Big Cicero Creek
Flood Control Study

VOLUME 2

Tipton County

Prepared for

Big Cicero Creek Joint Drainage Board
101 East Jefferson Street
Tipton, Indiana 46072

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Prepared by

Christopher B. Burke Engineering, Ltd.
115 West Washington Street, Suite 1368 South
Indianapolis, Indiana 46204

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1.0 INTRODUCTION

This report presents the results of the Christopher B. Burke Engineering, Ltd. (CBBEL) analysis of the Big Cicero Creek watershed completed as part of a flood control study for the Big Cicero Creek Joint Drainage Board (Board). As part of the study, CBBEL staff developed a hydrologic model of the watershed and a new hydraulic model.

Upstream portions of the Big Cicero Creek watershed are in Clinton, Boone, and Hamilton Counties. Water flows from these counties into the southwest part of Tipton County, through the town of Tipton, back into Hamilton County and empties into Morse Reservoir. The Big Cicero Creek watershed area is approximately 80 square miles at the confluence with Buck Creek in the City of Tipton and 135 square miles at the upstream limit of Morse Reservoir. Most of the watershed is in Tipton County. **Figure 1** shows the approximate watershed boundary for Big Cicero Creek**.** Big Cicero Creek is a County regulated drain, as are many of its tributaries.

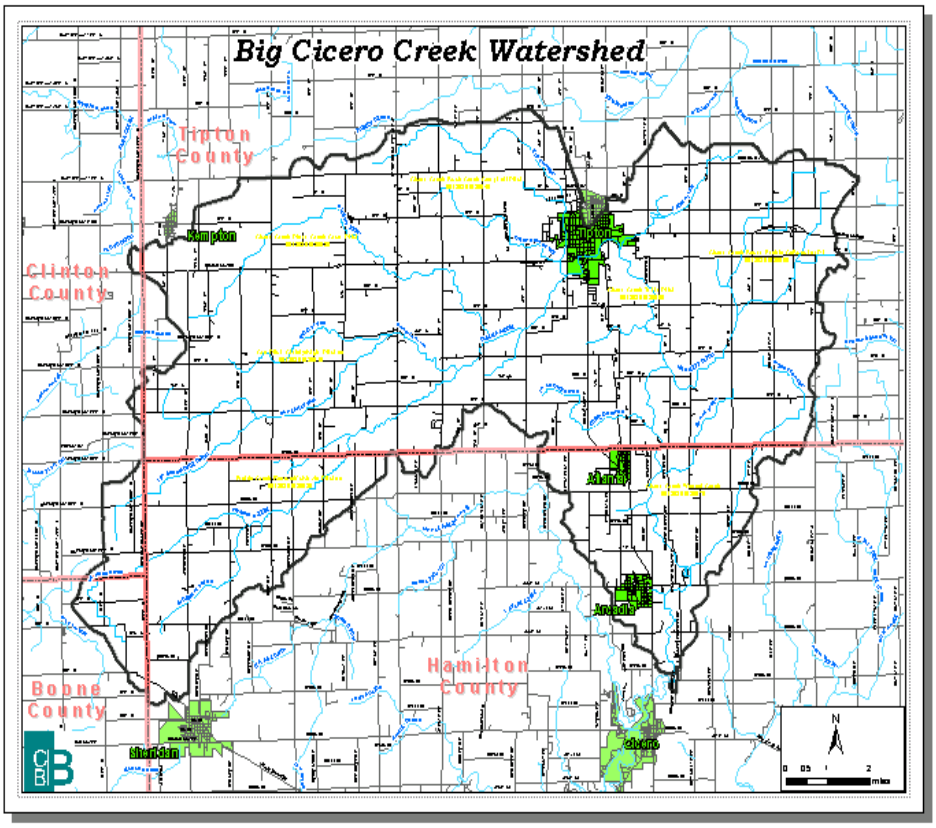


Figure 1: Big Cicero Creek Watershed Boundary

The hydraulic model created with this study extends from about 3,000 feet downstream of Tobin Ditch upstream about 7 miles to CR 500 W. This area is highlighted on the map in **Figure 2**.

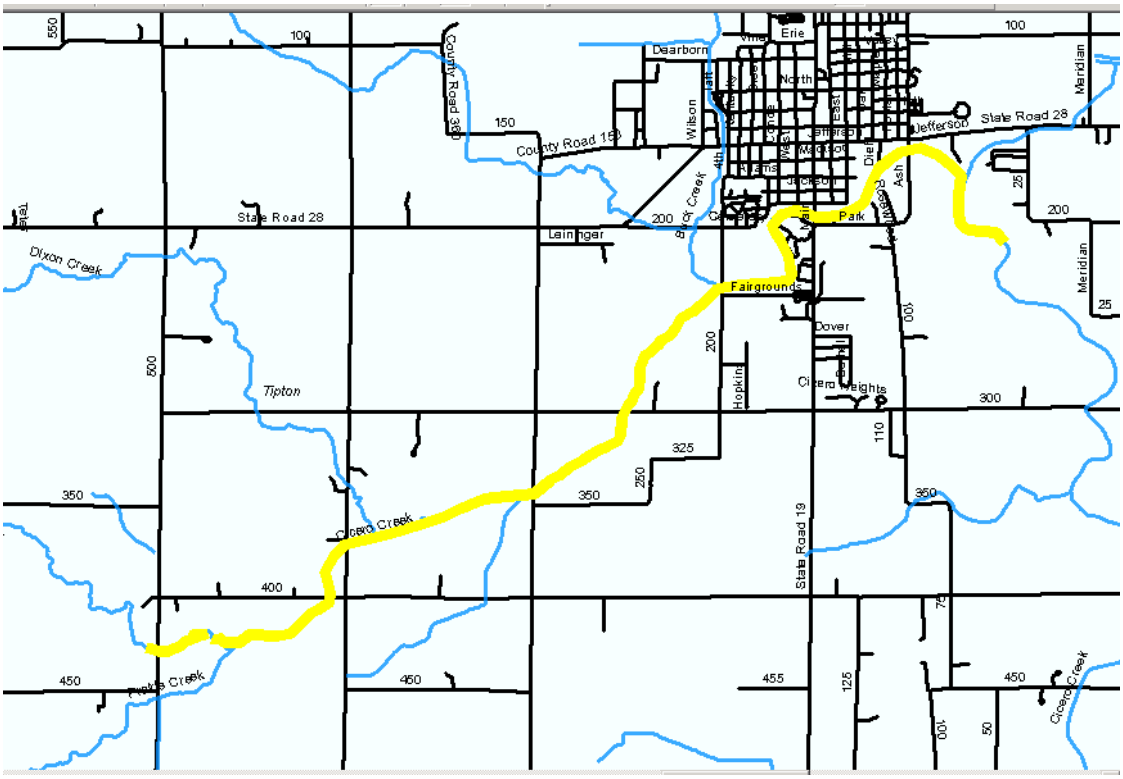


Figure 2: Highlighted Area Shows Reach of Big Cicero Creek Studied

The purpose of this document is to provide the details of the hydrologic and hydraulic modeling done for both the existing conditions and the analysis of various options for flood control analyzed for the project. Beyond the documentation of the work for analyzing flood control alternatives, the existing condition information is provided for submittal to the Indiana Department of Natural Resources (IDNR) as support documentation for both a revision to the Coordinated Discharge curve and for inclusion as a leverage study when the Tipton County Flood Insurance Study (FIS) is updated as part of the mapping initiative project by the Federal Emergency Management Agency (FEMA). For the purposes of this analysis, the 2-, 5-, 10-, and 100- year flood were analyzed. The 10- and 100- year hydraulic information is also provided in a slightly revised form to make it more applicable for use in the future FIS revision.

2.0 HYDROLOGIC ANALYSIS

2.1 INTRODUCTION

This section describes the process used to complete the analysis required to calculate the 2-, 5-, 10-, and 100-year discharges and hydrograph volumes for the study area of Big Cicero Creek. A hydrologic analysis was performed to determine the discharges using a HEC-HMS model. The relationship of drainage area to peak discharges for Big Cicero Creek was established by the IDNR in the Coordinated Discharge Graph titled "Big Cicero Creek", dated 1989. A copy of that graph is provided in **Appendix 1.** Although this regulatory peak discharge data is available, a detailed hydrologic model of existing conditions was needed to evaluate alternatives that would influence the timing or volume of stormwater runoff. The detailed modeling results also showed the need to revise the coordinated discharges. This data could be provided to IDNR for the purposes of revising the coordinated discharges for the stream.

2.2 HEC-HMS COMPUTER MODEL

CBBEL staff developed the hydrologic model for the Big Cicero Creek watershed upstream of the Arcadia gage using the United States Army Corps of Engineers (USACE) HEC-HMS (version 3.0.1) computer model. The model is based on National Resource Conservation Service (NRCS) TR-55 parameters.

2.3 SUBBASIN PARAMETERS

The NRCS dimensionless unit hydrograph method was selected as the basis of the rainfall-runoff generation for individual sub-areas. This method requires the following major inputs for a given subbasin: basin area, NRCS curve number (CN), and NRCS lag time. Other input parameters include the initial loss, percent impervious, and baseflow (if applicable). The development of major parameters is explained below.

2.3.1 Subbasin Drainage Area

Subbasins were delineated using the Hamilton County one-foot contour mapping and the USGS quad 10-foot contour interval mapping in Tipton County. Low areas and swales as seen on aerial photography were used as guides in determining basin divides when contours didn't make basin divides clear. Legal drains (closed or open) were also used as a guide when the contour data was not detailed enough.

Due to the flatness of parts of the watershed and the presence of many depressional areas which potentially drain in multiple directions, the delineations are not exact but should give a reasonable representation of the watershed. **Exhibit 1** includes a map of the subbasin delineations and the names used in the HEC-HMS model.

2.3.2 Subbasin Curve Number

Curve numbers were calculated based on the land use and soil types. The calculations were assisted by GIS tools that combined soil and land use parameters to determine individual curve numbers which were then "lumped" for each subbasin. Land use data was determined using the USGS National Land Use Dataset and soil data was derived from NRCS digital soil mapping. Curve number calculation sheets are included in **Appendix 2**.

2.3.3 Subbasin Time of Concentration and Lag Time

Time of concentration (Tc) is defined as the time required for the entire subbasin to contribute runoff at the subbasin outfall. The Tc calculations were performed by utilizing the TR-55 methodology which considers the ground cover and slopes found in the individual subbasins to calculate travel times for various reaches of a subbasin. Times of Concentration were based on lengths determined from the most detailed of the available contour mapping for each subbasin and the aerial photography used as a guide when topography was sparse. Time of concentration computation sheets for each subbasin are included in **Appendix 3**. Flow paths are shown on the Exhibit 1 map.

An initial estimate of the NRCS Lag Time parameter was calculated by multiplying the value of Tc by 0.6, as recommended by the computer program procedures. A summary of the calculated subbasin hydrologic parameters for the watershed is provided in **Table 1**.

Table 1
Calculated Subbasin Parameters

Subbasin	DA (mi^2)	DA (ac)	Lag Time (mins)	CN
BAtlanta	0.87	557	490.63	85
BBacon	2.21	1414	1393.63	86
BBrown	2.54	1626	1575.735	87
BBuckA	2.18	1395	969.22	87

Subbasin	DA (mi^2)	DA (ac)	Lag Time (mins)	CN
BBuckB	3.55	2272	1405.67	86
BBuckC	3.94	2522	1584.765	86
BBuscher	1.6	1024	893.97	87
BCampbell	4.4	2816	1888.775	87
BChristy	4.53	2899	1476.405	87
BCiceroA	2.67	1709	808.185	84
BCiceroA2	0.96	614	708.855	87
BCiceroB	1.49	954	586.95	84
BCiceroC	2.78	1779	1115.205	86
BCiceroD	1.96	1254	1991.115	85
BCiceroE	2.29	1466	994.805	86
BCiceroF	0.97	621	595.98	87
BCiceroG	2.46	1574	2188.27	87
BCoxDS	3.26	2086	1173.9	87
BCoxUS	4.47	2861	1345.47	87
BDixonA	3.83	2451	1464.365	87
BDixonB	4.1	2624	2483.25	86
BDixonC	2.65	1696	1265.705	86
BDunham	2.35	1504	1647.975	86
BEndicott	3.14	2010	1386.105	87
BEssig	3.73	2387	1348.48	86
BFolkner	3.49	2234	2180.745	87
BGarhart	2.43	1555	1089.62	87
BGordon	1.75	1120	1431.255	86
BGoxes	3.7	2368	1322.895	86
BHaskett	2.17	1389	4068.015	86
BHinde	2.8	1792	2617.195	86

Subbasin	DA (mi^2)	DA (ac)	Lag Time (mins)	CN
BJohnson	1.32	845	651.665	86
BKemp	1.71	1094	1092.63	86
BKigin	5.93	3795	2189.775	87
BLynch	2.57	1645	754.005	85
BMcKinzie	5.26	3366	1635.935	86
BMcMullen	2.74	1754	705.845	87
BPierce	5.3	3392	1438.78	87
BPrairieA	5.52	3533	1541.12	87
BPrairieB	4.17	2669	1604.33	87
BRecobs	1.72	1101	1476.405	86
BSowers	1.29	826	790.125	87
BStone	3.53	2259	1887.27	86
BTobinA	1.34	858	1410.185	86
BTobinB	1.85	1184	1134.77	86
BWeasel	3.79	2426	1607.34	87

2.3.4 Reach Routing

For reach routing in the watershed, Muskingum-Cunge 8-point channel routing parameters were derived from representative cross-sections from the two-foot contour mapping of Big Cicero Creek, Hamilton County one-foot contour mapping, and drain reconstruction plans for Cox Regulated Drain, Dixon Creek, Crum Drain, and tributaries. If none of these existed for a reach, data from the closest reach was used. Based on preliminary calibration efforts for the hydraulic model, Manning's N values of 0.05 for the channel and 0.06 for the overbank were used for the reaches on Big Cicero Creek itself. 0.035 and 0.05 were used for the channel and overbank, respectively, on other reaches as originally chosen based on general site conditions.

2.4 CALIBRATION STORM PARAMETERS

For calibration purposes, the June 1957, January 1962, March 1963, August 1973, and June 1980 rainfall data from the National Oceanic and Atmospheric Administration (NOAA) for gages in Burlington, Tipton, Anderson, Lebanon, Hartford,

Noblesville, and Frankfort was obtained and entered in the model. A copy of the available rainfall data is included in **Appendix 4**. The historic discharges used for model calibration were taken from the USGS gage records. A copy of those gage records is included in **Appendix 5**. Hydrograph volumes for the historic storms were calculated from the daily mean discharges at the gage from the historic flood records. A copy of these records is included in **Appendix 6.**

2.5 CALIBRATION TO HISTORIC FLOODS

Parameters that could be adjusted to achieve calibration were the curve number and time of concentration. Curve number affects the storm volume. Time of concentration affects the storm peak. Various calibration efforts produced a wide range of results. No combination of calibration efforts could reproduce the gage data for every storm.

Upon further investigation, the 1973 storm was dropped since the rainfall was higher than other floods yet the gage discharge was much lower. The June 1980 flood was dropped since no daily mean discharges were available for calculating the hydrograph volume at the gage. The January 1962 and June 1957 floods are the same ones that have high water marks so they were kept. The March 1963 flood appears similar to the January 1962 so it was dropped to simplify the number of runs.

The best results seemed to be achieved when the original calculated time of concentration was multiplied by a factor of 6 and 10 was added to the curve number. 10 more was added to the January 1962 flood curve number in an attempt to match the volume that the gage recorded. The justification for this is that in January the ground would be frozen and act like the curve number were higher. Even though the resulting curve numbers were almost 100, the historic volume at the gage could still not quite be duplicated.

After the public meeting and the number of comments about recent floods, data for the July 2003 and January 2005 flood events were investigated for addition to the analysis. These two floods seem to have been the larger of the recent floods. A new gage at Arcadia was started in 2004 so the January 2005 peak and volume at Arcadia was now available. Peak discharge and daily mean discharges for this flood are included in Appendices 5 and 6, respectively.

For the 2003 flood, IDNR was contacted for any available high water mark information. They found a high water mark at the old gage for the July 2003 flood. This value was added to the discharge versus stage graph for the gage and found to correspond to a discharge of approximately 4400 cfs. The high water mark and

discharge versus stage graph for the gage are provided in **Appendix 7**. Based on the recorded rainfall amounts, this was a much lower discharge than expected.

The July 2003 and January 2005 rainfalls from NOAA data provided in Appendix 4 were added to the model. The January curve numbers were adjusted by the additional 10 as done for the January 1962 rainfall.

The four floods used for the calibration efforts are listed in **Table 2** comparing the calculated with the gage-recorded values. Output for the calibrated model is included in **Appendix 8**. A digital copy of the model is included on a CD in **Appendix 9**.

Table 2

Comparison of Recorded Hydrograph Volumes and Peak Discharges at the Arcadia gage with Calibrated HMS Model results

	January 1962	January 2005	July 2003	June 1957
Q recorded, cfs	3020	4060	4400	6720
Q HMS, cfs	3471	4035	9401	6608
Q % difference	15	-1	-	-2
V recorded, ac-ft	14180	24655	-53	23711
VHMS, ac-ft	11234	23811	31715	22541
V, % difference	-21	-3	-	-5
Gage hydrograph length, days	4	5	NA	4
HMS hydrograph length, days	3	5	3	3
Avg. RF, in., duration	2-3, 12 hr	3.6 in 3 days	5.2 in 12 hr (5.7 in 24 hr)	4-5, 24hr

2.6 DESIGN STORM

As an additional check on the calibration, the design frequency rainfalls were input to the model in addition to the historic rainfalls and the resulting peaks and volumes compared to the frequency analysis of the gage values. A copy of the frequency analysis of the gage is included in **Appendix 10.** Precipitation depths and temporal distributions for twenty-four and forty-eight hour storm durations were chosen for this analysis from NOAA Precipitation Frequency Atlas 14, as summarized in **Table 3** and Appendix 4. A critical duration analysis showed the forty eight-hour storm duration to be the critical duration storm at the Arcadia gage. The 24-hour storm

duration controls in the City of Tipton area. 100-year discharges from the 24 hour or 48 hour storms are, however, relatively close at the Arcadia gage. Because of this, the 24 hour storm was used in the evaluation of alternatives.

Table 3

Precipitation Frequency Estimates (inches) from NOAA Atlas 14

(with aerial adjustment per NWS TP 49)

Storm Duration	Average Recurrence Interval				
	2-yr	5-yr	10-yr	50-yr	100-yr
24 hours (areal adjustment = 92.5%)	2.68	3.33	3.87	5.21	5.85
48 hours (areal adjustment = 94%)	3.17	3.90	4.49	5.95	6.63

The frequency volumes were determined by doing an FFA analysis of the one-, three- and seven- day mean discharges for the period of record as published in "Statistical Summary of Streamflow Data for Indiana". A copy of this data is provided in **Appendix 11**. The seven day mean was used for the 48 hour storm volumes. The three day mean plus 11% to create a four day mean was used for the 24 hour storm volume estimates. The 11% was based on the volume of the fourth day volume of actual storm hydrographs. A comparison of actual and modeled volumes and peak discharges is provided below in **Table 4**.

Table 4

Comparison of Frequency Flood Hydrograph Volumes and Peak Discharges at Arcadia gage for HMS Model Calibrated to Historic Floods Compared to FFA Analysis of Gage Data

	100 yr -24 hr	100 yr – 48 hr	10 yr – 24 hr	10 yr 48 hr	2 yr -24 hr	2 yr- 48 hr
$Q_{FFA, CFS}$	6460	6460	3430	3430	1850	1850
$Q_{HMS, CFS}$	8827	8840	4969	5305	2892	3186
Q % difference	37	37	45	55	56	72
$V_{FFA, ac\text{-}ft}$	27410	34988	16579	22770	9973	14440
$V_{, HMS, ac\text{-}ft}$	29585	35758	16865	21421	9644	12971
V, % difference	8	2	2	-6	-3	-10
Coor. Q, cfs	6700	6700	3400	3400	--	--

Because of the big difference between coordinated discharges (which are approximately the same as the FFA result discharges) and HMS discharges shown in Table 4 above, further investigation was done to determine if the coordinated discharges needed to be revised. Several sources were investigated including the Purdue Regression equations, the 95% confidence limit values from those equations, Flood Frequency Analysis (FFA) program run for the Arcadia gage records for the continuous portion of the record, and the 5% exceedance value from the FFA analysis. **Table 5** shows the values from these sources. The more detailed information upon which these values are based is included in **Appendix 12.** (The FFA output is provided in Appendix 10.)

TABLE 5
Comparison of Frequency Discharges Results From Various Sources

frequency	coordinated	Purdue Regression	Purdue Regression 95% limit	FFA	FFA 5% ex.	HMS calibration to FFA 5% ex.
Gage at Arcadia (DA = 131 sq. mi.)						
100 yr	6700	6940	10,300	6460	8390	7854
10 yr	3400	4360	6480	3430	4220	4467
2 yr	-	-	-	1830	2110	2672
D/S Buck Creek (DA = 80 sq. mi.)						
100 yr	4300	4640	6880	-	-	5342
10 yr	2500	2910	4340	-	-	3063
2 yr	-	-	-	-	-	1797
D/S Tobin Ditch (DA = 87 sq. mi.)						
100 yr	4650	4970	7370	-	-	5694
10 yr	2700	3110	4640	-	-	3261
2 yr	-	-	-	-	-	1914

Based on these results and the fact that there have been at least two floods in the last three years larger than all but the highest of discharges in the gage records (January 2005 discharge at the gage was 4060 cfs and the July 2003 discharge was estimated at 4400 cfs) it was decided that the 5% exceedance value from the FFA analysis would be the most defendable of the possible range of values to propose. With these values as the target values, the HMS model was calibrated further to create a model for the frequency storms and subsequent evaluation of alternatives.

At first, calibration of the 48-hour duration storm was attempted since it was the critical duration storm at the gage. To calibrate the 100-year 48-hour storm required

a multiplier of 9 for the time of concentration. When calibration of the 10-year was attempted, a multiplier of 7 was required. To avoid having different multipliers and because the previously calibrated discharge for the 100-year storm was almost the same regardless whether a 24- or 48-hour duration was used, the 24-hour storm was used for "calibration" Both the 10- and 100- year storms calibrated within reason using a multiplier of 7 for the Tc. The "calibrated" frequency discharges are also included in Table 5.

A copy of the output from the existing-condition "frequency calibrated" HEC-HMS modeling is provided in **Appendix 13.** Resulting frequency calibration model discharges for selected locations in the hydraulic model study reach are summarized in **Table 6**.

Table 6

Summary of "Calibrated" Peak Frequency Discharges at Select Locations in Study Reach

Location along Big Cicero Creek	Total Drainage Area (mi^2.)	River Mile	2-year Discharge (cfs)	10-year Discharge (cfs)	100-yr Discharge (cfs)
Just upstream of Prairie Creek	20.65	32.408	474	811	1402
Just downstream of Prairie Creek	44.01	32.407	1069	1830	3145
Just downstream of Dixon Creek	60.97	21.269	1404	2400	4157
Just upstream of Buck Creek	65.69	30.524	1483	2531	4404
Just upstream of Tobin Ditch	81.72	28.897	1832	3123	5449
Just downstream of Tobin Creek	87.08	26.800	1914	3261	5694

The values from Table 6 above were plotted. A line was drawn parallel to the HMS results and thru the FFA 95% confidence limit discharge at the gage. These lines are shown in **Figure 3** below. Discharge values were then taken from this line for use in the hydraulic modeling. The selected values are noted below in **Table 7.**

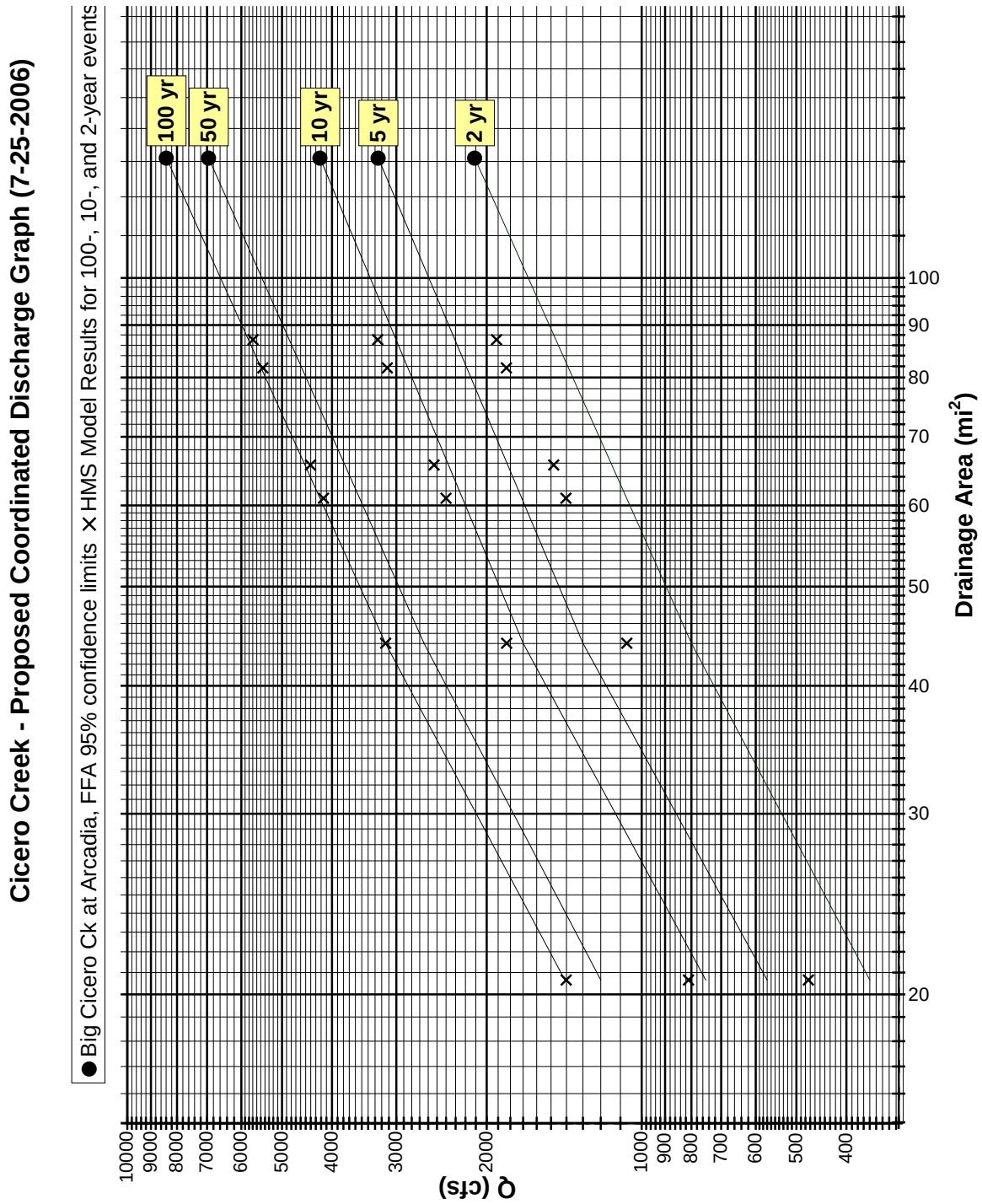


Figure 3: Proposed coordinated discharges and other frequencies used in analysis

Table 7

Summary of gage FFA 5% at HMS slope Discharges at Select Locations in Study Reach

Location along Big Cicero Creek	Total Drainage Area (mi^2.)	River Mile	2-year Discharge (cfs)	10-year Discharge (cfs)	100-yr Discharge (cfs)
Just u/s of Prairie Ck	20.65	32.408	360	750	1400
Just d/s of Prairie Ck	44.01	32.407	800	1700	3150
Just d/s of Dixon Ck	60.97	31.269	1070	2220	4220
Just u/s of Buck Ck	65.69	30.524	1150	2380	4550
Just u/s of Tobin D	81.72	28.897	1400	2850	5500
Just d/s of Tobin D	87.08	26.800	1480	3000	5800

The results of the “frequency calibrated” HMS hydrologic model will be considered “existing conditions” and used as the basis for evaluating flood reduction alternatives. A compact disk with a digital copy of the model is provided in Appendix 9.

3.0 HYDRAULIC ANALYSIS – EXISTING CONDITIONS

3.1 INTRODUCTION

This section describes CBBEL's development of a new hydraulic model of existing-conditions along Big Cicero Creek. The development of additional modeling to analyze applicable alternatives described in Volume 1, Chapter 6 is detailed in section 5 of this chapter.

CBBEL first reviewed regulatory floodplain information and the FIS hydraulic model provided by the IDNR. Copies of the regulatory FIS mapping for Big Cicero Creek, along with effective profiles and a copy of the FIS model output are provided in **Appendix 14** for reference. Using the Army Corps of Engineers HEC-RAS (version 3.1.1), a new hydraulic model was developed from about 3,000 feet downstream of the Tobin Ditch confluence upstream to County Road 500 West to calculate existing flood profiles and also to evaluate the impact of proposed solutions to the flooding problems.

3.2 HEC-RAS INPUT DATA

The nature and source of information that was used to develop the models required for the analysis of this reach of Big Cicero Creek are provided below.

3.2.1 Topographic Base Map

For a topographic work map, CBBEL used the 2-foot contour map of the study reach generated by EarthData. The delineation of floodplain boundaries was done using this map. The map datum is NAVD '88.

3.2.2 Study Reach

The study reach for the hydraulic model began at about 3,000 feet downstream of the confluence of Tobin Ditch, and extended upstream approximately 7 miles to just upstream of CR 500 W.

3.2.3 Flow Data

Flow data required for simulation of the historic, 2-, 5-, 10-, and 100-year events were obtained from the proposed new coordinated discharge curve discussed in Section 2 of this Volume. **Table 8** shows values for the 2-, 5-, 10-, and 100-year frequency floods.

Table 8
Peak Discharges Used in HEC-RAS Model

Upstream Section of Cross Section Range	Peak Discharges for Various Design Storm Recurrence Intervals (cfs)			
	100-yr	10-yr	5-yr	2-yr
33.227	1,400	750	570	360
32.554	3,150	1,700	1,300	800
31.269	4,220	2,220	1,700	1,070
30.524	4,550	2,380	1,820	1,150
28.897	5,500	2,850	2,200	1,400
26.800	5,800	3,000	2,300	1,480

Calibrating the model to the June 1957 and January 1962 floods required approximate discharges at various locations from these events. These discharges were determined by plotting the HEC-HMS results for these floods on log-log paper and drawing a line parallel to the HEC-HMS results through the respective Arcadia gage discharge value. These values are shown in **Figure 4**. **Table 9** below lists the HEC-HMS discharges and the discharges adjusted to create historic discharges for the reach of the calibration model. The table also lists the cross sections in HEC-RAS to which these historic flood discharges were applied.

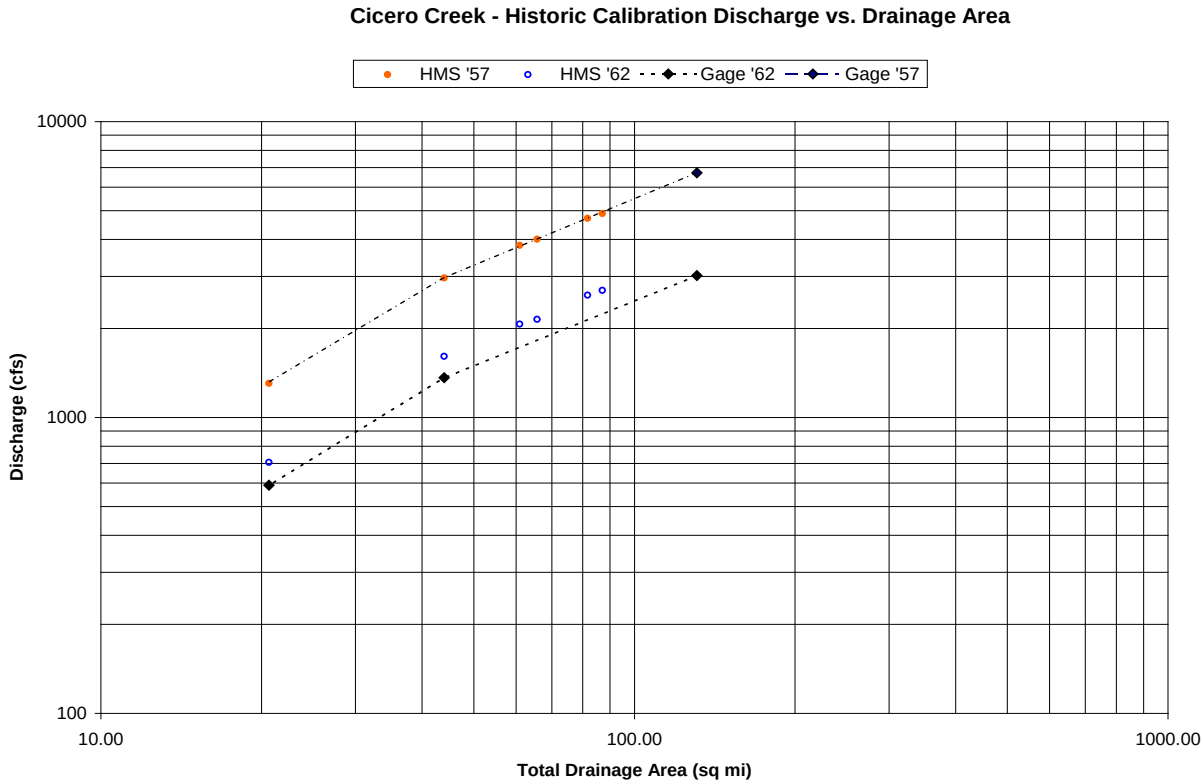


Figure 4: Graph used for Determination of Historic Discharges for Hydraulic Model

TABLE 9
Historic Discharges at Selected Locations for HECRAS Model

Location	DA	RM	Gage '62	Gage '57	HMS Q_{62}	HMS Q_{57}	Reach
JUS Prairie	20.65	32.877	(590)	(1304)	705	1305	32.877 - 33.227
JDS Prairie	44.01	32.407	(1365)	(2963)	1610	2964	31.541 - 32.407
JDS Dixon	60.97	31.269	(1685)	(3823)	2070	3821	30.753 - 31.269
JUS Buck	65.69	30.524	(1750)	(4013)	2146	4005	29.015 - 30.524
JUS Tobin	81.72	28.897	(2150)	(4722)	2591	4711	27.066 - 28.897
JDS Tobin	87.08	26.800	(2300)	(4903)	2688	4890	26.269 - 26.800

3.2.4 Starting Water Surface Elevations

The starting water surface elevations for the 2-, 5-, 10-, and 100-year events were set at “known” water surface elevations. Initially, a slope of 0.0009 based on the contour map was used. This slope to determine starting elevations did not allow the resulting profile to pass through the downstream most high water mark. In order to

pass through this mark, the start elevations for the 1957 and 1962 floods were taken from the high water mark profile. The start elevations for the frequency floods were then interpolated based on discharge differences compared to the 1962 and 1957 flood discharges.

3.2.5 Manning's Roughness Coefficients

Manning's roughness coefficients for Big Cicero Creek were originally chosen between 0.04 and 0.10 for the overbanks and 0.04 and 0.06 for the channel based on field visits and aerial photography. These values proved to be too low to produce a reasonable match with the 1962 and 1957 high water marks from IDNR. In order to achieve a reasonable calibration, Manning's N values were changed to 0.035 – 0.08 in the channel and 0.045 to 0.5 in the overbanks. The lower values were used in the park/golf course area. The higher values were used where there were a lot of obstructions such as buildings, knolls, or bends to obstruct the flow path.

3.2.6 Cross Section Geometric Data

Cross-section geometric data was taken from the 2-foot topographic base map with NAVD '88 datum. The USGS quad maps were used for additional data beyond the limits of the 2-foot contour mapping. The HEC-GeoRAS extension in ArcView GIS was used as a tool to electronically cut the cross-sections. The thalweg for each of the cross-sections was determined by using channel cross sections surveyed by Hamilton County Surveyor Office staff. All cross sections were aligned to best represent a perpendicular orientation to flow. A copy of the map showing the location and orientation of cross sections is provided as **Exhibit 2**.

3.2.7 Lateral Structures

Under existing conditions for the 10-year and lower elevations flood, water backs up Prairie Creek and the tributary along the east side of CR 500W then breaks across fields towards Garhart Ditch (upstream of Section 30.524) before rejoining Big Cicero Creek. To take this into account and be able to evaluate alternatives that might change this overflow path, a "lateral structure" was placed in the model just downstream of the confluence of Big Cicero Creek and Prairie Creek (section 32.554).

A rating curve for this "structure" was developed by creating a short HECRAS model for the overflow portion and running several discharges through it. Lack of detailed topographic information prevented creating a detailed model of the overflow path. Discharge versus elevation results at the upstream section of the overflow path model were input as the lateral structure rating curve.

Cross sections upstream of the “lateral structure” extended only to the effective flow area for Big Cicero Creek itself. Downstream of the lateral structure, the flows were separate until rejoining the main channel downstream of section 30.753. In between, ineffective limits were placed on sections on the main stem cross sections to prevent flow in the area assumed to be used by the “lateral structure” overflow from the CR 500 W tributary.

For the 2-, 5-, and 10-year floods that were of interest in this reach of the model, the separate flow path assumption was valid. However, for the 100-year flood, the overflow and main channel floodplains are connected through the reach making the assumption invalid.

Because the overflow path discharge is not that significant, and it was desirous to create a model that could be used for FIS purposes without having the 10- and 100-year flood events requiring different models, revisions were made to create a model appropriate for FIS use. The revisions included removal of the “lateral structure” and revision of the ineffective flow limits for cross sections from Garhart Ditch upstream to CR 500 W (sections 30.753 – 32.877) to account for the additional available flow area that was separate in the more frequent, lower floods. The input for this model is provided on disk in Appendix 9**.** The difference of up to 0.4 feet between the 100-year elevation from the model with and without the lateral structure is not significant enough to show up on the 2-foot contour mapping so the floodplain delineation for either model is essentially the same.

3.2.8 Ineffective Flow and Blocked Obstructions

Ineffective flow limits were used upstream and downstream of bridges for the purposes of modeling a restriction of flow by the bridges and their approaches, per IDNR guidelines. Many permanent ineffective flow areas were added for calibration purposes to reflect the many knolls in the overbanks. Multiple blocks and heights were used to reflect this “shadowing” of upstream and downstream topography. Ineffective flow was also used at section 30.753 and upstream to limit the 2-year through 10-year flows to the area connected with the channel since the lateral structure was used to model flows in the separate overflow path along Garhart Ditch.

3.2.9 Bridges

Bridge information was taken from the Tipton County Bridge Inventory, survey by the Hamilton County Surveyor Office, INDOT plans for the SR 19 bridge, and field measurements. Bridge roadway data was taken from the 2-foot topographic work map and combined with the bridge opening data.

3.2.10 Critical Depths

There were no critical depths calculated by this model.

3.2.11 Calibration

High water marks for the June 1957 and January 1962 floods were available from IDNR. These marks were therefore used for calibrating the model. Calibration was done by changing Manning's N values and adding many permanent ineffective flow areas to account for lots of knolls in the overbanks. Multiple ineffective flow areas were added to most sections in the reach upstream of CR 200W.

A profile showing the comparison of the calibrated profile and the high water mark elevations is included in **Figure 5**. The marks were matched within 0.8 ft. Because the two floods are near the 100-year flood and the 5-year flood, respectively, having both models calibrate to this extent made them able to be considered a reasonable representation of the stream. A digital copy of the calibration model is included on disk in Appendix 9. A copy of the output is included in **Appendix 15**.

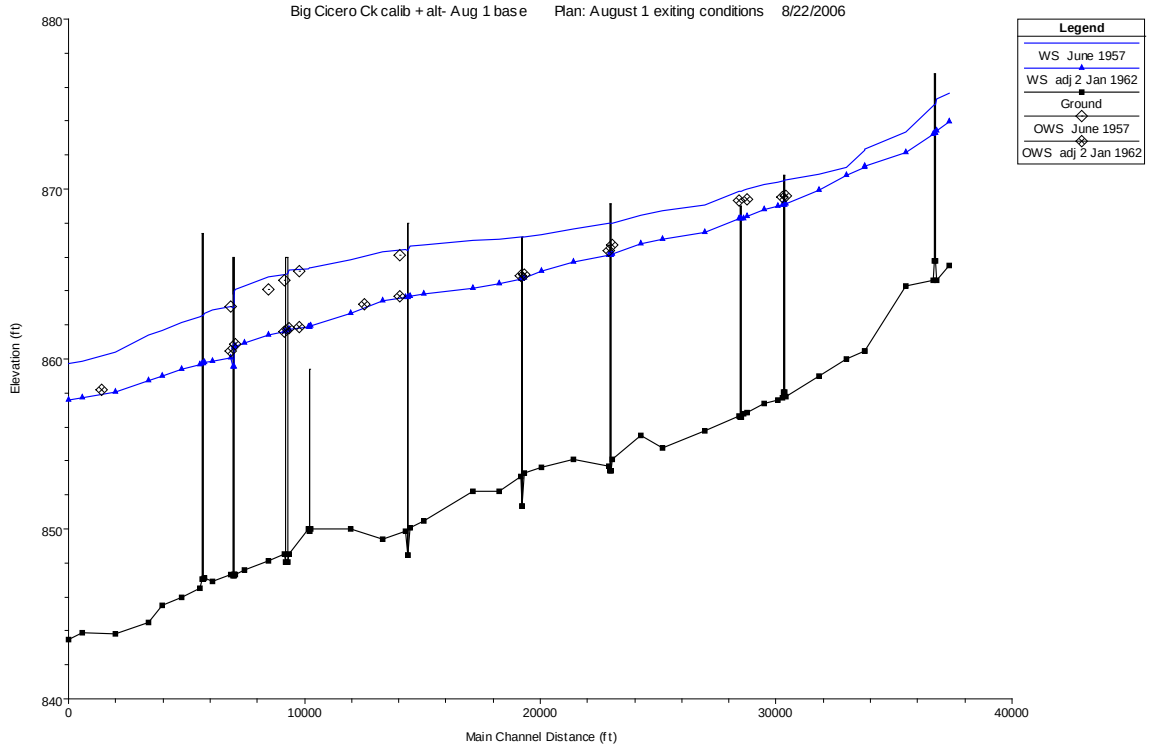


Figure 5 : Calibration of January 1962 and June 1957 floods

3.2.12 Floodway

The floodway was not determined as a part of this project. It is expected that from the downstream end of the model upstream to about CR 200 W, the model could be used to determine a floodway. However, upstream of that point, many ineffective flow areas were defined to mimic the knolls in the fields. This reach will require special consideration to properly define the floodway limits.

3.2.13 Check RAS

The FEMA Check RAS program was run to check the model. A copy of the output from Check RAS for the model proposed for use in the FIS is provided in the IDNR Modeling Guidelines Checklist in **Appendix 16.** The output has been annotated with remarks to explain the warning messages.

3.3 FINAL RESULTS OF EXISTING CONDITION HYDRAULIC ANALYSIS

Once the model was calibrated, the frequency discharges were added. The resulting floodplain limits for the 2-, 10-, and 100- year floods are displayed on **Exhibit 2**. It should be noted that some reaches along the south side of Big Cicero Creek and the west side of Buck Creek were not included on the 2-foot contour mapping. Delineations in these areas are therefore approximate and were based in part on wet areas as shown on the 2005 aerial mapping. **Exhibit 3** includes the limits of the current FIS floodplain and the 100-year floodplain from this modeling for comparison. (The 100-year floodplain for the project model and the revised model for the FIS are the same delineation.) A copy of the output from the existing-condition HEC-RAS modeling for the project and as revised for FIS use is provided in Appendix 5. A digital copy of the HEC-RAS model is provided on the compact disk in Appendix 9**.** Resulting water surface elevations at key locations in the project model are summarized in **Table 10**.

Table 10
Summary of Existing Condition Flood Elevations

Location along Big Cicero Creek	Station (mile)	2-yr Water Surface Elevation (ft, NGVD)	10-yr Water Surface Elevation (ft. NGVD)	100-yr Water Surface Elevation (ft, NGVD)
At pedestrian bridge at pool in Tipton Park	28.104	859.69	863.15	865.65
At confluence with Buck Creek	28.897	861.97	864.82	867.07
Upstream of CR 300 South	29.819	863.49	865.64	867.59
Upstream of CR 400 West	31.922	868.45	869.48	870.74

A copy of the proposed FIS Flood Profiles is provided in **Exhibit 4**. A copy of the completed IDNR computer modeling checklist for the proposed FIS model is contained in Appendix 16. A copy of the floodplain, contour, and cross section GIS shape files is also included on the disk in Appendix 9.

3.4 EVALUATION OF FLOOD CONTROL ALTERNATIVES

Using the existing condition project model described above, the alternatives described in Volume 1 were modeled. To aid in the evaluation of the alternatives, approximate elevations at which water would begin flooding fields were determined from the 2-foot contour mapping and entered as known water surface elevations at most cross sections. This provided a way to graphically determine if alternative results were within the channel. A copy of the resulting profile is shown below in **Figure 6**.

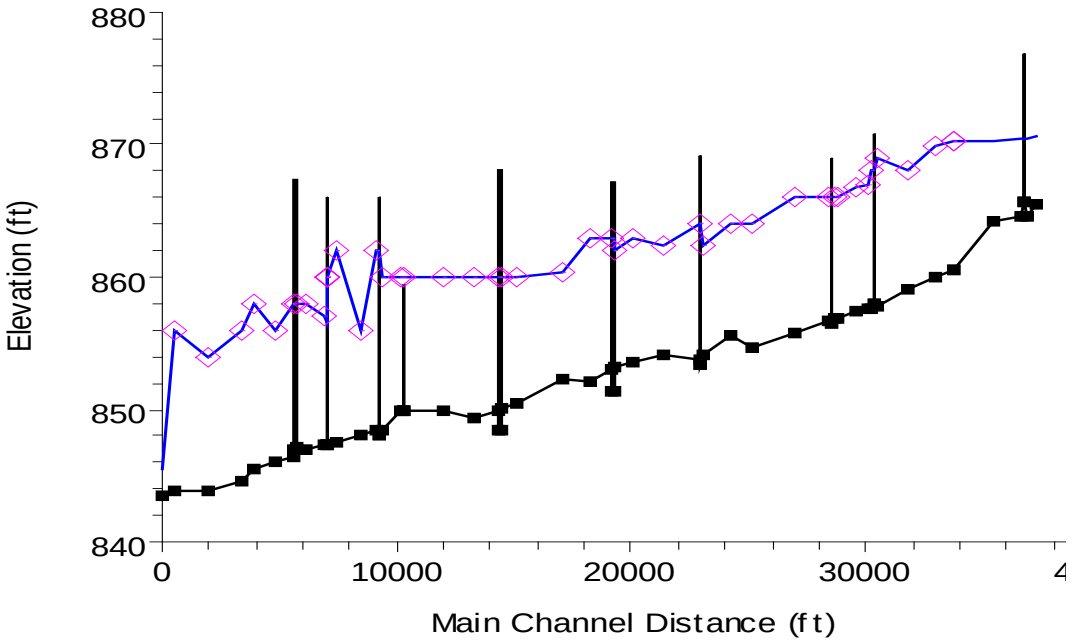


Figure 6: Approximate bank elevations at which overbank flooding begins to be a factor.

In order to evaluate the success of each of the alternatives in meeting the criteria, the study reach was divided into seven reaches. The reach limits generally correspond to each of the north-south roads that cross the stream. The exception is the reach along Buck Creek where backwater from Big Cicero Creek causes additional flooding.

In reach 1 (Ash St – Main St.) the lowest structure appeared to be at an elevation of 860 feet, NAVD ’88. It appeared to be the only structure so low. Others appeared to be above at least 861 feet, NAVD ’88. Section 27.761 was selected to represent this reach. If the 100-year water surface elevation at this section was 861 or less for an alternative, it was rated as successful. Elevations of 862.6 and 865.1 feet, NAVD ’88, the 10- and 100-year existing condition elevations, respectively, were the elevation used to determine the lesser protection ratings shown in **Table 12** at the end of this chapter.

In reach 2 (Main Street – CR 200 W) 862.0 feet, NAVD ’88 is the approximate elevation of the lowest structure. Elevations at Section 28.086 near this structure were used to determine the level of success of each alternative. A 100-year water surface elevation of 862 or below was considered to have met the goal. The existing condition 10- and 100- year elevations are 863.1 and 865.6, feet, NAVD ’88 respectively, and were used to determine the lesser protection levels in the reach.

For reach 3 (areas along Buck Creek) all structures appear to be above 863.4 feet, NAVD '88. The cross section applicable for water surface elevations affecting this area was 28.897. The existing condition 10- and 100-year elevations were 864.8 and 867.1 feet, NAVD '88, respectively.

For reaches 4-7 (CR 200W – CR 500W) the 10-year water surface elevation was compared to the plot of the low bank elevations to determine if the project goal was met by an alternative. The lesser levels of protection were defined by the 2- and 5-year profiles of the existing conditions.

3.4.1 Modeling of Flood Control Alternatives

A copy of the HEC-RAS model for each of the alternative described is included on disc in Appendix 9.

Alternative 1, **Option A** was modeled using the channel improvement option in HECRAS between sections 26.158 and 28.685 centering the cut in each cross section with a bottom width of 200', side slopes of 2:1 and a Manning's N value of 0.045. Channel inverts and bridge openings were unchanged from existing conditions. In reality, a low flow channel would have to be added.

Alternative 1, **Option B** was the same as Alternative 1 A except the bottom width was changed to 100 feet.

Alternative 1, **Option C** was just like Alternative 1 A except only from section 27.311 thru 28.685.

Alternative 1, **Option D** is the straightening of the channel from Main Street to CR 200 W. Presently, the channel flows in a "Z" shape between the two roads. Alternative 6 would make it a straight line between the two bridges. To model this in HECRAS, the total distance from sections 27.927 to 28.868 was changed to 3200 feet. Sections 28.104 and 28.428 were deleted. The park bridge was deleted assuming a new structure would be built that would not affect water surface elevations. Sections 28.685 and 28.086 were revised by the channel improvement option to a bottom width of 40 feet and side slopes of 2:1 and the inverts changed to agree with the slope from section 27.927 to 28.868. The old channel was filled in but still provided a swale for flow. Manning's N values in the channel stayed at 0.045. The same value was used for the overbanks in the park. Overbanks in the golf course were set at 0.08, an improvement from existing conditions based on the calibration efforts.

Alternative 1, **Option E** evaluated a channel improvement for the reach of Big Cicero Creek from the downstream side of the railroad to downstream of the Tobin

Ditch confluence. The channel improvement option was used on cross sections 27.311 – 26.533. It created a 30' wide shelf on the right bank at an elevation 3' above the main channel invert. No modifications were made to the bridge internal sections. Ineffective flow limits based on the bridge opening in the existing condition model were applicable to this alternative as well so were not changed.

Alternative 1, **Option F** was created by using the channel improvement option from CR 200W upstream thru section 32.407. The invert for the channel improvement was selected at 2 feet above the existing invert in order to maintain a low flow channel. A bottom width of 130 feet and side slopes of 2:1 were selected. The channel improvement was centered on the channel. A Manning's N value of 0.045 was selected for the improved channel. Bridge openings were unchanged.

Alternative 1, **Option G** was a combination of alternatives 1 A and 1 F.

Alternative 1, **Option H** consisted of a channel improvement from about 2,000 feet downstream of CR 300 S to CR 300 W. The channel improvement option was used for sections 29.613, 29.959, and 30.208. This created a shelf on the right bank 20 feet wide at an elevation 3' above the existing channel invert.

Alternative 1, **Option I** was created by adding alternative 1, Option D modeling downstream of this reach to a copy of the Alternative 1, Option H model.

Alternative 1, **Option J** represents a channel improvement from CR 200 W to CR 400W. This was modeled using the channel improvement option in HEC-RAS. From sections 29.015 – 31.269, a 60-foot wide "shelf" was cut on the left side of the cross section with the shelf elevation set at the channel invert plus 3 feet. No modifications were made to the bridge internal sections. Ineffective flow limits based on the bridge opening were moved to the edge of the modified channel so are slightly wider than would be expected if no modifications are made to the bridge openings.

Alternative 1, **Option K** consisted of copying the model for Alternative 1 J and adding the same 60-foot shelf channel improvement from about 2,800 feet downstream of Tobin Ditch upstream to Main Street. The channel improvement option was used for sections 27.575 – 27.894 and 26.533 – 26.80 assuming the work would be on the left bank and 26.910 – 27.461 and 27.485 – 27.5 assuming the work would be on the right bank. The shelf was placed at an elevation 3 feet above the channel invert at each section. The Manning's N value for the shelf portion was set at 0.045. All other parts of the section remained unchanged from the existing condition values. Side slopes for the new bank were set at 2:1. Excavation quantities were determined from the cut and fill calculations done by the channel improvement option in the model when applied to the existing condition model.

Alternative 1, **Option L** consisted of copying the model for Alternative 1 K and using the channel improvement option to add channel modification from section 28.685 – 28.868 on the left bank, 27.927 – 28.104 on the right bank and split between the two banks at section 28.428. The parameters for the channel modification were the same as used in Alternative 1 K. Excavation quantities were determined from the cut and fill calculations done by the channel improvement option in the model when applied to the existing condition model.

Alternative 1, **Option M** was then created by using the channel modification option to create revised cross sections from 26.533 – 31.541. This time, the channel improvement was the same as in Alternative 1 L except that it was only a 30' wide shelf. The revised sections and the excavation amounts were determined in the same manner as for Alternative 1 L.

Finally, **Alternative 1**, **Option N** was created by copying Alternative 1 M and continuing the channel improvement option up to section 31.893. Ineffective flow limits due to expansion and contraction were not changed since the bridge openings will not be changed.

A check was made on whether Buck Creek elevations would become the control elevations if Big Cicero Creek water surface elevations were lowered per Alternative 1, Option N. A HECRAS model used for a floodplain elevation determination was retrieved from the IDNR website. It appeared to be a copy of the Section 22 HEC-2 model that would have been used for the FIS. The 10-year discharge noted in the FIS book of 620 cfs upstream of Cemetary Road, was input to this model. The resulting Buck Creek 10-year water surface elevation did not exceed the Alternative 1 N 10-year elevation until upstream of Jefferson Street (SR 28). This is upstream of the area that this project is seeking to benefit. Big Cicero Creek 10-year elevations will still determine 10-year flood elevations downstream of Jefferson Street.

Alternative 2, **Option B** levees were drawn with generally a 140' - 200' spacing between them. The levee feature in Geo RAS was used and then imported to HECRAS. Ineffective flow limits that were inside the levees were removed. An unsteady flow model of the levee system was also created to evaluate the affects on elevations downstream of the levee. The steady flow models of existing conditions and of the levees were each truncated to the section downstream of the levees and the rating curve for that section from the stready flow model input as the downstream boundary condition (with additional assumed points to allow the model to run). Due to problems balancing calculations, the bridges were removed and all ineffective flow areas were changed to permanent. Hydrographs from the existing condition 10-year HEC-HMS model were linked to the appropriate places in the model as the upstream and lateral hydrographs. This provided a very rough model to evaluate the

levee affect. It showed almost a foot increase in water surface elevations downstream of the levee.

Alternative 3, **Option A** was evaluated by calculating the 100-year 24 hour hydrograph volume from the HECHMS at JDSBuck (just downstream of Buck Creek) above 2,250 cfs.

Alternative 3, **Option B** was the same thing but determining the volume above a discharge of 2,850 cfs.

For **Alternative 3**, **Option C** the volume of the 10-year- 24 hour hydrograph above the "bank full discharge" just upstream of Dixon Creek was calculated as the approximate required volume of storage. Depth of existing condition flow was used as the storage depth available to determine the required basin area. The "bank full discharge" was determined by running a range of discharges through the model and choosing the ones that most closely matched the plot of bank elevations. Each set of discharges chosen was based off a different line drawn parallel to the proposed coordinated discharge curve.

Alternative 3, **Option D** was evaluated by removing the basins for half of the drainage area above Tipton from the hydrologic model. The resulting discharges at points along Big Cicero Creek were compared to existing condition frequency discharges and that profile used to give an idea of to what degree the project goals were met.

In order to model the bypass channel from Garhart Ditch (upstream of section 30.524) to Recobs Ditch (downstream of study reach) for **Alternative 4**, **Option A**, a lateral structure was used in the HECRAS model at Garhart Ditch confluence to divert the water out of Big Cicero Creek. The lateral structure was defined as a rating curve. The values for the rating curve were determined by creating a short HECRAS model of several section of a trapezoidal channel with a 76' bottom width, 3:1 side slopes, a Manning's N value of 0.04, a slope of 0.001, and an initial depth of 6 feet. It was assumed that the invert at Big Cicero Creek for this bypass channel would be four feet above the Big Cicero Creek invert so that low flows would stay in Big Cicero Creek.

The rating curve for this diversion channel was also put into the 10-yr 24-hr HEC-HMS model as a diversion from Junctions JDSGarhart to JDSRecob to make sure that there was not flow entering the channel from tributaries downstream of Garhart that would make the 10-year flows exceed the channel capacity. The reduction in discharge caused by the diversion was found to be consitent downstream indicating that the assumption of the bypass channel capacity reducing the discharge downstream by that amount was appropriate.

Alternative 4, **Option B** was evaluated in the same way as Option A except that the bypass channel bottom width was 38 feet instead of 76 feet.

The bypass for **Alternative 4**, **Option C** was modeled by adding a reach to the HECRAS model from upstream of section 32.554 to downstream of section 30.753. Section 32.554 was selected to represent the bypass channel being connected to the main channel, Prairie Ck, or the tributary along CR 500 West because the water backs up Prairie Creek and the tributary from the confluence of Prairie and Big Cicero Creeks and thus the confluence elevation is closer to the controlling elevation. The reach consisted of a 14,250 foot long channel, at least six feet deep, 88 foot bottom width, 2:1 side slopes, and a slope of .0005. The optimization feature was used in the model to calculate the amount of flow split between this channel and the existing channel.

Alternative 4, **Option D** combines Alternative 4, Options A and C into one alternative.

Alternative 4, **Option E** evaluated a bypass channel from upstream of the railroad to downstream of the Tobin Ditch confluence. This was modeled by adding a reach from upstream of section 27.50 to upstream of section 26.533. This reach consisted of a trapezoidal channel approximately 2,600 feet long, sloping from 851 to 845, equivalent to over 3' above the main channel invert on the upstream end to almost 1' above the main channel invert on the downstream end. The bypass channel was assumed to have a 30' bottom width with 3:1 side slopes and a depth that varied depending on the natural ground elevations along the path. The natural ground elevations and shape above the proposed trapezoid were roughly estimated from the 2' topographic mapping. It was assumed that any structures that would be needed to carry roads over the bypass channel would be constructed large enough so as to have no impact on the flow elevations.

Alternative 6 was modeled by removing the railroad bridge at mile 27.485 and the obstructions for the interurban piers at sections 27.49 and 27.492.

Alternative 7 was created by removing the CR 300 West bridge and the ineffective flow limits due to that bridge deck at the bounding sections (bridge sections 2 and 3).

Alternative 11, **Option A** was evaluated by looking at the results of the model for Alternative 1, Option F using the "bankfull" discharge used in Alternative 3, Option C of 480 cfs plus the Dixon Creek contribution of 610 cfs. The resulting 1090 cfs is close to the existing condition 2-year flood downstream of Dixon Creek. In Alternative 1, Option F, the 2-year flood elevation is lowered to very close to the bank elevations.

Alternative 11, **Option B** is a combination of alternatives 4C, 1F, and 1A.

Alternative 11, **Option C** sized the detention basin assuming Alternative 1, Option B was constructed downstream. Alternative 1B can handle 4,722 cfs at an elevation of 862.6 at cross section 28.086 (reach 2's key location) and 2,850 cfs at an elevation of 860.1. A discharge of 4,350 cfs was therefore assumed capable of being handled at an elevation of 862, the approximate lowest grade of a structure. The detention basin was then sized based on the volume of the 100-year 24 hour hydrograph above the 4,350 cfs for Alternative 11 C.

Alternatives 2A, **5**, **8**, **9**, and **10** are self explanatory. No modeling was done to evaluate these alternatives.

3.4.2 Unsteady Flow Modeling of Flood Control Alternatives

Several alternatives were modeled using the unsteady flow option in HEC-RAS. These unsteady models are not necessarily exact due to the limitations surrounding the modeling methods available. However, they do give a relative idea of the benefit of each alternative, particularly in terms of how long specific areas may be flooded. Input for the HEC-RAS modeling of each of these alternatives is included on disk in Appendix 9.

Input for the flow portion of the unsteady model was the output hydrographs in the DSS files from the HEC-HMS model for the 10-year frequency discharges. A flow hydrograph at the upstream end of the model and lateral inflow hydrographs at key tributary confluences were used. **Table 11** below shows the HEC-HMS hydrograph locations and the point at which they were used in the unsteady flow model.

Table 11
HEC-RAS Sections and Corresponding HEC-HMS Hydrograph Names

Location along Big Cicero Creek	HEC-HMS Hydrograph
32.877	JUSPrairie
32.554	JPrairieM
31.541	JDixonM
30.753	BGarhart
29.015	JBuckM
26.910	JTobinM

At the downstream end, a rating curve was created using the 2-, 5-, 10-, and 100-year steady flow profile elevations and discharges and then adding interpolated points to make a smooth curve as needed for the model to run. There is inherently some room for interpretation in the resulting rating curve. Results are therefore approximate for the downstream most cross section at least. For the initial conditions at the upstream boundary, an initial flow of 200 cfs was assumed since that value was about the lowest that would allow the model to still run.

The geometry files from the steady flow models were imported to the unsteady flow model. A few cross sections upstream of the Prairie Creek confluence were deleted since they are not important for evaluation of the alternatives and their absence minimized potential instabilities in the model.

Alternatives were evaluated by using the option to show the water surface profile at the time interval increments. The water surface at each time increment was compared to the general bank elevation plot and the time that water started to leave each reach was noted and compared to the time it was again within the banks in order to compare the time out of banks for each alternative. The 10-year flood for Alternatives 1B, 1C, 1E, 1F, 1H, 1I, 1J, 1K, 1L, 1M, 1N, 4A, 4B, 6, and 7 were all evaluated in this manner. The 2-year frequency model was also created using the DSS files from the HEC-HMS 2-year frequency model and run for alternatives 4A, 4B, 1H, 1I, 1J, 1K, 1L, 1M, and 1N to determine the difference in reduction of time out of banks between the 10- and 2- year floods. Resulting time out of bank is shown below in **Table 12**.

Table #
Comparison of Alternatives

Alternative	Elevation Reduction Benefit Symbols refer to degrees to which technical goals were met. Numbers means the frequency flood for which the alternative provided the technical goal's level of protection.							Approximate Time out of Bank, hr. CR 200S – 300 W/ 300W – 500W		Maximum increase in downstream 10 yr. elevations, ft.	Costs: Construction costs (Excavation yds.) Land + construction costs	Disadvantages	Other benefits
	1) Ash - Main	2) Main – 200W	3) Along Buck Ck	4) 200W-300S	5) 300S – 300W	6) 300W – 400W	7) 400W – 500W	10yr	2 yr				
0 -Do nothing	(5)	5	(5)	<2	<2	< 2	< 2	104/95	99/93				
1A (1)- 2.6 mi., 200' BW channel improvement thru town	*+ [100]	*+ [100]	*(-) (100)	+- (2)	-0	0	0	X	X	X	$8,079,000 (1,080,000cy) $10.9M	Environmental mitigation not included in cost. Provides essentially no benefit to agricultural lands. May be combined with another alternative to provide agricultural benefits.	Excavated material could be stockpiled and made available for future development
1B (1.5) – 2.6 mi., 100' BW channel improvement thru town	+ 10	+ (-) 10	+ (-) 10	+ (-)	-0 < 2	0 < 2	0 < 2	103/94		X	$4,227,000 (438,000 cy) $5.5 M	Environmental mitigation not included in cost.	
1C (1.2) – Ash – 200W 200' BW channel improvement	- 10	- 10	+ (-) 10	+- < 2	-0 < 2	0 < 2	0 < 2	X	X	X	$4,256,000 (443,000 cy) $5.8 M	Environmental mitigation not included in cost. Alt. 1.5 provides slightly higher benefit with similar cost so this alternative will not be considered further.	
1D (6) –straighten channel Main – 200W	0 (5)	0 5	- 5	- (-) < 2	-0 < 2	0 < 2	0 < 2	Negl.	Negl.	0.22	$2,281,000 (110,000cy) $2.3 M	Environmental mitigation not included in cost. Accomplishes very little in terms of the goals and increases stages downstream. This alternative will not be considered further.	
1E (19) = 30' shelf channel improvement d/s RR – d/s Tobin Ditch	- 5	- [10]	0 (5)	0 < 2	0 < 2	0 < 2	0 < 2	93/95	X	X		Provides minimal benefits for the cost. This alternative will not be considered further.	
1F (7) – 130'BW channel improvement 200W – Prairie Ck	0 (5)	0 5	0 (5)	0- < 2	+- 2	+ (10)	* (-) 10	72/39		0.24	$6,143,700 (809,000 cy) $6.5 M	Environmental mitigation not included in cost.	Eliminates most road overflow for 10-year flood
1G (7.1) – combo Alts. 7 & 1	*+ [100]	*+ [100]	* (-) (100)	*+ 5	* (-) 5	* (-) 5	* 10				$14,223,000 (1,889,000 cy) $17.4 M	Environmental mitigation not included in cost.	
1H (21) – 200' shelf channel improvement d/s CR 300 S – 300 W	0 (5)	0 5	0 (5)	0 < 2	0 (+) < 2	0 < 2	0 < 2	90/84	72/75	0	$621,000 (29,000 cy) $0.7 M	Provides minimal benefits for the cost. This alternative will not be considered further.	
1I (21.6) – alts 21 & 6	- (5)	0 5	- (10)	- (-) < 2	0 (+) < 2	0 (+) < 2	0 < 2	85/75	66/66	0.22	$2,902,000 (139,000 cy) $3.2 M	Provides minimal benefits for the cost. This alternative will not be considered further.	
1J (17B) = 60' shelf channel improvement CR 200W – 400W	0 (5)	0 5	0 (5)	0 (+) < 2	- 2	-+ 2	0 (+) 2	72/58	59/50	0.18	$2,362,000 (200,000cy) $2.4 M	Environmental mitigation not included in the cost. Alt 23 provides more benefit at little additional cost so this alternative will not be considered further.	
1K (22.17B) – 60' shelf channel improvement d/s Tobin D. – Main St & Alt 17B	- 10	- 10	- 5	0 (+) <2	- (2)	-+ (2)	0(+) (2)	69/48	58/38	X	$2,981,000 (3,350,000 cy) $3.9 M	Environmental mitigation not included in the cost. Provides little flood relief along Buck Ck.	
1L (22B.17B) – same as 22.17B but 60' shelf channel improvement added between Main St. &	- 10	- 10	- 10	- 2	+ 2	+ 2	0- <2	48/48	24/24	X	$3,522,000 (400,000 cy) $4.9 M	Environmental mitigation not included in the cost.	Potentially benefits significant number of structures along Buck Ck from Big Cicero Ck backwater.

CR 200W													
1M (23) – 30' shelf channel improvement from d/s Tobin D. – CR 400W	- 10	- 10	- (10)	- <2	- <2	- <2	0 <2	59/59	41/41	X	$1,652,000 (160,400 cy) $2.5 M	Environmental mitigation not included in the cost. Alternative 24 gives additional benefit with little increase in cost so this alternative will not be considered any further.	
1N (24) – alt 23 extended up to CR 400S	- 10	- 10	- (10)	- <2	- <2	- <2	- 2	56/56	37/41[6]	X	$1,901,000 (195,000 cy) $2.7 M	Environmental mitigation not included in the cost.	
2A (14) – 100-yr protection levee in town	*	*	*	0	0	0	0	X	X	X	$1,175,000 (95,000 cy) $1.7 M	Doesn't appear to be adequate space, are internal drainage issues, potential for increased water surface elevations on unprotected property, requires a lot of maintenance. Would create future disaster potential in the event of a levee break. This alternative is not being considered further.	
2B (2) – 9.2 mi. ag levees	0	0	>	*	* NA	* NA	-* NA	X	X	2.2	$3,070,000 (217,000 cy) $3.2 M	Requires additional levees on tributaries and upstream of study reach to confine increased water surface elevations, increases downstream elevations up to 2.2 feet. Therefore, this alt. will not be considered further.	
3A (9) – detention basin u/s of CR 200W	* (-)	*	* (-)	0	0	0	0	X	X	X	$66,139,000 (10,295,000cy) $99.8 M	Environmental mitigation not included in cost. Due to cost, this alternative will not be considered further.	
3B (9.10) – Alt 9 w/ Q10-yr discharge	+	+	+	0	0	0	0	X	X	X	$47,349,000 (7,163,000 cy)	Environmental mitigation and land acquisition not included in cost. Due to cost, this alternative will not be considered further.	
3C (3) – detention on Cicero Ck u/s of Dixon Ck	0-	0-	0-	+	+	+	0+	X	X	X	$32,396,000 (5,042,000 cy) $46.5 M	Environmental mitigation not included in cost. Extended releases from detention pond would delay drainage tiles ability to empty. Due to costs, this alternative will not be considered further.	Reduce flooding of CR 400S & CR 400W if basin is located upstream of both roads.
3D (4) – several smaller detention in upper watershed	0-	0-	0-	-+	-+	+	-	X	X	X	-	Volume similar to alt 3 would be required and therefore have similar costs. This alternative will not be considered further.	
4A (5) – 76' BW bypass Garhart to Recobs D.	*+ [100]	* (-) (100)	*+ (100)	*+ 5	* (-) 10	+0 (2)	0 (2)	32/90	0/58	Approx. 0.2	$7,884,000 (534,000cy) $8.2 M	Increases d/s discharges up to 6%, requires 3-4 bridges to be built over channel, might put extreme floods on ground not previously flooded.	Environmental mitigation concerns would be minimal. Accomplishes the most in terms of overall flood elevation reduction and reduction in time out of banks in agricultural areas.
4B (5.5) – 38' BW bypass Garhart to Recobs D.	+- 10	+- 10	+ (-) 10	+ 2	+ 2	0+ (2)	0 < 2	42/90	21/62	X	$6,895,000 (390,000 cy) $7.1 M	requires 3-4 bridges to be built over channel, might put extreme floods on ground not previously flooded	Environmental mitigation concerns would be minimal.
4C (8) – 88' BW auxiliary channel 500W – Garhart D.	0	0	0	0 < 2	0 < 2	0- < 2	*+ 5	X	X	X	$4,497,000 (317,000cy) $4.7 M	Due to downstream elevations, this alternative creates little benefit for the cost. This alternative will not be considered further.	Environmental mitigation concerns would be minimal.
4D (8.5) – combo of Alts. 5 & 8	+ (+)	* (-)	+ (+)	*+ 5	*+ 5	+- (2)	*+ 5	X	X	X	$12,381,000 (851,000 cy) $12.9 M	Very little additional benefit compared to Alt 5 yet considerably more cost. This alternative will not be considered further.	Environmental mitigation concerns would be minimal.
4E (18) – bypass u/s RR to d/s Tobin Ditch	- (10)	0 (+) 10	0 (+) (5)	0 < 2	0 < 2	0 < 2	0 < 2	X	X	X	$1,473,000 (120,000cy) $1.8 M	Provides minimal benefits for the cost. This alternative will not be considered further.	
5 (15) – swales in farm	0	0	0	0 (+)	0(+)	0 (+)	0 (+)	X	X	X		Would not reduce flood elevations in	Benefits would be localized

ground												fields	
6 (16) – RR piers removed	- 5	0 5	0 (5)	0 < 2	0 < 2	0 <2	0 < 2	104/95	99/93	0.02	$83,000	Would need to be combined with other alternatives to produce any benefits in flood elevation reduction.	Lessen maintenance requirements due to removal of potential debris catching obstacles.
7 (20) – remove CR 300W	0 (5)	0 5	0 (5)	0 < 2	0 < 2	0 (+) < 2	0 < 2	100/90	Negl.	0		Provides minimal benefits for the cost. This alternative will not be considered further.	
8 (11) - buyouts				0	0	0	0	X	X	X	$12.5M	Expensive due to approximately 250 structures in the 100-year floodplain, 65 in the 10-year floodplain, and 0 in the 2-year floodplain. May be considered in combination with other alternatives	
9 (12) - floodproof				0	0	0	0	X	X	X	$6.3 M	Would not eliminate street flooding and sewer back-up problems. May be considered in combination with other alternatives.	
10 (13) – ordinance & enforcement	0 (+)	0 (+)	0 (+)	0	0	0	0	X	X	X	$10,000	Would not reduce existing damages.	Would lessen the likelihood that future development would be exposed to flood damages and would prevent increasing the present level of flood damage.
11A (7.3)– combo of Alts. 3 & 7	0-	0-	0-	+	* (-)	* (-)	0*	X	X	X	$38,540,000	Environmental mitigation and land acquisition not included in cost. Due to cost, this alternative will not be considered further.	Reduce flooding of CR 400S & CR 400W if basin is located upstream of both roads.
11B (8.7.1) – combo Alts 8, 7, & 1	*+	*+	* (-)	*+ 5	*+ 5	-+ 2	*+ (10)	X	X	X	$18,720,000 (2,206,000 cy) $22.0 M	Environmental mitigation not included in cost. Very little additional benefit compared to Alt. 7.1 yet more cost. This alternative will not be considered further.	Environmental mitigation concerns would be minimal.
11C (9.1.5) – alt 9 & 1.5	*(+)	* (+)	* (+)	0	0	0	0	X	X	X	$18,372,000 (1,630,00 cy) $105.3 M	Environmental mitigation not included in cost. Provided no benefit to agricultural areas. Due to cost, this alternative will not be considered further.	
1J (17B) – 60' shelf channel improvement CR 200W – 400W	0 (5)	0 5	0 (5)	0 (+) < 2	- 2	-+ 2	0 (+) 2	72/58	59/50	0.18	$2,362,000 (200,000cy) $2.4 M	Environmental mitigation not included in the cost. Alt 23 provides more benefit at little additional cost so this alternative will not be considered further.	
4E (18) – bypass u/s RR to d/s Tobin Ditch	- (10)	0 (+) 10	0 (+) (5)	0 < 2	0 < 2	0 < 2	0 < 2	X	X	X	$1,473,000 (120,000cy) $1.8 M	Provides minimal benefits for the cost. This alternative will not be considered further.	
1E (19) – 30' shelf channel improvement d/s RR – d/s Tobin Ditch	- 5	- [10]	0 (5)	0 < 2	0 < 2	0 < 2	0 < 2	93/95	X	X		Provides minimal benefits for the cost. This alternative will not be considered further.	
7 (20) – remove CR 300W	0 (5)	0 5	0 (5)	0 < 2	0 < 2	0 (+) < 2	0 < 2	100/90	Negl.	0		Provides minimal benefits for the cost. This alternative will not be considered further.	
1H (21) – 200' shelf channel improvement d/s CR 300 S – 300 W	0 (5)	0 5	0 (5)	0 < 2	0 (+) < 2	0 < 2	0 < 2	90/84	72/75	0	$621,000 (29,000 cy) $0.7 M	Provides minimal benefits for the cost. This alternative will not be considered further.	
1I (21.6) – alts. 21 & 6	- (5)	0 5	- (10)	- (-) < 2	0 (+) < 2	0 (+) < 2	0 < 2	85/75	66/66	0.22	$2,902,000 (139,000 cy) $3.2 M	Provides minimal benefits for the cost. This alternative will not be considered further.	
1K (22.17B) – 60' shelf channel improvement d/s Tobin D. - Main St & Alt 17B	- 10	- 10	- 5	0 (+) <2	- (2)	-+ (2)	0(+) (2)	69/48	58/38	X	$2,981,000 (3,350,000 cy) $3.9 M	Environmental mitigation not included in the cost. Provides little flood relief along Buck Ck.	
1L (22B.17B) – same as	-	-	-	-	+	+	0-	48/48	24/24	X	$3,522,000	Environmental mitigation not included	Potentially benefits significant number of

22.17B but 60' shelf channel improvement added between Main St. & CR 200W	10	10	10	2	2	2	<2				(400,000 cy) $4.9 M	in the cost.	structures along Buck Ck from Big Cicero Ck backwater.
1M (23) – 30' shelf channel improvement from d/s Tobin D. – CR 400W	- 10	- 10	- (10)	- <2	- <2	- <2	0 <2	59/59	41/41	X	$1,652,000 (160,400 cy) $2.5 M	Environmental mitigation not included in the cost. Alternative 24 gives additional benefit with little increase in cost so this alternative will not be considered any further.	
1N (24) – alt 23 extended up to CR 400S	- 10	- 10	- (10)	- <2	- <2	- <2	- 2	56/56	37/41*	X	$1,901,000 (195,000 cy) $2.7 M	Environmental mitigation not included in the cost.	

For areas 1-3: * means the alternative lowered 100-year flood elevations to or below the lowest grade of structures in that reach
+ means the alternative lowered 100-year elevations to about the existing 10-year frequency elevations in that reach
- means the alternative lowered 100-year elevations below the existing 100-year frequency elevations in that reach
0 means the alternative did not lower 100 year flood elevations in the reach

For areas 4-7: * means the alternative lowered the 10-year flood elevations to the approximate channel bank elevations
+ means the alternative lowered the 10-year flood elevations to approximately the existing 2-year flood elevations
- means the alternative lowered the 10-year flood elevations to approximately the existing 5-year flood elevations
0 means the alternative did not lower the 10-year flood elevations

(+) or (-): means slightly higher or lower benefit, respectively, but not enough to be classified as the next benefit level.

(#): means the #-year frequency flood is within ½ foot of the damage free elevation
[#]: means the #-year frequency flood is within 1 foot of the damage free elevation

X: means not determined because alternative is not being pursued or model doesn't extend far enough to determine, or information not available with present degree of modeling.

* The additional benefit to the reach upstream of CR 400W is hidden because the water stays out of bank in the downstream portion of the reach longer than the added upstream portion.

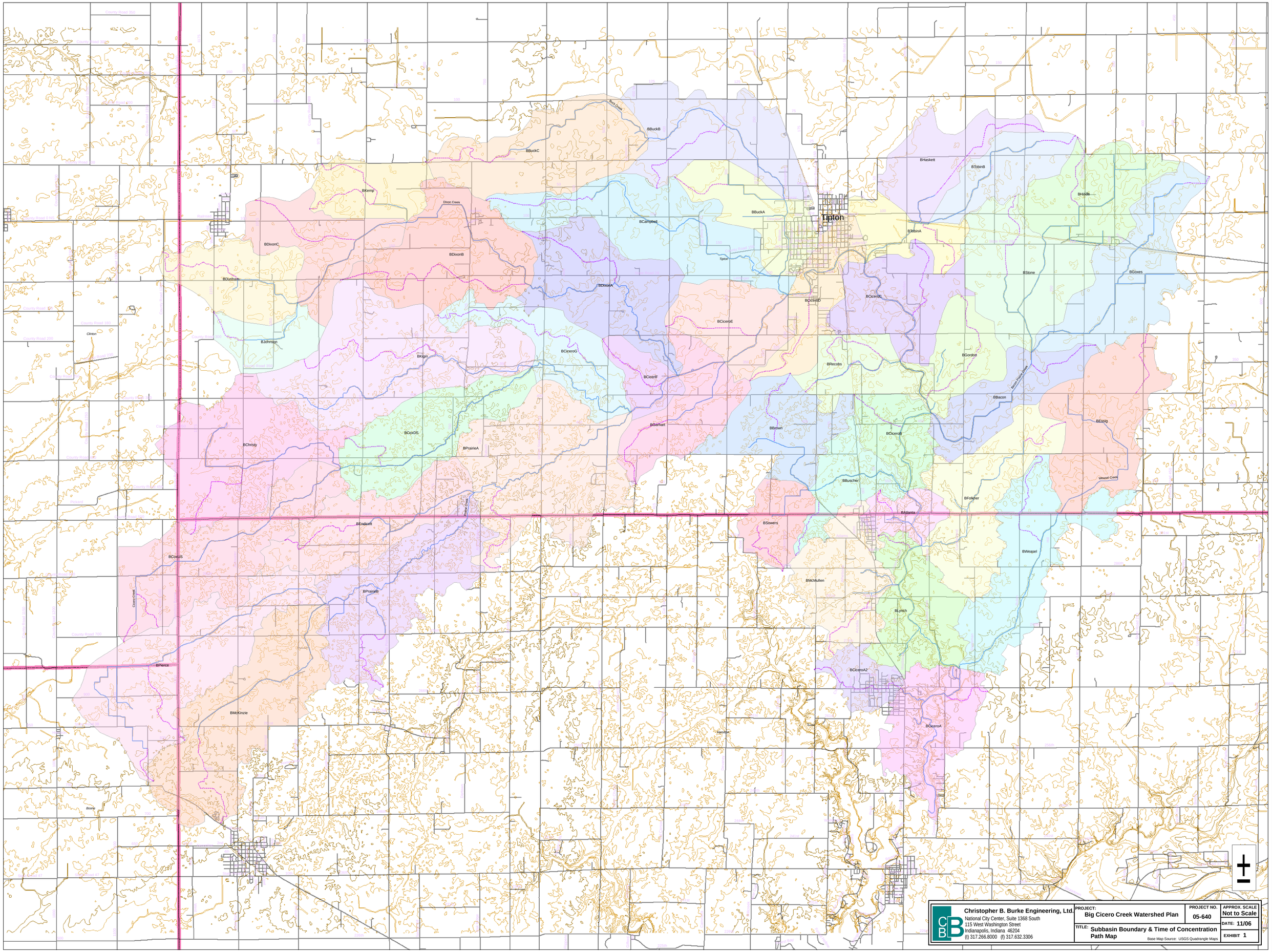
Costs include construction cost, design & construction engineering costs, and a 20% contingency.

4.0 DATA SOURCES

Data used in this study was collected from the Joint Drainage Board, Tipton County and Hamilton County staff, the Indiana Department of Natural Resources (IDNR), United States Geological Survey (USGS), Natural Resource Conservation Service (NRCS), EarthData, and the Federal Emergency Management Agency (FEMA). Each data source is listed below, along with the data obtained from that source:

Source	Data Provided
Indiana Department of Natural Resources (IDNR)	• Regulatory hydraulic model (HEC-2) • Coordinated Discharge Graph titled Big Cicero Creek • Coordinated Point Discharges at Arcadia • Historic Profile for Big Cicero Creek, June 1957 and January 1962 floods • B17B analysis of USGS gage records through 2003
City of Tipton, Indiana Flood Insurance Study (FIS) dated March 5, 1996	• Flood profiles • Flood Insurance Rate Maps • Floodway Data Tables, and other information
Tipton County Surveyor	• Tipton county watershed map (1994) • Various flood photos • Railroad Right-of-Way maps showing culverts through the RR bed • Regulated drain reconstruction plans for Cox, Dixon, and Crum Ditch (1/2001, 3/1984) • Bridge inventory information for the structures inventoried over Big Cicero Creek (most recent inventory as of May, 2006)
Hamilton County Surveyor	• Channel and some bridge deck survey information (February, 2005)

USGS	• Peak flow data at area gages • Daily Mean Discharges at Big Cicero Ck at/near Arcadia gage • Digital 10' Contour interval quad mapping for watershed portions of Tipton, Clinton, and Boone Counties • 1999 National Land Cover Dataset
EarthData	• Digital 2' contour interval mapping for a 4000' wide corridor of the study reach of the creek (developed as part of the study and received May, 2006)
Natural Resource Conservation Service (NRCS) SSURGO	• Digital Soils data Boone County (12/6/05) Clinton County (8/1/05) Hamilton County (12/22/01) Tipton County (12/6/05)
National Climatic Data Center	• Historic hourly rainfall for area gages



Christopher B. Burke Engineering, Ltd.
National City Center, Suite 1368 South
115 West Washington Street
Indianapolis, Indiana 46204
(t) 317.266.8000 (f) 317.632.3306

PROJECT: Big Cicero Creek Watershed Plan

PROJECT NO. 05-640

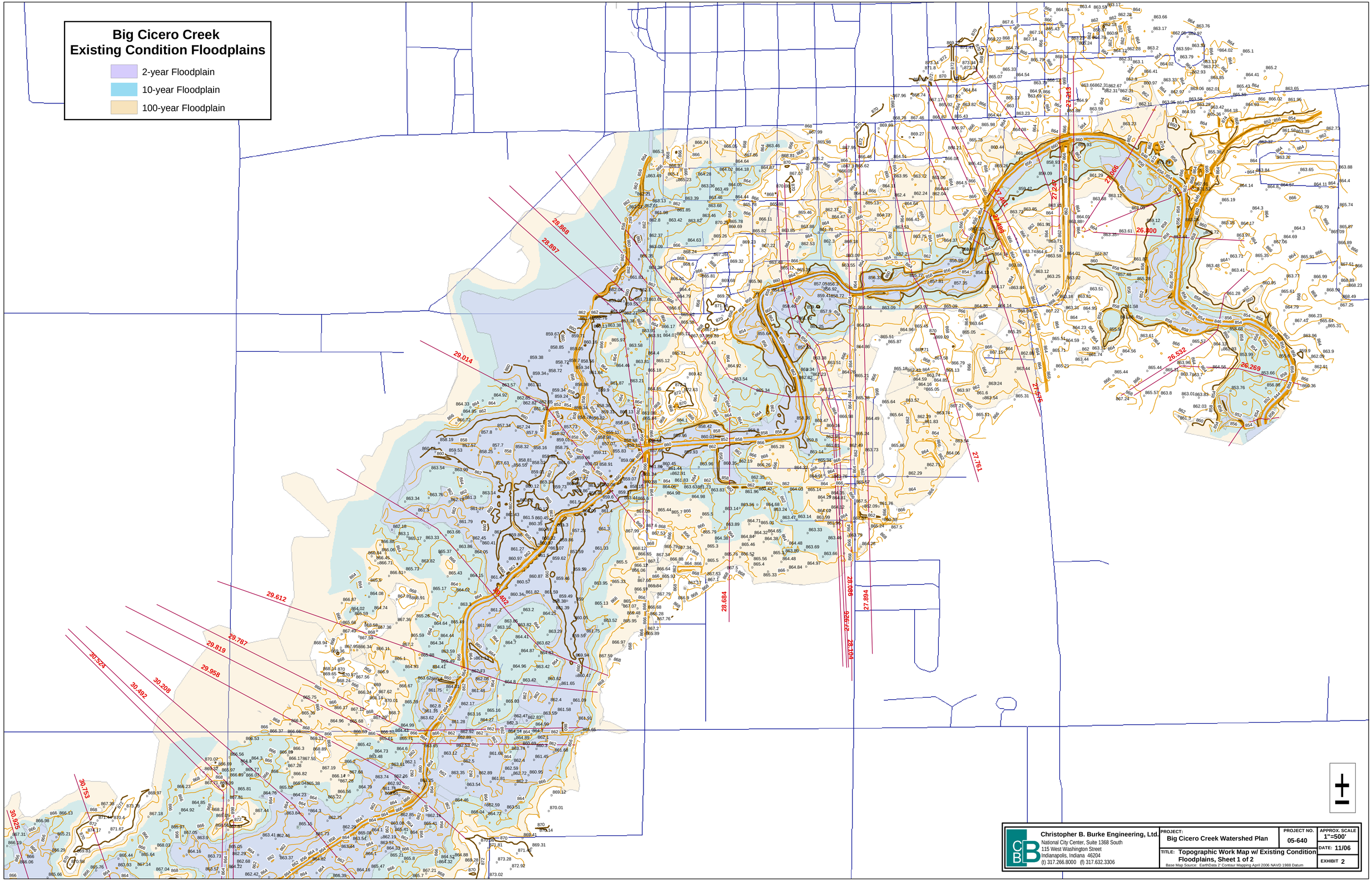
APPROX. SCALE Not to Scale

DATE: 11/06

TITLE: Subbasin Boundary & Time of Concentration Path Map

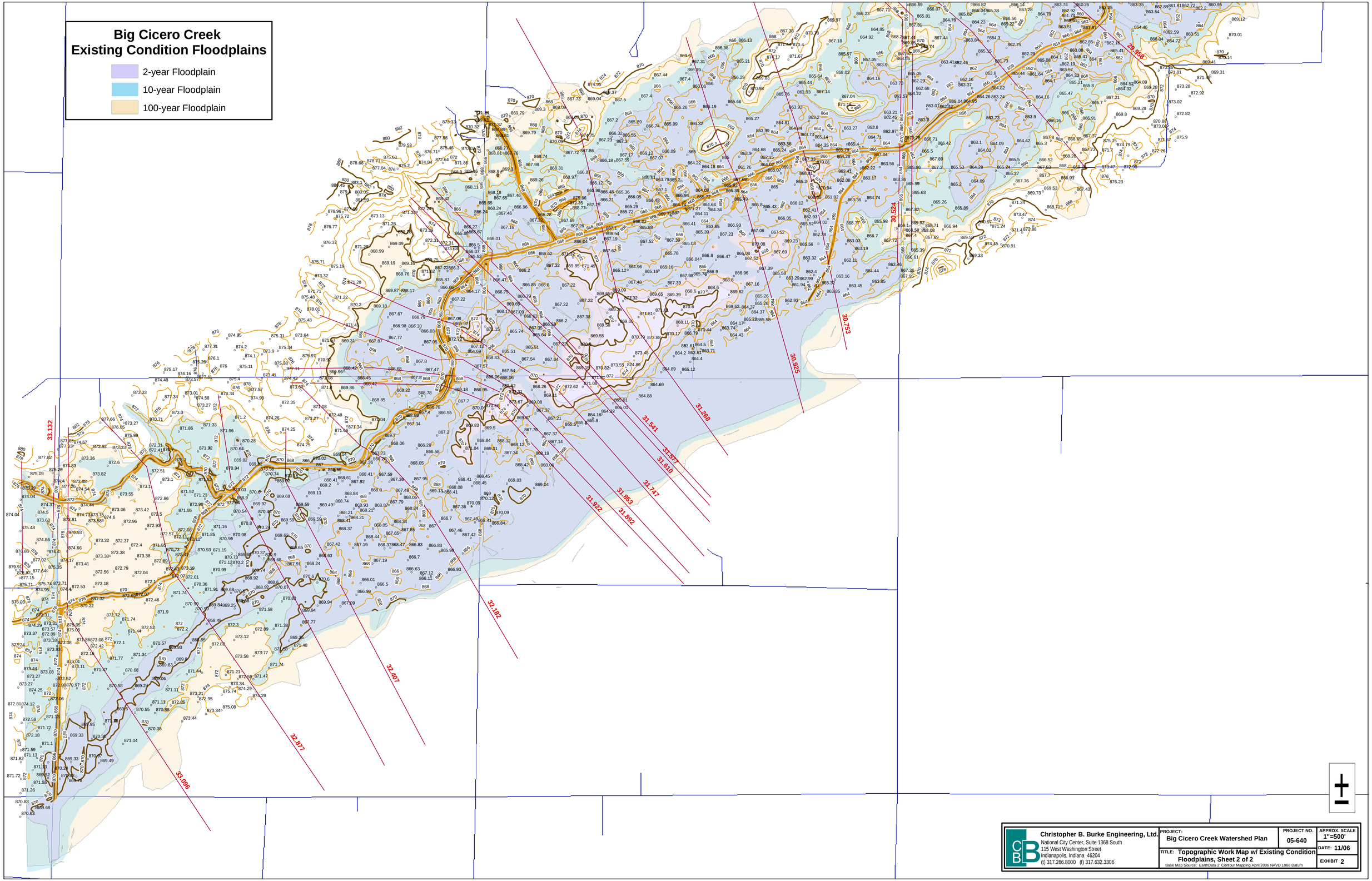
Base Map Source: USGS Quadrangle Maps

EXHIBIT 1



Christopher B. Burke Engineering, Ltd.
National City Center, Suite 1368 South
115 West Washington Street
Indianapolis, Indiana 46204
(t) 317.266.8000 (f) 317.632.3306

PROJECT: Big Cicero Creek Watershed Plan	PROJECT NO. 05-640	APPROX. SCALE 1"=500'
TITLE: Topographic Work Map w/ Existing Condition Floodplains, Sheet 1 of 2	DATE: 11/06	
Base Map Source: EarthData 2' Contour Mapping April 2006 NAVD 1988 Datum	EXHIBIT 2	



Christopher B. Burke Engineering, Ltd.
National City Center, Suite 1368 South
115 West Washington Street
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(t) 317.266.8000 (f) 317.632.3306

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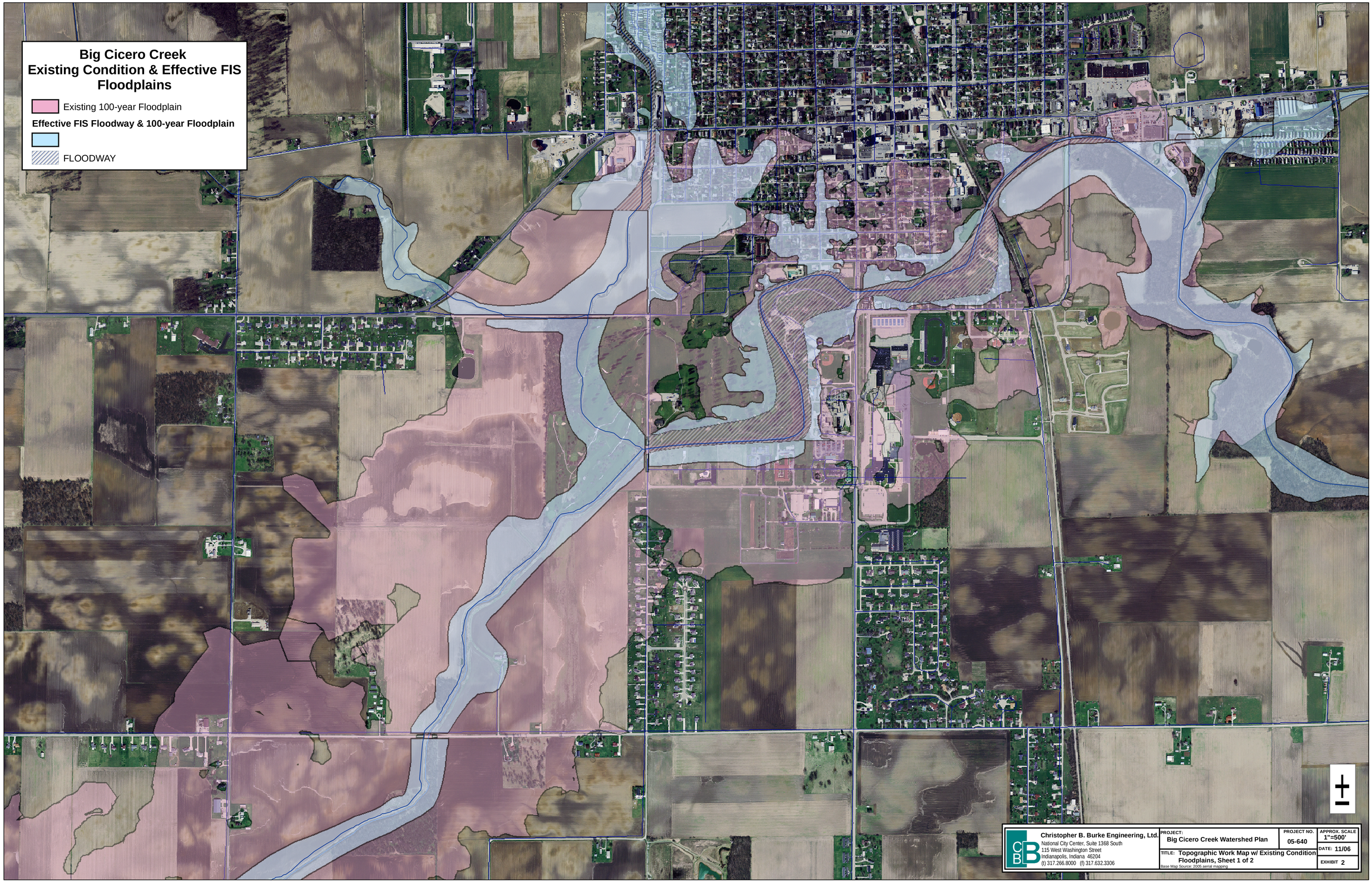
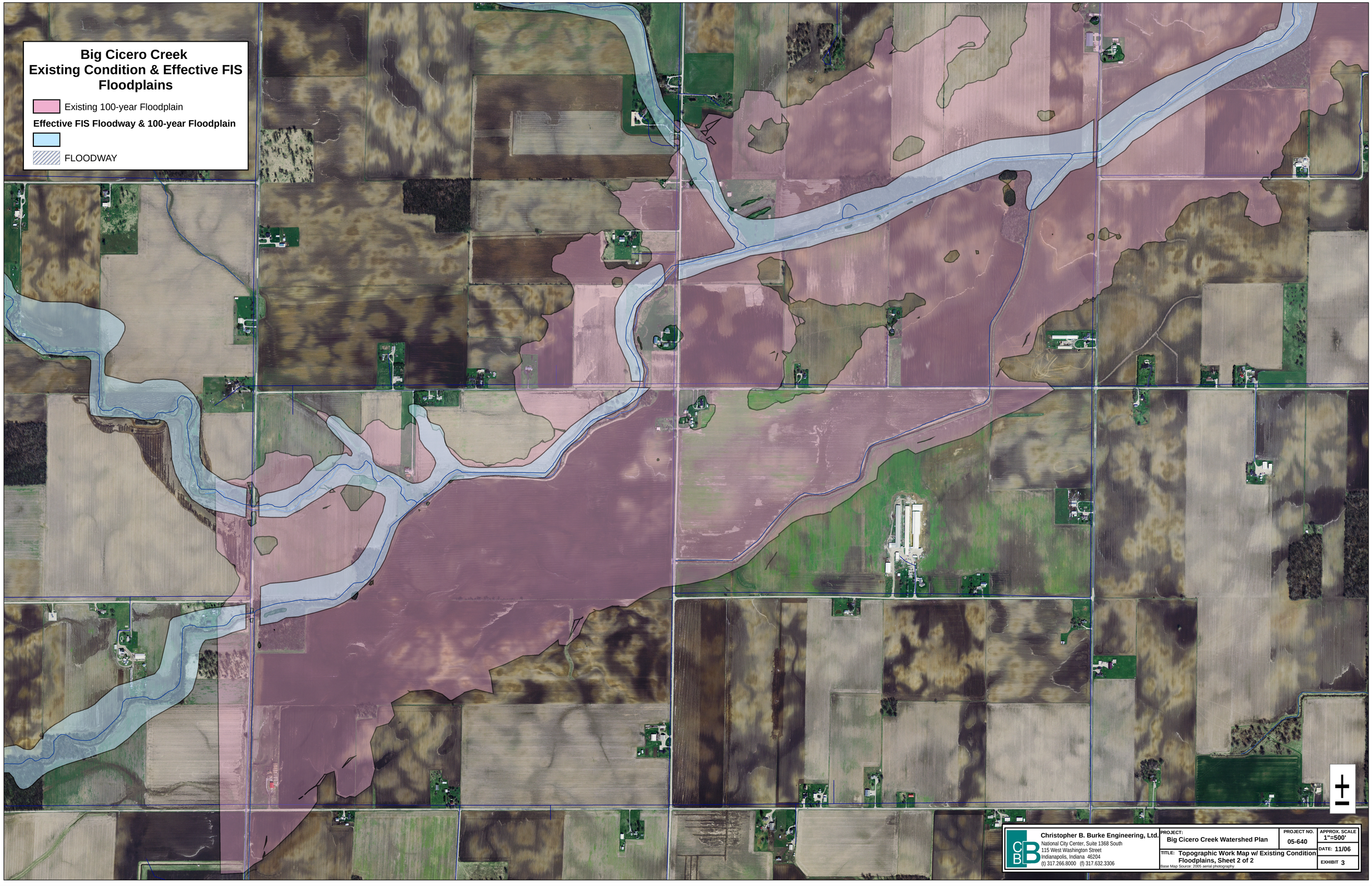


Christopher B. Burke Engineering, Ltd.
National City Center, Suite 1368 South
115 West Washington Street
Indianapolis, Indiana 46204
(t) 317.266.8000 (f) 317.632.3306

PROJECT: Big Cicero Creek Watershed Plan	PROJECT NO. 05-640	APPROX. SCALE 1"=500'
TITLE: Topographic Work Map w/ Existing Condition Floodplains, Sheet 1 of 2		DATE: 11/06
Base Map Source: 2005 aerial mapping		EXHIBIT 2


Big Cicero Creek
Existing Condition & Effective FIS Floodplains
Existing 100-year Floodplain
Effective FIS Floodway & 100-year Floodplain
FLOODWAY
Christopher B. Burke Engineering, Ltd.
National City Center, Suite 1368 South
115 West Washington Street
Indianapolis, Indiana 46204
(t) 317.266.8000 (f) 317.632.3306
PROJECT:
Big Cicero Creek Watershed Plan
PROJECT NO.
05-640
APPROX. SCALE
1"=500'
TITLE:
Topographic Work Map w/ Existing Condition Floodplains, Sheet 2 of 2
DATE:
11/06
EXHIBIT
3
Base Map Source: 2005 aerial photography.