part of the model revision, ineffective flow area and levee designations were added at appropriate locations within the City limits. Assumptions developed in the 2007 HEC-RAS model related to Manning's n values, contraction and expansion losses were retained. The revised model of existing conditions also retained the same hypothetical storm events provided in the 2007 HEC-RAS model.

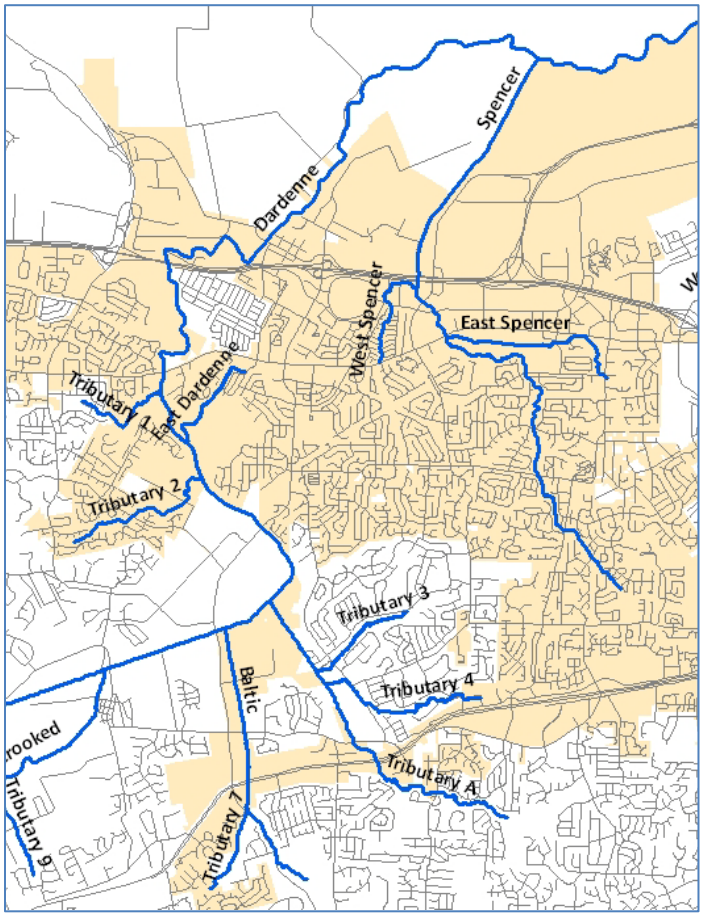

FIGURE 7. HEC-RAS MODEL STREAM CENTERLINES

Downstream boundary conditions were updated in the refined model of existing conditions. In order to represent more realistic boundary conditions with each hypothetical storm event, the 2-year water surface elevation was assumed for the Mississippi River.

The following table presents changes in water surface elevations at main points throughout the City as a result of these revisions to the HEC-RAS model. Figure 7 shows the names of the reaches referenced in Table 5. It is important to note that the revised cross sections are not at the exact location of the original cross section and therefore the comparison of water surface elevation is not exact. The inundation maps present the most accurate comparison of any changes in water surface elevation.

TABLE 5. WATER SURFACE ELEVATION COMPARISON, HEC-RAS EXISTING CONDITIONS MODELS

Stream	Reach	Comment	USACE Model		Refined Model		Difference
			River Station	*W.S. Elev (ft)*	*River Station*	*W.S. Elev (ft)*	*(ft)*
Baltic	2	Just downstream of Robertridge Court and Pittman Hill Road	10909	486.71	10939.1	483.34	-3.37
Baltic	1	Just upstream of Highway 94	7350	481.17	7589.226	475.32	-5.85
Baltic	1	Upstream of Dye Club Drive	2511	473.33	2747.017	473.38	0.05
Dardenne	9	Downstream of Mid Rivers Mall Drive	76865	473.25	76914.59	472.94	-0.31
Dardenne	7	Upstream of Mid Rivers Mall Drive	70035	469.85	70036.03	467.65	-2.2
Dardenne	4	Downstream of Confluence with Tributary 1	61578	463.48	61578	463.47	-0.01
Dardenne	4	Upstream of Mexico Road	60613	462.54	60613	462.52	-0.02
Dardenne	4	Adjacent to Dardenne Drive and Maple Street	54412	460.95	54412	460.93	-0.02
Dardenne	4	Adjacent to bend in Brown Road	49463	455.37	49463	455.35	-0.02
Dardenne	4	Upstream of Main Street	46762	454.9	46762	454.89	-0.01
Dardenne	3	Downstream of Confluence with Spencer Creek	27167	442.56	27167	439.82	-2.74
East Dardenne	1	Adjacent to Settlers Circle	383	465.09	3113.922	464.69	-0.4
East Spencer	1	Upstream of First Executive Drive	5447	468.61	5472.929	469.96	1.35
Spencer	3	Adjacent to Waterbury Court	29026	500.94	28950.23	498.99	-1.95
Spencer	3	Upstream of Willott Road	26168	488.46	26304.94	488.05	-0.41
Spencer	3	Upstream of Sutters Mill Road	23927	482.72	23873.98	479.74	-2.98
Spencer	3	Adjacent to Lantana Lane West	22447	475.73	22479.98	474.11	-1.62
Spencer	3	Adjacent to White Falls Court	20095	469.71	20040.57	467.7	-2.01
Spencer	2	Upstream of I-70	12737	451.98	12844.4	451.59	-0.39
Spencer	1	Adjacent to Spencer Road	8271	442.79	8277.905	440.8	-1.99

Stream	Reach	Comment	USACE Model		Refined Model		Difference
			River Station	*W.S. Elev (ft)*	*River Station*	*W.S. Elev (ft)*	*(ft)*
Spencer	1	Upstream of confluence with Dardenne Creek	672	442.57	630.0797	439.9	-2.67
Tributary 1	1	Adjacent to Carpenter Court cul-de-sac	4077	464.61	4047.269	464.62	0.01
Tributary 1	1	Upstream of confluence with Dardenne Creek	511	464.3	613.7575	464.5	0.2
Tributary 2	1	Adjacent to Country Squire Circle	5512.5	467.34	5528.229	468.33	0.99
Tributary 2	1	Upstream of confluence with Dardenne Creek	685	465.8	665.5778	465.24	-0.56
Tributary 3	1	Upstream end of Tributary 3	5066	504.05	4851.662	503.83	-0.22
Tributary 3	1	Downstream of St. Peters Howell Road	695	472.4	747.4638	470.75	-1.65
Tributary 4	1	Upstream of Woodstream Drive	7370	509.69	7383.865	509.8	0.11
Tributary 4	1	Adjacent to St. Peters Howell Road	1689.5	476.18	1885.409	476.53	0.35
Tributary A	3	Upstream of Dingledine Road	13469	495.17	13528.44	494.05	-1.12
Tributary A	3	Adjacent to Pine Bluff Drive Cul-de-sac	12090	487.71	12127.7	486.21	-1.5
Tributary A	3	Upstream of Central School Road	7864	475.21	7852.112	472.98	-2.23
Tributary A	2	Upstream of Fox Pointe Circle	4561	472.42	4449.625	471.28	-1.14
West Spencer	1	Adjacent to Madrid Court cul-de-sac	4536	456.94	4542.916	457.13	0.19
West Spencer	1	Upstream of Executive Centre Pkwy	2770	456.08	2788.375	456.02	-0.06
West Spencer	1	Upstream of Costco Way	1150	452.14	1020.302	451.96	-0.18
Tributary 7	1	Adjacent to San Marco Way	1404	483.25	1540.299	483.55	0.3

On average, there is less than one foot of change in 100-year water surface elevation between the revised existing conditions model and the 2007 USACE model. Typically, the water surface elevation decreased. There was a maximum increase within the City limits of 1.35 feet associated with the 100-year water surface elevation, located at Station 5447 on East Spencer Creek, upstream of First Executive Drive. There was a maximum decrease of 5.85 feet associated with the 100-year water surface, located

at Station7350 on Baltic Creek, just upstream of the Highway 94 crossing. This is due to a culvert project that was installed in 2010-2011.

2.3.5 Calibration Assessment

The calibration of the 2007 HEC-RAS model compared the computed flood elevations to observed data along Dardenne Creek. There are two recording stream gauges along Dardenne Creek; these gauges are located at O'Fallon and St. Peters and provide data from late 1999 to the present. The 2007 study used the observed high water marks from the storm of January 2005 and the hydrograph from the November 2003 storm event. The 2007 study modified Manning's n values and some bridge and culvert data to calibrate the model to the observed high water marks. The 2007 study acknowledged that the limited number of rainfall gauges and various antecedent moisture conditions could not accurately portray complex storm patterns for the HEC-HMS analysis.

For this study, the same flows were used for each hypothetical storm event because the revised HMS model was calibrated to the USACE HMS model. These flows produced water surface elevations that were within 1 foot of those in the 2007 hydraulic model, on average. Calibrating the revised model to match the water surface elevations with the new topography would require changing the Manning's n values beyond the reasonable range.

Rain events in April and June (2011) produced localized flooding and additional high water marks that were recorded by the City. The anecdotal records from this event were used to evaluate specific areas of the existing conditions model. Two particular locations were highlighted by the City: flooding at Jungermann Road and Rachel's Trail. The flooding recorded at Jungermann Road is not accurately represented in the existing model because the hydrologic drainage basin in this area overlaps the roadway and the storm event discharge is associated with a flow change location downstream of the bridge, rather than at a point upstream. Figure 8 presents the HEC-HMS subarea B-SP7 that overlaps Jungermann Road. The upstream basin produces a flow of 2279 cfs during the 100-year event and B-SP7 produces a flow of 4378 cfs during the same event. The higher flow is applied to a cross section downstream of Jungermann Road in the HEC-RAS model and therefore the evaluation of Jungermann Road in the existing conditions model was not based on an appropriate break point.

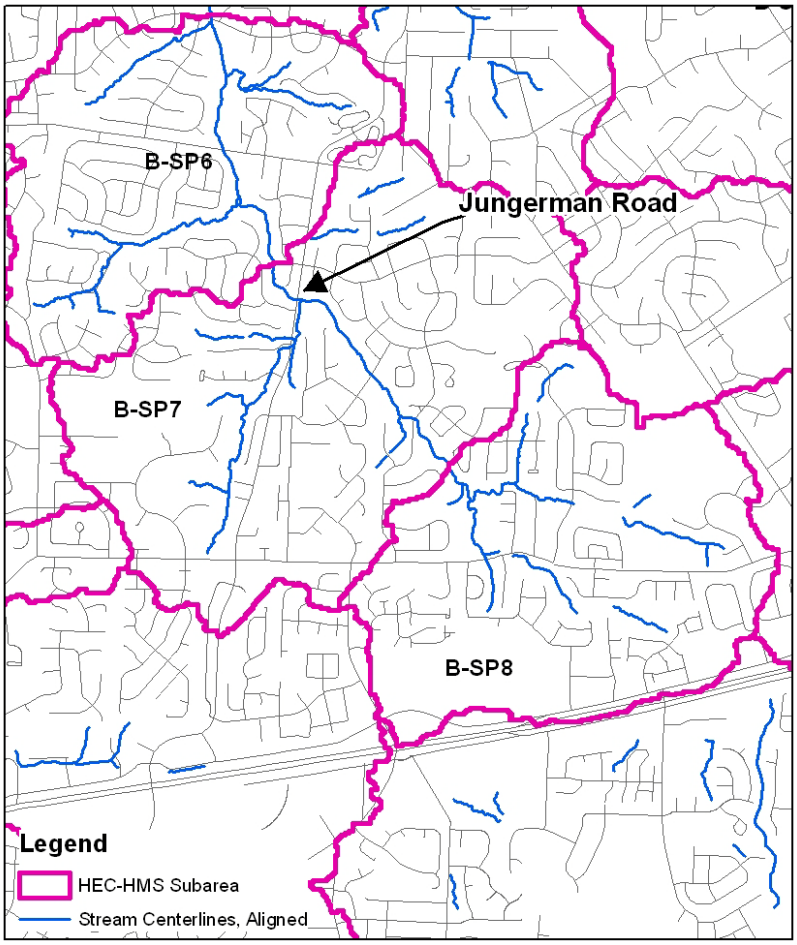

FIGURE 8. HEC HMS SUBAREAS, JUNGERMANN ROAD VICINITY

The second location that the City highlighted was at Rachel's Trail Drive. This area is influenced by the capacity of the stormwater pipe conveyance system and is not within the scope of this master plan.

2.3.6 Flooding Identified in Existing HEC-RAS Model

The area of inundation was established for the revised existing conditions model during the 100-year event. The 2007 USACE hydraulic model area of inundation was also developed for the purposes of comparison. The revised existing conditions hydraulic model results in a water surface elevation that will be used for comparison to conceptual recommendations. This inundation map is presented in Figure 9.

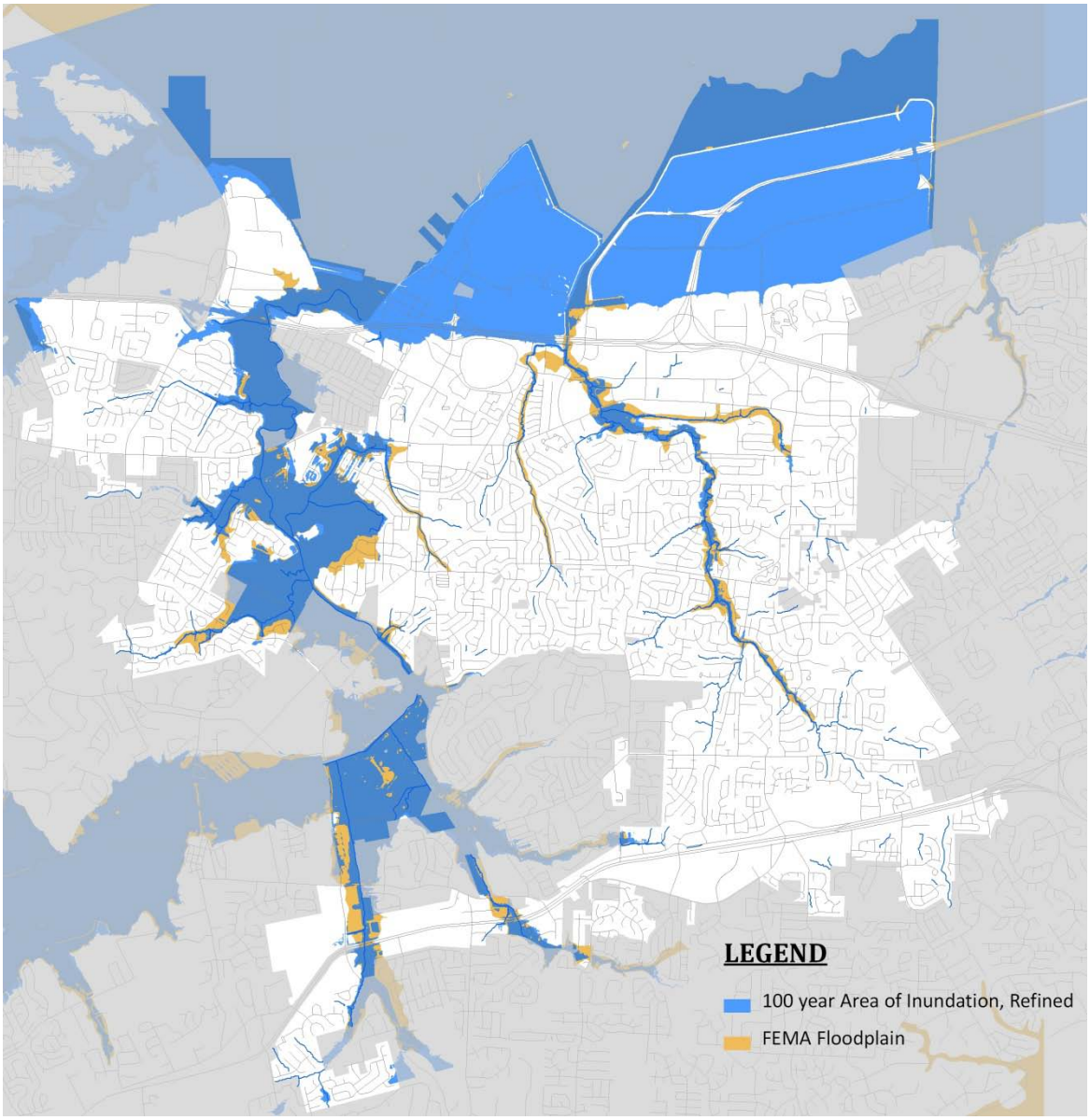

FIGURE 9. COMPARISON OF HEC-RAS INUNDATION AREAS (EFFECTIVE FEMA FLOODPLAIN, 1996)

Along Spencer Creek, the revised area of inundation lies mostly within the FEMA floodplain and is significantly narrower in many reaches. Several residential structures are within this floodplain and area of inundation. Low spots were documented along Jungermann Road and Boone Hills Drive, presented in Figures 10 through 14.

FIGURE 10. JUNGERMANN ROAD OVERTOPPING ALONG SPENCER CREEK

FIGURE 11. BOONE HILLS DRIVE OVERTOPPING ALONG SPENCER CREEK

FIGURE 12. JUNGERMAN ROAD OVERTOPPING ALONG EAST SPENCER CREEK

The area of inundation along Dardenne Creek is significantly wider. Within the City limits, development within this floodplain is mostly recreational. The revised model of existing conditions shows overtopping of Mexico Road and the residential area south of Mexico Road is within the area of inundation.

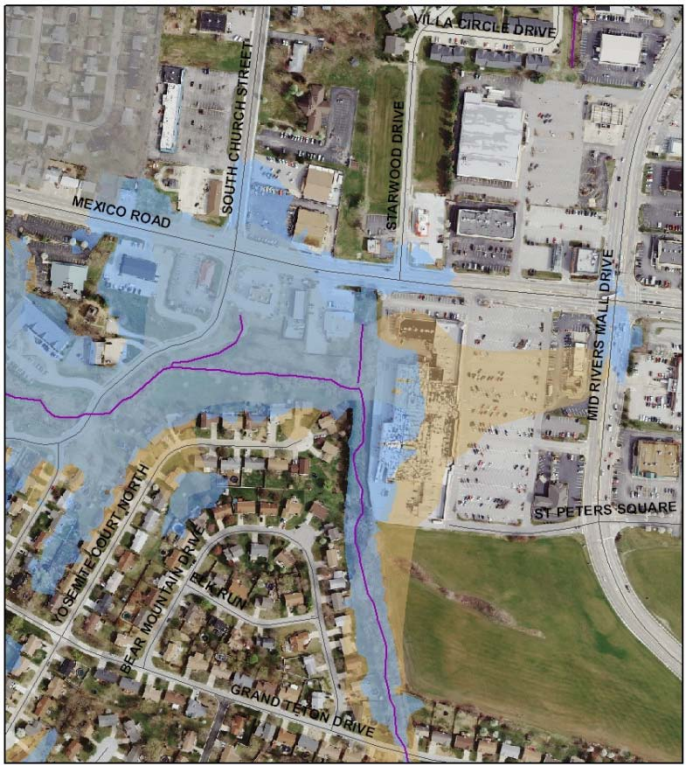

FIGURE 13. MEXICO ROAD OVERTOPPING NEAR MID RIVERS MALL DRIVE, TRIBUTARY TO DARDENNE

The 2007 hydraulic model and the revised existing condition model produce inundation areas that are different from the FEMA flood insurance study maps. As the 2007 USACE study highlighted, FEMA's St. Charles County Flood Insurance Study is still the official document for determining the floodplain and floodway areas. The revised model of existing conditions does not estimate the floodway width or define encroachments on the floodplain. This FEMA flood insurance study is the official document for decisions related to floodplain development.

2.3.7 Bridge and Culvert Crossings

Bridge and culvert crossings within the City limits were evaluated within the revised hydraulic model. Bridge design guidance is provided in Section 4.040 of the St. Louis MSD 'Rules, Regulations, and Design Requirements'. This document states that the "lowest point of bridge superstructure shall have 2-ft of freeboard from the 15-yr water surface elevation and 1-ft freeboard from the 100-yr water surface elevation." Table 6 presents the freeboard at each bridge and culvert crossing within the City limits, as calculated in the hydraulic model. Thirteen crossings were considered for project locations because of overtopping or insufficient freeboard during the 15- or 100-yr event. For individual reasons (i.e. the bridge crossing is located at a private property), eight of these crossings were not included as projects in the CIP. A summary of the data for each crossing is provided in the table below.

TABLE 6. BRIDGE AND CULVERT CROSSINGS

Reach	Project ID	Crossing	Low Chord or Low Point on Deck	Refined Existing Conditions Model Water Surface Elevation		Freeboard *(ft)*		PROJECT
				15 year	*100 year*	*15 year*	*100 year*	
Baltic		Dye Club Road Bridge	463.5	470.35	473.38	-6.85	-9.88	Considered but excluded because private ownership
Baltic		Central School Road	477.5	471	471	6.5	6.5	NO
Baltic		Hwy 94						NO
Baltic		Private Road Bridge	489	484.92	485.6	4.08	3.4	NO
Dardenne		N&W RR Bridge	450	449.61	450	0.39	0	Considered but excluded because private ownership
Dardenne	IP-7071-4-2	Hwy C Bridge	454.07	450.23	453.73	3.84	0.34	YES
Dardenne	IP-6970-4-3	I-70	456.9	454.77	460.34	2.13	-3.44	YES
Dardenne	IP-6970-4-3	Mexico	460.5	459.06	462.22	1.44	-1.72	YES
Dardenne - 7		Mid Rivers Mall	473.5	464.43	467.34	9.07	6.16	NO
Dardenne - 9		Mid Rivers Mall	474	470.14	473.5	3.86	0.5	Considered but excluded because of relatively new construction
East Spencer		St. Peters Walkway Bridge	457.5	456.49	458.76	1.01	-1.26	Considered but excluded because of pedestrian usage
East Spencer		St. Peters Rec Plex	464.61	457.77	459.56	6.84	5.05	NO
Spencer		N&W RR Bridge	442	442.89	444.04	-0.89	-2.04	Considered but excluded because private ownership

Spencer		I-70	449.22	444.77	447.25	4.45	1.97	NO
Spencer		Spencer Road Bridge	461.95	450.72	452.67	11.23	9.28	NO
Spencer		Executive Center	459.5	452.17	454.4	7.33	5.1	NO
Spencer		Mexico	457.45	454.11	457.46	3.34	-0.01	Considered
Spencer		Boone Hills	469.94	466.17	467.31	3.77	2.63	NO
Spencer		Sutters Mill Road	480	474.5	476.51	5.5	3.49	NO
Spencer		Willott	489	484.14	485.9	4.86	3.1	NO
Spencer	IP-7368-1-3	Jungermann	500.12	498.24	502.35	1.88	-2.23	YES
Spencer		Burning Leaf	514.93	509.69	511.69	5.24	3.24	NO
Spencer		Millwood Drive	526.6	517.4	517.4	9.2	9.2	NO
Tributary 2		Woodlands Parkway	454	462.18	465.26	-8.18	-11.26	Considered but excluded because of park location
Tributary 2		Ohmes Road Culvert	471.63	462.85	465.31	8.78	6.32	NO
Tributary 7	IP-7066-4-1	Pittman Hill Road	480.4	480.81	482.01	-0.41	-1.61	YES
Tributary A		Central School Road	480.5	470.58	472.68	9.92	7.82	NO
Tributary A		Private Road Culvert	478.51	478.51	478.51	0	0	Considered but excluded because private ownership
Tributary A		Hwy 94						NO
Tributary A		Dingledine Road Culvert	500.15	489.92	491.23	10.23	8.92	NO
West Spencer		Costco Culvert	454.7	449.48	451.61	5.22	3.09	NO
West Spencer		Suemandy Culvert	454	450	450	4	4	NO
West Spencer		Suemandy Culvert	458.21	451	451	7.21	7.21	NO

2.3.8 Closed System Hydraulics

The closed system hydraulic analysis was performed with the stormwater analysis software, XP-SWMM 2011. The model was developed initially using the MIKE SWMM files provided by the city of St. Peters, but much of the pipe system shown in the model has been updated and no subcatchment delineations were provided with it making it impossible to delineate new catchments for the new system. Therefore, a new version of the Old Town stormwater system was developed, which utilized infiltration parameters from the MIKE SWMM model and the same general stormwater system configuration with updates to the pipe system and the pump station. The pipe system updates were taken from the Old Town St. Peters Storm Sewer Improvements Plan drawings and the pump station information was provided by the City. These drawings represent improvements constructed in 2000. When new data was required to complete the model, the GIS datasets (specifically, the 2-ft contours, the impervious area, and the storm sewer coverages) were utilized, and if further information was needed, reasonable assumptions made for the model.

Stormwater Conveyance

The Old Town stormwater drainage basin is a relatively flat basin that is bordered on the west by the Dardenne Creek levee, and road and highway embankments in the other directions; however there is a significant amount of drainage area to the south of I-70 that is conveyed underneath the highway and makes its way into the Old Town drainage system. The stormwater is conveyed in a pipe system to the Old Town Pump Station and discharges to Dardenne Creek. When the water surface elevation in Dardenne Creek is higher, flow is then pumped into Dardenne Creek. In a system such as this, not only do the pipes and storm water inlets need to have adequate capacity for the level of service design event, but so does the pump station. It should be noted that the modeling performed in this basin only evaluated the pipe system shown and did not evaluate stormwater inlets. If stormwater inlets are inadequate, then localized flooding issues can occur; however, their impact is generally limited as excess flows will usually flow down roads to other entry locations into the stormwater system. If the pipe system or pump station capacity is inadequate, more severe flooding can occur. The following Figures 14 and 15 show an overview of the Old Town basin and a schematic of the stormwater system analyzed.

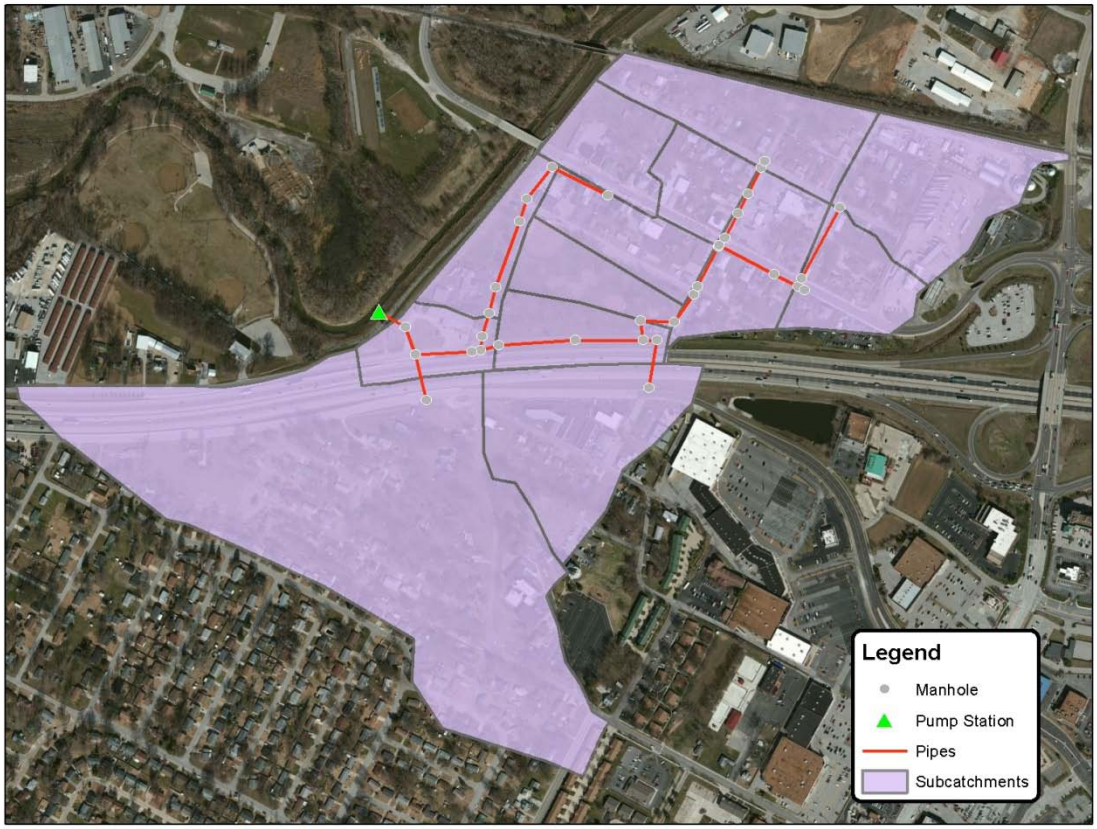

FIGURE 14. OLD TOWN DRAINAGE SYSTEM AND SUBAREAS

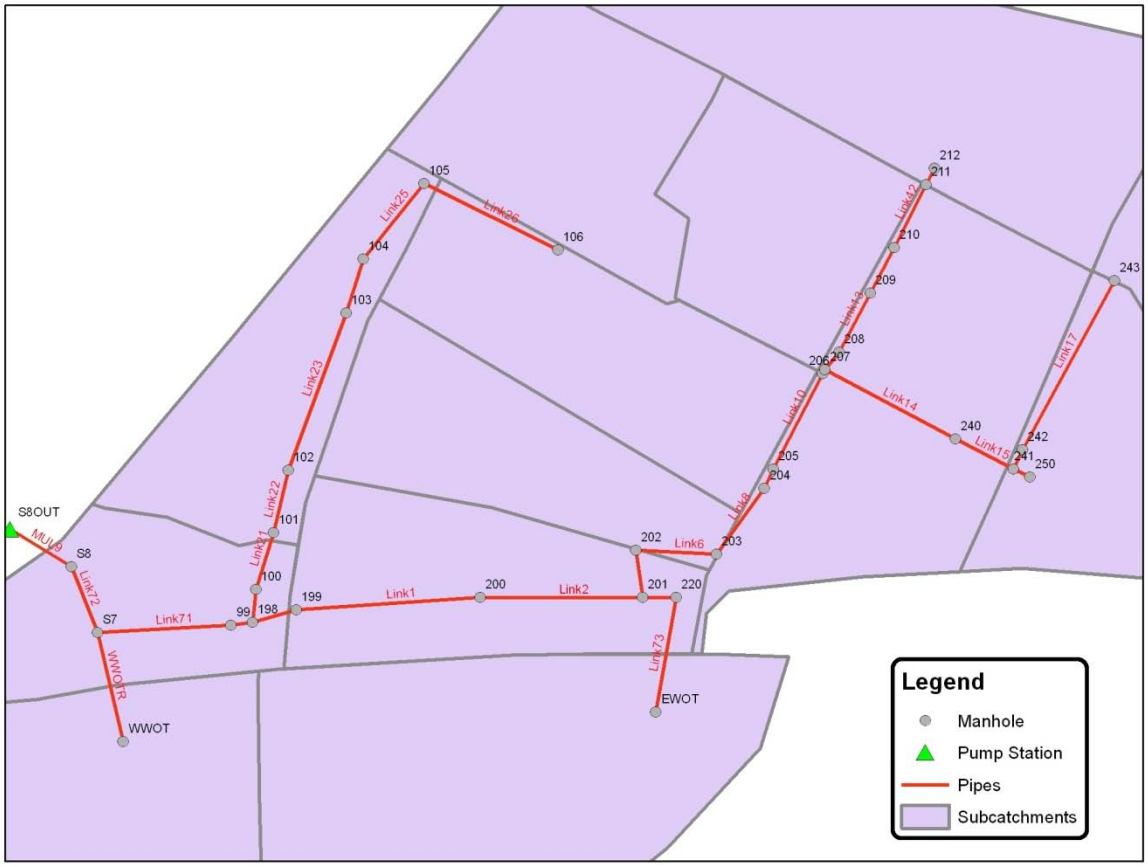

FIGURE 15. OLD TOWN PIPE SYSTEM, XP-SWMM MODEL

Pipe System Capacity Analysis

Within the XP-SWMM model, the pipes are represented by links and the manholes are represented by nodes. The pipe system should be able to convey the design storm without flooding the manholes. To analyze just the pipe system conveyance performance, the pump station was excluded. This prevents the pump station from creating a backwater condition that controls the stormwater conveyance system, which does not allow for the individual pipe capacities to be evaluated. The pump station was evaluated separately. Using this model, the 2-yr, 24-hr and 15-yr, 24-hr SCS Type II storms were simulated in the model. The 15-yr, 20 minute design storm is contained within the 15-yr, 24-hr storm and provides the same peak flow although the 24-hr storm analysis provides conservative volume results. The model indicated that the system experiences minor surcharging (i.e., when the water surface elevation exceeds the top of the pipe crown) and flooding (i.e., when water would flow out of the tops of manholes). For the pipe system analysis, the cause of surcharging and flooding is inadequate pipe capacity. Table 7 summarizes the flows and pipe capacities for the Old Town system (pipes highlighted in grey surcharged or contributed to flooding during the 15-yr, 24-hr storm):

TABLE 7. EXISTING OLD TOWN PIPE SYSTEM CAPACITY ANALYSIS

Pipe	2-yr flow (cfs)	15-yr flow (cfs)	Full Pipe Capacity (cfs)	Current Pipe Size (ft)*	15-year Flow to Full Pipe Capacity Ratio
Link43	7	22	19.6	3	113%
Link42	7	22	31.2	3.5	70%
Link41	7	21	32.24	3.5	67%
Link13	11	31	44.1	4	71%
Link12	11	31	44.5	4	70%
Link17	18	34	13	2.5	260%
Link16	18	34	32.29	3.5	106%
Link20	9	21	12.7	2.5	163%
Link15	26	53	42.6		124%
Link14	26	52	49		106%
Link11**	47	102	N/A	4.5	
Link10	47	102	60.44	4.5	169%
Link9	56	119	68	4.5	175%
Link8	56	119	64	4.5	187%
Link6	57	121	82	5	147%
Link5	57	125	88	5	141%
Link73	22	49	67	3	73%
Link3	22	49	157	3	31%
Link2	77	167	135	6	123%
Link1	77	167	135	6	123%
Link44	81	176	171.5	6.5	103%
Link26	7	15	22	2.5	69%
Link25	7	15	22	2.5	68%
Link24	7	15	22	2.5	68%
Link23	7	15	36	3	41%
Link22	7	15	36	3	43%
Link21	11	25	33	3	76%
Link46	11	25	38	3	66%
Link45	92	201	158.5	6.5	127%
Link71	93	204	268	Twin 6-ft RCPs	76%
WWOTR	60	133	382	6x3 RCB	35%
Link72	149	325	268	Twin 6-ft RCPs	121%
* - All dimensions are pipe diameters except where indicated					
** - This is a short segment of flat pipe, which therefore does not have a calculated full flow by Manning's equation					

Based on Table 7, it can be seen that there are a number of pipe segments that have more flow than their full pipe capacity can convey by gravity; however, exceeding the pipe capacity by a limited amount does not necessarily cause flooding.

Pump Station Conveyance Analysis

In a basin such as Old Town, the most severe flooding can occur if the pump station cannot convey the stormwater produced by the design storm. The information provided by the City indicates that the current pump station can convey approximately 44 cfs (20,000 gpm), which corresponds with two pumps that can convey 22 cfs at the Old Town Pump Station. From the Old Town stormwater model, approximately 160 cfs is conveyed to the pump station during the 2-yr, 24-hr storm and approximately 340 cfs during the 15-yr, 24-hr storm. It should be noted that for the analysis, it was assumed that the water surface within Dardenne Creek corresponded with the same return interval as the design storm. For the 2-yr storm, this does not impact the pump station as the pump station discharges at a greater elevation; however for the 15-yr storm, there is some submergence of the pump discharge, which reduces the capacity of the pumps (however, this is submergence is offset by lower lift requirements on the suction side of the pump during the peak flow of the design storm).

When the pump station is introduced into the model, it acts as a bottleneck in the system for both the 2-yr and 15-yr storms effectively causing most of the low lying areas in Old Town to flood. From a modeling standpoint, it is difficult to determine the exact extent and depth of the surface flooding within a relatively flat basin such as Old Town. Essentially, as the flow escapes from the tops of the low lying manholes, it spills onto the surface and begins to spread out, forming a pool of inundation at the surface. In a one dimensional (1D) model such as SWMM, the challenge is to determine the surface storage volume where excess flow temporarily pools until the pump station can drain the system.

XP-SWMM 2011 has an optional two dimensional (2D) model that can be coupled with the one dimensional model to address this situation. For reference, the one dimensional model used by XP-SWMM is based on the EPA SWMM's model while the two dimensional model is called TUFLOW. For the 2D model, a surface model created from terrain data, such as contour or point data, is added to the top of the one dimensional model manholes. As the water surface elevation increases, eventually the water will spill out the tops of the manholes onto the surface, and the model determines the inundation extents on the surface.

Using the coupled 1D-2D XP-SWMM model, the extents of the flooding were determined for Old Town. Figure 16 shows the extent of the flooding within Old Town:

FIGURE 16. OLD TOWN AREA OF INUNDATION, 15-YR, 24-HR EXISTING CONDITIONS

In the existing conditions, approximately 29 acres of Old Town is inundated to a water surface elevation of 440.3 ft (as the water surface elevation is a two dimensional surface, there is variation across the surface, but for this analysis the concern is the elevation near the lower elevations of the basin). Most of the basin located within the 29 acre inundation area has a surface elevation of approximately 440, which means that much of the area would experience relatively shallow inundation (0.3 ft) and much of the deeper inundation would be in localized low areas.

East Old Town Conveyance System

The City noted that the existing swale draining east Old Town to the regional detention basin along Iffrig Road may be causing additional flooding of the Old Town area. This conveyance system was analyzed in a new XP SWMM model and it was determined that the swale and pipe capacity was undersized.

2.4 Assessment of Water Quality, Existing Conditions

Water quality in the City is influenced by several factors including the rate of development, consistent application of wet and dry detention basins, and stream stability.

There are 248 stormwater detention ponds within the City limits. Using the screening process described in Section 2.1.4, the most significant 74 detention ponds were selected. Each basin's potential to remove total suspended solids (TSS) was evaluated. The resulting data was evaluated in the context of each of the eight regional drainage areas in the City. Four of the drainage areas are in the Spencer Creek watershed and four are in the Dardenne Creek watershed.

Stream stability strongly influences water quality as well. There are 47 miles of stream corridor within the City limits. Stream bank and bed degradation was observed during field reconnaissance in April and May 2011. Nearly 12 miles of stream were documented as experiencing systemic instability, represented by mass wasting of banks, streambed degradation, or lack of riparian corridor.

2.4.1 Stormwater Runoff Model

The P8 urban stormwater runoff model was used to predict pollutant removal efficiencies for each of the 74 existing stormwater detention basins. The model simulated the generation and transport of stormwater runoff pollutants in the watersheds contributing to the detention basins. The simulations were driven by continuous hourly rainfall based on ten years of data recorded at the Lambert International Airport provided by the National Climatic Data Center. During the period of 2001 to 2009, the annual precipitation averaged 40.6 inches and ranged from a minimum of 29.8 inches in 2006 to a maximum of 58.0 inches in 2008.

The P8 model was pre-calibrated by its developer using the Nationwide Urban Runoff Program (NURP) database. Since data characterizing the quantity and quality of stormwater entering the basins were not available, a site-specific calibration of the P8 model was not possible. Although P8 is capable of simulating total suspended solids (TSS), total phosphorus, total Kjeldahl nitrogen, copper, lead, zinc, and total hydrocarbons, TSS was assumed to be a surrogate pollutant because in stormwater runoff the other pollutants are almost all associated with the suspended solids.

P8 input parameters for each detention basin included the area and impervious fraction of the detention basin's contributing drainage area, the detention basin area and volume, and whether the basin was normally dry (extended detention) or contained a permanent pool (wet detention). The watershed areas and imperviousness were based on GIS data provided by the City and the HEC-HMS hydrology model. Detention basin volumes were determined by one of two methods. The method used depended upon the information that was available from the City. If a complete hydrologic design report was available from the city for a given basin, the stage/storage/outflow information from the hydrologic report was incorporated into the model. If a hydrologic design report was not available from the City, the stage/storage curve was developed using the LiDAR contours. In this case, outflow information was developed based on field observations of the outlet works of the basin.

2.4.2 Removal Efficiencies of the Existing Stormwater Detention Basins

Of the 74 stormwater detention basins evaluated, 19 were located in the Spencer Creek watershed, upstream of Interstate-70. This watershed is named "Spencer (US)" and is shown in Figure 17.

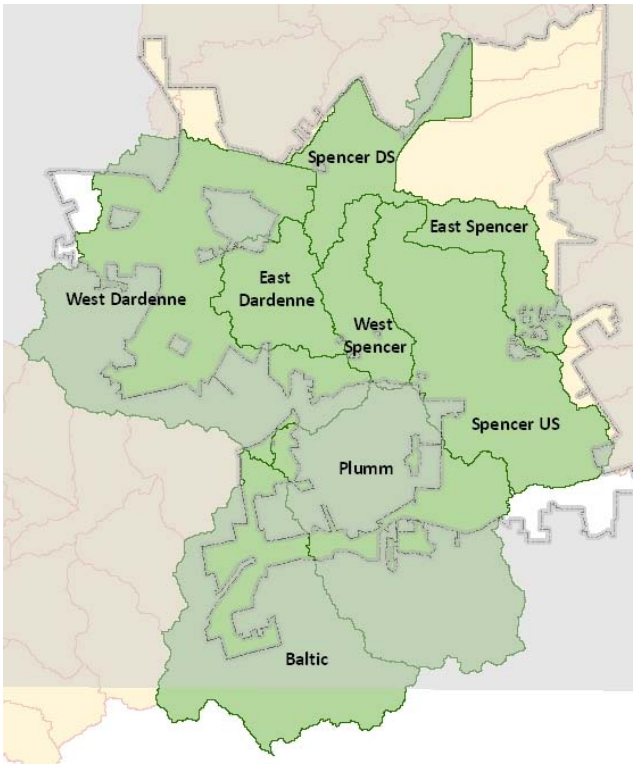

FIGURE 17. REGIONAL DRAINAGE AREAS IN ST. PETERS

The P8 model predicted that the wet basin removal efficiencies are significantly higher than the dry detention, averaging over 90 percent; the average removal of the dry detention is approximately 25 percent. Wet detention basins were more efficient in TSS removal because a full pond provides a longer hydraulic retention time, allowing more time for the TSS to settle out of the water column.

The average TSS removal of all the detention basins in the upstream Spencer drainage area was 65 percent. Approximately 87,000 lbs of TSS is removed by the 19 detention basins each year, under existing conditions. The P8 model predicted that approximately 689,000 lbs per year of TSS would be contained in stormwater runoff from the entire drainage area (Spencer US); therefore, the overall TSS removal in the 19 detention basins, with respect to the loading from the entire regional drainage area, is 12.6 percent. The total TSS loading for eight regional drainage areas is approximately 3.5 million lb/yr, including 1.1 million lb/yr contributed by the four Dardenne Creek drainage areas, Baltic, Plumm, West Dardenne, and East Dardenne.

Of the 74 basins evaluated, 67 are located in regional drainage areas. The following table summarizes the removal overall efficiencies for the 67 detention basins in the eight regional drainage areas.

TABLE 8. REMOVAL EFFICIENCIES FOR EXISTING STORMWATER DETENTION BASINS

Regional Drainage Area	Number of Detention Basins	Total Regional Drainage Area	Detention Basin				Overall Removal
			In	Out	Removed	Removed	
		TSS-lb/yr	*TSS-lb/yr*	*TSS-lb/yr*	*TSS-lb/yr*	*%*	*%*
Baltic	8	298,595	44,878	14,184	30,694	68	10.3
West Dardenne	17	915,435	86,769	26,087	60,682	70	6.6
East Dardenne	9	252,434	73,836	41,436	32,400	44	12.8
East Spencer	5	271,909	61,911	21,479	40,432	65	14.9
Plumm	4	590,753	39,304	30,969	8,335	21	1.4
Spencer Downstream	0	226,128	0	0	0	0	0.0
West Spencer	5	232,583	7,215	1,868	5,347	74	2.3
Spencer Upstream	19	688,865	134,126	47,549	86,577	65	12.6
Totals	67	3,476,702	448,039	183,572	264,467	59	7.6

**Only 67 of the 74 detention basins evaluated in this study are within the regional drainage areas.*

Tables of removal efficiencies and other characteristics for each of the existing basins are included in Appendix C.

2.5 Geomorphic Assessment of Existing Conditions

In April and May 2011, Black & Veatch and PBA staff joined the City to evaluate stream stability and habitat within the City limits. As a team, Black & Veatch and PBA conducted a geomorphology and stream health field survey of the 47 miles of streams in the City of St. Peters. The team applied standards developed by the Kansas City Metropolitan Chapter of American Public Works Association and various state and federal natural resource agencies. The field assessment data was used to characterize each reach of the stream network with a rating that reflects the level of stability. Field data was collected in a Trimble unit that allowed for each data point to be georeferenced. Photographs were linked to these data points as well. The resulting GIS data, with scores and photographs along each reach, is provided in the geodatabase deliverable.

2.5.1 Channel Condition Scoring Matrix

Black & Veatch applied a scoring matrix based on the APWA 5600 Channel Condition Scoring Matrix (CCSM) to determine the stability of stream reaches throughout the City. The CCSM indices used include the channel geometry, bank slope stability, streambed and bank material composition, and erosion issues. Existing channel bed and bank geometry was established using the LiDAR data. Stream bank and bed material were evaluated based on soil texture, sediment composition, consolidation, armoring

materials, and vegetative protection. Bank cutting, mass wasting, and excessive sediment deposition in localized areas were recorded. At select locations, existing riparian vegetation was documented. For each channel reach, the score of the indicator and the weighted rating for the stream were compiled in a GIS database.

The matrix provides a comparative score of reaches throughout the City and was used to identify areas of significant instability and to prioritize projects. The resulting score was divided into three groups that range between 10 and 25. A rating of 13 or less indicates a stream of moderate stability and may require only standard levels of protection during construction. A rating between 13 and 19 indicates limited instability and special measures may be needed to address specific issues that were rated poorly (i.e. debris jam). A rating that is greater than 19 indicates that the stream may be experiencing significant system-wide instability.

2.5.2 Stream Asset Inventory

The Stream Asset Inventory (SAI) methodology provided rapid and scientifically defensible indicators of water quality, stream stability, and habitat conditions at a given location that is selected to be representative of a larger stream reach. Assessment criteria of the SAI include erosion indicators; bed and bank composition; aquatic habitat features; tree canopy and understory coverage and composition; and indirect water quality indicators (visible or detectable degradation and presence/absence of aquatic life). The assessment criteria are assigned individual weighted scores to create a composite score of stream quality at each sampled location and a relative ranking of stream quality throughout the entire watershed.

Sample locations were selected in the field based on preliminary map assessments, the assessor's professional judgment, as well as accessibility and ability to observe as many assessment criteria as practical. Data was collected at least 100 feet from major human-made influences if possible (such as roads, bridges and culverts), as past experience indicates that these localized features negatively bias the assessment results. Some stream reaches smaller than 4th order streams were not sampled due to insufficient reach length necessary for sampling. Finally, the SAI procedure is not intended for ephemeral streams or man-made channels. In other locations not all components were observable but data were collected if the locations otherwise met assessment criteria. PBA staff completed a survey checklist at 120 locations, or approximately every 3/8th stream mile, which exceeded the recommended data density of at least one survey point every stream mile.

The SAI procedure has four major categories with each category composed of five scoring components. Each component has a maximum potential score of 10 which creates a possible total SAI score of 200. By dividing the total SAI score by 20 (or by the number of measured components), the assessment provides a qualitative numerical score ranging from 0.0 to 10.0. A score of 10.0 would be considered optimal stream conditions while a score of 0.0 would indicate poor stream conditions. Some components were not applicable or observable in certain situations, and if so, the evaluation team did not assess that component. The final sampling point Total Score was calculated by dividing the sum of all four SAI categories by the number of components observed at that sampling location. For example,

where bed composition could not be observed due to high flows or turbid conditions, no points are assigned for the bed composition component and the total site score was divided by 19 rather than 20.

As explained in the methodology description, stream reaches were assigned a composite score from 10.0 to 0.0 and were classified Type 1 (highest quality - 10.0) through Type 5 (lowest quality - 0.0) based on the statistical distribution of all study data. For example, a Type 3 stream score falls one standard deviation above or below the mean score, a Type 2 stream scores more than one standard deviation above the mean, etc. Stream segments were classified into five types:

Type 1 - Highest Quality: Generally described as the highest quality naturally occurring stream with little negative impact. Erosion and sedimentation is low, water quality indicators are positive and the surrounding riparian zone is healthy, mature woodland or other high-quality environment. Approximately 1% of evaluated streams were placed in this category.

Type 2 - High Quality: This type of stream may have some down or side-cutting; however, bank and bed composition (bedrock) assist in keeping the impact low. Water quality is generally good and the riparian zone is largely intact, although vegetation may be negatively altered from that of a typical native plant association. Approximately 12% of evaluated streams were placed in this category.

Type 3 – Restorable: Deterioration of the channel and riparian corridor are more noticeable. While some remnant plant associations may be present, overall vegetative canopy cover is comprised of immature tree species. Water quality may be fair to marginal. The potential for restoration exists although erosion and sedimentation can be greater than desirable. Approximately 63% of evaluated streams were placed in this category.

Type 4 - Low Quality: Impacts are greater on this stream type with significant indicators of bank erosion and channel instability. The adjoining riparian corridor may be intact but vegetation is not representative of a native plant association. Water quality is typically poor. Approximately 23% of evaluated streams were placed in this category.

Type 5 - Lowest Quality: The channel in this type is the most changed. The riparian corridor is impaired to the point of providing little protection or benefit, and erosion and sedimentation indicators are significantly high. Water quality is poor with degradation and absence of macro-invertebrates, fish, mollusks, and amphibians. Approximately 1% of evaluated streams were placed in this category.

The surveyed stream segments were classified relative to the sample population of surveyed streams, rather than applying an absolute score. The relative ranking is used for several reasons: (1) Scoring streams on an absolute scale may imply that the break points between classes are based on some quantitative linkage between the score and stream function, which is not the case; (2) Streams should be classified in comparison to general, regional conditions so that streams are assigned scores reasonable for their physiographic and development settings; and (3) Relative distribution allows the assessor to identify the truly high-quality and low-quality streams within the study area.

2.5.3 Geomorphic Stream Survey - Impairment of Streams

A majority of the stream network is incised, as documented in the 2002 watershed study and as shown in Figure 18. Channel incision is a response to changes in the hydrology of the contributing drainage area as well as channel bed materials and downstream conditions. The stream network within the City limits has varying levels of stability and incision. The downstream ends of the main tributaries to Spencer Creek have already experienced changes in structure and are now very incised. In these streams, further incision is likely to occur as a small low flow channel cutting into the hard clay bed was documented in the field. Reaches higher up in the watershed have very steep profiles and debris jams of roots or other material were often found to provide temporary vertical grade control.

FIGURE 18. INCISED REACH, EAST SPENCER TRIBUTARY NEAR CONFLUENCE WITH SPENCER CREEK

The field assessment highlighted soils in the lower part of both watersheds as a predominant concern. These soils significantly contribute to the sediment load in Spencer and Dardenne Creeks. The sandy clay soil composition found in several of these reaches has little resistance to shear stresses of flow. This type of soil is stable at a very mild slope and therefore will continue to degrade more rapidly in these areas. Many of the channels are incised and the banks with sandy or loamy clay material that will continue to degrade at a more rapid pace than hard clay or rocky banks. Rock check structures are highlighted in Section 3 as a recommendation for many of these incising streams in order to control the rate of degradation and soil loss.

PBA analyzed the SAI data to identify obvious trends. The quality riparian vegetation and water quality indicators appear to have the greatest correlation with overall stream quality (R= 0.60). When PBA narrowed the analysis to sample locations where all 20 components were scored (i.e. where all of the conditions were present and could be observed), the terrestrial habitat and water quality components correlated more strongly with the overall stream condition (R = 0.67). The presence or absence of quality riparian vegetation appears to have the greatest influence on overall stream quality, and water quality appears to be better in streams that are in better overall condition. Field observations and SAI scores support the theory that urbanization and degradation or loss of stream buffers negatively impact the overall stability and quality of streams throughout St. Peters. This is consistent with regional and national research indicating that an intact riparian buffer can reduce the impacts of impervious cover and intense runoff conditions in urban watersheds (Center for Watershed Protection 2003; PBA 2005a and 2005b; PBA 2007; Schulte, Scott A., Patricia A. Elbert Noll and Jeffrey Henson, 2008). These findings further support the importance of riparian buffers for protecting stream quality.

A stream classification summary is provided in Appendix D.

Immediate Concerns

Several localized issues were identified during the field assessment as immediate need or concern areas. These locations are typically pipe crossings or outfalls that are immediately threatening existing infrastructure. A database of immediate concern locations was presented to the City.

3.0 Management Measures

The existing conditions of stormwater management in the City demonstrated concerns related to flooding, stormwater detention, stream stability, water quality, and existing policies. This section introduces management measures that were considered to address these stormwater concerns. The following chapter describes how these measures were evaluated. Typical renderings of management recommendations are provided in Appendix E.

Three sections of management measures were considered: stream management, stormwater best management practices throughout the watershed, and policy and ordinance updates. Each of these measures will play a part in achieving load reductions.

3.1 Stream Management Measures

Hydraulic modeling and field reconnaissance documented a variety of flooding and stability conditions, as presented in Section 2.

3.1.1 Flooding Management Measures

Flooding locations were identified during the assessment of existing conditions with the use of hydraulic models and the geodatabase of anecdotal concerns provided by the City. Most flooding locations were addressed with a conceptual recommendation as part of the CIP. Some bridge crossings with flooding or insufficient freeboard were excluded, such as crossings on private roads.

3.1.2 Stream Stability Measures

The stream network in St. Peters includes approximately 12 miles of stream that are highly degraded and unstable, rated with a CCSM score of 19 or more. System stability is based not only on the stability of the banks, but also the vertical stability of the channel bed.

Bank Stability

The areas of mass wasting, bank slumping, and bank scour are representative of the channel's response to the rapid development that occurred over the last 30 years in the City. To prevent further migration of the bank in critical areas, several stability measures were considered.

Over the past decade, the City has applied a variety of bioengineered techniques incorporating synthetic and vegetative materials – with varying levels of success. Based on field observation, one of the most critical elements of successful stabilization projects is to ensure that sufficient flow and sediment conveyance are provided.

FIGURE 19. TRM SYNTHETIC STABILIZATION METHOD FAILURE

Three levels of bank stabilization were evaluated with consideration to materials that have demonstrated successful application in past City projects. The basic level includes riprap toe protection and establishment of vegetation to protect banks. The second level builds upon the basic cross section to include the integration of a floodbench so that higher flows can spread out over a wider conveyance area. The final level applies to reaches where there is not sufficient area to establish a stable bank slope

or flood bench and a wall structure is integrated into the stream bank. This last case only applies to a few locations throughout the City.

Vertical Stability of the Channel Bed

The recommended management measure for vertical grade control is the establishment of rock check dams using riprap. A durable riprap should be used; in the field, evidence of riprap deterioration was prevalent. Grade control structures are installed perpendicular to flow. They are designed with respect to the channel bed and bedload materials and their height and spacing depends upon achieving an equilibrium slope, preventing channel incision from migrating upstream. The riprap size is designed to resist velocities and shear stresses during a typical design event (i.e. 100-yr). Scour analyses is also necessary to understand localized channel hydraulics and critical shear stress values. The depth of excavation and placement of riprap below the channel bed is dependent on these calculations of scour. Hydraulic analysis is also used to evaluate the location of shear stress on the banks. To establish vegetation, riprap voids may be filled with a granular soil mixture, sometimes using on-site excavated materials. Root cuttings such as willows have shown successful establishment in restoration projects in the City as well, as shown in Figure 20. Table 9 provides a summary of project locations where rock check stabilization is recommended. The geodatabase provides more detailed information at each of these project sites.

TABLE 9. CIP PROJECTS WITH ROCK CHECKS

Project_ID	Location Description	Estimated Rock Check width	Channel Bed Improvement (No. of Rock Checks)
IP-6965-2-1	Wolfrum Way	Category 2: Rock Check (10-15 feet across)	3
IP-6965-2-2	Deer Meadow Ct.	Category 2: Rock Check (10-15 feet across)	7
IP-6968-1-1	NW Ohmes Farm	Category 2: Rock Check (10-15 feet across)	6
IP-6969-2-2	Hope Drive	Category 2: Rock Check (10-15 feet across)	2
IP-6970-1-1	Calwood	Category 2: Rock Check (10-15 feet across)	5
IP-6970-4-2	Dardenne Creek between Mexico Road and I-70	Category 4: Rock Check (>20 feet across)	4
IP-7065-1-3	Highlands	Category 2: Rock Check (10-15 feet across)	4

Project_ID	Location Description	Estimated Rock Check width	Channel Bed Improvement (No. of Rock Checks)
IP-7067-1-1	Baltic Creek, downstream of Central School Road	Category 3: Rock Check (15-20 feet across)	4
IP-7067-2-3	Old Hickory GC	Category 3: Rock Check (15-20 feet across)	2
IP-7068-2-1	Bruns Place	Category 2: Rock Check (10-15 feet across)	7
IP-7068-4-1	Dardenne Creek at Fairfax	Category 4: Rock Check (>20 feet across)	1
IP-7069-1-1	Crystal Lake Detention	Category 2: Rock Check (10-15 feet across)	2
IP-7069-1-2	Crystal Lake	Category 1: Rock Check (5-10 feet across)	2
IP-7069-2-3	Parkdale	Category 1: Rock Check (5-10 feet across)	4
IP-7069-3-2	Woodlands GRG	Category 2: Rock Check (10-15 feet across)	4
IP-7071-3-1	Dardenne Creek, downstream of I-70	Category 4: Rock Check (>20 feet across)	10
IP-7166-1-1	Old Farmhouse Rd.	Category 2: Rock Check (10-15 feet across)	2
IP-7168-1-1	Athens Drive	Category 1: Rock Check (5-10 feet across)	3
IP-7169-1-2	Dubray Middle School	Category 1: Rock Check (5-10 feet across)	10
IP-7169-2-1	Carrington Place	Category 3: Rock Check (15-20 feet across)	22
IP-7170-2-1	Eldorado	Category 2: Rock Check (10-15 feet across)	5
IP-7267-4-3	Park Ridge Estates	Category 2: Rock Check (10-15 feet across)	1

Project_ID	Location Description	Estimated Rock Check width	Channel Bed Improvement (No. of Rock Checks)
IP-7268-2-2	Wood Path	Category 2: Rock Check (10-15 feet across)	8
IP-7269-1-1	Colby Drive	Category 1: Rock Check (5-10 feet across)	21
IP-7269-4-1	Oak Creek Park	Category 2: Rock Check (10-15 feet across)	12
IP-7270-1-1	St. Peters Executive Centre	Category 2: Rock Check (10-15 feet across)	10
IP-7270-1-2	Executive Centre	Category 4: Rock Check (>20 feet across)	1
IP-7270-3-2	Carrington Estates	Category 2: Rock Check (10-15 feet across)	4
IP-7270-4-1	City Centre	Category 3: Rock Check (15-20 feet across)	4
IP-7368-1-4	Tanglewood	Category 2: Rock Check (10-15 feet across)	2
IP-7368-2-1	Burning Leaf Drive	Category 1: Rock Check (5-10 feet across)	1
IP-7368-2-3	Kelly Leaf	Category 2: Rock Check (10-15 feet across)	6
IP-7368-3-1	Jungermann Office Center	Category 2: Rock Check (10-15 feet across)	6
IP-7368-3-2	Hidden Pine Estates	Category 2: Rock Check (10-15 feet across)	8
IP-7368-4-3	Spencer Creek in Millwood Subdivision	Category 2: Rock Check (10-15 feet across)	7
IP-7369-1-1	Spencer Creek between Sutters Mill Road and Boone Hills Drive	Category 2: Rock Check (10-15 feet across)	4

Project_ID	Location Description	Estimated Rock Check width	Channel Bed Improvement (No. of Rock Checks)
IP-7369-2-1	Boone Hills Drive and Jungermann Road	Category 2: Rock Check (10-15 feet across)	3
IP-7369-3-2	Spencer Trail	Category 4: Rock Check (>20 feet across)	2
IP-7369-4-1	West Drive	Category 2: Rock Check (10-15 feet across)	6
IP-7370-4-1	East Spencer Creek downstream of Jungermann Road	Category 1: Rock Check (5-10 feet across)	5
IP-7467-4-1	Magnolia Manor	Category 2: Rock Check (10-15 feet across)	5
IP-7467-4-3	Pointe of Heritage	Category 3: Rock Check (15-20 feet across)	6

FIGURE 20. STABLE STREAM EXAMPLE WITH WILLOW CUTTING ESTABLISHMENT

3.1.3 Riparian Renovation

The primary goal of riparian preservation is to protect the function and aesthetics of streams through proper management of the riparian corridor vegetation. Preservation begins with floodplain management or stream setback policies (or both) that help limit encroachment of structures in the riparian corridor, and regulations and education to discourage inappropriate uses and activities that degrade natural function. However, preservation does not end there; there are three key components to consider with riparian preservation: management, vegetation type, and corridor width.

First, preservation does not mean taking a hands-off approach. Vegetative management must occur if the riparian corridor is to function properly. Second, management practices generally focus on reducing or eliminating non-native vegetation in an effort to support the growth of more desirable native varieties. Even some aggressive native plant species like bittersweet and trumpet vine can be undesirable within a riparian corridor. This may include replanting native species where non-native or aggressive vegetation is removed to prevent it from re-establishing, and planting natives where existing vegetation is thin (particularly along streambanks where the vegetation helps prevent erosion). Third, the width of the riparian zone is equally important in preserving function. A narrow riparian corridor is far less effective in buffering streams from adjoining land uses than a wider corridor. Buffer widths vary with stream type (ephemeral, intermittent, or perennial) but, the general rule of thumb is to place wider

buffers on smaller headwater streams that are often most neglected and impacted, and where benefits are often the greatest due to the preponderance of smaller streams in a watershed. Wider buffers are also desirable where high-quality riparian corridors exist to protect the integrity of these resources; and where stream restoration is a high priority.

A list of tree and shrub species, provided in Appendix F, includes varieties that are present in healthy, regional stream corridors. These species should be used for preservation and restoration efforts.

"Reference sites" are healthy stream corridors that provide helpful guides for riparian preservation and restoration. There are a few sites in the Spencer Creek and Dardenne Creek watersheds that would be considered reference sites for proper vegetation and buffer width. Most of these reference sites are located within parkland already owned and managed by the City. These sites include:

- Spencer Creek from Jungermann Road to Millwood Drive.
- Spencer Creek upstream of extended wet detention facility east of McClay Valley Boulevard.
- Tributary to Dardenne Creek in the southern portion of Old Hickory Golf Course located within the wooded area surrounding the golf course maintenance facility (western tributary)

Figures 21 and 22 illustrate a healthy riparian corridor, and a stream corridor with undesirable vegetation for comparison.

Desirable riparian corridor: this photo is of an area east of Jungermann Road and illustrates a good mix of canopy and understory species with very little undesirables; trees are a mix of ages, and the canopy is open enough to allow for good understory vegetation.

FIGURE 21. DESIRABLE RIPARIAN CORRIDOR CONDITIONS

Undesirable riparian corridor: this photo shows undesirable vegetation (honeysuckle) dominating one side of corridor; and a thin, almost non-existent corridor on the other side.

FIGURE 22. UNDESIRABLE RIPARIAN CORRIDOR CONDITIONS

3.2 Watershed Improvements

The management of stormwater as it flows across the landscape is the second critical section of management measures. This section presents the approach to developing detention basin retrofit options and general stormwater best management practices. Regional detention opportunities evaluated in this study are presented in Section 4.

3.2.1 Detention Basins

The detention basin improvements considered for this study considered the hydrologic as well as water quality needs of the watershed. The hydrologic needs were addressed largely by recommending improvements to the dam structure or outlet works, or by recommending increased storage behind the structure. Water quality improvements were addressed by recommending an alternative storage regime (dry detention to wet detention), improved plantings, or by the construction of a sediment forebay.

Three retrofit options were evaluated in this study. These options were based the most current best management practices for improving stormwater runoff quality.

Dry Detention Retrofit

A dry detention retrofit established a temporary pool that is sized to detain the water quality volume. Typically, this retrofit includes the establishment of native vegetation and amendment to the soil in the bottom of the basin to enhance infiltration.

Dry to Wet Detention Retrofit

Upon retrofit of a dry detention to a wet pond, a permanent pool that is equal to the volume of the water quality volume was established for each basin. Typically, a wetland bench of planting is recommended around the basin perimeter.

Wet Detention Retrofit

A wet basin retrofit includes establishment of a wetland buffer planting around the perimeter of the basin.

Additionally, sediment forebays were suggested for some detention basins based upon watershed characteristics or anecdotal information that sediment is an issue for that basin. In general, it is suggested that the City reference the Mid-America Regional Council (MARC) guidance on the construction and features of a sediment forebay. Key features of a sediment forebay include:

- Maintenance Schedule: Sediment forebays must be maintained on a frequent basis. Sediment must be removed periodically and hauled off-site. If a forebay is not periodically maintained, it will become choked with sediment, cease to function, and actually become a detriment to downstream facilities.
- Proper sizing: according to the maintenance schedule determined for the forebay, the facility must be properly sized to accept sediment between maintenance events.
- Access: Proper access must be afforded to maintenance crews to allow for upkeep of the facility. Typically, a backhoe can be expected to handle the excavation of accumulated sediment.
- Plantings: Plantings within the sediment forebay are welcomed. During maintenance events, the plantings can simply be scraped off the top of the sediment, placed to the side, and then replaced in the sediment forebay after excavation is completed.

3.2.2 Stormwater Best Management Practices

A water quality best management practices (BMP) locator assessment was used to identify optimal locations for BMPs within the City. The objective was to assess and classify the relative value of land for capturing, storing, and infiltrating stormwater runoff; and to identify vacant land that could be used to construct new water quality BMPs if desired to meet the City's stormwater management objectives.

3.3 Ordinance and Policy Improvements

This section briefly summarizes existing policies, ordinances, and design criteria and identifies regulations and other factors that may influence future policies. Following the review of the existing situation recommendations for updating and adopting polices to meet regulations and achieve the goals for storm water management in St. Peters are presented.

3.3.1 Existing Ordinances and Regulations

The City's current existing ordinances and regulations provide for management of storm water. State and federal regulations also impact storm water management.

Current City Storm water Management Regulations

Existing ordinances and standards that address storm water and flood risk management in the City are briefly summarized below.

- Chapter 410 – Floodplain Management. Criteria in this section are consistent with FEMA requirements and limit development in the 100-year floodplain.
- Chapter 530 – Grading Regulations. These regulations require a permit when development will grade more than 300 cubic yards, or 20,000 square feet and are in general consistent with MDNR requirements.
- Chapter 550 – Storm Sewer and Drainage Facility Guidelines. A storm water management plan is required for every development under these guidelines sized for the 15-year, 20 minute design storm. These guidelines also require detention if runoff is increased by 15 % or more as a result of development. A 25 foot setback from streams is also required.
- Standards for Erosion and Sediment Management Practices, January 2001. This document provides design criteria for many erosion and sediment control best management practices. Many additional effective best management practices have been developed since 1991.

- Chapter 550 references "Standard Construction Specifications for Sewers and Drainage Facilities of the Metropolitan St. Louis Sewer District," 1992 and Chapter 4 "The Metropolitan St. Louis Sewer District – Rules and Regulations and Engineering Design Requirements for Sanitary Sewage and Storm Water Drainage Facilities", February 1997. Both of these documents have been updated.

State Stormwater Management Regulations

Construction Site Runoff Controls

The MDNR storm water regulations for Construction or Land Disturbance Activity MO-R101000 and MO-R100000 for existing sites and M0-R10A000 for new sites address construction site runoff controls. These permits are valid through February 7, 2012. The City's existing ordinance and design standards appear to address these requirements. However, newer design criteria are available that expand the number of BMPs that can be used to limit sediment from being discharged. These new design criteria are available in the following document: Protecting Water Quality: A field guide to erosion, sediment and storm water best management practices for development sites in Missouri and Kansas, January 2011.

Stormwater Discharge Permit

The MDNR storm water regulations for discharge from the Municipal Separate Storm Sewer System MO-R040044 (MS4 Permit) require that the storm water management program address the six minimum control measures. This permit is in effect until June 12, 2013.

Control measure 5 requires implementation of post construction storm water management in new development and redevelopment. From the permit, "prevent or minimize water quality impacts by reasonably mimicking pre-construction runoff conditions on all affected new development projects and by effectively utilizing water quality strategies and technologies on all affected redevelopment projects to the maximum extent practicable". The City's existing design criteria require predevelopment peak flows to be managed for 2-year and 25-year design storms. The largest impact on water quality resulting from runoff is from more frequent events.

Federal Stormwater Management Regulations

EPA expects to publish new storm water regulations by the end of 2012 as result of the Natural Resources Commission study and report on the effectiveness of the current storm water regulations. Key findings and recommendations from that report are listed below.

Key findings of NRC report

- Current approach is unlikely to produce an accurate picture of the problem and unlikely to adequately control storm water's contribution to water body impairment
- Current requirements leave a great deal of discretion to dischargers to set their own standards and ensure compliance, which results in inconsistency across the nation
- Poor accountability and uncertain effectiveness in current approach to stormwater management

Key Recommendations

- A straightforward way to regulate storm water contributions to water body impairment would be to use flow or a surrogate like impervious cover as a measure of storm water loading
- Efforts to reduce storm water flow will automatically achieve reductions in pollutant loading. Moreover, flow is itself responsible for additional erosion and sedimentation that adversely impacts surface water quality
- Stormwater control measures that harvest, infiltrate and evapo-transpire storm water are critical to reducing the volume and pollutant loading of small storms

3.3.2 Recommended Best Management Practices

The current MS4 NPDES permit will expire in June 2013. Compliance with minimum control 5 – post construction runoff controls is the biggest issue. The City's current storm water regulations partially address this by requiring post construction runoff rates to match predevelopment runoff rates for the 2-year and 25-year design storms. While this minimizes changes in peak flows, it does not specifically address minimizing impacts to water quality. Therefore, it is recommended that the City adopt criteria to capture and treat runoff from common rainfall events to ensure compliance with the permit.

Communities in Missouri, including Kansas City and St. Louis MSD, have adopted criteria that capture and treat runoff from the Water Quality storm. The Water Quality storm is a storm in which the rainfall depth is equal to or less than the rainfall depth from 90 to 95% of the storms in a given year. St. Louis has adopted criteria that use the 90% value or 1.14 inches of rainfall and it is recommended that St. Peters adopt these criteria as well. Similar to the existing criteria, it is recommended that this apply

when the volume of runoff from a site will increase by 15 percent or more as a result of a 2-year 24 hour storm event.

The volume that is captured and treated is defined as the Water Quality Volume, WQ_v, where:

$$WQ_v = [(P)(R_v)(A)]/12$$

Where:

WQ_v = Water Quality Volume (acre-feet)

P = Rainfall depth in inches equal to the 90% runoff event (inches)

R_v = Volumetric runoff coefficient (dimensionless)

A = Area (acres)

$R_v = 0.05 + 0.009(I)$, I is the percent (whole number) impervious cover

Best management practices (BMP) are then designed to capture and treat this volume of runoff. BMP manuals tailored to a specific community and developed from scratch are typically expensive to develop. Much reference material has been developed within Missouri, the Midwest and across the country, so it is not recommended that the City develop its own BMP manual. Rather, it is recommended that appropriate sections of existing manuals and reference materials be adopted.

Existing BMP references used in Missouri include:

- Manual of Best Management Practices for Stormwater Quality, Mid America Regional Council, March 2008, used in the Kansas City area (MARC manual).
- Erosion Prevention and Sediment Control Guidelines, City of Springfield Department of Public Works Storm Water Services Division, October 2008.
- Joplin, Missouri Stormwater Management Criteria, Allgeier, Martin and Associates, Inc., April 2009.
- Maryland Stormwater Design Manual, Volumes I & II, Center for Watershed Protection and the State of Maryland Department of the Environment, October 2000 effective date July 1, 2001, used by St. Louis MSD (Maryland manual).

It is recommended that the focus be placed on using extended dry or wet detention to capture and treat the water quality volume storm. This will minimize impacts to the local development community because detention is already required by existing criteria. The basins should be configured to provide 40 hours of detention for the water quality volume storm. Extended dry detention typically provides 60 to 80 percent removal of sediments while extended wet detention typically removes 90 percent of the sediments. Both the MARC and Maryland manuals provide criteria for design of extended detention basins.

In addition to upgraded detention, the City should require hydrodynamic separators, filtration practices or proprietary BMP devices for "hot spots". Hot spots are land uses that contribute higher concentrations of metals, hydrocarbons and other pollutants and include the following (from the MARC manual):

- Fuel dispensing facilities
- Aboveground storage of liquid materials
- Exterior storage of bulk materials
- Material transfer areas and loading docks
- Equipment and vehicle washing facilities
- Covered vehicle parking areas
- High-use vehicle and equipment traffic areas, parking and vehicle storage
- Dog kennels, doggie day care, and veterinary clinics

Both the MARC and Maryland manuals provide design guidance BMPs for these areas. In addition, St. Louis MSD has developed specific information on hydrodynamic separators and proprietary devices for hot spots that can be found at the following websites:

- http://www.stlmsd.com/engineering/planreview/PlanReviewInformation/ProprietaryBMPs
- http://www.stlmsd.com/portal/page/portal/engineering/planreview/PlanReviewInformation/ProprietaryBMPs/MSDProprietaryBMPProgramGuidance-080213rev090105.pdf

3.3.3 Low Impact Development Recommendations

Creating an ordinance or other regulatory mechanism to require low impact development is not recommended because little developable property is left. Therefore, it is recommended that low impact development (LID) be encouraged but not required in new development or redevelopment projects. The purpose of encouraging LID is to minimize the runoff that occurs from development which reduces the detention and capture and treatment requirements discussed above. Principles of low impact development include the following:

- Plan first
- Prevent, then mitigate
- Minimize the disturbance
- Manage stormwater as a resource – not a waste
- Mimic the natural water cycle
- Disconnect, decentralize, distribute
- Integrate natural systems
- Maximize multiple benefits
- Make maintenance a priority

Planning for stormwater management from the earliest stages of the development process helps to ensure that natural resources are protected and the impacts of the site are minimized. Minimizing the amount of runoff generated from the site is the most effective way to manage stormwater. This is

accomplished by **minimizing the disturbance** of existing vegetation and soils as well as minimizing the impervious area of the site. BMPs should be designed to capture and treat the runoff that does occur.

Designing sites to take advantage of stormwater runoff instead of getting rid of it can create community amenities, reduce irrigation needs and costs through rainwater harvesting for building and site uses, and protect natural resources. Designing ways to capture and store runoff supports all of these benefits. Planning a development around streams, ponds, wetlands, and rain gardens often attracts residents and adds value to lots near them.

Mimic the natural water cycle. Designing the site to control peak flows as well the volume of runoff and infiltration of precipitation minimizes the impacts on water quality and stream health. This results in management of the full range of precipitation from frequent rainfalls to the infrequent flood events.

Disconnect, decentralize, distribute. Capturing rainfall where it falls is a very effective stormwater management technique. This is accomplished by disconnecting impervious areas from the drainage system, installing BMPs at individual lots and neighborhoods, and spreading them throughout the development.

Integrate natural systems. Protecting and taking advantage of native soils, vegetation, and natural resources minimizes the impacts of a development and can increase its value. Natural resources are effective stormwater management systems that provide water quality benefits and reduce flood peaks.

Maximize multiple benefits. Designing the site to preserve natural resources and incorporate BMPs using native vegetation can add to the social and economic value of a development and community as well as provide water quality benefits including:

- creating open space for recreation and amenity value,
- increasing property values, and
- decreasing construction and maintenance costs.

Make maintenance a priority. BMPs often require different types of maintenance than typical crews are used to performing. Designing BMPs with maintenance requirements in mind reduces their impacts. They may often require more frequent maintenance than inlet and pipe systems. Placing priority on training crews to properly care for BMPs and planning for and committing to scheduled maintenance programs is important for their long-term function.

It is suggested that implementation of LID can be encouraged through reductions in any fees collected for storm water management in the future.

3.3.4 Policy Recommendations Impact on Development Code

Section 405 of the City's Code of Ordinances is known as the "The City of St. Peters, Missouri, Zoning and Subdivision Codes" (Section 405). Section 405 has been reviewed to identify potential conflicts between these policy recommendations and the existing code.

Low Impact Development

Many of the concepts promoted in LID involve minimizing impervious areas. Areas of conflict identified in Section 405 include the following:

- Minimum residential lot size of 10,000 square feet
- Overall density of a development should be 12,000 square feet per lot
- Impervious area can not cover more than 50% of the area of a lot
- Minimum home size of 1800 square feet for a one story house

Communities have adopted or allowed reduced requirements for these criteria in concepts such as cluster development where homes and businesses are concentrated in a portion of the overall development. The overall imperviousness of the development can be maintained or reduced and open space or natural resource areas increased.

PHOTO: CLUSTER DEVELOPMENT, METROPOLITAN DESIGN CENTER

A Planned Unit Development (PUD) can be developed to allow these concepts under Section 405. The PUD allows an increase in density of development of up to 30%. Also, PUD's commonly also allow other criteria to be modified as part of the agreement between the developer and the City. This existing mechanism can be promoted to encourage developers to employ LID principles.

Longer-term, adoption of Conservation Development code language is recommended. This code would specify the terms and conditions of the use of LID and reduce costs and staff-time in reviews compared to achieving LID through the PUD. The city of Kansas City, Missouri recently adopted conservation development and the code language can be accessed at the following website.

http://library.municode.com/index.aspx?clientID=10156&stateID=25&statename=Missouri

Once at this website, the Conservation Development code is Section 80-209 within Chapter 80.

Reduced Parking Space Requirements

Many communities have considered reducing parking space requirements to minimize impervious areas and impacts to stormwater runoff. Another method for reducing these impacts is to promote use of permeable pavement in low volume traffic areas. This is especially viable in large mall areas where the majority of the parking is used seasonally.

Reduced Pavement Widths

Communities have also used narrower pavement widths in the proper setting such as residential streets. Emergency vehicle access needs must be considered. If the City considers this as a means to reduce runoff, the emergency response groups should be involved in the development of the criteria.

3.3.5 Stream Setback Recommendations

The City's current stream setback requirement provides basic separation of development from the streams. Much of the City is already developed therefore; an extensive stream setback ordinance is not recommended. However, several enhancements are recommended to reduce the potential for new development in flood-prone areas, development of areas susceptible to erosion along the streams, and to preserve and restore the natural setting of the City's streams.

1. Prevent construction of habitable buildings resulting from new development or redevelopment in the 100-year floodplain. Updated mapping has been developed that shows the limits of the 100-year floodplain. For stream reaches that do not have a defined floodplain, require that the floodplain be delineated as part of the development or redevelopment plan. Keep the 25 foot setback preventing all development from occurring in the 25 foot setback zone, and allow storage sheds and other small structures in the area outside of the 25 foot setback.
2. Prevent new development or redevelopment adjacent to areas of significant stream instability until the stream segment is stabilized.
3. Develop an education program for neighborhoods that provides information on stream stability and how the residents' actions impact the streams. An example brochure is provided in Appendix I. Meetings with neighborhoods should be scheduled to provide an overview of the City's stream protection/enhancement program and identify ways they can help protect/enhance the streams.
4. A figure presenting stream segments identified for protection and target recommendations for no new development is presented in Appendix J. This should be included in the City's Comprehensive Plan to bolster their legitimacy and effectiveness.
5. Encourage use of native and riparian species in the 25 foot setback zone and discourage turf grasses and mowing.
6. Establish setbacks as wide as practicable when completing improvement projects or easements along the streams. The goal of this program will be to achieve the performance as outlined in item 1, but will need to be determined on a project-by-project basis.

3.3.6 FEMA Community Rating System Recommendations

The Community Rating System (CRS) is a program within the National Flood Insurance Program (NFIP). The stated objective of the CRS is to "reward communities that are doing more than meeting the minimum NFIP requirements to help their citizens prevent or reduce flood losses".

The CRS offers discounts of up to 45-percent on flood insurance premiums for property owners in participating communities. Other benefits to citizens and the City of performing CRS creditable activities include:

- Enhanced public safety
- Reduction in damage to property and public infrastructure
- Avoidance of economic disruption and losses
- Reduction of human suffering
- Protection of the environment

- The ability to evaluate the effectiveness of its flood program against a nationally recognized benchmark
- Technical assistance in designing and implementing some activities is available at no charge
- Implementing some CRS activities, such as floodplain management planning, can help a community qualify for certain federal assistance programs

It is recommended that the City contact the local FEMA office to discuss participation in this program. FEMA staff can assist with an evaluation of on-going activities to determine if reduced insurance premiums are likely and will identify the types of information required by FEMA for participation.

3.3.7 Street Sweeping Recommendations

Recent research has shown that street sweeping is most effective when it occurs every two weeks. While that may not be practical for St. Peters, targeting the roadways that are nearest the streams and detention basins is recommended. This should be completed monthly.

Research has also shown that street sweeping following thawing of significant snow events can be effective at removing the sand and salt and other debris buildup from these events. If budgets allow, this practice is recommended and at a minimum the streets should be cleaned early in the Spring.

3.3.8 Sediment and Erosion Control Recommendations

The silt load in the City's streams has been identified as an on-going problem. One of the identified sources of the silt load is silt-laden runoff from construction sites. The existing ordinance provides the regulatory mechanism to address this issue for larger sites. Enforcement and education are recommended to minimize the impacts of construction. Enforcement requires adequate staff resources to inspect each site frequently enough to identify issues and require remedies before multiple storm events occur. Two existing staff are dedicated to conducting Inspecting erosion and sediment control facilities weekly and after rain events. In addition, four engineers can fill in if needed. Another concern is the amount of sediment eroded from individual building lots. The current regulations exempt most residential home construction. Many examples exist where the exemption limits are much lower than St. Peters. One good example is in St. Louis County where the limits for exemption are a volume of 30 cubic yards or surface area of 2000 square feet. Lowering your limits to these values would significantly increase the number of home construction sites that are required to include sediment and erosion control. Building permits range from 30 to 100 annually. The City of Lincoln, Nebraska has recently implemented a program that addresses this issue. It can be found at the following website:

http://www.lincoln.ne.gov/city/pworks/watrshed/require/erosion/ilnoi.htm

It requires an individual lot notice of intent to comply with the City's requirements as well as a storm water pollution prevention plan to be submitted with the building permit.

The design standards used in St. Peters were adopted in the early 2000's. A new design guideline has been published by MDNR titled: **Protecting Water Quality: A field guide to erosion, sediment and storm water best management practices for development sites in Missouri and Kansas, January 2011.** It provides design standards for many newer BMPs for sediment and erosion control as well as updates

to many included in the existing design standards which incorporate lessons learned over the last decade. It is recommended that this document be adopted as a supplement to the existing design standard. MDNR completed a series of training sessions on this document. The City should send selected staff if additional training is scheduled. Further, developers and homeowners operating in St. Peters should be encouraged to attend.

The existing City code prohibits dumping of lawn waste into stream corridors in section 215.010.J.5. Dumping of other types of refuse is prohibited by 215.010.F. Abatement measures are also addressed in section 215.030. Therefore, additional ordinance language or policies are not required. However, current staff resources are not adequate to identify the sources of illegal dumping and enforce the existing ordinances. A public education program to inform residents of the existing rules, impacts of illegal dumping, and reporting mechanisms is recommended to reduce the amount and frequency of illegal dumping.

3.3.9 Homeowner Drainage Issues Recommendations

During field investigations, it was noted that downspouts and pool drainage pipes extended beyond the stream banks. These cause erosion and are vulnerable to being swept away during large runoff events. Development of a program to educate homeowners of alternatives to this is recommended. Alternatives for downspouts include discharge to a rain garden, swale with bed protection as needed, or burying the discharge pipe in the bank of the stream and directing its discharge to a pool or riffle in the stream. The MARC Manual includes rain garden design guidelines and many other on-line resources are available.

For pool drainage, use of temporary pipes to direct the flow into a pool riffle area of the stream is recommended.

Education and Awareness

A comprehensive education and awareness program is recommended to involve the community in the new approaches to storm water management. This should include not only the residents as noted above, but also developers, builders, and the designers that work in the City.

Homeowner Training

- Use of the City's compost program for grass clippings
- Importance of maintaining a buffer in your backyard
- Connection of downspouts to rain gardens or proper locations in the streams
- Native plant identification

Developer Training

- Participation program to inform them of the new policies and incorporate appropriate modifications
- Education on new extended detention basin and "hot spot" BMP design guidelines
- Education on erosion and sediment controls

Staff Training including other Departments that help manage Storm Water

- Checklist and training on proper installation and maintenance of erosion and sediment controls
- Vegetation identification for honeysuckle management
- BMP maintenance
- New policies

Water Quality Related Programming

- Clean Stream Teams to remove litter from streams
- Adopt-a-Wetland Teams

3.3.10 Summary of Recommendations

BMPs

- Require capture and treatment of the Water Quality Volume storm for new development and redevelopment when the volume of runoff from a site will increase by 15 percent or more.
- Focus on capturing and treating the runoff through extended dry or extended wet detention.

LID

- Require BMPs such as hydrodynamic separators or filters to treat runoff from "hot spots".
- Encourage LID and provide incentives for using LID in new development and redevelopment.

Stream Setbacks

- Prevent construction of habitable buildings in the 100-year floodplain.
- Keep the 25-foot setback preventing any development in this zone.
- Allow non-habitable structures outside of the 25 foot setback zone.

FEMA

- Investigate participation in the Community Rating System through the National Flood Insurance Program to reduce insurance rates for residents and businesses.

Street Sweeping

- Sweep streets near streams monthly beginning in early Spring.

Sediment and Erosion Control

- Reduce the threshold for requiring sediment and erosion control to movement of 30 cubic yards of material or disturbing 2000 square feet.
- Adopt the new guideline published by MDNR: Protecting Water Quality: A field guide to erosion, sediment, and storm water best management practices for development sites in Missouri and Kansas, January 2011.

Homeowner Drainage Issues

- Develop a program to educate homeowners about alternative downspout and pool discharge practices.

EDUCATION AND AWARENESS

- Develop a comprehensive education program that targets homeowners, developers, staff and water quality related programming.

4.0 Modeling of Recommendations

This section provides a summary of the modeling completed to apply the management measures discussed in Section 3 to solve the City's stormwater concerns.

4.1 Hydrology Model

4.1.1 Detention Basin Analysis

The HEC-HMS model received from the USACE was previously updated to include the screened detention basins. This updated HEC-HMS model was used to evaluate potential detention improvements throughout the watershed.

The need for hydrologic improvements to a detention basin was generally indicated by either downstream flooding or by overtopping of the dam structure. Hydrologic improvements to the detention basins were modeled and determined using HEC-HMS. This process involved determining the available potential on-site for increasing the flood pool of the facility. If additional area was not available for increased storage, the field observation materials were reviewed to determine the feasibility of improvements to the outlet works or embankment of the facility. Once a plan was developed for the detention basin, the stage-storage and storage-outflow tables within HEC-HMS were modified to reflect the proposed improvements.

Detention improvements were modeled by altering the basin's stage-storage and storage-discharge tables. These tables were also used to evaluate the extent of overtopping by expanding the table to capture the overtopping flow.

4.1.2 Regional Detention Analysis

Regional detention modeling was also performed to evaluate the potential of this watershed improvement measure. Regional detention opportunities to prevent large-scale flooding were evaluated, in contrast to the design of most detention basins in the City which address local peak flow management. Regional flooding was observed in several locations around the City. The regional detention locations considered were largely located in the Spencer Creek watershed. Figure 23 presents all regional detention locations considered in the City. Table 10 presents the list of locations that were investigated in HEC-HMS:

TABLE 10. REGIONAL DETENTION ANALYSIS

Location	Project Identifier	100-year Flow Goal	Reduction in 100-year Flow
Structure Flooding along Main Spencer Creek	IP-7368-4-3	1200 cfs	1073 cfs
Burning Leaf Drive	IP-7368-2-1	850 cfs	1423 cfs
Structure Flooding along Main Spencer Creek	IP 7369-3-1	3500 cfs	895 cfs

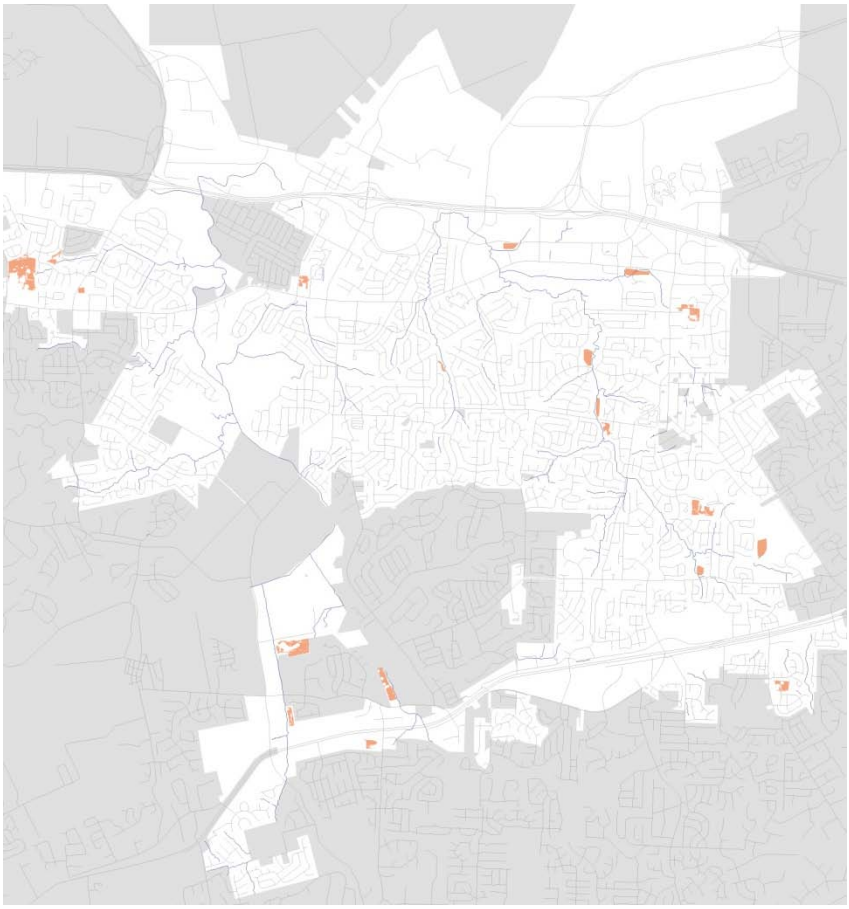

FIGURE 23. REGIONAL DETENTION OPPORTUNITIES

The first avenue when considering regional detention was to increase the detention provided by existing facilities. The first and best opportunity to impact the flows along Spencer Creek at an existing facility is the McClay Valley detention basin (DB-7468-06). However, even if the McClay Valley facility detained all flow draining to it during the 100-year event, only 593 cfs would be removed from Spencer Creek. This flow reduction does not come close to solving the regional flooding downstream. Even removing all flow exiting each of the existing detention basins contributing to Spencer Creek would not alleviate the regional flooding.

Regional flooding was observed within the Dardenne Creek watershed. Sufficient area within the City was not available to provide regional detention that would address this flooding. In order to detain flows along Dardenne Creek within the City limits, a large facility would be required. This is mainly due to the location of the City in the watershed; the City is located at the downstream end of a large drainage area. Additionally, regional detention along Dardenne tributaries was not deemed feasible for two reasons. Again, sufficient vacant area was not available within the Dardenne tributary watershed in the City's limits. In addition, even if sufficient space were available to detain the flows produced by Dardenne tributaries within the City, the reduction in flow provided by these facilities would not be sufficient to alleviate regional flooding in Dardenne Creek. Municipalities within the Dardenne Creek watershed can improve the flooding conditions by preventing fill and development in the floodplain. In the Crystal Lake subdivision, many structures are flood prone because they were built within the 100-year area of inundation.

4.2 Hydraulic Model

4.2.1 HEC-RAS Modeling

The analysis of existing conditions documented several flooding locations in neighborhood locations and at stream crossings. These locations are summarized in Section 2.3.

Level of Service

Each conceptual solution was designed to achieve a specific level of service. For overbank flooding, the objective was to remove as many buildings as possible from the 100-year floodplain. For bridge and culver crossings, conceptual crossings were input to the model to achieve sufficient freeboard. Bridge design guidance is provided in Section 4.040 of the St. Louis MSD 'Rules, Regulations, and Design Requirements'. This states that the "lowest point of bridge superstructure shall have 2-ft of freeboard from the 15-yr water surface elevation and 1-ft freeboard from the 100-yr water surface elevation."

Modeling the Concepts

The hydraulic model was used to develop conceptual recommendations at each of these locations. Additionally, opportunities for regional detention were explored based on a proposed peak 100-year flow. Table 11 summarizes the hydraulic modeling updates developed to address flooding at each project location.

TABLE 11. HEC-RAS MODELING OF PROJECTS TO ADDRESS FLOODING

Project ID	Location Description	Stream Improvement
IP-6970-1-2	Richmond	Enlarged XS 53654, lowered inverts on either side of Mexico, raised low steel on Mexico, widened downstream of Mexico
IP-6970-4-2	Dardenne Creek between Mexico Road and I-70	
IP-7069-1-2	Crystal Lake	
IP-6970-4-1	Calwood	
IP-7069-1-1	Crystal Lake Detention	
IP-7368-2-1	Burning Leaf Drive	Install 3 10x10 culverts with wingwalls, each is 32' long
IP-7369-1-1	Spencer Creek between Sutters Mill Road and Boone Hills Drive	Fill in sump area along Boone Hills Road, Replace culvert array with 4 - 12x15 (W*H), and fill in flloodplain areas just north of Showboat Circle on both sides of channel to raise floodplain grade by 2 feet, slope back to channel.
IP-7369-3-1	Mill Race Ct.	Lower channel invert and develop more consistent slope for approximately 1800 linear feet of stream between Stations 26969.20 and 25137.03.
IP-7066-4-1	Pittman Hill Road	Replace culvert with bridge crossing.
IP-7368-4-3	Spencer Creek in Millwood Subdivision	Fill in floodplain along the reach 50 feet downstream of Millwood Drive, channel improvements to reduce invert elevation, AND complete project IP-7368-2-1 (Install 3 10x10 culverts with wingwalls, each is 32' long)
IP-7368-1-3	Jungermann Road at Spencer Creek	Need to add a berm on the downstream side along 160 linear feet or buyout 1 Waterbury Court property. Culvert improvements would include: Replace existing culvert with raised bridge crossing and fill sump area. Replace stormwater infrastructure. Overall impacts to 600 LF of roadway.

The hydraulic model developed for this study was not sufficient to analyze stream reach design velocities; however, the velocities produced in the model were used as reference during concept development.

4.2.2 XP-SWMM Model Development

PUMP STATION ANALYSIS

To eliminate the flooding completely within Old Town during the 15-yr, 24-hr storm, a model was constructed using the Old Town improved condition model (model with pipe improvements shown in Table 2) and the pump station capacity was increased until all flooding was eliminated during the 15-yr, 24-hr storm. This analysis showed that it would take a pump station that could convey approximately 275 cfs (~123,000 gpm) to completely eliminate flooding within Old Town for the 15-yr event. As this is a significant increase in pump station capacity, two additional scenarios were evaluated with pump station capacities between the existing capacity and the capacity required to eliminate flooding. Table 12 summarizes those results:

TABLE 122. PUMP STATION CAPACITY ANALYSIS

Pump Station Capacity (cfs)*	Approximate Inundation Water Surface Elevation (ft)	Maximum Inundation Area (ac)
54	440.3	28.8
110	440.2	28.3
220	440.1	26.4
275	N/A	0

* - Although the Old Town Pump Station nominal capacity is listed as 44 cfs, during a flood event the static lift of the pump is reduced due to the high water surface elevation in the wet well. The flows listed in the table are based on the maximum flow produced by the model for the pumps.

The existing pump station is undersized and additional capacity is needed. The table above shows that significant improvements are necessary however there was not sufficient detail at this study level to determine the exact recommendation for the pump station upgrade.

EAST OLD TOWN CONVEYANCE SYSTEM

This system should be studied in more detail to verify elevations of swale. From the information provided and a rough model of the drainage system, the following recommendations have been developed: 1) reduce the amount of vegetation in the swale to reduce roughness (ongoing maintenance not included in capital cost), and 2) replace pipes with 48" RCP, set at slightly lower elevations.

4.3 Water Quality Model

P8 input parameters for each retrofit detention basin included the area and impervious fraction of the detention basin's contributing drainage area, the detention basin adjusted area and volume, and whether the detention basin was normally dry (extended detention) or contained a permanent pool (wet detention). For each retrofit, the area was defined within ArcView, using LIDAR contours and assuming a 4 ft deep average pool depth to determine the average pool area. The watershed areas and imperviousness were based on GIS data provided by the City and the HEC-HMS hydrology model. A summary of the design standards applied to detention retrofits is provided in Section 3.

4.3.1 Dry Detention Retrofit Modeling

To simulate a dry detention retrofit, only a flood pool volume was used. A permanent pool does not exist in a dry detention basin. The flood pool volume was established using a stage storage curve (if available) or estimated with LIDAR contours. The existing depth was established by taking the flood pool volume and dividing by the revised area. The flood pool volume was assumed to be the storage up to the spillway elevation. For a dry to wet basin retrofit, the flood pool volume does not change.

The flood pool volume was updated to include the water quality volume and any additional storage capacity included in the recommendation. A revised flood pool area was calculated using the new flood pool volume and the existing depth.

4.3.2 Dry to Wet Detention Retrofit Modeling

To simulate the retrofit of a dry detention basin to a wet detention basin, a permanent pool that is equal to the volume of the water quality volume was established for each basin. The area of this pool was assumed to remain constant to existing conditions. The existing flood pool volume was established using stage storage curve (if available) or LIDAR contours. The existing depth was established by taking the flood pool volume and dividing by the revised area. The flood pool volume was assumed to be the storage up to the spillway elevation. For a dry to wet basin retrofit, the flood pool volume and area does not change.

4.3.3 Wet Detention Retrofit Modeling

For a wet basin retrofit, the permanent pool was not changed. There was no modeled benefit for wet detention retrofits.

4.3.4 Removal Efficiencies of Retrofitted Detention Basins

As discussed in Section 2.4.2, the removal efficiencies of 74 existing stormwater detention basins were evaluated using the P8 model. Fifty-eight of these basins were recommended for retrofitting to improve

their TSS removal efficiencies. For the West Dardenne regional drainage area, 11 of the 17 existing detention basins were recommended for retrofits. Of those, 7 were dry to wet retrofits and 4 were wet basin retrofits. The P8 model predicted the most improvement in the dry to wet retrofits because of the increase in the hydraulic retention times. Detention basins that were previously less than 30 percent efficient would be over 90 percent efficient with the retrofits.

The P8 model predicted that the average TSS removal of all the ponds in the West Dardenne regional drainage area, including the 11 retrofits, was 74 percent compared to 70 percent for the sub-basin without retrofits, as shown in Table 13. The P8 model predicted that approximately 69,000 lb/yr removal by these 17 detention basins. Approximately 915,000 lbs per year (lb/yr) of TSS would be contained in stormwater runoff from the entire drainage area; therefore, the overall TSS removal of the 17 detention basins with respect to the loading from the entire regional drainage area was 7.5 percent compared to 6.6 percent removal of the detention basins without the retrofits, as shown in Table 13. Table 13 summarizes the removal overall efficiencies for the retrofitted stormwater detention basins in the eight regional drainage area. Tables of removal efficiencies and other characteristics for each of the retrofitted basin are included in Appendix C.

TABLE 133. REMOVAL EFFICIENCIES FOR RETROFITTED STORMWATER DETENTION BASINS

		Total Regional Drainage Areas	Detention Basins			
			In	Out	Removed	Removed
Regional Drainage Areas	Number of Retro/Total Basins	TSS-lb/yr	TSS-lb/yr	TSS-lb/yr	TSS-lb/yr	%
Baltic	8/8	298,595	44,878	11,739	33,139	74
West Dardenne	11/17	915,435	86,769	18,077	68,692	79
East Dardenne	9/9	252,434	73,836	26,479	47,357	64
East Spencer	3/5	271,909	61,911	21,479	40,432	65
Plumm	4/4	590,753	39,304	30,298	9,006	23
Spencer Downstream	0	226,128	0	0	0	0
West Spencer	5/5	232,583	7,215	819	6,396	89
Spencer Upstream	18/19	688,865	134,126	31,725	102,401	76
Totals	58/67	3,476,702	448,039	140,617	307,423	69

4.4 GIS Modeling

The application of geo-spatial databases is an efficient approach to asset management. The ESRI ArcView GIS was integrally related to the progress of this study and the development of the products. This section describes the GIS deliverables that result from this study and a GIS process used to determine optimal locations for stormwater BMPs.

4.4.1 Environmental Sensitivity Index

A water quality best management practices (BMP) locator assessment was completed for the watersheds located within the City. The objective was to assess and classify the relative value of land for capturing, storing, and infiltrating stormwater runoff; and to identify vacant land that could be used to construct new water quality BMPs if desired to meet the City's stormwater management objectives. The following sections document the BMP locator process and summarize the inventory results.

The basic principal for the environmental sensitivity index (ESI) comes from the weighted factor overlay technique by Ian McHarg (1992). In GIS, the weighted overlay is done with numeric combinations of factor values that produce results which are good for spatial and statistical analysis, and map visualization. In the St Peters ESI model, 14 factors were combined into five categories for analysis: soils, hydrology, surface cover, topography, and social value.

After completing this initial analysis, the following additional analyses were completed to identify locations that are actually feasible for consideration as future BMP locations.

1. Surface flow patterns were analyzed using the DEM, and screened out parcels that are not on significant flow lines.

2. The analysis isolated parcels that are either public, semi-public, or institutionally owned, and privately owned vacant parcels that could potentially be acquired for BMP creation (either for this project, or future projects). This subset of parcels was then sorted by size to help categorize each opportunity's relative potential benefit into tiers. The tiers were: less than 2 acres; 2 to 10 acres; 10 to 40 acres; and greater than 40 acres.

In St. Peters, Missouri, high stormwater BMP suitability exists where soil, hydrologic, and vegetative systems overlay in a distinctive dissected, dendritic landscape matrix pattern with upland connections through suburban open space, roadways, and linear easements.

4.4.2 GIS Deliverables

Table 14 summarizes the deliverables within the database.

TABLE 144. GIS DATABASE DELIVERABLES

Layers Identifying WQ Challenges	
Water Quality Concern Points	This layer identifies locations where a water quality concern was noted during field reconnaissance.
Water Quality Concern Reaches	This layer identifies locations where a water quality concern was noted during field reconnaissance.
Existing Stream Geomorphology Layers	
CCSM Reaches	This layer includes information gathered in the field and the CCSM score for each reach in the City.
CCSM Points	This layer includes information gathered in the field at the point it was taken.
Comments	This layer provides comments from the field at the point taken.
SAI Stream Type	This layer provides the SAI stream type and associated data gathered in the field.
Flood Prone Areas Layer	
Refined 100-year	This layer provides the refined 100-year area of inundation generated as a part of this study.
Detention Basin Utilization Layers	
Selected Detention Basins	This layer presents the 75 detention basins that were selected for the 2011 study. It includes DB-7270-05, a basin that was not analyzed due to lack of available data.
Detention Basins CDA	This layer provides approximated contributing drainage area information, generated using ArcHydro tools
Selected Detention Basins CDA	This layer provides approximated contributing drainage area information generated using ArcHydro tools
Layer of existing Stormwater BMPS and Proposed BMPS	
ESI Results	This layer presents the results of the environmental sensitivity index, identifying future BMP locations.
Stormwater Improvements CIP Layer	
CIP Projects	This layer includes all 2011 CIP projects.
Other	
Impervious Cover	This layer is an aggregate of the impervious cover layers provided by the City.
P8 Watersheds	This layer includes P8 regional drainage areas referenced in the report.
XP SWMM Subcatchments	This layer includes subcatchments used in the XP SWMM modeling.
Spencer Watershed	This layer is a polygon of the Spencer Creek watershed.
CCSM Reaches in Projects	This layer includes aggregated CCSM reaches within CIP project polygons.

5.0 Stormwater CIP

5.1 Project Locations and Conceptual Improvements

Field data and GIS analysis of existing conditions was used to establish project locations. The majority of flooding locations were selected as project locations. Stream reaches with scores greater than 13 were selected for stability improvements and then reaches were grouped with consideration to constructability. Special consideration was given to documentation of stream stability relative to infrastructure or buildings. Stream reaches with SAI scores of 1 and 2 were identified as preservation project areas. Detention basins that showed evidence of flooding or water quality issues such as excessive erosion or algae were selected as project locations. Project polygons were developed to encase project areas. The following outlines the project selection process.

1. Known Flooding Locations
    a. HEC-RAS Model
    b. Anecdotal
2. Detention Basins
    a. Undersized (Model)
    b. Anecdotal Issues or Problem Spots
    c. Opportunity to place or retrofit to In-Line Structure
    d. Downstream Concerns (Field Notes)
    e. Opportunities (Field Notes)
3. Stream Stability
    a. CCSM Score (>18)
    b. SAI Score (3-5)
    c. Field Notes, Anecdotal Selections
4. Water Quality Concern Locations
    a. Noted in Field
    b. Nutrients
    c. Sediment Contributions
5. Restoration Opportunities
    a. Select SAI Score (1-2)
    b. Select CCSM Score (>13)
6. Preservation Opportunities
    a. Select SAI Score (1)
    b. Select CCSM Score (<13)

The selected conceptual improvements are described in Section 3.0. Additional information related to plant palettes and plant species to be controlled is provided in Appendices F and G.

5.2 Opinion of Probable Costs

Black & Veatch compiled an opinion of probable cost for each project based on unit cost data that reflects recent project experience in the region. A general contingency of 25% was added to each cost.

Mobilization and erosion control is estimated at 15% and engineering at 15% of the subtotal cost. A 10% contingency is added for potential utility conflicts. The following table presents the unit cost of each project component.

TABLE 15. UNIT COSTS

General	Unit Cost	Unit
Category 1: Asphalt, 30-ft wide	$150	LF
Category 2: Asphalt, 50-ft wide	$250	LF
Category 3: Asphalt, 75-ft wide	$375	LF
General Concrete (CY)	$1,000	CY
General Site Grading	$2	SY
Category 1: Pipe removal cost (24-36 inch)	$90	LF
Category 2: Pipe removal cost (>36-inch)	$150	LF
Category 1: 4x4 Culvert Box	$210	LF
Category 2: 10x10 Culvert Box (Burning Leaf Drive)	$520	LF
Category 3: 12x15 Culvert Box (Boone Hills Drive)	$800	LF
Detention		
Structure Excavation	$20	CY
Category 1: Replacement Structure	$5,000	EA
Category 2:Replacement Structure	$10,000	EA
Category 1: Modification Structure	$3,000	EA
General	**Unit Cost**	**Unit**
Category 2: Modification Structure	$5,000	EA
Remove concrete swale	$10	LF
CUT	$15	CY
FILL	$25	CY
Reinforced Concrete (Slab on Grade)	$550	CY
Category 1: Small Forebay	$10,000	EA
Category 2: Large Forebay	$15,000	EA
Wetland Plantings (area, sf)	$3	SF
Vegetation Buffer area, (sf)	$0.16	SF
Establish native seeding in basin, include amendment of soils* update cost	$3	SF
Grading and Vegetation Establishment on Slopes for Erosion Stabilization	$0.16	SF
Riprap (D50 18-inch through 36-inch)	$65	CY
Detention - Community Asset Cost		LS
Stream Stability		
Category 1: Rock Check (5-10 feet across)	$3,000	EA
Category 2: Rock Check (10-15 feet across)	$4,000	EA
Category 3: Rock Check (15-20 feet across)	$7,500	EA

Category 4: Rock Check (>20 feet across)	$10,000	EA
Category 1: Left Bank, Slope and Vegetate (Bank Height: 5 ft)	$195	LF
Category 2: Left Bank, Slope and Vegetate (Bank Height: 10 ft)	$286	LF
Category 3: Left Bank, Slope and Vegetate (Bank Height: 15 ft)	$430	LF
Category 4: Left Bank, Slope and Vegetate (Bank Height: 20 ft)	$879	LF
Category 5: Left Bank, Low Flow Bench, Slope and Vegetate (Bank Height: 10 ft)	$247	LF
Category 6: Left Bank, Low Flow Bench, Slope and Vegetate (Bank Height: 15 ft)	$346	LF
Category 7: Left Bank, Low Flow Bench, Slope and Vegetate (Bank Height: 20 ft)	$645	LF
Category 8: Left Bank, 5-ft Wall, Slope and Vegetate (Bank Height: 5 ft)	$565	LF
Category 9: Left Bank, 5-ft Wall, Slope and Vegetate (Bank Height: 10 ft)	$656	LF
Category 10: Left Bank, 10-ft Wall, Slope and Vegetate (Bank Height: 10 ft)	$1,050	LF
Category 11: Left Bank, 15-ft Wall, Slope and Vegetate (Bank Height: 10 ft)	$1,507	LF
Category 1:Right Bank, Slope and Vegetate (Bank Height: 5 ft)	$195	LF
Category 2:Right Bank, Slope and Vegetate (Bank Height: 10 ft)	$286	LF
Category 3:Right Bank, Slope and Vegetate (Bank Height: 15 ft)	$430	LF
Category 4:Right Bank, Slope and Vegetate (Bank Height: 20 ft)	$879	LF
Category 5:Right Bank, Low Flow Bench, Slope and Vegetate (Bank Height: 10 ft)	$247	LF
Category 6:Right Bank, Low Flow Bench, Slope and Vegetate (Bank Height: 15 ft)	$346	LF
Category 7:Right Bank, Bench, Slope and Vegetate (Bank Height: 20 ft)	$645	LF
Category 8:Right Bank, 5-ft Wall, Slope and Vegetate (Bank Height: 5 ft)	$565	LF
Category 9:Right Bank, 5-ft Wall, Slope and Vegetate (Bank Height: 10 ft)	$656	LF
Category 10:Right Bank, 10-ft Wall, Slope and Vegetate (Bank Height: 10 ft)	$1,050	LF
General	**Unit Cost**	**Unit**
Category 11:Right Bank, 15-ft Wall, Slope and Vegetate (Bank Height: 10 ft)	$1,507	LF
Filter strips	$4	LF
Riparian Buffer (RPM Trees, Native Seeding)	$0.25	SF
Riparian Renovation (Management and Replanting)	$4,300	AC
Tree/Shrub Landscape (area,sf)	$17	SF
Signage (EA)	$5,000	EA
New Trail (LF)	$40	LF
Category 1: 15-inch RCP	$75	LF
Category 2: 24-inch RCP	$120	LF
Category 3: 30-inch RCP	$180	LF
Category 4: 36-inch RCP	$216	LF
Category 5: 48-inch RCP	$288	LF
Category 6: 60-inch RCP	$300	LF
New Inlets or Junctions	$5,000	EA
Pipe Crossing Protection	$5,000	EA
Drainage Outfalls to Channel	$3,000	EA

5.3 Prioritization of Improvements

The integration of stormwater best management practices with management of the City's natural waterways may substantially benefit flooding, water quality, habitat, and provide opportunities for public education regarding water resource issues. A prioritization scheme was developed based on the conditions established during this study. To evaluate the priority of each CIP project, the following categories were evaluated: flooding, stream stability, and ecology. These categories were weighted and summed to develop a subtotal value.

TABLE 156. WEIGHTED CATEGORIES FOR PRIORITIZATION

Flooding	Stream Stability	Ecology
40%	35%	25%

5.3.1 Flooding

Flooding locations were identified using hydrologic and hydraulic modeling results and anecdotal reports documented by the City and provided in a GIS database. The hydrologic model was used to determine when detention basin overtopping occurred during the 25-year event. The refined hydraulic model of the City waterways was used to generate a mapped area of inundation. This layer was evaluated to determine flooding locations. At each flooded location, the number of impacted structures or the impacted roadway was documented. Each bridge crossing was also evaluated within the hydraulic model to determine overtopping of bridge or culvert crossings. Project areas identified in this study, particularly regional flooding areas, should be further evaluated to characterize the extent of flooding in more detail.

5.3.2 Stream Stability

Stream stability was prioritized using the Channel Condition Scoring Matrix (CCSM) and the assessment of a stream reach's impact on infrastructure or property. During the field evaluation, each stream reach was evaluated using the CCSM. A resulting score was documented for each reach. Project areas often overlapped multiple stream reach and therefore, a weighted CCSM score was developed for each project reach. Each project was evaluated to determine if structures or property was negatively impacted by stream stability.

5.3.3 Ecology

The final category of prioritization relates to the ecology of the system. A water quality benefit was established for each detention basin retrofit. The categories were developed based on the potential increased removal of total suspended solid (TSS) provided by a retrofit option. This accounts for the contributing watershed characteristics and is influenced by the basin retrofit type. During the field evaluation, each stream reach was evaluated using the Stream Asset Inventory, as presented in Section 2.4.2. The resulting SAI score was used to establish a priority for restoring or protecting the stream reach within each project.

5.3.4 Other

Additional multipliers were used to influence the ultimate score, including frequency, visibility, and education.

TABLE 167. PRIORITIZATION SCHEME

Flooding Multipliers	
Regional Flooding	10
6 - 15 Homes &/or Businesses	2
1-5 Homes &/or Businesses	1
Major Street	2.5
Other Street	1
Dam Overtopping	2
Stream Stability Multipliers	
CCSM Score > 19	2
CCSM Score 13 - 19	1
Threatens Private Structure	1.5
Threatens Public Utility or Infrastructure	1.5
Threatens Public or Private Property	1
Ecology Multipliers	
Detention Basin WQ Benefit (Significant)	3
Detention Basin WQ Benefit (Good)	2
Detention Basin WQ Benefit (Average)	1
Detention Basin WQ Benefit (None)	0
SAI Category 5	2
SAI Category 3 & 4	1
Project Multipliers	
Frequency	
Common	2
Annual	1.5
Infrequent	1

5.3.5 Final Prioritization

Once the scores for the flooding, stream stability, and ecology were tallied and multiplied as described in the form and paragraphs above, the result is a final prioritization score. This can be considered a measure of the project's benefit to the City stormwater system. This score is one important measure to be considered as the City allots capital funds to stormwater improvements over the coming years.

Another important measure to be considered is the cost benefit ratio that can be calculated for each project. This ratio is simply calculated by dividing the cost by the benefit score. For this study, the cost

benefit scores were uniformly multiplied by 100 to avoid cumbersome decimal displays. Those projects with the smallest cost benefit ratios will provide the City with the most efficient use of improvement funds.

5.4 Schedule

The schedule for improvements will be based upon the prioritization scheme presented in Section 5.3 and the capital allotted to improvements in a given year. Some consideration of the prioritization scheme's inherent bias must occur when discussing the schedule of improvements.

For instance, if the City were to use the cost benefit ratio as the sole means to select projects, only a series of small projects would be completed. This is due to the fact that many of the projects with low cost benefit ratios are small projects that, while providing benefit to the watershed, have lower costs.

Alternatively, if the City were to use the benefit scores as the sole means to select projects, the City would likely only complete a very few, perhaps only a single, project(s) each year. This is due to the fact that many of the projects that provide a large benefit to the City's stormwater system are also large and expensive.

To avoid these scenarios, it is suggested that the City develop a hybrid selection and scheduling process. By this method, the City will develop a balanced schedule that will address multiple projects across the watersheds, but will also address important (high benefit) projects in as timely a manner as funding will permit.

5.5 GIS Databases

Several GIS database were developed during this study to manage data collected in the field and data created during analysis. These geodatabases will be a valuable tool to manage assets during implementation of the stormwater CIP. The geodatabase organization is provided in Appendix I and the database is stored on the CD attached with this report.

6.0 Recommendations

This study resulted in the development of over 100 projects with a total cost of $125,000,000. These projects are located throughout the City and are classified as flooding, stream stability, detention, or preservation projects. Some projects are multi-functional and are associated with two or more of these categories. A comprehensive approach to solving stormwater-related concerns will include projects that are rated with a high priority score, projects that have a low cost benefit ratio and projects that preserve the City's existing resources. Sections 6.1, 6.2, and 6.3 include the top 10 projects in each of the following categories: high priority, low cost benefit and preservation value. One project is represented as both a high priority and a low cost benefit project. The total estimated cost for these 29 projects is approximately $17,500,000.

Summaries for each project developed for the CIP are provided in Appendix H.

6.1 Top 10 Priority Projects

The following locations received the highest priority rating. These projects are not the most cost effective but represent significant stormwater concerns in the City.

1. Dardenne Creek at Mexico Road and I-70 (IP-6970-4-3)

This flooding project received a priority score of 1061 points and an estimated project cost of $4,915,000.

Existing Description

The project location is located at the Mexico Road and I-70 crossings of Dardenne Creek. The bridge information provided in the HEC-RAS model showed that these bridges cause constrictions during high flow events in Dardenne Creek, resulting in flooding upstream.

Improvement Description

Address flooding by replacing the bridge at Mexico Road and raising the low steel. Channel widening is recommended along 2,000 linear feet, extending through the I-70 bridge crossing. This solution lowers the water surface elevation and removes some houses from the 100-year floodplain.

2. Burning Leaf Drive (IP-7368-2-1)

This flooding project received a priority score of 795 points and an estimated project cost of $258,000.

Existing Description

The crossing of Burning Leaf Drive in the Tanglewood subdivision is currently overtopping in the hydraulic model. Six structures are impacted by the 100-year inundation area.

Improvement Description

The low area at Burning Leaf Drive should be raised along approximately 300 linear feet and a new triple array of 10x10 culverts should replace the existing bridge crossing. Additionally, 150 linear feet of channel improvements will be necessary as part of the culvert installation. One other option evaluated was new regional detention upstream; however, there was not sufficient land available to add the capacity needed to reduce peak 100-year discharge.

3. Jungermann Road at Spencer Creek (IP-7368-1-3)

This flooding project received a priority score of 730 points and an estimated project cost of $2,347,000.

Existing Description

The Jungermann Road Bridge project is located along the main channel of Spencer Creek. This has been an area of concern during each of the significant 2011 storm events. The existing hydraulic model did not accurately reflect flooding at this location because the USACE hydrologic analysis for the area was not representative of contributing drainage area. Flooding is the result of insufficient conveyance area and downstream conditions. The hydrology of this area should be studied further.

IMPROVEMENT DESCRIPTION

To address overtopping that resulted from the conservative modeling approach, the existing culvert at Jungermann should be replaced with a bridge. The sump area along Jungermann should be filled - impacting approximately 800 linear feet of this road. Additionally, a berm should be constructed on the north side of the channel, downstream of Jungermann Road.

4. Pittman Hill Road (IP-7066-4-1)

This flooding project received a priority score of 607 points and an estimated project cost of $1,453,000.

EXISTING DESCRIPTION

Pittman Hill Road is overtopping in the hydraulic model. Two structures are shown within the FEMA floodplain and this study's refined area of inundation during the 100-year event. There was one tall shear bank noted on the downstream side of Pittman Hill Road. Owners on the adjacent property said that this bank was eroding quickly. Upstream of Pittman Hill Road, the riparian corridor is in fair condition but very narrow on the west side. The main channel banks show evidence of continuous scouring. Debris jams were noted in the field study.

IMPROVEMENT DESCRIPTION

The concept includes replacement of the Pittman Hill Road culverts with a bridge to address overtopping. Additionally, bank stabilization of the 24-ft tall shear bank downstream of Pittman Hill Road is recommended. This bank is upstream of the area that was protected as part of the Hwy 94 construction project (2010-2011). This project also includes vegetative and riprap toe protection of the reach upstream of the culvert crossing where shorter banks show evidence of continuous scouring. The corridor upstream of Pittman Hill Road should be renovated. The corridor downstream of Pittman Hill Road will require the establishment of a new riparian corridor and limited stream widening.

5. Old Town I-70 Service Road North (IP-7171-2-1)

This flooding project received a priority score of 540 points and an estimated project cost of $765,000.

EXISTING DESCRIPTION

Swale draining east Old Town toward regional detention basin along Iffrig Road is flooding Old Town.

IMPROVEMENT DESCRIPTION

This system should be studied in more detail to verify elevations of the swale. From the information provided and a rough model of the drainage system, recommendations include: reducing the amount of vegetation in the swale to reduce roughness (ongoing maintenance not included in capital cost) nad replacing pipes with 48" RCP, at a lower elevation.

6. McClay Valley Detention (IP-7468-3-3)

This detention and stream stability project received a priority score of 525 points and an estimated project cost of $1,650,000.

Existing Description

The McClay Valley Detention Basin (DB-7468-1) receives a significant amount of debris and flow from upstream sources and presents a unique demonstration opportunity as a larger detention basin in the watershed. Additionally, the City noted that the contributing storm sewer infrastructure is not designed to adequately drain the surrounding area to the basin. Hydrologic modeling showed overtopping of the downstream road during the 15-yr event.

Improvement Description

The conceptual solution is focused on addressing the issues at the McClay Valley detention basin and developing a second detention upstream of the large basin. A berm embankment and outlet works would be constructed for the upstream detention basin. At the main basin, a forebay would be developed with wetland plantings and the outlet structure may be modified. Local drainage to the main detention basin would be replaced to allow functional drainage. Edge treatment may include a vegetative buffer, wetland bench, or combination established on the perimeter of the main basin.

7. West Drive (IP-7369-4-1)

This detention and stream stability project received a priority score of 428 points and an estimated project cost of $990,000.

Existing Description

This project area includes the West Drive wet detention and stream reaches in the contributing drainage area, which extends over City and county property. The West Drive detention basin is currently used for recreation (i.e. fishing) and is owned by the City. There are three contributing tributaries that drain into the basin: Elm Tree Tributary is at the far west, Hi Pointe Tributary is the center tributary, and Peach Street Tributary is at the far east of the project area. The upstream two thirds of Elm Tree Tributary were characterized as a fairly incised channel with unconsolidated leaf litter debris and loose sediment bed material. Between pools in this reach, there were nick points (dropping as much as 2 feet). The Hi Pointe Tributary was characterized as an incised channel with debris dumping in the downstream half. In the upstream half it is a meandering intermittent channel with poor understory vegetation. The Peach Street Tributary flows through backyards and has been associated with flooding problems at Peach Street. The City has made some improvements to remedy flooding problems, however, neighbors complain of bank erosion encroaching on backyards. The reach upstream of Peach Street is a low flow channel through a backyard that functions as a detention basin. The reach downstream of Peach Street lacks a buffer and dumping is prevalent.

Improvement Description

This concept includes improvements to the West Drive detention basin and each of the contributing tributaries. At the detention basin, the capacity should be increased by raising the profile of West Drive and replacing the basin outlet structure. The West Drive detention basin should be further studied to

refine the proposed solution. Wetland benching and a buffer should be established on the southwest and a vegetated buffer should be installed around the entire perimeter. Stabilization of the Elm Tree tributary should include vertical grade control to prevent further degradation, establishment of a riparian buffer when it is not present, renovation of the existing corridor and bench slopes on the incised channel (upstream two thirds of the tributary). The Hi Pointe Tributary should be routed to drain into the south end of the detention basin and the existing tributary should be cleared and graded to be included as part of the detention basin (reducing the stream length by 529 linear feet). Riparian renovation is recommended throughout the remaining reach. Stabilization of the Peach Street Tributary includes two strategies, 1) the reach between Peach Street and the wet detention basin should be stabilized and revegetated, 2) the reach upstream of Peach Street should have a narrow riparian corridor re-established.

8. Spencer Creek between Sutters Mill Road and Boone Hills Drive (IP-7369-1-1)

This stream stability and flooding project received a priority score of 402 points and an estimated project cost of $3,186,000.

Existing Description

This project reach is along the main channel of Spencer Creek between Sutters Mill Road and Boone Hills Drive. There is a section in the middle of the reach, close to Showboat Circle cul-de-sac, where concrete chunks have been used as toe protection and the banks have successfully vegetated (although vegetation was not diverse). The reach upstream of this stabilized area showed evidence of significant bank erosion and debris jams which should be addressed to reduce sediment loading into the channel

Improvement Description

The proposed solution addresses flooding by replacing the existing culvert array at Boone Hills Drive with 4 12x15 boxes and filling floodpain areas just north of Showboat Circle Drive on both sides of channel to raise floodplain grade by 2 feet. This project also includes imited stream stabilization approaches are included along the upstream 2414 linear feet of this project reach. Four rock checks are included to stabilize the bed profile of the channel. Riparian renovation is included in the entire project area (3597 linear feet).

9. Spencer Creek in Millwood Subdivision (IP-7368-4-3)

This stream stability and flooding project received a priority score of 380 points and an estimated project cost of $ 520,000.

Existing Description

This project reach is along both the main channel of Spencer Creek and the Millwood tributary in the Millwood subdivision. Minor flooding, stream stability, debris jams, and backyard erosion were documented in the study. A significant amount of riprap is already present in the main channel of Spencer Creek, however in localized areas bank protection is needed (such as the high bank threatening the existing trail along Spencer Creek).

Improvement Description

The proposed solution addresses flooding of 2 residences by adding fill in the floodplain downstream of Millwood Drive and completing channel improvements along Spencer Creek. Additionally, IP-7368-2-1, a project that includes improvements to Burning Leaf Drive, must be completed to address flooding. Limited stream stabilization approaches are included along 25% of the Spencer Creek main channel (the most upstream reach within project boundary) and riparian renovation is included in the entire project area. Seven rock checks are included to stabilize the bed profile. Upstream regional detention was evaluated as an alternative, however, it was determined that insufficient area was available to add a level of detention that would significantly reduce the 100-year water surface elevation.

10. Boone Hills Drive and Jungermann Road (IP-7369-2-1)

This stream stability and detention project received a priority score of 375 points and an estimated project cost of $ 492,000.

Existing Description

This project area includes a stream reach along East Spencer Branch, existing detention at DB-7369-11, and a proposed detention location. These improvements may be related to potential flooding at the intersection of Boone Hills Drive and Jungermann Road. The existing aerated wet detention basin is well landscaped and the shoreline protected with a stacked stone wall and tree buffer on the west. Very turbid water was documented during the field visit. The stream has good overall structure in the upstream section, although an excessive amount of debris associated with brush and dumping at the Tieman Property was noted. This dumping has also compromised bank stability along 400 feet of the reach. The existing corridor has a significant amount of honeysuckle brush that is preventing the growth of a diverse riparian corridor.

Improvement Description

This concept includes improvements to the existing detention basin, development of a new dry detention basin west of Jungermann, and limited channel improvements. At the existing basin, the outfall structure should be further studied with additional information about backwater influences. To operate effectively, this structure must be maintained. Debris and sediment from the upstream channel reach should be managed with a forebay structure. A vegetative buffer should be established along the street that runs adjacent to the basin along 700 linear feet. In the upstream reach, the left bank at Tieman property (approximately 400 linear feet) should be repaired with a 5-ft wall and sloped/vegetated section. Use of the wall prevents further encroachment on the property and the channel is already connected to a floodplain on the right bank. Three rock check structures are included to stabilize the bed profile of the channel. The riparian corridor should be renovated along the entire reach (950 lf). To the west of Jungermann Road behind 410 Jungermann Road, a new dry detention facility is proposed with the addition of a 70-ft long berm that raises the grade 4 feet. A new outlet structure will be replaced and tie into the existing piped system.

6.2 Top 10 Cost Benefit Projects

The following locations will provide the greatest benefit for the associated cost. Preservation projects with low cost benefit were excluded from this list. The benefit is assumed to be correlated with the priority score.

1. Spencer Rd. Storage (IP-7270-3-1)

This flooding project received a priority score of 319 points and an estimated project cost of $22,000.

Existing Description

This project includes the downstream portion of the Spencer Place Tributary, located between Spencer Road and the confluence with main Spencer Creek in the Avemco subdivision. Three storage structures are located within the area of inundation during the 100-year event in Spencer Creek.

Improvement Description

Construct a 300-foot long berm, 2-4 feet tall around the northern and eastern sides of the storage facilities, bringing the elevation up to 462-ft.

2. Magnolia Manor (IP-7467-4-1)

This stream stability project received a priority score of 245 points and an estimated project cost of $37,000.

Existing Description

This project area includes a reach directly west of Magnolia Manor Court. The field notes documented headcutting at the 90 degree bend behind the residence at 1343 Magnolia Manor Court. A twin CMP discharge in the right bank may have been abandoned. Invasive vegetation was present in this channel and preventing the establishment of a diverse corridor.

Improvement Description

This concept includes 5 vertical grade control structures and renovation of the riparian corridor. There will be significant grading and clearing to establish a stable slope and remove debris in the channel.

3. Athens Drive (IP-7168-1-1)

This stream stability project received a priority score of 320 points and an estimated project cost of $81,000.

Existing Description

This existing channel, in Pegasus Farms, is formed by a pipe outlet from Morningside Drive. This pipe discharges to a steep ravine and re-enters a piped system to Kenworth Drive. This very steep channel has an average slope of 0.054 ft/ft with bank heights up to 12 feet. Active bank erosion and dumping of lawn waste was noted in the upstream portion of this reach. The existing woodland corridor should be preserved on this upstream portion. On the downstream portion of the project reach, the corridor is lacking. Access for construction may be difficult because the channel is located in backyards and there are not street crossings in the project area.

IMPROVEMENT DESCRIPTION

This concept utilizes a series of 13 rock checks to stabilize the invert elevation of the channel. The upstream corridor (200 feet) should be preserved and a downstream corridor, along 315 linear feet, should be enhanced. Large debris should be removed from the channel (shopping cart, old truck).

4. Koenig Orchard (IP-7370-3-1)

This stream stability project received a priority score of 170 points and an estimated project cost of $54,000.

EXISTING DESCRIPTION

The existing reach of East Spencer Creek/Koenig Tributary was characterized as having a bedrock bottom, some bank cutting, and good rock material on most banks. The channel is located in an agricultural area without buffers and significant invasive vegetation in the riparian corridor was documented during field reconnaissance.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor. Additionally, a vegetated filter strip should be added to all agricultural buffers that are adjacent to this reach. Debris on the culvert face should be removed during typical maintenance activities.

5. Burning Leaf Drive (IP-7368-2-1)

This flooding project received a priority score of 795 points and an estimated project cost of $258,000 and is also listed as a top 10 priority project.

6. Bruns Place (IP-7068-2-1)

This stream stability project received a priority score of 245 points and an estimated project cost of $89,000.

EXISTING DESCRIPTION

This stream reach extends through a neighborhood at Bruns Place Court, under Mid Rivers Mall Drive and then confluences with Dardenne Creek. The channel has a very steep profile in the reach upstream of Mid Rivers Mall Drive, dropping approximately 20 feet over a distance of 850 feet. The channel transitions to a braided channel before flowing to Dardenne Creek. A section of the channel has been piped through the backyard of the residence at 109 Bruns Place Court - the pipe entrance has become blocked and is therefore not functional.

IMPROVEMENT DESCRIPTION

The proposed concept includes the installation of grade control structures and riparian renovation of the corridor. The piped system at 109 Bruns Place Court should have the inlet structure replaced to allow appropriate drainage of the upstream channel. No additional bank stabilization is recommended at this time. The downstream end of the tributary should be stabilized with a series of vertical grade controls as well to prevent further degradation and incision.

7. Applewood (IP-7369-1-2)

This detention project received a priority score of 209 points and an estimated project cost of $98,000.

EXISTING DESCRIPTION

This existing wet detention DB-7369-04, located in the Applewood subdivision, is south of Bartley Street. The basin does not have sufficient capacity and is overtopping Bartley Street.

IMPROVEMENT DESCRIPTION

The concept recommendation includes replacement of the outfall structure, implementing a shoreline buffer of vegetation and riprap, and increasing capacity by excavating an additional 2 feet from the bed of the basin. The basin has borderline 25-year capacity and this additional storage will prevent street flooding.

8. Kelly Leaf (IP-7368-2-3)

This stream stability project received a priority score of 95 points and an estimated project cost of $49,000.

EXISTING DESCRIPTION

This stream reach along the Tanglewood tributary was characterized as slightly incised with sharp meanders, significant debris jams, and good cobble bars. The buffer on the left bank was documented to be in good condition.

IMPROVEMENT DESCRIPTION

The recommendation for this project reach is to preserve the corridor through riparian renovation. Six rock checks are included for vertical grade control.

9. Calwood (IP-6970-1-1)

This stream stability project received a priority score of 137 points and an estimated project cost of $52,000.

EXISTING DESCRIPTION

The existing channel reach is a small tributary within the Dardenne Creek watershed. During the field visit, excessive debris in the channel and invasive vegetation on the banks were noted. The channel profile is too steep for the bed materials to establish equilibrium. During high events, flow in this channel may be influenced by Dardenne Creek.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor. Debris jams should be removed. Additionally, 2 rock checks should be installed to stabilize the invert elevation.

10. Bella Vista Subdivision and MC-1 Stream (IP-7367-2-1)

This detention and stream stability project received a priority score of 248 points and an estimated project cost of $156,000.

EXISTING DESCRIPTION

This project area, located south of McClay Road in the Bella Vista subdivision includes an existing wet detention basin, DB-7367-20, and a channel reach that was modified during a previous stabilization project. The channel area is impacted by the upstream detention basin and very turbid water was documented in the field.

IMPROVEMENT DESCRIPTION

This concept includes improvements to the detention basin and riparian renovation of the stream corridor with some spot fixes for areas with erosion. Wetland plantings and a vegetated buffer should be established around the perimeter of the detention basin. A forebay structure should be added to control sediment in runoff from the contributing drainage area which is currently under development. Additional volume should be considered to prevent overtopping.

6.3 Top 10 Preservation Projects

The following locations will provide the greatest preservation benefit for the associated cost.

1. Spanish Trails (IP-6870-1-1)

This preservation project received a priority score of 95 points and an estimated project cost of $3,000.

EXISTING DESCRIPTION

The existing channel has good structure but too much shade inhibits understory growth. Riparian corridor should be restored.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor.

2. Trailside Court, Villages of Windwood (IP-7467-3-2)

This preservation project received a priority score of 95 points and an estimated project cost of $6,000.

EXISTING DESCRIPTION

This existing reach showed evidence of polluted water and is too shaded for a healthy riparian corridor.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor.

3. Laurel Park Stream, McClay Valley Boulevard (IP-7468-3-1)

This preservation project received a priority score of 95 points and an estimated project cost of $8,000.

EXISTING DESCRIPTION

The existing channel has very good structure but the presence of honeysuckle and other invasive vegetation needs to be addressed.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor.

4. Crescent Hills (IP-7368-1-2)

This preservation project received a priority score of 95 points and an estimated project cost of $11,000.

EXISTING DESCRIPTION

This reach of the Crescent Hills Tributary was characterized with good structure and vegetation.

IMPROVEMENT DESCRIPTION

The recommendation in this project reach is to preserve the existing corridor through riparian renovation (1313 linear feet).

5. Ohmes Mitigation (IP-6969-1-1)

This preservation project received a priority score of 95 points and an estimated project cost of $20,000.

EXISTING DESCRIPTION

This reach of stream, located adjacent to the mitigation site for the Ohmes Farm Development, has a good riffle pool structure but significant amount of honeysuckle that should be managed.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor.

6. McClay Valley /Woodstream (IP-7468-3-2)

This preservation project received a priority score of 95 points and an estimated project cost of $17,000.

EXISTING DESCRIPTION

This existing channel was identified as a potential linkage between two channels in good condition.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor.

7. Harvestowne (IP-7166-2-1)

This preservation project received a priority score of 75 points and an estimated project cost of $7,000.

EXISTING DESCRIPTION

The existing channel, south of Hwy 94, has very good structure, with stable sloped banks and a diverse riparian corridor but the presence of invasive vegetation should be addressed.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor. The buffer on the east side should be enhanced.

8. Spencer Crossing (IP-7269-2-1)

This preservation project received a priority score of 75 points and an estimated project cost of $23,000.

EXISTING DESCRIPTION

This reach of the Oak Creek Park Tributary was documented as a meandering sandy, cobble bed channel with bank cutting around bends, scour under TRM, and riprap on some bank toes.

IMPROVEMENT DESCRIPTION

Riparian renovation is recommended for this stream corridor.

9. Country Crossing (IP-6868-2-1)

This preservation project received a priority score of 75points and an estimated project cost of $26,000.

EXISTING DESCRIPTION

The existing channel has very good structure but the presence of honeysuckle and other invasive vegetation needs to be addressed.

IMPROVEMENT DESCRIPTION

Riparian Renovation suggested for stream corridor (1602 linear feet).

10. Millwood (IP-7368-1-1)

This preservation project received a priority score of 75 points and an estimated project cost of $46,000.

EXISTING DESCRIPTION

This project includes the preservation and enhancement of a high-quality (SAI Type 1 and 2) riparian corridor on Spencer Creek between Jungermann Road and Millbrook Court/Mill Run Lane. The project intent is to maintain the corridor's existing health and function, and to demonstrate how vegetative plantings on residential properties that border riparian corridors can protect and enhance stream health.

IMPROVEMENT DESCRIPTION

The concept is to remove invasive species such as shrub honeysuckle in up to 20 acres of riparian corridor, and to re-vegetate these areas with appropriate riparian trees, forbs and grasses. Doing so will help ensure that these invasive species don't get a foothold and degrade this otherwise high-quality stream corridor. In addition, residential properties border the riparian corridor on the northern side of the stream along a half-mile reach from Jungermann Drive to Millwood Drive; the concept would be to plant an appropriate, 25-foot vegetated buffer along most or all of this reach. The plantings would

consist of native shrubs, forbs and grasses that would form an attractive, defined edge between residential lawns and the stream corridor.

6.4 General Detention Basin Recommendations

6.4.1 Detention Basin Management Program

The 248 detention basins in the City have a significant impact on stormwater management reducing peak flows and improving runoff water quality. However, many of these are not maintained properly and therefore do not reduce peak flows and improve water quality as designed and originally constructed. Therefore, the City is developing a plan to manage these basins so they function properly and provide benefits to the community. A prioritized approach has been developed for the Detention Basin Management Program.

One of the most important considerations in management of the basins is the annual costs the City will incur. General operations and maintenance (O&M) costs have been estimated from the Water Environment Research Foundation's "Performance and Whole-Life Costs of Best Management Practices (BMPs) and Sustainable Urban Drainage Systems (SUDS)" (WERF Report). The WERF Report examined BMP performance and costs primarily in the US and the UK and created a whole-life cost spreadsheet that can be used to estimate O&M costs. Many assumptions are made in the spreadsheet and the most significant is the level of maintenance provided with choices of low, medium, or high. For the purposes of this project, a medium level of maintenance was chosen. This choice then defines the frequency of the following:

- Inspection of facilities – once every three years
- vegetation management – once a year
- vector control – once every three years
- intermittent maintenance – once a year
- sediment removal – once every ten years

The Detention Basin Management Plan includes a prioritization of the basins for acquiring them and taking over responsibility for O&M. The highest priority basins are the 57 included in the Capital Improvements Program CIP. When they are upgraded as recommended in the CIP, the City will consider management of these basins. The estimated annual O&M cost for these basins is approximately $110,000. These include five basins currently owned by the City of St. Peters.

Seven basins owned by the City that are not included in the CIP are also in the group of highest priority basins. The estimated annual operations and maintenance cost for these basins is approximately $36,000.

TABLE 178. DETENTION BASINS OWNED BY THE CITY

Basin	Location	Approximate Drainage Area (*acres)*
DB-6870-04	SPANISH TRAILS	3
DB-6871-01	I-70 TRADE CENTER	33
DB-6969-02	WOODLANDS SPORK PARK	16
DB-6969-04	WOODLANDS SPORT PARK	27
DB-7169-05	COMMUNITY & ARTS CENTER	2
DB-7171-01	CITY OF ST PETERS	875
DB-7467-06	HERITAGE MANOR	13

The 68 residential basins are the next priority. The basins are prioritized in Table 18 by their type with wet basins having a higher priority than dry basins because they provide more water quality benefit. The drainage area served is also used to prioritize the basins with the larger tributary areas having a higher priority. The estimated annual cost for O&M for these basins is approximately $142,000.

TABLE 1918. RESIDENTIAL DETENTION BASINS

Basin	Location	Type	Approximate Drainage Area *(acres)*
DB-7468-06	LAUREL VILLAGE	WET	167
DB-7066-03	MEADOWRIDGE	WET	152
DB-6968-03	COUNTRY CROSSING ESTATES	WET	77
DB-6869-02	FAWN LAKE	WET	40
DB-7269-03	HICKORY RIDGE	WET	19
DB-6970-03	LAKES OF DEVONDALE	WET	17
DB-7069-08	LAKES OF DEVONDALE	WET	12
DB-7369-05	CAP-AU-GRIS	WET	7
DB-7169-03	MID RIVERS CHRISTIAN CHURCH	WET	5
DB-7268-05	COUNTRY CREEK	WET	4
DB-6870-11	RICHMOND	WET	2
DB-7269-02	HICKORY RIDGE	DRY	124
DB-6870-01	ESTATES OF CHATEAU WOODS	DRY	62
DB-7467-11	TRIANGLE APARTMENTS	DRY	59
DB-7168-03	PEGASUS FARMS	DRY	52
DB-7065-06	THE HIGHLANDS	DRY	43
DB-7270-02	VANDERBILT APARTMENTS	DRY	37
DB-6869-01	HOLLOW CREEK	DRY	29
DB-7267-02	PARK RIDGE ESTATES	DRY	27

Basin	Location	Type	Approximate Drainage Area *(acres)*
DB-7370-09	TWILL VALLEY	DRY	26
DB-7467-01	MC CLAY MEADOWS	DRY	19
DB-7467-07	REGENCY ESTATES	DRY	18
DB-7168-02	SPRING VALLEY WAY	DRY	12
DB-7367-09	SUGARWOOD	DRY	12
DB-6968-04	COUNTRY CROSSING ESTATES	DRY	12
DB-7468-03	MC CLAY VALLEY	DRY	11
DB-7268-03	HANOVER CROSSING	DRY	11
DB-6870-07	FORT ZUMWALT SOUTH	DRY	9
DB-7268-01	HANOVER CROSSING	DRY	9
DB-7468-02	FRANCIS HOWELL SCHOOL DISTRICT	DRY	8
DB-7368-03	BRIARWICK	DRY	8
DB-7066-02	WHITTINGTON PLACE	DRY	8
DB-6868-01	ASPEN RIDGE	DRY	8
DB-6870-08	CANDLEWICK ESTATES	DRY	7
DB-7069-01	BUCHHOLZ	DRY	7
DB-7270-04	SPENCER HILL	DRY	7
DB-6870-13	DEER RIDGE	DRY	6
DB-7367-05	SUGARWOOD	DRY	5
DB-7369-10	CHAPEL OF THE CROSS LUTHERAN CHURCH	DRY	5
DB-7367-02	HARVEST POINT	DRY	5
DB-7267-01	FAIRFIELD PLACE	DRY	5
DB-7468-04	MC CLAY VALLEY	DRY	5
DB-7267-12	ST JOACHIM & ANN CATH SCHOOL	DRY	4
DB-7367-13	LIENEMANN FOREST	DRY	4
DB-7268-02	HANOVER CROSSING	DRY	4
DB-7367-08	HARVEST POINT	DRY	4
DB-7468-05	FRANCIS HOWELL SCHOOL DISTRICT	DRY	3
DB-7267-15	CALVARY TEMPLE ASMBLY OF GOD	DRY	3
DB-6870-03	BELLEAU CREEK ESTATES	DRY	3
DB-7367-03	HEATHER RIDGE	DRY	3
DB-6970-01	DARDENNE LAKE ESTATES PLAT 1	DRY	3
DB-6870-02	ESTATES OF CHATEAU WOODS	DRY	3
DB-7170-01	LEWIS & CLARK ELEMENTARY SCHOOL	DRY	2
DB-7368-07	COUNTRY MANOR	DRY	2
DB-6870-05	BROOKWOOD ESTATES	DRY	2
DB-7467-02	MC CLAY MEADOWS	DRY	2
DB-7366-02	CANYON CREEK	DRY	2

Basin	Location	Type	Approximate Drainage Area *(acres)*
DB-7070-04	FORT ZUMWALT PLACE	DRY	1
DB-7070-06	FORT ZUMWALT PLACE	DRY	1
DB-7367-14	SUMMERGATE TOWNHOMES	DRY	1
DB-7366-05	PARKWAY VILLAS	DRY	1
DB-7369-09	CHAPEL OF THE CROSS LUTHERAN CHURCH	DRY	1
DB-6870-06	MID RIVERS SEVENTH DAY ADVENTIST	DRY	1
DB-6870-12	CANTERBURY PARK	DRY	1
DB-7367-06	HEATHER RIDGE	DRY	1
DB-7070-01	OLD TOWN SOUTH	DRY	1
DB-6970-02	FIRE STATION	DRY	1

The remaining 116 basins are in commercial or industrial areas. The maintenance of these should be provided by their owner. Therefore, it is recommended that the City not take over O&M of these basins and ensure their performance through inspection and enforcement of existing ordinances requiring maintenance by the owner.

The O&M costs of the 57 basins included in the CIP and the seven basins owned by the City totals approximately $146,000. The schedule for taking over ownership and O&M of the 68 residential basins will be determined by available funding and staff resources.

6.4.2 Low-flow Flume Replacement

Many detention basins in the City include concrete flumes that convey low-flows from the discharge pipes to the basin outlet. These eliminate most wet areas and erosion in the bottom of grass-lined basins; however they convey the first flush of runoff through the basin with no storage time or exposure to the vegetation. This allows what is typically the most polluted runoff to move through the basin with no treatment or attenuation. The following recommendations are made to upgrade the flumes so they provide some water quality benefit while not impacting the detention basin's peak flow control performance significantly.

Intact Flumes

Flumes that are not in need of structural repair do not need to be replaced to provide some water quality benefit. A small berm made of selected rip-rap can be placed across the flume in the vicinity of the outlet structure. The berm will cause water to pond allowing it to spread out into the vegetated area of the basin. This will slow the water down allowing sediment to settle and for some of the water to be exposed to the vegetation allowing further removal of sediments and other water quality benefits. The berm should be permeable or have a low flow outlet to allow the ponded water to drain within 40 hours. Provisions should be made to allow sediment removal and the basins should be inspected once a year to identify maintenance needs and ensure that small semi-permanent pools are not being created.

The size of the rip-rap should be determined based on anticipated velocities in the flume. Also, the need for some type of stabilization in the vegetated area should be evaluated based on anticipated velocities and visual inspection.

A pilot program should be initiated to test the performance of the berm in trapping sediment and draining within 40 hours. A few basins should be retrofitted with the berms and inspected after storm events to monitor their performance and identify adjustments needed for satisfactory performance.

Flumes Needing Repair

For flumes needing concrete repair, it is recommended that the concrete sections be removed and replaced with rip-rap. The rip-rap should extend from the area needing repair to the low flow outlet. A berm should be placed near the outlet as described above. The rip-rap should be planted with appropriate plants as listed in Appendix F. The rip-rapped section will slow the water down allowing it to pond and trap sediment. Provisions should be made to allow sediment removal and the basins should be inspected once a year to identify maintenance needs and ensure that small semi-permanent pools are not being created.

The size of the rip-rap should be determined based on anticipated velocities in the flume. Also, the need for some type of stabilization in the vegetated area should be evaluated based on anticipated velocities and visual inspection.

A pilot program should be initiated to test the performance of the rip-rapped section and berm in trapping sediment and draining within 40 hours. A few basins should be retrofitted and inspected after storm events to monitor their performance and identify adjustments needed for satisfactory performance.

Flumes Needing Replacement

The concrete should be completely removed and replaced with a rip-rap channel. If possible, the channel should be designed with meanders to further slow the water down thereby trapping more sediment. The rip-rap should be planted with appropriate plants as listed in Appendix F. Provisions should be made to allow sediment removal and the basins should be inspected once a year to identify maintenance needs and ensure that small semi-permanent pools are not being created.

The size of the rip-rap should be determined based on anticipated velocities in the flume. Also, the need for some type of stabilization in the vegetated area should be evaluated based on anticipated velocities and visual inspection.

A pilot program should be initiated to test the performance of the rip-rapped channel in trapping sediment and draining within 40 hours. A few basins should be retrofitted and inspected after storm events to monitor their performance and identify adjustments needed for satisfactory performance.

6.4.3 Forebays

New detention basins should be designed with a forebay to capture sediment and trash from all inlets to the basin. The forebay should be sized to accumulate 5-years of sediment before requiring sediment

removal. A maintenance ramp and access should be provided to allow the required equipment to be used for forebay maintenance (MARC Manual). Forebays should be inspected annually to ensure they drain properly and that small pools are not being created.

6.4.4 Basin Retrofits

The 57 basins recommended for upgrades in the CIP include provisions for adding forebays. As the City considers management of additional basins as part of the Detention Basin Management Program, the basins should be evaluated for the presence of a forebay or whether one can be added over time. The rip-rap dam recommended in the Flumes discussion above creates a sediment trap and in essence creates a forebay. This is a low-cost way to provide sediment removal and water quality benefit if the addition of a dedicated forebay is not feasible.

6.4.5 Detention Basin Location

Detention basins have been shown to be effective in providing water quality benefit and to reduce peak flows in the portion of the City south of I-70. Additionally, those basins located in areas protected by levees north of I-70 are effective at providing storage areas for subsequent pumping when the Mississippi River is in flood stage. For areas north of I-70 not protected by levees, the area within the City is flood prone from Mississippi River, Dardenne Creek and Spencer Creek flooding. Therefore, these areas are not going to benefit as much from local detention and can be excluded from the detention basin requirements.

7.0 References

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Appendix

To see full plan with Appendices, go online to http://www.stpetersmo.net/storm-water-management.aspx.