



State of Illinois
Department of Natural Resources

OFFICE OF WATER RESOURCES

Kingston Flood Control Study Village of Kingston DeKalb County, Illinois March 2021



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KINGSTON FLOOD CONTROL STUDY

Village of Kingston
DeKalb County, Illinois

I. PURPOSE AND AUTHORITY

This report summarizes the results of a flood control study for the Village of Kingston, Illinois. This study is intended to identify the sources and extent of the existing flood problems, and to evaluate alternatives for reducing future flood damages.

The Illinois Department of Natural Resources/Office of Water Resources (OWR), is granted the authority to conduct flood control surveys and studies for all watersheds of the state through the Flood Control Act of 1945, 615 ILCS 15, Illinois Compiled Statutes.

II. WATERSHED DESCRIPTION

The Village of Kingston, Illinois is located in the northern part of DeKalb County, approximately 60 miles northwest of Chicago, Illinois and about 21 miles southeast of Rockford, Illinois (**Figure 1**). Kingston is located about 13 miles north of DeKalb, Illinois. Kingston is located along Illinois State Route 72 about 7.6 miles east of Interstate 39.

The Village of Kingston has a population of 1,164 based on the 2010 census figures. Land use outside the Village limits is primarily agricultural with some wooded areas. Land use within the Village limits is mostly residential with some commercial and business development (**Figure 2**).

The Village is served by an unnamed tributary which flows south to north through the central part of the Village. The unnamed tributary has a slope of about 51.3 feet per mile through the study area.

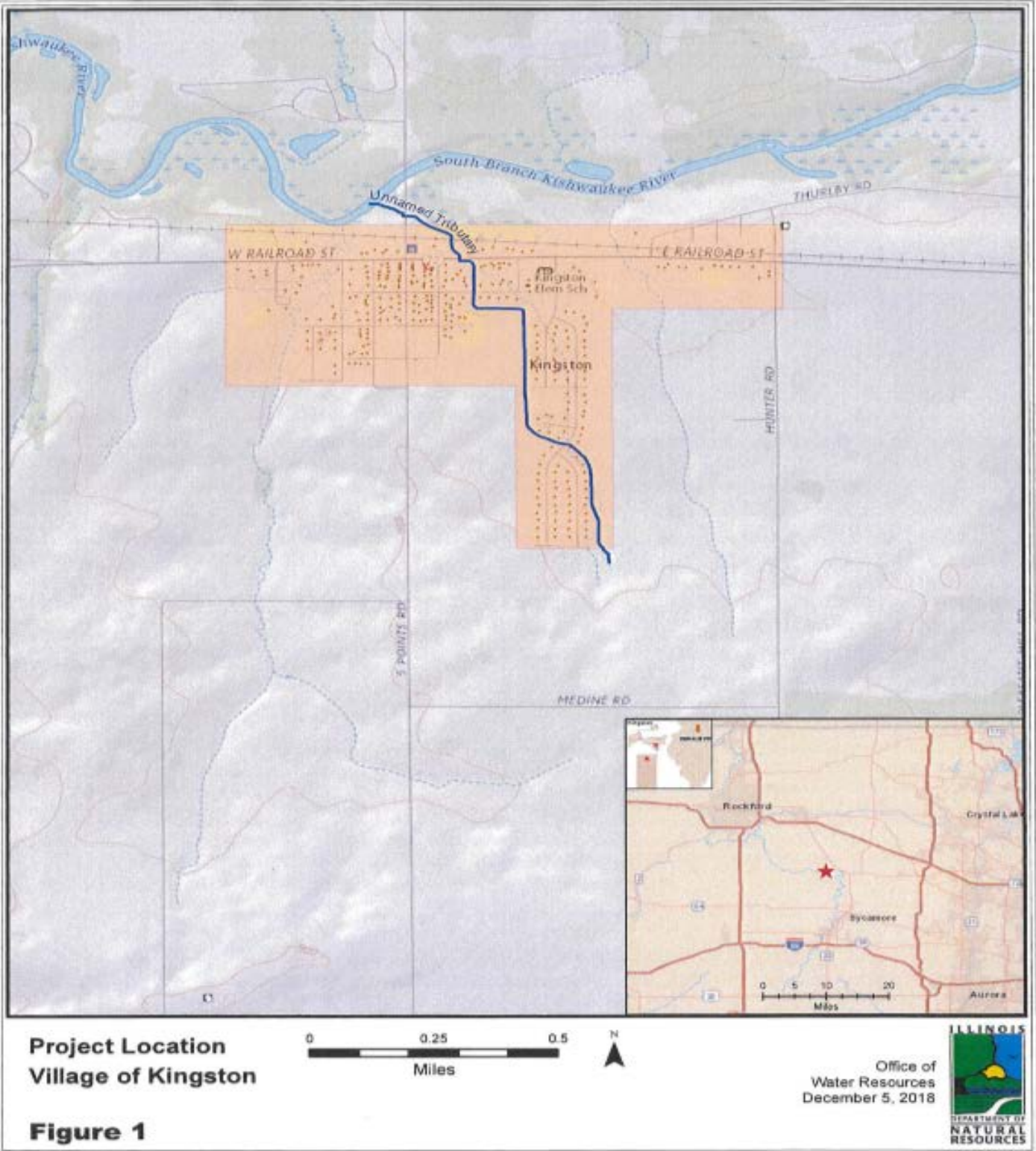


FIGURE 1 - Location Map

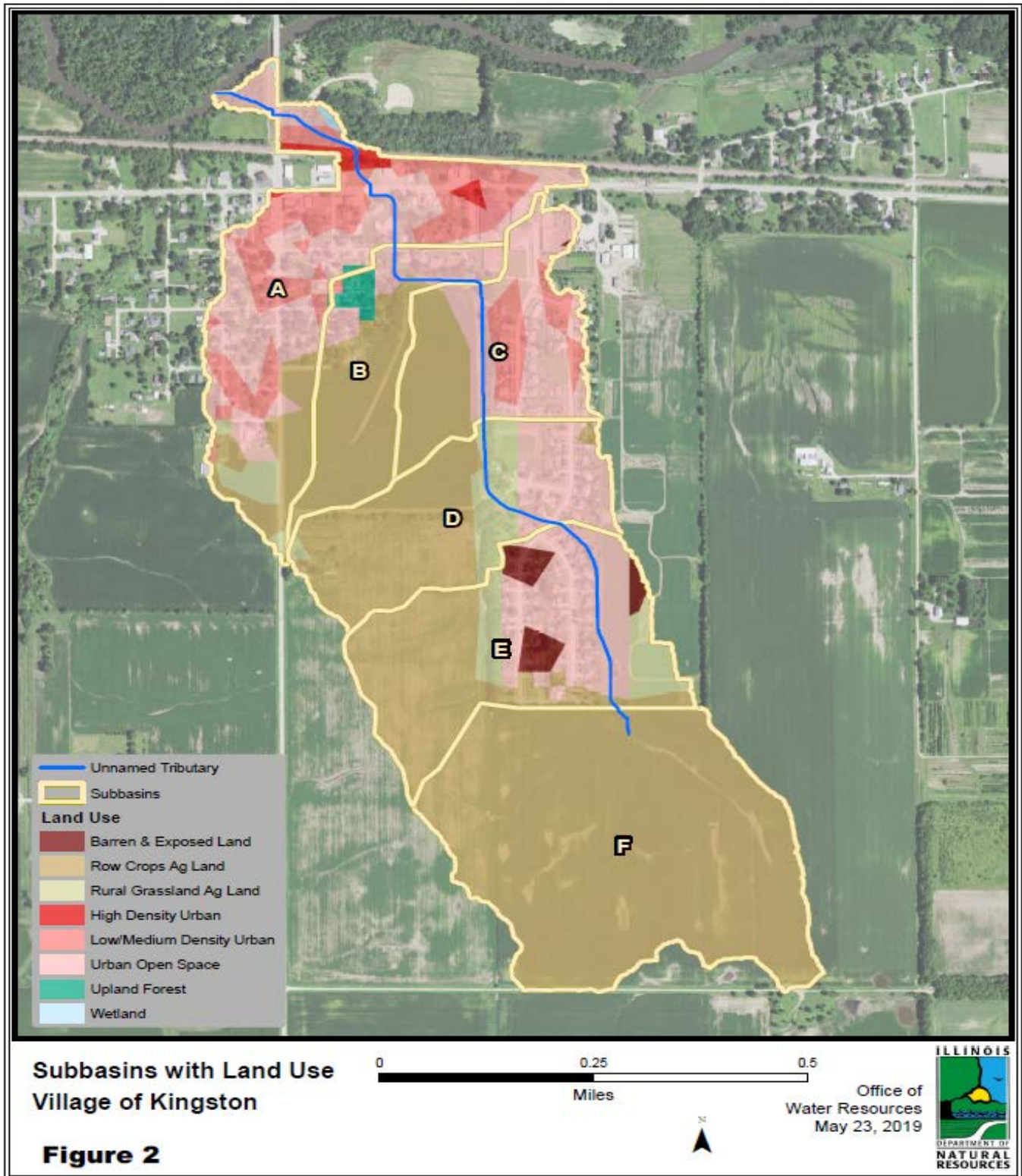


FIGURE 2 - Watershed Sub-Basins Map With Land Use

This unnamed tributary commences about 208 feet south of the Kingston Village limits and extends to about 980 feet downstream of the Village limits. This tributary flows through the Village for a distance of about 4,860 feet (0.9 miles) and outlets to the South Branch Kishwaukee River. The South Branch

Kishwaukee River flows into the Kishwaukee River.

The total unnamed tributary watershed area is approximately 0.44 square miles. For this study, the watershed was divided into 6 subareas (**Figure 3**). The tributary conveys flow to the north through Kingston, beneath six roadway structures, including the Illinois State Route 72 bridge, and two railroad bridges. Cropland, located south of the Village, is designated as Subarea "F", and has a drainage area of 0.13 square miles. Runoff from this area enters the unnamed tributary and these flows continue to the north through Kingston.

III. HYDROLOGY

HEC 1

The watershed under consideration does not have stream gauging stations to provide historical or factual stream flow information. Therefore, synthetic methods were utilized to arrive at estimates of discharges for various frequency storm events, at strategic locations being studied. The US Army Corps of Engineers' HEC-1 computerized watershed hydrology model was used. The SCS Curve Number Loss Rate method was chosen to analyze the watershed. These curve numbers were generated by analyzing soil types and land use in GIS.

The SCS unit hydrograph method was selected to compute synthetic unit hydrographs for each subarea. The time of concentration (T_c) for each subarea was determined. TLAG was computed using the time of concentration in equation $TLAG = 0.6 \times T_c$. Hydrographs were computed as necessary to obtain storm hydrographs and peak discharges at strategic locations.

A total of seven (1-, 3-, 6-, 12-, 18-, 24-, and 48-hour) duration 100-year frequency were input into the HEC1 model and analyzed to determine the critical duration. The critical duration is that duration along with the corresponding point rainfall that produces the highest discharge amounts. The 1-hour

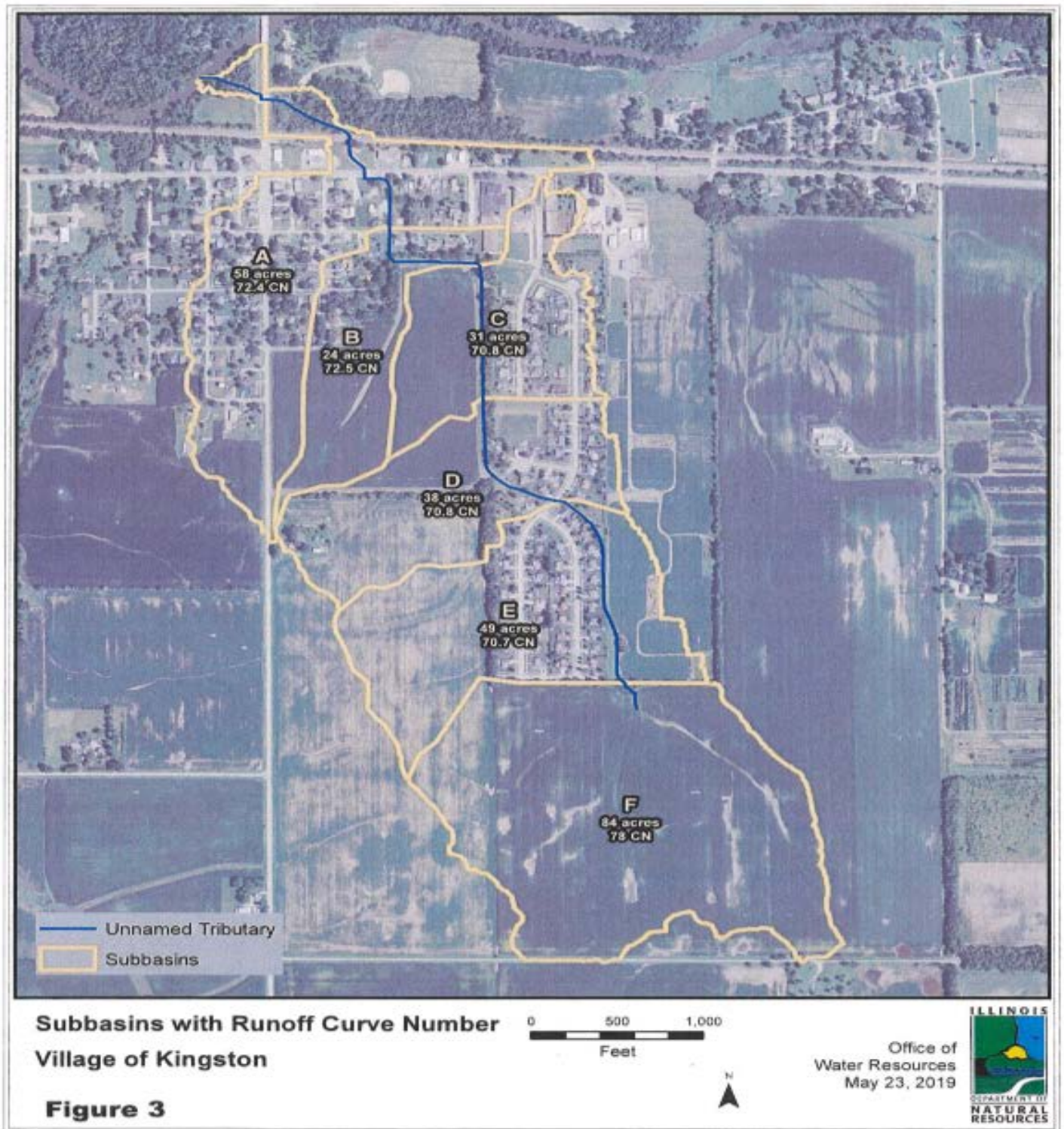


Figure 3 - Watershed Sub-Basins Map

duration yielded the greatest runoff discharge and therefore was selected as the design storm event. The design HEC-1 model simulates the runoff from a 3.56-inch rain distributed over a 1-hour duration. (Appendix A). The 100-year flood event has a 1% chance of occurring in any 1-year.

Meteorological models were made for six different storm frequency events which were the 2-,5-,10-,25-,50-,100-year for each sub-watershed which included the use of various hydrologic parameters (**Table 1**). The subareas were delineated in GIS using 2-foot contour interval mapping based on lidar and field investigations.

IV. HYDRAULICS

HEC-RAS

The US Army Corps of Engineers HEC-RAS backwater model utilizes surveyed bridge, channel, and floodplain cross section information to compute water surface profiles when a discharge is input into the model. An existing conditions HEC-RAS Unsteady combination 1-Dimensional and 2-Dimensional Flow Model was developed (**Appendix B**), based on the survey information obtained from 2014 OWR field surveys and Lidar also.

The HEC-RAS hydraulic modeling commences at the confluence of the unnamed tributary and the South Branch Kishwaukee River and continues upstream(south) through the Village for a distance of about 6,047 feet. The model terminates about 208 feet south of 9th Street. The slope area method was used to estimate the starting water surface elevation, assuming the slope of the energy grade line was similar to the bed slope. Cross section locations for the HEC-RAS model and the Existing Conditions 100-year floodplain are shown in **Figure 4**.

KINGSTON

CALCULATE TLAG USING TC (TIME OF CONCENTRATION) FOR SCS HEC1 MODEL

Sub-basin	AREA (SQ. MI.)	TC	R	CN	SCS TC	SCS TLAG (TC*0.6) (HR)	Computation Interval or Minimum Duration Interval TLAG x .06 (HR)	Computation Interval or Minimum Duration Interval TLAG x .06 (Minutes)
A	0.091	0.621	0.501	72.4	0.621	0.3726	0.108054	6.48324
B	0.038	0.318	0.277	72.5	0.318	0.1908	0.055332	3.31992
C	0.048	0.268	0.522	70.8	0.268	0.1608	0.046632	2.79792
D	0.059	0.289	0.29	70.8	0.289	0.1734	0.050286	3.01716
E	0.077	0.318	0.358	70.7	0.318	0.1908	0.055332	3.31992
F	0.131	0.333	0.305	78	0.333	0.1998	0.057942	3.47652

Table 1 - Hydrologic (HEC1) Parameters

Several lateral structures were used in the HEC-RAS model. One of which was located along the north side of the Railroad Tracks Bridge. The minimum weir elevation at this location is 782.37 feet and the weir elevations correspond to the existing ground elevation at that location. This weir conveys flows out of the Unnamed Tributary system to the South Branch (S.B.) Kishwaukee River. Because the discharge conveyed over this weir flows to the S.B. Kishwaukee River, this flow was permanently

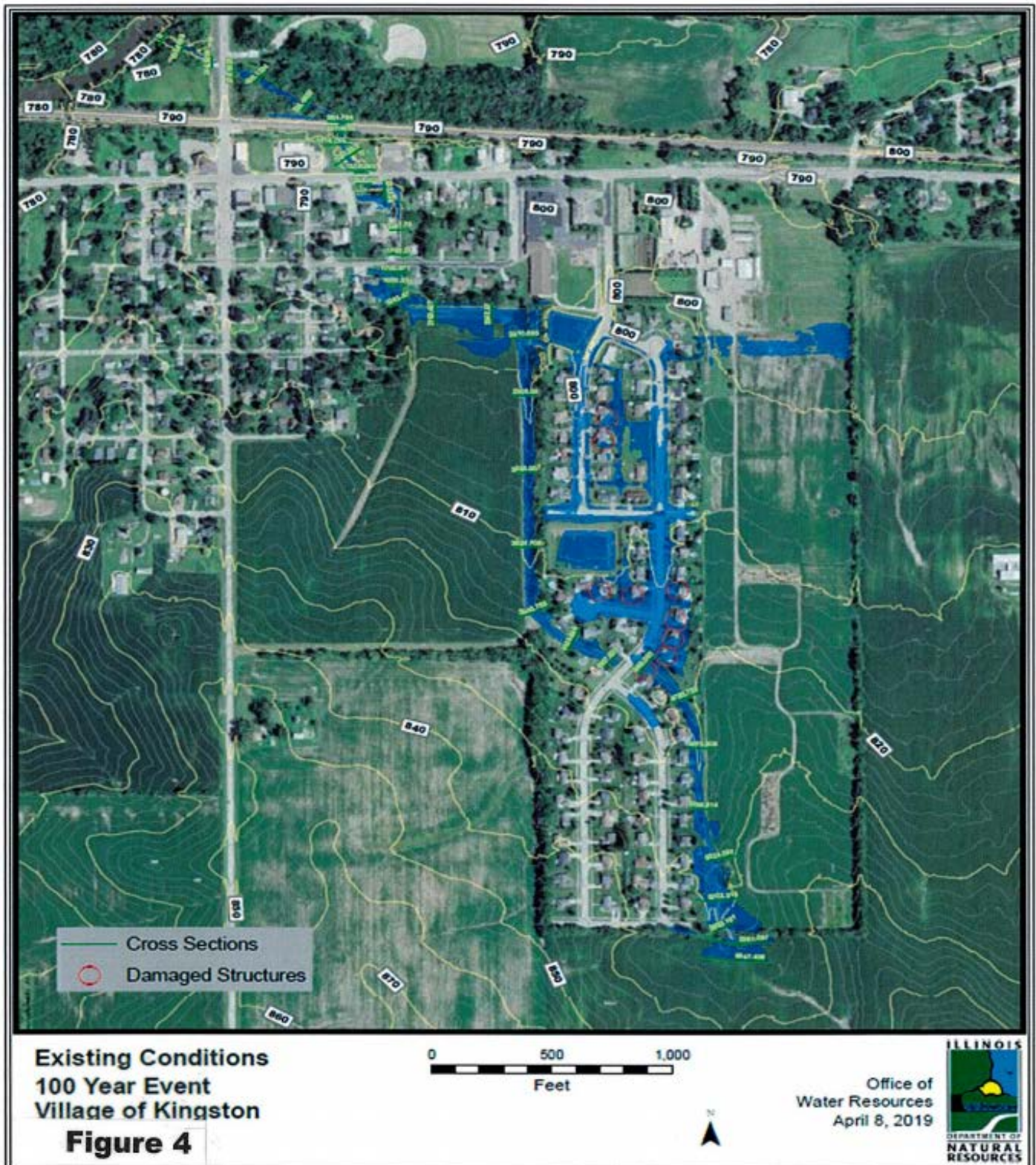


FIGURE 4 - Cross Sections And Existing Conditions 100 Year Floodplain Map

removed and not recombined with the remaining Unnamed Tributary flows continuing downstream to its confluence.

There is an increase in the 100-year frequency water surface elevation upstream of the 1st Street box culvert of 2.88 feet. Most of this increase is because of the change in the channel invert elevation just upstream of this culvert. The invert increases from 786.6 feet to 788.79 feet which is a change of 2.19 feet. There is a drop structure located just upstream of the box culvert for transition through this abrupt elevation change. This structure was modeled in HEC-RAS as an in-line structure.

A second lateral structure was modeled in HEC-RAS at the detention basin located upstream of 1st Street just south of the Kingston Grade School. This structure connects the 1-dimensional flow area to the 2-dimensional flow area located south of the school and extending to the Sandra Street Culverts. The lateral structure minimum weir elevation is 798.0 feet.

A lateral Structure was also input into the HEC-RAS model upstream of the Sandra Street roadway culverts along the low area on the north side of the channel. The lateral structure weir corresponds to the ground elevations at this location and has a minimum elevation of 815.0 feet. The 100, 50, and 25-year frequency flows generate water surface elevations upstream of Sandra Street that exceed this minimum elevation. When this occurs, flow overtops the lateral structure weir and is conveyed overland through the Windhaven Drive and Sandra Street subdivision area resulting in structure damages. The amount of flow conveyed by the weir is controlled by the length and elevation of the weir. The maximum 100-year water surface elevation at the lateral structure weir is 817.78 feet.

The subdivision area located north of the Lateral weir upstream of Sandra Street, was modeled as a 2- dimensional area. Flow conveyed over the Lateral Structure weir was then modeled as it moved overland using about 3,581 cells referred to as a mesh. These cell area elevations were determined using an elevation tin developed from Lidar elevation data. Flows travel through each cell of the mesh based on elevation. Thus, flow will be conveyed throughout this 2-dimensional area based on the lowest elevation within each cell section inside the mesh. The average cell size area calculated in the model was 634.8 square feet.

CALIBRATION

USGS Stream Stats discharges were obtained and the 90% confidence limits discharges were plotted along with the OWR discharges. The USGS discharges plots compared very well with the OWR discharges. The OWR discharges were within the range between the 90% minimum and 90% maximum confidence limits of the U.S.G.S. StreamStats discharges.

The HEC-RAS model was calibrated using the May 13, 2011 flood event. The precipitation amount and storm duration were determined by using NEXRAD data from the National Weather Service in conjunction with gage data from precipitation gages at Kingston, Belvidere, Genoa, DeKalb, and the DeKalb Airport. The HEC1 discharge was developed using the precipitation data. The precipitation data was input into the HEC1 model to generate discharges for the flood event. These discharges were input into the existing conditions HEC-RAS model and the resulting water surface elevations were calculated.

High water mark locations were determined using photos of the flooded area provided by the residents. The highwater marks noted were surveyed to obtain elevation data (**Figure 5**). The elevations from the 2011 flood HEC-RAS model were then compared to high water mark elevations determined within the Village. These highwater elevations matched reasonably close to the elevations from the HEC-RAS model within the Windhaven Drive and Sandra Street subdivision damage area. The elevation differences ranged from 0.35 feet to 0.02 feet between the HEC-RAS model and the high water marks. Thus, the HEC-RAS model developed for the unnamed tributary was considered to be reasonably accurate (**Appendix C**).

The critical duration Existing Conditions discharges from the HEC-1 model were input into the Existing Conditions HEC-RAS model for the entire range of storm frequencies. This hydraulic model provided the flood frequency-elevation information for existing conditions. Once the HEC-RAS model was finalized, the 100-year floodplain was delineated on the plan map using the 100-year profile elevations from this model. Similar hydrologic and hydraulic modeling was then done for the project alternatives to determine project impacts on flood flows and elevations.



Figure 5 - High Water Mark Locations Map

V. FLOOD PROBLEM

A. Description

The drainage system that was studied in the Village of Kingston consists of an unnamed tributary channel that conveys flows through the central portion of the Village and eventually drains into the South Branch Kishwaukee River. The total drainage area of the unnamed tributary to its confluence is 0.44 square miles. The tributary conveys flows from south to north through the central part of the Village which includes the Windhaven Drive and Sandra Street subdivision area.

The total drainage area upstream of Sandra Street and south of Kingston totals approximately 0.21 square miles. This drainage area includes 0.13 square miles of cropland south of the Village which has a slope of .0183 feet per foot or 97 feet per mile, which is quite steep. This contributes to higher runoff rates from this drainage area which exceed the channel capacity of the tributary at the two 3.5' x 2.5' corrugated metal pipe arch culverts under Sandra Street.

The total discharge upstream of the Sandra Street culverts is about 277 cubic feet per second (cfs) at the 100-year frequency existing conditions event. This discharge amount exceeds the capacity of the two culverts at Sandra Street. The minimum elevation of the right bank upstream of the culverts is 815.0 feet. When the water surface exceeds this elevation, flows overtop the channel bank and continue northward overland along the old channel alignment. The maximum overland flow is about 120 cfs for the 100-year frequency event. These overland flows cause damages to nine structures within the Windhaven Drive and Sandra Street areas of this subdivision based on this Study. The total head loss through the two Sandra Street culverts is 1.8 feet at the 100-year frequency existing conditions discharge. Approximately 189 cfs continues to be conveyed along the existing channel downstream of Sandra Street.

Structure damages result when overland flow occurs at this location for the higher 25, 50, and 100-year frequency runoff events which of course produce greater flow rates. These less frequent flood events have a lower percentage chance of occurring in any one year. The 100-year frequency has a one percent chance of occurring in a one year period whereas a 2-year frequency event has a 50 percent chance of occurring in a 1-year period.

B. Historic Events and Damages

The Village most recently experienced a flood event which occurred in May 2011 following an

approximate 5.29 - inch rainfall with a duration of 8 hours. Damage Survey forms were provided by IDNR/OWR to the Village to be used to provide structure damage information for this flood event. These forms were completed by residents and were returned by the Village, to OWR. The completed forms show that damages totaling \$78,536 occurred to structures along the unnamed tributary which was studied and is described in this report.

VI. DAMAGES MODEL

For the One-Dimensional section of the Study Area, an Average Annual Damages computer model was compiled to calculate damages based on the 100, 50, 25, 10, 5, and 2-year frequency discharge water surface profiles. The first floor and low water entry elevations were surveyed for all damageable structures. All the structures were identified by structure type and were then input into the damage model along with the structure first floor and low water entry elevations.

This damage model uses US Army Corps of Engineers depth-damage curves to determine structure and contents damages. Based on the depth of flooding, damages are calculated for each structure using the depth-damage curves. A HEC-RAS hydraulic model was used to calculate existing conditions water surface elevations for the 100, 50, 25, 10, 5, and 2-year frequency discharges. The depth of flooding is determined from the first floor elevations and the computed water surface elevations. The existing conditions delineated 100-year floodplain shows that no structures located in the 1-dimensional study area were located within the floodplain inundation. Therefore, no damages were calculated for any structures there.

To calculate Average Annual Damages for the 2-Dimensional Study Area a different method was required. It was necessary to provide Raster Tifs for all 6 flood frequencies, 100-YR, 50-YR, 25-YR, 10-YR, 5-YR and 2-YR. This was done for "Plan97_Existing_Cond", which was the Existing Conditions HEC-RAS Model.

Point locations of 316 structures were evaluated to determine which ones were impacted by flood waters from the raster profiles. ESRI ArcGIS 10.2.2 ArcMap desktop software was used for this evaluation. This included the Extract Multi Values to Points tool of the Spatial Analyst extension. The 24 plan/profile evaluations were conducted using a python script listed in Attachment B. The meta

data output from running the script is posted as Attachment A. Refer to **Appendix D** to view these Attachments. The results are given as shape files.

There is a separate shapefile for each plan that contains the 316 point locations with attribute columns added for flood elevation at each profile i.e. – “F100YR”, “F50YR” “F25YR”, “F10YR”, “F5YR”, and “F2YR”.

HEC-RAS Existing Conditions Model had 9 structures that were impacted by the 100-year flood event. Refer to Map 1 in **(Appendix D)**. Some of these structures were also contacted by the lower frequency profiles as indicated in the relevant fields for each profile.

The final results with only structures that were contacted by flood profiles are provided as feature classes in the “working.gdb” file geodatabase. A separate feature class is given for plan, “P97_exist_contacts”. These results are also listed in a Microsoft Excel spreadsheet, “Kingston_Plan_Eval_Results.xlsx” in **(Appendix D)**. It includes the relevant parameters of the structure such as value and elevations in addition to the flood elevation that reached the structures for each of the given profiles when it occurred.

Alternatives 1, 2, and 3 did not have any structure points in contact with the rasters of the flood profiles. Therefore, there were no flood damages in the 2-Dimensional Model damage area for this alternative and 2-D area damages equal \$0.0.

The final results with only structures that were contacted by flood profiles are provided as feature classes in the “working.gdb” file geodatabase. A separate feature class is given for each of the three plans, “P97_exist_contacts”, and “P81_ChExCul_contact”. These results are also listed in a Microsoft Excel spreadsheet, “Kingston_Plan_Eval_Results.xlsx”. It includes the relevant parameters of the structure such as value and elevations in addition to the flood elevation that contacted the structures for each of the given profiles when it occurred for each of 3 different plans. These plans were “Plan97_Existing_Cond”, “Plan81_Channel_Excavation_Culv” and “Plan93_Reservoir.”

A Damages Excel Spreadsheet was set up which contained the U.S. Army CORPS of Engineers Depth Damage Curves and corresponding percent damage. Damage occurs to those structures when the water physically comes into contact with the structure. Thus, these structures were

identified as described above and were entered into the Damages Spreadsheet. The surveyed first floor elevations for each structure and the water surface elevations provided for each frequency event were used to determine depth of flooding. This information was also entered into the Damages Spreadsheet. Thus, based on the depth of flooding, damages are calculated for each structure using the depth-damage curves and percent damage values. The Damages Excel spreadsheet is too extensive to be included in the body of this report but can be found in **Appendix D**.

Based on this damages analysis there is a total of 9 structures in the Windhaven Drive and Sandra Street Subdivision area which are damaged. The total Average Annual Damages were calculated to be \$10,017.71. This amount includes 15% non-tangible damages which is a standard amount used to account for other losses associated with flooding issues that include flood clean up such as, acquiring dumpsters, hauling, traffic control due to road closures, and setting up barricades. Average Annual Damages is the average amount of damages caused by flooding each year. See **Appendix D** for a list of the structure data and a summary of the damages.

VII. ALTERNATIVES INVESTIGATED

A detailed investigation of three alternatives was conducted to determine the amount of flood damages reduction that could be provided with implementation of each alternative. These alternatives are listed and described below.

Alternative 1 – Culverts Replacement Excavated Channel Conveyance (plan 41)

Alternative 2 – Culverts Replacement Excavated Channel Conveyance (Plan 81)

Alternative 3 – Upstream Storage Reservoir (Plan 93)

Alternative 1

This alternative investigated includes construction of three 6' x 3' concrete box culverts at Sandra St. to replace the existing two 3.5 foot by 2.5-foot pipe arch culverts. This alternative includes channel excavation commencing at the downstream face of Sandra St. which extends downstream to X-

Section 3265.067, a distance of 1,093 feet. The cut depth is 3.75 feet with a 4-foot bottom width and 3h to 1v side slopes (**Figure 6**). The proposed channel slope is .00954 foot per foot.

Approximately 922 feet downstream of this channel improvement, another channel improvement begins. This channel excavation commences at X-Section 2342.97 and ends at the upstream face of the 1st Street Culvert. This is an additional 590 feet of channel excavation to 1st Street. The excavation slope is .0121 foot per foot. The excavation depth is 2.5 feet with a 2-foot bottom width and a 3h to 1v side slope to X-Section 1943.47. At this location the bottom width changes to 10 feet and the channel side slopes become 2h to 1v with rip rap. The concrete drop structure just upstream of 1st Street would be removed since the new channel slope would tie into the existing box culvert invert elevation at 1st Street.

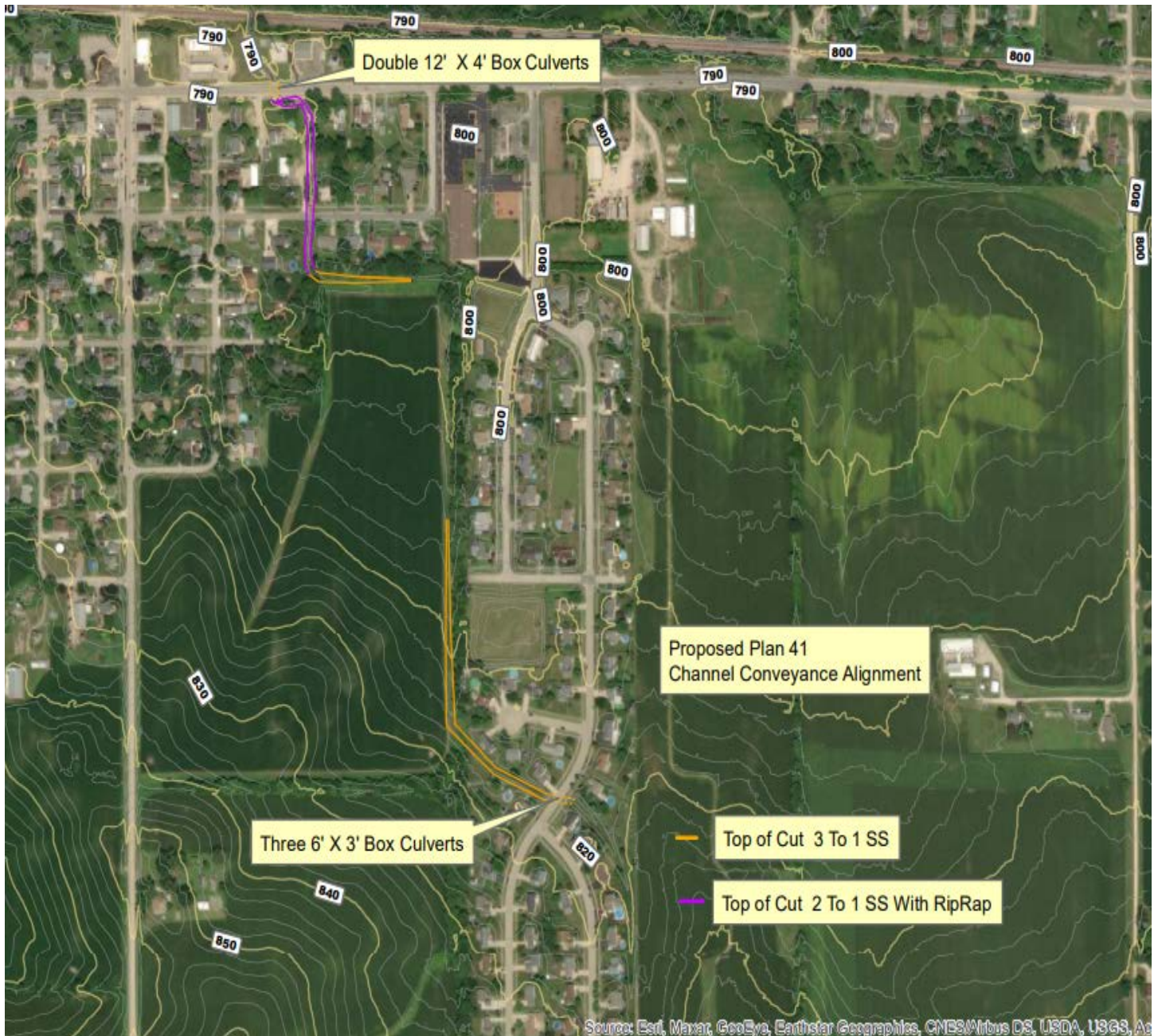


FIGURE 6 - Alternative 1 Plan View Channel Project Description Map

Additional channel excavation from the downstream face of the of 1st Street culvert to Illinois Route 72 would be included using the existing channel invert elevations. A 10-foot bottom width and an excavation depth of 5.0 feet is used with side slopes at 2h to 1v. The 2h to 1v side slopes will be lined

with rip rap. This additional improvement reach totals 545 feet. In addition, the double 8' x 4' RC Box Culvert at Illinois Route 72 would be replaced with a double 12' x 4' RC Box Culvert.

This alternative reduces the maximum water surface elevation for the 100-year frequency beginning at the upstream face of the Sandra St. box culverts by about 1.1 feet from 817.48 ft. to 816.35 ft. This reduction continues upstream but decreases to 0.5 feet about 180 feet upstream. The reduction in elevation decreases to 0 about 500 feet upstream of Sandra Street. Because of the 1.1-foot reduction in water surface elevation upstream of Sandra Street, overflows into the subdivision which typically have caused flooding, are reduced considerably with this alternative. Even though there is still some overflow into the subdivision, it is relatively minor and there are no structures that are flooded. This alternative minimizes overflow into the Windhaven Drive and Sandra Street subdivision area and eliminates structural damages. **(Figure 7)**.

As a result of the channel excavation downstream of the proposed Sandra St. Box Culverts, the maximum water surface elevation for the 100-year frequency is reduced even though the discharge increases by about 88 cfs. The water surface reduction downstream varies from 0.04 feet just downstream of the Sandra St. Culverts to 1.2 feet at X-Section 3983.13, and then to 0.01 feet at X-Section 3364.77. The elevations then begin to gradually increase and are 0.26 feet greater than existing conditions at X-Section 3245.26 and then the change decreases to 0 feet at X-Section 2362.09. Then the elevations drop below existing conditions and vary from 0.13 feet at X-Section 2303.89 to around 1.95 feet at X-Section 1961.81. The reduction then gradually dissipates to 0.07 feet at X-Section 1822.36 just upstream of 1st Street.

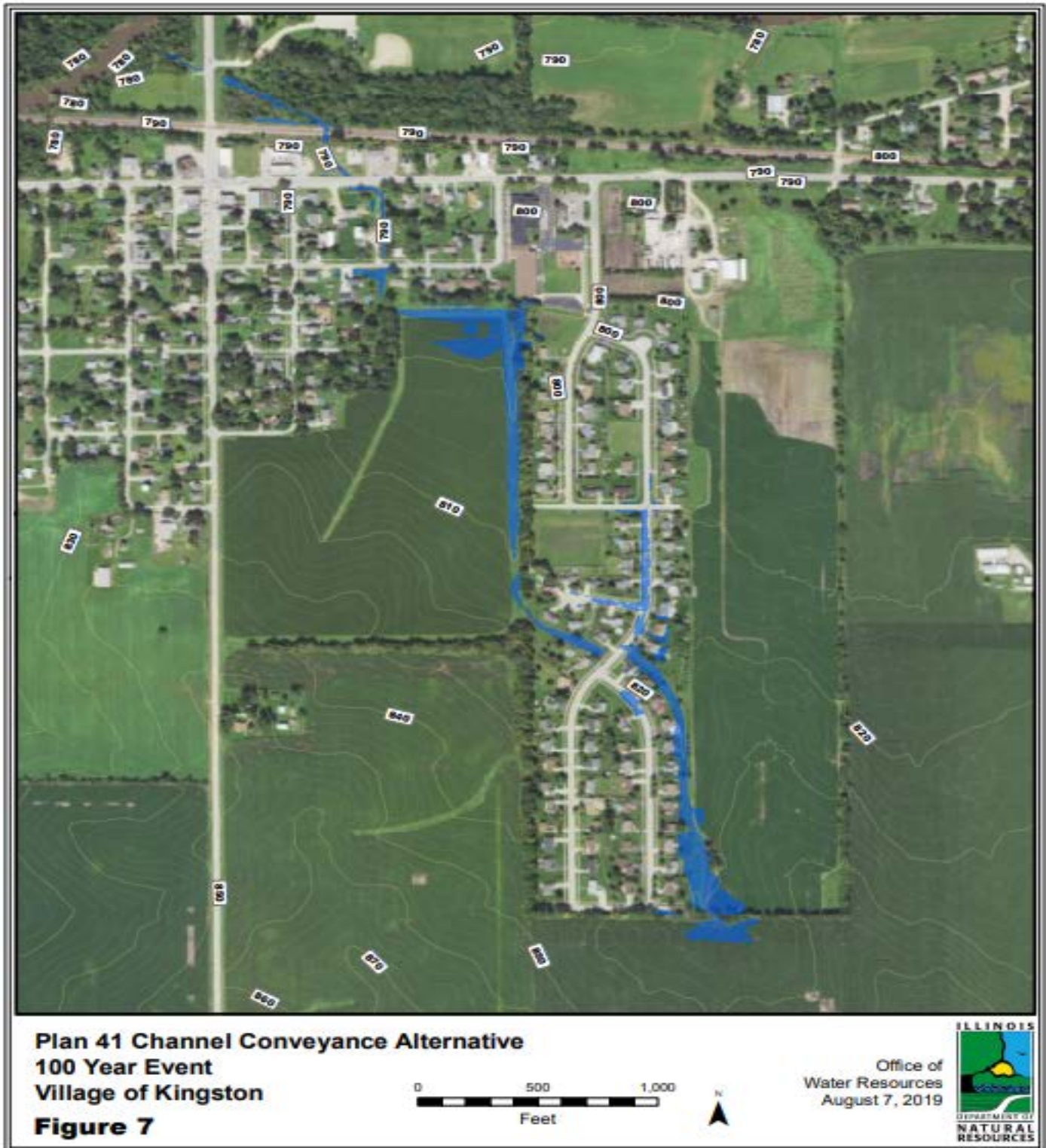


FIGURE 7 - Alternative 1 Channel Project 100 Year Floodplain Map

The 100-year water surface elevations are reduced just downstream of 1st St. by 0.22 feet and range to 0.48 feet at X-Section 1431.49 about 320 feet downstream of 1st St. The reduction begins to gradually decrease to 0.05 feet at X-Section 1320.75 about 430 feet downstream. At the next

downstream X-Section there is a gradual increase in water surface elevation to ranges up to 0.71 feet just downstream of the Il. Route 72 bridge. The increase then ranges from 0.67 feet to 0.03 feet over the next 400 feet and goes to 0.0 feet increase at X-Section 754.19.

As a result of the modified channel, velocities increase in some reaches throughout the conveyance improvement. At several locations, rip rap protection is proposed to protect the channel from erosion where the increased velocities exceeded scour velocity thresholds.

Alternative 2

This alternative investigated was Plan 81 which is almost identical to Plan 41 just described above. The only difference between these alternatives was the channel excavation in the channel reach just upstream of 1st Street at X-Section 1943.47 to Illinois Route 72. The channel excavation in this reach differs in that a 20 foot bottom width is used instead of a 10 foot bottom width. Also, vertical walls are used instead of 2h to 1v side slopes (**Figure 8**). The vertical walls would be constructed using split face block. This Alternative requires less land rights to be acquired from property owners for construction through the channel reach where the vertical walls are used. This alternative reduces the maximum water surface elevation for the 100-year frequency beginning at the upstream face of the Sandra St. box culverts by about 1.1 feet from 817.48 ft. to 816.38 ft. This reduction continues upstream but decreases to 0.5 feet about 180 feet upstream. The reduction in elevation decreases to 0 about 425 feet upstream of Sandra Street.

Because of the 1.1 foot reduction in water surface elevation upstream of Sandra Street, overflows into the subdivision which typically have caused flooding, are reduced considerably with this alternative (**Figure 9**). Even though there is still some overflow into the subdivision, it is relatively minor and there are no structures that are damaged by flooding. This alternative minimizes



FIGURE 8 - Alternative 2 Plan View Channel Project Description Map



Figure 9 - Alternative 2 Channel Project 100 Year Floodplain Map

overflow into the Windhaven Drive and Sandra Street subdivision area and eliminates structural damages.

As a result of the channel excavation downstream of the proposed Sandra St. Box Culverts, the maximum water surface elevation for the 100-year frequency is reduced even though the discharge increases by about 88 cfs. The water surface reduction downstream varies from 0.04 feet just downstream of the Sandra St. Culverts. to 1.2 feet at X-Section 3983.13, and then to 0.01 feet at X-Section 3364.77. The elevations then begin to gradually increase and are 0.26 feet greater than existing conditions at X-Section 3245.26 and then the change decreases to 0 feet at X-Section 2381.22. Then the elevations drop below existing conditions and vary from 0.07 feet at X-Section 2362.09 to around 2.07 feet at X-Section 1961.81. The reduction then gradually dissipates to 0.05 feet at X-Section 1822.36 just upstream of 1st Street.

The 100-year water surface elevations are reduced just downstream of 1st St. by 0.56 feet and range to 0.43 feet at X-Section 1431.49 about 320 feet downstream of 1st St. The reduction begins to gradually decrease to 0.03 feet at X-Section 1337.336 about 412 feet downstream. At the next downstream X-Section there is a gradual increase in water surface elevation to ranges up to 0.70 feet just downstream of the II. Route 72 bridge. The increase then ranges from 0.66 feet to 0.03 feet over the next 400 feet and goes to 0.0 feet increase at X-Section 754.19.

As a result of the modified channel, velocities increase in some reaches throughout the conveyance improvement. At several locations, rip rap protection is proposed to protect the channel from erosion where the increased velocities exceeded scour velocity thresholds

Alternative 3

Alternative 3 would consist of constructing a reservoir upstream of the subdivision in the farm field south of the Village (**Figure 10**). This reservoir would be a gravity in and gravity out design in that the embankment would be constructed at the lower end of the reservoir area to impound runoff above ground behind the embankment. One 24-inch diameter RCP culvert would be constructed through the embankment at an invert elevation of 834.0 feet. This culvert was sized to convey some

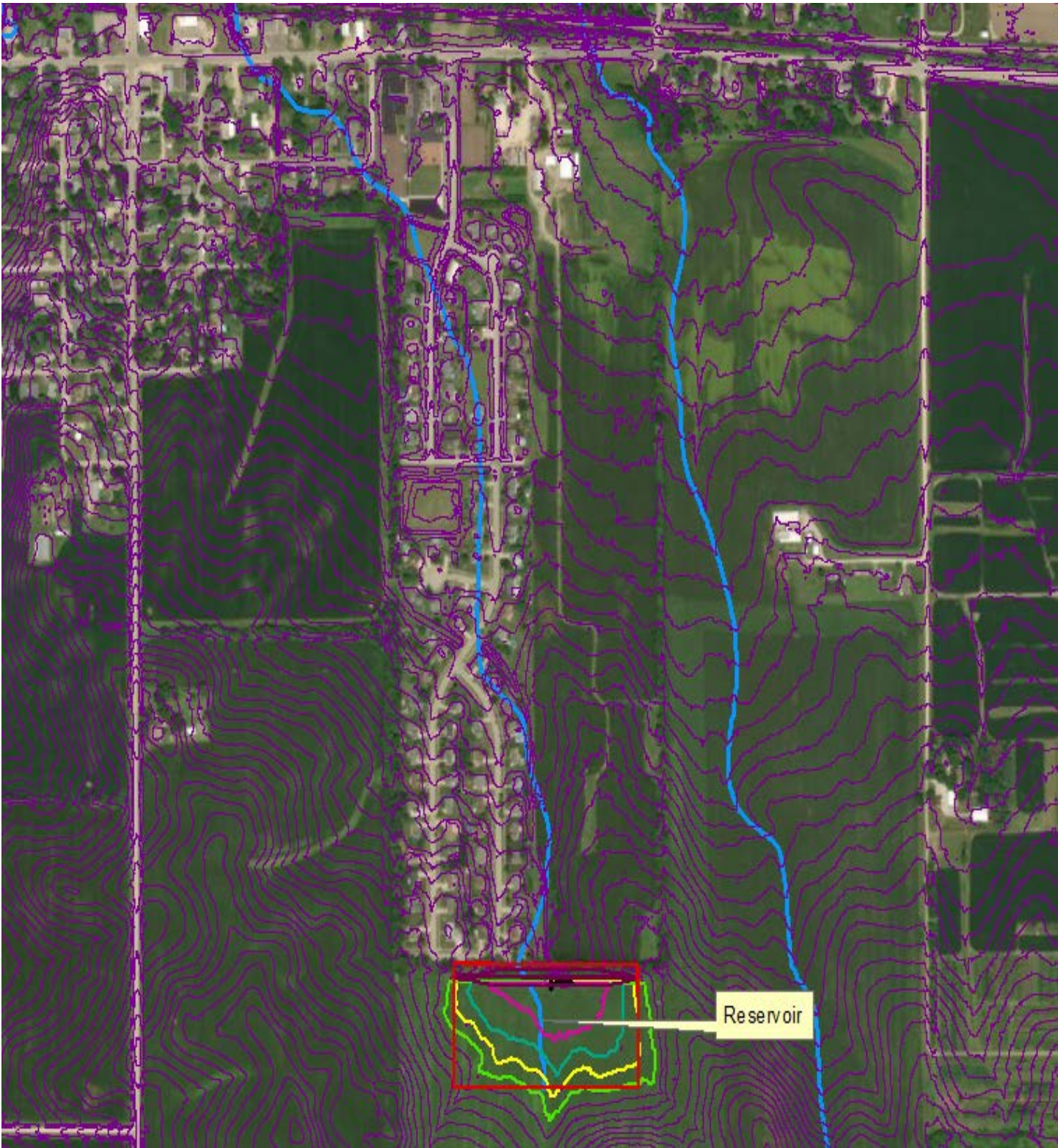


Figure 10 - Alternative 3 Reservoir Project Location Map

discharge but restrict flow such that water would back up into the reservoir. As storm event runoff decreases the culvert would drain or dewater the reservoir. This reservoir would store runoff from Sub Area F which would reduce discharges downstream at the Sandra St. culverts.

An earthen embankment would be constructed at a height of 7.15 foot to elevation 841.15 feet, to impound storm water runoff. The total storage volume of the reservoir at the 100-year frequency discharge would be 12.9 acre-feet at an elevation of 839.07 feet. Total discharge through the 24-inch RCP would be 32.0 cfs. The total area inundated at the 100-year frequency would be 5.8 acres. The reservoir would have a rip rap armored earthen emergency spillway with a crest elevation of 839.15 feet and a length of 100 feet.

By storing runoff volume in the reservoir upstream, flows are reduced at the two existing 3.5 foot by 2.5 foot corrugated metal pipe arch culverts at Sandra Street. As a result, this alternative reduces the maximum water surface elevation for the 100-year frequency event at the upstream face of the Sandra Street. The 100-year frequency elevation at the Sandra Street culverts is reduced by 1.56 feet from 817.48 ft. to 815.92 ft. This reduction minimizes overflow into the Windhaven Drive and Sandra Street subdivision area and eliminates structural damages. Refer to **Figure 11** which shows the 100-year floodplain for this alternative 3.

Downstream of Sandra St. the maximum water surface elevation is reduced by 0.79 feet and continuing downstream the reduction gradually decreases. At a distance of 1,211 feet downstream, the reduction is 0.65 feet. Continuing further downstream the reduction ranges from 0.65 feet to 1.08 feet to 1st Street. Downstream of 1st Street the reduction in elevation ranges from 1.08 feet to 0.89 feet all the way to the most downstream railroad bridge. Downstream of the railroad bridge the reduction ranges in elevation from 1.9 feet to 0.5 feet.

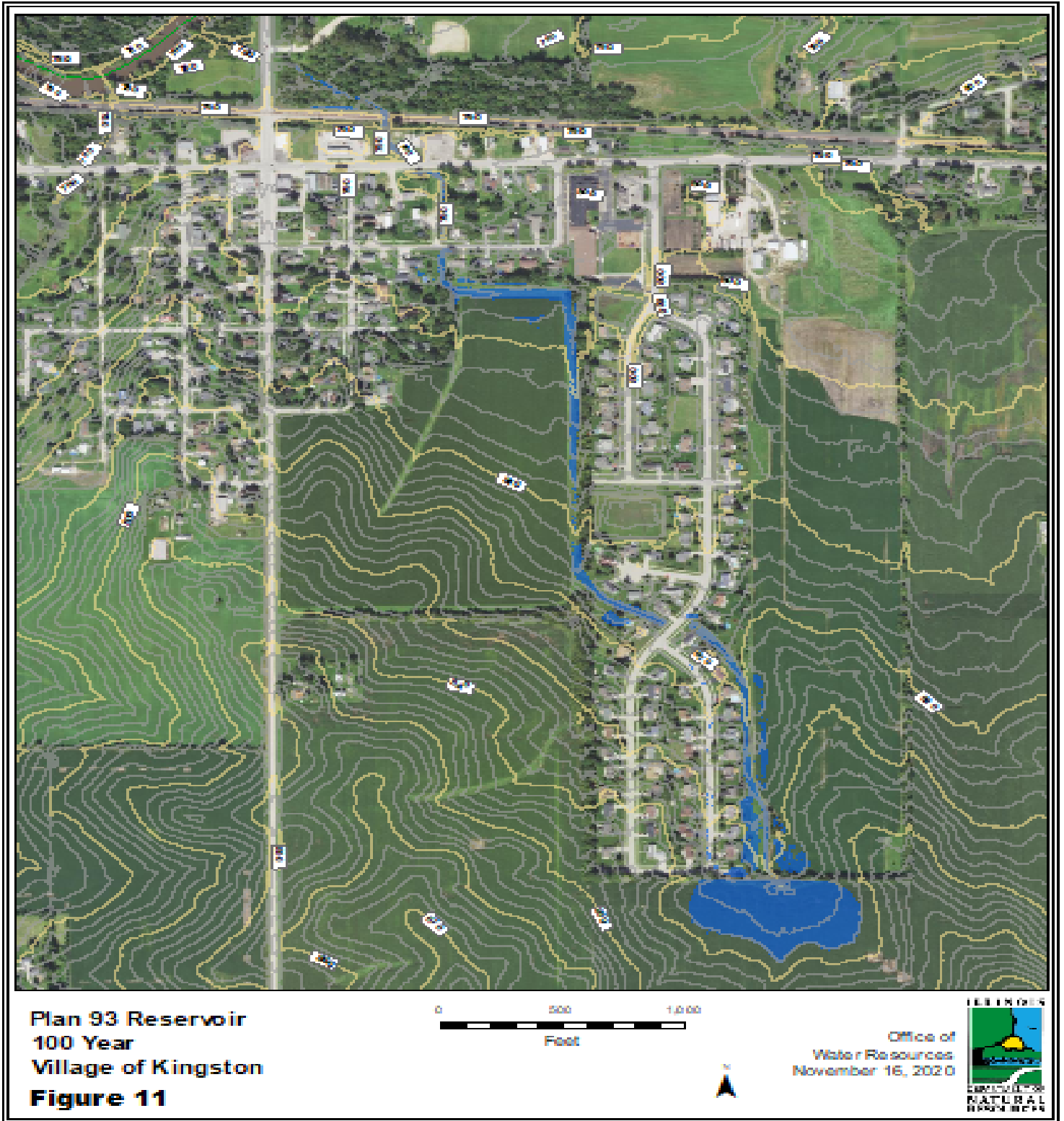


FIGURE 11 - Alternative 3 Reservoir Project 100 Year Floodplain Map

VIII. DAM SAFETY PERMIT MODELING

This Reservoir would likely be classified as a Small High Hazard Dam by the OWR Dam Safety Program because of the residences located downstream of the reservoir. This would require that a permit be obtained from the Dam Safety Program for the reservoir. There are several different calculations and several different Hydrologic and Hydraulic models that are required for a Dam Safety permit (**Appendix E**). As part of the permit application an analysis of the reservoir is required using 0.5 times the Possible Maximum Precipitation (PMP). The PMP is 32 inches of rainfall in a 24-hour period and 0.5 times the PMP is 16 inches of precipitation. The 24-hour duration is used because that is the duration required by OWR Dam Safety Section. Using 0.5 PMP results in the 0.5 Possible Maximum Flood (PMF). Thus, 0.5 PMF is the amount of runoff that is generated from the drainage area by one half of the PMP.

The total 0.5 PMF runoff amount from the drainage area located upstream of the reservoir must be routed through the reservoir. The amount of runoff from the 0.5 PMF storm that can't be stored in the reservoir, must be conveyed by the 24-inch diameter RCP outlet and the emergency spillway without overtopping the dam embankment. The HEC-RAS analysis shows that the 0.5 PMF runoff is conveyed by the 24 -inch diameter outlet pipe and the emergency spillway at elevation 839.63 feet. Thus, the 0.5 PMF runoff does not overtop the reservoir embankment which is at elevation 841.15 feet. Therefore, the reservoir outlet system would meet the required criteria. The maximum storage volume for the 0.5 PMF storm event is 16.4-acre feet. The maximum discharge over the emergency spillway and through the 24-inch diameter pipe combined is 149 cfs. The emergency spillway would convey 117 cfs and the 24-inch diameter outlet pipe would convey 32 cfs.

The HEC-RAS analysis for the 100-year storm event with the proposed Alternative 3 Reservoir, results in a maximum elevation of 839.07 feet. Therefore, the maximum water surface elevation in the reservoir is below the crest of the emergency spillway which is at elevation 839.15 feet.

Wave runup and wind setup calculations were performed for the reservoir for the 100-year frequency event. The increase in elevation that resulted was 0.93 feet. When this increase is added to the 100-year elevation of 839.07, the total water surface elevation in the reservoir is 840.0 feet. Thus, this will not overtop the reservoir embankment at 841.15 feet and results in 1.15 feet of freeboard. The 100-year reservoir storage volume of 12.9 acre feet would be dewatered by the 24-inch diameter pipe in 20 hours.

Figure 12 shows the 100-year dam breach analysis that was analyzed using HEC-RAS modeling. The final breach width used was 30 feet with 0.5h to 1v side slopes and a final bottom elevation of 834.0 feet. The overtopping method was used along with a breach time of 0.25 hours. No damages occurred to any structures as a result of the dam breach analysis.

The 0.5 PMF dam breach analysis was analyzed using HEC-RAS modeling (**Figure 13**). The final breach width used was 30 feet with 0.5h to 1v side slopes and a final bottom elevation of 834.0 feet. The overtopping method was used along with a breach time of 0.2 hours. The failure was triggered using a water surface elevation plus duration. The water surface used was 834.0 feet along with a duration of 15 hours above the 834.0 feet threshold elevation.

The 0.5PMF dam breach does result in damages to four structures (**Figure 14**). The four structures would have water inside the attached garages but not inside the first floor. Three of the four structures would sustain fewer damages than the at the 25-year frequency because the 0.5 PMF dam breach water surface is less than the 25-year water surface elevation. The fourth structure would have fewer damages than at the 50-Year frequency event since the 0.5PMF dam breach water surface elevation is less than the 50-year water surface elevation.

The existing conditions 0.5 PMF storm event was also modeled using HEC-RAS and therefore, no reservoir is included in this analysis. The existing conditions analysis was used for comparison to the 0.5 PMF dam breach analysis which of course does include the proposed reservoir (**Figure 15**). As mentioned above, the 0.5 PMF is based on one half of the PMP which is 16 inches of rainfall in a 24-hour period. Thus, 0.5 PMF is the amount of runoff that occurs in the watershed from one half of the PMP amount.

Four structures are flooded by the 0.5 PMF Plan 93 Reservoir dam breach. The 0.5 PMF dam breach analysis elevations range from .031 feet to .327 feet lower than the 0.5 PMF existing conditions elevations at these same structures. Therefore, all four structures have less flooding than occurs with the existing conditions 0.5 PMF flood event.

A formal maintenance plan would be required to be developed which specifies what maintenance activities will be performed. A timetable for each of the specified maintenance activities would also need to be specified. In addition, an Emergency Action Plan would need to be developed which is a

formal document that identifies the possible emergency conditions at the dam. This plan would identify all action that would be taken in the event of an emergency and by which entities.

IX. CONCLUSIONS

A total of three Alternatives were investigated as described in this report. Two of these alternatives increased channel conveyance and were effective to address the flood issue without causing any additional flood issues. The Channel Conveyance Alternatives eliminated all structure damages. These alternatives were Plan 41 and Plan 81. The Plan 93 Reservoir was a storage alternative which also eliminated all damages (**Table 2**).

A total of 9 structures flooded at the 100-year frequency event, 6 at the 50-year, and 5 at the 25-year frequencies located in the Windhaven Drive and Sandra Street subdivision area. None of these structures were damaged at the 10-, 5-, and 2-Year frequency events. These three alternatives all provide 100-year frequency flood protection for these 9 structures that sustained damages, based on this study.

Although the three alternatives eliminate all structure damages, the Alternative recommended is the Plan 93 Reservoir. This Alternative generates average annual benefits of \$10,017.71 and has a benefit to cost ratio of 0.56. This is the highest benefit to cost ratio of the alternatives considered.

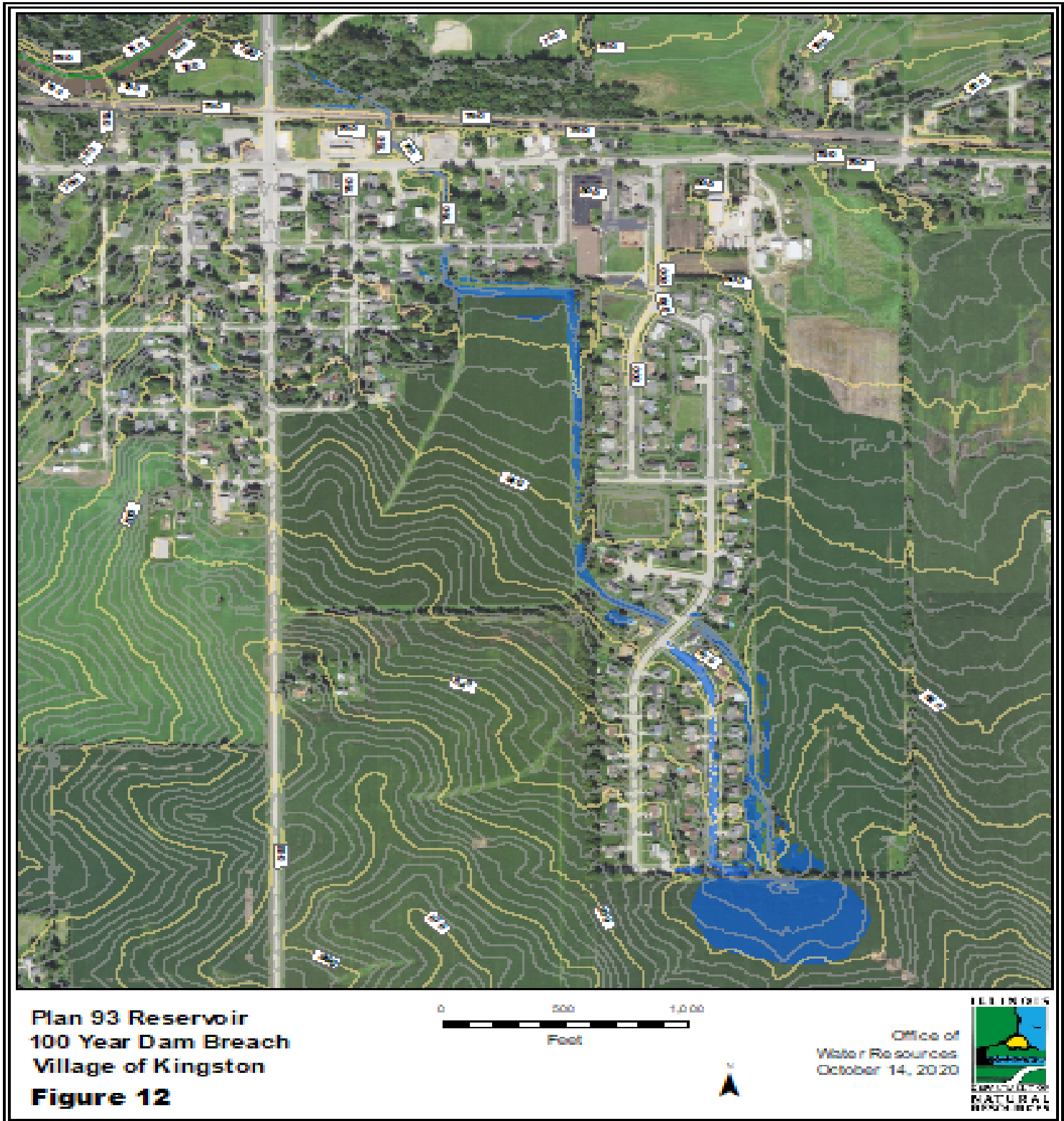


FIGURE 12 - Alternative 3 Reservoir Project 100 Year Dam Breach Floodplain Map

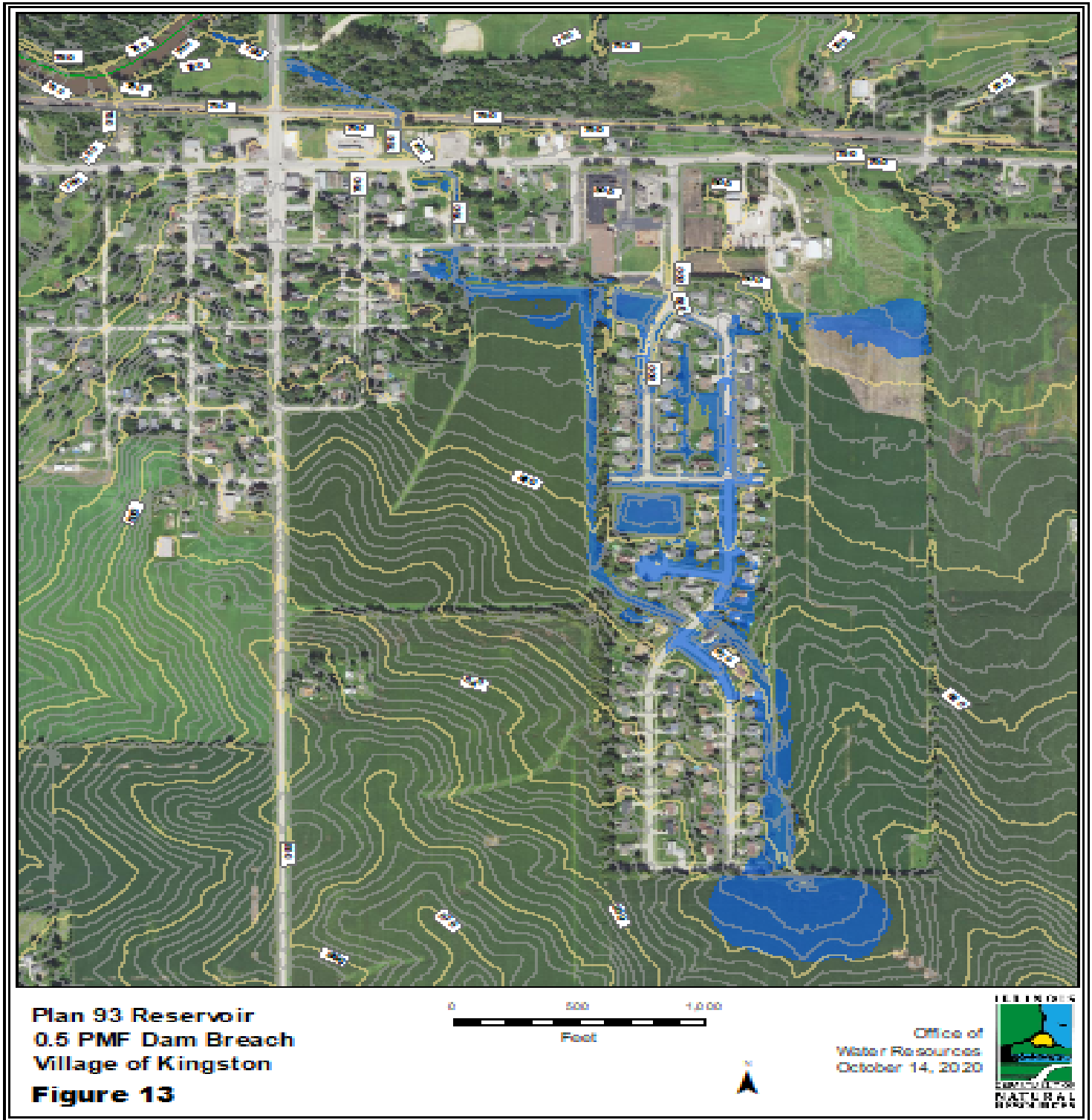


FIGURE13 - Alternative 3 Reservoir Project 0.5 PMF Dam Breach Floodplain Map



FIGURE 14 - Alternative 3 Reservoir Project 0.5 PMF Dam Breach Damaged Structures Map (Red Dots)

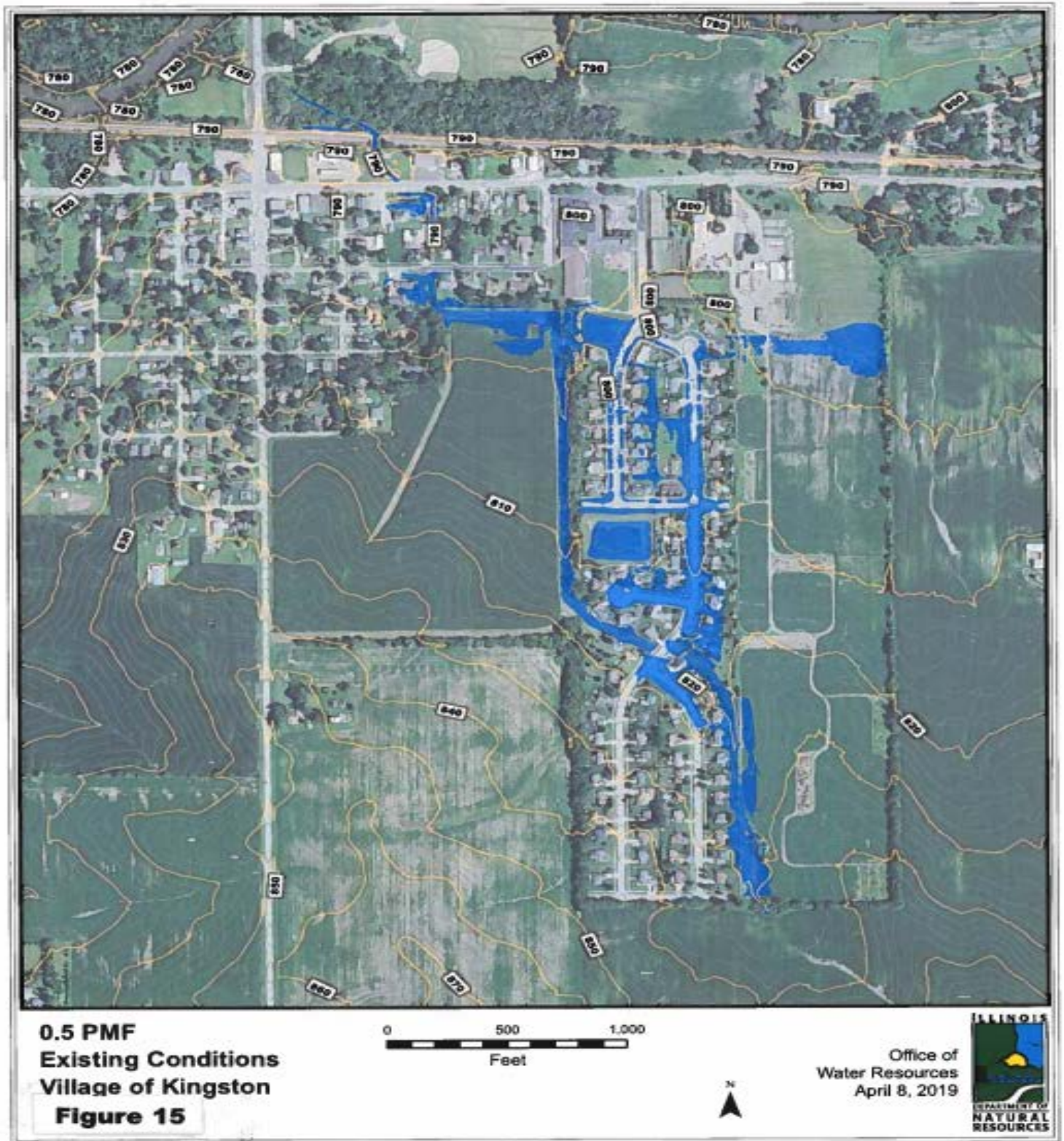


FIGURE 15 - Existing Conditions 0.5 PMF Floodplain Map

The benefit to cost ratio for Plan 41 Channel Conveyance and Plan 81 channel conveyance was 0.26 and 0.18 respectively.

Alternative 3 Reservoir Project is also the lowest cost alternative of the alternatives. This alternative would have a total first cost of \$393,221.33 and the total annual cost would be \$17,914.16. The total annual benefits would be \$10,017.71. As stated above this alternative has a benefit to cost ratio of 0.56. Alternative 1 Channel Project would have a total first cost of \$935,666.54 and total annual cost of \$39,057.34. Alternative 2 Channel Project would have a total first cost of \$1,343,331.73 and a total annual cost of \$55,653.94.

Since Alternative 3 Reservoir Project has the highest benefit to cost ratio, it should be considered for implementation. The total first cost of this alternative is \$393,221.33 which includes construction, mobilization, contingencies, land rights, engineering and design (Appendix F). The land rights costs are \$195,071.05 and the construction costs are \$198,150.28. The land rights costs are based on an appraisal obtained by the Village. OWR could fund all construction costs and the Village would be required to fund the land rights costs of \$195,071.06.

The amount of funding OWR can provide is based on capitalized benefits and these are computed by multiplying the Average Annual Benefits by 26.35185 which is the reciprocal of the Capital Recovery Factor (CRF). The CRF is calculated using the U.S. Army CORPS of Engineers interest rate of 2.875% for Fiscal Year 2021. This interest rate is applied based on a project life of 50 years to yield the CRF. The capitalized benefits are calculated to be \$263,985.19 based on the amount of benefits. Thus, OWR could fund construction costs of a project up to this amount. However, this is a greater amount than the Alternative 3 construction costs and OWR can only fund up to the amount of the construction costs, which is \$198,150.28.

The Village is required to purchase all necessary land rights for the project. The Village would also be responsible to pay for the relocation of any utilities necessary for construction of the Project. Utilities relocation costs have not been quantified. It should be noted that all damages, benefits, and costs are October 2018 dollars and can be found in **Appendix F**.

Table 2, a Kingston Alternatives Comparison Matrix spreadsheet shows the AAB, project construction costs and total project costs for each of the alternatives. OWR funding is based on the amount of benefits generated and is equal to the amount of capitalized benefits determined using the current Federal Discount Rate referenced above. However, the funding amount provided can't exceed the total project construction costs. Also, all dollar amounts used are October 2018 dollars.

ALTERNATIVES		AAD	AAB	Structures Removed From 100-YR Floodplain	Project Construction Costs	Total Project Cost	B/C RATIO	TOTAL REDUCTION OF DAMAGES
		OCTOBER 2018 DOLLARS	OCTOBER 2018 DOLLARS					
(NONE) EXISTING		\$10,017.71	\$0.00	0	\$0		0.00	\$0
ALTERNATIVE 1								
PLAN 41								
CHANNEL CONVEYANCE								
Construct three 6'X3' RC Box Culverts at Sandra St.	Excavate Channel Improvement D.S. of Sandra St.	\$0.00	\$10,017.71	9	\$914,602.60	\$935,666.54	0.26	\$213,086.00
Excavate Channel Improvement D.S. of lateral	Structure 2880; Remove inline Structure at 1St St.							
Excavate Channel D.S. of 1st St. to IL. Route 72	Use RR4 Rip Rap From X-Sec 1943.47 to Il Rt. 72 W/ 2:1 SS							
Replace Existing Box Culverts at IL Route 72	with Two 12'X 4' RC Box Culverts							
ALTERNATIVE 2								
PLAN 81								
CHANNEL CONVEYANCE								
Construct three 6'X3' RC Box Culverts at Sandra St.	Excavate Channel Improvement D.S. of Sandra St.	\$0.00	\$10,017.71	9	\$1,324,261.21	\$1,343,331.72	0.18	\$213,086.00
Excavate Channel Improvement D.S. of lateral	Structure 2880; Remove inline Structure at 1St St.							
Excavate Channel D.S. of 1st St. to IL. Route 72	Use Segmental Concrete Block Wall D.S. 1ST St.							
Replace Existing Box Culverts at IL Route 72	with Two 12'X 4' RC Box Culverts							
ALTERNATIVE 3								
PLAN 93								
RESERVOIR								
Construct Earthen Embankment to Elevation 841.15	Construct 1- 24" Diameter RCP outlet at Elev. 834.0 Ft.	\$0.00	\$10,017.71	9	\$198,150.28	\$393,221.33	0.56	\$213,086.00
Construct 100 Ft. Emergency Spillway at Elev. 839.1	100 YR Pool Elev. At 838.06 Ft.; Storage Vol. = 13.8 Ac-Ft.							
100 YR. Pool Surface Area = 8.05 Acres								

Table 2 - Alternatives Comparison Matrix

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