Appendix A
Hydrology and Hydraulics (H&H)

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1.1 HYDRAULIC MODELING SUMMARY

Hydrologic simulations used in this study were conducted using PCSWMM 7.2 for the hydrology and HEC-RAS 5.0.7 for the hydraulics. Using a 2020 flood event, existing models were recalibrated, and the flood of record results were verified for accuracy.

The PCSWMM model extents encompass the entire River Des Peres watershed. The model includes both the open channel hydraulics mixed with a large number of closed conduits, combined sewers with overflow, and flow splits throughout the River Des Peres watershed. Specific focus was provided to the hydrology in the upper River Des Peres.

The modeling extents for the HEC-RAS model start upstream at Warson Road in Olivette, MO. The reach flows downstream until it reaches the entrance of the underground sewerage system in the areas between Vernon and Dartmouth Avenues in University City, MO. The start of the underground network is referred to as the River Des Peres "Tubes". For the purposes of this project, PCSWMM will be used to generate the anticipated HEC-RAS flow input.

The existing condition model problem areas were compared against conditions documented in the prior USACE studies for the upper River Des Peres in University City. The model results are presented using frequency rainfall events and the resulting river levels/depth grids on the River Des Peres in University City, MO.

Throughout this report and in this appendix, flood events and their resultant inundation will be referred to by Annual Exceedance Probability (AEP), which is the probability that this level of flooding may be realized or exceeded in any given year. For example, a flood event with a 1% AEP would have a 1% probability of occurring every year. This is a change in terminology from the recent commonly used term "annual chance of exceedance" (ACE). Additionally, in the past, flood events have often been described by their "return period" – or the estimated average length of time between flood events of a similar magnitude. A 1% AEP event would have been referred to as having a 100-year return period or being a 100-year event. This terminology is no longer used because it falsely conveys a sense of time and lowers public risk perceptions. Table 1 provides a list of AEP flooding events that were considered during the study, with their equivalent "return period." It is important to note that all AEP references in this report are for expected water levels, not the AEP of meteorological events (i.e. a 1% flood event is not the same as, nor does it necessarily occur as a result of, a 1% storm event).

Table 1. Comparison of AEP, ACE, and Return Period Terminology

AEP/ACE	Return Period ("x-year flood")*
20%	5-year
10%	10-year
4%	25-year
2%	50-year
1.3%	75-year
1%	100-year
0.5%	200-year
0.2%	500-year
*Note: Return Period is a term that can be misleading, is often misunderstood, and is no longer used by USACE (see ER 1110-2-1450).	

1.2 EXISTING MODELS

The models used in this study were recently assembled from a Zone AE designated streams hydrology study prepared for the Missouri State Emergency Management Agency by Wood Environment and Infrastructure Solutions (June 2017). The study analyzed several watersheds in the Cahokia North Watershed. Particular to this project study area, the River Des Peres watershed hydrology was analyzed using PCSWMM. The watershed and pipe network geometry of the River Des Peres watershed is illustrated in Figure 1. PCSWMM simplifies the river system by conduits and junctions with transverse elements representing surface junction overflow.

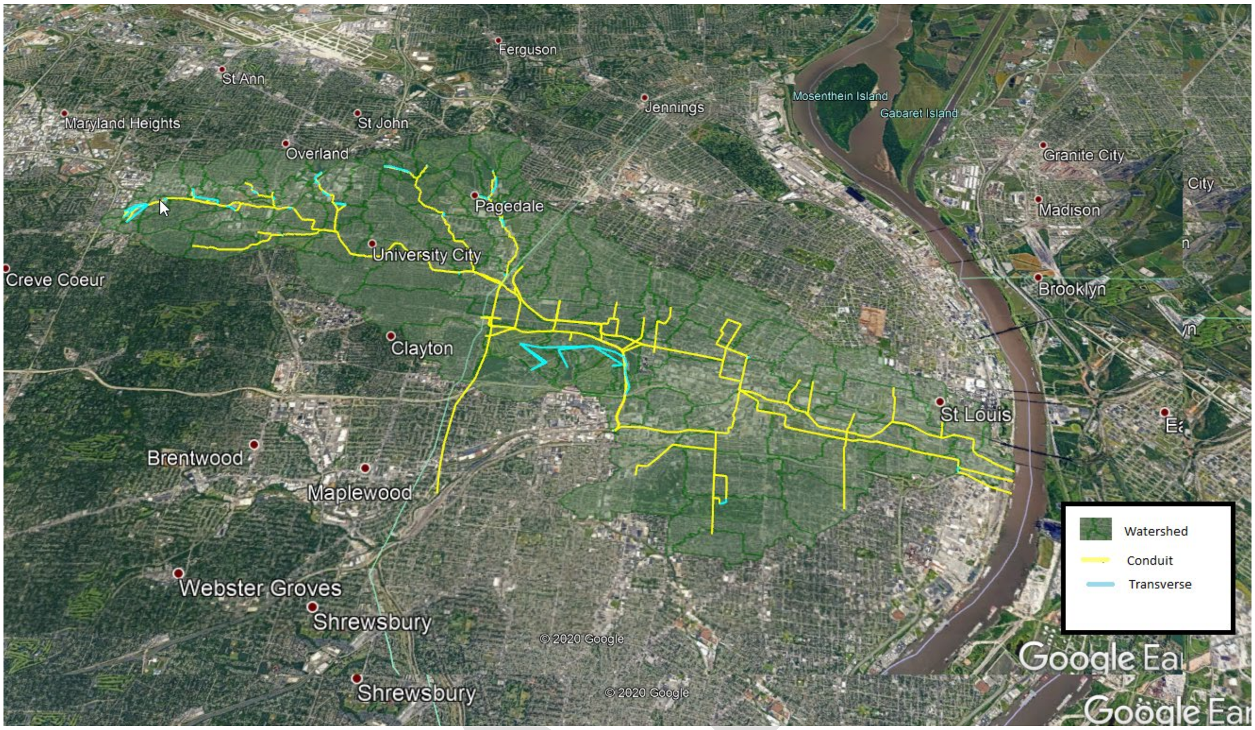

Figure 1 - PCSWMM Geometry of the Upper River Des Peres Watershed

In tandem with the Cahokia North hydrologic analysis, Wood Environment and Infrastructure Solutions created or updated several hydraulic models for a FEMA FIS update of St. Louis County, Missouri. The model used to capture the University City branch of the River Des Peres was constructed using HEC-RAS. The HEC-RAS cross-section geometry of the University City branch study reach is illustrated in Figure 2.

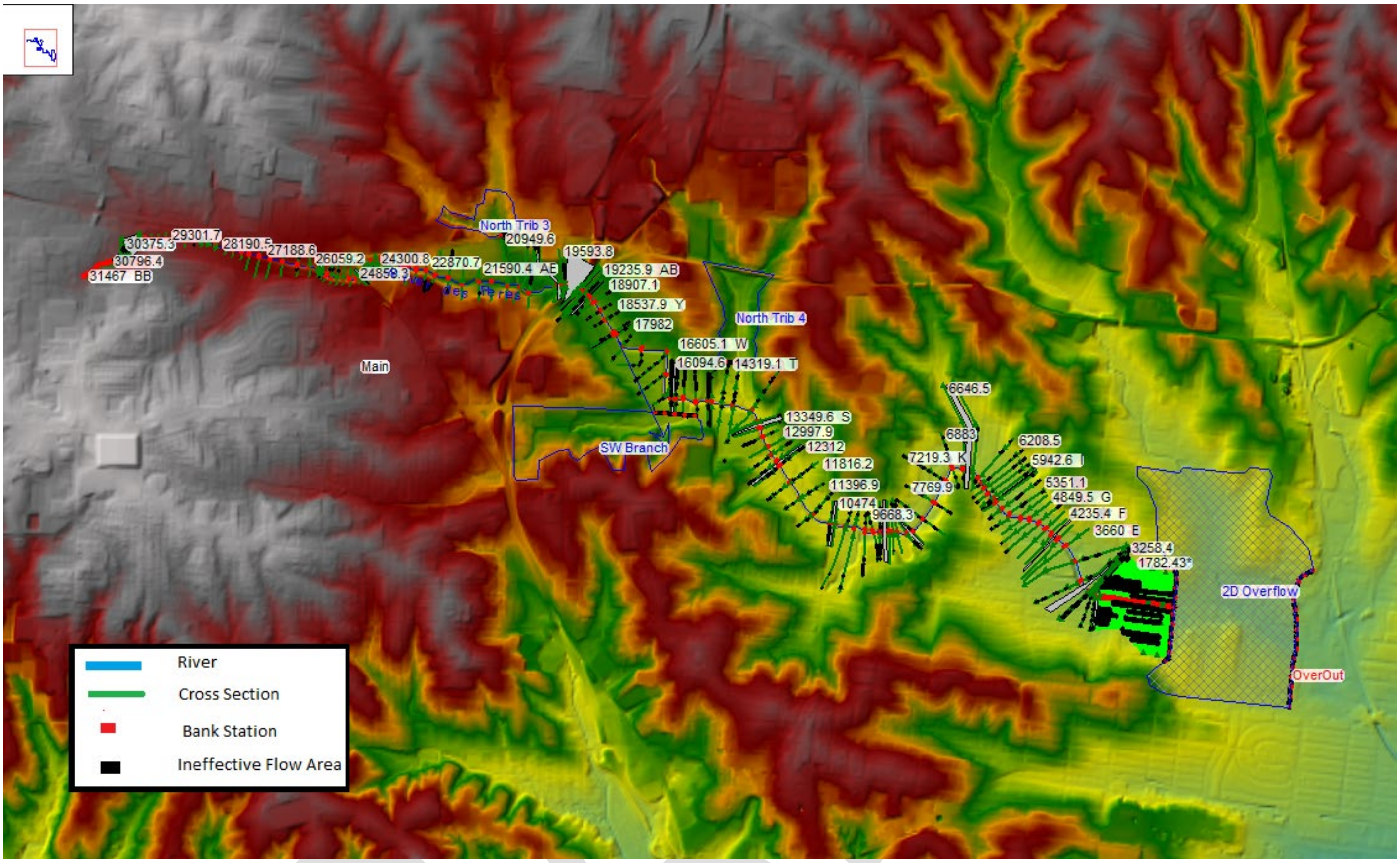

Figure 2 - HEC-RAS Model of the University City Branch of the River Des Peres

1.3 PCSWMM CALIBRATION AND VALIDATION

An existing PCSWMM 2D model of the River Des Peres watershed, originally built using the 5.1.011 SWMM computational engine, was imported into PCSWMM version 7.2. The model was converted to use the 5.1.013 SWMM engine. Originally calibrated to the 2008 flood of record, the model was calibrated to a recent flood event as well as re-calibrated against the 2008 flood. Like the individual watershed rainfall hyetographs used in the 2008 event simulation, NEXRAD Stage 3 radar-based rainfall hyetographs were used as rainfall input.

More specific to calibration, the PCSWMM model was used to simulate observed events that occurred on 8 August 2020 and 14 September 2008. Based on the simulation results, the model was further refined.

The SWMM subcatchment runoff block parameters include:

- Subcatchment Width
- Curve Number
- Depression Storage
- and Impervious Manning's Roughness Coefficients

These parameters were adjusted to better capture the flow and stages on the River Des Peres at University City, MO. The results of the September 2008 calibration are illustrated in Figure 3. PCSWMM August 2020 calibration results Calibration of the August 2020 flood of record is illustrated in Figure 4. PCSWMM September 2008 verification results. Goodness of fit comparisons of the peak discharges and hydrograph volumes are tabulated in Table 7.

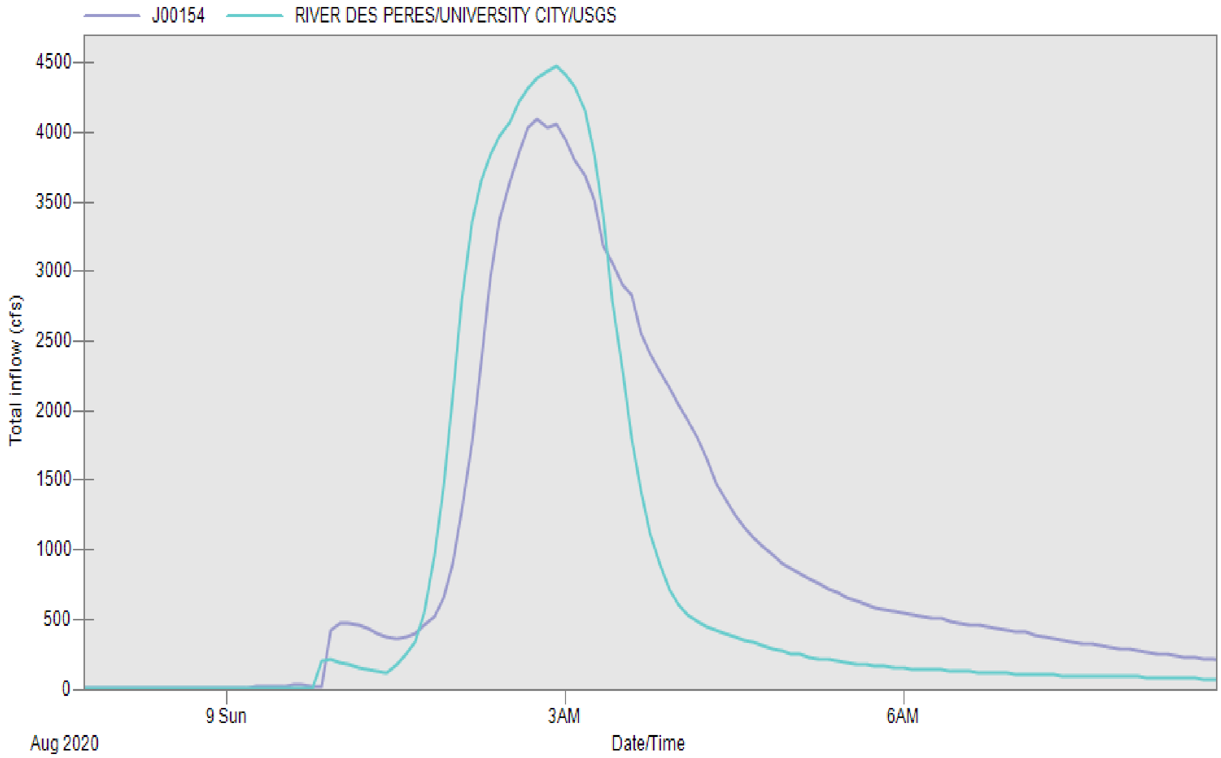

Figure 3. PCSWMM August 2020 calibration results

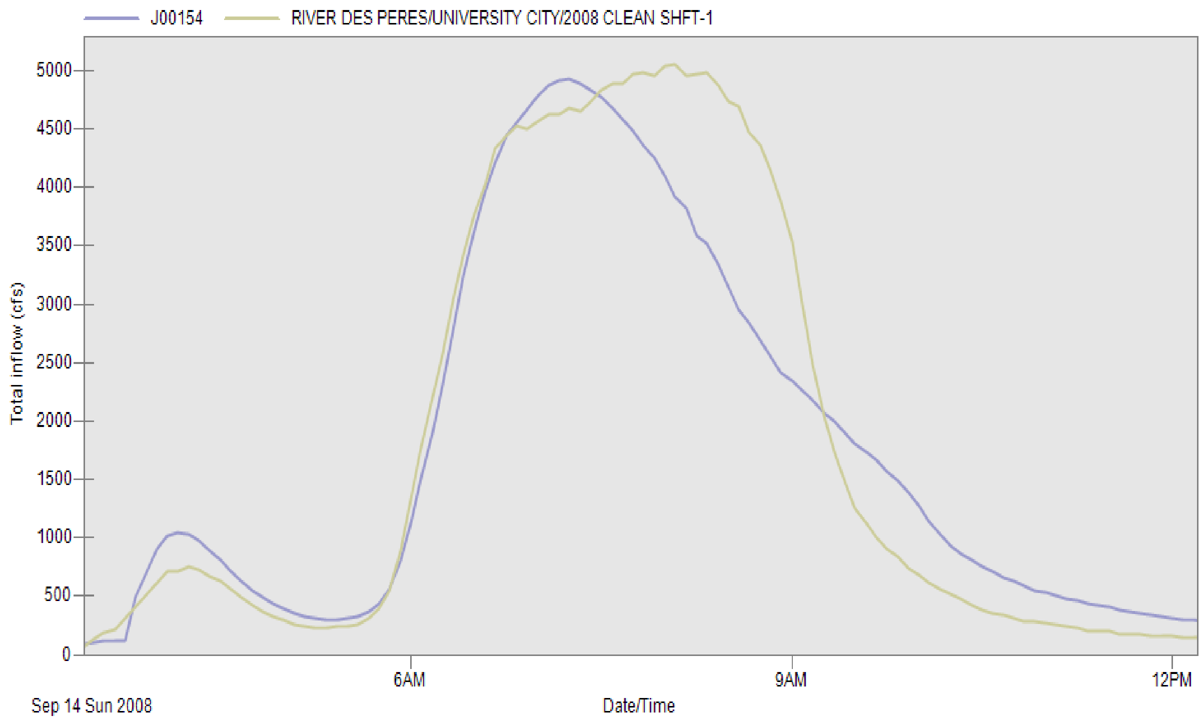

Figure 4. PCSWMM September 2008 verification results

Table 2 - Comparison of PCSWMM Simulated versus Observed Discharges

		Simulated		Observed		Differences	
Storm Event	Nash-Sutcliffe Efficiency	Volume (ac-ft)	Peak Discharge (cfs)	Volume (ac-ft)	Peak Discharge (cfs)	Volume (ac-ft)	Discharge (cfs)
September 2008	0.904	1277	4932	1333	5050	-56	-118
August 2020	0.823	800	4096	637	4480	-163	-384

Simulation of the 2020 calibration event yielded peak discharge results that were within 9% of the observed measurements at the University City, MO gage. For the September 2008 flood of record, flows were within 2% of the observed.

1.4 PCSWMM FREQUENCY ANALYSIS

The PCSWMM model was used to simulate frequency level inflows to HEC-RAS using point precipitation frequency estimates from NOAA Atlas 14. Using an SCS type-II 24 hour duration rainfall distribution, the point precipitation frequency estimate were uniformly distributed over the subbasins. No areal reduction was applied as the project watershed is less than eight square miles. For the purposes of feasibility, the 24-hour storm duration is an adequate assumption but additional analysis regarding which storm duration yields the highest runoff.

Storm duration will be reassessed in greater detail during the planning and engineering design phase of this project.

1.5 HEC-RAS MODELING

The existing HEC-RAS model of the River Des Peres watershed was built using HEC-RAS 5.0.7. Using only the HEC-RAS geometry data supplied by Wood Environment and Infrastructure Solutions, a new unsteady state HEC-RAS 5.0.7 model of the project area was created. LIDAR and aerial photographs were used to verify accuracy of the geometry. Recent changes to conditions were incorporated in the final geometry. The original channel survey gathered during the 2017 study was used for current channel conditions in the new model.

Observed and frequency inflows were computed using the PCSWMM model discussed in the previous sections. Locations used for the PCSWMM computed lateral inflows were either at the downstream end or a point of notable tributary inflow for the contributing watershed.

1.6 HEC-RAS MODEL CHALLENGES

Initial simulations of the model required a significant amount of work to improve stability. The model will need to run both high and low flows because of the flashy nature of the watershed. The model simulates numerous transitions from mild to steep slopes yielding mixed flow conditions throughout. There is significant head cutting seen on the downstream side of several bridges. This left significant elevation differences between the upstream and downstream cross-sections around the bridges. The following actions were employed to ensure stability and accuracy of the hydraulic results:

- <u>Pilot Channels</u>. Pilot channels were added throughout the model reach extents to maintain water depths through bridges with downstream channel head cuts.
- <u>Cross-section Interpolation</u>. Numerous cross-sections were interpolated near the downstream boundary at the tubes entrance. Because of the steep channel drop off as the channel descends into the Tubes, additional cross-sections were added to improve stability at higher time steps. Other interpolated cross-sections were also added to improve model stability around select bridges throughout the model.
- <u>Cross-section Extension</u>. Cross-sections were extended to ensure that the 0.2% AEP flood extents were adequately captured.
- <u>Bridge HTab Parameters</u>. Bridge HTab Parameters were set to improve computation stability through the entire simulations. Peak flows were specified to keep the model from computing errant high flows when the computed water surface hits the point where bridge overflow begins.
- <u>Bridge Energy Equations</u>. More so an issue during low flow conditions, stage-flow determination was set to compute using the Standard Step energy equations at several bridges.

- Mixed-Flow Regime. Mixed flow regime was employed to improve stability on the steep and mild channel slope transitions. Without this option, the model will not run.
- Storage Areas. Storage Areas were added to ensure that the extents of the 0.2% AEP event were adequately covered on the tributaries to the study reach.
- 2D Storage Area at Tubes. A 2D storage area was added surrounding the downstream boundary condition in a manner to ensure adequate overflow around the Tubes during the high magnitude less frequent storm events.
- Minimum Flow at Upstream Boundary. A flow minimum was used at the upstream boundary on the River Des Peres. Inflows selected were the minimum allowable that allowed for a stable simulation. As a result, calibration of the low flow stages may not be reliable.

1.7 DOWNSTREAM BOUNDARY CONDITION

The downstream boundary condition for the HEC-RAS model was defined as a rating curve. Bentley's CivilStorm was used to estimate a stage discharge relationship for the culvert segments that begin at the Tubes (Figure 5). Because of the volume of runoff and close proximity to the Engleholm Creek confluence with River Des Peres it was verified through results of the PCSWMM analysis that backwater does affect the River Des Peres at the Tubes. Further analysis shows that the backwater extent of influence primarily extends from the Tubes upstream to Pennsylvania Avenue. To account for this backwater from the tubes, PCSWMM was used to simulate the effects of backwater at the entrance to the tubes. The upper end of the curve was approximated using flows and stages within 200 cfs of the frequency event computed peak. For non-backwater conditions, the 50 percent AEP storm event was used to estimate the lower end of the curve. Graphically analyzing computed stage and flow at the Tube entrance, a new adjusted rating curve was computed to account for downstream backup at the times of peak stage and discharge (Figure 6). The culvert parameters captured in the PCSWMM model were used and adjusted to begin discharges at the HEC-RAS final channel invert just prior to the Tubes entrance.

In order to capture overflow conditions to ensure adequate overflow around the Tubes during the high-magnitude, less frequent storm events, a 2D Area was created encircling the River Des Peres reach downstream boundary. The 2D Area was extended downstream far enough to ensure that normal channel flow conditions developed before the 2D Area boundary condition outfall.

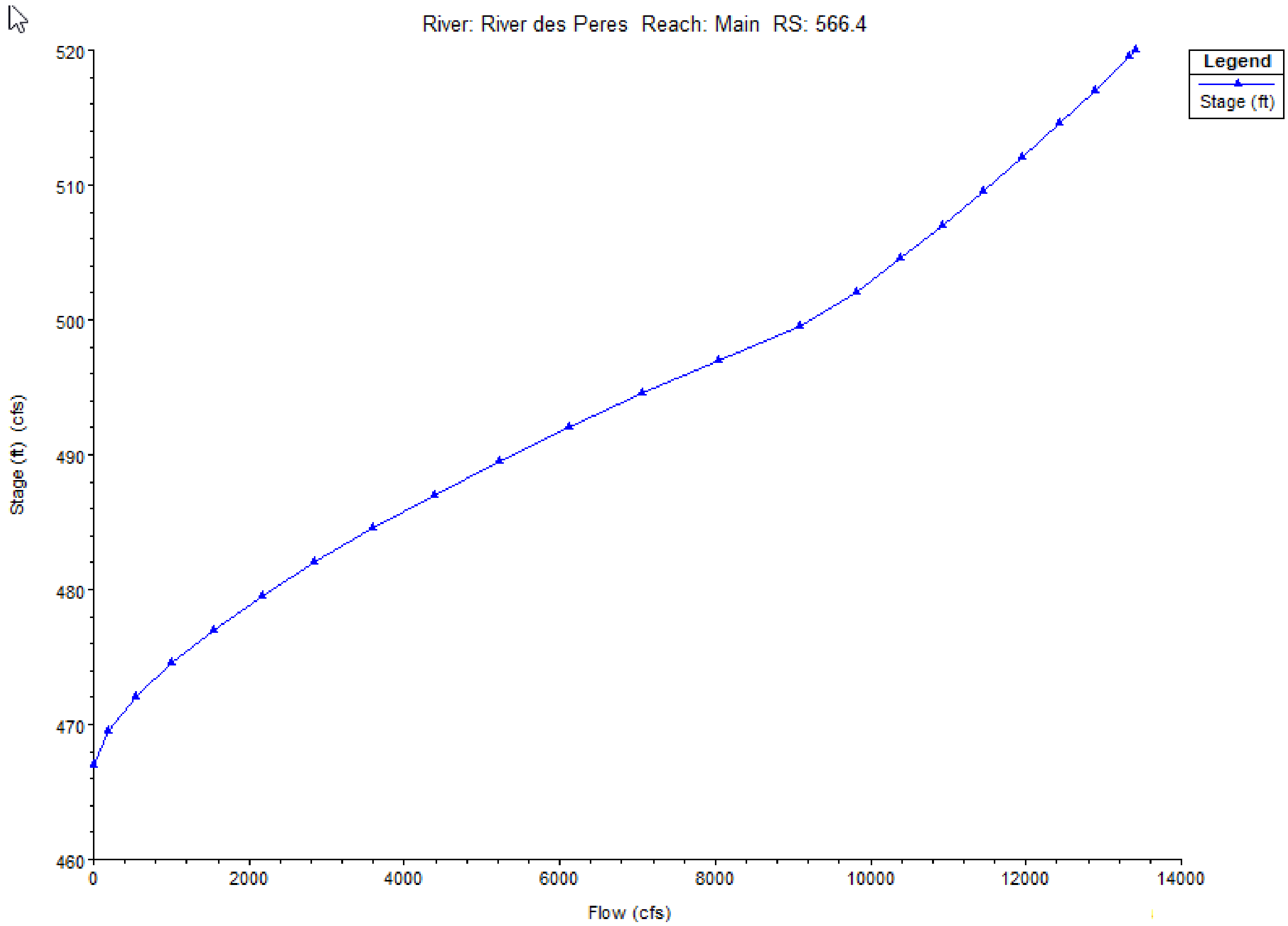

Figure 5. Downstream culvert Boundary Rating Curve Without Backwater Adjustment

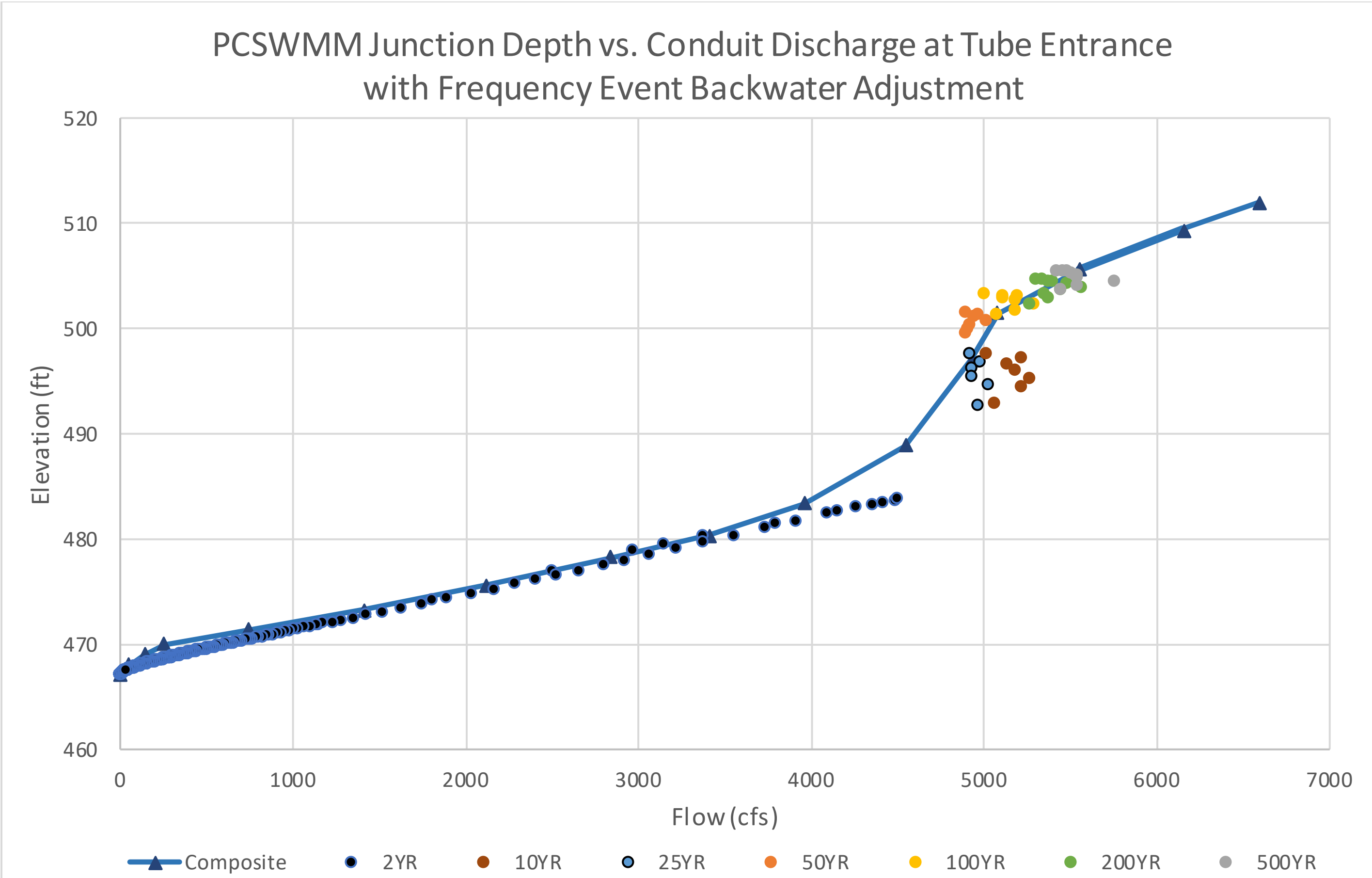

Figure 6 - Adjusted Rating Curve at Tube Entrance Accounting for Backwater Influence

1.8 HEC-RAS CALIBRATION AND VALIDATION

The HEC-RAS model was calibrated to a recent flood event as well as against the 2008 flood of record. The selected calibration event occurred on 8 August 2020. The USGS gage at University City, MO was primarily used to calibrate both stage and flow on was located on the River Des Peres. The gage is located on the foot bridge at Purdue Avenue. The corresponding cross-section on the River des Peres reach Main, is at river station 5669.3.

Based on the results, parameters such as Manning "n" values and flow roughness factors were adjusted to ensure computed stages effectively match observations. Bridge computational approach parameters were adjusted as well to further fine-tune bridge drawdown profiles. The results of the August 2020 calibration are illustrated in Figure 7. Computed versus observed water surface at River Des Peres, University City gage August 20207. Verification of the September 2008 flood of record is illustrated in Figure 8. Computed versus observed water surface at River Des Peres, University City gage September 20088. A comparison of the peak flows and stages simulated versus observed are tabulated in Table 3.

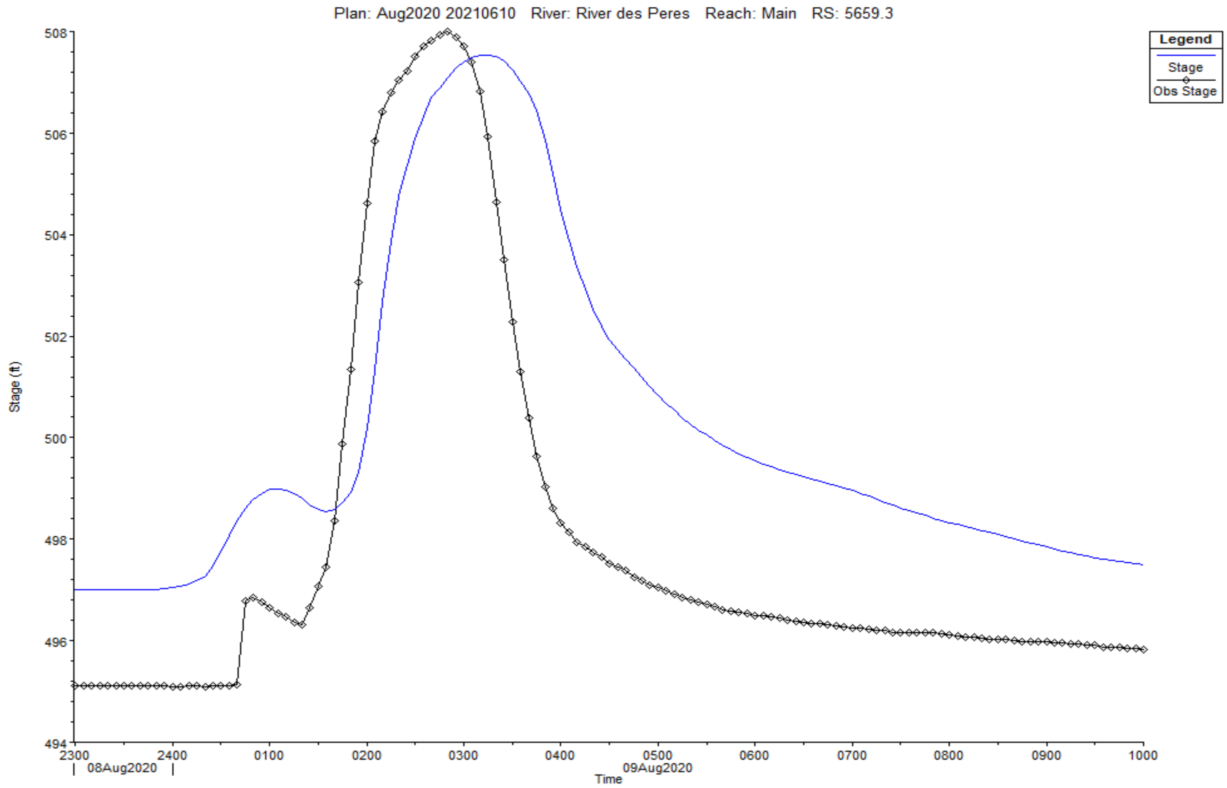

Figure 7. Computed versus observed water surface at River Des Peres, University City gage August 2020

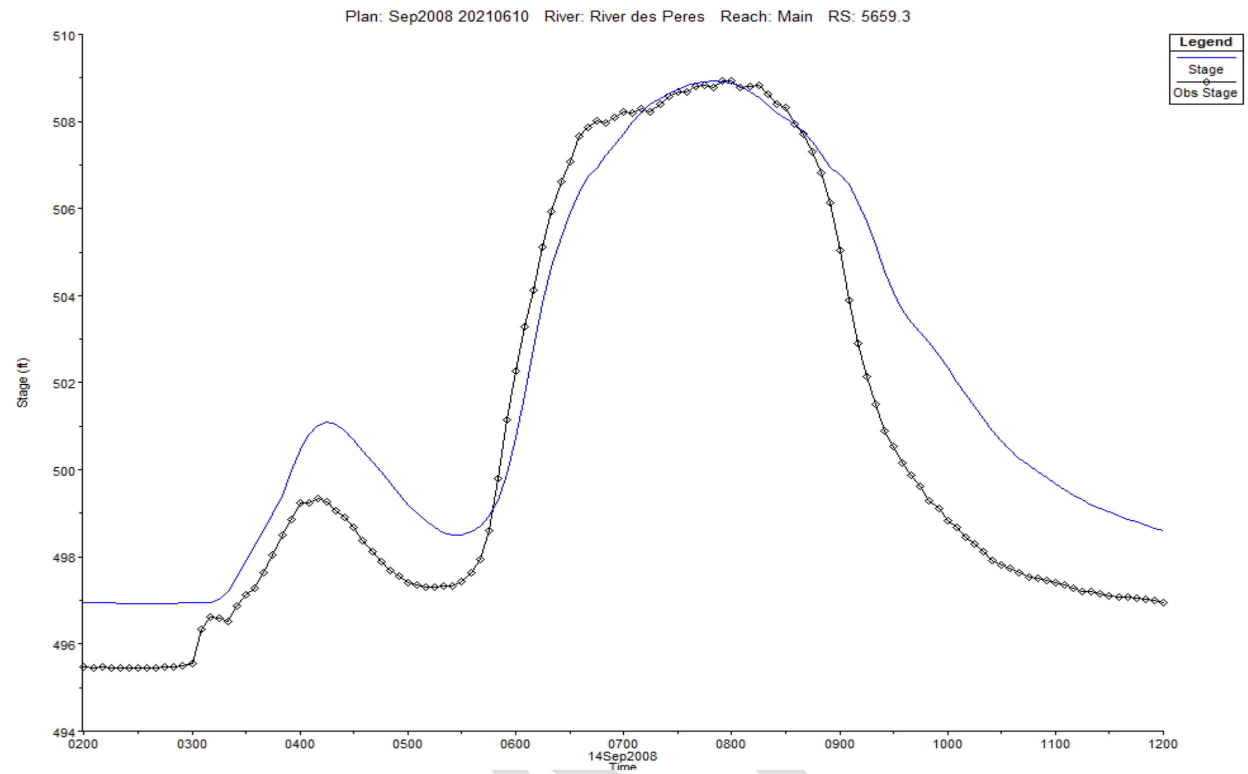

Figure 8. Computed versus observed water surface at River Des Peres, University City gage September 2008

Table 3. Computed versus observed water surface and discharge at River Des Peres, University City gage September 2008 and August 2020

	Simulated		Observed		Differences		
Storm Event	**Peak Elevation NAVD88 (ft)**	**Peak Discharge (cfs)**	**Peak Elevation NAVD88 (ft)**	**Peak Discharge (cfs)**	**Elevation (ft)**	**Discharge (cfs)**	**Volume Difference (%)**
September 2008	508.92	4973	508.94	5050	0.02	-77	2.7
August 2020	507.74	4016	508.0	4480	-0.26	-464	37.4

High-water mark information was captured after the 2008 storm event. The location of the high-water marks are listed in Table 4 by address and frequency. The locations of the high-water mark data were all situated along Wilson Avenue where the FEMA buyout parcels are located. Figure 9 illustrates the results of the high-water mark calibration.

Table 4. Location and Frequency of High-Water Marks

Address	High-Water Mark Elevation (NAVD 88)	FEMA Frequency
1158 Wilson Ave	514.62	Greater than a 10% AEP

1106 Wilson Ave	515.94	Greater than a 10% AEP
Handley Rd, near 7401 Balson Ave	519.92	Greater than a 10% AEP

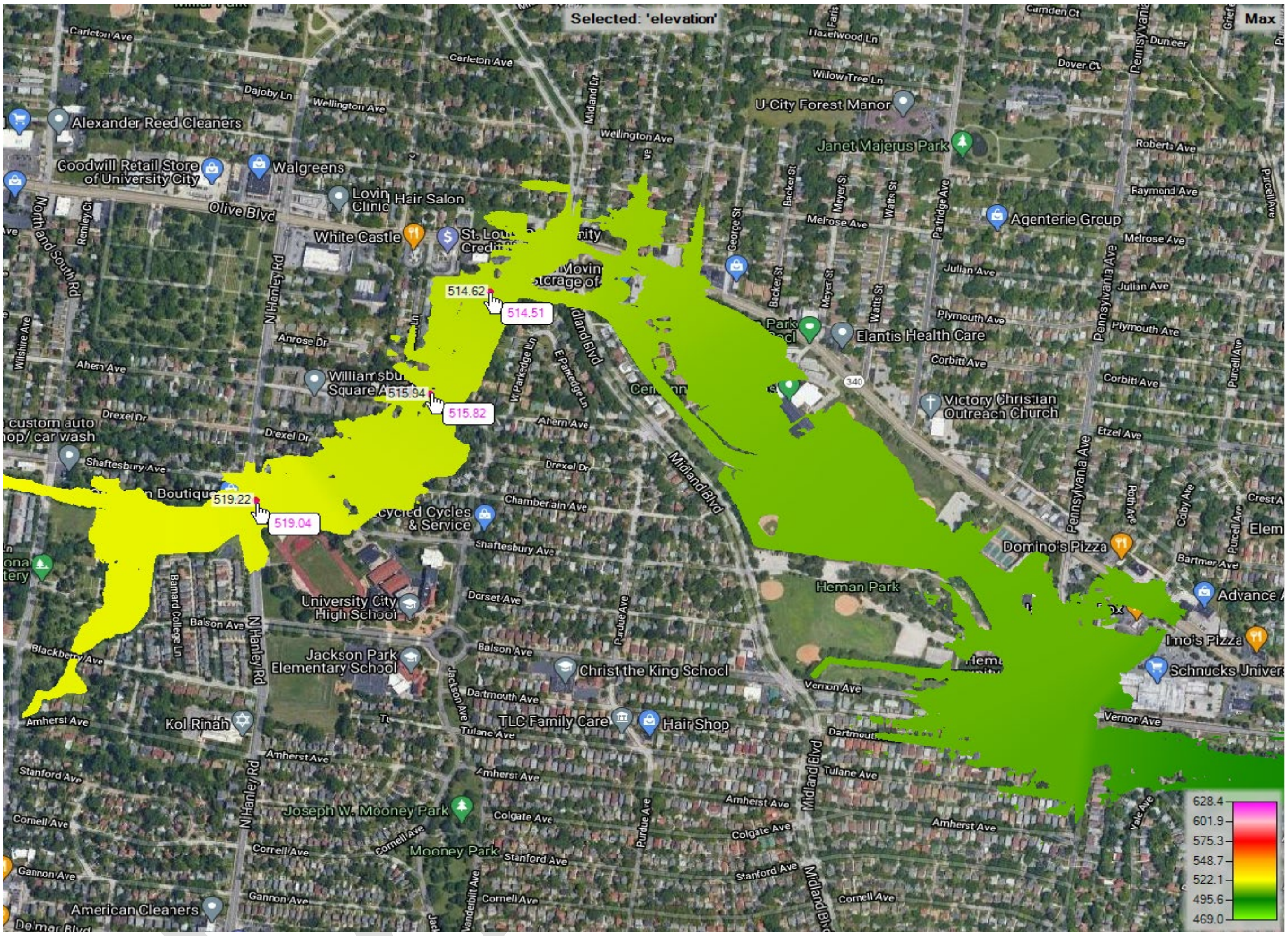

Figure 9. Calibration Results of 2008 High Water Mark Data

Simulation of the 2008 calibration events yielded peak stage results that were less than 0.1 feet off the observed measurements at the University City, MO gage. HEC-RAS flows were within 1% of the flow observation. The 2008 storm event high water mark calibration also fell within 0.2 ft of the recorded observations.

Simulation of the 2020 calibration events yielded peak stage results that were approximately 0.25 feet off the observed measurements at the University City, MO gage. HEC-RAS flows were within 10% of the flow observation.

The 2020 event calibration was not as good as the 2008 event. However, the event was within a reasonable deviation from observed measurements. Because the 2008 event was the flood of record, more weight was applied to 2008 calibration.

1.9 FREQUENCY EVENT ANALYSIS

Frequency event analysis was performed using the calibrated PCSWMM and HEC-RAS models. The frequencies analyzed were the 0.2, 0.5, 1, 2, 4, 10, 20, and 50 percent annual exceedance probabilities (AEP). The inundation of the 1% and 10% AEP events is illustrated in Figure 10. The resulting profiles for the 1% and 10% AEP event are illustrated in Figure 11.

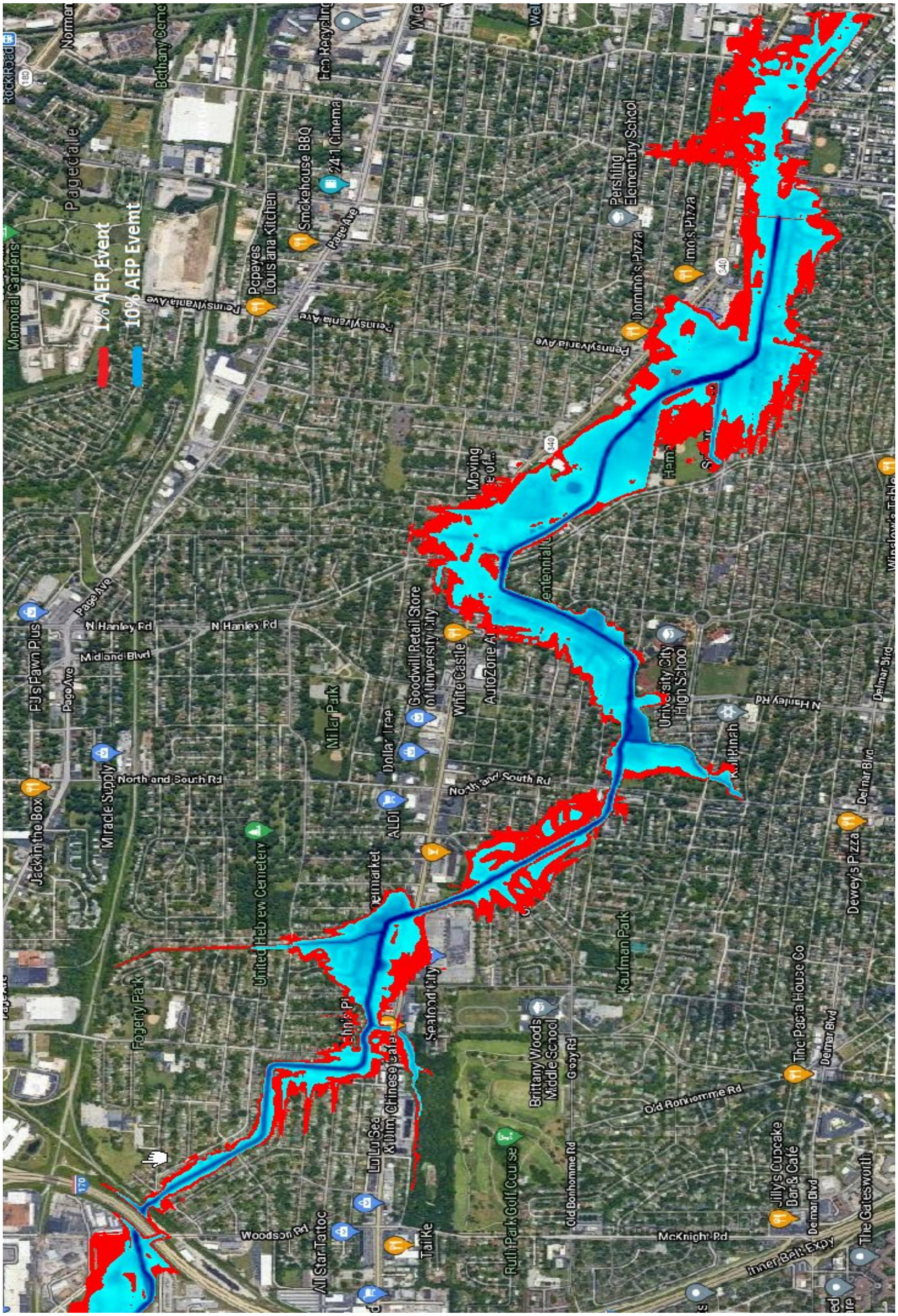

Figure 10. Existing Condition 1% and 10% AEP Event Inundation

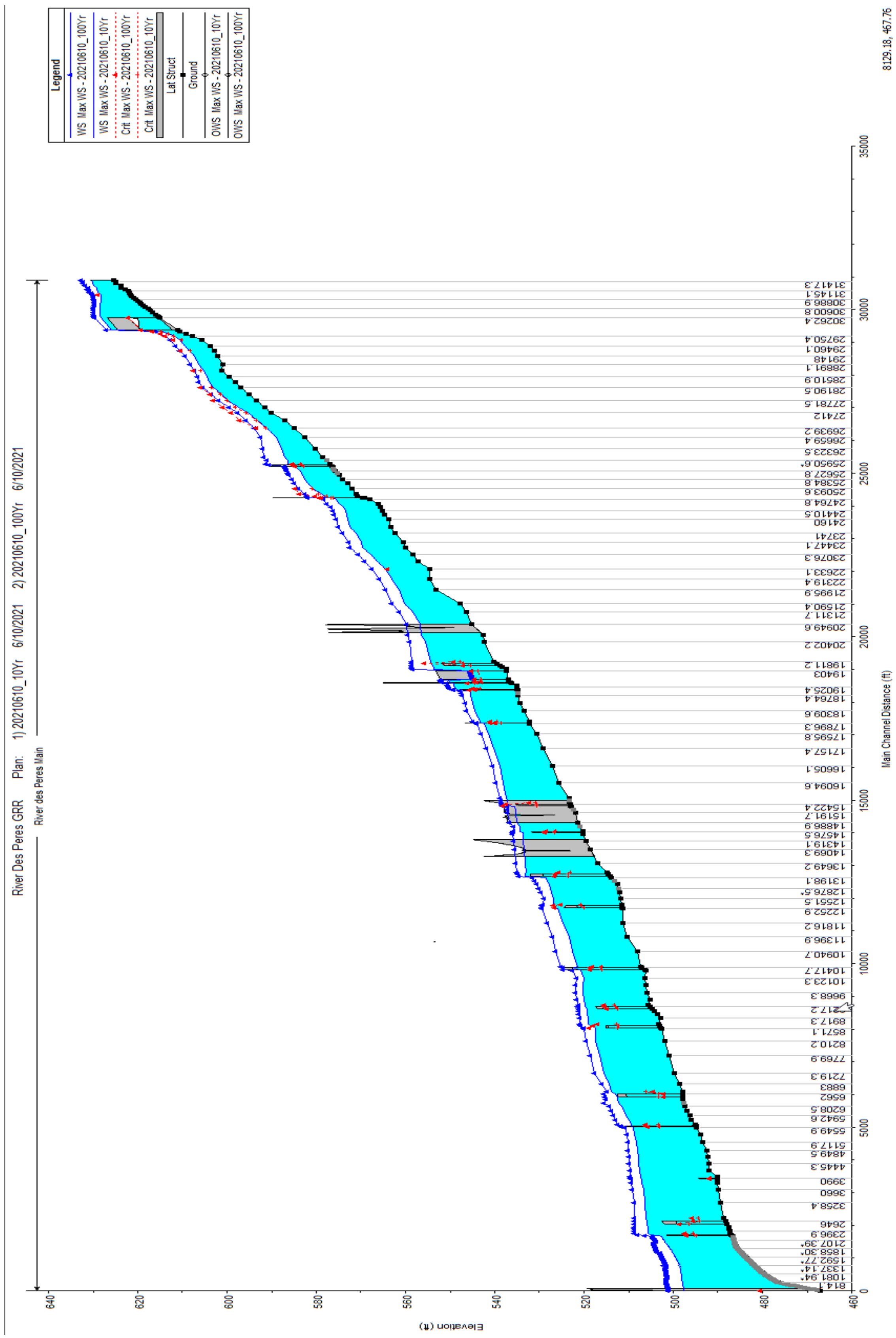

Figure 11. Frequency Profiles 1% and 10% Annual Exceedance Probability

The frequency analysis comparisons to a prior 2007 re-evaluation study of the 1988 FONSI is shown in Table 5. Comparison of Past Frequency Analysis with Current Conditions. Computed versus observed water surface and discharge at River Des Peres, University City gage September 2008 and August 2020. The main difference between the model approach between the 2007 analysis and the current study is in the manner storage is accounted for in the current model. The 2007 model was a steady state representation of the channel where the current model computes using unsteady state representation. Taking into account storage will be necessary during the alternatives analysis of detention storage.

Table 5. Comparison of Past Frequency Analysis with Current Conditions

	2007 Steady State Frequency Analysis		**Current Conditions (2021)**	
Annual Exceedance Probability (%)	**Elevation (ft) NAVD88**	**Flow (cfs)**	**Elevation (ft) NAVD88**	**Flow (cfs)**
0.2	512.93	11979	513.82	9709
1	511.79	9982	512.20	8419
2	511.33	8819	511.51	7757
4	510.73	7830	510.46	6776
10	509.92	6573	509.40	5594
20	507.57	5419	508.70	5008
50	505.14	3460	507.51	4049

In 2010, the USGS in cooperation with MSD conducted a study of ungagged watersheds in Missouri (Estimation of the Magnitude and Frequency of Floods in Urban Basins in Missouri; USGS 2010). In the study, the University City gage on the River Des Peres was utilized and a Bulletin 17B analysis was performed. Using the study created weighted least-squares regression equations a final weighted frequency discharge was computed adjusting the 17B results for the University City gages limited period of record. Comparison to the 2010 frequency analysis of the University City, MO gage on the River Des Peres is listed in Table 6. As shown, discharges of the same frequency have risen since 2010. This is most likely due to increased urbanization along with the trend of increasing frequency of extreme precipitation events. Refer to Appendix B Section 1.7 for Non-Stationarity of Monthly Maximum Precipitation at Lambert St. Louis Airport.

Table 6. Model Comparison to 2010 USGS Frequency Analysis

Annual Exceedance Probability (%)	**2010 USGS Adjusted Frequency Discharge (cfs)**	**2021 Current Condition Discharge (cfs)**
1	7440	8419
2	6790	7757

4	5470	6776
10	4770	5594
20	3840	5008
50	2840	4049

1.10 MSD PROJECTS IN UNIVERSITY CITY AND FUTURE WITHOUT PROJECT CONDITIONS

MSD identified 55 projects funded through its Operation Maintenance Construction Improvement (OMCI) program within the River Des Peres-University City watershed. An incomplete list of University City OMCI projects upstream of the previously authorized project area is provided in Table 7.

MSD anticipates that even if all these projects are constructed within the 50-year period of analysis, these future projects combined will not impact flow in the River Des Peres to the extent that the difference would be significant enough to affect USACE's H&H modeling effort (Riepe, 2020). Considering this, the future without project conditions are assumed to be the same as the current model conditions.

Table 7. Incomplete University City Watershed OMCI (#5584) projects upstream of USACE authorized project area as of July 2020

Project Name	Project Number	Municipality	Financial Year	Project Cost	Phase of Work	Scope of Work	Problem Description
Sims Ave 2201 Storm Buyout	10880	Overland	FY21	200,000	Land Purchase	Buyout and demolish 2201 Sims Ave to reestablish the overland flow path	Structure constructed within overland flow path and existing sewer under structure is deteriorating
Glenmary to White Rose Storm Improvements	11314	Olivette	FY23	380,000	Construction	Construct 1,160 feet of 15-in to 42-in storm sewer	Erosion and flooding due to inadequate storm system
Trenton Ave 9400 Block Channel Improvements	11313	Overland	FY23	386,000	Construction	Construct 350 feet of 6-ft high modular block wall	Creek erosion threatening structures
Collingwood Drive Consolidation Sewer	12127		FY21	Unknown	Construction		
University City I/I Reduction-East (UR-08 & UR-09)	11984		FY22	Unknown	Construction		
82nd Street to I-170 Sanitary Relief (UR-08 and UR-09)	11993		FY21	Unknown	Construction		
Price to Pioneer Sanitary Relief	12388		FY23	Unknown	Design		
Lindley Drive Sanitary Relief (I-70 to Ashmont Dr)	12329		FY21	Unknown	Design		
Cherry Tree Lane Storm Sewer Improvement	10209		Unfunded	Unknown	Identified		
Olive 8200 Block Bank Stabilization	10316	University City	Unfunded	1,024,000	Identified	Construct approx. 130 feet of concrete retaining wall, 605 feet of composite revetment, 125 feet of biostabilization, 300 feet of rock clock toe, 605 feet of heavy stone revetment, and 15 feet of 15-in storm sewer	Erosion threatening parking lot and fence on property at 8144 and 8162 Olive Blvd
Dielman Road to Appleseed Storm Sewer	10317	Olivette	Unfunded	168,000	Identified	Construct approx. 220 feet of rocklined trapezoidal channel, and 10 feet of 48-in diameter storm sewers, and appurtenances, from Dielman Rd to Appleseed	Yard flooding and erosion

Project Name	Project Number	Municipality	Financial Year	Project Cost	Phase of Work	Scope of Work	Problem Description
Dolores Ave Storm Improvements	11307	Olivette	Unfunded	704,000	Identified	Construct 535 feet of 42-in diameter RCP to 5-ft by 5-ft box culvert	Creek erosion
Edward Dr to Alice Pl Storm Sewer	11315	Olivette	Unfunded	137,600	Identified	Construct 325 feet of 42-in diameter RCP to 5ft x 3ft box culvert	Damage to property due to flooding
Echo Lane and Woodson Rd Storm Sewer	11457	Overland	Unfunded	504,000	Identified	Construct 995 feet of 12-in to 24-in storm sewer	Yard flooding of 4 years
Flore and Wismer Storm Sewer	11456	Overland	Unfunded	280,000	Identified	Construct 610 feet of 18-in storm sewer	Yard ponding due to inadequate street drainage
Lackland Ave 9900 Block Storm Sewer	11455	Overland	Unfunded	208,000	Identified	Construct 440 feet of 12-in to 15-in storm sewer	Yard ponding
Lackland Rd Wismer Ave Storm Sewer	11305	Overland	Unfunded	544,000	Identified	Construct 1,180 feet of 15-in to 30-in storm sewer	Localized runoff affecting parking lot
Ridge Ave #9408 Storm Sewer	11311	Overland	Unfunded	184,000	Identified	Construct 360 feet of 15-in RCP storm sewer	Yard ponding
Locust Ave to Maddox Pl Storm Sewer	11452	Unincorporated	Unfunded	960,000	Identified	Construct 1,985 feet of 12-in to 24-in RCP storm sewer	Inadequate road drainage throughout subdivision

2.1 MEASURES AND ALTERNATIVES

The following sections describe benefits and assumptions for the project's potential hydraulic structural modifications. Impacts will be compared in terms of water surface profile and inundation changes. The alternatives discussed include detention basin storage, channel widening, and levee construction. The original selected plan (authorized plan) in the 1988 study was channel widening from Purdue Avenue to 82nd Street.

2.1.1 Detention Storage Alternatives

Visual inspection of the watershed yielded five possible detention basin locations. The locations are illustrated in Figures 12 and 13. Detention Basin (DB) 1 and 2 are located on opposite sides of the River Des Peres in Heman Park. DB3 is located in the plaza at 8020 Olive Blvd currently occupied by the business Seafood City. DB4 is located in the City of Overland's Woodson Road Park and the adjacent field owned by the US Army Publications Distribution Center (1655 Woodson Rd). Of the five locations, DB 1, 2, and 5 were found to not have a significant impact on river stages. Based on their locations in the watershed and size/volume of storage, DB3 and 4 proved to have a measurable reduction in river stages.

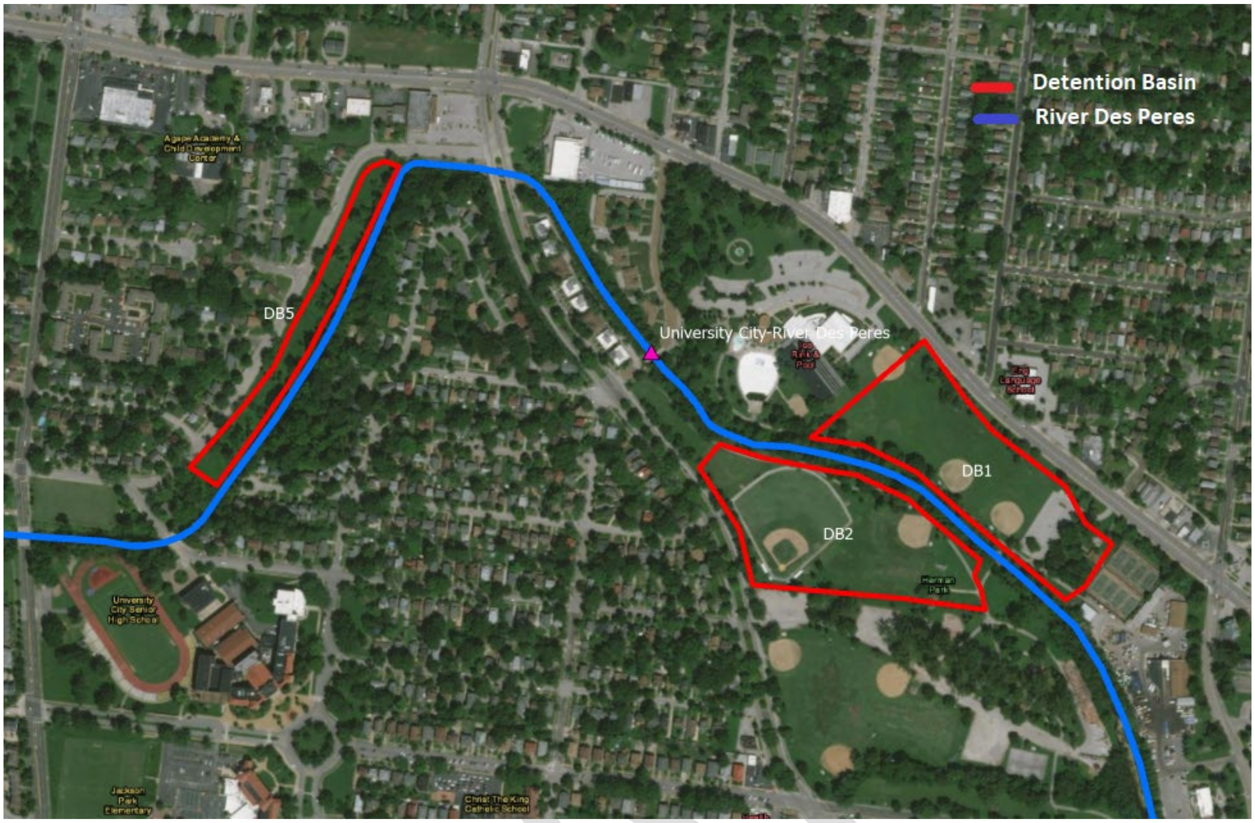

Figure 12. Location of Detention Basins 1, 2, and 5 (screened)

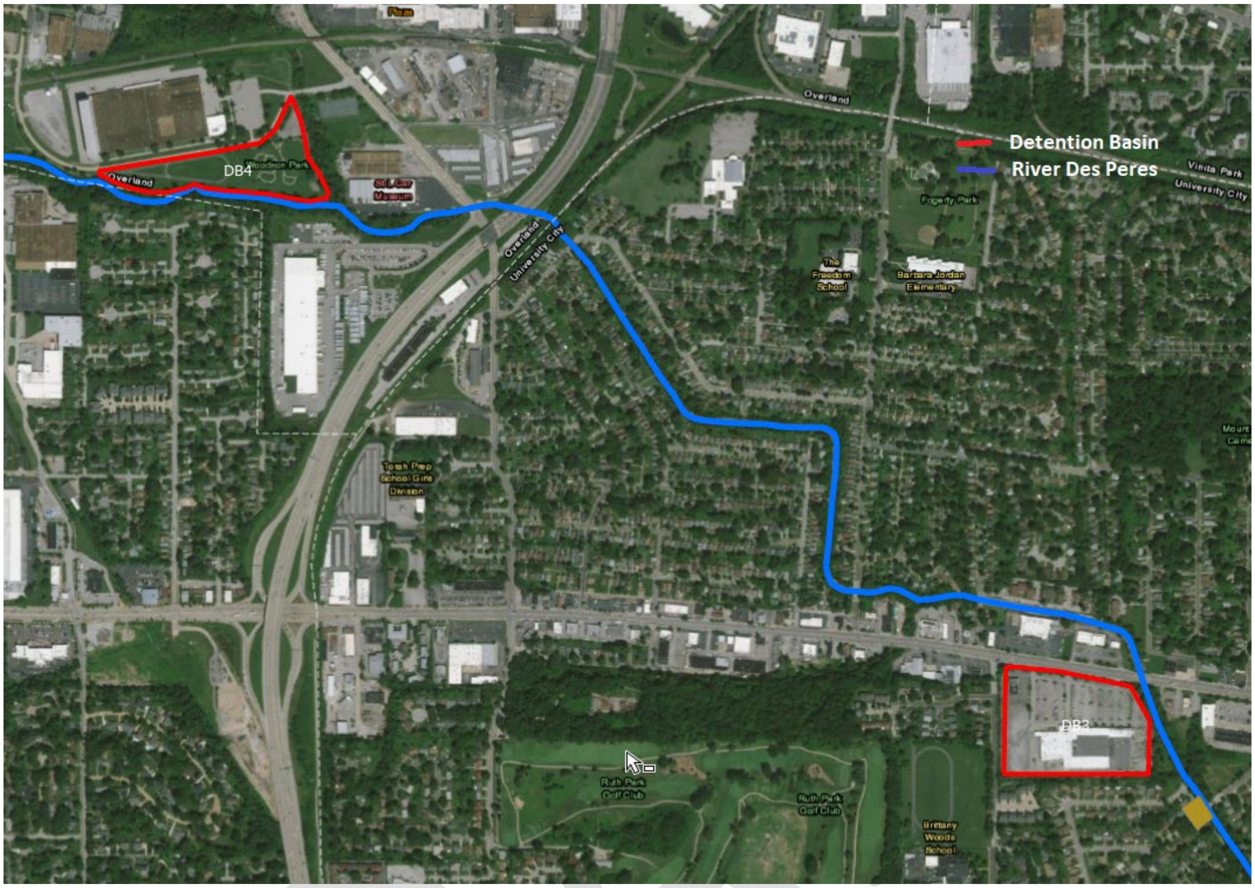

Figure 13. Location of Detention Basins 3 and 4 (retained)

The HEC-RAS model was modified to capture the effects of three different detention basin scenarios:

- DB3 and DB4
- DB3 Only
- DB4 Only

The detention basins were designed to have an unregulated weir as the inflow structure and a closed conduit outflow structure. The detention basins were treated as one-dimensional storage areas with a constant area and depth. Lateral structures were used to model the embankments, inlet, and outlet configurations. The HEC-RAS model geometry configuration is illustrated in Figures 14 and 15.

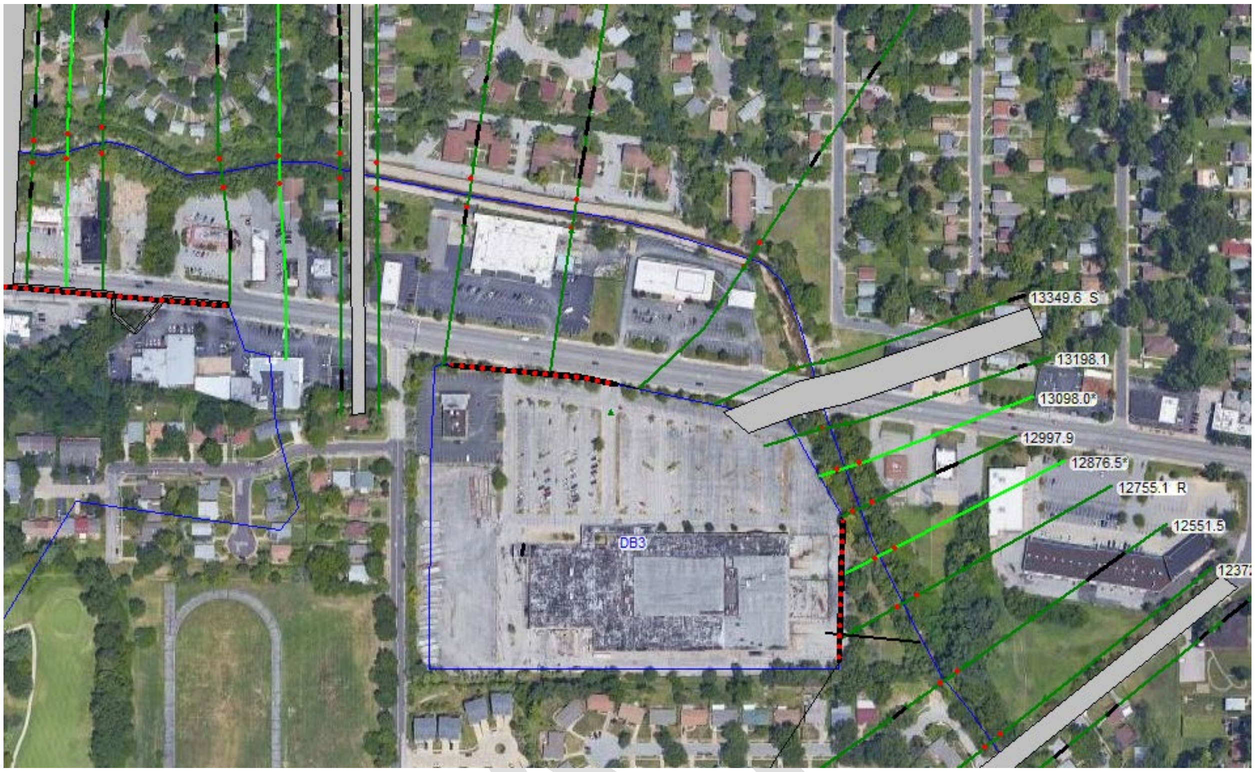

Figure 14. HEC-RAS Detention Basin 3 Configuration

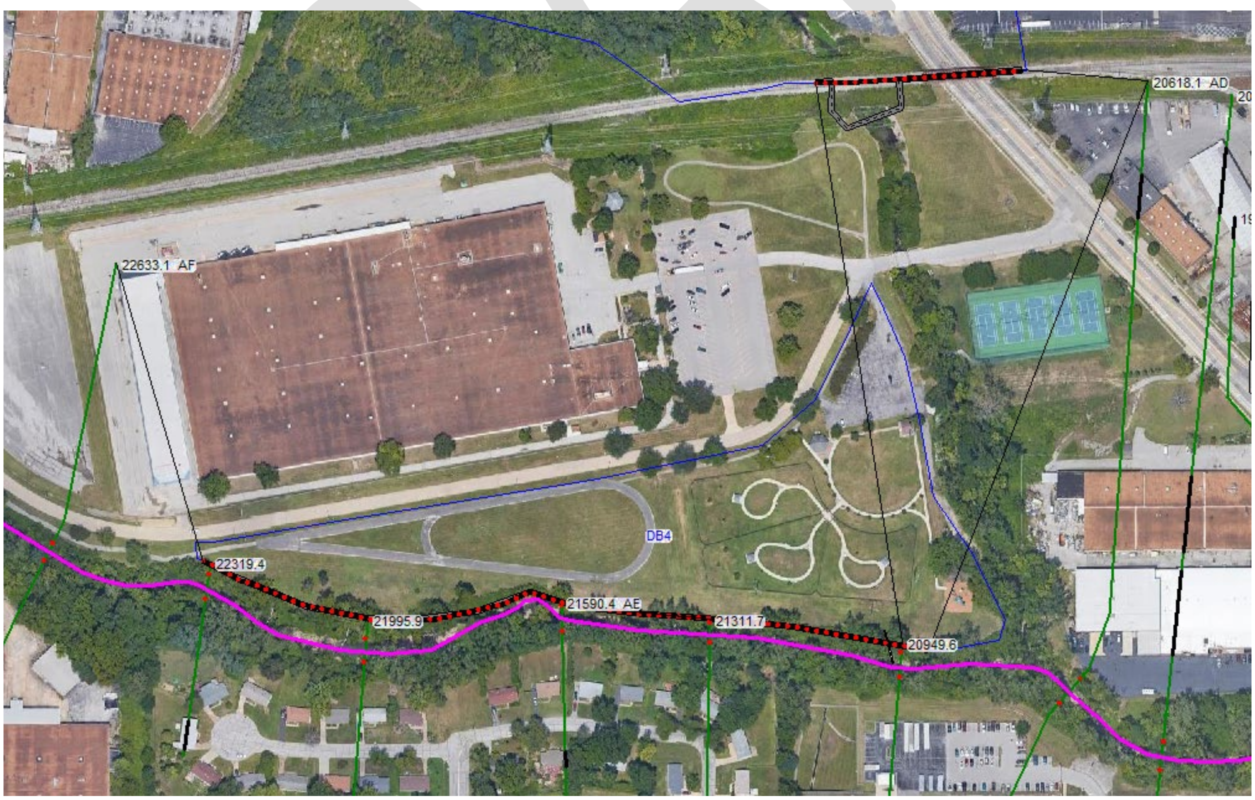

Figure 15. HEC-RAS Detention Basin 4 Configuration

The detention basin configuration was optimized for the 50%, 10%, and 1% AEP events. The assumptions employed were:

- Weir inlet elevation was set 1 to 2 feet below the 50% AEP water surface profile at DB's respective location along the River Des Peres.
- The depth and weir length of the DB was optimized to ensure that it was filled by the 10% AEP storm event. This would be to the level of the inlet control weir.
- The DB embankments would be overtopped during the 1% AEP storm event.
- The closed conduit outfall was sized to ensure that the DB would be drained within 12 to 24 hours. To ensure adequate storage volume, backflow preventers will be employed as to not allow tailwater backup through the outlet pipe before weir overtopping.

The final DB 3 and 4 dimensions were optimized together using a single HEC-RAS geometry. These same dimensions were used for the single DB scenario geometries as well. The final structural dimensions of the detention basins 3 and 4 are listed in Table 8 and Table 9.

Table 8. Detention Basin 3 Design Configuration

Detention Basin 3		
Design Volume	**DB Area (Ac)**	14.7
	DB Base Elevation (ft)	519.0
	Embankment Elevation (ft)	528.0
Inlet Design	**Inlet Control Weir Elevation (ft)**	524.0
	Weir Length (ft)	150.0
Outlet Design	**Outfall Pipe Diameter (ft)**	3.0

Table 9. Detention Basin 4 Design Configuration

Detention Basin 4		
Design Volume	**DB Area (Ac)**	8.9
	DB Base Elevation (ft)	550.0
	Embankment Elevation (ft)	563.0
Inlet Design	**Inlet Control Weir Elevation (ft)**	557.5
	Weir Length (ft)	125.0
Outlet Design	**Outfall Pipe Diameter (ft)**	3.0

The resulting 1% AEP water surface profiles for the three scenarios are illustrated in Figures 16 through 18. Focusing on the Groby/Shaftbury/Wilson Ave vicinity flooding, the resulting reduction in water inundation is shown in Figures 19 through 21.

As seen in the water surface profiles for the DB3 and DB4 scenarios there is no increase in water stages. It would be expected that if an embankment was constructed next to the river that there would be an increase in water level from the encroachment. Because the locations of the detention basins are dry during most flood events and since the detention basins only add volume to the floodplain through excavation there is not an increase to the water surface profiles for any of the frequency events.

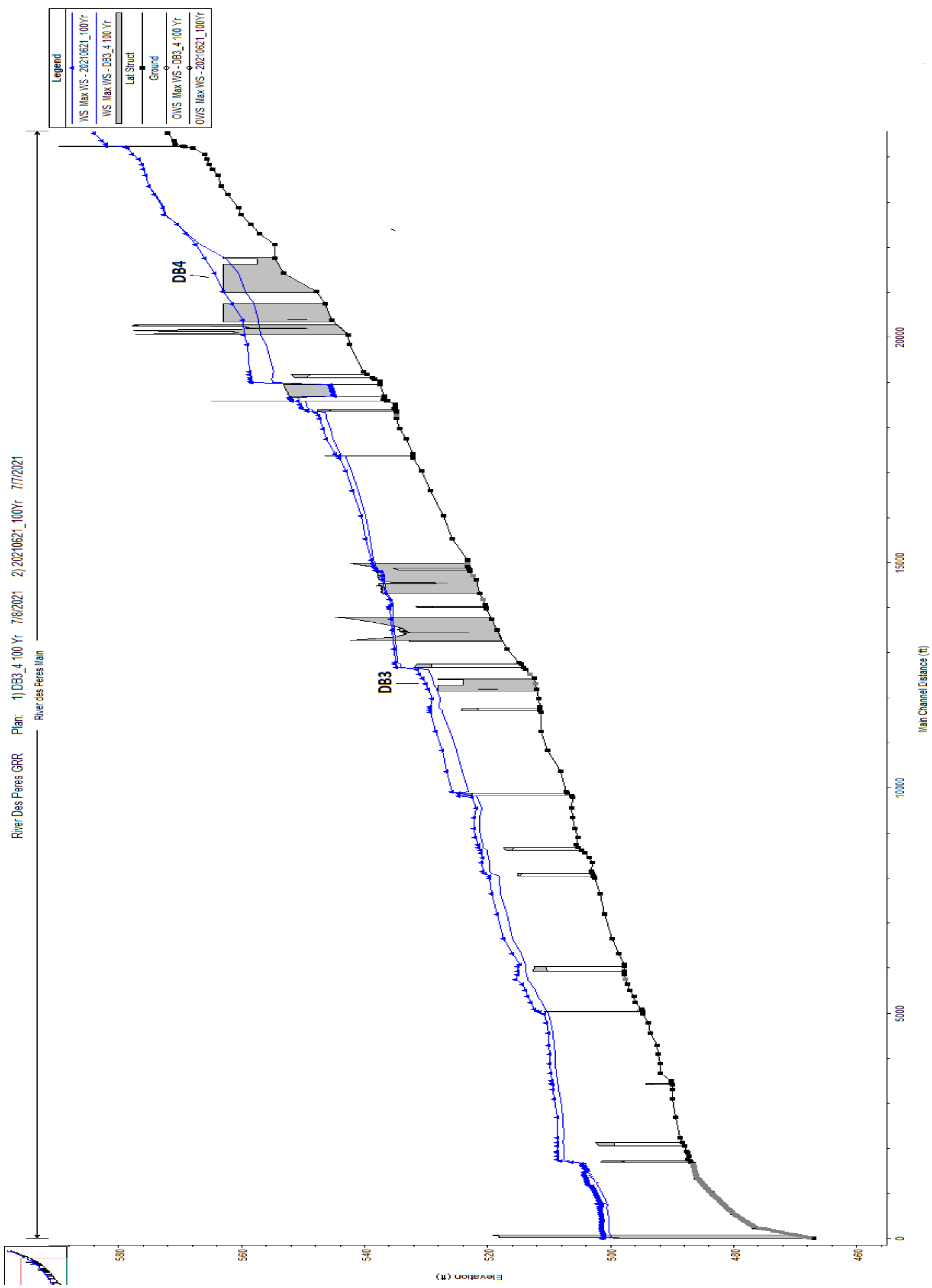

Figure 16. Proposed Detention Basin 3 and 4 Alternative vs. Existing Conditions - 1% AEP Profile

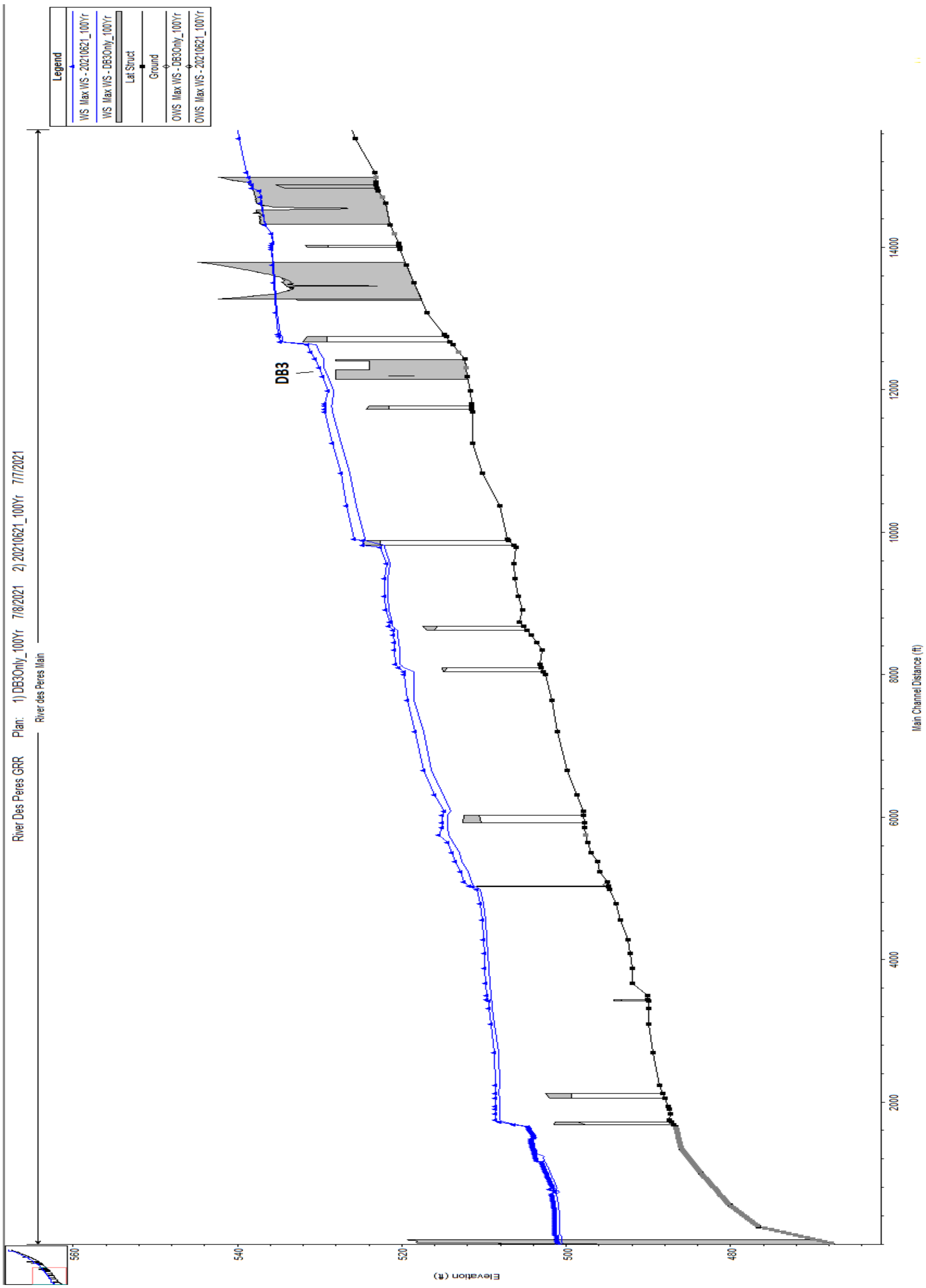

Figure 17. Proposed Detention Basin 3 Alternative versus Existing Conditions - 1% AEP Profile

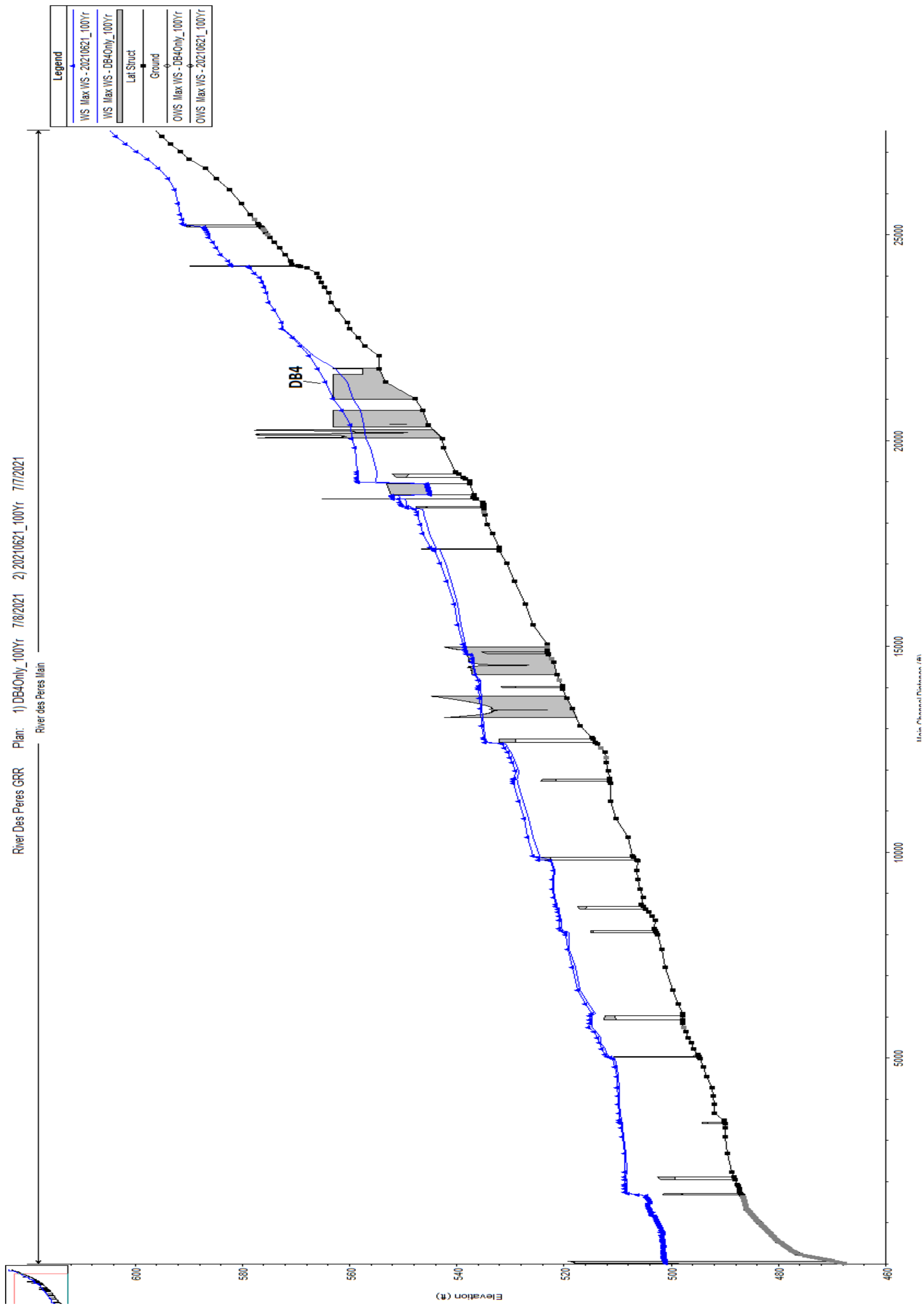

Figure 18. Proposed Detention Basin 4 Alternative versus Existing Conditions - 1% AEP

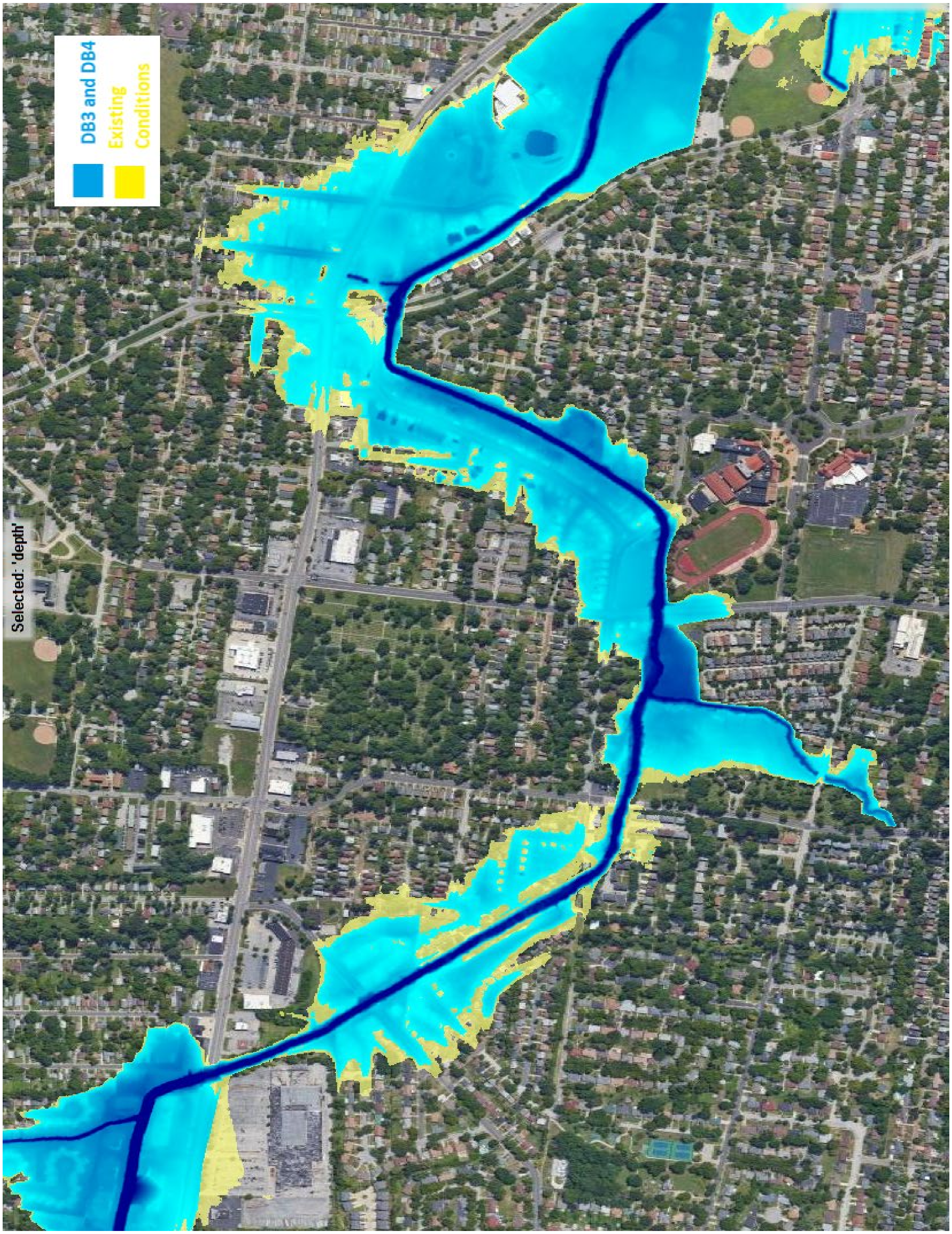

Figure 19. Proposed DB3 and 4 Alternative versus Existing Conditions Inundation - 1% AEP - Groby/Shaftsbury/Wilson Vicinity

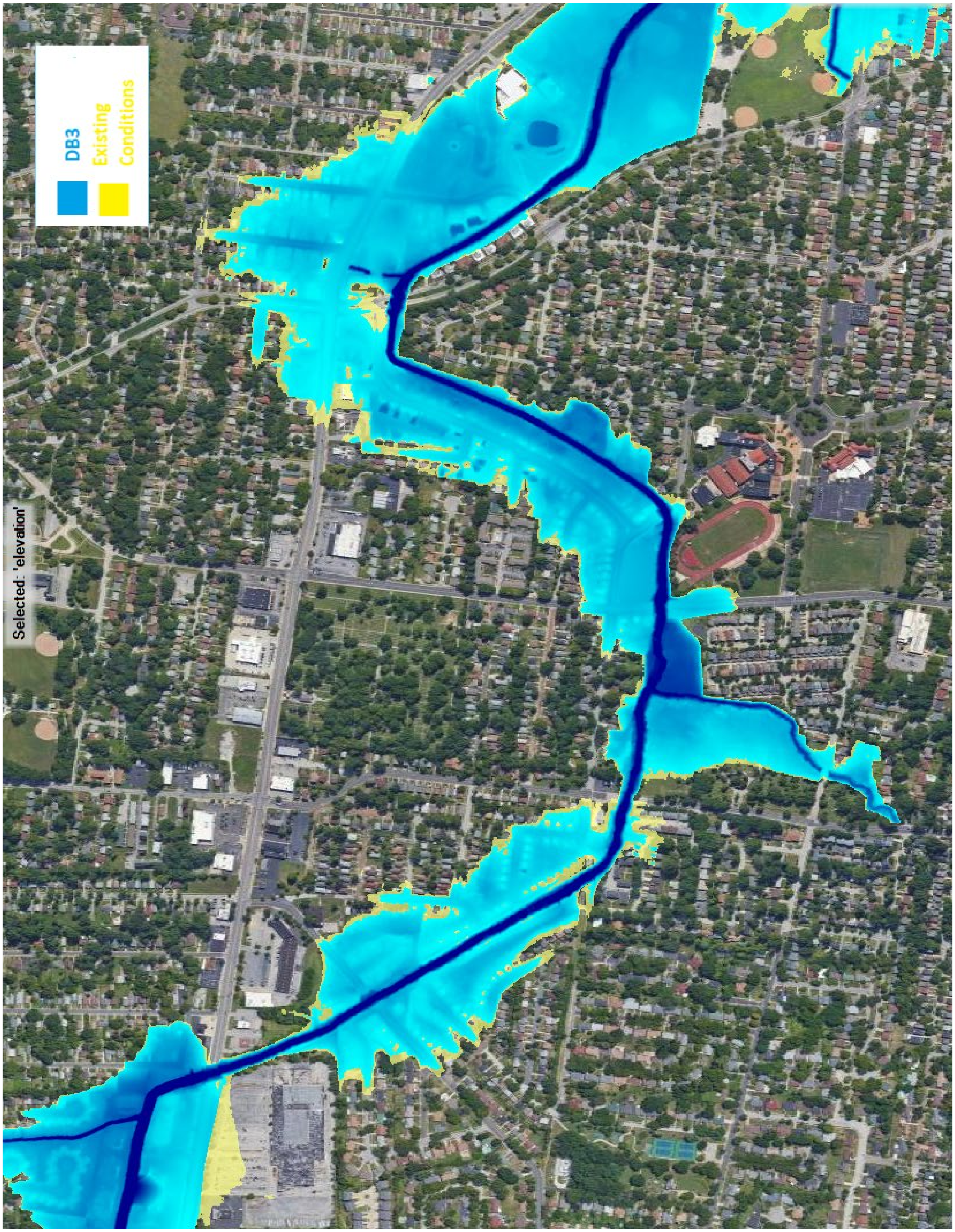

Figure 20. Proposed DB3 Alternative versus Existing Conditions Inundation - 1% AEP - Groby/Shaftsbury/Wilson Vicinity

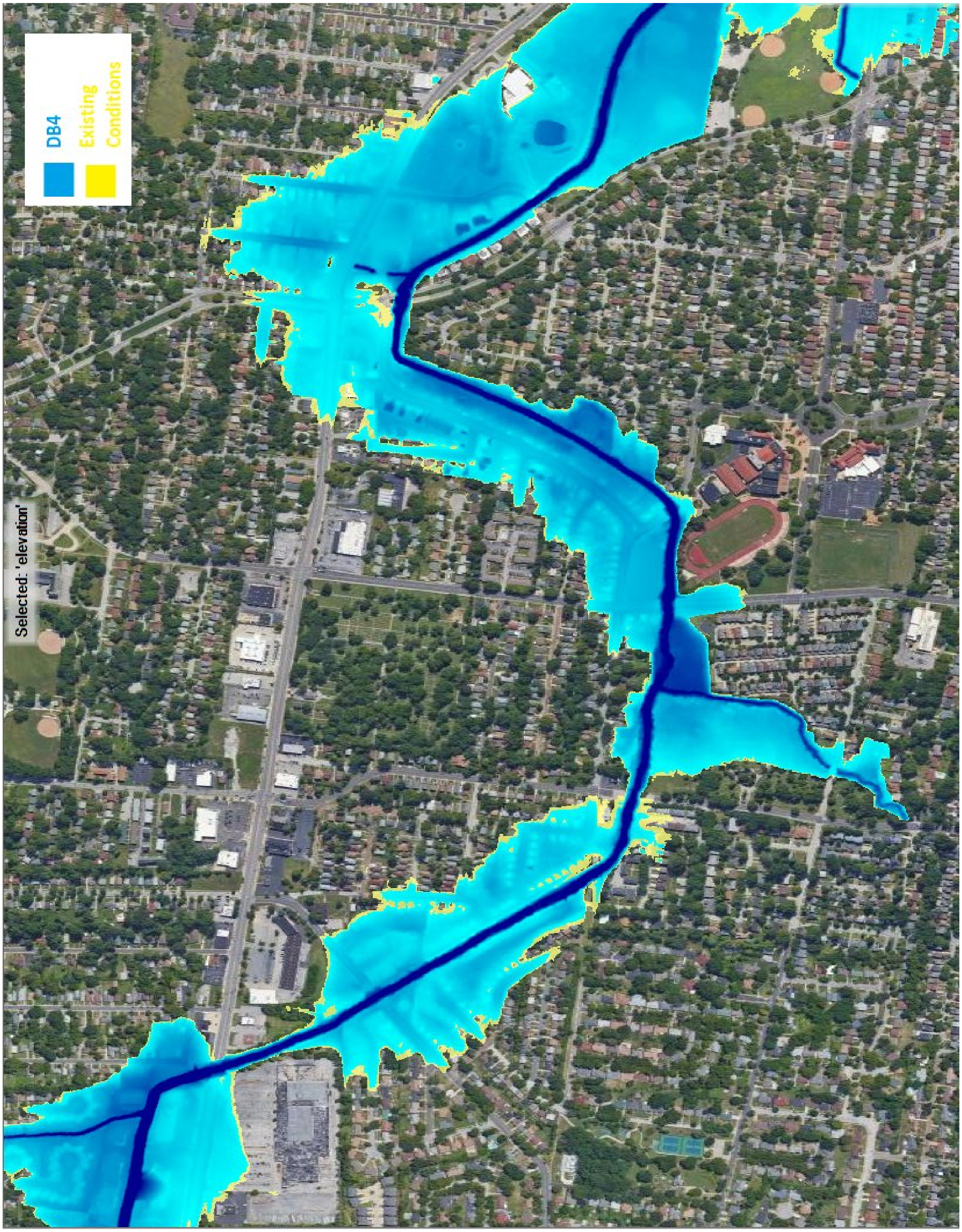

Figure 21. Proposed DB4 Alternative versus Existing Conditions Inundation - 1% AEP - Groby/Shaftsbury/Wilson Vicinity

2.1.2 Levee Alternative

HEC-RAS was used to examine the potential for levees along River Des Peres in University City. The levee configurations tested are illustrated in Figure 22.

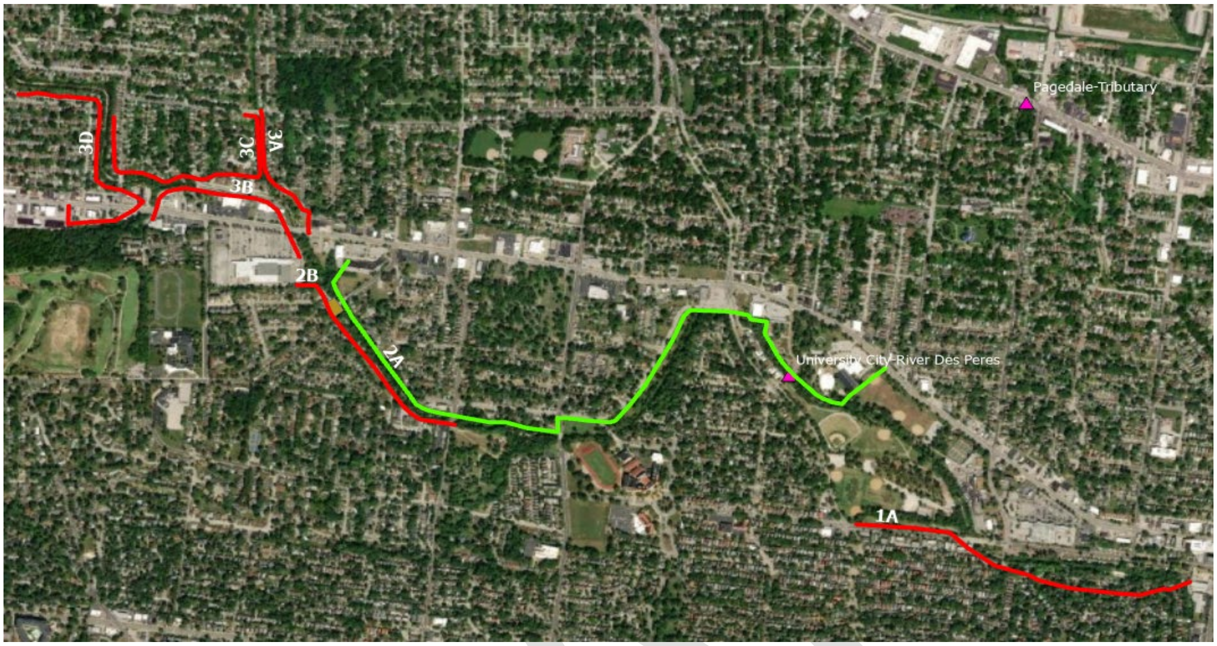

Figure 22. Proposed Levee Alignments

None of the levee reaches could be constructed without induced flooding. They would also increase discharges into the Tubes. Levee reach 2A (highlighted in green) was the only option that worked without induced flooding after incorporating detention basins 3 and 4.

Because of the high cost of construction and real estate acquisition the levee alternatives were discounted as the project benefit is too low. As a result, no further investigation of the levee alternatives was considered.

2.1.3 Channel Widening and the Original Selected Plan U-12

The selected plan from the 1988 River Des Peres Feasibility study was a channel widening from the Purdue Ave foot bridge upstream to 82nd St. The widening included 1.43 miles of rip rapped channel enlargement, 0.42 miles of gabion-lined channel, and bridge replacements. The channel modification itself would be an enlargement having an average base width of 28 feet, a depth of 12 feet, and 3 on 1 side slopes for the riprap-lined channel and a top width of 65 feet for the gabion lined channel. For the analysis in this study all bridges between Purdue Ave and 82nd bridge would be required to be at least 75 feet with no piers.

The results comparing the proposed with the existing conditions water surface profile is illustrated in Figure 23.

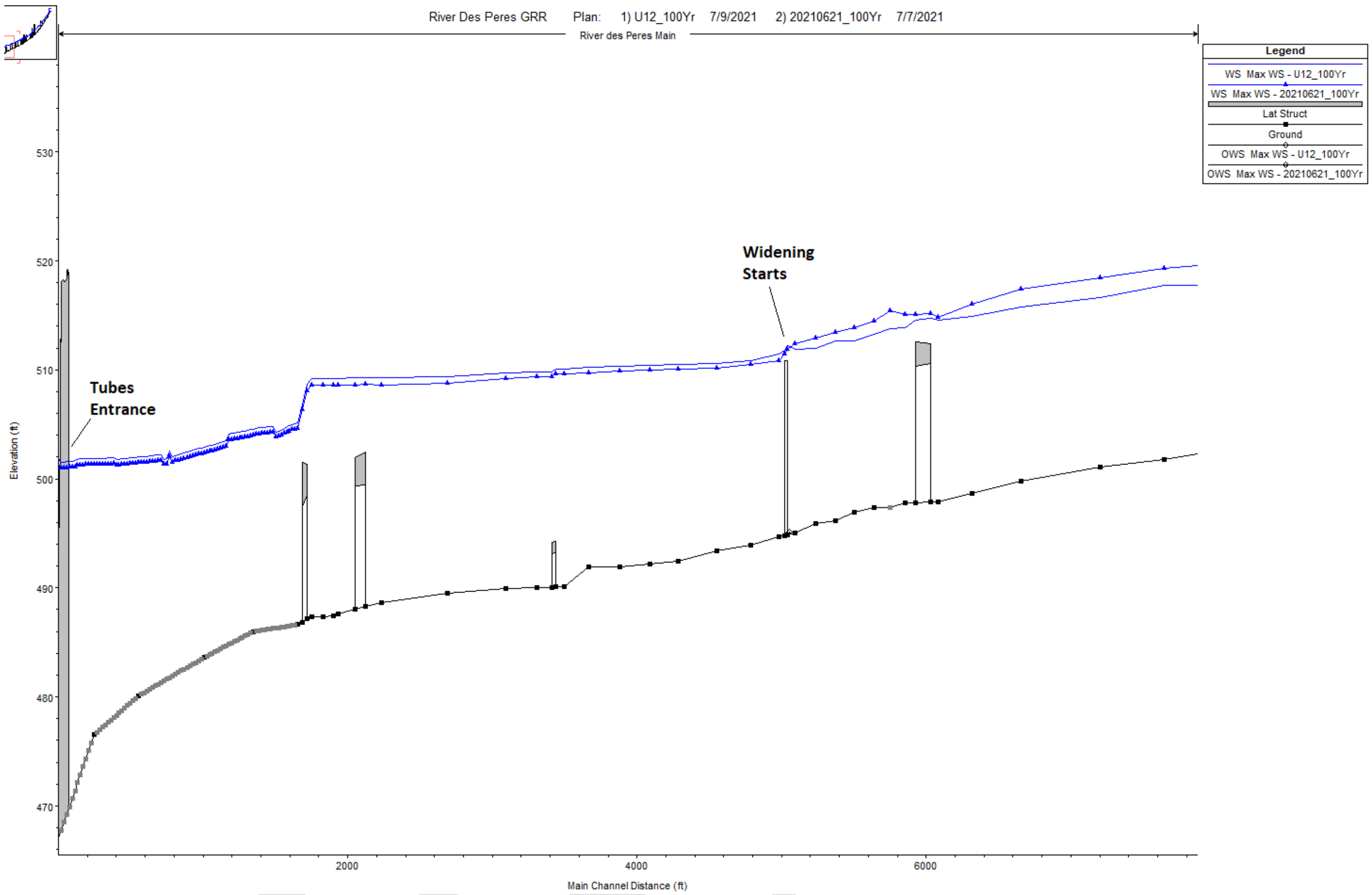

Figure 23. Original TSP, Alternative U12, Channel Widening - 1% AEP Water Surface Profile

From the HEC-RAS analysis, the U12 profile does reduce the water surface upstream of the widening, however it increases the water surface profile downstream of the widening. This would also indicate that because of this widening, higher discharges would enter the Tubes. The increases in discharge upstream of the tubes is tabulated in Table 10.

Table 10. Flow Comparison at Tubes Entrance - 1% AEP

Annual Exceedance Probability	Existing Conditions Discharge (cfs)	Proposed Condition U12 Widening (cfs)	Difference (cfs)
0.2	9257	10042	+784
0.5	8560	9491	+931
1	8079	8669	+590
2	7453	7760	+307
4	6565	6900	+335
10	5404	5852	+448
20	4629	4861	+231
50	4057	4154	+97

Increasing discharges into the Tubes would most likely exacerbate conditions downstream of the study area. Not to mention that when the Tubes are at capacity this increase will almost certainly add to the overflow volume around the Tubes. Because of the potential for significant impact to the hydraulics outside of the study area, it is not recommended to continue with U12 as the selected plan.

2.1.4 Modified Channel Widening Plan

Because of this increase in discharges to the Tubes, the original selected plan (U12) was changed to include detention storage for the purposes of mitigating this increase in discharge to the Tubes. By adding both detention basin 3 and 4, enough volume can be removed effectively assuring lower stages continue downstream of the channel widening. The modified profile is shown in Figure 24. The reduction in inundation is illustrated in Figure 25 for the Groby/Shaftsbury/Wilson Avenue vicinity.

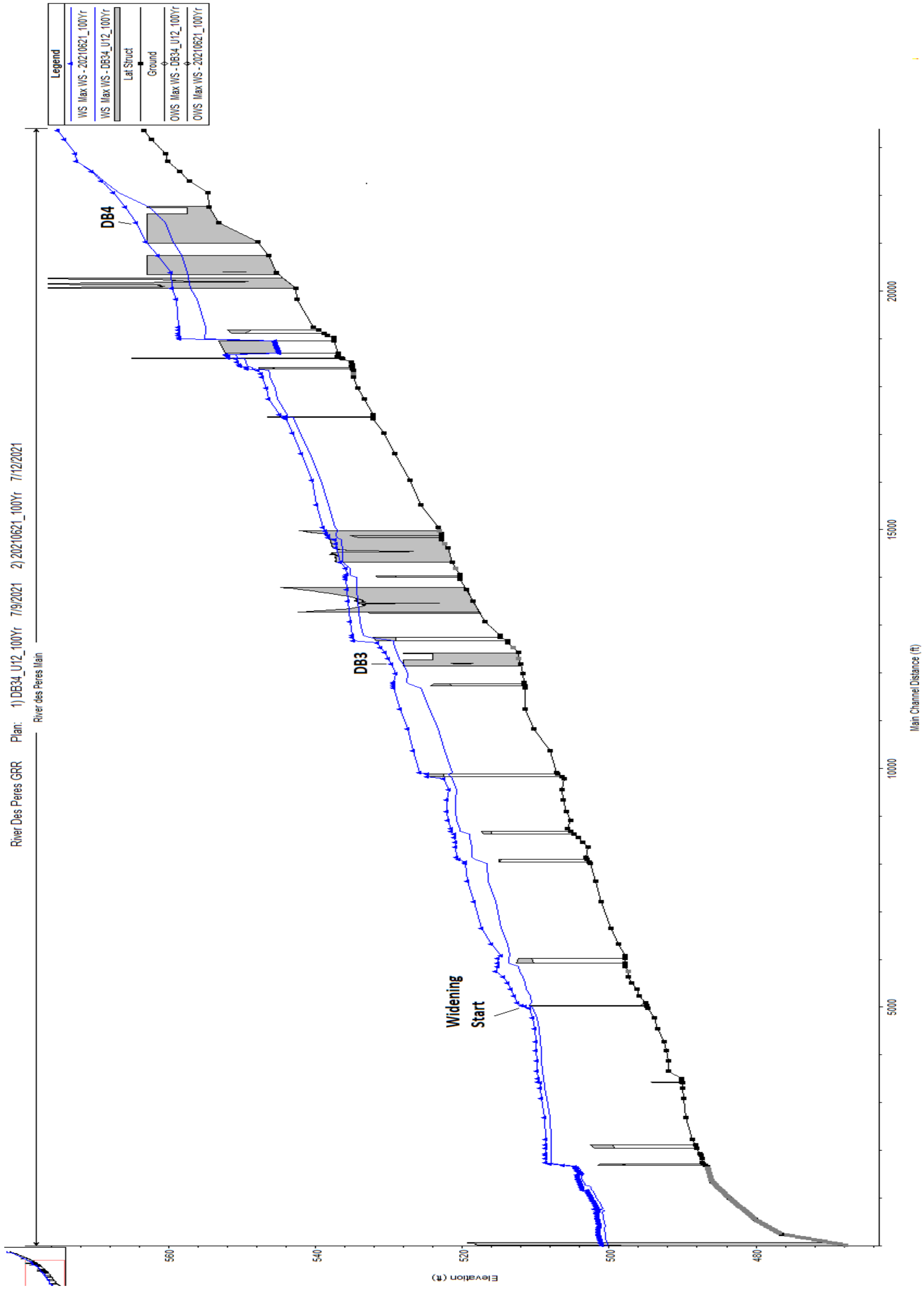

Figure 24. Modified Channel Widening, U12 - 1% AEP Water Surface Profile

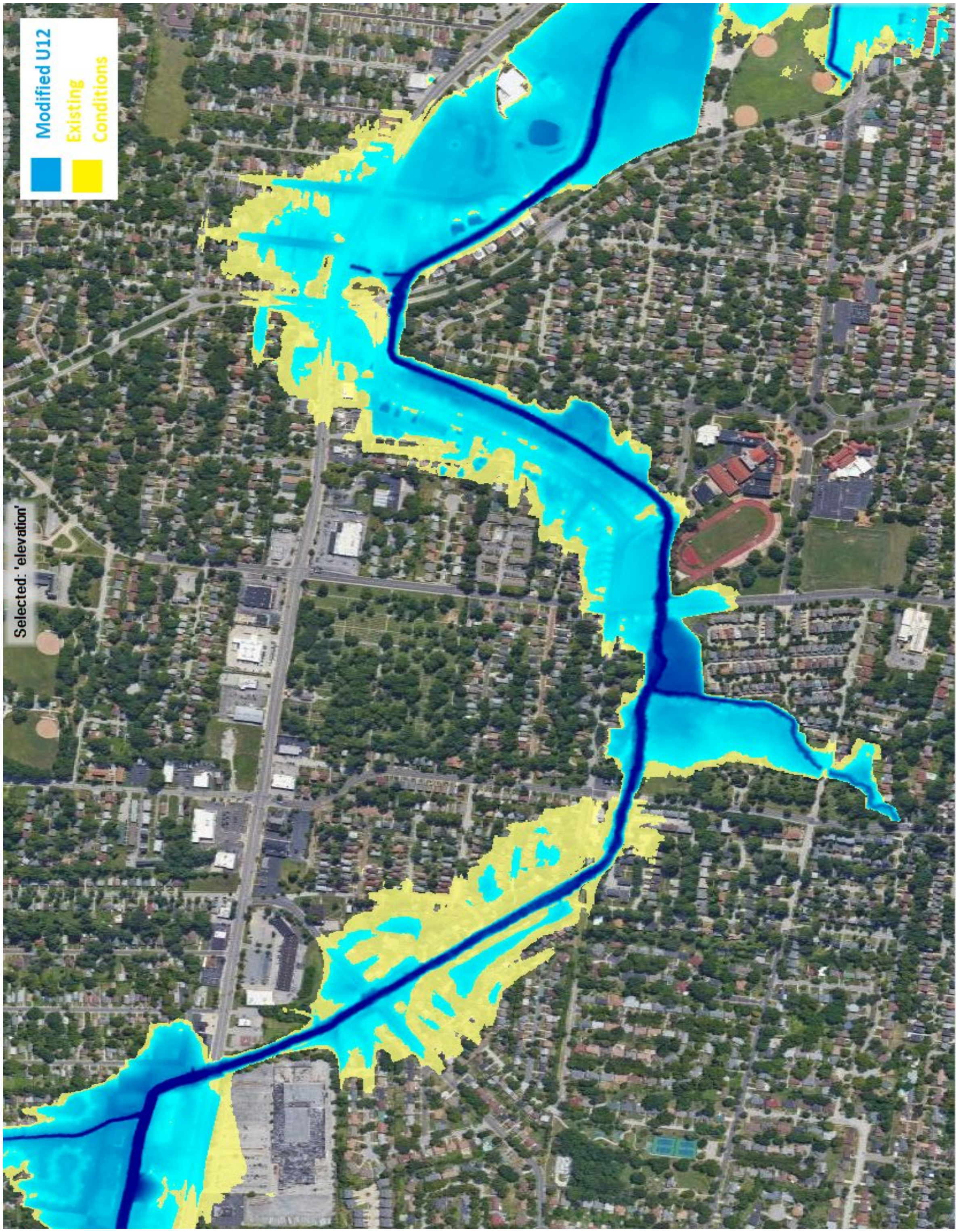

Figure 25. Proposed Modification to U12 Alternative versus Existing Conditions Inundation - 1% AEP - Groby/Shaftsbury/Wilson Vicinity

2.2 ALTERNATIVE COMPARISONS

In review of the alternatives the benefits of each alternative can be assessed at various locations throughout the project area. The focus will be on three locations: at the Tubes entrance (XS 814.1), at the University City gage on the Purdue Ave foot bridge (XS 5659.3), and at the Olive Blvd near Westlove Ave (XS 13349.6). Table 11 compares the water surface levels for the 1% AEP at these locations for each of the hydraulically feasible alternatives.

Table 11. Comparison of Alternatives' 1% AEP Water Surface Elevations

	Water Surface Elevation NAVD 88 (ft)			Differences		
Alternative (# if included in final array)	**Tubes Entrance [XS 814.1]**	**Purdue Ave Gage [XS 5659.3]**	**Olive Blvd near Westlove Ave [XS 13349.6]**	**Tubes Entrance [XS 814.1]**	**Purdue Ave Gage [XS 5659.3]**	**Olive Blvd near Westlove Ave [XS 13349.6]**
Existing Conditions	501.31	512.3	535.03			
U12/Channel Widening	501.86	511.84	534.12	0.55	-0.46	-0.91
Modified U12/Channel Widening (Alternative 2)	500.4	510.55	533.48	-0.91	-1.75	-1.55
DB3 and DB4 (Alternative 3a)	500.26	510.69	534.59	-1.05	-1.61	-0.44
DB3 Only	500.79	511.47	534.81	-0.52	-0.83	-0.22
DB4 Only (Alternative 3b)	501.01	511.77	534.77	-0.3	-0.53	-0.26

The results of the analysis of alternatives shows that the Modified U12/Channel Widening (Alternative 2) offers the highest reduction in river stages starting within and upstream of the widened portions. It also shows that Alternative 3a has a similar impact on stage reduction. All alternatives analyzed will reduce expected flood levels at Purdue Ave and Olive Blvd near Westlove Ave. The originally selected plan, U12/Channel Widening is the only plan that would result in induced flooding at the entrance to the Tubes.

2.3 RISK ANALYSIS

For the selected and locally preferred plan that has the construction of DB4 as the structural option, the worst-case breach condition was examined.

The detention basin works more like a dam than a levee. It behaves like a dam by containing water to reduce the peak of flow downstream during certain storms then releasing it after the peak has passed when river stages are lowered. The highest head difference between the interior storage area and the river would occur at the outfall structure. The highest achievable head would be when the detention basin is full up to the weir control inlet and the river drops out. This would equate to a head of 7.5 feet or the difference between the weir inlet elevation and the outlet pipe invert. Though this is the actual worst-case scenario for a breach, the volume is small enough to not impact downstream conditions.

The worst-case condition hydraulically would be if a breach formed at the outlet pipe during the peak stage in the detention basin during the higher magnitude frequency events. Though this would breach before the highest head difference between the detention basin interior and the river, this would yield impacts when the river was already out of bank. For this breach condition, a complete blow-out breach with a width of 75 feet at the outlet pipe was simulated. Degradation progression would have normally been used, but because of the short duration of flooding on the River Des Peres the containment embankment would not adequately degrade.

Breach analysis for a Probable Failure Mode Analysis (PFMA) would also include breaches at other probable failure points. A breach at the inlet would not be as detrimental as water would still be attenuated on breach release. Breaches in the embankments upstream from the downstream outlet would have less impact the further upstream from the outlet. As such, this abbreviated analysis focused on a detention basin breach only at the outlet pipe.

Table 12. Water Surface Elevations Downstream of DB4

AEP	**Water Surface Elevation NAVD 88 (feet) XS 20402.2**			**Proposed DB4 Difference**	
	Existing	**Proposed DB4 Alternative**	**Proposed DB4 Breached**	**Difference with Existing**	**Difference with Breach**
0.2%	562.05	560.1	560.58	1.95	0.48
0.5%	560.53	557.05	558.01	3.48	0.96
1%	558.95	556.05	556.05	2.9	0
2%	557.03	555.36	555.36	1.67	0
4%	556.72	555.11	555.11	1.61	0
10%	555.66	554.02	554.02	1.64	0

Table 12 compares the existing, proposed, and proposed with-breach condition water surface elevations. Figure 26 shows a profile comparison of the 0.5% AEP with-project condition comparing outlet breach versus non-breach. As seen in the illustration, impacts are within 1500 ft of the outlet. After the second bridge it becomes less than 0.2 feet.

It was determined that highest with-project reduction immediately downstream of the detention basin is seen at the 0.5% AEP frequency (3.48 ft). The average reduction in water surface below the 1% AEP is 1.9 feet. Below the 2% AEP, water surface reduction trends similar differences for those more frequent events. The benefit stops once the weir control level is reached. The highest reduction in water level being the 0.5% AEP event is most likely due to the embankment having less of a constricting effect as it is overtopped during this and the less frequent events.

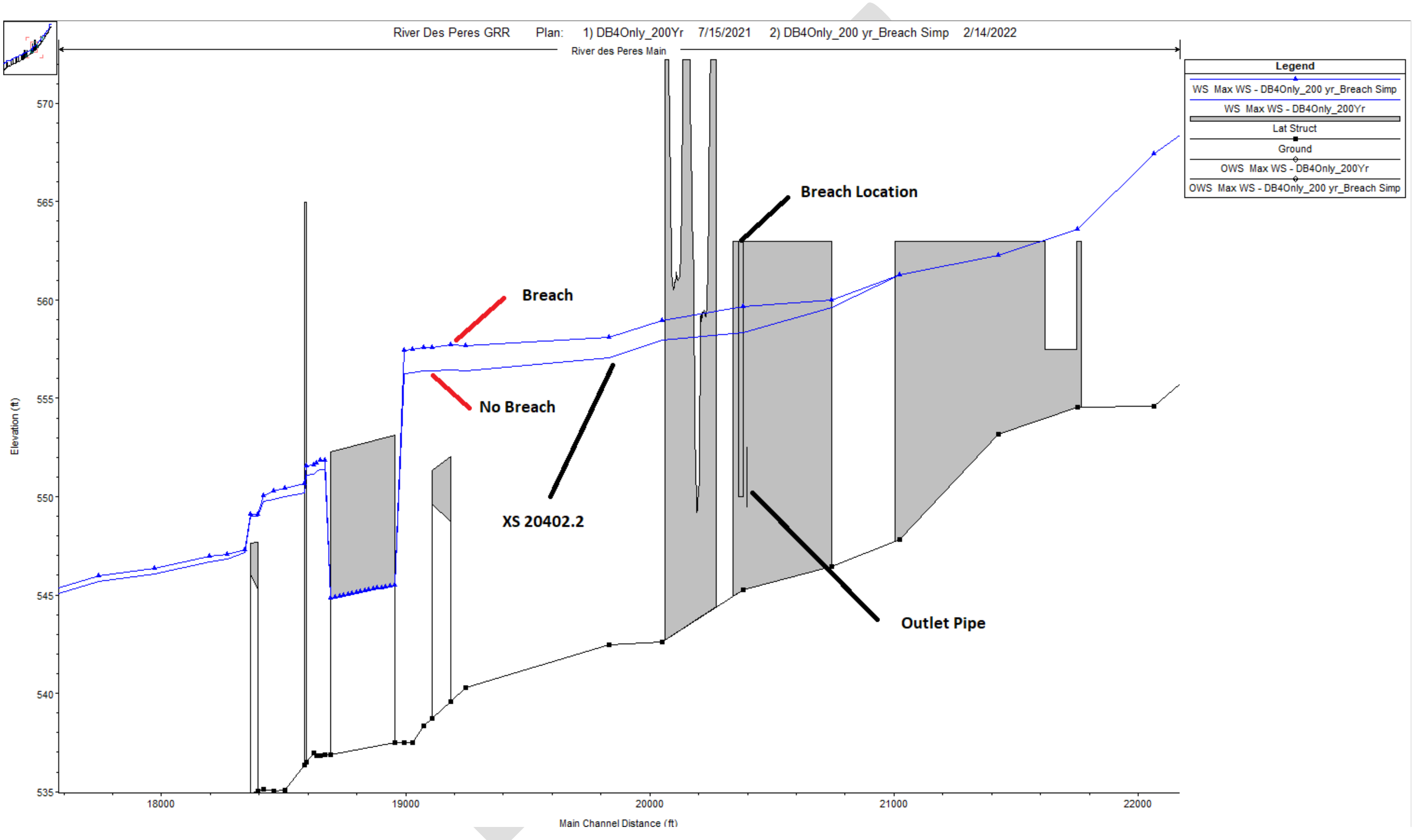

Figure 26 – 0.5% AEP With-Project Breach versus No-Breach

APPENDIX A-1

PCSWMM FREQUENCY MODEL SUBCATCHMENT PARAMATERS – 100 YEAR/1% AEP