US Army Corps of Engineers
Omaha District

SECTION 22, PLANNING ASSISTANCE TO STATES STUDY

YELLOWSTONE RIVER

CITY OF GLENDIVE/DAWSON COUNTY, MONTANA

DECEMBER 2014

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PLANNING ASSISTANCE TO STATES STUDY

FINAL

DECEMBER 2014

Prepared by

U.S. Army Corps of Engineers

Omaha District

For

City of Glendive, Montana

And

Dawson County, Montana

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YELLOWSTONE RIVER
DAWSON COUNTY, MONTANA
PLANNING ASSISTANCE TO STATES STUDY

1.0 Introduction

Purpose and Authority of Study

City of Glendive and Dawson County officials have partnered with the US Army Corps of Engineers, Omaha District (Corps) to conduct a Section 22 Planning Assistance to States study to investigate the conceptual flood risk management alternatives for the City of Glendive and surrounding area. Section 22 of the Water Resources Development Act (WRDA) of 1974, as amended, provides authority for the Corps of Engineers to assist the States, local governments, Native American Tribes and other non-Federal entities, in the preparation of comprehensive plans for the development and conservation of water and related land resources.

Flooding, typically from ice jams, is a high risk to residents and business owners in Glendive. Damaging ice jam floods have occurred on the Yellowstone River in Glendive 30 times since 1890 including major ice jam floods in 1899, 1920, 1936, 1969, 1986, and 1994. A total of 16 deaths have occurred as a result of these events. The Corps of Engineers constructed a levee to provide flood protection to West Glendive in 1959. The levee was designed to protect against an open water flood discharge of 200,000 cubic feet per second with 3 feet of freeboard. The levee was not designed specifically to protect against ice jams, but the design elevation of the levee was compared against the 1936 ice jam elevation and was found to still have 3 feet of freeboard. The ice jam floods of 1969, 1986, and 1994 all came to within 0.5-1.5 feet of overtopping the West Glendive Levee.

A Section 22 study was recommended to the City of Glendive to examine the flood risks and evaluate alternative measures which could be implemented to improve flood risk management. While the Section 22 study will not in itself lead to construction of a flood risk management project, it has provided an opportunity for the Corps to work with the City and County of Glendive to develop a better understanding of the flood risk, and quantify the frequency of flooding, the depths of flooding for various events, and likely damages associated with flooding. Although this study focuses on a few potential flood risk management alternatives, it is not a comprehensive feasibility study and does not study all potential alternatives nor recommend a specific alternative for implementation. Since this analysis was completed as part of the PAS Study, the purpose is not to seek construction authorization for a recommended alternative, but rather to provide planning assistance to the City of Glendive and Dawson County as they consider their options in relation to their existing flood risk conditions as well as potential opportunities for flood risk reduction in the future.

1.1 Scope of the Study

1.1.1 Area Description

The City of Glendive is located in Dawson County in Eastern Montana along the Yellowstone River approximately 90 miles upstream from the river's confluence with the Missouri River (Figure 1). The Yellowstone River flows from southwest to northeast through the city dividing it into Glendive proper and West Glendive. Glendive proper is located on the high, right bank bluff overlooking the Yellowstone River valley. West Glendive is located on the left bank floodplain and is protected from flooding by the West Glendive Levee (Figure 2).

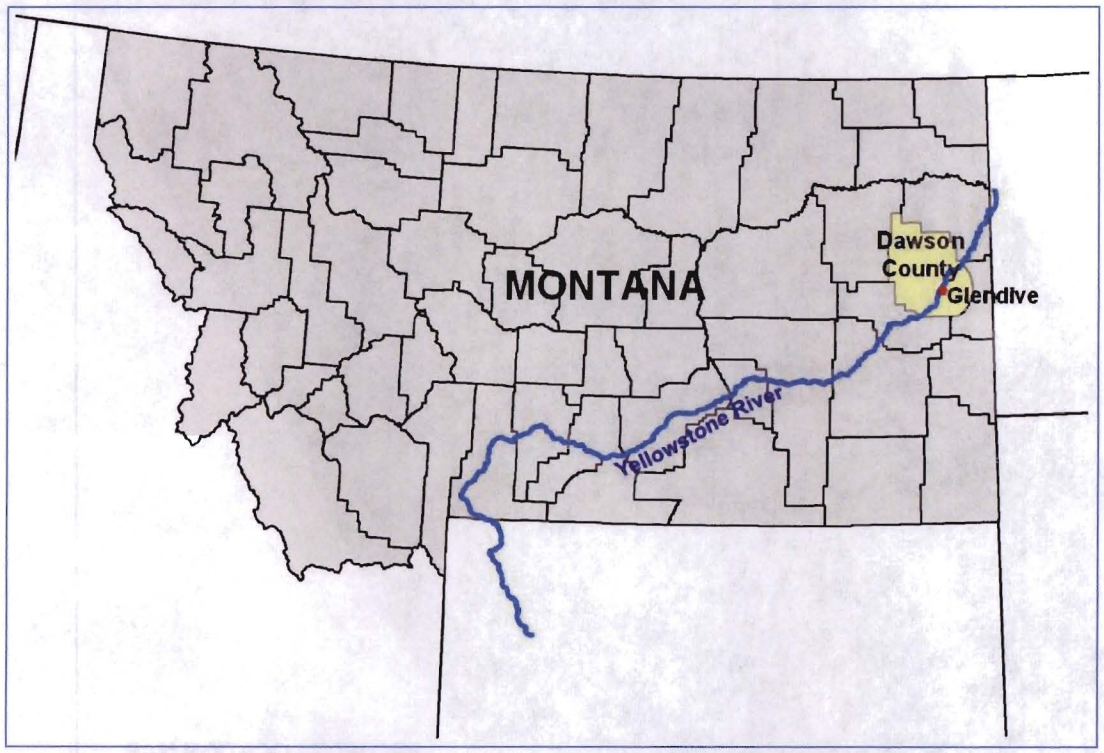

Figure 1. Location Map Yellowstone River, Dawson County and the City of Glendive, Montana

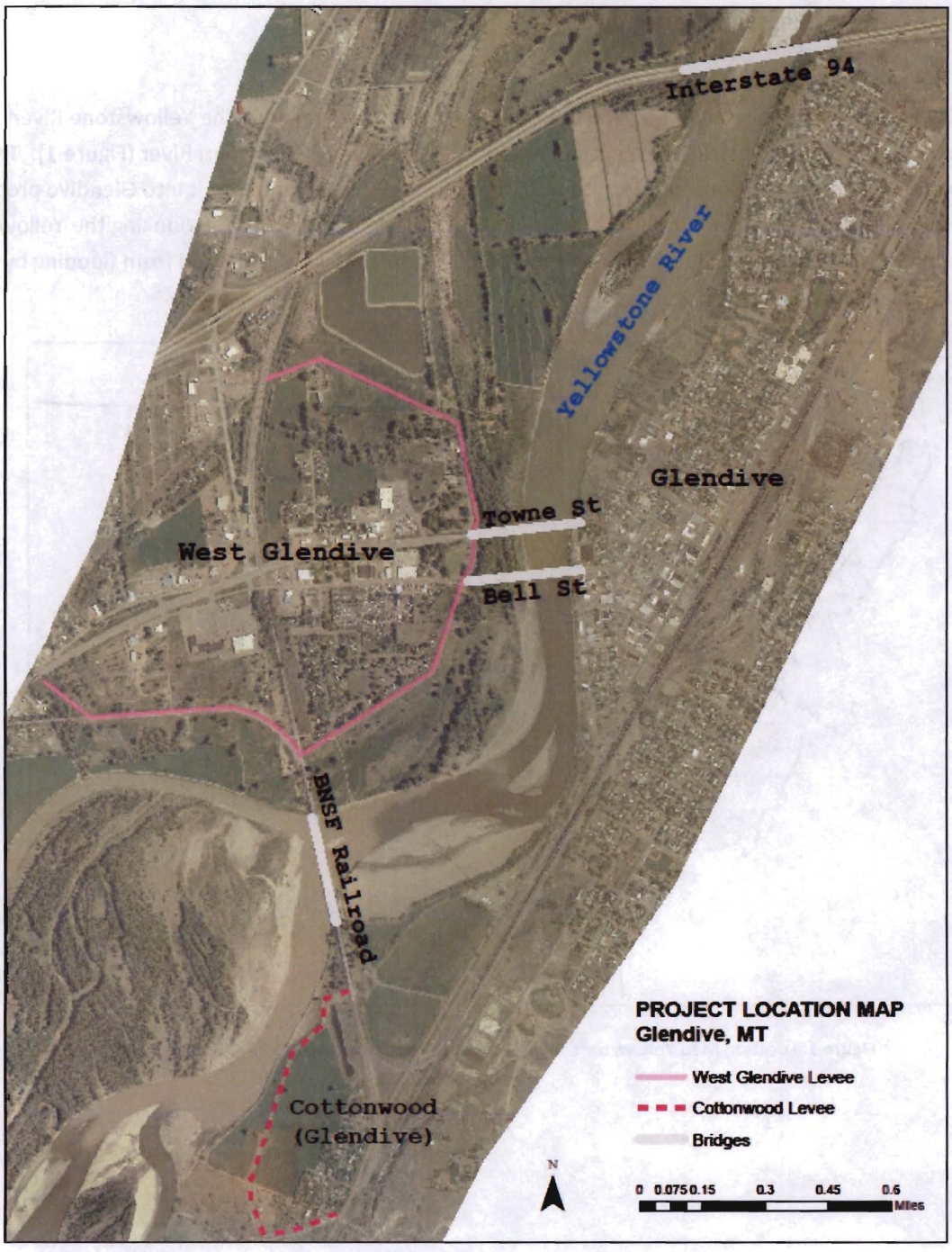

Figure 2. Location Map Yellowstone River at Glendive, Montana

Existing Project. The West Glendive Levee, completed in 1959, was constructed by the U. S. Army Corps of Engineers–Garrison District to provide protection from flooding on the Yellowstone River. The existing levee was designed for an open water discharge of 200,000 cfs with three feet of freeboard. The top of levee design was cross-checked with historic ice jam events and indicated that the levee top elevation would be approximately three feet above the highest recorded ice stage at that time. The existing

project consists of a levee which, operating in conjunction with the Burlington Northern Railroad (BNRR) embankment, provides some level of protection for the unincorporated area known as West Glendive and a portion of the city of Glendive. A 1,100-foot diversion channel diverts Dry Creek directly into the Yellowstone River about 1/2-mile upstream from the Burlington Northern Railroad (BNRR) Bridge. Approximately 870 feet of levee, connecting the embankments of U.S. Highway 10 (also called the business loop of I-94) and the Circle Branch of the railroad, blocks the old channel of Dry Creek, and closes the upper end of the area to flood waters from this source. An impervious fill approximately 2,860 feet long has been placed on the riverside of the Circle Branch railroad embankment to reinforce it. About 3,150 feet of levee connects the embankment of the BNRR near the junction of the Circle and Sidney Branches north of the Yellowstone River railroad bridge with the U.S. Highway 10 embankment near the left bank of the Yellowstone River. From this highway the levee extends northward downstream along the west bank of the Yellowstone River about 2,100 feet and then west about 2,760 feet to the BNRR (Sidney Branch) embankment.

The existing levee section as constructed consists of a 10-foot crown width topped with a 3-inch gravel roadway. The landside of the levees has one vertical on two 1/2 horizontal (1V on 2.5H) side slopes, and the riverside has 1V on 3H slopes, except where features that were existing required slight modifications. Control berms are located in certain reaches to prevent underseepage. The borrow areas were located riverward from the levee at a minimum distance of 300 feet from the levee toe.

The foundation materials of the existing levee are predominantly characterized by silty, sandy silty, or gravelly sandy silty materials with relatively low plasticity. Underlying these silts are poorly graded gravels and sands.

The General Design Memorandum (GDM) for the West Glendive levee was completed in May 1954. The design levee top elevations were based on the profile for an open-water flood discharge of 200,000 cubic feet per second (cfs) with 3 feet of freeboard for the levee section upstream from the U.S. Highway 10 bridge. This required a levee top elevation of 2,072 feet, msl at the Dry Creek Diversion and 2,065 feet, msl at the U.S. Highway 10 bridge. The design was also checked to provide protection from floods caused by ice jams. The levee top elevation of 2,065 feet msl at the U.S. Highway 10 bridge was approximately 3 feet above the highest known ice jam flood at that time. The crest grade was set at 2,065 feet, msl for the entire levee section downstream from U.S. Highway 10.

The Yellowstone River is a typical meandering stream as is associated with mature rivers. The Glendive reach has seen much modification due to human development beginning in the late 19th century. In 1902, the Northern Pacific Railroad (currently Burlington Northern–Santa Fe Railroad -- BNSFRR) constructed a nine span iron truss bridge over the Yellowstone River at the upstream side of the city. This bridge is commonly referred to as the "Black Bridge." The elevation of the BNSF Railroad Bridge resulted in the approaches on the left bank requiring embankments as much as 10 feet above the floodplain. These large embankments effectively prevented Yellowstone River flood flows from getting into a large portion of the left bank flood plain. Additionally, the naturally meandering stream was contained to the bridge opening. Two different Highway 10 bridges (one of which is now a pedestrian bridge) were later built in the central part of the city to connect West Glendive and Glendive. These

bridges, in combination with the West Glendive Levee, further helped to contain the river to its current alignment and prevent meandering. During the late 1960's, the Interstate 94 Bridge was constructed just downstream from the city. This large bridge was constructed at a sufficient height to tie into the right bank bluff line which required the construction of a long, elevated road embankment across the left bank floodplain. This embankment varies in height from approximately 10- to 20- feet above the floodplain and acts as a significant obstruction to flood flows forcing nearly all of the flows to go through the bridge opening.

In 1980, the Federal Emergency Management Agency (FEMA) conducted a Flood Insurance Study (FIS) for Dawson County, Montana and the City of Glendive respectively. The 1980 FIS demonstrated that the existing levee did not provide adequate protection from ice jam flooding and, as a result, the majority of the West Glendive portion of the city was included in the 100- year flood plain. After the 1980 FIS the City of Glendive did not adopt the required flood plain management ordinances for participation in the National Flood Insurance Program (NFIP) and consequently FEMA suspended the city's participation in the NFIP. The city continued to allow major development to take place behind the levee during the 1980's. This development was in violation of the NFIP and the city now has 13 structures that are not in compliance with NFIP regulations. FEMA has granted the city temporary reinstatement into the NFIP on the condition that the city actively pursue mitigation of the 13 non-compliance structures.

1.2 Prior Studies

The Yellowstone River has been studied by several agencies and organizations, including the U.S. Army Corps of Engineers (Corps). Three recent USACE studies of the Yellowstone River reach in Dawson County, Montana are described below.

- Flood Plain Management Plan, Glendive, Montana. Omaha District, U.S. Army Corps of Engineers, dated March 2002. This study used HEC-HMS and HEC-RAS to investigate alternatives to reduce the occurrence and severity of flood hazards through nonstructural measures, modification of the Interstate 94 Bridge, and flood warning systems along the Yellowstone River in Glendive, Montana. Results of the study indicate under ice jam conditions, the existing West Glendive Levee provides approximately 30-year flood protection with no freeboard and 10-year protection with four feet of freeboard. The study offers three alternatives to reduce flood damage hazards: levee raise, historic channel restoration at the Interstate 94 Bridge with levee removal and flood plain buyout, and historic channel restoration at the Interstate 94 Bridge with levee raise. The study shows the historic channel restoration at the Interstate 94 Bridge with levee removal and flood plain buyout as the most beneficial alternative; however, a General Investigation Study would be required to pursue this alternative.
- Intake Diversion Dam Modification Project, Lower Yellowstone Project, Montana. Omaha District, U.S. Army Corps of Engineers, Ongoing. This ongoing study includes the National Environmental Policy Act (NEPA) efforts and final design for a fish screen structure located at the U.S. Bureau of Reclamation diversion dam at Intake, located in Dawson County, Montana. The proposed project is a priority for recovery of pallid sturgeon on the Missouri River System. The dam acts as a significant impediment to the migration of fish up and down the river, and

providing fish passage at Intake Diversion Dam would open up a minimum of 165 river miles of additional habitat that pallid sturgeon could utilize for spawning and other purposes. Survey data of the diversion dam obtained for this study was applied to the Dawson County hydraulic model.

- Yellowstone Corridor Study, Montana. Omaha District, U.S. Army Corps of Engineers, Ongoing. This ongoing study is a joint effort of the River Conservation Districts Council and the U.S. Army Corps of Engineers, Omaha District. The interdisciplinary study examines hydrologic, hydraulic, geomorphic, biologic and socioeconomic characteristics of and impacts on the Yellowstone River and adjacent flood plain in a 460-mile reach from Gardiner, Montana, to the confluence with the Missouri River in western North Dakota. Study results are necessary to support the cumulative impacts assessment for the entire river corridor and develop a comprehensive plan that provides for sustainable use of the river and its flood plain for both economic and environmental needs. The hydrologic and hydraulic analysis of the Yellowstone River resulted in producing updated floodplain mapping for the entire river corridor including Dawson County and the City of Glendive.

2. Existing Conditions

2.1 Hydrologic Analysis

Since the main source of flooding for the city is ice jams, flow-frequency curves were required to determine the necessary percent chance exceedance flood flows for Dawson County under ice cover and ice jam conditions. Flow records for the Glendive, MT reach were obtained from the Montana USGS draft report developed to support the Yellowstone River Corridor Cumulative Effect Assessment (USGS, 2013).

A discharge probability relationship was developed utilizing the methodology presented in Bulletin 17b (WRC, 1981) based on the 75 events. Peak flows evaluated are listed in Table 1 and include the 50, 20, 10, 4, 2, 1, 0.5 and 0.2-percent (%) annual chance exceedance (ACE) probabilities.

Table 1. Annual Flow Statistics

Exceedance Probability Percent (%)	Probability Flow (cfs) - Computed	
	Annual	Winter
0.2	151,000	151,000
0.5	136,000	133,000
1.0	125,000	106,000
2.0	114,000	83,600
4.0	103,000	64,900
10.0	86,800	44,800
20.0	74,000	32,300
50.0	54,200	18,400

2.2 Hydraulics Analysis

Because of the frequent recurrence of ice-affected flooding on the Yellowstone River in the City of Glendive, the effects from a floating ice cover and ice jam were analyzed. Significant ice jams have been documented just downstream of the Interstate 94 Bridge, resulting in backwater flooding. The bathymetric data indicate the presence of sandbars and channel constrictions downstream of the Interstate 94 Bridge where ice jams have been reported. These factors typically contribute to ice jam formation. In addition, the HEC-RAS model suggests that the constriction of flow through the Interstate 94 Bridge opening may exacerbate the effects of the backwater flooding caused by the ice jam.

The hydraulic model geometry file includes an ice data table. User defined ice thicknesses, roughness values, and other ice jam parameters can be defined for each cross section. The ice data was incorporated into the model, calibrated to the limited available data, and the results of the current conditions were evaluated.

The composite stage probability was calculated for each cross section in the City of Glendive area to determine the composite ice profiles for the 50, 20, 10, 4, 2, 1, 0.5 and 0.2% chance exceedance events. Composite ice profiles were only calculated for the Glendive area where water surface elevations for ice jam and ice cover conditions exist.

The results from the calculated composite ice stages are shown in Figure 3 for the eight computed profiles. The 50 to 2% exceedance events show a water surface elevation that is consistently below the West Glendive Levee crest. The 1% exceedance event overtops a portion of the levee, and the 0.5 and 0.2% exceedance events overtop the entire levee.

The Cottonwood Grove Levee, located on the right bank of the flood plain just upstream from the BNSF Railroad Bridge, provides protection against the river stages for all modeled profiles. Although the levee shows adequate height for protection of the Cottonwood Community, an indication of backwater flooding across the railroad tracks is apparent. An elevation dip along the railroad embankment is below the composite ice flood stage for the 0.5 and 0.2% exceedance events. Overbank, backwater flooding across the railroad could cause the Cottonwood Community to become inundated. However, flood damage to the Community would depend on the length of time the flood maintained this stage. In addition, terrain data used for the model and mapping consists of one meter contours. Therefore, terrain data may contain an error of ±1.6 feet. A detailed survey of the levee and adjacent road and railroad embankments may be beneficial to determine the extent of potential backwater flooding.

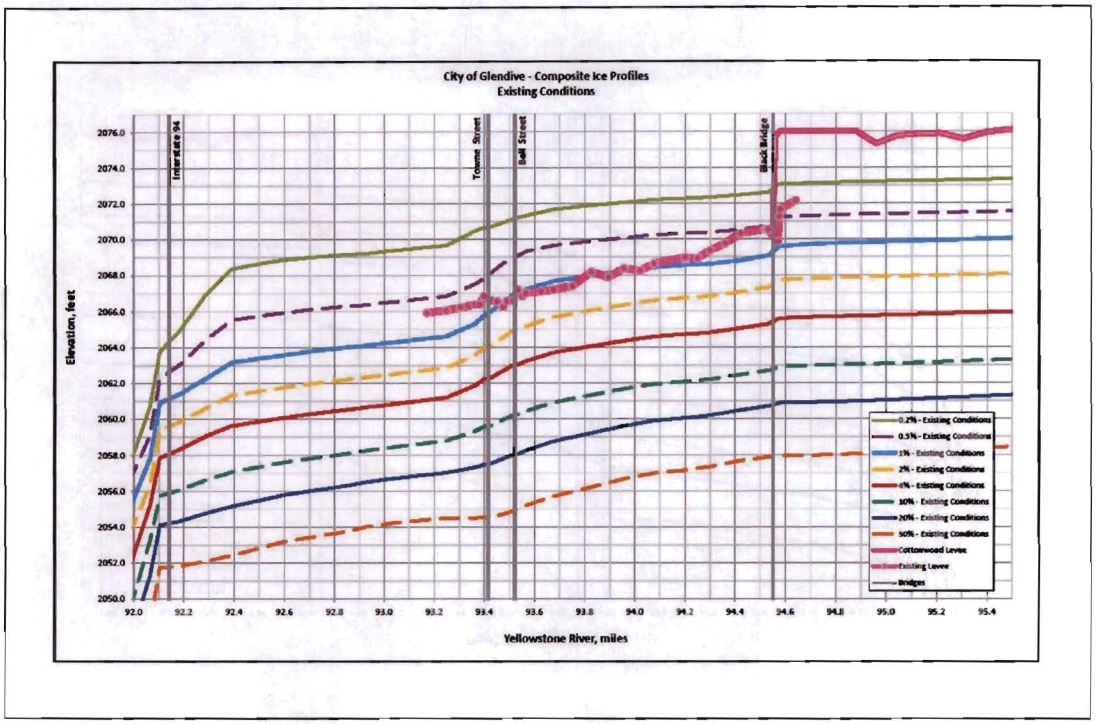

Figure 3. Water Surface Profiles for Existing Condition

The Flood Insurance Study (FIS) for the City of Glendive, published in 1980 by the Federal Emergency Management Agency, calculated an ice-impacted flood stage. The study data indicate that based on 70 years of record, the 1% chance exceedance event, open-water flood stage was exceeded approximately every 5 years due to ice-impacted flood events. It was determined that a combined stage (open water and ice affected) for the 1% chance exceedance event yielded an elevation of 2068.8 feet NAVD88 at the Bell Street Bridge. Upstream and downstream stages were calculated based on the Bell Street Bridge stage utilizing the Water Surface Profile Program developed by the Bureau of Reclamation.

A comparison of the current HEC-RAS composite ice profile and the 1980 FIS combine ice profile is shown in Figure 4. Results show that at Bell Street Bridge, the HEC-RAS composite ice stage is 1.9 feet less than the FIS stage. At most, the FIS stage exceeds the HEC-RAS composite ice by 2.5 feet. The FIS combined profile remains high downstream of the Interstate 94 Bridge, where the HEC-RAS jam was modeled, since a location of ice jam impacts was not specified in the FIS.

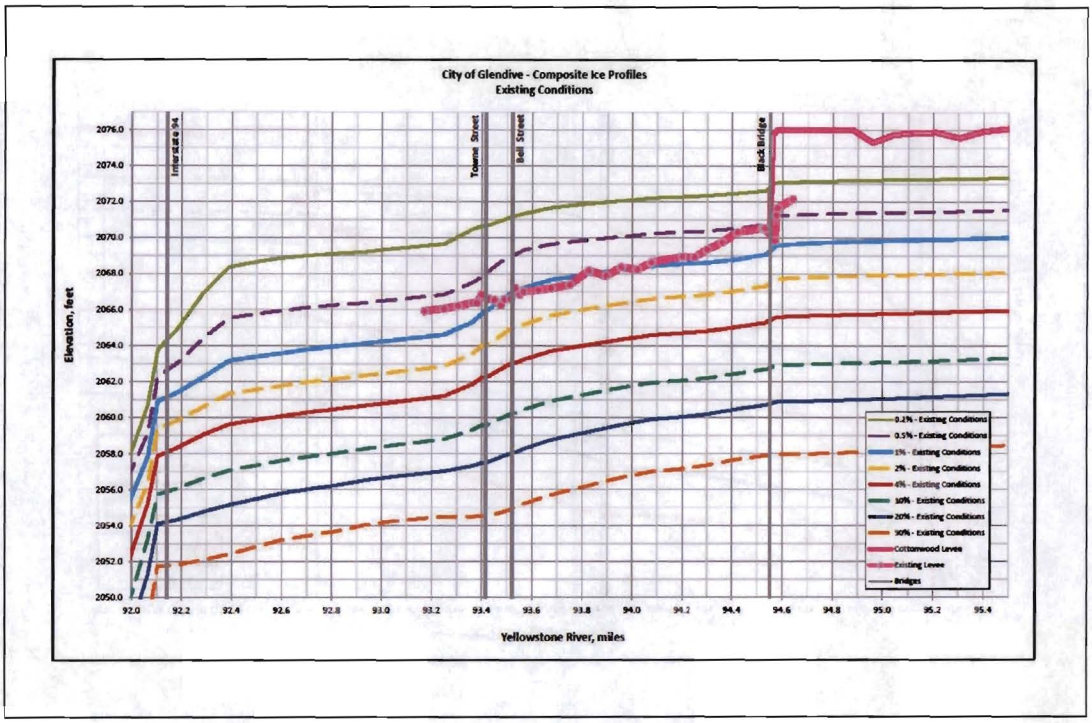

Figure 4. Water Surface Profiles for 1%ACE Composite Ice and FIS Report

The existing conditions hydraulic analysis concluded with the development of the existing conditions composite ice inundation map (Figure 5). Mapping of additional flow events can be found in Appendix B.

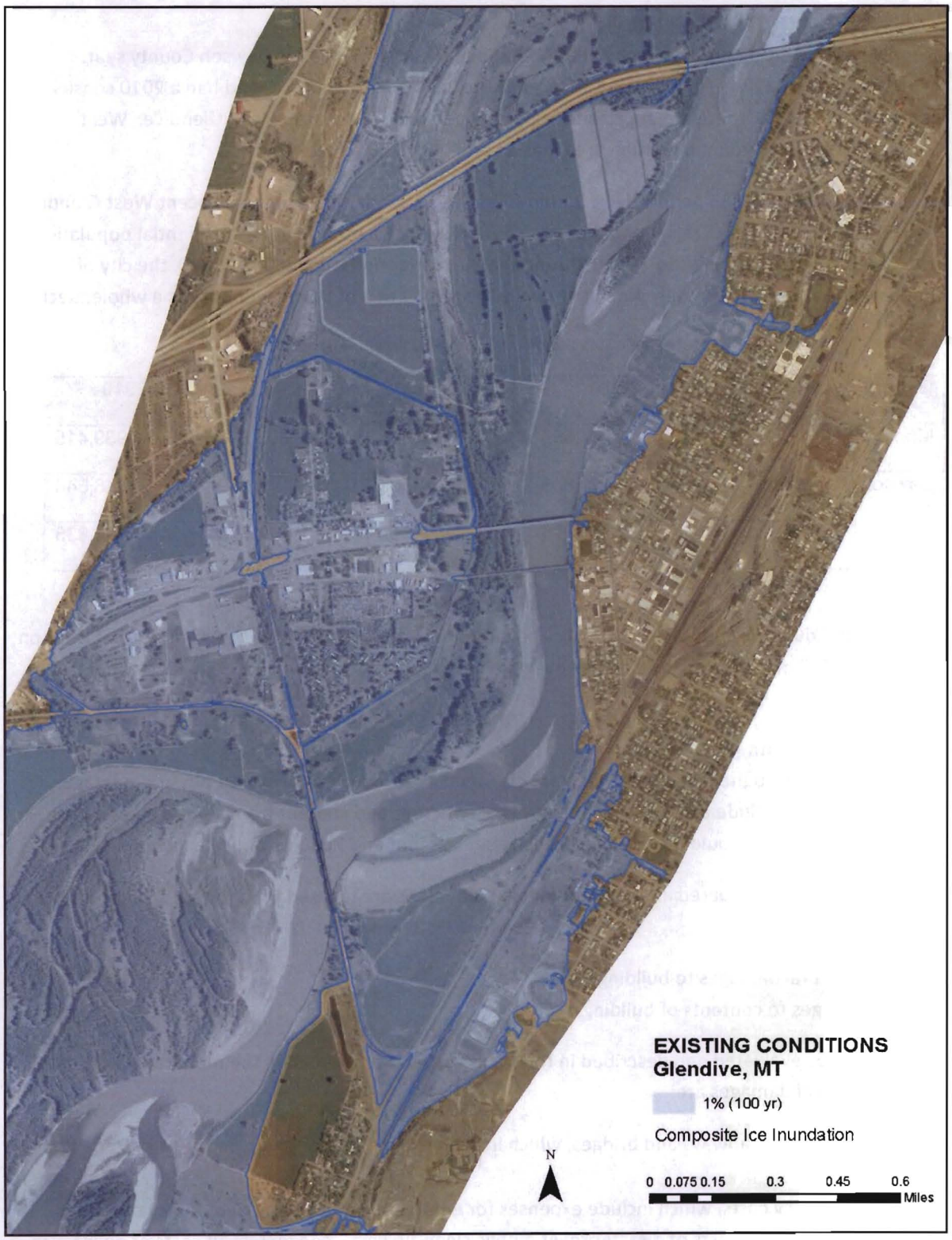

Figure 5. Inundation Map of 1% ACE (100 yr) Existing Conditions

2.3 Economic Analysis

Glendive is the only incorporated city in Dawson County and serves as the Dawson County seat. According to the City of Glendive website, Glendive covers 3.3 square miles and had a 2010 census population of 4,935. There are approximately 1,495 persons per square mile in Glendive. West Glendive, and the surrounding urban area, has an additional 1,948 people.

In 2010, there were 6,883 people living within Glendive's boundaries or in the adjacent West Glendive Census Designated Place. This is equal to over 75 percent of Dawson County's residential population (estimated at 8,640 residents in 2010). During the 10-year period from 2000 to 2010, the city of Glendive grew by approximately 4 percent, while the population of Dawson County as a whole, declined.

Table 2. Population by Geographic Area, 1980 to 2010

Geographic Area	1980	1990	2000	2010
Montana	787,690	799,065	902,195	989,415
Dawson County	11,805	9,505	9,059	8,640
Glendive	5,978	4,802	4,729	4,935

The Dawson County/Glendive Growth Policy document (draft December 2013), estimates that Dawson County population will gradually increase over the next few years reaching 8,777 in 2020 and 9,012 in 2025.

2.3.1 Major Damage Categories

Flood damages throughout the study area are classified as either physical or non-physical damages. Physical damages include damage to buildings, infrastructure and content from a flood event, while non-physical damages could include emergency management and flood response expenditures.

Damage categories evaluated and quantified for this analysis are focused primarily on structures located within the study area:

- structural damages to buildings;
- damages to contents of buildings;

Other damages evaluated and described in this analysis, but not included in the monetary value of expected annual damages are:

- Damage to roadways and bridges, which include expenses to clean up and repair after a flood event.
- Emergency costs, which include expenses for emergency evacuation, emergency measures, administrative costs of disaster relief, public clean-up costs, and increased costs of police, fire, and military patrol, along with temporary relocation costs.

- Traffic re-routing caused by flooding, which can temporarily impede traffic by inundating roads and bridges. Even the threat of flooding and concern for public safety may make it necessary to close roads, bridges, and detour traffic.

For the purpose of the economic analysis, the study area was based upon the estimated flood inundation boundaries of a 500-year flood event (.2% event), with this area divided into eight reaches. Five of these reaches were further subdivided into left and right bank reaches because of existing levees on either one or both sides of the river. One of the reaches (4L) was subdivided again to capture differences in levee heights and water surface elevations. The study area reaches are described in Table 3 and shown in Figure 6.

Table 3. Study Area Reaches

Reach Name	Reach Station	Reach Description	Index Station
1	95.5 to 97.8	downstream of City of Glendive and Cottonwood Levee	96.77
2L	94.64 to 95.5	left bank across from Cottonwood Levee, downstream of Black Bridge (railroad bridge)	95.04
2R	94.64 to 95.5	right bank of Cottonwood Levee downstream of Black Bridge	95.04
3L	94.55 to 94.64	left bank from downstream of Black Bridge to Black Bridge	94.58
3R	94.55 to 94.64	right bank upstream of Cottonwood Levee to Black Bridge	94.58
4LU	94.08 to 94.55	left bank from Black Bridge to upstream of Bell St Bridge	94.30
4LD	93.52 to 94.08	left bank from midway upstream of Bell St. Bridge to Bell St. Bridge	93.83
4R	93.52 to 94.55	right bank from Black Bridge to Bell St. Bridge	94.08
5L	93.42 to 93.52	left bank from Bell St. Bridge to Towne St. Bridge	93.47
5R	93.42 to 93.52	right bank from Bell St. Bridge to Towne St. Bridge	93.47
6L	93.15 to 93.42	left bank from Towne St. Bridge to end of W. Glendive Levee	93.36
6R	93.15 to 93.42	right bank from Towne St. Bridge to end of W. Glendive Levee	93.36
7	92.15 to 93.15	upstream of W. Glendive Levee to Interstate Bridge	92.59
8	90.76 to 92.15	Interstate Bridge to upstream of City of Glendive	91.61

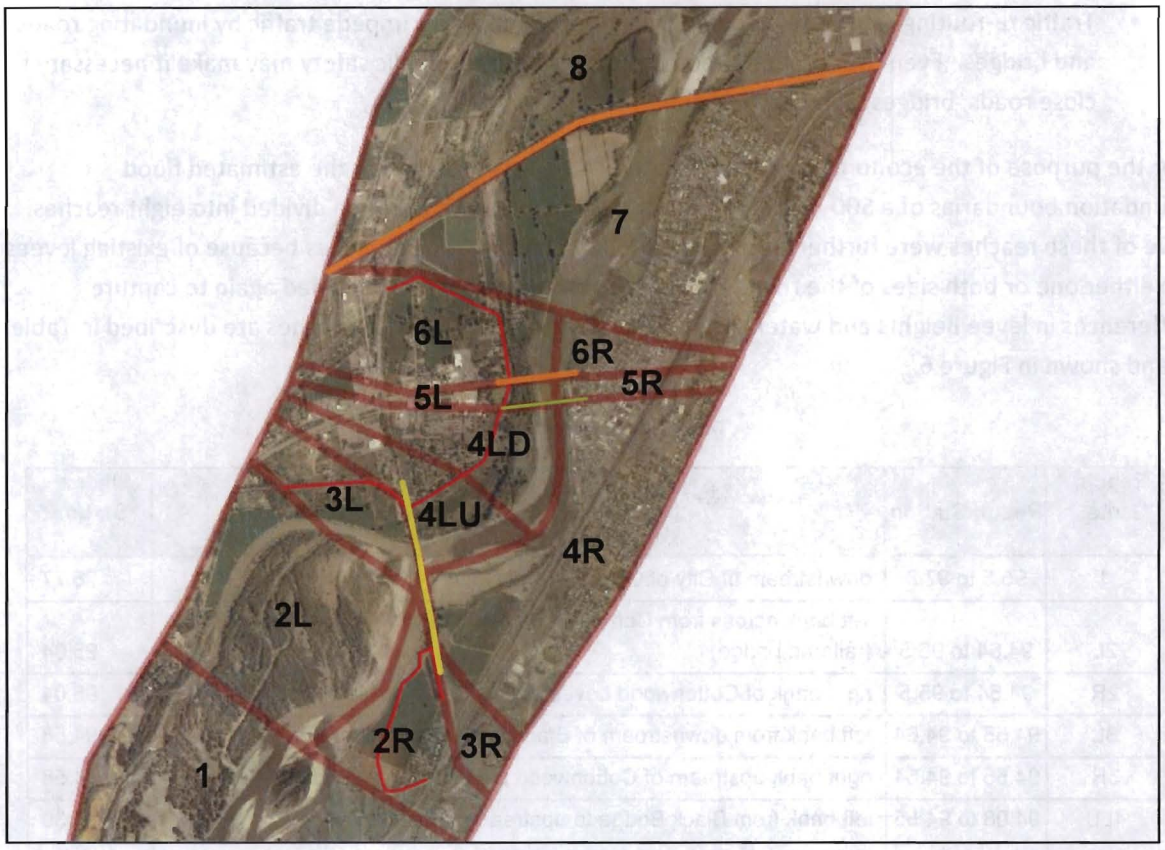

Figure 6. Study Area Reaches

2.3.2 Existing Conditions Flood Damages

The monetary value of potential flood damages are described in terms of expected annual damages given existing conditions. The following sections present the results of the analysis completed to quantify existing conditions expected annual damages and maximum affordable project.

Expected annual damages are based on fiscal year 2014 price levels, a FY14 discount rate of 3.5 percent, and a 50-year project life. The expected annual flood damages (EAD) under existing without project conditions are $1,265,230. This amount is based upon the value of likely damage to residential, commercial and public structures and content within the Glendive study area.

The EAD summary by reach and damage category is presented in Table 4. West Glendive would incur approximately 75 percent of the total damages (this includes reaches 3L, 4LU, 4LD, 5L and 6L). The highest expected annual damages would occur in reaches 4LU and 4LD, which are located between the Black Bridge and Bell Street Bridge and includes several commercial and industrial structures.

Table 4. Existing Conditions Expected Annual Damages (FY 2014$ Prices, $1,000)

Damage Reach Name	Commercial	Industrial	Public	Residential	Total Damage
Reach 8	$0.00	$0.00	$0.00	$0.00	$0.00
Reach 7	$174.74	$0.00	$2.99	$11.14	$188.87
Reach 6L	$254.25	$0.00	$0.00	$73.28	$327.54
Reach 6R	$1.65	$0.00	$5.40	$4.18	$11.23
Reach 5L	$91.09	$0.00	$0.00	$11.55	$102.64
Reach 5R	$0.90	$0.00	$0.03	$0.21	$1.14
Reach 4LD	$29.58	$64.29	$0.00	$166.00	$259.87
Reach 4LU	$189.17	$5.74	$0.00	$13.35	$208.25
Reach 4R	$0.87	$0.45	$0.07	$36.67	$38.06
Reach 3L	$31.04	$0.00	$0.00	$27.99	$59.03
Reach 3R	$0.00	$0.00	$0.00	$0.00	$0.00
Reach 2L	$0.01	$0.00	$0.00	$23.97	$23.98
Reach 2R	$0.11	$0.00	$0.00	$44.50	$44.61
Reach 1	$0.00	$0.00	$0.00	$0.00	$0.00
	$773.41	**$70.48**	**$8.50**	**$412.85**	**$1,265.23**

2.3.3 Maximum Affordable Project

Corps procedures calculate benefits based on the difference between the expected annual damages with and without alternative flood damage reduction plans. The implicit assumption incorporated into this procedure is that the reduction in flood damages is directly translatable into increased net income to floodplain land uses. Benefits from flood damage reduction alternatives focus on inundation reduction benefits that would result from reduced physical damages to structures and contents, and reduced non-physical losses.

To begin the evaluation of potential alternative flood damage reduction plans, the calculation of an array of maximum affordable projects has been completed. The term 'maximum affordable project is in itself a bit misleading in that the costs of potential project alternatives have not been calculated, however the value of maximum affordable project is solely based upon an estimate of the reduced physical damages to structures and content.

Using the FY14 Federal Discount Rate of 3.5% and assuming a project life of 50 years, the maximum flood risk reduction for a project that would reduce flooding beyond the 500 year event would provide a $1,265,230 annual benefit. The maximum affordable project based on this annual benefit is $29,700,000 (Table 5). Table 5 also displays a range of maximum affordable projects based upon lower targets of flood risk reduction – a 50-year level of flood risk reduction and a 100-year level of flood risk reduction.

Table 5. Maximum Affordable Project by Reach (3.5% Discount Rate)

Reach Name	50 Year Level of Flood Risk Reduction	100 Year Level of Flood Risk Reduction	500 Year Plus Level of Risk Reduction
Reach 8	$0	$0	$0
Reach 7	$4,000,000	$4,100,000	$4,500,000
Reach 6L	$0	$4,400,000	$7,700,000
Reach 6R	$0	$0	$300,000
Reach 5L	$0	$1,400,000	$2,400,000
Reach 5R	$0	$0	$0
Reach 4LD	$0	$3,200,000	$6,100,000
Reach 4LU	$0	$3,000,000	$4,900,000
Reach 4R	$0	$200,000	$900,000
Reach 3L	$0	$800,000	$1,400,000
Reach 3R	$0	$0	$0
Reach 2L	$300,000	$400,000	$600,000
Reach 2R	$0	$0	$1,000,000
Reach 1	$0	$0	$0
Total	**$4,400,000**	**$17,500,000**	**$29,700,000**

3 Plan Formulation – Alternative Development

The PAS study does not include a comprehensive plan formulation exercise and does not identify all potential measures and screen them into alternatives. Instead, the Corps met with the City and County and prioritized a few potential alternatives that would help give the sponsor an idea of available options and conceptual costs of alternatives. It was agreed that the Corps would look at four potential alternatives, including: 1) flood warning system; 2) providing increased conveyance at the I-94 bridge; 3)

setting back the existing levee and providing more conveyance by extending the railroad bridge; and 4) raising the existing levee. Conceptual costs were developed for Alternatives 3 and 4. There are likely other feasible alternatives that could be considered should the City/County and Corps pursue a feasibility study.

3.1 Alternative 1 – Flood Warning System

A flood warning system consisting of five gage locations was developed to provide the City of Glendive suitable warning times for evacuation due to an ice jam flood event. The five recommended locations (Figure 7 and Figure 8**Error! Reference source not found.**) would be equipped with automated ultrasonic transducers attached to bridges crossing over the Yellowstone River. The two existing USGS gage locations and three new sites are:

1) USGS 06309000, Yellowstone R. at Miles City. This is an existing USGS gage location approximately 80 miles upstream of Glendive. This site uses the satellite based Data Collection Platform (DCP) to transmit data. At a minimum, WaterAlert subscriptions should be set for designated personnel with the National Weather Service (NWS) and Glendive emergency managers.
2) Fallon, MT I-94 Bridge. Fallon is 31 river miles upstream of Glendive, approximately halfway between Miles City and Glendive. This gage would provide early warning to Glendive emergency managers in the event a large jam forms or releases downstream of the Miles City gage. It would also be beneficial in determining ice jam flow travel times between Miles City and Glendive.
3) Railroad Bridge upstream of Bell St. Bridge, Glendive, MT. This bridge is about one mile upstream of the Bell St. Bridge and is located downstream of an island at a wide section of the river channel known to experience ice jam events. An automated gage at this location would help provide warnings for flood events similar to the 1978 and 2003 floods wherein stages increased at rates up to 1.1 ft every 10 minutes.
4) USGS 06327500, Yellowstone R. at Glendive. This is the existing USGS DCP gage location and the Glendive water treatment plant gage location on the Bell St. Bridge. At a minimum, WaterAlert subscriptions for the NWS action and flood stages (51.5ft and 53.5 ft) should be activated for the Glendive emergency managers and city officials.
5) Glendive I-94 Bridge. This location is downstream of the Bell St. Bridge and also prone to ice jam floods that can cause backwater at Bell St. A gage at this location was recommended in the 2001 Flood Warning System Evaluation report.

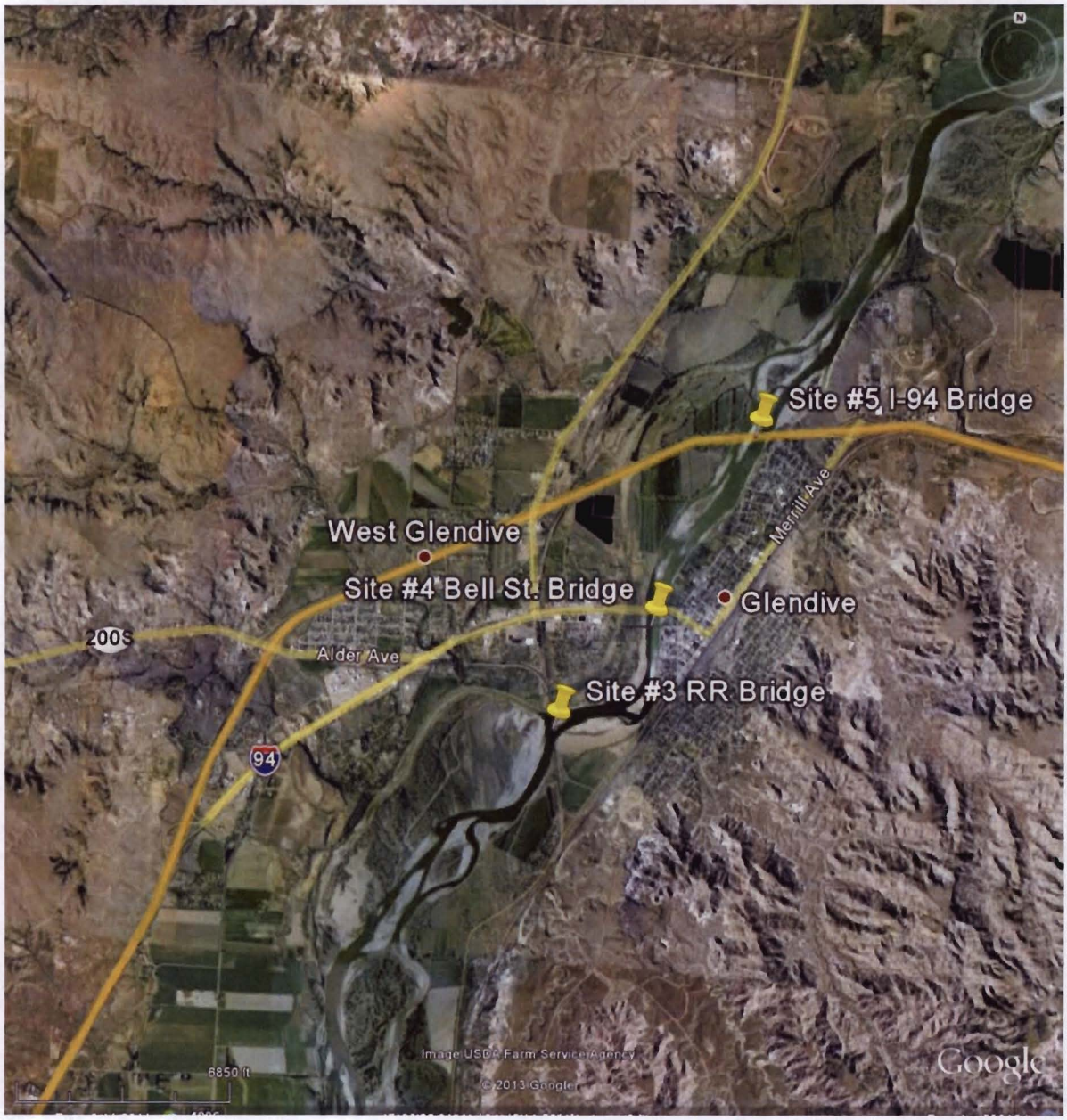

Figure 7. Proposed Flood Warning System Gage Locations in Glendive, MT

Figure 8. Location Map of Flood Warning Gages for Glendive, MT

Further information on the flood warning system can be found in Appendix A.

3.2 Alternative 2 - Interstate 94

Ice jams have historically occurred immediately downstream of the Interstate 94 Bridge. The impact of the Bridge on the system was evaluated by modeling variations to the structure that increased conveyance and then comparing the results. The alternative solutions for increasing conveyance at the crossing include incorporating an additional opening on the left overbank, where the Interstate 94

embankment cut off a historic side channel, at a width of 100ft and 500ft and complete removal of the Interstate approach road embankment.

The openings and Bridge were modeled as multiple openings in the bridge editor, see Figure 9. The modeled side channel followed the natural terrain where the historic side channel existed prior to the construction of the Interstate 94 and Bridge. The channel is approximately 10,000 feet in length with an invert approximately 10 feet higher than the main channel invert. An opening of either 100 feet or 500 feet was created in the embankment to serve as a relief outlet, allowing additional drainage of upstream backwater. The location of the modeled bridge opening coincides to the elevation sag in the left embankment where the previous side channel existed. A third alternative to address the restricted conveyance at the interstate bridge evaluated the complete removal of the bridge, bridge piers and approach road embankment.

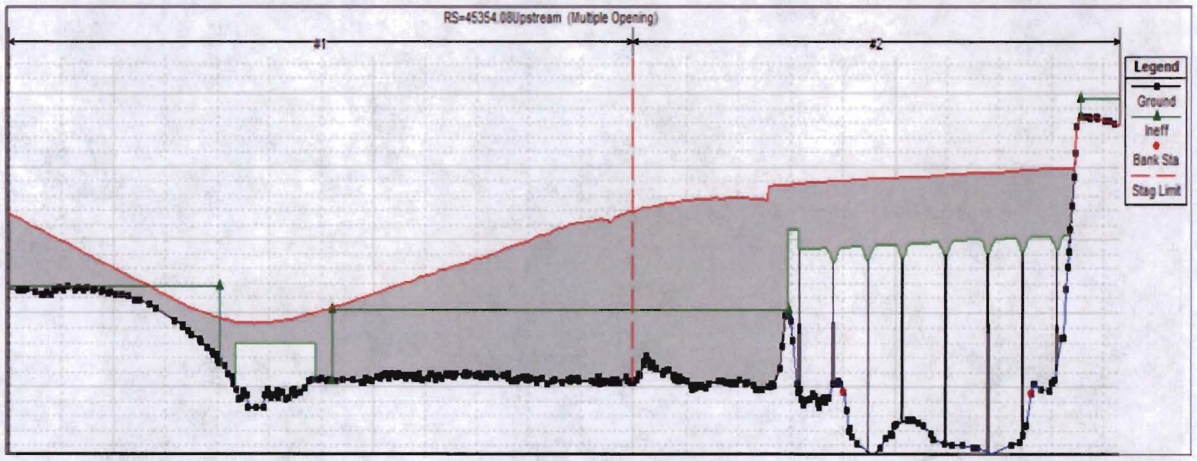

Figure 9. Cross-Section of Interstate 94 with 500-ft Opening

The resulting water surface profiles for the three options for the 1% and 0.2% ACE are shown in Figure 10 and Figure 11. Further information can be found in Appendix B. The increased conveyance yielded a lower water surface between the Interstate 94 Bridge and Towne Street Bridge. However, upstream of Towne Street, the water surface elevation and ice jam thickness remained largely unchanged. In addition to the lower water surface between I-94 and Towne Street, the model also computed a change in the ice jam shape. As the conveyance was increased and the water stage lowered, the ice jam toe became longer.

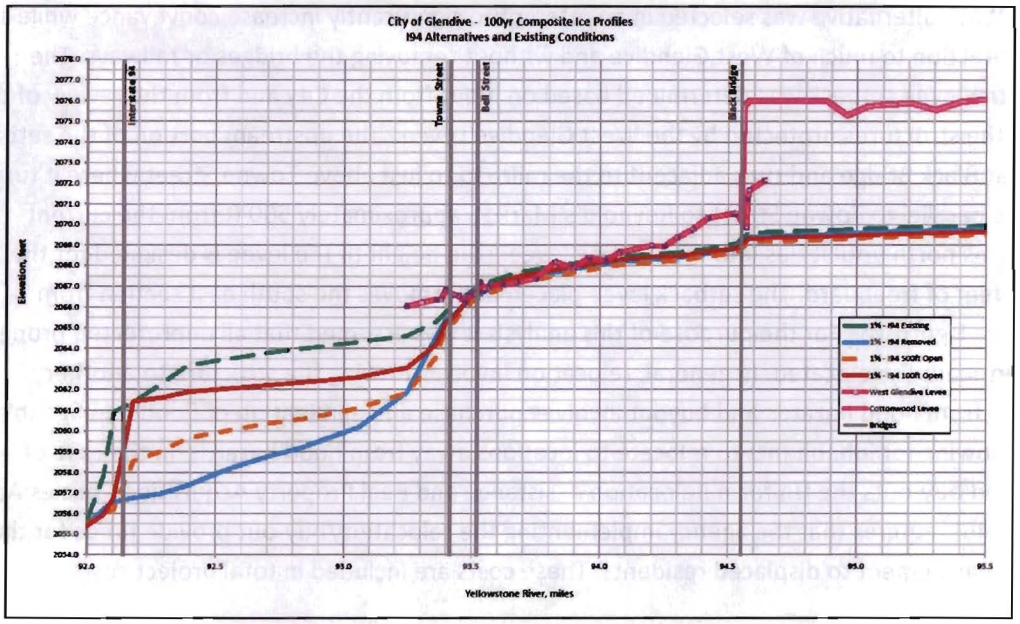

Figure 10. 1% ACE Water Surface Profiles for the Interstate 94 Options

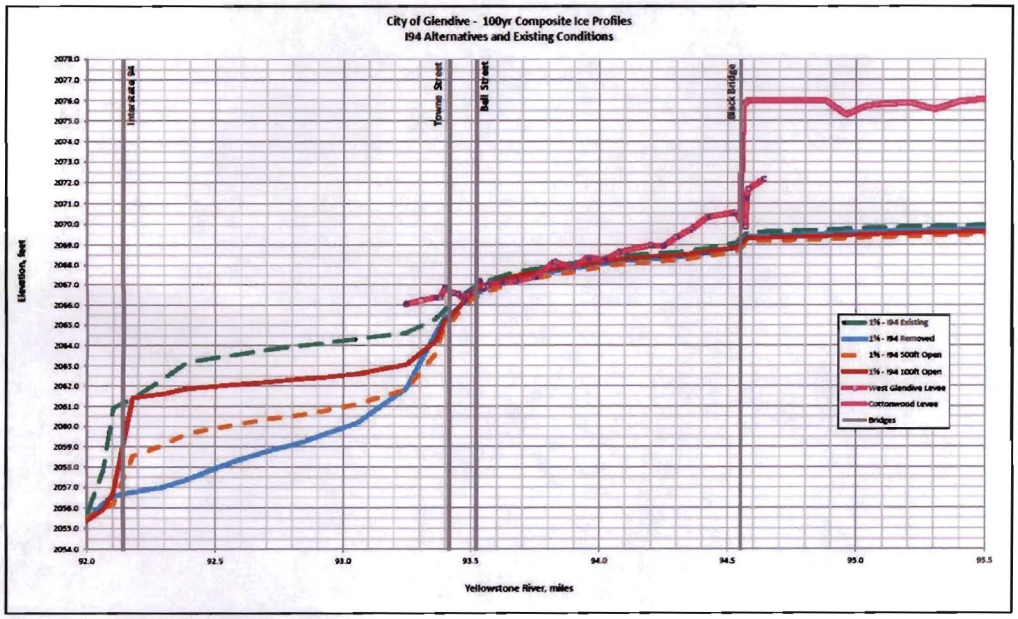

Figure 11. 0.2% ACE Water Surface Profiles for the Interstate 94 Options

3.3 Alternative 3 - Levee Setback and Culverts

The levee setback alternative was selected in an attempt to significantly increase conveyance while still providing protection to much of West Glendive and without removing the bridges or railways. The footprint of the levee setback was determined based on input from the City and from the review of the valuation of the structures protected by the West Glendive Levee. The upstream portion of the setback levee starts at Black Bridge and runs adjacent to the railroad to just above Towne Street where it turns east and runs parallel to Towne Street to Reynold's Market, approximately 500 ft from the current levee, and turns north until it ties into the existing levee. The height of the levee is designed for the 1% event plus 3 feet of freeboard. The setback levee placement removes the south east section from protection, see Figure 12. For the purpose of this analysis it was assumed that all unprotected property would be permanently relocated. In general, relocation involves moving the structure to another location away from flood hazards and buyout involves purchase and elimination of flood damageable structures, allowing for inhabitants to relocate to locations away from flood hazards. In the case of relocations and buy-outs the Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970 (PL 91-646) requires that the agency implementing the relocation/buy-out provide funds for the moving and resettlement to displaced residents. These costs are included in total project costs.

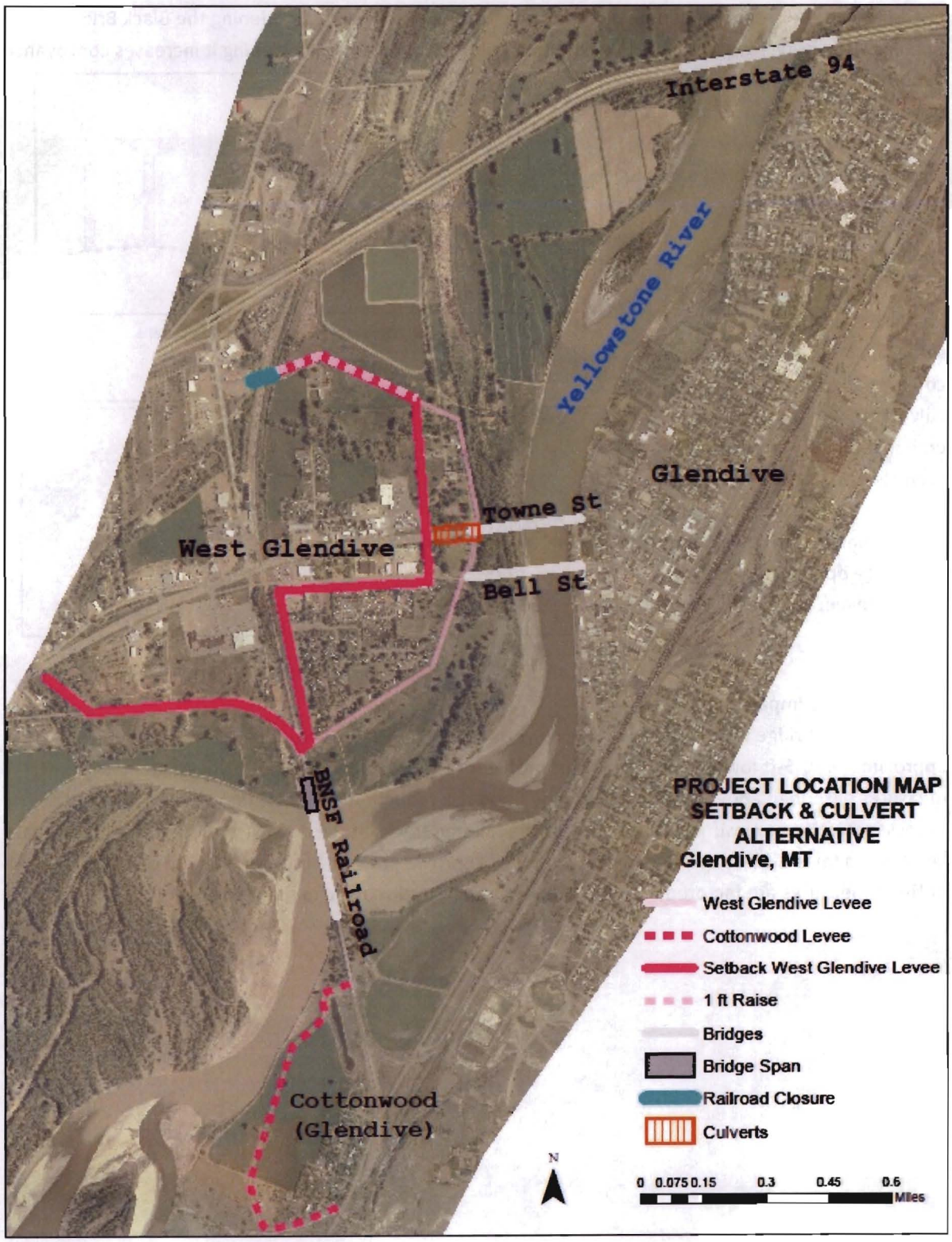

Figure 12. Location Map of the Setback Levee Alternative

In addition to the setback, additional conveyance would be achieved by widening the Black Bridge opening. The bridge is a significant constriction point for the river and widening it increases conveyance in both the upstream and downstream direction. Therefore, two 150-foot spans were incorporated into the bridge on the left bank (Figure 13).

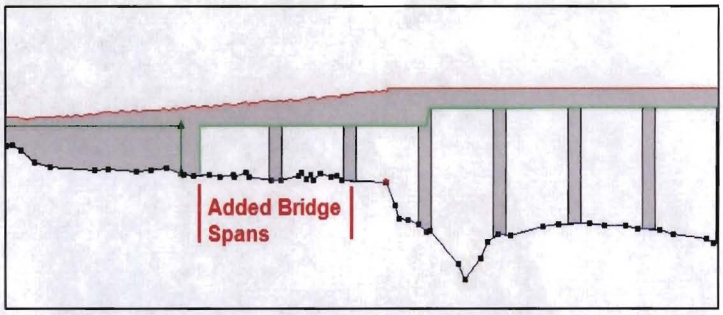

Figure 14. Black Bridge with Added Bridge Spans

Similarly, Towne Street Bridge is a constriction point for the river. To increase the benefit of the levee setback, additional conveyance could be achieved by placing culverts at the bridge embankment. Eighteen large culverts, sized 24 feet wide by 12 feet high were incorporated into the left embankment, between the bridge opening and the setback levee, shown in Figure 14.

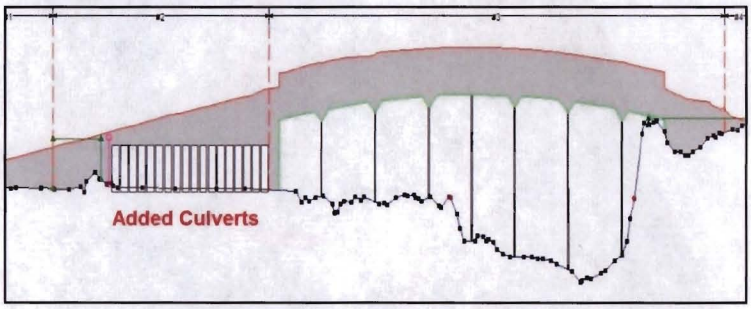

Figure 13. Towne Street Bridget with Added Culverts

The cumulative impacts of the setback levee, bridge widening and bridge culverts resulted in a water surface elevation that is approximately 1.5-ft lower than the current profile for the 1% flood event. However, along with the measures described above in order to provide flood risk reduction at the 1% flood event, it would be necessary to place a closure structure across the railroad tracks at the downstream tie-off of the setback levee and a raise of the downstream levee crest of approximately one foot for a length of 2,000 feet. All of these measures are factored in to the economic analysis discussed in the next section.

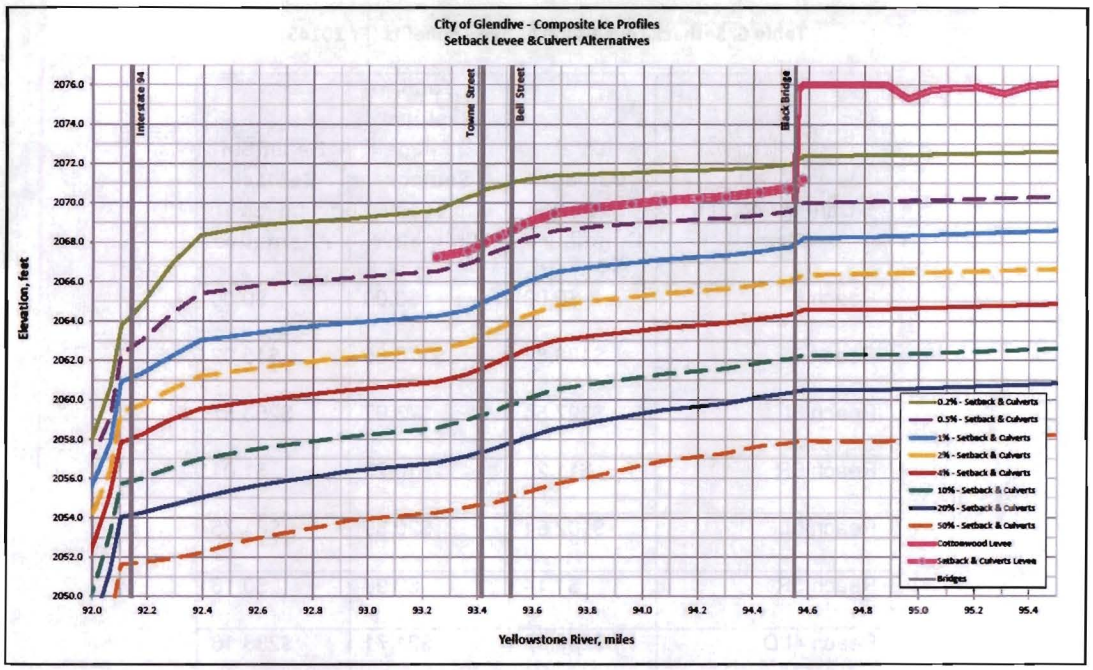

Figure 15. Water Surface Profiles of the Setback Levee Alternative

3.3.1 Economic Benefits and Costs

Corps procedure calculates benefits based on the difference between the expected annual damages with and without alternative flood damage reduction plans, therefore the flood risk reduction benefits that could be achieved if the proposed setback levee alternative were constructed are compared to the flood risk conditions under existing conditions. Benefits from flood damage reduction alternatives are inundation reduction benefits from reduced physical damages to structures and contents. As shown in Table 6, benefits from reduced expected annual damages would occur on both the left and right banks of the Yellowstone River, with the majority of the benefits occurring on the left bank of the river (Table 6). The benefit of the proposed setback levee alternative are estimated at almost $838,000 annually, in reduced structure and content damages.

Table 6. Setback Levee Alternative - Benefits (FY 2014$)

Damage Reach Name	Total Damage Existing Conditions	Total Residual Damage Setback Levee Alternative	Benefit of Setback Levee Alternative
Reach 8	$0.00	$0.00	$0.00
Reach 7	$188.87	$173.86	$15.02
Reach 6L	$327.54	$73.97	$253.57
Reach 6R	$11.23	$10.23	$1.01
Reach 5L	$102.64	$20.89	$81.75
Reach 5R	$1.14	$0.98	$0.16
Reach 4LD	$259.87	$21.71	$238.16
Reach 4LU	$208.25	$38.75	$169.50
Reach 4R	$38.06	$25.70	$12.36
Reach 3L	$59.03	$8.99	$50.04
Reach 3R	$0.00	$0.00	$0.00
Reach 2L	$23.98	$18.71	$5.27
Reach 2R	$44.61	$33.45	$11.16
Reach 1	$0.00	$0.00	$0.00
	$1,265.23	$427.23	$837.99

As part of the setback levee alternative several properties on the river side of a newly constructed set-back levee would need to be relocated. Structure removal would contribute to the reduction in property damage, along with reduced levee fragility due to new levee construction, and lower water surface profiles provided by the increased water conveyance and decreased constriction points associated with a setback levee. Opportunities for ecosystem restoration and recreation could be incorporated into this alternative if a feasibility level analysis were pursued.

Table 7 provides a draft cost estimate for the setback levee alternative by item. The proposed setback levee alternative includes levee construction costs, railroad bridge extension, closure structure, and real estate costs. Based upon the proposed setback levee alignment approximately seven residential

parcels, seven commercial parcels and two trailer parks would be removed. Total project costs are estimated at $20,600,000. Total project costs are annualized or a 50-year period using the FY14 Federal discount rate of 3. 5%. Annual operation & maintenance (O&M) costs are estimated at $6,000/year. Total annual cost including O&M is $864,562.

Table 7. Setback Levee Alternative Costs (in FY14$)

Category	Cost
Levee Setback	$5,500,000
RR Bridge Extension	$5,500,000
Closure Structure	$300,000
Real Estate	$9,300,000
Total First Costs	**$20,600,000**
Total Annualized Costs (50 years, 3.375%)	$878,000
Annual O&M Costs	$6,000
Total Annual Cost	**$884,000**

3.3.2 Summary

Economic benefits for Alternative 3 are estimated at about $838,000 annually (Table 6). The total first costs, or initial investment costs for the levee setback alternative is $22,600,000 (Table 7). Total average annual costs are approximately $884,000 including $6,000 annual operation and maintenance costs. Comparing annual benefits and costs, the proposed setback levee alternative has a net annual benefit of approximately -$46,000 and an annual benefit cost ratio of .95. Note, that the full array of benefits and costs has not been completed for this analysis. For example benefits such as reduction in emergency management and response costs and reduced traffic rerouting costs are not included in the benefit estimate. Likewise interest during construction costs have not been included in the analysis due and other project costs could be identified as alternative plans are refined.

3.4 Alternative 4 - Levee Raise

Modeling of the existing levee yielded water surface elevations that exceeded the existing West Glendive levee for the 1-, 0.5- and 0.2% ACE, therefore a raise of the existing levee was considered. The computed 1% ACE composite ice profile was selected and three feet of freeboard was applied to determine the raised levee height. Results of the modeling indicate the levee would need to be raised

0-4 feet with an average of 2 feet depending on the location. The resulting profiles for eight frequency events are shown below in Figure 16, and inundation maps are located in Appendix B.

The levee raise provides protection for the 1% ACE with three feet of freeboard and the 0.5% ACE with approximately 0.5 feet of freeboard. The 0.2% ACE overtops the raised levee by as much as two feet. The levee raise would require a closure structure across the railroad tracks near the downstream tie off. The bridge embankments provide adequate height for the raised levee such that closure structures on the roadways will not be necessary.

The levee raise alternative would raise the West Glendive levee, which is on the left bank of the Yellowstone River, and increase protection or reduce damages to structures also located on the left bank. The levee raise will result in increases in the water surface profiles of the Yellowstone River. Approximately 97 structures located along the right-bank and just upstream of the federal levee along the left-bank of the Yellowstone River would experience a slight increase in damages from a West Glendive Levee Raise (above existing condition damages), due to the increased water surface profile associated with a levee raise. These increased damages are termed induced damages and would require non-structural mitigation to maintain their existing level of flood risk reduction.

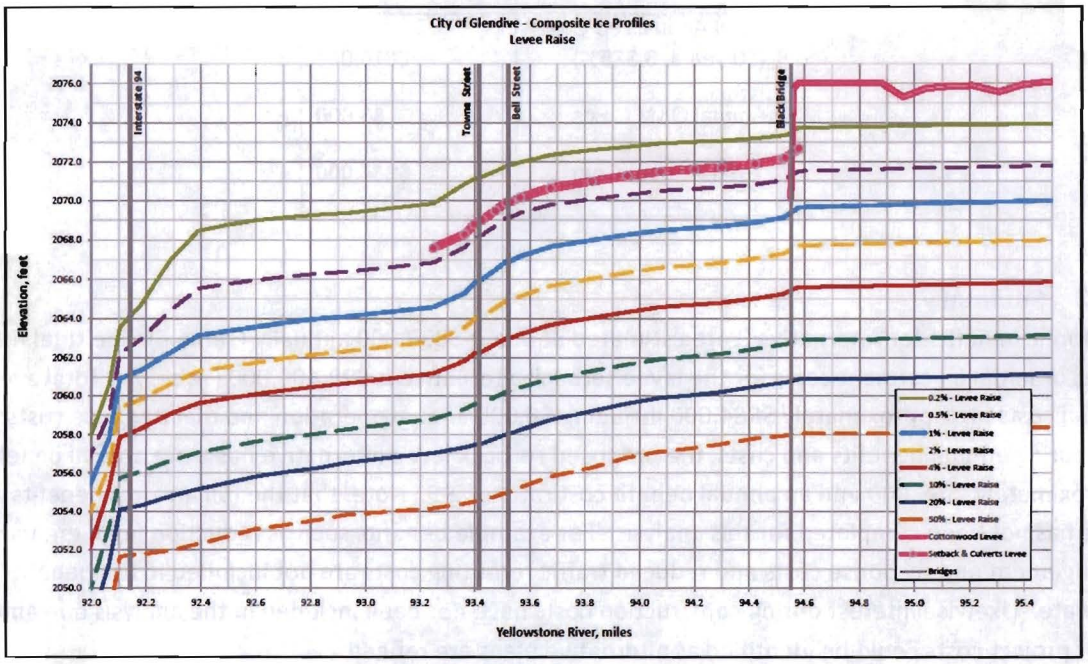

Figure 16. Water Surface Profiles of the Levee Raise Alternative

Table 2 indicates the structures located along either the left-bank or right-bank and those nonstructural flood risk adaptive measures considered for nonstructural mitigation (see Appendix D for a detailed description of each technique).

Table 8. Nonstructural Measures Identified by Streambank Location

Technique	Left-Bank Structures	Right-Bank Structures
No Action Required	2	23
Acquisition	2	4
Dry Flood Proofing	0	3
Elevation	0	8
Remove Basement	4	63
Elevate/Remove Basement	2	11
Total Structures	10	112

3.4.1 Levee Raise Benefits and Costs

As discussed above, Corps procedures calculate benefits based on the difference between the expected annual damages with and without alternative flood damage reduction plans. Table 9 shows the results of the expected annual damage analysis for the levee raise alternative by study reach. The benefit of the levee raise alternative is estimated at over $525,000 annually in reduced structure and content damages.

Table 9. Levee Raise Alternative - Benefits (FY2014$)

Damage Reach Name	Total Damage Existing Conditions	Total Residual Damage Levee Raise Alternative	Benefit of Levee Raise Alternative
Reach 8	$0.00	$0.00	$0.00
Reach 7	$188.87	$190.24	($1.36)
Reach 6L	$327.54	$143.52	$184.02
Reach 6R	$11.23	$12.07	($0.83)
Reach 5L	$102.64	$46.02	$56.62
Reach 5R	$1.14	$1.73	($0.59)
Reach 4LD	$259.87	$114.38	$145.49
Reach 4LU	$208.25	$100.06	$108.19
Reach 4R	$38.06	$41.87	($3.81)
Reach 3L	$59.03	$19.39	$39.64
Reach 3R	$0.00	$0.00	$0.00
Reach 2L	$23.98	$24.75	($0.77)
Reach 2R	$44.61	$45.51	($0.90)
Reach 1	$0.00	$0.00	$0.00
	$1,265.23	$739.52	$525.71

Table 10. Levee Raise Alternative Costs

	Cost
Levee Raise	$2,000,000
Real Estate	$200,000
NonStructural Mitigation	$4,400,000
Total First Costs	$6,600,000
Total Annualized Costs (50 years, 3.5%)	$275,000
Annual O&M Costs	$6,000
Total Annual Cost	**$281,000**

3.4.2 Summary

Economic benefits for Alternative 4 are estimated at about $526,000 annually (Table 9). The total first costs, or initial investment costs for the levee raise alternative is $6,600,000 (Table 10). Total average annual costs are $281,000 including $6,000 annual O&M costs. Comparing annual benefits and costs, the proposed levee raise alternative has a net annual benefit of approximately $245,000 and an annual benefit cost ratio of 1.87. Note, that the full array of benefits and costs has not been completed for this analysis. For example likely benefits such as reduction in emergency management and response costs and reduced traffic rerouting costs are not included in the benefit estimate. Likewise interest during construction costs have not been included in the analysis and other project costs could be identified as alternative plans are refined.

US Army Corps
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Omaha District

Yellowstone River at Glendive, MT

Planning Assistance to States Report

DRAFT ECONOMICS APPENDIX

May 2014

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1. Introduction

1.1 Purpose and Scope

The purpose of this appendix is to present the results and economic analysis completed to evaluate flood risk conditions for the City of Glendive from the Yellowstone River. The economic analysis was completed as part of a Planning Assistance to States (PAS) Study on flood risk reduction opportunity for Glendive, MT. It provides an evaluation of existing conditions, as well as an evaluation of a preliminary flood risk reduction alternative.

Since this analysis was completed as part of the PAS Study, the purpose is not to seek construction authorization for the proposed alternative, but rather it is to provide planning assistance to the local government of Glendive as they identify their best path forward in relation to flood risk reduction and to help them assess their existing flood risk conditions as well as potential opportunities for flood risk reduction in the future. In addition to this PAS Study, several other studies have been completed on flood risk reduction in Glendive. The most recent study by the US Corps of Engineers (USACE) was a Section 205 Report followed by a Flood Plain Management Plan that was published in March 2002.

1.2 References

The economic analysis described in this appendix is consistent with Federal water resources policies and practices, including *Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies* (P&G, 1983), the Corps *Planning Guidance Notebook* (ER-1105-2-100, 22 April 2000), Planning, Risk Analysis For Flood Damage Reduction Studies, dated 3 January 2006; and Risk-Based Analysis for Evaluation of Hydrology/Hydraulics and Economics in Flood Damage Reduction Studies (EC 1105-2-205), dated 25 Feb 1994.

1.3 Water Resource Problem

The Yellowstone River is prone to ice jam flooding, typically between February and April when temperature swings are most extreme. A pattern of freezing and thawing allow ice to form, break apart, float downstream and potentially jam at constriction points in the river. The ice jam

can result in abrupt upstream flooding. Properties along the Yellowstone River, especially in West Glendive have been identified as susceptible to flooding form the Yellowstone River. These are predominantly commercial and residential developments, including a large trailer home development that is directly behind the West Glendive Levee.

2. Background

2.1 Study Area Location

The City of Glendive is located in Dawson County in Eastern Montana along the Yellowstone River approximately 90 miles upstream from the river's confluence with the Missouri River. The Yellowstone River flows from southwest to northeast through the city dividing it into Glendive proper and West Glendive.

The Yellowstone River is a typical meandering stream as is associated with mature rivers. The Glendive reach has seen much modification due to human development beginning in the late 19th century. In 1902, the Northern Pacific Railroad (currently Burlington Northern–Santa Fe Railroad -- BNSFRR) constructed a nine span iron truss bridge over the Yellowstone River at the upstream side of the city. This bridge is commonly referred to as the "Black Bridge." The elevation of the Black Bridge resulted in the approaches on the left bank requiring embankments as much as 10 feet above the floodplain. These large embankments effectively prevented Yellowstone River flood flows from getting into a large portion of the left bank flood plain. Additionally, the naturally meandering stream was contained to the bridge opening. Two different Highway 10 bridges (one of which is now a pedestrian bridge) were later built in the central part of the city to connect West Glendive and Glendive. These bridges, in combination with the West Glendive Levee which was constructed in 1959 by the Corps of Engineers, further helped to contain the river to its current alignment and prevent meandering.

During the late 1960's, the Interstate 94 Bridge was constructed just downstream from the city. This large bridge was constructed at a sufficient height to tie into the right bank bluff line which required the construction of a long, elevated road embankment across the left bank floodplain. This embankment varies in height from approximately 10- to 20- feet above the floodplain and acts as a significant obstruction to flood flows forcing nearly all of the flows to go through the bridge opening.

2.2 Socioeconomic Conditions

Glendive is the only incorporated city in Dawson County (there is one unincorporated town as well), and serves as Dawson County seat. According to the Glendive website, Glendive covers 3.3 square miles and had a 2010 census population of 4,935. There are approximately 1,495 persons per square mile in Glendive. West Glendive, and the surrounding urban area, has an additional 1,948 people.

In 2010, there were 6,883 people living within Glendive's boundaries or in the adjacent West Glendive Census Designated Place. This is equal to over 75 percent of Dawson County's

residential population (estimated at 8,640 residents in 2010). During the 10-year period from 2000 to 2010, the city of Glendive grew by approximately 4 percent, while the population of Dawson County as a whole, declined.

Table 1. Population by Geographic Area, 1980 to 2010

Geographic Area	1980	1990	2000	2010
Montana	787,690	799,065	902,195	989,415
Dawson County	11,805	9,505	9,059	8,640
Glendive	5,978	4,802	4,729	4,935

The Dawson County/Glendive Growth Policy document (draft December 2013), estimates that Dawson County population will gradually increase over the next few years reaching 8,777 in 2020 and 9,012 in 2025.

3. Economic Analysis

The economic analysis completed to evaluate existing conditions and the preliminary action alternatives are presented in this section. The following section describes the methodology utilized for evaluating flood damages under existing conditions and the potential benefits that could be captured with an action alternative.

3.1 Methodology

Flood damages are expressed in terms of expected annual damages, which are defined as the monetary value of physical damages and non-physical losses that can occur in any given year based on the magnitude and probability of losses from all possible events. For the Glendive PAS study, the primary focus of the analysis is the physical damages to structures and content. Other damages categories such as public infrastructure (utilities, roads, etc.) are described and quantified but not included in the average annual damage calculation.

The basis for determining damages is an examination of losses sustained in historical floods along with an evaluation of likely annual damages utilizing the Hydrologic Engineering Center Flood Damage Analysis (HEC-FDA) version 1.2.5a tool. The HEC-FDA model is supplied with detailed hydrologic and hydraulic engineering and flood damage data in order to establish a relationship between flooding conditions and damages.

3.1.1 Risk and Uncertainty

The HEC-FDA model is used to estimate the economic value of physical damages to structures and content while considering the risk and uncertainty inherent to estimating flooding conditions and economic values. The driving assumption underlying this software program is that the inputs of the program contain uncertainty and imperfect knowledge. Using the most likely values for input parameters, HEC-FDA uses specified levels of uncertainty over tens of thousands or even hundreds of thousands of model iterations to incorporate the uncertainty in the values used in the program. Throughout these iterations, the HEC-FDA program determines whether a flood event

resulted in damage and how much damage would have occurred. The result is a computed expected annual damage (EAD) with incorporated risk and uncertainty. Uncertainty parameters used in the HEC-FDA study for this analysis include:

- first floor elevations
- structure values
- content to structure ratios
- appurtenant uses to structure ratios
- percent depth-damage functions;
- discharge-exceedance functions;
- stage-discharge functions; and
- levee fragility curves.

Uncertainty parameters such as content to structure ratios or depth damage functions are assigned a probability distribution (based on prior analysis), that is included HEC-FDA model. Other uncertainty parameters, such as levee fragility curves which provide an estimate of the likelihood of geotechnical levee failure at various water surface elevations. For example, 1 foot below the top of the levee may be expected to have a 5 or 10 percent chance of failure based upon historical information or physical conditions

Damages are initially expressed as a stage-damage relationship (i.e., each foot of potential flooding at an index point is associated with an estimated amount of "primary damage"). However, the ultimate goal is expression of damages in an average annual form. The calculated total for existing conditions or without project flood damages is an estimate of the average annual damages that could be expected in any given year for an extended period of time if no flood risk reduction project is constructed. To estimate the potential with-project alternative benefits, the annualized damage of the without-project condition is compared to the annual damage of the with-project alternative. This average annual benefit is then compared with the annualized cost for each alternative to obtain the benefit-cost ratio (BCR) and net annual benefits (the difference between annual benefits and annual costs) for each alternative.

3.1.2 Major Damage Categories

Flood damages throughout the study area are classified as either physical or non-physical damages. Physical damages include damage to buildings, infrastructure and content from a flood event, while non-physical damages could include emergency management and flood response expenditures.

Damage categories evaluated and quantified for this analysis are focused primarily on structures located within the study area:

- structural damages to buildings;
- damages to contents of buildings;

Other damages evaluated and described in this analysis, but not included in the monetary value of expected annual damages are:

- Damage to roadways and bridges, which include expenses to clean up and repair after a flood event.
- Emergency costs, which include expenses for emergency evacuation, emergency measures, administrative costs of disaster relief, public clean-up costs, and increased costs of police, fire, and military patrol, along with temporary relocation costs.
- Traffic re-routing caused by flooding, which can temporarily impede traffic by inundating roads and bridges. Even the threat of flooding and concern for public safety may make it necessary to close roads, bridges, and detour traffic.

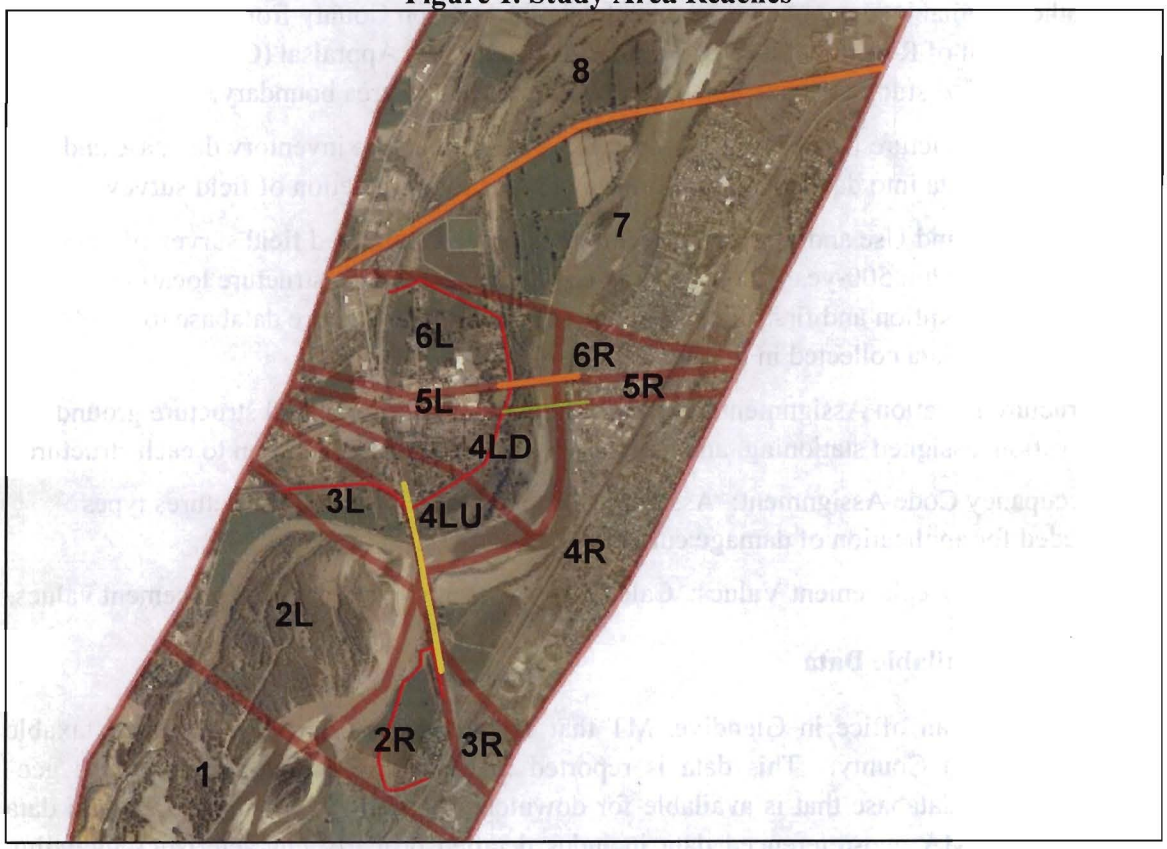
3.1.3 Study Area and Planning Reaches

The study area is located along the Yellowstone River through Glendive in Eastern Montana. To facilitate the analysis of existing levees, as well as preliminary action alternatives, the study area was divided into eight reaches. Five of these reaches were further subdivided into left and right bank reaches because of existing levees on either one or both sides of the river. One of the reaches (4L) was subdivided again in to capture differences in levee heights and water surface elevations. The defined reaches are described in Table 2 and shown in Figure 1.

Table 2. Study Area Reaches

Reach Name	Reach Station	Reach Description	Index Station
1	95.5 to 97.8	downstream of City of Glendive and Cottonwood Levee	96.77
2L	94.64 to 95.5	left bank across from Cottonwood Levee, downstream of Black Bridge (railroad bridge)	95.04
2R	94.64 to 95.5	right bank of Cottonwood Levee downstream of Black Bridge	95.04
3L	94.55 to 94.64	left bank from downstream of Black Bridge to Black Bridge	94.58
3R	94.55 to 94.64	right bank upstream of Cottonwood Levee to Black Bridge	94.58
4LU	94.08 to 94.55	left bank from Black Bridge to upstream of Bell St Bridge	94.30
4LD	93.52 to 94.08	left bank from midway upstream of Bell St. Bridge to Bell St. Bridge	93.83
4R	93.52 to 94.55	right bank from Black Bridge to Bell St. Bridge	94.08
5L	93.42 to 93.52	left bank from Bell St. Bridge to Towne St. Bridge	93.47
5R	93.42 to 93.52	right bank from Bell St. Bridge to Towne St. Bridge	93.47
6L	93.15 to 93.42	left bank from Towne St. Bridge to end of W. Glendive Levee	93.36
6R	93.15 to 93.42	right bank from Towne St. Bridge to end of W. Glendive Levee	93.36
7	92.15 to 93.15	upstream of W. Glendive Levee to Interstate Bridge	92.59
8	90.76 to 92.15	Interstate Bridge to upstream of City of Glendive	91.61

Figure 1. Study Area Reaches



3.1.4 Analysis Years and Period of Analysis

The 50-year period of analysis is from 2012 through 2062, and the structure inventory database is used to characterize existing and future conditions. According to Glendive's Floodplain Management Regulations and discussions with city planning officials, it is unlikely that there will be major changes to land use within Glendive's flood plain over the next several years. Therefore, for purposes of this analysis it is assumed that the existing level of development will remain the same for the period of analysis under future without-project conditions.

3.1.5 Land Use and Structure Inventory Database

The purpose of the structure inventory is to identify the existing floodplain structures and collect the data necessary to determine expected flood damages for various flood events. HEC-FDA modeling requires specific structure information, along with a set of occupancy types and associated depth-damage curves in order to calculate expected damages. Structure information needed includes structure location; the depreciated replacement value of the structure and its associated contents; and the elevation at which the structure is susceptible to flooding. This information is then used to compute existing and future conditions flood damages.

Development of the structure inventory included the seven steps listed below:

Step 1: Gather Available Data: Acquired parcel data for Dawson County from the Montana Department of Revenue (DOR) Computer Assisted Mass Appraisal (CAMA) database, and identified study area parcels that fall within the study area boundary.

Step 2: Organize Structure Database and Maps: Developed structure inventory database and organized data into data collection forms and maps for completion of field survey.

Step 3: Complete Land Use and Structure Inventory Survey: Completed field survey of structures within 500-year floodplain to verify occupancy type, structure location, structure condition and first floor elevation (FFE). Edited structure database to incorporate data collected in the field.

Step 4: Structure Location Assignment: Created parcel centroids, computed structure ground elevation, assigned stationing, and assigned left and right bank location to each structure.

Step 5: Occupancy Code Assignment: Assigned occupancy categories and structures types needed for application of damage curves.

Step 6: Depreciated Replacement Values: Calculated structures' depreciated replacement values.

3.1.5.1 Gather Available Data

The MT DOR has an office in Glendive, MT that establishes assessed values for all taxable property in Dawson County. This data is reported to the MT DOR and input into a geo-referenced CAMA database that is available for download through the Montana Cadastral data website. The CAMA geo-referenced data includes detailed property characteristics including

occupancy type, structure size (square footage, number of floors, etc.), structure building material, condition, and parcel boundaries.

The CAMA data layer was added to an existing map of the Glendive study area, defined as the 500-year floodplain boundary. Parcels within the 500-year flood plain boundary were selected and their associated data exported to begin the development of the structure inventory database. In total there were 944 parcels within the 500-year flood event boundaries for the 8 reaches.

Figure 2 shows an example of parcels selected within a particular reach – in this case reach 6L. The reddish-brown line is the boundary for the reach.

Figure 2. Parcels Selected with Reach 6L - Towne Street Bridge to End of West Glendive Levee

Some parcels within the study area did not contain structures, so one of the first steps in preparing the structure inventory database was removing parcels without structures from the database. The study area included 288 parcels that did not include any structures, and 656 parcels with structures.

3.1.5.2 Organize Structure Database and Maps

Next, the parcel/structure data for parcels extracted from the CAMA database were reviewed and organized into an easy-to-use format for the structure and land use survey.

In addition to providing a geo-spatial reference for study area parcels, the CAMA database included detailed structure information that was used as a starting point for assembling the data needed to estimate damages. All data variables included in the CAMA database were reviewed so only variables relevant to this analysis would be utilized for the structure inventory database. Examples of data retained for use in the analysis includes:

1. Parcel Address – These data were used to assign property addresses to structures, and to serve as a linking variable to the structure inventory provided by the Omaha District.
2. Parcel ID Number – These data were used to serve as a linking feature to structure characteristics and structure valuation data (see description of data obtained below).
3. Property Type – These data were used along with other information, to categorize structures by occupancy type for calculating depreciated replacement cost and assigning damage curves.
4. Structure First Floor Square Feet – These data were used to estimate the depreciated replacement cost of the structure
5. Structure Second Floor Square Feet – These data were used to estimate the depreciated replacement cost of the structure and determine the structure 'type' (1-story, split level, 2-story home)
6. Basement square feet – These data were used to evaluate if a structure has a basement and if it's full or partial basement.
7. Exterior Wall –These data described the exterior wall type (wood siding, masonite, brick, stucco, etc.), and were used along with other structure information to assign an estimate of the cost per square foot for each structure.
8. Physical Condition – These data indicated the assessed physical condition of the structure, ranging from (1 poor to 6 excellent), and were used along with other information, to determine the structure grade in terms of quality of construction for calculating the depreciated replacement value.
9. Year Built – These data indicate the year the structure was originally built.
10. Effective Year – These data are calculated by the MT DOR based upon the year a structure was built and if any major renovations have been completed on the structure. This information was used to estimate an appropriate depreciation factor.
11. Business Name – Business name information was provided in the CAMA database. It was also verified and updated during the structure inventory survey.

3.1.5.3 Complete Land Use and Structure Inventory Survey

Next, using the CAMA data as background information, a field survey of all structures within the 500-year flood plain was completed in May 2012. The field survey involved examining the extent of the floodplain limits, verifying structure characteristics including occupancy types, and gathering information on the first floor elevation of structures.

As described above the CAMA database has several variables that were utilized for the structure inventory database and ultimately for the calculation of the depreciated replacement cost for the structures and contents. Variables that were evaluated during the land use and structure survey were property type, basements, structure condition, exterior wall type, and business name.

Foundation heights for structures were recorded in half-foot increments, and were established through the direct examination of structures. Uncertainty values were assigned to first floor elevations based on tables 6-4 and 6-5 of EM 1110-2-1619, Risk-based Analysis for Flood-damage-reduction Studies dated August 1996. The errors in first floor elevations were assumed to have a normal distribution with a standard deviation of 0.5 foot.

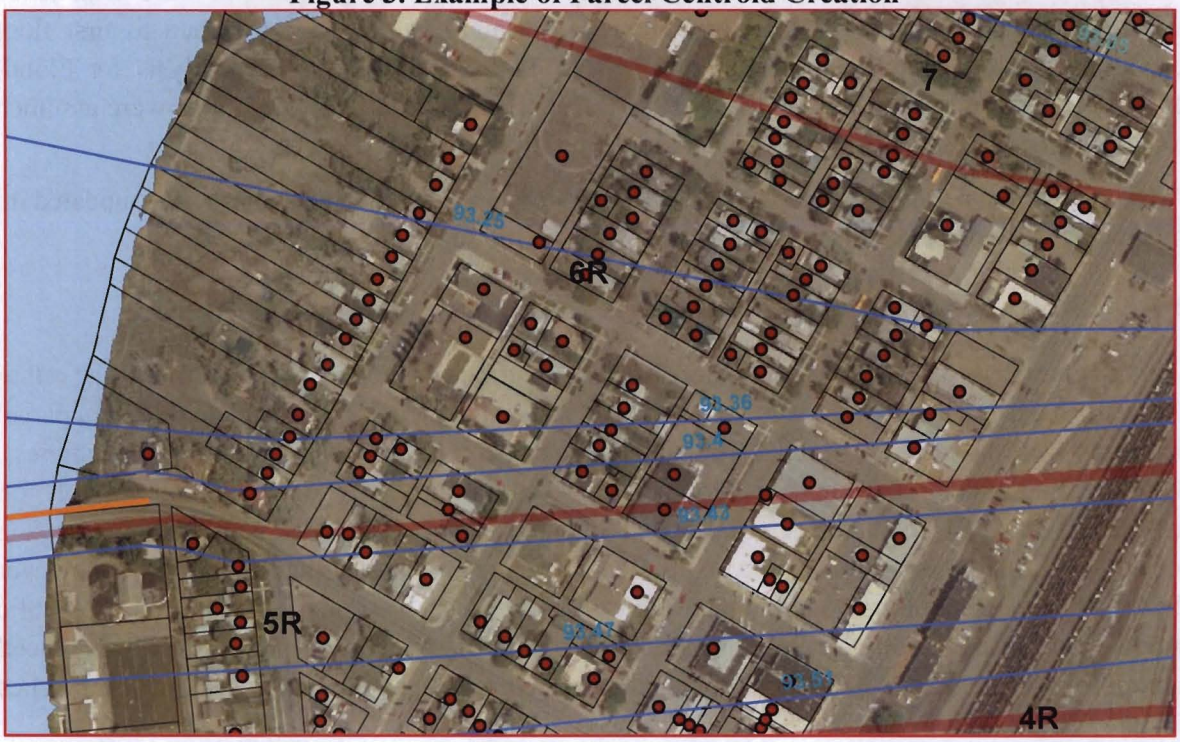
After the structure survey was completed, the structure inventory database was updated to include new or corrected information gathered during the survey.

3.1.5.4 Structure Location Assignment

Ground elevations and locations in relation to cross sections cannot be assigned to parcels; rather these attributes must be assigned to the structures within the parcel. To complete this step, parcel centroids were created in the GIS database for each parcel within the study area boundaries[1]. When created in GIS, centroids are automatically located in the center of each parcel (or polygon as defined in GIS) so many centroids were manually moved to more accurately reflect the location of structure(s) within a parcel. In general, there was one structure per parcel however there are some parcels that contain multiple structures, such as mobile home parks, public schools or apartment complexes. For parcels with multiple structures, one centroid was placed central to all structures. An example of parcel centroid creation and location is shown for reaches 5R and 6R below in Figure 3.

[1] In geometry, the centroid, or geometric center, of a plane figure or two-dimensional shape is the intersection of all straight lines that divide the shape into two parts of equal moment about the line. Informally, it is the "average" (arithmetic mean) of all points of the shape.

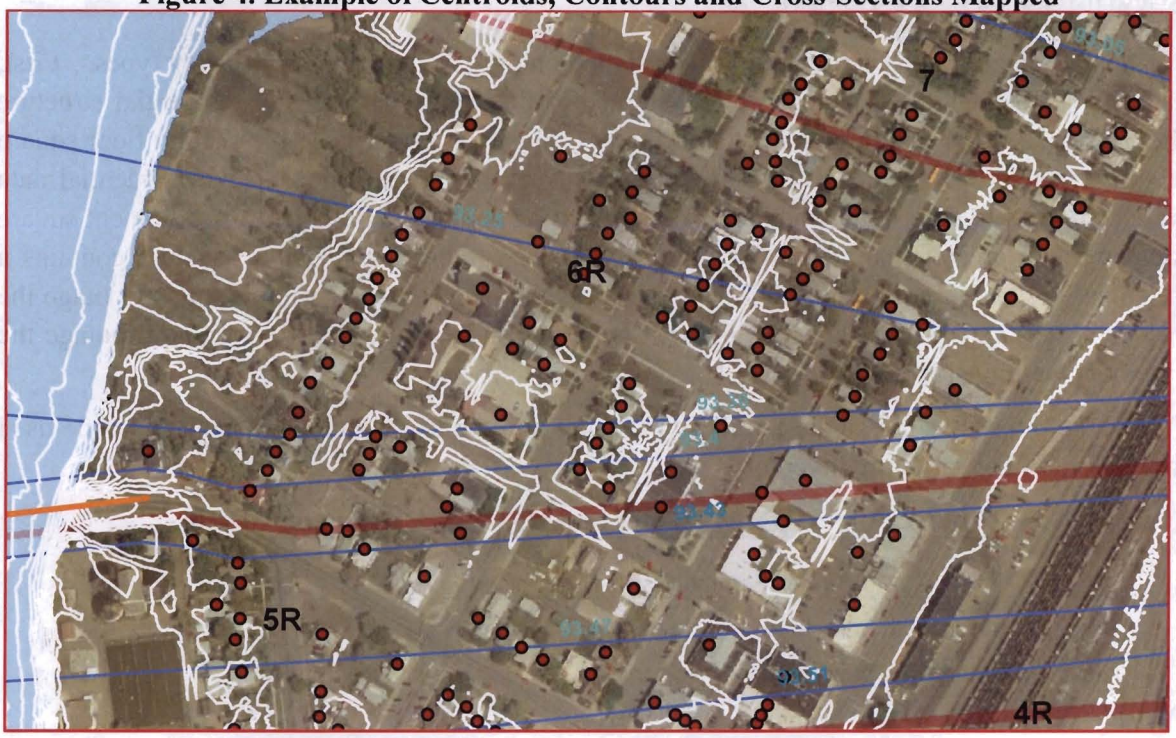
Figure 3. Example of Parcel Centroid Creation



After the structure centroids were added (now proxies for the structure locations[2]), one-foot topographic contours were used to create a digital elevation model (DEM), which was then used to assign a ground elevation for each centroid. Next, structures were assigned to cross-sections based upon their location so that each structure would be assigned a hydraulic station.

Figure 4 shows a map of structure centroids, topographic contours and cross-sections for reaches 5R and 6R.

Figure 4. Example of Centroids, Contours and Cross-Sections Mapped



3.1.5.5 Occupancy Code Assignment

Structure occupancy classification codes were assigned to parcels based on information collected during the structure inventory survey along with the assessor data from the CAMA database. Parcels containing structures were assigned to one of six damage categories:

- Residential Single-Family Dwellings (426 parcels)
- Residential Duplexes or Apartments (19 parcels)
- Mobile Homes (148 parcels)
- Industrial Businesses (8 parcels)
- Commercial Businesses (148 structures)
- Public (19 structures)

As described above, there was generally one structure per parcel, however for some unique parcel types multiple structures exist. For example, the number of mobiles homes assigned to a specific parcel varied from 1 to 119 mobile homes. Some parcels, such as public schools, apartment complexes or industrial buildings may include multiple connected structures, or multiple similar structures on one parcel. For these parcels the total number of buildings along with total square feet of the buildings was included in the analysis.

Residential Single Family Dwellings

Residential single-family structures were assigned one of five structure occupancy types. First, data for all residential structures were examined to determine whether the residential structure should be classified as a one- or two-story home. The assessor variable of "second floor square feet" was used to determine if the structure was 1 or 2 story residence. Next, residential data were examined to determine whether or not a dwelling contains a basement. A basement square footage value greater than zero was assumed to be an indication that the structure contains a basement. If the basement square footage was less than half of the first floor square footage the basement was considered partial, while if it were over half of the first floor square footage the basement was considered full.

Table 3 below shows the single family home types and the corresponding occupancy code used for the HEC-FDA structure inventory.

Table 3. Single Family Homes and HEC-FDA Occupancy Code

Single Family Home Type	Occupancy Code
1-story with basement	R01
2-story with basement	R02
Split level with basement	R03
1-story without basement	R04
2- story without basement	R05

Duplexes and Apartments

Occupancy codes for residential duplexes or apartments were assigned using a methodology similar to that for single-family dwellings. Data were examined to determine whether the structure should be classified as a one- or two-story building, and to determine whether or not the structure contains a basement. No duplexes or apartments in the study area were more than 2 stories tall. Table 4 below shows the classification of residential duplexes and apartments in the study area, with the corresponding occupancy code used in the HEC-FDA structure inventory.

Table 4. Duplexes or Apartments and Occupancy Code

Single Family Home Type	Occupancy Code
1-story apartment with basement	R08
2- story apartment with basement	R09
1- story duplex without basement	R10
2-story duplex without basement	R11

Mobile Homes

Occupancy codes for mobile homes were assigned based upon information gathered during the land use and structure inventory, along with mobile home information provided from the MT

DOR. Most of the mobile homes in the study area are part of a mobile home park, where multiple mobile homes are located on an individual parcel. The number of mobile homes per parcel was identified from MT DOR information and is included in the structure inventory database. All mobile homes in the study area were assigned the occupancy code R07.

Commercial Businesses

Occupancy codes for commercial businesses were assigned based on data obtained from the CAMA database along with information gathered during the land use and structure inventory. Fields such as "Business Name", "Business Type", and "Occupancy Code" provided insight as to the business type, along with communication with the City of Glendive to either clarify business type or to collect information from businesses.

Custom occupancy codes were developed for a handful of businesses unique to the Study Area. There were also a handful of vacant buildings in the Study Area. Commercial structure types and their associated occupancy codes are outlined in Table 5 below.

Table 5. Commercial Businesses and HEC-FDA Occupancy Name

Commercial Business Type	HEC FDA Occupancy Code
Expert Engineered:	
Clothing Retail Store Pre-Engineered	NR02
Convenience Store Engineered	NR03
Convenience Store Pre-Engineered	NR04
Electronic Retail Store Pre-Engineered	NR08
Fast Food Restaurant Pre-Engineered	NR10
Furniture Retail Store Pre-Engineered	NR12
Grocery Pre-Engineered	NR14
Hotel Pre-Engineered	NR18
Medical Office Pre-Engineered	NR22
Non-Fast Food Restaurant Pre-Engineered	NR24
Office Building Pre-Engineered	NR26
Recreational Facility Pre-Engineered	NR30
Service Station Pre-Engineered	NR36
Warehouse, Non-Refrigerated Engineered	NR37
Warehouse, Non-Refrigerated Pre-Engineered	NR38
Warehouse, Refrigerated Pre-Engineered	NR40
Custom:	
Industrial Pump Manufacturer	CP41
Funeral Home	CF42
Funeral Home Tribute Center	CT43
Service and Autobody Shop	CD44
Outdoor Recreation Center	CY45

Commercial Business Type	HEC FDA Occupancy Code
Auto Sales	CA46
Auto Warehouse	CA47
Carwash	CW48
Surveyed Grocery	CA49
Hardware Store	CP50
Implement Dealer	CI51
Vacant Convenience Store Pre-Engineered	V04
Vacant Office Building Pre-Engineered	VO26
Vacant Warehouse, Non-Refrigerated Pre-Engineered	VW38

Note: The designation of "Expert Engineered" or "Custom" defines which percent depth damage function is to be used in the FDA analysis. Depth-Damage Functions are discussed in Section 6 of this document.

3.1.6 Estimated Structure Depreciated Replacement Values

Depreciated replacement costs for structures in the study area were calculated using information from the CAMA database, land use and structure survey and cost information available from RSMeans Square Foot Costs and Marshall Valuation Service.

3.1.6.1 Residential Structures

Depreciated replacement costs (DRC) for residential structures, not including mobile homes, were calculated using the RSMeans Square Foot Costs. RSMeans uses square foot costs to approximate the value of the structure; then depreciation and locality factor are applied, resulting in a depreciated replacement value.

Depreciated replacement values for structures are subject to considerable uncertainty. In order to account for this, uncertainty measures were assigned to structure values, as suggested in Corps EM 1110-2-1619, dated 1 August 1996. Residential and non-residential structure value uncertainties were assumed normally distributed.

The DRC was calculated using a cost per square foot estimate for both residential and commercial structure, and a specific set of parameters, including:

- Building type: based upon structure occupancy
- Class of construction: Economy, Average, Custom and Luxury
- Exterior wall construction: wood, brick, concrete, etc.
- Square feet of structure

In addition to the above parameters, depreciation rates were applied to the cost estimates based upon the effective age and general condition of the residence. Table 6 shows the depreciation rates suggested by RS Means for residential structures.

Table 6. RSMeans Residential Depreciation Rates

Age in Years	Good	Average	Poor
2	2%	3%	10%
5	4%	6%	20%
10	7%	10%	25%
15	10%	15%	30%
20	15%	20%	35%
25	18%	25%	40%
30	24%	30%	45%
35	28%	35%	50%
40	32%	40%	55%
45	36%	45%	60%
50	40%	50%	65%

Note: values for intervening years are interpolated, and depreciation factors are held constant structures older than 50 years

A depth-damage curve was also applied to residential structures in the floodplain to account for appurtenant uses. Appurtenant uses include vehicles, equipment and landscaping, which can be accounted for in the HEC-FDA model by applying other-to-structure value ratios to all residential structures in the floodplain. For this analysis, a decision was made to focus on vehicle values only. Although other appurtenant use values exist in the Glendive study area, it was determined that they would contribute only a minor increase in value, but would require a significant number of hours to obtain incorporate the information into the model, so a decision was made to focus on vehicles only.

To incorporate the cost of vehicles that may be damaged during flood events the average price of 176 used vehicles was taken from three different Glendive car dealerships. The figure calculated was $12,250, which is acceptable due to the number of four-wheel-drive pick-up trucks and SUVs in the area. Using the 2006-2010 American Community Survey 5-Year Estimates for Selected Housing Characteristics, it was calculated that there would be an average of 1.79 vehicles per residence. To account some vehicles being moved to higher ground, it was assumed that there would be one vehicle per residential household.

3.1.6.2 Commercial Structures

The study area contains approximately 145 commercial structures. Each of these structures was examined in detail, and a variety of data sources were used to: (1) identify the type business currently on site and (2) collect information needed to calculate the DRC of the structure. Data sources included: DOR CAMA database, data gathered during the structure and land use survey completed in May 2012, and information gathered from City and County employees. Marshall Valuation Service was used to calculate the DRC for commercial structures and mobile homes. Similar to RSMeans, Marshall uses square foot costs to approximate the value of the structure. Then a depreciation and locality factor are applied, resulting in a depreciated replacement value.

3.1.7 Structures and Investment Value

A summary of investment value in the study area, including structure, content and appurtenant uses, as well as the number of structures, are shown in Table 7.

Table 7. Summary of Investment Value in Study Area ($1,000, FY14 Dollars)

Damage Category	No. of Structures	Structure Value	Content Value	Appurtenant Value	Total Value
Commercial	148	$36,536	$19,737	$1,205	$57,478
Industrial	8	$1,725	$3,295	$0	$5,020
Public	19	$34,981	$2,733	$0	$37,713
Residential					
Houses	426	$45,307	$45,307	$5,310	$95,925
Apartment buildings	19	$3,744	$3,744	$1,446	$8,934
Mobile Homes	156	$1,928	$2,814	$1,944	$6,686
Outbuildings	29	$437	$437	$361	$1,236
Total Residential	630	$51,416	$52,303	$9,061	$112,780
Total	**805**	$124,658	$78,068	$10,266	$212,992

The structure values are the total depreciated replacement cost. Content value is an estimate of the replacement cost for items stored within a structure. The generic content depth-damage curves for residential structures provided in the Economic Guidance Memorandum (EGM) 04-01, Generic Depth-Damage Relationships for Residential Structures with Basements, October 2003, represented the content depth-damage functions for residential structures in HEC-FDA. Using these relationships, it is not necessary to define the value of contents for a residence in HEC-FDA. As shown in Table 7, the content value houses and apartment buildings are valued at 100% of the structure value and over 100% for mobile homes. The expert-opinion depth-damage curves developed by FEMA and USACE experts for commercial, industrial and public structures represents content value as part of the curves as well, so it was not necessary to define the value of contents for these structures either. Content values were estimated for a few businesses that were contacted and questioned about potential flood damages.

3.1.8 Depth Damage Functions

Depth damage functions are used in the HEC-FDA program to estimate the damage caused to structures, structure contents, and appurtenant uses for given depths of flooding.

3.1.8.1 Residential Depth-Damage Functions

Each occupancy type has its own damage curve which estimates the stage-percent of value damaged for structures and contents. Residential structure depth-damage curves are based upon the structure and content curves published in the Economic Guidance Memorandum (EGM) 04-01, Generic Depth-Damage Relationships for Residential Structures with Basements; the EGM also include depth-damage relationship for residential structures without basements. The EGM

uses data developed by the Institute for Water Resources (IWR) from post-flood residential damage claim records provided by the Federal Emergency Management Agency (FEMA). As described above (Section 3.7), the damage-curve functions account for both structural and content damage to homes, and a curve was also applied to residential structures to account for appurtenant uses, which for this study only included vehicles.

Depth-damage functions are not known with certainty. To account for this and be consistent with guidelines specified in Corps EM 1110-2-1619, uncertainty variables were incorporated into the depth-damage functions. These variables included standard deviations for structure and content damages as reported in EGM 04-01.

3.1.8.2 Commercial Depth-Damage Functions

An expert-opinion elicitation process, conducted by the Federal Emergency Management Agency (FEMA) and the U.S. Army Corps of Engineers (USACE) for identification of the valuation and extent of damage to nonresidential structures and their contents from flooding was provided to the USACE for application to flood risk management studies (Solicitation of Expert Opinion Depth-Damage Function Calculations for the Benefit-Cost Analysis Tool, DRAFT, October 2008). Twenty-one nonresidential structure occupancy types were evaluated by a panel of experts recruited from across the United States. This resulted in the development of nationally relevant depth-damage curves and content-to-structure value ratios for use in estimating the value of damages expected to occur from a flood event. These FEMA/USACE expert-engineered depth-damage functions were used to estimate structure, content and appurtenant damages for most of the commercial structures in the study area.

Depth-damage functions for non-residential structures not adequately represented in the 21 non-residential structure occupancy types were developed from information gathered directly from Glendive businesses by the city of Glendive. Less than 10 businesses were contacted and six provided information used to develop custom depth-damage and content-to-structure value ratios.

3.1.9 Additional Damage Categories

This section provides a discussion of the additional damage categories that are oftentimes evaluated as part of the flood risk management study. Given that this effort is a Section 22 Planning Assistance to States study, a decision was made to expend more effort on evaluating the higher value items (i.e. potential structure and content damages), and to complete a more cursory evaluation of other damage categories. Potentially these other damage categories could possibly increase the EAD and therefore maximum affordable project by as much as 10 to 15 percent. This section presents information on these other damage categories but they are not currently included in the EAD calculation.

Additional damage types evaluated are:

- Infrastructure damages and emergency costs including temporary relocations

- Traffic detour and delay costs
- National Flood Insurance Program (NFIP) administrative costs

3.1.9.1 Infrastructure Damages and Emergency Costs

A major flood event in Glendive would damage public infrastructure, require emergency flood response operation during the flood event, and cleanup after the flood event. Damage to infrastructure would require repair and cleanup costs of municipal property including streets, storm sewers, sanitary sewers and debris removal from bridges and public property. Emergency flood fighting costs include emergency personnel and barricades to block off flooded streets that pose a safety threat as a result of flooding. A major flood event could also require residents of damaged homes to seek alternate quarters until their homes become habitable. Temporary housing and meal costs are included for residents temporarily displaced while flood damaged residential structures are cleaned up and repaired to a condition fit for habitation.

Costs associated with infrastructure damage, emergency response and cleanup can be included as part of the HEC-FDA modeling analysis, and are typically completed in this manner for a detailed general investigation analysis. However, for the purposes of this study, these costs are estimated based as a percentage of total EAD developed from prior studies and professional experience. Thus, infrastructure damage and cleanup is estimated to cost 10 percent of the calculated EAD or approximately $126,000. Emergency costs and temporary housing is estimated to cost five percent, or approximately $63,000.

3.1.9.2 Traffic Detour and Delay

Flooding or even the threat of flooding and public safety concerns may cause road closures and associated traffic detours. Road closures can last for the duration of actual flooding plus the time required for road cleanup and road repairs. As described in National Economic Development Procedures Manual-Urban Flood Damage, IWR Report 88-R-2, March 1988, the costs of traffic disruption are based on the vehicle operating costs for additional miles traveled because of the detour, and on traffic delay costs per passenger.

Although local city roads would experience closures if a major flood event occurred in Glendive, it's not possible to estimate detour and delays within the City. Instead the focus of the traffic detour and delay analysis is the closure of Interstate-94 (I-94) that runs through West Glendive and Glendive, crossing the Yellowstone River in northern Glendive. I-94 is the northernmost east-west Interstate Highway connecting the Great Lakes and the Intermountain regions of the United States.

Based upon H&H modeling of various flood events, traffic detours and delays are expected to occur on I-94 and the I-94 Bridge beginning around the .004 probability event. It is assumed that bridge closure would be necessary for safety, but the road itself may not actually be inundated. The length of closure is estimated at three days. At the .002 road inundation is expected on the I-94 Bridge, and the bridge would be closed for approximately one week.

Traffic counts for I-94 were available through the Montana Department of Transportation, including estimates of the percent of vehicles that were large trucks, small trucks and cars. Detour routes were measured and compared with mileage for the disrupted route to determine the additional miles that would be traveled with the detour. Additional sources and a brief overview of the methodology for calculating operating costs and delay costs, as well as the results, are below.

The traffic that crosses the I-94 Bridge is mixed and has a relatively high percent (about 25 percent) of heavy truck traffic. Passenger vehicles comprise the majority of vehicles that regularly cross the bridge. To calculate the detour costs for passenger vehicles, the latest version of AAA values of automobile operating costs was used (AAA, 2013). A composite of small, medium, and large sedans was created from the average operating costs for each type. The composite operating cost (including gas, maintenance, and tires) for passenger vehicles was calculated to be $0.2042 per mile. AAA was also used to calculate the operating cost of small trucks, which have a per-mile operating cost of $0.2615. The variable operating costs for heavy trucks was calculated using the "Per-Mile Costs of Operating Automobiles and Trucks" by Barnes and Langworthy (2003). The per-mile operating costs for heavy trucks were calculated using the same variable cost items as non-trucks and pickup trucks/SUVs (gas, maintenance, and tires). The variable operating costs per mile for heavy trucks was calculated to be $0.7443.

Table 8 shows the vehicles per day that cross the I-94 Bridge, and the operating costs and detour calculation for the I-94 bridge.

Table 8. Operating Costs Associated with Traffic Detour (FY14)

Vehicles Per Day	Vehicle Type	Operating Cost Per Mile	Average Detour (miles)	Detour Cost/Day	Detour Costs/250-Year Event	Detour Costs/500-Year Event
3,111	Passenger	$0.2042	16.9	$10,736	$32,208	$75,151
414	Small Truck	$0.2615	16.9	$1,830	$5,489	$12,808
1,096	Large Truck	$0.7443	16.9	$13,786	$41,357	$96,499
4,620			Total:	$26,352	$79,054	$184,459

Delay costs for drivers and passengers of vehicles were calculated in accordance with Engineer Regulation (ER) 1105-2-100, Appendix D, Economic and Social Considerations, Amendment #1, June 30, 2004. According to the 2010 U.S. Census Bureau, Dawson County, MT median family income is estimated at $63,982 per year, or approximately $30.76/hour (in 2012 dollars).

Table 9 below displays the delay costs associated with the closure of the I-94 Bridge.

Table 9. Opportunity Cost of Time of Traffic Delays (FY12

***Drivers Per Day	****Hourly Wage	Type of Trip	Average Delay (minutes)	Delay Factor	Delay Cost/Day	Delay Cost/250-Year Event	Delay Cost/500-Year Event
755	$30.76	Farm Work	20	0.538	$12,488	$37,464	$87,416
2,207	$30.76	Other Work (including industrial)	20	0.538	$36,517	$109,552	$255,620
1,659	$30.76	Other (e.g. shopping, recreation)	20	0.600	$30,616	$91,847	$214,309
4,620				**Total:**	**$79,621**	**$238,862**	**$557,345**

Based on the estimated additional detour and delay costs associated with closure of the bridge for the 250-year and 500-year events, the average annual damage estimate is approximately $2,700.

3.1.9.3 NFIP Administrative Cost Savings

Glendive is currently instated in Federal Emergency Management Agency's (FEMA) National Flood Insurance Program (NFIP.) As a part of this program, Glendive must adopt and enforce strict land use controls, including building codes, zoning regulations and subdivision regulations designed to limit flood damage to future construction within FEMA's specified Special Flood Hazard Area (SFHA). This area comprises the 100-year floodplain. Due to the City's participation in this program, property owners and renters are eligible to purchase national flood insurance. These policies provide for the reimbursement for eligible damages in the event of a flood.

Savings which accrue to the government as a result of the implementation of a flood control project are considered NED benefits (ER 1105-2-100, App. E). The savings in policy premiums are not considered however, because to do so would cause double counting of flood damages reflective in the policy premiums. The savings of flood insurance policy administration costs are not reflected as flood damages, and would constitute a benefit. Savings would occur to the extent present and future policy holders are removed from the SFHA and NFIP policies are no longer required.

According to FEMA records, there are 15 policies within Glendive. There are currently an estimated 58 residential structures (not including mobile homes) located within the 100-year floodplain. Flood insurance is mandatory for Federally-backed mortgage loans on structures located in the 100-year SFHA. For this reason, the numbers of policies is expected to increase over time as structures change ownership or are refinanced. Therefore it was estimated that in 2033 there will be 35 policies in force, or roughly 60 percent of the 58 residential structures. Commercial structures were not analyzed since recent estimates show that approximately only 5% of total NFIP policies are for commercial structures. To the extent commercial structures are not included in this analysis, estimated NFIP administrative costs savings are somewhat conservative.

The current annual savings is based on the 15 policies which cost on average $192/year (EGM 06-04, National Flood Insurance Program Operating Costs, Fiscal Year 2006), or $2,880 annually

for 50 years (the period of analysis). To estimate the annual value of likely future policies, an additional policy was added to the total each year for twenty years. After twenty years, the number of NFIP policies remains constant through 50 years. The value of each additional policy is discounted each year that a new policy comes on line. This is done for twenty years, then discounting continues for the remaining thirty years. The present value of the total cost over 50 years is then annualized (at 3.5 percent) and added to the current annual savings for a total annual flood insurance administrative cost savings of approximately $5,601.

3.2 Existing Conditions Flood Damages

The monetary value of potential flood damages are described in terms of expected annual damages given existing conditions. The following sections present the results of the analysis completed to quantify existing conditions expected annual damages and maximum affordable project.

Expected annual damages are based on fiscal year 2014 price levels, a FY14 discount rate of 3.5 percent, and a 50-year project life. The expected annual flood damages (EAD) under existing without project conditions are $1,265,230. This amount is based upon the value of likely damage to residential, commercial and public structures and content within the Glendive study area.

The EAD summary by reach and damage category is presented in Table 10. West Glendive would incur approximately 75 percent of the total damages (this includes reaches 3L, 4LU, 4LD, 5L and 6L). The highest expected annual damages would occur in reaches 4LU and 4LD, which are located between the Black Bridge and Bell Street Bridge and includes several commercial and industrial structures.

Table 10. Existing Conditions Expected Annual Damages (FY 2014 Prices, $1,000)

Damage Reach Name	Commercial	Industrial	Public	Residential	Total Damage
Reach 8	$0.00	$0.00	$0.00	$0.00	$0.00
Reach 7	$174.74	$0.00	$2.99	$11.14	$188.87
Reach 6L	$254.25	$0.00	$0.00	$73.28	$327.54
Reach 6R	$1.65	$0.00	$5.40	$4.18	$11.23
Reach 5L	$91.09	$0.00	$0.00	$11.55	$102.64
Reach 5R	$0.90	$0.00	$0.03	$0.21	$1.14
Reach 4LD	$29.58	$64.29	$0.00	$166.00	$259.87
Reach 4LU	$189.17	$5.74	$0.00	$13.35	$208.25
Reach 4R	$0.87	$0.45	$0.07	$36.67	$38.06
Reach 3L	$31.04	$0.00	$0.00	$27.99	$59.03
Reach 3R	$0.00	$0.00	$0.00	$0.00	$0.00
Reach 2L	$0.01	$0.00	$0.00	$23.97	$23.98
Reach 2R	$0.11	$0.00	$0.00	$44.50	$44.61
Reach 1	$0.00	$0.00	$0.00	$0.00	$0.00
	$773.41	**$70.48**	**$8.50**	**$412.85**	**$1,265.23**

3.3 Maximum Affordable Project

Corps procedures calculate benefits based on the difference between the expected annual damages with and without alternative flood damage reduction plans. The implicit assumption incorporated into this procedure is that the reduction in flood damages is directly translatable into increased net income to floodplain land uses. Benefits from flood damage reduction alternatives focus on inundation reduction benefits that would result from reduced physical damages to structures and contents, and reduced non-physical losses.

To begin the evaluation of potential alternative flood damage reduction plans, the calculation of an array of maximum affordable projects has been completed. The term 'maximum affordable project is in itself a bit misleading in that the costs of potential project alternatives have not been calculated, however the value of maximum affordable project is solely based upon an estimate of the reduced physical damages to structures and content.

Using the FY14 Federal Discount Rate of 3.5% and assuming a project life of 50 years, the maximum flood risk reduction for a project that would reduce flooding beyond the 500 year event would provide a $1,265,230 annual benefit. The maximum affordable project based on this annual benefit is $ 29,700,000.

Table 11 also displays a range of maximum affordable projects based upon lower targets of flood risk reduction – a 50-year level of flood risk reduction and a 100-year level of flood risk reduction.

Table 11. Maximum Affordable Project by Reach (3.5% discount rate)

Reach Name	50 Year Level of Flood Risk Reduction	100 Year Level of Flood Risk Reduction	500 Year Plus Level of Risk Reduction
Reach 8	$0	$0	$0
Reach 7	$4,000,000	$4,100,000	$4,500,000
Reach 6L	$0	$4,400,000	$7,700,000
Reach 6R	$0	$0	$300,000
Reach 5L	$0	$1,400,000	$2,400,000
Reach 5R	$0	$0	$0
Reach 4LD	$0	$3,200,000	$6,100,000
Reach 4LU	$0	$3,000,000	$4,900,000
Reach 4R	$0	$200,000	$900,000
Reach 3L	$0	$800,000	$1,400,000
Reach 3R	$0	$0	$0
Reach 2L	$300,000	$400,000	$600,000
Reach 2R	$0	$0	$1,000,000
Reach 1	$0	$0	$0
Total	$4,400,000	$17,500,000	$29,700,000

4. Evaluation of Preliminary Alternative

An evaluation of preliminary flood risk reduction alternatives was completed to assess the potential benefits that could be gained through a flood risk reduction project, as well as to assess the potential feasibility of action alternatives. The first alternative evaluated is a levee raise alternative and the second is a levee setback alternative. These two alternatives were selected for evaluation based on discussions among project delivery team members on what types of flood risk reduction measures may best address the flooding issues along with consultation with the City of Glendive. A description of each preliminary action alternative is provided in the next sections, followed by the alternatives analysis completed and results.

4.1 Levee Raise

As described in the PAS Study (City of Glendive/Dawson County Report), the levee raise alternative would raise the existing West Glendive levee above the project 100-year flood event to the computed 1 percent ACE composite ice profile plus 3-feet of freeboard.

Corps procedures calculate benefits based on the difference between the expected annual damages with and without alternative flood damage reduction plans. Benefits from flood damage reduction alternatives are inundation reduction benefits from reduced physical damages to structures and contents.

Table 12 shows the results of the expected annual damage analysis for the levee raise alternative by study reach. The benefit of the setback levee alternative is estimated at over $525 thousand annually in reduced structure and content damages. As described in the City of Glendive/Dawson County Report, the levee raise alternative would raise the West Glendive levee, which is on the left bank of the Yellowstone River, and increase protection or reduce damages to structures also located on the left bank. Some structures located on the right bank of the Yellowstone River would experience a slight increase in damages from a West Glendive Levee Raise (above existing condition damages), due to the increased water surface profile associated with a levee raise. These increased damages are termed induced damages and would require non-structural mitigation to maintain their existing level of flood risk reduction.

Table 12. Levee Raise Alternative – Benefits (FY 2014$)

Damage Reach Name	Total Damage Existing Conditions	Total Damage Levee Raise Alternative	Benefit of Levee Raise Alternative
Reach 8	$0.00	$0.00	$0.00
Reach 7	$188.87	$190.24	($1.36)
Reach 6L	$327.54	$143.52	$184.02
Reach 6R	$11.23	$12.07	($0.83)
Reach 5L	$102.64	$46.02	$56.62
Reach 5R	$1.14	$1.73	($0.59)
Reach 4LD	$259.87	$114.38	$145.49
Reach 4LU	$208.25	$100.06	$108.19
Reach 4R	$38.06	$41.87	($3.81)
Reach 3L	$59.03	$19.39	$39.64
Reach 3R	$0.00	$0.00	$0.00
Reach 2L	$23.98	$24.75	($0.77)
Reach 2R	$44.61	$45.51	($0.90)
Reach 1	$0.00	$0.00	$0.00
	$1,265.23	$739.52	$525.71

4.2 Setback Levee Alternative

Another alternative evaluated for the PAS Study is the setback levee alternative. As described in the City of Glendive/Dawson County Report, with the setback levee alternative a new setback levee would be constructed along the left bank of the Yellowstone River. The setback levee would add additional conveyance to the river by widening the channel and decreasing constriction points, which would decrease water surface elevations by approximately 1.5 feet when compared to the current 1 percent flow profile.

As described above for the levee raise alternative, Corps procedures calculate benefits based on the difference between the expected annual damages with and without alternative flood damage

reduction plans. Benefits from flood damage reduction alternatives are the inundation reduction benefits from reduced physical damages to structures and contents.

Table 13 shows the results of setback levee alternative plan analysis by study reach. As shown in the table, benefits from reduced expected annual damages would occur on both the left and right banks of the Yellowstone River, with the majority of the benefits occurring on the left bank of the river. The benefit of the setback levee alternative is estimated at over $837 thousand annually, in reduced structure and content damages. As part of this alternative several properties on the river side of a newly constructed set-back levee would need to be removed and relocated. Structure removal would contribute to the reduction in property damage, along with reduced levee fragility due to new levee construction, and lower water surface profiles provided by the increased water conveyance and decreased constriction points associated with a setback levee.

Table 13. Setback Levee Alternative – Benefits (FY 2014$)

Damage Reach Name	Total Damage Existing Conditions	Total Damage Setback Levee Alternative	Benefit of Setback Levee Alternative
Reach 8	$0.00	$0.00	$0.00
Reach 7	$188.87	$173.86	$15.02
Reach 6L	$327.54	$73.97	$253.57
Reach 6R	$11.23	$10.23	$1.01
Reach 5L	$102.64	$20.89	$81.75
Reach 5R	$1.14	$0.98	$0.16
Reach 4LD	$259.87	$21.71	$238.16
Reach 4LU	$208.25	$38.75	$169.50
Reach 4R	$38.06	$25.70	$12.36
Reach 3L	$59.03	$8.99	$50.04
Reach 3R	$0.00	$0.00	$0.00
Reach 2L	$23.98	$18.71	$5.27
Reach 2R	$44.61	$33.45	$11.16
Reach 1	$0.00	$0.00	$0.00
	$1,265.23	$427.23	$837.99

US Army Corps of Engineers
Omaha District

SECTION 22, PLANNING ASSISTANCE TO STATES

YELLOWSTONE RIVER
CITY OF GLENDIVE, MONTANA

HYDRAULIC ANALYSIS WITH COMPOSITE ICE

October 2014

SECTION 22, PLANNING ASSISTANCE TO STATES

YELLOWSTONE RIVER
CITY OF GLENDIVE, MONTANA
HYDRAULIC ANALYSIS WITH COMPOSITE ICE

October 2014

Prepared by
U.S. Army Corps of Engineers
Omaha District

For
City of Glendive, Montana
and
Dawson County, Montana

SECTION 22, PLANNING ASSISTANCE TO STATES

YELLOWSTONE RIVER
CITY OF GLENDIVE, MONTANA
HYDRAULIC ANALYSIS WITH COMPOSITE ICE

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SECTION 22, PLANNING ASSISTANCE TO STATES

YELLOWSTONE RIVER
CITY OF GLENDIVE, MONTANA
HYDRAULIC ANALYSIS WITH COMPOSITE ICE

1. INTRODUCTION

1.1. PURPOSE AND AUTHORITY OF STUDY

The purpose of this analysis was to develop the water surface profiles and flood boundary delineations for selected exceedance floods under ice conditions in the reach of the Yellowstone River in the City of Glendive, Montana. The analysis was conducted to support the Section 22, Planning Assistance to States (PAS) Study on the flood risk reduction opportunities for Glendive, MT by evaluating both existing conditions and potential flood risk reduction alternatives. While the Section 22 program will not in itself lead to construction of a flood risk management project, it will offer the City and County the opportunity to work closely with the Corps on the development of a study which will identify the flood risk, and quantify the frequency of flooding, the depths of flooding for various events, and the damages associated with the flooding. The Section 22 study will focus on potential feasible flood risk management alternatives which could be implemented to increase the level of protection the levee system provides.

2. SCOPE OF STUDY{TC \L1 "2. SCOPE OF STUDY}

2.1. AREA DESCRIPTION{TC \L2 "2.1. AREA DESCRIPTION}

The hydraulic study area includes the Yellowstone River reach within Dawson County, Montana. Dawson County is located in east-central Montana, approximately 12.5 miles from the North Dakota border to the east and 130 miles from the Wyoming border to the south. The Dawson County reach of the Yellowstone River consists of 56.5 river miles. A location map is shown in Figure 1 and identifies the Yellowstone River, Dawson County and the City of Glendive.

The City of Glendive is the only city located in the Yellowstone River flood plain in Dawson County. The city is approximately 90 miles upstream from the river's confluence with the Missouri River. The Yellowstone River flows from southwest to northeast through the city dividing it into Glendive proper and West Glendive. Glendive proper is located on the high, right bank bluff overlooking the Yellowstone River valley. West Glendive is located on the left bank flood plain and is protected from flooding by the West Glendive Levee, depicted in Figure 2. The Glendive reach of the Yellowstone River has seen much modification due to human development beginning in the late 19th century.

There are no significant tributaries of the Yellowstone River in Dawson County. However, a diversion dam and an irrigation canal are located at Intake at River Mile (RM) 71.3. This U.S. Bureau of Reclamation (USBR) irrigation project was constructed in the early 1900's and supplies water to approximately 53,000 acres in Eastern Montana and Western North Dakota. Canal flows vary seasonally with a maximum design flow of approximately 1,400 cubic feet per second (cfs). The canal returns excess water to the Missouri River at a point approximately 1.75 miles upstream of the Yellowstone-Missouri Rivers confluence.

The Yellowstone River is prone to ice jam flooding, typically between February and April when temperatures swings are the most extreme. A freezing and thawing trend allows ice to form, break apart, float downstream and potentially jam at a bend or other constricting point in the river. Ice jams can occur with little or no warning and result in abrupt upstream flooding.

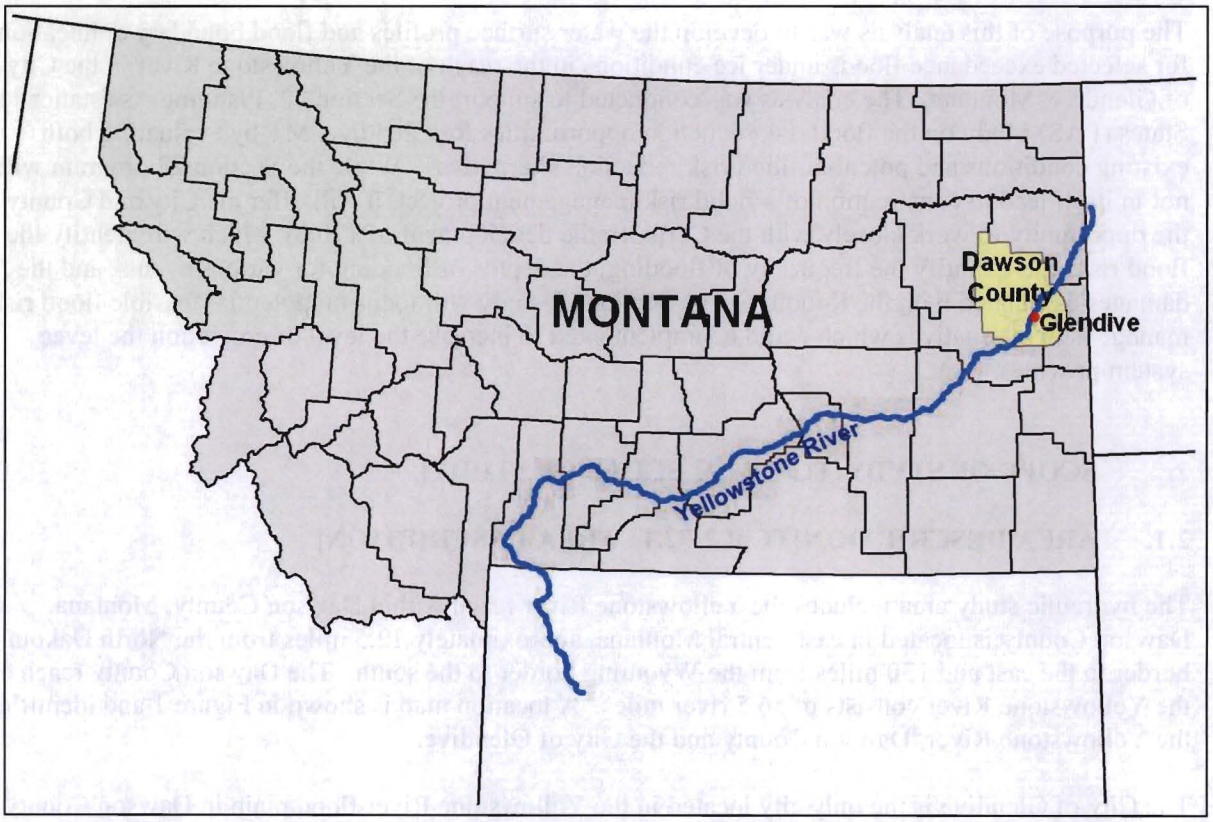

Figure 1. Location Map Yellowstone River, Dawson County and the City of Glendive location map in the State of Montana.

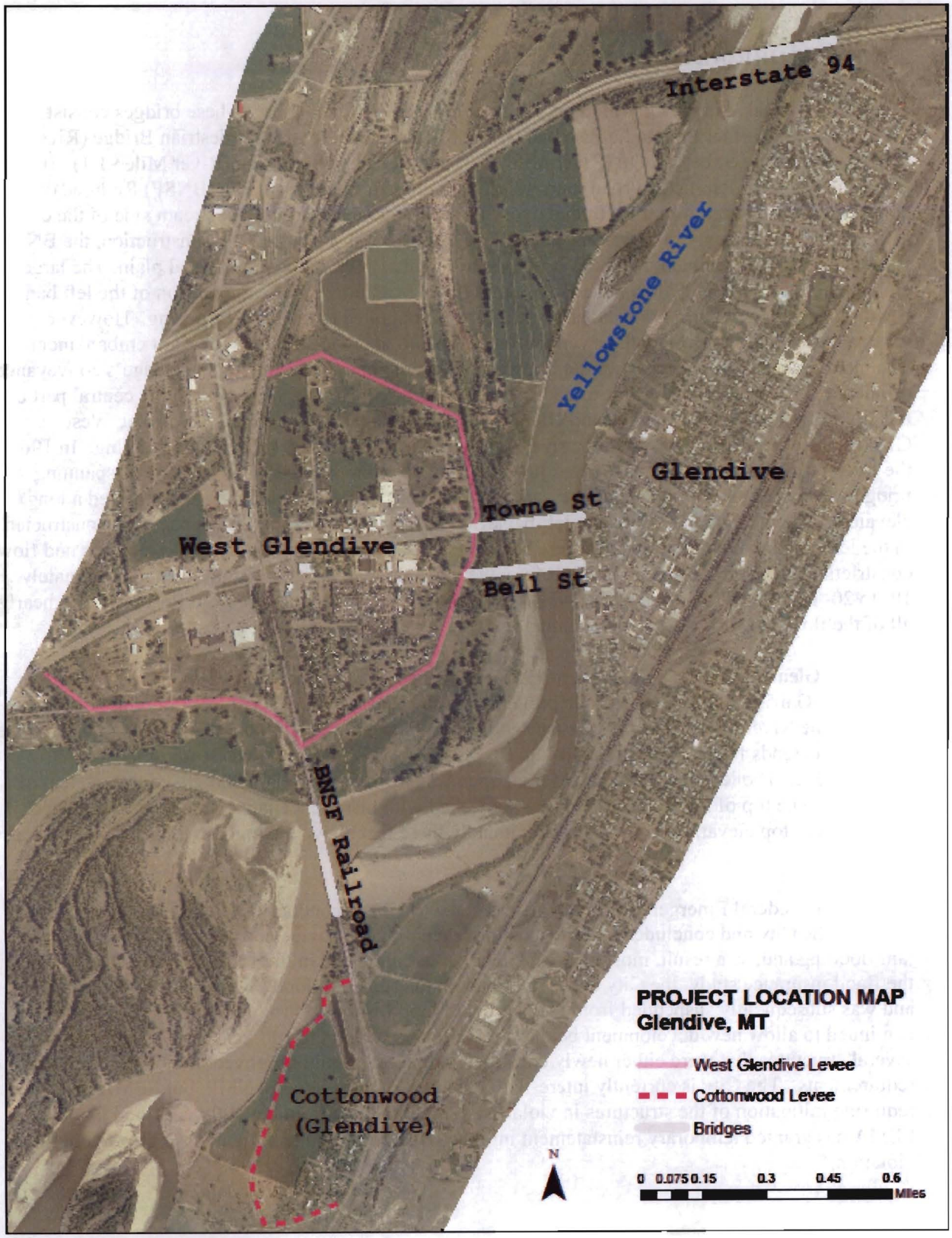

Figure 2. Project Location Map

2.2. HISTORICAL INFORMATION

Four bridges located in the City of Glendive span the Yellowstone River. These bridges consist of the Burlington Northern-Santa Fe Railroad (River Mile 93.5), Bell Street Pedestrian Bridge (River Mile 92.4), Towne Street Bridge (River Mile 92.3) and Interstate 94 Bridge (River Mile 91.1). In 1902, the Northern Pacific Railroad (currently Burlington Northern–Santa Fe (BNSF) Railroad) constructed a nine span iron truss bridge over the Yellowstone River at the upstream side of the city. This bridge is commonly referred to as the "Black Bridge." As a result of the construction, the BNSF Railroad Bridge left embankment elevation was as much as 10 feet above the flood plain. The large embankment prevented Yellowstone River flood flows from utilizing a large portion of the left bank flood plain, and the naturally meandering stream was contained to the bridge opening. However, upstream of the Black Bridge, the river continues to shift, encroaching upon the right embankment. The main river channel approaches the bridge piers at a sharp angle, reducing the bridge's conveyance efficiency. The Bell Street Bridge and later the Towne Street Bridge, were built in the central part of the city to connect West Glendive and Glendive. These bridges, in combination with the West Glendive Levee, further contained the river to its current alignment and restrict meandering. In 1968, the Interstate 94 Bridge was constructed just downstream from the city. This large, dual-spanning bridge was constructed at a height necessary to tie into the right bank bluff line and required a long, elevated road embankment across the left bank flood plain. An elevation dip, or sag, was constructed in the left embankment to alleviate severe backwater flooding that may occur due to ice jams and flow constriction at the bridge opening. Therefore, the embankment varies in height from approximately 10- to 20- feet above the flood plain and acts as a significant obstruction to flood flows, forcing nearly all of the flows through the bridge opening.

The West Glendive Levee, completed in 1959, was constructed by the U. S. Army Corps of Engineers–Garrison District to provide protection to the West Glendive area from flooding on the Yellowstone River. The federally constructed levee is located on the left overbank of the Yellowstone River and extends from just upstream of the BNSF Railroad Bridge to downstream of the Towne Street Bridge. The levee was designed for an open water discharge of 200,000 cfs with three feet of freeboard. The top of levee design was cross-checked with historic ice jam events which indicated that the levee top elevation would be approximately three feet above the highest recorded ice stage at that time.

In 1980, the Federal Emergency Management Agency (FEMA) conducted a flood insurance study (FIS) for the City and concluded that the existing levee did not provide adequate protection from ice jam flooding and, as a result, most of West Glendive was included in the 100-year flood plain. After the flood insurance study, the City chose not to adopt the required flood plain management ordinance and was subsequently suspended from the National Flood Insurance Program (NFIP). The City continued to allow new development behind the levee and within the 100-year flood plain resulting in several structures that were either newly constructed or substantially improved in violation of NFIP requirements. The City is currently interested in being reinstated into the NFIP, but FEMA is requiring mitigation of the structures in violation to remove them from the 100-year flood plain. FEMA has granted temporary reinstatement into the NFIP pending the mitigation of the structures in violation.

Open water flood events have not been a problem since the construction of the West Glendive Levee. However, the City of Glendive is still threatened by flooding from ice jams on the Yellowstone River. The city has experienced at least 30 ice jam floods since 1890 including major ice jam floods in 1899, 1936, 1969, 1986, and 1994. A total of 16 deaths have occurred from these flood events. The 1994 flood was due to the break-up of a major ice jam upstream from the community and the wave of water, debris, and ice rushing through the city. All other events were due to ice jam formation in the reach immediately downstream from the Interstate 94 Bridge. The 1969, 1986, and 1994 ice jam floods all came within 0.5- to 1.5-feet of overtopping the West Glendive Levee. Overtopping of the West Glendive Levee would result in catastrophic damages and high potential for loss of life as the depth and duration of flooding would be exacerbated by ponding behind the levee.

The majority of ice jams form near the county fairgrounds downstream from Interstate 94 and back up through the bridge opening. The Interstate 94 Bridge acts as a flow obstruction under ice jam conditions because of the high embankment that crosses the left flood plain of the Yellowstone River. The embankment cuts off a significant side chute of the Yellowstone River, which may have historically provided a relief valve for floodwaters to flow around the ice jams.

In 2002, the US Army Corps of Engineers conducted a Section 206 Study which reported that the total value of real property protected by the levee was approximately $19 million, which included approximately 130 residential structures and 57 commercial structures. Evaluation during recent studies shows the existing levee provides approximately 30-year flood protection under ice jam conditions and at least 50-yr protection under composite ice conditions.

2.3. RELATED STUDIES

The Yellowstone River has been studied by several agencies and organizations, including the U.S. Army Corps of Engineers (USACE). Four recent USACE studies of the Yellowstone River reach in Dawson County, Montana are described below.

"Section 206 Technical Assistance Study, Flood Plain Management Plan, Glendive, Montana." Omaha District, U.S. Army Corps of Engineers, dated March 2002.

This study used HEC-HMS and HEC-RAS to investigate alternatives to reduce the occurrence and severity of flood hazards through nonstructural measures, modification of the Interstate 94 Bridge, and flood warning systems along the Yellowstone River in Glendive, Montana. Results of the study indicate under ice jam conditions, the existing West Glendive Levee provides approximately 30-year flood protection with no freeboard and 10-year protection with four feet of freeboard. The study offers three alternatives to reduce flood damage hazards: levee raise, historic channel restoration at the Interstate 94 Bridge with levee removal and flood plain buyout, and historic channel restoration at the Interstate 94 Bridge with levee raise. The study shows the historic channel restoration at the Interstate 94 Bridge with levee removal and flood plain buyout as the most beneficial alternative; however, a General Investigation Study would be required to pursue this alternative.

"Lower Yellowstone Project at Intake, Montana." Omaha District, U.S. Army Corps of Engineers, Ongoing.

This ongoing study includes the National Environmental Policy Act (NEPA) efforts and final

design for a fish screen structure located at the U.S. Bureau of Reclamation diversion dam at Intake, located in Dawson County, Montana. The proposed project is a priority for recovery of pallid sturgeon on the Missouri River System. The dam acts as a significant impediment to the migration of fish up and down the river, and providing fish passage at Intake Diversion Dam would open up a minimum of 165 river miles of additional habitat that pallid sturgeon could utilize for spawning and other purposes. Survey data of the diversion dam obtained for this study was applied to the Dawson County hydraulic model.

"Yellowstone River, Dawson, County, Montana, Flood Plain Study: Open Water and Ice Conditions" Draft December 2009.

This draft study was part of the Yellowstone River Corridor Hydraulics Analysis to characterize the impacts of ice jams at the Interstate 94 Bridge in Glendive and evaluate the effects of increasing the conveyance through the bridge and left approach road at the location of the side channel cut off. Results showed that opening the bypass with a span of 100-feet or 500-ft did lower the water surface profile immediately upstream of the interstate. However, the lower water surface resulted in additional jamming at the Towne St Bridge. Water surfaces upstream of the Towne St Bridge were virtually unchanged despite the added conveyance at the I-94 Bridge.

"Yellowstone River Corridor Study, Hydraulic Analysis, Modeling and Mapping Report." Omaha District, U.S. Army Corps of Engineers, Draft February 2014.

This report describes the hydraulic analysis performed to support the Yellowstone River Cumulative Effects Assessment. The study is a joint effort of the River Conservation Districts Council and the U.S. Army Corps of Engineers, Omaha District. The interdisciplinary study examines hydrologic, hydraulic, geomorphic, biologic and socioeconomic characteristics of and impacts on the Yellowstone River and adjacent flood plain in a 460-mile reach from Gardiner, Montana, to the confluence with the Missouri River in western North Dakota. Study results are necessary to support the cumulative impacts assessment for the entire river corridor and develop a comprehensive plan that provides for sustainable use of the river and its flood plain for both economic and environmental needs. The hydraulic analysis involves modeling and mapping of various flows under both natural and human-impacted conditions.

3. HYDROLOGIC ANALYSIS{TC \L1 "3. HYDROLOGIC ANALYSES}

Peak Winter flow-frequency curves were required to determine the necessary percent chance exceedance flood flows for Dawson County under ice cover and ice jam conditions.

Flow records for the Glendive, MT reach were obtained from the Montana USGS draft report developed to support the Yellowstone River Corridor Cumulative Effect Assessment (USGS, 2013). The regulated-flow data sets were modified using HEC-DSSVue (vs. 2.0.1) to extract the daily flows between 01Jan and 15Apr for the period of record 1928 - 2002. The peak daily flows for each winter period were then selected using the math functions in HEC-DSSVue. Peak daily flows were then multiplied by 1.04 to convert them to an instantaneous peak flow. 1.04 was derived from averaging the ratio of the observed daily stream flows occurring on the same day as the observed instantaneous peak flow published on the USGS data website.

A discharge probability relationship was developed utilizing the methodology presented in Bulletin 17b (WRC, 1981) based on the 75 events. The methodology presented in Bulletin 17B utilizing the log-Pearson type III distribution was used and no regional skew was applied. The Hydrologic Engineering Center's (HEC) statistical software package program, HEC-SSP, was used to derive the frequency relationships. The station statistics from the analysis are listed in Table 1. Peak flows necessary for the hydraulic modeling are listed in Table 2 and include the 50, 20, 10, 4, 2, 1, 0.5 and 0.2-percent (%) annual chance exceedance (ACE) probabilities.

These discharges are smaller than those used in the 2009 ice study which utilized the Sidney, MT gage discharges (USACE 2009).

Table 1. Flow Frequency Statistics

Calculated Winter Flow Frequency Statistics	
Mean Logarithm	4.284
Standard Deviation	0.277
Station Skew	0.477
# Years Record	75
High Outlier	1952
Historic Events	No

Table 2. Annual Flow Statistics

Exceedance Probability Percent (%)	Flow (cfs) - Computed	
	Annual	Winter
0.2	151,000	151,000
0.5	136,000	133,000
1.0	125,000	106,000
2.0	114,000	83,600
4.0	103,000	64,900
10.0	86,800	44,800
20.0	74,000	32,300
50.0	54,200	18,400

Flow values were obtained from the regulated-flow draft data set and may vary from final values reported in the hydrologic analysis Yellowstone River Corridor Study (USGS, 2013).

4. HYDRAULIC ANALYSIS{TC \L1 "4. HYDRAULIC ANALYSIS}

The purpose of this analysis is to determine the hydraulic characteristic of the Yellowstone River reach in Glendive, MT under ice impacts. Delineation of the inundated flood plain and surface water profiles for the 50, 20, 10, 4, 2, 1, 0.5 and 0.2-percent (%) exceedance probabilities flood events were developed for ice conditions.

4.1. METHODOLOGY
{tc \l2 "4.1. METHODOLOGY}

Water surface profile models of the Yellowstone River were developed using the Corps of Engineers Hydrologic Engineering Center-River Analysis System (HEC-RAS) backwater computer program. The HEC-RAS program, Version 3.1.3 (released in May 2005), was developed for calculating water surface profiles for steady, gradually varied flow in natural or man-made open-channels. Newer versions of the model gave an error during the ice jam iterations and were not resolved by the completion of this study.

4.1.1. Survey Data

Topographic data of the Yellowstone River flood plain in Dawson County was collected in 2004 by Merrick and Company as Light Detection and Ranging (LiDaR) data utilizing a Merrick's ALS40 sensor and supplied as Triangulated Irregular Network (TIN) supported Digital Terrain Model (DTM). The data were provided in Montana State Plane Coordinates, NAD83, suitable for generation of 1-meter contours with spot elevations, accurate to National Map Accuracy Standards.

A limitation of the LiDAR technology is that it cannot penetrate water surfaces. Therefore, bathymetric data was collected in 2004 by Chris Ransome and Associates for a 15-mile reach near the City of Glendive. Bathymetry data was integrated into the TIN dataset. The surface model was an ESRI compatible TIN. All elevations are referenced to the North American Vertical Datum of 1988 (NAVD88). Aerial imagery was collected by Aerial Surveys International, LLC in May and June 2004.

4.1.2. Model Geometry

The Dawson County Yellowstone Corridor open-water model was used as the base model. Some cross sections throughout the City of Glendive were re-cut to better represent flow through/over the bridges and levee under ice conditions.

The terrain data set was used with HEC-GeoRAS, an ArcView GIS extension specifically designed to process geospatial data for use with HEC-RAS. The extension allows the creation of an HEC-RAS import file containing geometric attribute data from an existing DTM and complementary data sets. Similarly, results exported from HEC-RAS may also be processed.

Utilizing the HEC-GeoRAS extension, the hydraulic system was characterized by defining river, reach and station identifiers; cross-sectional attributes including cut lines, bank stations and roughness coefficients; and downstream reach lengths for overbanks and channel. Information was extracted and used to create a geometric import file compatible with the HEC-RAS model.

Structure information was manually entered into the HEC-RAS geometric input file. Bridge data was supplied by the USGS for the four bridges spanning the Yellowstone River in the City of Glendive. Bridge information included pier numbers, width and shape; high and low chord elevations; span lengths; and bridge deck width and are listed in Table 3. Bridge and pier skew are defined in the HEC-RAS bridge editor. Based on site observations and aerial photographs, the BNSF Railroad Bridge and piers span the main channel at approximately 70 degrees. However, at higher flows, the overbanks become inundated and the skew factor is reduced. The bridges were assumed to be free of debris and obstructions for this analysis.

Table 3. Bridge Parameters

HEC-RAS Station	Bridge Description	Span Length (ft)	Width (ft)	Number of Piers	HEC-RAS Bridge, Pier Skew (degrees, °)	Minimum Low Chord Elevation (ft NAVD 88)	Minimum Top of Road Elevation (ft NAVD 88)
49216	BNSF Railroad Bridge	1068	40	6	35°, 35°	2071.3	2067.6
47536	Bell Street Bridge	1340	40	6	0°, 0°	2071.2	2058.7
47357	Towne Street Bridge	1325	40	7	0°, 0°	2073.4	2059.1
45354	I-94 Bridge (dual)	1675	120	8	30°, 0°	2081.0	2065.0

Two levee systems were defined in the model. The federally built West Glendive Levee and non-federal Cottonwood Grove Levee were incorporated as existing conditions. The diversion dam at Intake was modeled as an inline structure. Dimensions of the rock structure and upstream and downstream bathymetric data were obtained from the concurrent USACE fish passage study at Intake (USACE, Ongoing).

A vegetative cover map for the Yellowstone River Corridor was supplied by the Montana Natural Resources Conservation Service (NRCS). Channel and overbank roughness values were assigned as Manning's roughness coefficients and determined from previous studies, field observation, aerial photography, and engineering judgment and based on land use, soil type and vegetation size, density and type. Values were incorporated into the HEC-RAS geometry file. A composite Manning's "n" value of 0.028 was assigned to the channel proper, and overbank values ranged from 0.035 to 0.1. The Dawson County flood plain is primarily agricultural rangeland but does consist of areas of dense vegetation and urbanization. The area behind the West Glendive Levee was assigned a high Manning's "n" value of 0.3 to reflect flow impedance.

Expansion and contraction coefficients for channel cross sections were 0.3 and 0.1, respectively. Additional losses near bridges were accounted for by establishing ineffective conveyance areas and increasing the expansion and contraction coefficients to 0.5 and 0.3, respectively.

This analysis utilized constant peak discharges with steady flow simulations. The exceedance discharges obtained from the hydrologic analysis were input as boundary conditions and modeled as constant flows throughout the reach. A starting water surface for each profile was computed by HEC-RAS using the normal depth calculation based on the energy slope for subcritical flow.

4.2. OPEN WATER MODEL CALIBRATION

{tc \l2 "4.2. MODEL SETUP}

Rating curves for the Glendive gage were obtained from the USGS and the National Weather Service (NWS). However, a gage datum discrepancy of one foot exists between the cited elevations. A USGS rating curve, developed in 2002, for the Bell Street Bridge gage cites a datum of 2040 ft NGVD29 with an offset of 39.00, and the NWS cites a datum of 2000 ft NGVD29. The HEC-RAS results were plotted against the two rating curves and, in general, were between the NWS and USGS rating curve data. However, due to the gage datum discrepancy, the model was not explicitly calibrated to either rating curve.

High water marks for the reach are limited, but a set was collected in Glendive following the May 2011 flood event (Tetra Tech Inc., 2011). A comparison to the high water marks was conducted and showed close agreement in stage at the Interstate 94 and Towne Street Bridges. The model yielded a higher stage at the BNSF Bridge than the reported high water marks. Due to the nature of high water mark identification and the length of time between the flood event and high watermark collection, the data were used for comparison but not as a calibration tool.

4.2.1. Low-Flow Calibration

A low flow analysis was conducted on the study reach. Underwater geometry data was only provided for a 15-mile reach encompassing the City of Glendive. In areas upstream and downstream of the bathymetry data, the minimal terrain elevation was limited to the water surface. Therefore, an analysis was performed based on the discharge in the river at the time of the aerial photography. A trapezoidal channel with a calculated depth was sized based on the average width of the main channel and channel slope, roughness and flow. The trapezoidal channel was incorporated into the HEC-RAS model to account for the unavailable underwater data. The dimensions of the trapezoidal channel were verified by plotting the resulting HEC-RAS water surface elevations on the aerial photographs. Adjustments to the channel were made at cross sections where the water surface elevation significantly deviated from the water line in the aerial photograph.

This method is an acceptable calibration tool for high flow studies. However, caution should be used in low flow studies due to the generalization of the underwater geometry.

4.2.2. Sensitivity Analysis

Sensitivity analyses were performed on the 1% chance exceedance flood event. Effects of "n" value adjustments were analyzed for both channel and overbank. Based on the *Engineer's Manual 1110-2-1619*, Figure 5-4, and the observed channel roughness, "n" values for the channel and overbanks were deviated by ±0.007. The slope used to calculate normal depth was also varied by ±20%, and expansion and contraction coefficients at bridges were varied by ±0.05. Results from the sensitivity analyses were compared to verify that changes to the system yielded minimal impact.

5. ICE CONDITIONS{TC \L1 "5. FLOODPLAIN MANAGEMENT APPLICATIONS}

Because of the frequent recurrence of ice-affected flooding on the Yellowstone River in the City of Glendive, the effects from a floating ice cover and ice jam were analyzed. Significant ice jams have been documented just downstream of the Interstate 94 Bridge, resulting in backwater flooding. The bathymetric data indicate the presence of sandbars and channel constrictions downstream of the Interstate 94 Bridge where ice jams have been reported. These factors typically contribute to ice jam formation. In addition, the HEC-RAS model suggests that the constriction of flow through the Interstate 94 Bridge opening may exacerbate the effects of the backwater flooding caused by the ice jam.

The hydraulic model geometry file includes an ice data table. User defined ice thicknesses, roughness values, and jamming options can be defined for each cross section. The ice data was incorporated into the model, calibrated to the limited available data, and the results of the current conditions were evaluated.

5.1. ICE DATA
{tc \l2 "5.1. FLOOD BOUNDARIES}

Ice has historically impacted the Yellowstone River. In the Glendive area, the damaging and fatal impacts of ice jams have been recorded since the 1890's. Despite the recurrent ice jam problems in the City of Glendive, detailed ice data is limited.

The USACE's Cold Regions Research and Engineering Laboratory maintains a database of historic ice jams throughout the United States, including Glendive. The database contains approximate dates and minimal descriptions of the jams; however, the database is not a comprehensive record of all ice jams. In addition, gage heights have been recorded at the Bell Street Bridge by the City of Glendive during the presence of ice. However, corresponding ice thicknesses were not recorded due to the difficulty of assessment. Ice out dates (i.e., the dates that the ice broke apart and floated downstream) and corresponding gage elevations were recorded.

Noted changes in the hydrology, channel morphology and overbank structures since the city's establishment directly impact the river system and resulting impacts from ice. Therefore, to evaluate current conditions of the Glendive area, hydrology and ice data subsequent to the completion of the Interstate 94 Bridge was used. The Interstate 94 Bridge was constructed in 1968 and is the most recent, significant change to the system.

5.2. ICE COVER
{tc \l2 "5.1. FLOOD BOUNDARIES}

Ice cover thickness was determined utilizing the USACE Engineering Manual for Ice Engineering (EM 1110-2-1612, 2002) and regional temperature data. Freezing degree days were calculated based on daily temperature data from the National Climatic Data Center (NCDC) for the period of 01Jan1928 through 31Dec2002. Ice thicknesses were computed based on ice equations provided in EM 1110-2-1612, limited historic ice information and ice characteristic assumptions. The calculated values are listed in Table 4.

Severe ice jams tend to occur in years with colder than average temperatures, based on the ice jam data obtained for Glendive, MT and as noted in ice jam studies on the Missouri River just below the mouth of the Yellowstone River. It is therefore reasonable to use an ice thickness greater than the average. For purposes of modeling an ice jam over the range of discharges for Dawson County, a thickness of 31 inches, the first standard deviation above the mean, is recommended. This thickness is also consistent with estimates of ice thicknesses based on photographs of past events near Glendive, as well as ice thickness measurements by USACE personnel following an ice run near Intake in March 2014.

Table 4. Range of Ice Thicknesses

	Ice Thickness
2nd Standard Deviation Above Mean	39 inches
1st Standard Deviation Above Mean	31 inches
Mean	26 inches
1st Standard Deviation Below Mean	21 inches
2nd Standard Deviation Below Mean	17 inches

Six ice jams have been documented in Glendive subsequent to the construction of the Interstate 94 Bridge. The initial ice cover thicknesses were calculated based on temperature data corresponding to each of these jams and are listed in Table 5. These initial ice cover thicknesses were used to calibrate other ice parameters in the ice jam model.

Table 5. Calculated Average Ice Thicknesses at Breakup

Year	Calculated Ice Thickness
1969	33 inches
1971	28 inches
1978	35 inches
1982	30 inches
1986	30 inches
1996	27 inches

The HEC-RAS open water model was utilized for the ice analysis. Ice cover thickness was entered in the model using the 'ice cover table' feature in the geometry file. An ice thickness of 31-inches (2.583 feet) was used as the initial channel ice thickness for the 50, 20, 10, 4, 2, 1, 0.5, 0.2% exceedance probability events. Ice cover was limited to the main channel only, and was not calculated in the overbanks. An ice roughness value of 0.03 was used for the ice cover simulations. Typical roughness values for ice can range from 0.03 to 0.1 and tend to increase with ice jam thickness, according to the USACE EM 1110-2-1612 (2002). Similarly, the HEC-RAS Hydraulic

Reference Manual suggests roughness values from 0.008 to 0.06 for a single layer of ice (sheet ice and frazil ice). The model was run with the computed peak winter flows listed in Table 2.

The winter 1% ACE event with and without ice cover profiles were plotted for the Glendive area in relation to the four bridges and federal levee and are depicted in Figure 3. As shown, the winter 1% ACE is below the levee crest by nine feet. Under ice cover conditions, the minimum freeboard along the Federal Levee, as calculated by HEC-RAS, is 4.4 feet. The winter 0.2% ACE with ice cover yielded a minimum freeboard of one foot.

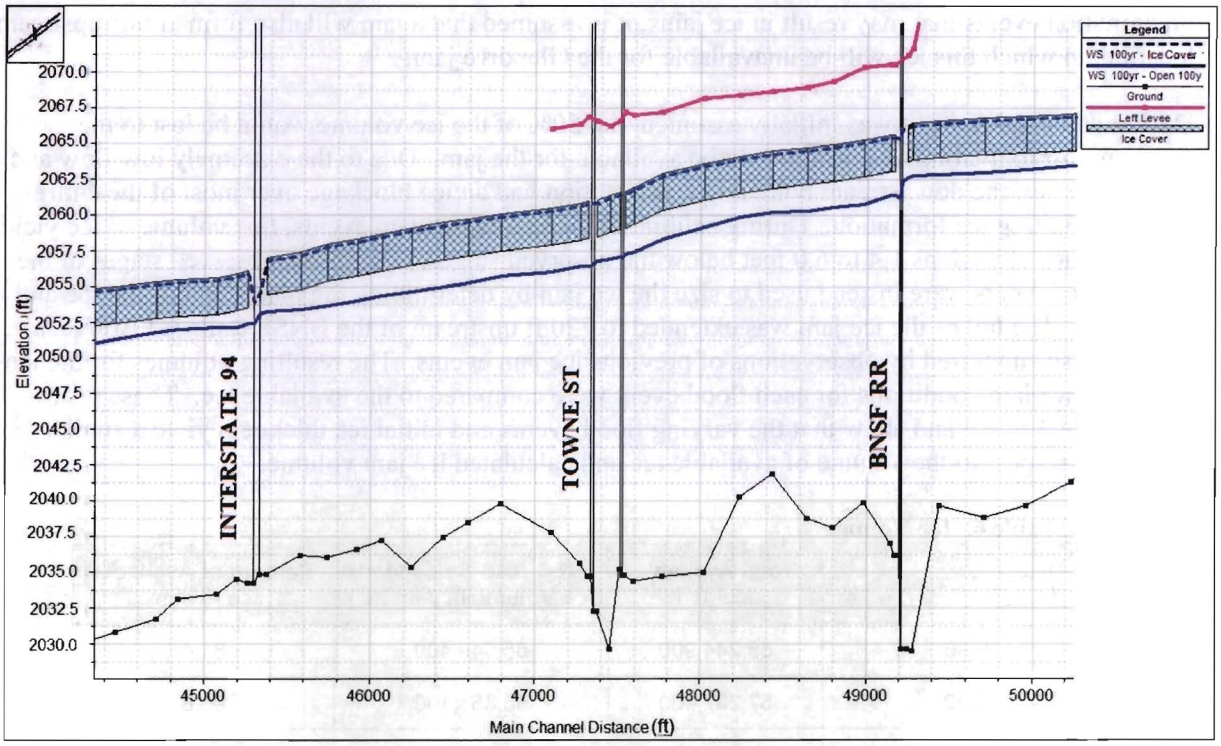

Figure 3. Water profile for ice cover and open water in Glendive, 1% ACE event.

5.3. ICE JAM

The HEC-RAS ice cover model was utilized for the ice jam analysis. Ice jam characteristics were entered in the model using the 'ice cover table' feature in the geometry file. The table allows the user to define the extents of the jam, initial ice thickness and ice roughness values. The locations of the downstream toe and the upstream tail of the jam were selected based on historic ice jam events, available ice volume and model stability. Residents of Glendive have reported historic ice jam formation just downstream of the Interstate 94 Bridge, near the fair grounds. Sand bars, vegetated islands and channel constrictions are apparent in the aerial photographs and bathymetric surveys. These factors can contribute to ice jam formation. The ice jam toe was placed 3,200 feet downstream of the Interstate 94 Bridge.

An ice thickness of 31-inches was used as the initial channel ice thickness for the 50, 20, 10, 4, 2, 1, 0.5, 0.2% exceedance probability events. The ice jam was limited to the main channel only, and was not calculated in the overbanks. An ice roughness value of 0.03 was used for the initial ice cover, and the option for the ice jam roughness values calculated by HEC-RAS was selected.

The volume of ice available for the jam was determined by modeling the February 50% flow duration discharge with ice cover. The volume of ice was calculated from the cross section downstream of the Interstate 94 Bridge, where the jam was selected to start, to approximately ten miles upstream of the BNSF Railroad Bridge, where ice jams have also been reported. During temperature changes or precipitation events that may result in ice jams, it is assumed that a jam will also form at this upstream location, in which this ice will be unavailable for the Glendive jam.

During jam conditions, it was initially assumed that 50% of the ice volume would be lost to the overbanks or to melting and 50% would be available for the jam. Due to the extremely low flow and thick ice cover, the downstream bounding cross section had an ice blockage over most of the entire channel, limiting ice formation. During calibration of historic ice jam events, this volume of ice yield water surface elevations 1.4 to 6.0 feet below the observed stages. Therefore, observed stages of the historic jam events were instead used to size the ice jam by determining the placement of the toe and tail extent. The tail of the ice jam was extended to 720 ft upstream of the BNSF Railroad Bridge, a location also supported by observations of previous ice jam events. The resulting volumes for the ice cover and ice jam conditions for each flood event were compared to the available ice. These results are listed in Table 6 and show that the varying flood events and initial ice thickness yield a similar trend with respect to the volume of available ice and calculated ice jam volume.

Table 6. Ice Volume

Flood Event	Total Available Ice (ft^3)	HEC-RAS Calculated Ice Volume in Jam (ft^3)	Percent of Available Ice in Jam
1996	68,244,900	55,549,400	81.4
1982	57,207,400	55,351,100	96.8
1978	94,051,600	64,362,600	68.4
1986	83,682,100	65,732,200	78.5
1971	75,764,000	66,206,200	87.4
1969	89,532,400	69,130,300	77.2
1% Exceedance Event	84,305,600	81,051,900	96.1

NOTE: Available ice was calculated based on ice cover formation, listed in Table 5, during each flood event from XS 66212 (10 miles upstream of the BNSF RR Bridge) to XS 44345 (0.5 miles downstream of the I-94 Bridge.)

The ice jam events listed in Table 7 were modeled with constant ice cover in the reach of Dawson County just downstream of the Interstate 94 Bridge. The ice jam forms 3200 ft downstream of the Interstate 94 Bridge and extends 720 ft upstream of the BNSF Railroad Bridge. The reach upstream of the jam was not modeled with ice. Calibration results for the historic flood events are shown in Figure 4.

Table 7. Historic Ice Jams and Resulting Jam Thicknesses

Flood Event	Discharge (cfs)	Initial Ice Thickness (ft)	Maximum Ice Jam Thickness Calculated by HEC-RAS (ft)
1996	15,800	2.25	13.57
1982	17,600	2.33	14.12
1978	36,300	2.92	16.87
1986	44,100	2.5	17.64
1971	49,000	2.5	18.12
1969	58,400	2.25	19.96

A sensitivity analysis was performed on the ice jam model for the 1969 ice jam event. This event was modeled with an initial ice cover of 2.75 ft and a flow of 58,400 cfs. Changes in ice roughness values, fixed roughness values, ice porosity and under-ice velocity were made to verify the resulting impacts on the system were minimal.

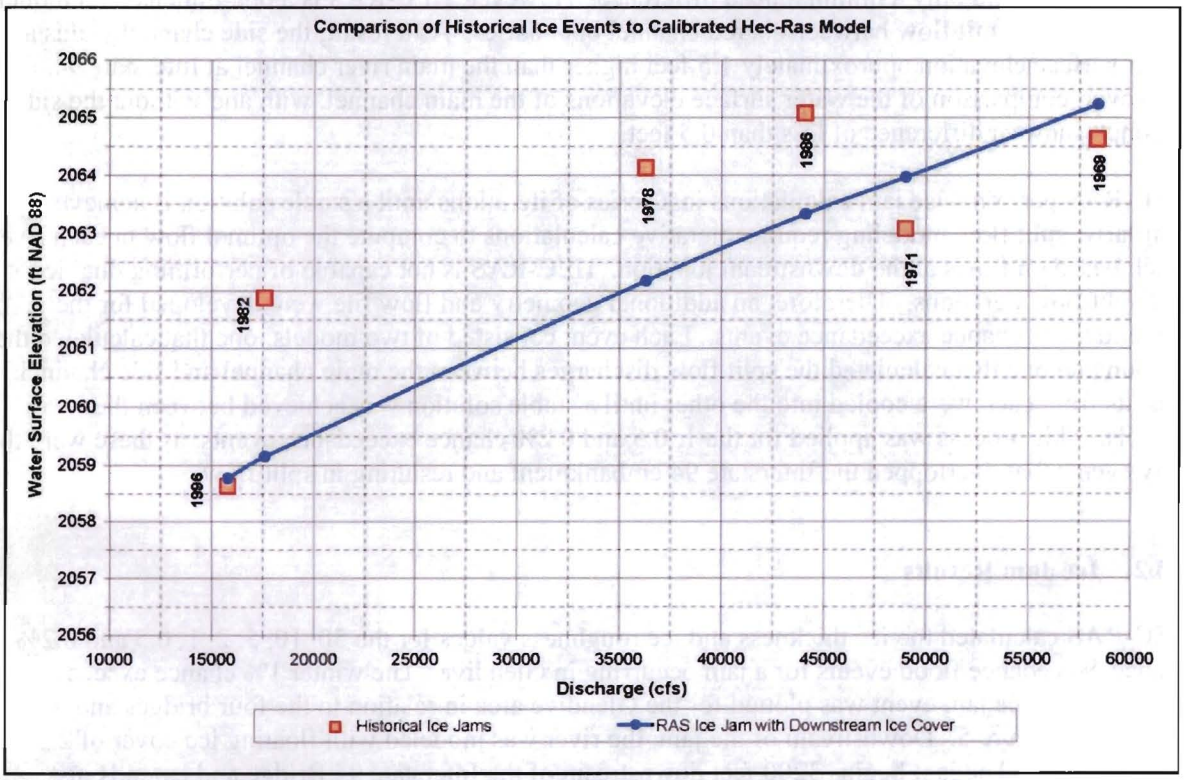

Figure 4. HEC-RAS Water Surface Elevations for Historic Ice Jam Events

5.3.1. Ice Jam Modeling Difficulties

{TC \L2 "5.1. FLOOD BOUNDARIES}

The Winter Season 50, 20, 10, 4, 2, 1, 0.5, 0.2% exceedance probability flood events were modeled with HEC-RAS calculating jams. The 2% exceedance with ice jam event yielded a backwater flood that overtopped the upstream portion of the Federal Levee. A revised geometry file with adjusted areas of ineffective flow was created to adequately model the overtopping. The 1 and 0.2% exceedance probabilities with ice jam result in overtopping of the entire Federal Levee and overtopping of the Interstate 94 left embankment. Revised geometry files were created for these events to address the reduced areas of ineffective flow and the flow split created by the overtopping of the Interstate 94 embankment. A side channel was incorporated into the geometry file on the left overbank and followed the natural geometry where the previous channel existed prior to the construction of Interstate 94. The side channel is approximately 10,000 ft in length with an invert approximately 10 ft higher than the main channel invert. To model the split flow, an upstream junction was incorporated 200 ft downstream of the Towne Street Bridge and a downstream junction was incorporated 1600 ft downstream of the Interstate 94 Bridge. The side channel allows flow across the low elevation (sag) in the Interstate 94 embankment.

The side channel and split flow become effective at very high flows and elevated stages due to the ice jam. At these elevations and flows, there is a high water interface between the two defined channels which would yield only a minimal head difference. However, HEC-RAS is a one-dimensional model and does not permit flow between shared channel boundaries. As a result, the side channel yields a water surface elevation approximately 1.5 feet higher than the main river channel at Interstate 94. However, comparison of the water surface elevations of the main channel, with and without the side channel, shows a difference of less than 0.5 feet.

HEC-RAS performs ice jam calculations in a series of iterations until a stable solution is achieved. Similarly, split flow modeling requires iterative calculations to compute the optimal flow in each river reach based on flows at the downstream junction. HEC-RAS is not capable of performing dual ice and split flow iterations. Therefore, an additional geometry and flow file were developed for the 1, 0.5, and 0.2% chance exceedance events. Each event consisted of two models: one that calculated the ice jam and one that calculated the split flow discharges between the main channel and side channel. Results from each were copied into the other until a stable solution was achieved between the two models. This process was applied for the 1, 0.5 and 0.2% chance exceedance events, as these were the only events that overtopped the Interstate 94 embankment and resulting in split flow.

5.3.2. Ice Jam Results

HEC-RAS calculated the ice thickness and ice roughness values for the 50, 10, 5, 2, 1, 0.5 and 0.2% chance exceedance flood events for a jam occurring in Glendive. The winter 1% chance exceedance flood with an ice jam event was plotted for the Glendive area in relation to the four bridges and is depicted in Figure 5. Downstream of the jam, the river was modeled with floating ice cover of 2.58 feet thickness. The jam begins 3200 feet downstream of the Interstate 94 Bridge and extends just upstream of the BNSF Railroad Bridge.

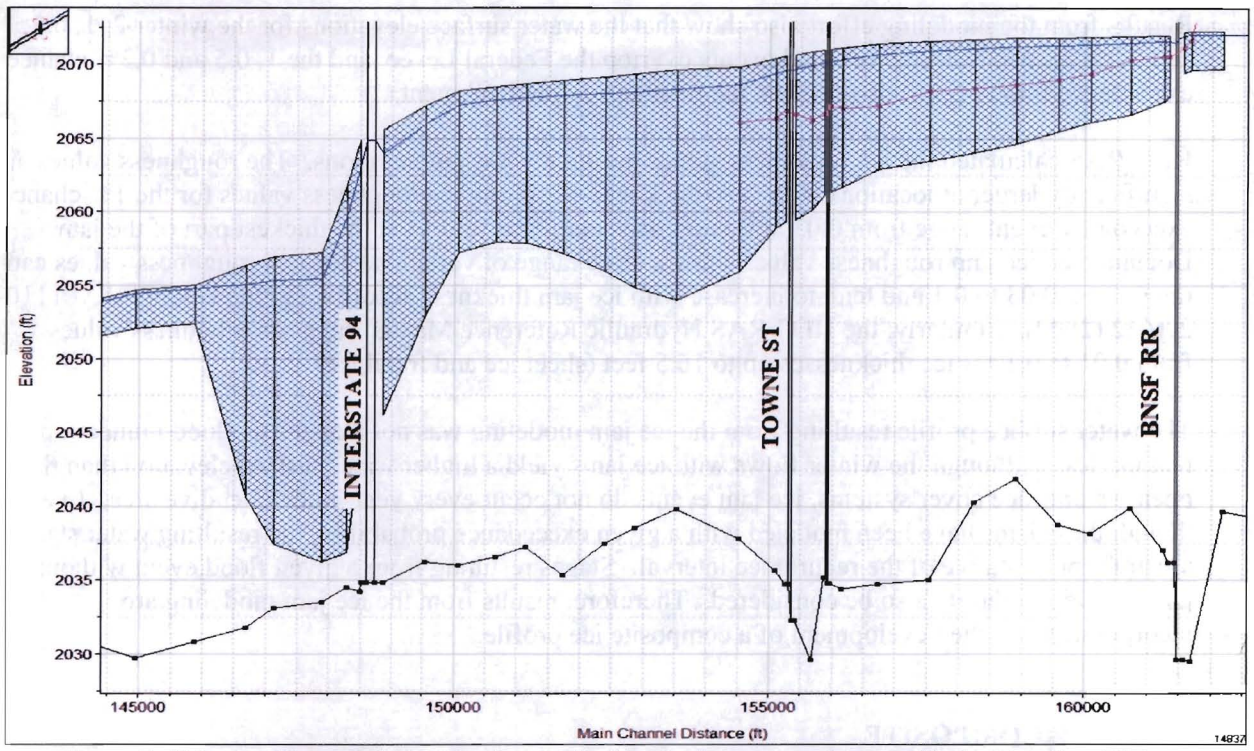

Figure 5. Winter 1% exceedance flood event ice jam and water surface profile in Glendive, MT.

Jam thicknesses were calculated by HEC-RAS and maximum thicknesses are listed in Table 8 for each flood event. The maximum ice jam thickness computed by HEC-RAS assumes a stable, floating ice jam. Actual thicknesses could be much greater if the toe of the ice jam were grounded; however, HEC-RAS cannot compute ice jam thicknesses for grounded jams.

Table 8. HEC-RAS Calculated Maximum Ice Jam Thicknesses

Flood Event (% chance exceedance)	Maximum Ice Jam Thickness Calculated by HEC-RAS (ft)
50	16.2
20	17.0
10	18.4
4	21.2
2	23.1
1	25.1
0.5	26.5
0.2	27.5

Results from the modeling effort also show that the water surface elevations for the winter 2, 1, 0.5 and 0.2% chance exceedance flood events overtop the Federal Levee, and the 1, 0.5 and 0.2% chance exceedance flood events also overtop the Interstate 94 embankment.

HEC-RAS calculated the ice roughness values during the ice jam iterations. The roughness values are significantly larger at locations with very thick ice. Resulting ice roughness values for the 1% chance exceedance event range from 0.03 at floating ice conditions to 0.14 at the thickest part of the jam. Documented ice jam roughness values show a large range of variation. Ice jam roughness values can range from 0.03 to 0.1 and tend to increase with ice jam thickness according to the USACE EM 1110-2-1612 (2002). Similarly, the HEC-RAS Hydraulic Reference Manual suggests roughness values from 0.01 to 0.1 for ice thicknesses up to 16.5 feet (sheet ice and frazil ice).

The water surface profile resulting from the ice jam modeling was not mapped as flood inundation boundaries. Although the winter flows with ice jams yield a higher water surface elevation than the open water or ice cover systems, ice jam events do not occur every year in the Glendive area. Even though these jams have been modeled with a given exceedance probability, the resulting water stages are not representative of the recurrence interval. Stages resulting from a given flood event without ice jam conditions should also be considered. Therefore, results from the ice jam modeling are incorporated into the development of a composite ice profile.

5.4. ICE COMPOSITE

Composite stage-frequency rating curves were developed for an area encompassing the City of Glendive using the indirect approach outlined in FEMA's *Appendix F: Guidance for Ice-jam Analyses and Mapping.*

Water surface profiles were developed for three hydraulic conditions using HEC-RAS:

1. Open water, winter season discharges
2. Ice-cover, winter season discharges
3. Ice-jam, winter season discharges

FEMA's *Appendix F* was used to determine the stage for the winter 1% chance exceedance event based on ice impacts.

For each condition, water surface profiles for the 50, 20, 10, 4, 2, 1, 0.5, 0.2% exceedance probability discharges were plotted for each cross section to develop stage frequency rating curves for the ice cover and ice jam conditions. The stage-frequency rating curves were used to develop the ice-affected composite stage using a variation of Appendix F, Equation 3. This equation is a modified version of Appendix F, Equation 1 used to combine curves:

$$P(s) = [P(sw)*P(si=\text{ice-jam event}) + P(so)*P(si=\text{free-flow event})] + P(sq) - [(P(sw)*P(si=\text{ice-jam event}) + P(so)*P(si=\text{free-flow event})) * P(sq)]$$

where

P(s) = Probability of the annual-maximum stage exceeding a given stage "s" in any year, by either type of event

P(sw) = Probability of exceeding a given stage "s" in the ice-jam season developed using the discharge-frequency relationship for the ice-jam season and ice-jam hydraulic conditions

P(si=ice-jam event) = Fraction of years during the ice-jam season that peak stages are affected by ice jams

P(so) = Probability of exceeding a given stage "s" in the ice-jam season developed using the discharge-frequency relationship for the ice-jam season and free-flow hydraulic conditions

P(sq) = Probability of the annual-maximum stage exceeding a given stage "s" in the free-flow season

P(si=free-flow event) = Fraction of years during the ice-jam season that peak stages are free-flow events

A variation of this equation is necessary to calculate the composite ice profile in the City of Glendive by including a factor for ice cover, in addition to open water and ice jam conditions. Ice cover occurs in the winter season, occurs every winter (i.e., more frequently than ice jams) and contributes to a higher stage than the open water events. Therefore, it was necessary to incorporate the effects of ice cover separate from the effects of ice jams:

P(s) = [P(sw)*P(si=ice-jam event) + (P(sc)*P(si=ice-cover event)) + P(so)*P(si=free-flow event)]+ P(sq) – [(P(sw)*P(si=ice-jam event) + (P(sc)*P(si=ice-cover event))+ P(so)*P(si=free-flow event)) * P(sq)]

where

P(sc) = Probability of exceeding a given stage "s" in the ice-cover season developed using the discharge-frequency relationship for the ice-jam season and ice-cover hydraulic conditions

P(si=ice-cover event) = Fraction of years during the ice-jam season that peak stages are affected by ice cover

Meteorological records for Dawson County show that ice formation has the potential to occur every winter. Therefore, it is assumed that peak stages occur under ice-cover or ice-jam conditions and the probability of a peak stage occurring during open flow conditions in winter is near zero. Similarly, the probability of peak stages occurring during open flow summer season that equal or exceed the peak ice-affected stages is near zero.

Historical records indicate that ice jams have a 0.303 chance of occurring in a year, and ice cover without ice jams have 0.697 chance of occurring in a year. Figure 6 compares the HEC-RAS ice cover stages to the historical stages. Significant increase in stage for a given discharge when compared to the corresponding ice cover stage is assumed to be a result of impacts from an ice jam. The annual ice jam recurrence percents of 0.303 and 0.697 are applied to the revised Equation 3 as P(si=ice-jam event) and P(sc=ice-cover event), respectively.

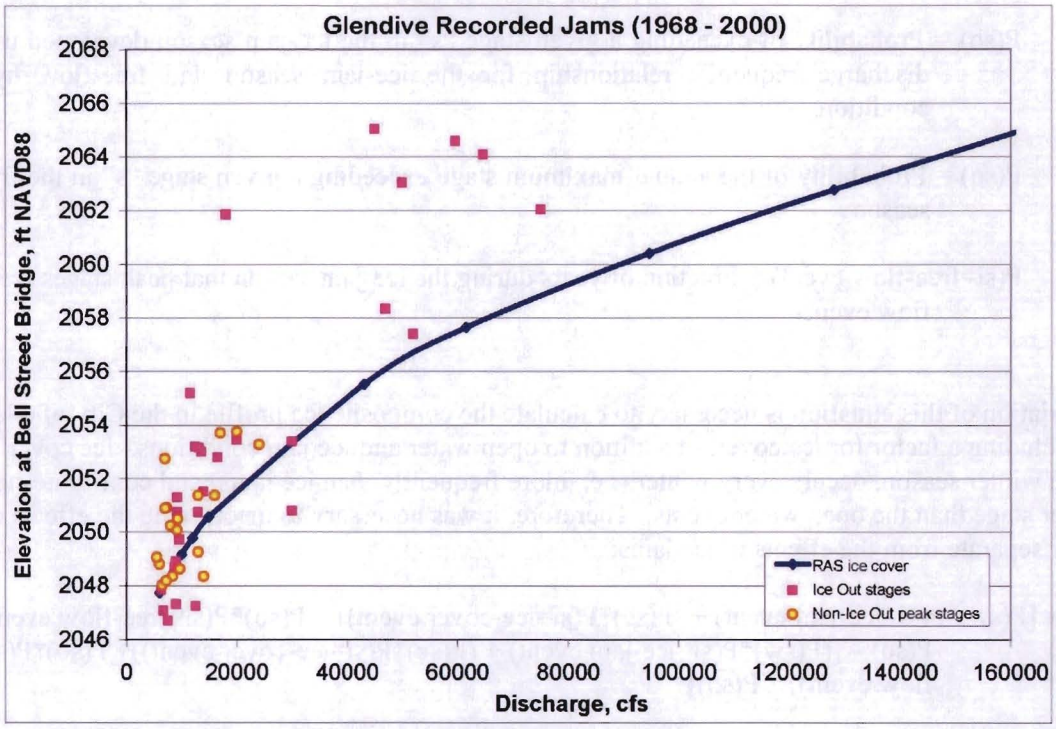

Figure 6. Historical ice-affected stages in Glendive, MT

The probability of an ice jam occurring at and withstanding the forces from flows as large as the 1% chance exceedance event, as modeled in the Glendive area, is unknown, and therefore was not incorporated into the composite ice calculations.

Composite ice stage-probability curves were developed for each cross section within the jam reach to generate a water surface profile (see Figure 8) for each flood event. An Excel spreadsheet was created to compute the composite ice stage-frequency relationship. A resulting composite ice stage-probability curve is shown in Figure 7 for the Towne Street Bridge cross section, with stages reported in meters. For each stage, a probability was calculated based on the corresponding ice cover and ice jam probabilities and then weighted with the jam occurrence probability (i.e., 0.303 for ice jam recurrence and 0.697 for ice cover only recurrence).

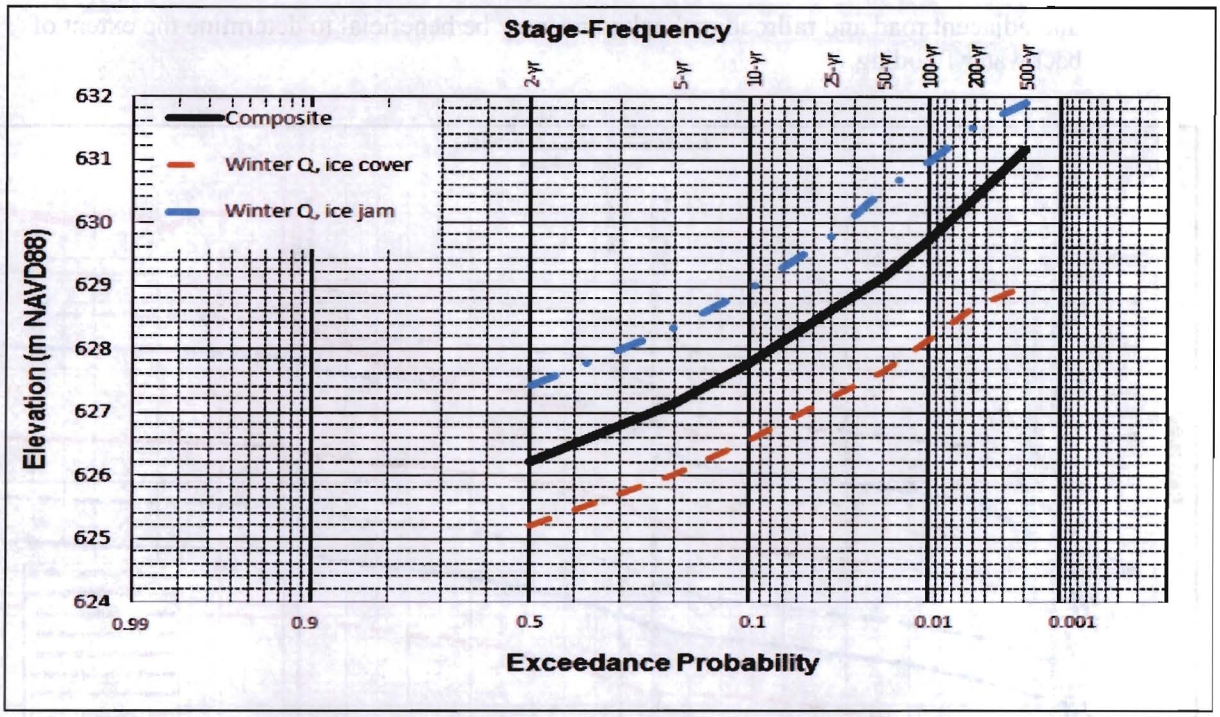

Figure 7. Composite Ice Probability Curves for Towne Street Bridge

5.5. COMPOSITE ICE MODELING RESULTS

The composite stage probability was calculated for each cross section in the City of Glendive area to determine the composite ice profiles for the 50, 20, 10, 4, 2, 1, 0.5 and 0.2% chance exceedance events. Composite ice profiles were only calculated for the Glendive area where water surface elevations for ice jam and ice cover conditions exist. The existing conditions results and project alternatives are described below. The corresponding profiles and flood boundaries for the composite ice profile are shown in Appendix A.

The results from the calculated composite ice stages are shown in Figure 8 for the eight computed profiles. The 50 to 2% exceedance events show a water surface elevation that is consistently below the West Glendive Levee crest. The 1% exceedance event overtops a portion of the levee, and the 0.5 and 0.2% exceedance events overtop the entire levee.

The Cottonwood Grove Levee, located on the right bank of the flood plain just upstream from the BNSF Railroad Bridge, provides protection against the river stages for all modeled profiles. Although the levee shows adequate height for protection of the Cottonwood Grove Community, an indication of backwater flooding across the railroad tracks is apparent. An elevation dip along the railroad embankment is below the composite ice flood stage for the 0.5 and 0.2% exceedance events. Overbank, backwater flooding across the railroad could cause the Cottonwood Grove Community to become inundated. However, flood damage to the Community would depend on the length of time the flood maintained this stage. In addition, terrain data used for the model and mapping consists of one meter contours. Therefore, terrain data may contain an error of ±1.6 feet. A detailed survey of

the levee and adjacent road and railroad embankments may be beneficial to determine the extent of potential backwater flooding.

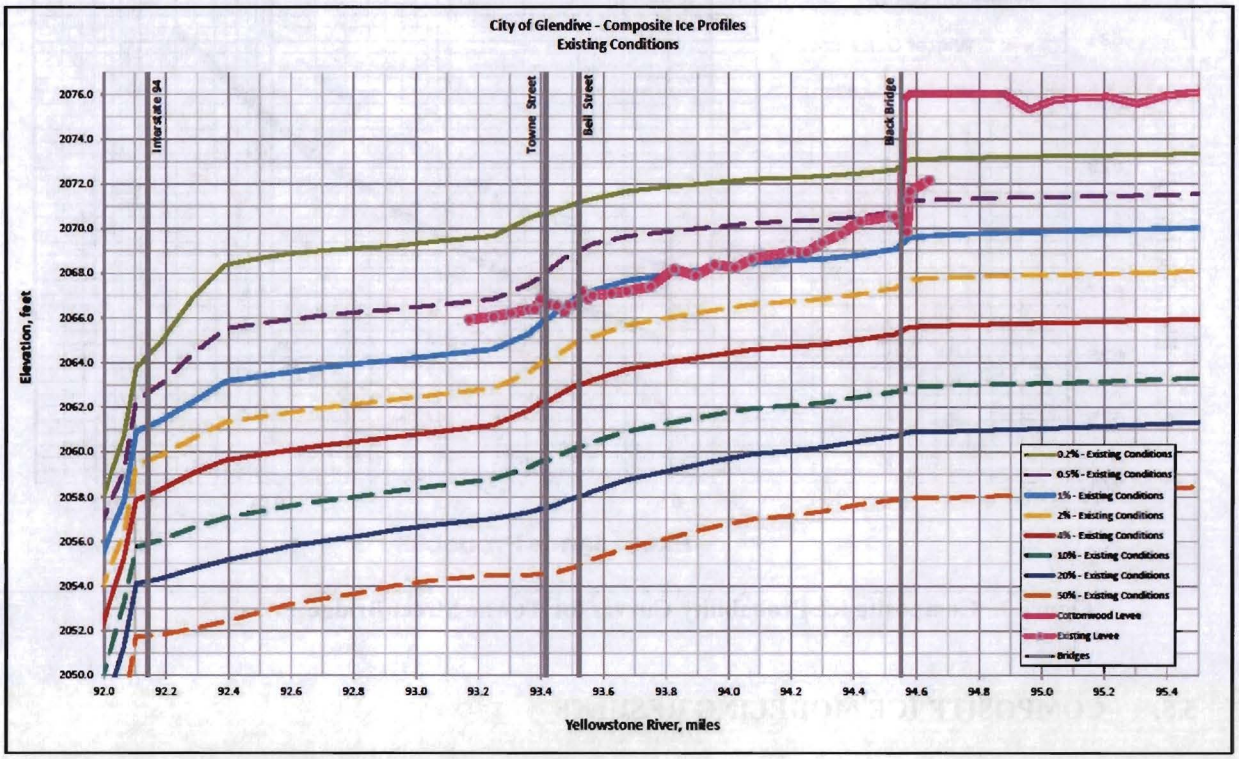

Figure 8. Water Surface Profiles for Existing Conditions

The Flood Insurance Study (FIS) for the City of Glendive, published in 1980 by the Federal Emergency Management Agency, calculated an ice-impacted flood stage. The study data indicate that based on 70 years of record, the 1% chance exceedance event, open-water flood stage was exceeded approximately every 5 years due to ice-impacted flood events. It was determined that a combined stage (open water and ice affected) for the 1% chance exceedance event yielded an elevation of 2068.8 feet NAVD88 at the Bell Street Bridge. Upstream and downstream stages were calculated based on the Bell Street Bridge stage utilizing the Water Surface Profile Program developed by the Bureau of Reclamation.

A comparison of the current HEC-RAS composite ice profile and the 1980 FIS combine ice profile is shown in Figure 9. Results show that at Bell Street Bridge, the HEC-RAS composite ice stage is 1.9 feet less than the FIS stage. At most, the FIS stage exceeds the HEC-RAS composite ice by 2.5 feet. The FIS combined profile remains high downstream of the Interstate 94 Bridge, where the HEC-RAS jam was modeled, since a location of ice jam impacts was not specified in the FIS.

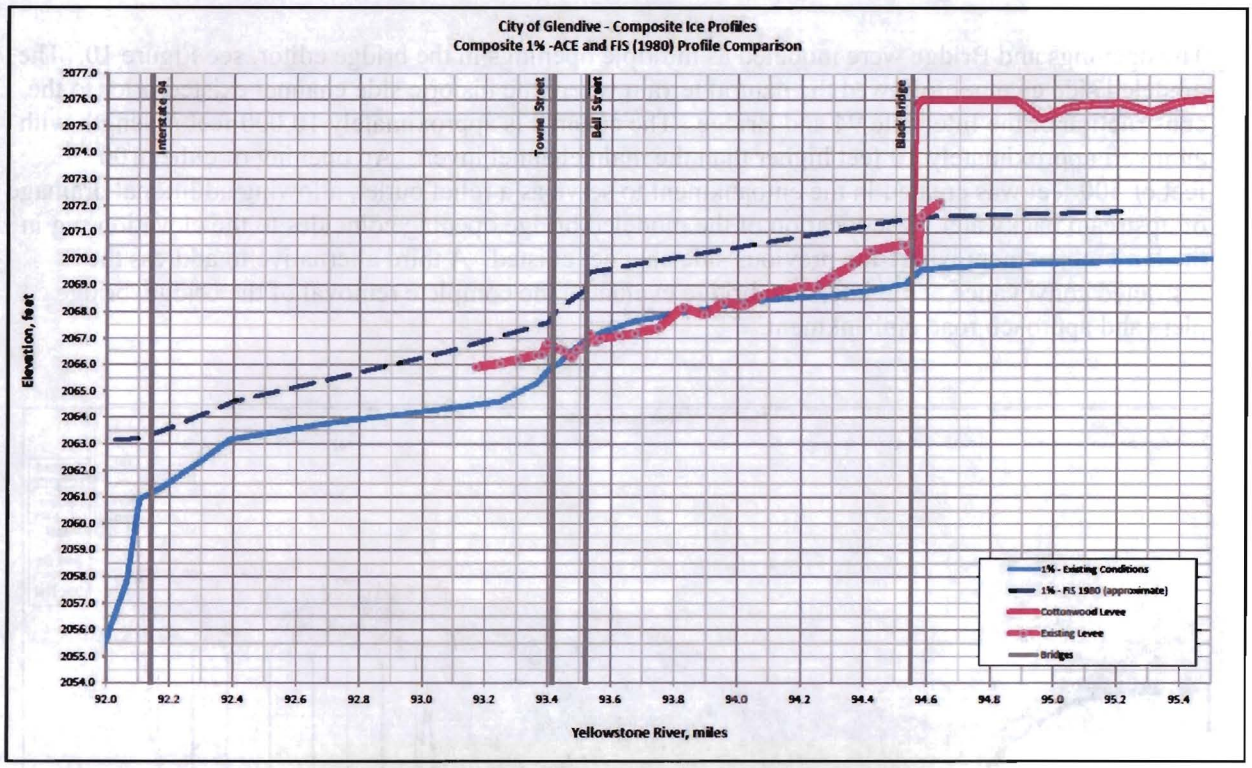

Figure 9. Water Surface Profiles for 1% ACE Composite Ice and FIS Report.

6. CONCEPTUAL MODELS

Alternatives intended to enhance flood protection by increasing conveyance, lowering the water surface elevation and reducing the ice jam impacts on the system were determined, modeled and evaluated. The selected alternatives targeted the levees and bridge crossing, which are the most constricting structures. In addition, raising of the existing levee was considered to provide additional protection without enhancing the river hydraulics. The changes to the structures were incorporated into the conceptual models and are described in the following sections. Corresponding inundation maps and water surface profiles are located in Appendix A through Appendix D.

6.1. INTERSTATE 94 ALTERNATIVE

Ice jams have historically occurred immediately downstream of the Interstate 94 Bridge. The impact of the Bridge on the system was evaluated by modeling variations to the structure that increased conveyance and then comparing the results. The alternative solutions for increasing conveyance at the crossing include incorporating an additional opening on the left overbank, where the Interstate 94 embankment cut off a side channel, at a width of 100ft and 500ft and complete removal of the Interstate approach road embankment.

The openings and Bridge were modeled as multiple openings in the bridge editor, see Figure 10. The modeled side channel followed the natural terrain where the historic side channel existed prior to the construction of the Interstate 94 and Bridge. The channel is approximately 10,000 feet in length with an invert approximately 10 feet higher than the main channel invert. An opening of either 100 feet or 500 feet was created in the embankment to serve as a relief outlet, allowing additional drainage of upstream backwater. The location of the modeled bridge opening coincides to the elevation sag in the left embankment where the previous side channel existed. A third alternative to address the restricted conveyance at the interstate bridge evaluated the complete removal of the bridge, bridge piers and approach road embankment.

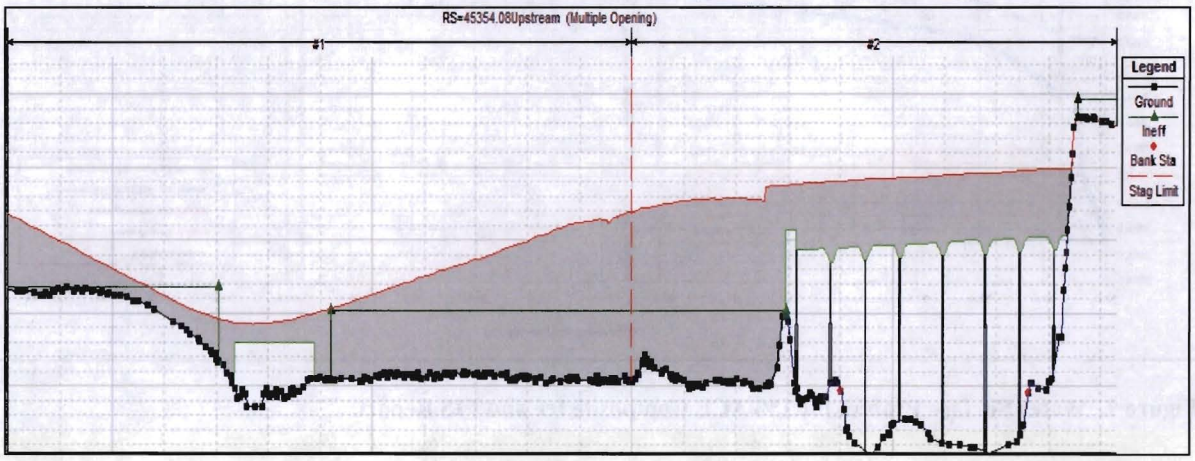

Figure 10. Cross-Section of Interstate 94 with 500-ft Opening

The resulting water surface profiles for the three alternatives for the 1% and 0.2% ACE are shown in Figures 11 and 12 and Appendix A. The increased conveyance yielded a lower water surface between the Interstate 94 Bridge and Towne Street Bridge. However, upstream of Towne Street, the water surface elevation and ice jam thickness remained unchanged. In addition to the lower water surface between I-94 and Towne Street, the model also computed a change in the ice jam shape, as shown in Figures 13 to 16 for the 1% ACE. As the conveyance was increased and the water stage lowered, the ice jam toe became longer.

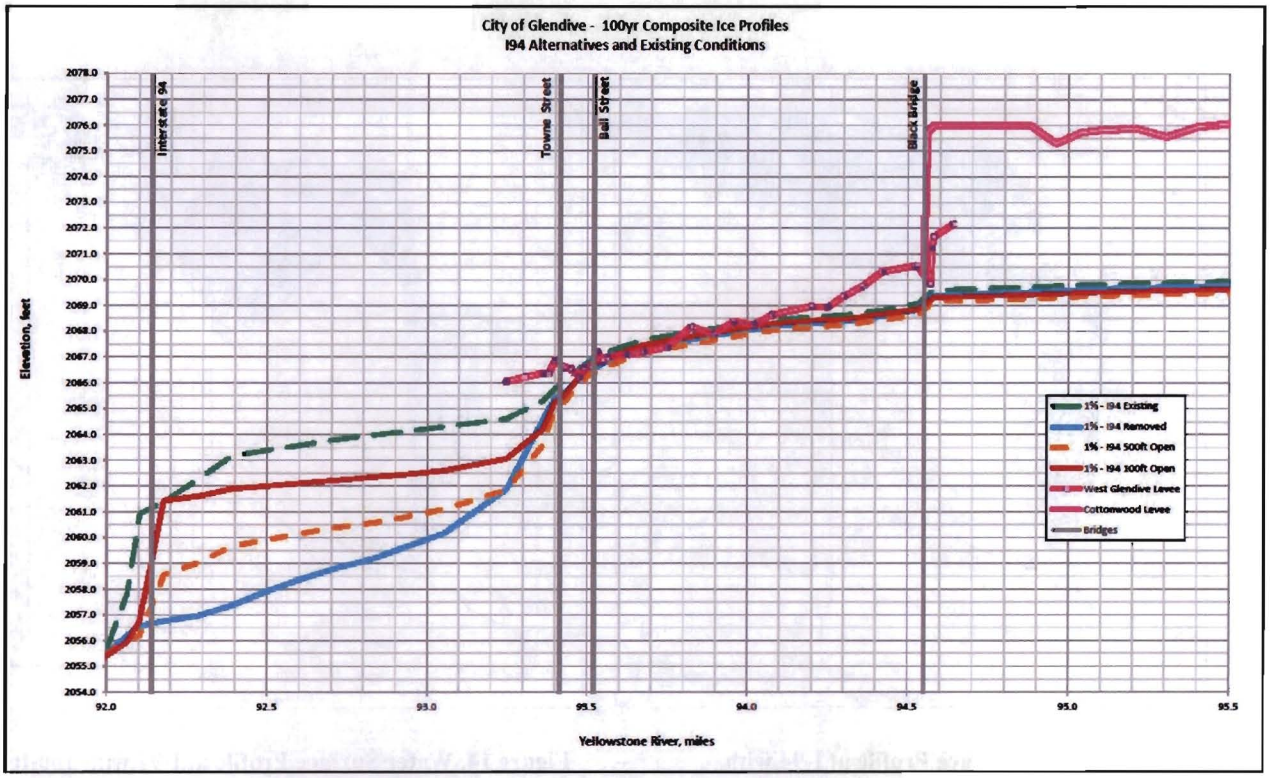

Figure 11. 1% ACE Water Surface Profiles for the Interstate 94 Alternatives.

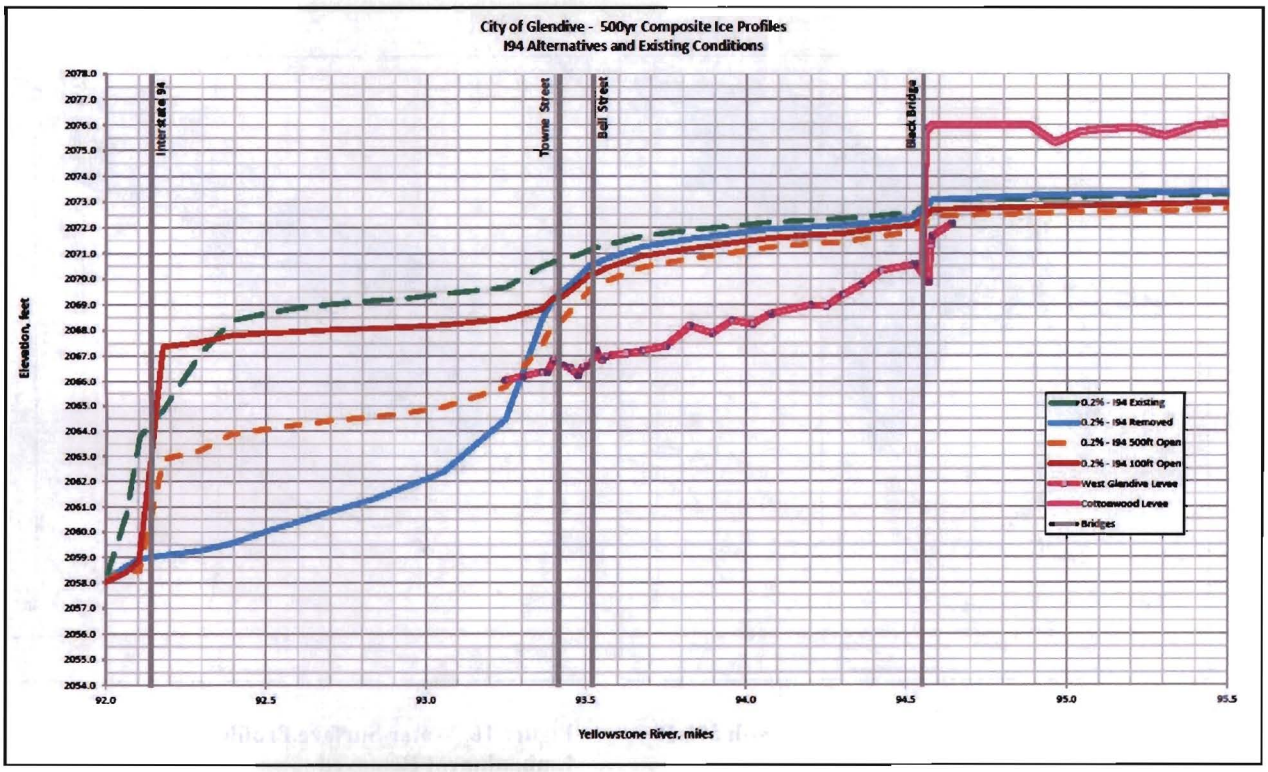

Figure 12. 0.2% ACE Water Surface Profiles for the Interstate 94 Alternatives.

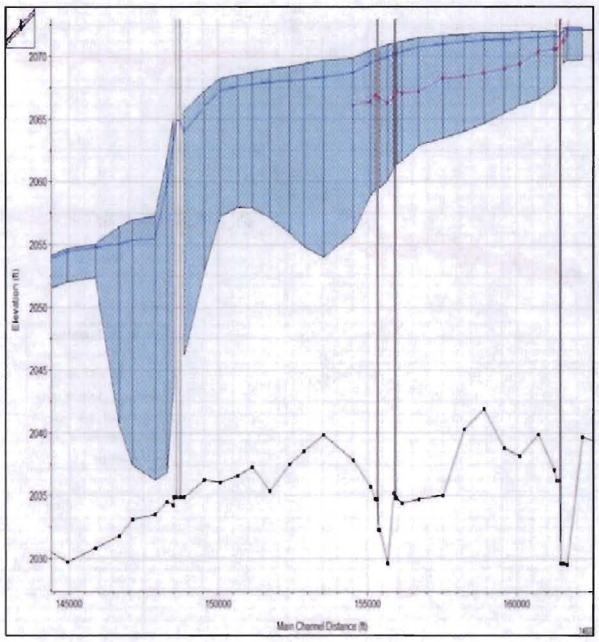

Figure 13. Water Surface Profile at I-94 with Existing Bridge

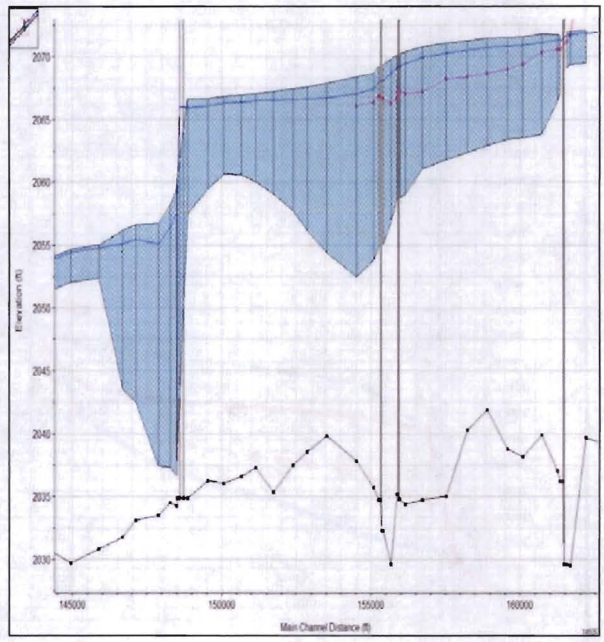

Figure 14. Water Surface Profile at I-94 with 100-ft Opening

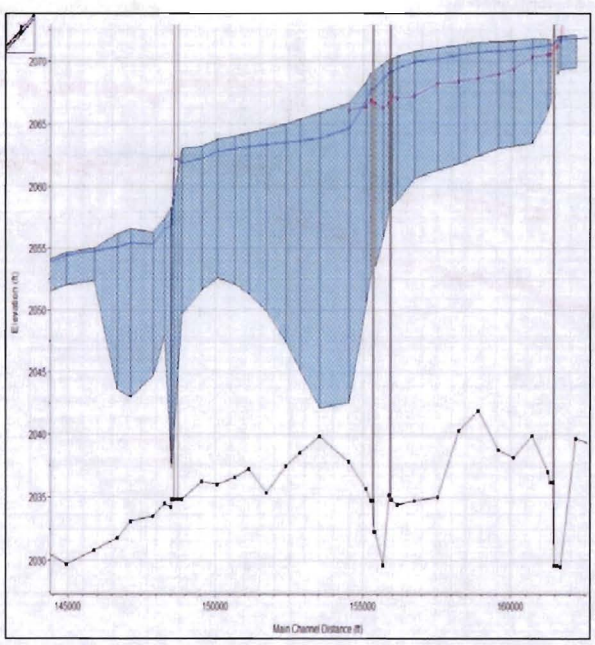

Figure 15. Water Surface Profile at I-94 with 500-ft Opening

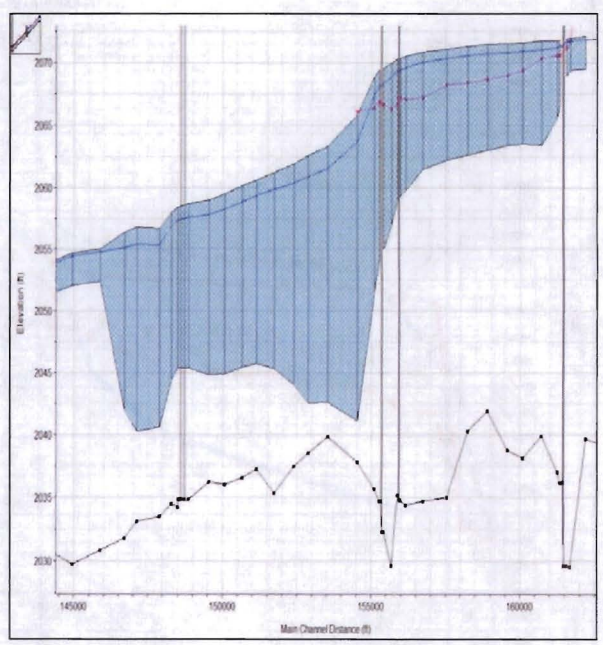

Figure 16. Water Surface Profile at I-94 with Bridge and Embankment Removed

6.2. REMOVED LEVEE

The West Glendive levee features were removed from the model geometry and ineffective flow areas were reconfigured. Manning's n-values were unchanged, reflecting high flow friction due to urbanization. Existing embankments for the road and railway remained in the geometry providing some flow constrictions; however, the scenario assumes inundation of existing roads in West Glendive rendering the roads and bridges inaccessible. Resulting profiles are approximately 1.5-feet lower than current conditions. Water surface profiles are shown in Figure 17 and inundation maps are located in Appendix B.

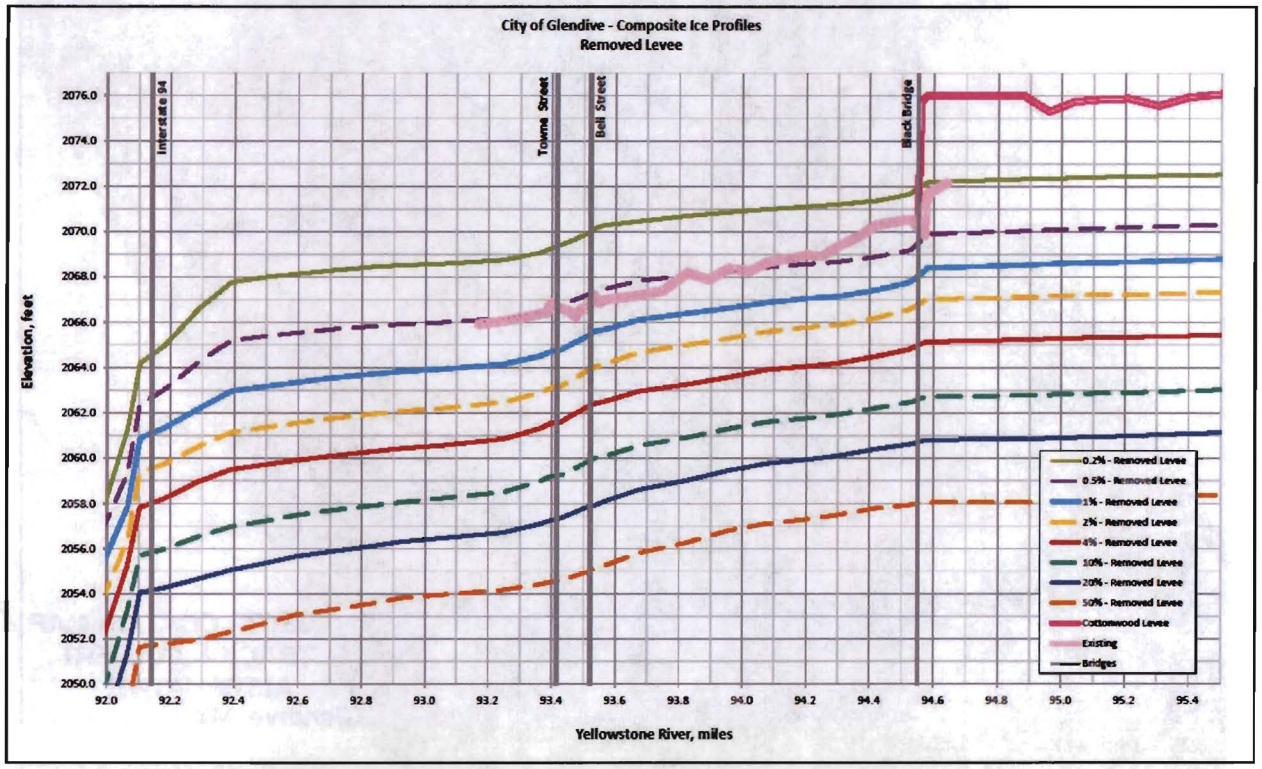

Figure 17. Water Surface Profiles for the Removed Levee Alternatives.

6.3. LEVEE SETBACK AND CULVERTS

The levee setback alternative was selected in an attempt to significantly increase conveyance while still providing protection to much of West Glendive and without removing the bridges or railways. The footprint of the levee setback was determined based on input from the City and from the review of the valuation of the structures protected by the West Glendive Levee. The upstream portion of the setback levee starts at Black Bridge and runs adjacent to the railroad to just above Bell Street where it turns east and runs parallel to Bell Street to Reynold's Market, approximately 500 ft from the current levee, and turns north until it ties into the existing levee. The setback levee placement removes the south east section from protection, see Figure 18. Resulting profiles are shown in Figure 20, and inundation maps are located in Appendix C.

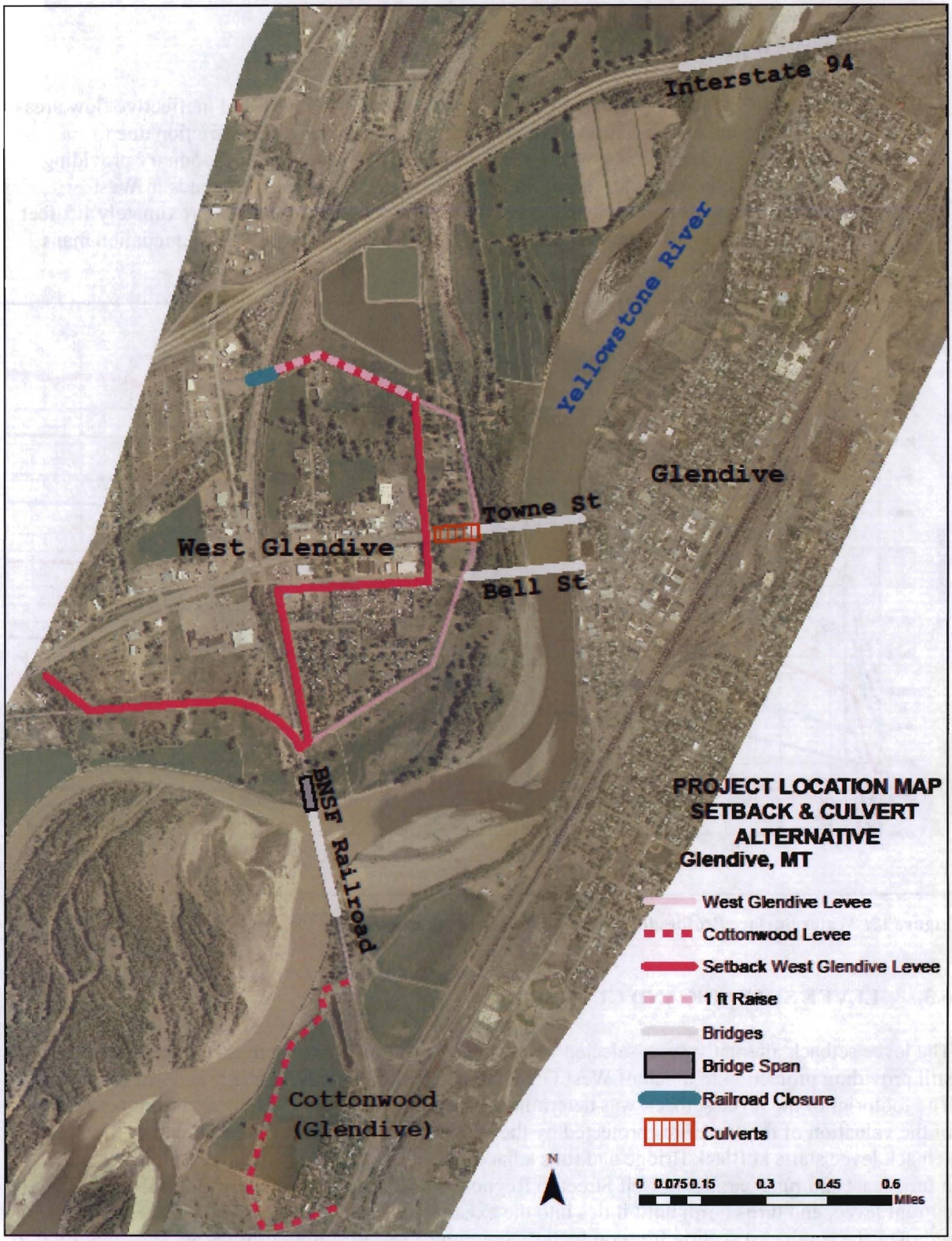

Figure 18. Location Map of the Setback Levee Alternative

In addition to the setback, additional conveyance was achieved by widening the Black Bridge opening. The bridge is a significant constriction point for the river and widening it was necessary to increase conveyance in both the upstream and downstream direction. Therefore, two 150-foot spans were incorporated into the bridge on the left bank.

Similarly, Towne Street Bridge is a constriction point for the river. To increase the benefit of the levee setback, additional conveyance was needed at the bridge. Eighteen large culverts, sized 24 feet wide by 12 feet high were incorporated into the left embankment, between the bridge opening and the setback levee, shown in Figure 19.

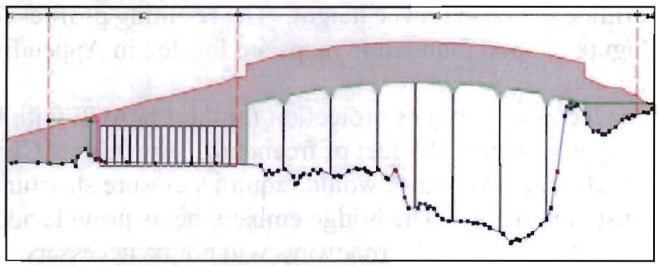

Figure 19. Towne St Bridge with Culverts

The cumulative impacts of the setback levee, bridge widening and bridge culverts resulted in a water surface elevation that is approximately 1.5-ft lower than the current profile for the 1%. Therefore, in addition to widening the Black Bridge, placing culverts through the Towne Street approach road and setting back the levee, a closure structure is necessary at the downstream tie-off across the railroad tracks and a raise of the downstream levee crest of approximately one foot for a length of 2,000 feet.

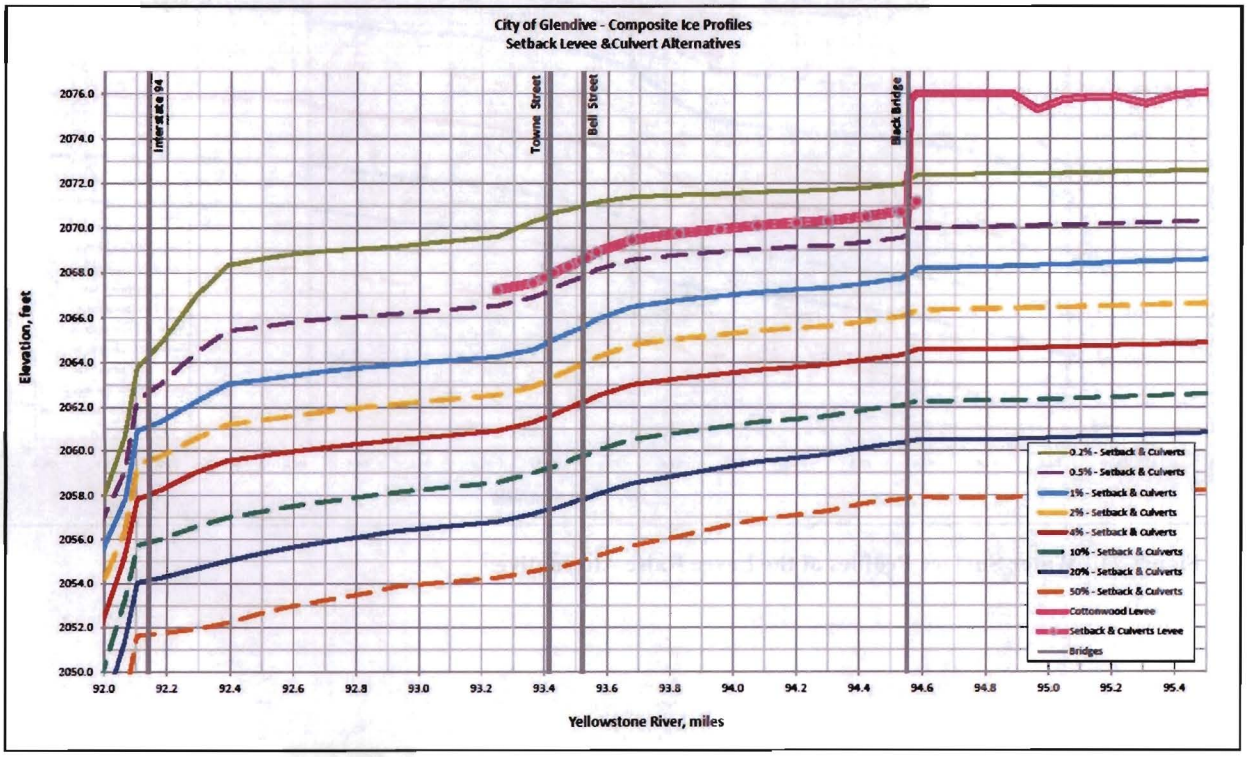

Figure 20. Water Surface Profiles of the Setback Levee Alternative.

6.4. LEVEE RAISE

Modeling of the existing levee yielded water surface elevations that exceeded the existing West Glendive levee for the 1-, 0.5- and 0.2% ACE. A raise of the existing levee was considered. The computed 1% ACE composite ice profile was selected and three feet of freeboard was applied to determine the raised levee height. The resulting profiles for eight frequency events are shown below in Figure 21, and inundation maps are located in Appendix D.

The levee raise provides protection for the 1% ACE with three feet of freeboard and the 0.5% ACE with approximately 0.5 feet of freeboard. The 0.2% ACE overtops the raised levee by as much as two feet. The levee raise would require a closure structure across the railroad tracks near the downstream tie off. The bridge embankments provide adequate height for the raised levee such that closure structures on the roadways will not be necessary.

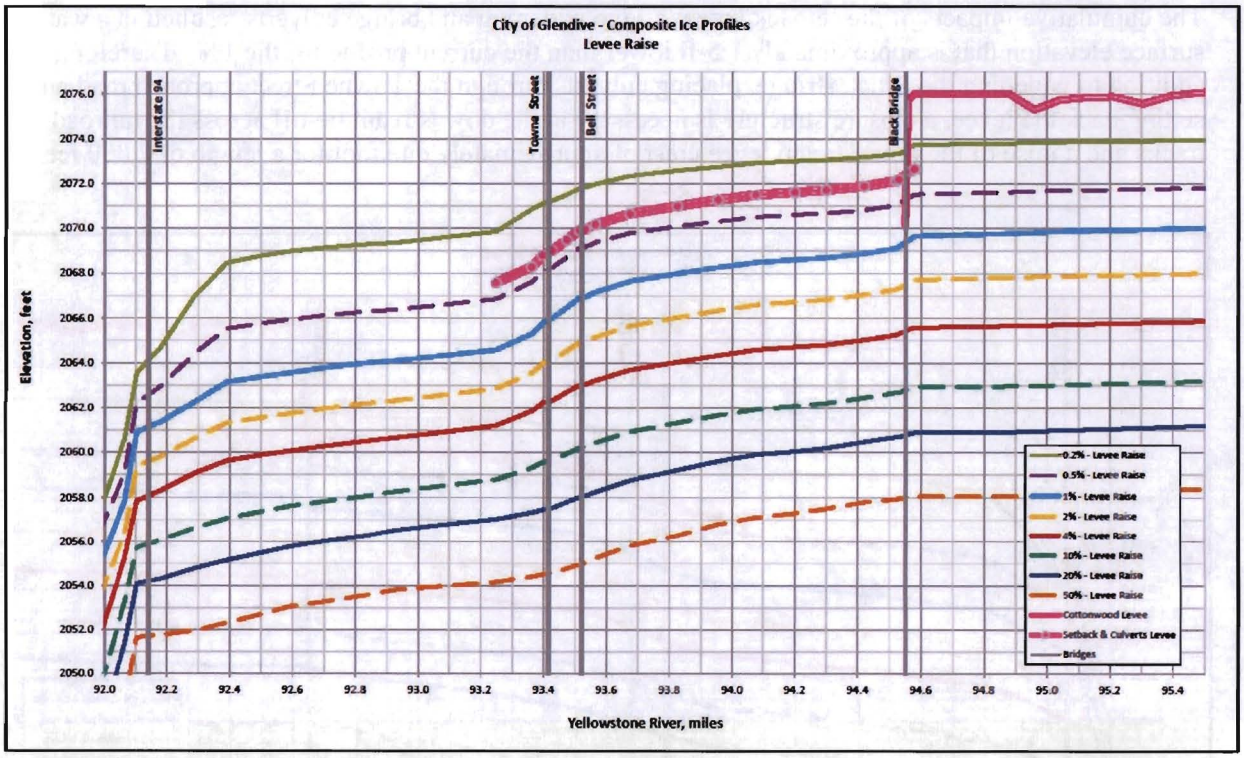

Figure 21. Water Surface Profiles of the Levee Raise Alternative.

7. CONCLUSION

A chart comparing the 1% (100-yr) and 0.2% (500-yr) composite ice profiles of the different scenarios affecting the West Glendive Levee is shown in Figure 22. The existing condition is compared to the removed levee, levee raise and levee setback alternatives. The profiles are plotted against the existing and raised West Glendive Levee, the Cottonwood Grove Levee and the four bridges in Glendive.

The composite ice profiles and inundation maps for the existing conditions and conceptual model runs were computed for the 50, 20, 10, 4, 2, 1, 0.5 and 0.2-percent (%) exceedance probabilities. These hydraulic results were generated as part of the Section 22, Planning Assistance to States (PAS) Study on the flood risk reduction opportunities for Glendive, MT. The profiles and maps support the feasibility determination of potential flood plain alternatives through the evaluation of the flood risks, the depths of flooding, potentially induced damages associated with the flooding, and the level of protection provided a levee alternative.

This hydraulic analysis serves as a comparison of the Glendive area under various changes to the flood plain. If an alternative to the system is pursued, revising the hydraulic ice model to include additional survey data at critical areas of the railroads, bridges and levee and collecting additional ice thicknesses and stages for better calibration is recommended due to the higher level of uncertainty from ice jam modeling.

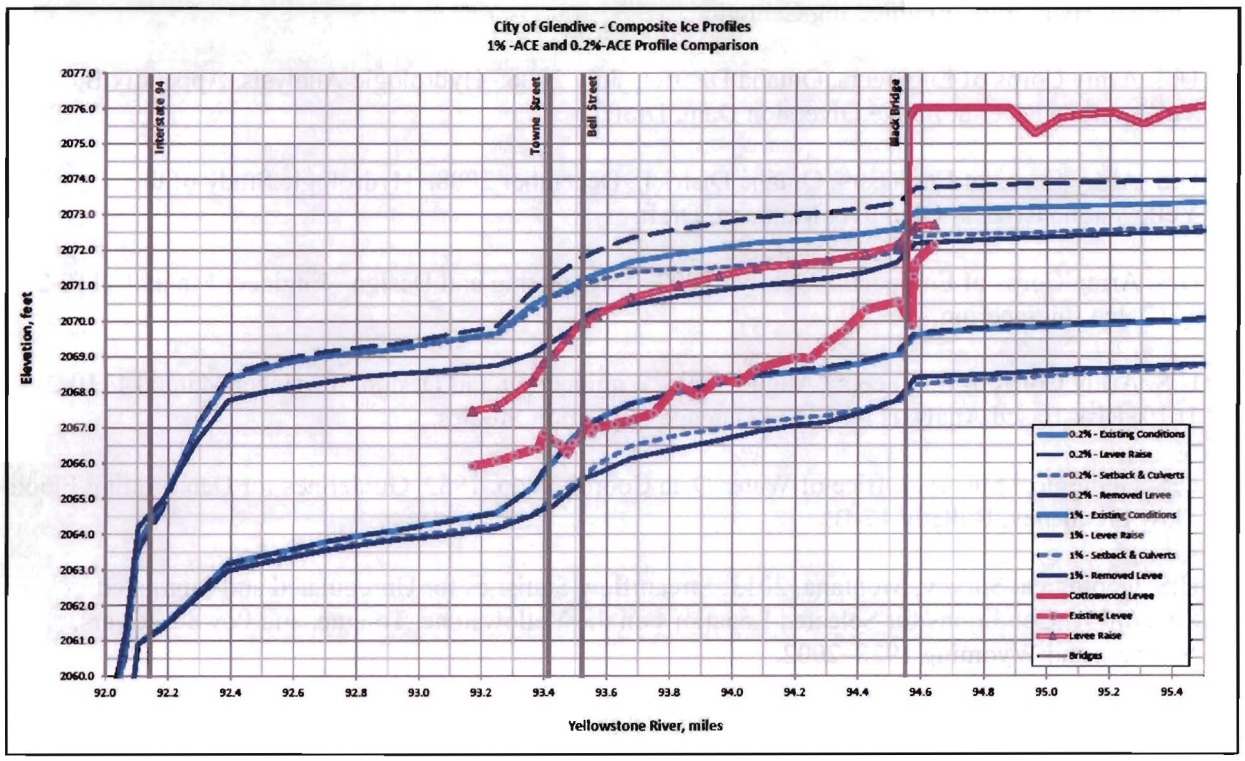

Figure 22. 1% and 0.2% Water Surface Profiles Comparison

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Glendive Nonstructural Assessment

December 2014

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1.0 Introduction

This nonstructural assessment has been conducted in support of the U.S. Army Corps of Engineers (USACE), Omaha District (NWO), to assess structures, consisting of residential and commercial buildings, to identify potential flood risk adaptive measures (FRAM), generally referred to as nonstructural mitigation measures, located along the Yellowstone River in conjunction with a proposed levee raise of the existing left-bank federal levee project at Glendive, Montana. The existing conditions 1% annual exceedance floodplain is shown in Figure 1 with the general location of the assessment area being located along the right-bank and just upstream of the federal levee along the left-bank.

The data collected for this nonstructural assessment is for inclusion in the ongoing NWO Section 22 Study report with the City of Glendive for consideration in mitigating the current flood risk and addressing potential future flood risks associated with these structures.

The entire assessment area contains numerous structures, most of which are privately owned. While not analyzed as part of this assessment, several of the structures may be culturally significant due to their construction techniques, construction materials, and age of the properties to the establishment and evolution of this area.

If flood risks to these structures and others located within the vicinity of the assessment area occur as postulated by the impacts associated with increasing the height of the existing levee, then, the impacted area will be pressed to implement nonstructural FRAM measures, consider the construction of significant barriers to be located along the Yellowstone River, or face losing these buildings to cumulative flood effects as structures and development may be forced to withdraw from the hazard area.

This assessment focuses on potentially at-risk structures and contains the detailed technical assessment used for investigating the incorporation of nonstructural FRAM measures within the assessment area. It is conclusive that without the incorporation of nonstructural flood risk adaptive measures or other measures such as levees and floodwalls, a significant number of structures could be damaged along the right-bank and just upstream from the federal levee under conditions of raising the height of the federal levee and extensive flooding along the Yellowstone River.

While nonstructural FRAM measures are specific to the structure being investigated, when considered for the mitigation of flood damages, the cumulative effect is to determine a strategy for incorporating a full range of nonstructural FRAM measures which are economically feasible and will reduce the cumulative risk of flooding. Each individual structure assessed may require a different nonstructural technique from the structure located next to it. While this assessment relies heavily upon an inventory of data collected in the field for implementation, each structure would be required to be inspected by a team consisting of a floodplain engineer, architect or structural engineer, cost engineer, civil engineer, and real estate specialist in order to determine the mitigation details relative to each type of nonstructural FRAM measure employed. Because of the limited nature of this level of investigation, this assessment was conducted as reconnaissance level detail.

Nonstructural FRAM measures require different implementation methods than structural measures. Since each structure is owned and typically occupied, nonstructural implementation agreements must be entered into with each individual owner.

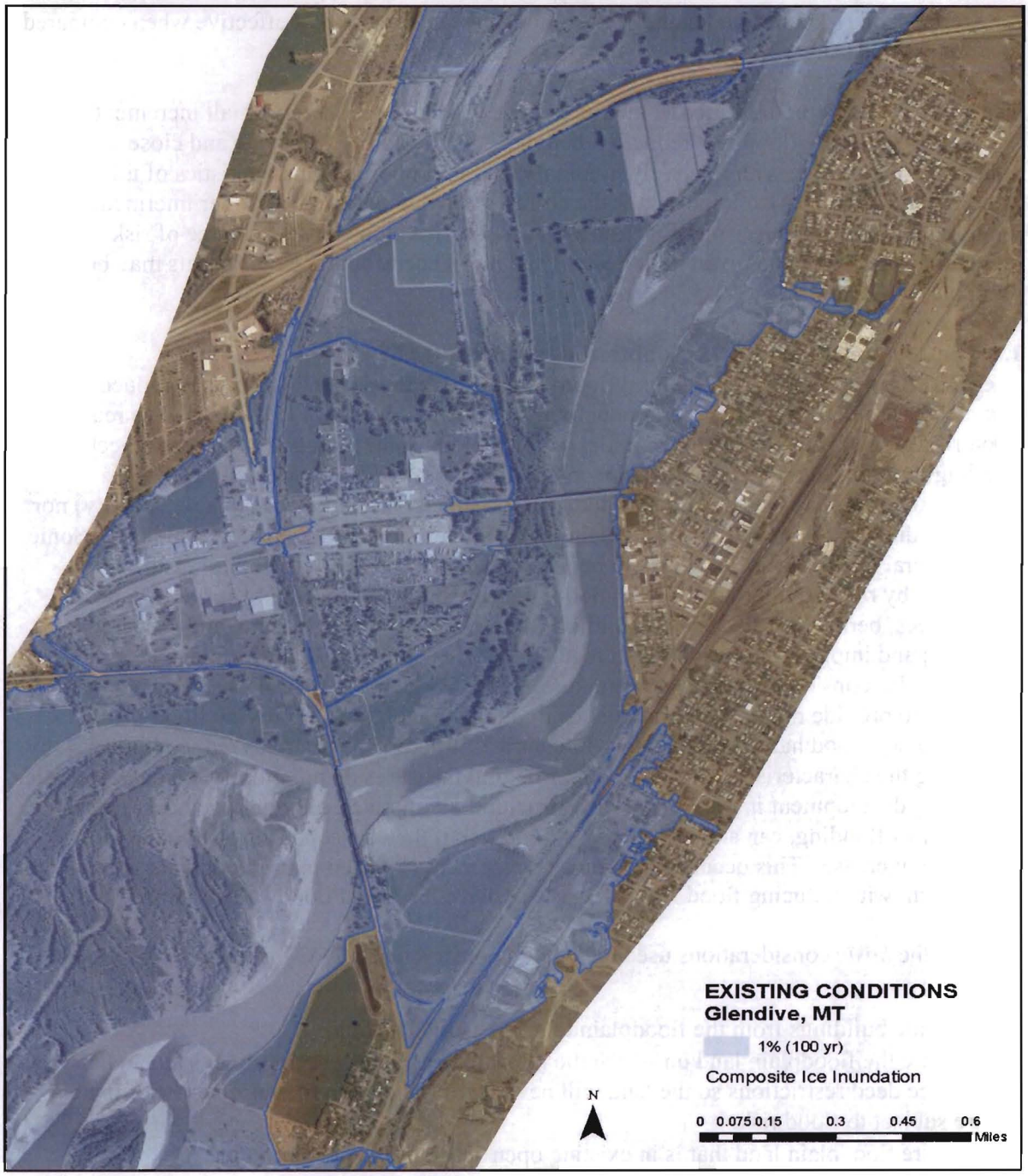

Figure 1
Existing Conditions 1% Annual Exceedance Floodplain

Nonstructural FRAM measures are proven methods and techniques specifically directed at reducing flood risk and flood damages in floodplains. Numerous structures across the nation are subject to reduced risk and damage or no risk and damage due to implementation of nonstructural measures. Nonstructural mitigation measures are very effective for both short and long term flood risk and flood damage reduction and can be very cost effective when compared to structural measures.

The ability of nonstructural FRAM measures to be implemented in very small increments, each increment producing flood risk reduction benefits, and the ability to initiate and close a nonstructural program with relatively minimal costs are important characteristics of this form of flood risk reduction. Also important is the ability to implement measures over intermediate and long periods such that layering of measures, each one providing a higher degree of risk reduction, is possible and given both Federal and non-Federal funding constraints may be probable.

1.1 Nonstructural Flood Risk Adaptive Measures (FRAM)

The overall purpose of a nonstructural flood risk adaptive measure (FRAM) is to reduce flood risk, decrease flood damages, and to potentially eliminate life-loss. FRAM measures reduce flood risk by modifying the characteristics of the buildings and structures that are subject to flooding or modifying the behavior of people living in or near floodplains. In general, nonstructural FRAM measures do not modify the characteristics of floods (stage, velocity) nor do they induce development in a floodplain that is inconsistent with reducing flood risk. Some nonstructural FRAM measures that can be formulated include removing buildings from the floodplain by relocation or acquisition; flood proofing buildings; elevating structures, placing small levees, berms or walls around buildings; implementing flood warning and preparedness activities; and implementing floodplain regulation. The National Flood Insurance Program (NFIP) is also considered among nonstructural flood risk adaptive measures since it contains programs to provide minimum standards for floodplain regulation, to provide flood insurance, and to provide flood hazard mitigation. In contrast, structural alternatives reduce flood risk by modifying the characteristics of the flood. Structural measures do not modify the characteristics of existing development in the floodplain. Structural alternatives, although they decrease the frequency of flooding, can actually increase flood risk if the consequences of flooding are allowed to increase. This occurs when new development is placed in the floodplain that is inconsistent with reducing flood risk, such as extensive levee and floodwall systems.

Some of the *basic* considerations used to develop nonstructural FRAM measures are as follows:

- Relocate buildings from the floodplain to a flood-free location.
- Acquire the floodplain land on which the relocated buildings previously existed and enforce deed restrictions so the land will never be developed in the future for uses that are subject to flood risk.
- Acquire floodplain land that is in existing open space use to prevent future development that could be at flood risk.
- "Acquire" buildings within the floodplain, destroy them, and enforce deed restrictions to prevent future development that could be at flood risk.

- Elevate buildings above a particular flood elevation.
- Dry flood proof buildings (traditional building waterproofing)
- Wet flood proof buildings (retrofitting existing buildings below a design flood elevation with water resistant materials and allowing flood water to easily flow into and out of the building).
- Install small ring levees, berms, and walls around one building or a few buildings that are in close proximity to one another. Such levees, berms, and walls are not to be accredited for the National Flood Insurance Program.
- Install flood warning systems.
- Develop and implement emergency flood preparedness plans.
- Employ communication and educational outreach programs aimed at no flood risk.

Each of these general categories of nonstructural FRAM measures can be applied as a single measure or can be applied in combination with one another or with structural measures to reduce or eliminate flood risk. The range of benefits, costs, and residual damages associated with application of each measure is broad. The extent and severity of social and economic impacts associated with the various measures can be likewise broad and must be identified for any plan. Depending upon the nonstructural measures selected for application and the relative percentage of each applied, the future land use pattern of the area could look considerably different in specific areas.

The consequences associated with locating damageable property and people within floodplain areas can be extreme to property owners and floodplain occupants. Within the context of this assessment, an objective is to identify strategies and measures that can be used in tandem to reduce flood risk. Some strategies and measures may be more appropriate for Federal action while others will be more attuned to local regulatory action and administration. In either case, these measures must be effective, socially acceptable, environmentally suitable, and mindful of the existing neighborhood and community social and economic systems within which they would be implemented. It is the intent of this assessment to identify such nonstructural measures.

1.2 Floodplain and Flood Risk Characteristics

The source of the most major historic damaging floods are ice induced events and riverine processes causing flood problems. Because of the characteristics of ice formation and jamming on the Yellowstone River, flood warning would be favorable to enable human intervention to reduce some structure content flood damages. Because of the riverine flood characteristics, actual flood duration can last from several days up to over a week or longer.

The floodplain within the Glendive nonstructural assessment area consists primarily of residential and commercial development, with a few public structures. Basements and crawl spaces exist in many of the buildings. Age of development is from very old to relatively new.

1.3 Executive Order 11988; Floodplain Management (EO11988)

This Executive Order (EO11988) was issued by President Carter on 24 May 1977. In issuing EO11988 the President stated "in order to avoid to the extent possible the long and short term adverse impacts associated with the occupancy and modification of floodplains and to avoid direct and indirect support of floodplain development wherever there is a practicable alternative,

it is hereby ordered that each agency shall provide leadership and shall take action to reduce the risk of flood loss, to minimize the impact of floods on human safety, health and welfare, and to restore and preserve the natural and beneficial values served by floodplains in carrying out its responsibilities…". The nonstructural FRAM measures contained herein was conducted in complete compliance with EO11988 meaning that any nonstructural FRAM measures that are incorporated into alternatives recommended for implementation support the vision of EO11988.

1.4 Critical Facilities

Structures/facilities which become inoperable during a flood event and result in additional adverse impacts or hardship on the effected population are critical facilities. They are essential during and after a flood to provide human safety, health, and welfare. Critical facilities are generally those services required during the flood such as police and fire protection, emergency operations, people evacuation sites, and medical care. Facilities which house the elderly, disabled, or requiring medical assistance, require extensive evacuation time and would also be considered critical. Facilities that could, if flooded, add to the severity of the disaster such as power stations, waste water treatment plants, and toxic material storage sites are considered critical. Each critical facility within the guidelines of EO11988 should be located at a flood free site. If this is not possible or practicable, the facility should be located external to the 500-year floodplain. If this is not possible or practicable, the facility must be, at a minimum, protected to the extent that it can function as intended during all floods up to and equal to a 500-year event.

1.5 Common Nonstructural Flood Risk Adaptive Measures (FRAM)

The following nonstructural FRAM measures are commonly utilized for reducing flood risk within urban and rural areas across the nation. Each measure must meet specific criteria that would make it acceptable to the flood characteristics and site conditions. Most highly utilized nonstructural FRAM measures are described in detail, not all measures were found to be acceptable for implementation within the Glendive assessment area.

1.5.1 Acquisition and Demolition of the Structure. This measure consists of buying the structure and the associated land as part of the flood mitigation measures. The structure is either demolished or the structure is sold and relocated to a site which is external to the floodplain. Development sites, if needed, can be part of a project in order to have locations where displaced people can build new homes or businesses. This measure is applicable anywhere in the within the study area.

1.5.2 Elevation of Structures. This measure requires lifting the entire structure or the habitable area to be located at an elevation above a particular flood event. If a basement exists and had been fully developed prior to elevation and could not be developed post-elevation, partial compensation of the basement space would be in order to the owner. This measure is applicable anywhere within the study area unless the required elevation is greater than a maximum of 12 feet above the adjacent grade. Velocity and hydrodynamic forces on the structure would also have to be considered. An example is shown in Figure 2.

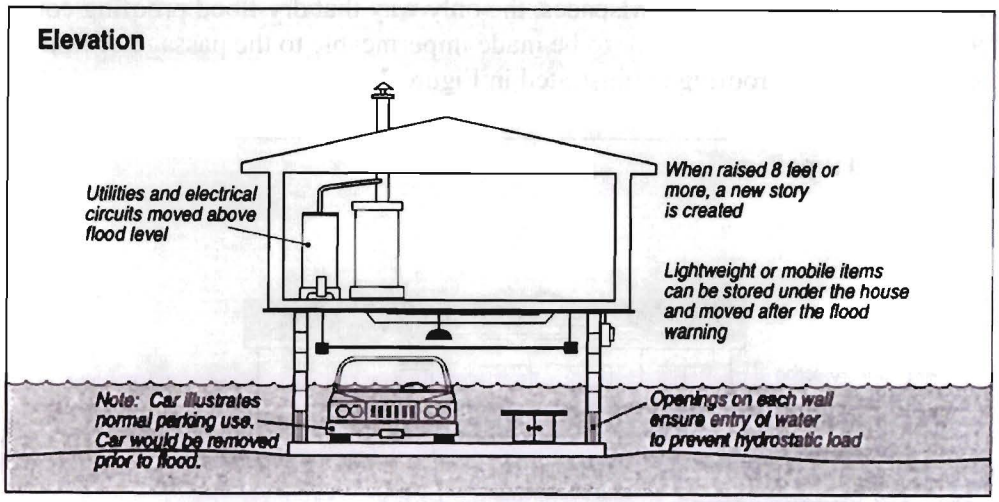

Figure 2
Structure Elevation (Diagrammatic Section)

1.5.3 Removal of Basement. This measure consists of filling in the existing damageable basement without elevating the remainder of the structure. This would occur if the structures' first floor was located above the Base Flood Elevation (BFE) or above the design elevation or whichever is higher. With this measure, placing an addition onto the side of the structure as part of the project to compensate for the lost basement space to the owner should be considered. If the addition could not be developed because of limited space within the property parcel or because the owner did not want it, partial compensation for the lost basement space would be in order to the owner. Hydrodynamic forces on the structure would also be a consideration.

1.5.4 Relocation of Structures. This measure requires physically moving the at-risk structure and buying the land upon which the structure is located. This measure achieves a high level of flood risk reduction when structures can be relocated from a high flood hazard area to an area that is located completely outside of the floodplain. Development of relocation sites where structures could be moved to achieve the planning objectives and retain such aspects as community tax base, neighborhood cohesion, or cultural and historic significance can be considered as part of any relocation project. This measure could be applicable anywhere within the study area.

1.5.5 Dry Flood Proofing. This measure basically consists of waterproofing the structure. This can be done to residential homes as well as other types of structures. This measure achieves flood risk reduction benefits but it is not recognized by the NFIP for any flood insurance premium rate reduction if applied to residential structures. Based upon testing, a "conventional" built structure can generally be dry flood proofed up to 3 feet on the walls. A structural analysis of the wall strength would be required if it was desired to achieve higher protection. A sump pump and perhaps French drain system may be required as part of the project. Closure panels are required for all openings. This concept does not work with basements or crawl spaces due to the possible long duration of flood and the probability of floodwaters entering the lower level. Hydrodynamic forces should also be a consideration. For

buildings with basements and/or crawlspaces, the only way that dry flood proofing could be considered to work is for the first floor to be made impermeable to the passage of floodwater. An example of dry flood proofing is illustrated in Figure 3.

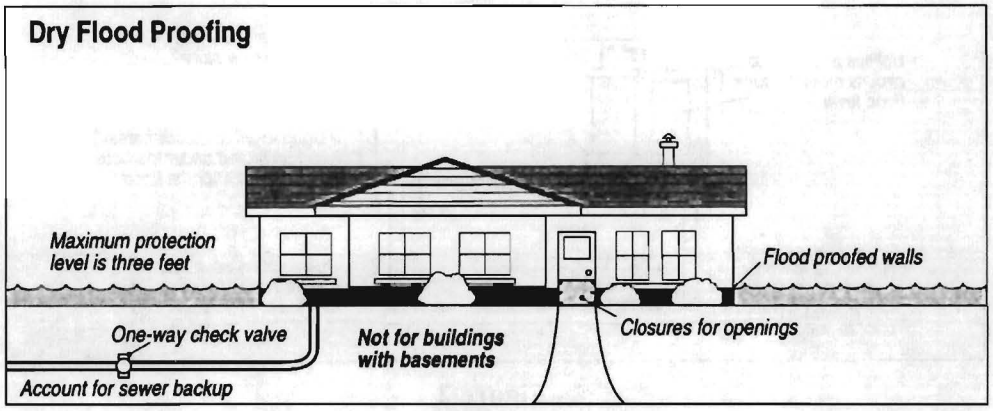

Figure 3
Dry Flood Proofing (Diagrammatic Detail)

1.5.6 Wet Flood Proofing. This measure is applicable as either a stand-alone measure or as a measure combined with other measures such as elevation. As a stand-alone measure, all construction materials and finishing materials are required to be water resistant. All utilities must be elevated above the design flood elevation. Because of these requirements, wet flood proofing of finished residential structures is generally not recommended. Wet flood proofing is quite applicable to commercial and industrial structures and should be considered for being combined with a flood warning, flood preparedness, and flood response plan. This measure is generally not applicable to large flood depths and high velocity flows. An example of wet flood proofing is shown in Figure 4.

Figure 4
Wet Flood Proofing (Diagrammatic Detail/Section)

1.5.7 Berms, Levees, and Floodwalls. This measure could applicable to several locations within the assessment area. As nonstructural measures, berms, levees, and walls should be constructed to no higher than 5 feet above grade and are not certifiable for the National Flood Insurance Program (NFIP), meaning that flood insurance and floodplain management requirements of the NFIP are still applicable within the protected area. These nonstructural measures are intended to reduce the frequency of flooding but not eliminate floodplain management and flood insurance requirements. These measures can be used for all types of structures located within the study area. They can be placed around a single structure or a small group of structures. With application of these measures to be nonstructural, they should not adversely impact the water surface elevation of the 100-year flood by any more than 0.00 feet. An example of berms, levees, and floodwalls is illustrated in Figure 5.

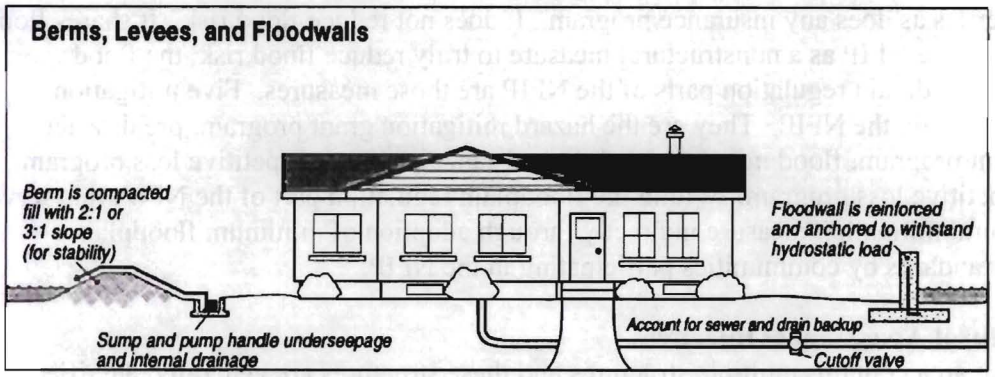

Figure 5
Berms, Levees, Floodwalls (Diagrammatic Detail)

1.5.8 Flood Warning, Preparedness, Evacuation Plans and Pertinent Equipment Installation. These measures are applicable to the entire assessment area. Any nonstructural plan should consider the development and implementation of flood warning systems and emergency preparedness planning. The development of such plans and the installation of pertinent equipment such as data gathering devices (rain gages, stream gages) and data processing equipment can become an integral feature of a project.

1.5.9 Land Acquisition. Land acquisition can be in either the form of fee title or permanent easement with fee title. Land use after acquisition is open space use via deed restriction that prohibits any type of development that can sustain flood damages or restrict flood flows. Land acquired as part of a nonstructural project can be converted to a new use such as ecosystem restoration and/or recreation that is open space based such as trails, shoreline access, and interpretive markers. Conversion of previously developed land to open space means that infrastructure no longer needed such as utilities, streets, and sidewalks can be removed as part of the project. The conversion to new use (ecosystem restoration and/or recreation) can also be part of a nonstructural project. By incorporating " new uses of the permanently evacuated floodplains" into the nonstructural flood risk reduction project, economic feasibility of the buyout or relocation projects is enhanced due to transfer of some flood risk reduction costs to ecosystem restoration and by adding benefits and costs of recreation.

1.5.10 Floodplain Regulation and Floodplain Management. Floodplain regulation and floodplain management have proven to be very effective in reducing flood risk and flood damage. The basic principles of these tools are founded nationally in the NFIP which requires minimum standards of floodplain management and floodplain regulation for those communities that participate in the NFIP. While the minimum standards have not resulted in substantial flood risk reduction, incorporation of more stringent building codes and zoning ordinances may meet community objectives of no flood risk.

1.5.11 National Flood Insurance Program (NFIP). The NFIP contains 3 basic parts; flood insurance, flood mitigation, and floodplain regulation. In terms of reducing flood risk, only flood mitigation and floodplain regulation have a direct impact in theory. In regard to the flood insurance part of the NFIP, flood insurance simply allows spreading the flood risk across multiple properties as does any insurance program. It does not reduce flood risk. It shares flood risk. In terms of the NFIP as a nonstructural measure to truly reduce flood risk, the flood mitigation and floodplain regulation parts of the NFIP are those measures. Five mitigation programs exist within the NFIP. They are the hazard mitigation grant program, pre disaster mitigation grant program, flood mitigation public assistance program, repetitive loss program, and severe repetitive loss program. Within the floodplain regulation part of the NFIP, this serves as a nonstructural mitigation measure indirectly through adoption of minimum floodplain management standards by communities participating in the NFIP.

2.0 Nonstructural Assessment Objectives

The assessment area contains multiple structures and these structures are generally classified as residential, commercial, and public (government). For a nonstructural assessment, each structure must be examined for purposes of what type of nonstructural measure is most appropriate for that particular structure given what it is, where it is located within the floodplain, what the flood characteristics are (velocities and stages), and other site conditions. A 1% (100-year) exceedance flood event was considered as the benchmark for implementation of FRAM measures to mitigate the at-risk structures.

This assessment conducted an investigation of 122 structures, which are detailed later in this report. Specific structure information was collected in the field, combined with additional study information, and used to develop the nonstructural recommendations.

2.1 Description of Nonstructural Structure Dataset

For the nonstructural FRAM assessment, structure information was collected along the left- and right-bank of the Yellowstone River at Glendive for the area impacted by a left-bank levee raise. A list of the types of structures assessed is summarized in Table 1.

Table 1
Structures Affected by Levee Raise

Structure Type	Left-Bank Structures	Right-Bank Structures
Residential	10	96
Commercial	0	14
Public	0	2

3.0 Description of Nonstructural Flood Risk Adaptive Measures Assessment

Limited detailed field data was collected and a desktop assessment was conducted for each of the 122 at-risk structures. Details regarding structure characteristics, site conditions, site elevations and flood elevation data were prepared for each structure from which the assessment is based. Observations of structure characteristics were noted and compared to the hydrologic conditions to determine potential flood risk, from which nonstructural FRAM measures, or those measures which adapt to the characteristics of the floodplain, could be identified and recommended.

Each of the structures was assessed using a similar format for each. The recommendations include measures described in section 1.5 of this report, and are described as individual measures, or in combination with other measures to provide the most effective level of flood risk reduction.

While USACE will try to recommend measures which support the National flood Insurance Program (NFIP), the ultimate goal is to identify economically feasible and environmentally acceptable solutions, which ultimately reduce flood damages and flood risk.

3.1 Nonstructural Flood Risk Adaptive Measures Assessment Guidelines

This assessment established specific guidelines for recommendation of nonstructural flood risk adaptive measures. The 1% annual Chance flood event (100-year) was utilized due to its importance in floodplain management as the flood stage to which the assessment was conducted. A description of where and how those guidelines are used in this assessment follows.

3.1.1 Floodway. Under no circumstances will USACE consider any flood risk reduction activity for a structure located in the regulated floodway, other than relocation or acquisition. While flood proofing or elevation could substantially reduce potential flood damages, life risk would increase, where as there would be the possibility of loss to the structure inhabitants or to first responders during a major flood event.

3.1.2 Flood Proofing. This measure was considered for all structure types, when the depth of flooding was four feet or less for the 1% annual chance flood event. Dry flood proofing of commercial structures may result in a flood insurance rate decrease when conducted in association with NFIP requirements. While dry flood proofing may reduce future flood damages from occurring, there is no flood insurance rate decrease when applied to residential structures.

3.1.3 Elevation. For all structures, commercial, residential, or public, where the structure type, compared to the depth of flooding would allow for elevation of the structure, that measure was considered. Elevation on the appropriate foundation was considered to a depth of up to eight feet for the1% annual chance flood event. For greater depths, the structure was considered only for relocation or acquisition.

3.1.4 Acquisition and Buy-out. This measure was considered for all structure types depending upon location, within or near the regulatory floodway, and depth of flooding for the 1% annual chance flood event, where the depth would be greater than eight feet.

3.1.5 Remove Basement. This measure consists of removing the basement or crawl space by demolition of habitable space and furnishings, then drilling holes into floor to eliminate hydrostatic pressure between the interior and exterior of the structure. A clean run sand or pea gravel material is placed into the basement and crawl space then covered with a concrete cap. If the property parcel has sufficient room, a small addition is placed on the addition above the 1% annual chance flood elevation. The addition would contain utilities and appliances such as the furnace, water heater, water softener, washer and dryer.

4.0 Nonstructural Assessment

This nonstructural flood risk adaptive measures assessment was conducted at a reconnaissance level of detail considering the 1% annual chance flood event and those stages and their potential flood impact on each structure. In order to develop a detailed assessment and cost estimates for the structures analyzed, additional and specific information would be required for each structure. A detailed assessment would require a team of engineers potentially consisting of hydraulic, mechanical, electrical, geotechnical, and structural, as well as an architect to possibly investigate the exterior and interior of each structure.

From the field data collected, a review of all 122 structures was conducted, where it was determined that 25 of the structures had no impact by flood waters. In other words, these structures were not damaged by the 1% annual chance flood event. Table 2 indicates the structures located along either the left-bank or right-bank and those nonstructural flood risk adaptive measures considered.

Table 2
Nonstructural Measure Identified by Streambank Location

Technique	Left-Bank Structures	Right-Bank Structures
No Action Required	2	23
Acquisition	2	4
Dry Flood Proofing	0	3
Elevation	0	8
Remove Basement	4	63
Elevate/Remove Basement	2	11
Total Structures	10	112

The 122 structures within the inventory were also categorized by structure type; residential, commercial, and public/government, which are summarized in Table 3. The table indicates the total number of nonstructural flood risk adaptation measures applied to the three types of structure categories. Note, three of the four residential structures listed as acquisition are outbuildings and one is a residence.

Table 3
Nonstructural Measure Identified by Structure Type

Technique	Residential	Commercial	Public
No Action Required	14	9	2
Acquisition	4	1	1
Elevation	8	0	0
Remove Basement	55	12	0
Elevate/Remove Basement	13	0	0
Dry Flood Proofing	0	3	0
Total Structures	94	25	3

Table 4 illustrates the reconnaissance level costs developed for each nonstructural technique and reported by streambank. These costs do not consider any boundaries (political, cultural) other than split between left-bank and right-bank.

Table 4
Nonstructural Measure Cost Estimate by Streambank Location

Technique	Left-Bank Structures	Right-Bank Structures
No Action Required	-	-
Acquisition	$74,040	$1,388,590
Elevation	$0	$622,360
Remove Basement	$83,790	$1,134,720
Elevate/Remove Basement	$272,090	$644,630
Dry Flood Proofing	-	$211,160
Total Estimated Costs	$429,920	$4,001,460

Table 5 illustrates the reconnaissance level costs developed for each nonstructural technique and reported by structure category (residential, commercial, and public). These costs do not consider any boundaries (political, cultural, and geographic). Total cost is approximately $4,431,380.

Table 5
Nonstructural Measure Cost Estimate by Structure Type

Technique	Residential	Commercial	Public
No Action Required	-	-	-
Acquisition	$342,090	$30,490	$1,090,050
Elevation	$622,360	-	-
Remove Basement	$988,810	$229,690	-
Elevate/Remove Basement	$916,730	-	-
Dry Flood Proofing	-	$211,160	-
Total Estimated Costs	$2,869,990	$471,340	$1,090,050

4.1 Nonstructural Assessment Economic Feasibility

This nonstructural flood risk adaptive measures assessment was completed at a reconnaissance level of detail, which was based upon a gross inventory of structures. The assessment was also based upon the 1% annual chance flood stages associated with the federal levee raise alternative, as well as those flood damages resulting from the existing conditions without levee raise. In other words this assessment considered all potential damages for the without- and with-levee raise conditions. Table 6 illustrates a summary of economic results for the assessment which indicate that further detailed analyses may be supported by the amount of annual benefits derived from implementing nonstructural measures to address damages through the 1% annual chance flood event. A 50 year repayment period and the FY2014 federal discount rate of 3. 5% were used to estimate annual cost, which is compared to annual benefits to estimate net annual benefits and the benefit to cost ratio.

Table 6
Nonstructural Assessment Economic Results

Annual Benefits	$177,140
Annual Costs	$188,820
Net Annual Benefits	-$7,440
Benefit to Cost Ratio	0.94

It is important to note that this assessment did not incorporate contingency factors for estimated costs such as real estate costs or proposed nonstructural measure costs, nor did it consider all potential benefits, which could result in different net benefits conducted through a detailed analysis. It is also important to note that of the 122 structures contained within the structure inventory, 97 structures were identified as being at flood risk from the 1% annual chance flood event. While some structures had a positive benefit to cost ratio, some structures were not

feasible on their own accord, but were included in the cumulative assessment. Further analyses may consider geographical, political, or cultural boundaries which could result in variations to the economic analysis.

5.0 Glossary of Terminology

The following Glossary of Terminology is provided as a tool for becoming more familiar with general flood risk management, floodplain management, and nonstructural flood risk adaptive measures terminology.

100-year flood – The 1% annual chance exceedance (ACE) expressed as a return period.

Annual chance exceedance flood – The flood that has a (stated percent - %) chance of being exceeded in any given year, such as the 1% annual chance exceedance (ACE) flood.

Berms, Levees, Floodwalls - Freestanding structure(s) located adjacent to a structure that prevents the encroachment of floodwaters, as illustrated in Figure 6.

Figure 6
Berm/Floodwall Examples

Breakaway Panel – A panel designed and constructed to collapse under water loads without causing collapse, displacement, or other structural damage to a structure's bearing walls or supporting foundation system.

Closures / Shields - Closures act to close the openings in flood barriers and prevent water from entering, as illustrated in Figure 7. They can be of a variety of shapes, sizes, and materials. In some cases closures are permanently attached using hinges so that they can remain open when there is not flood threat. They may also be portable, normally stored in a convenient location and slipped into place when a flood threatens.

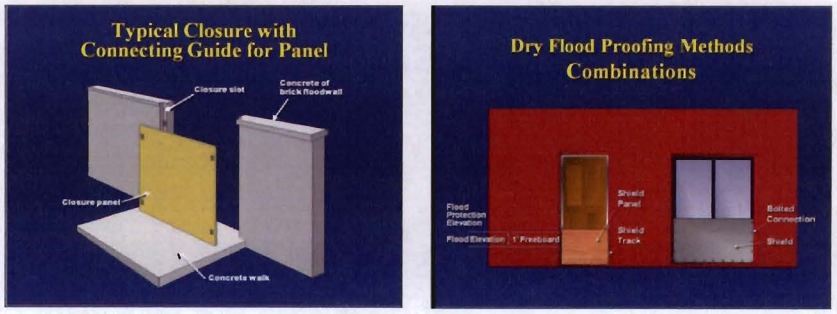

Figure 7
Closures / Shields (Detail)

Community – Any state or area or political subdivision thereof, or any Indian tribe or authorized tribal organization, or Alaska Native village, or authorized native organization that has the authority to adopt and enforce flood plain management regulations for the areas within its jurisdiction.

Consequences (of inundation) - The effect, result, or outcome of inundation/flooding as reflected in the potential loss of life, economic losses, and adverse social - environmental impacts.

Dry Flood Proofing involves temporary or permanent sealing building walls with waterproofing compounds, impermeable sheeting, or other materials to prevent the entry of floodwaters into damageable structures. Dry flood proofing is applicable in areas of shallow, low velocity flooding. Examples are illustrated in Figures 8 and 9.

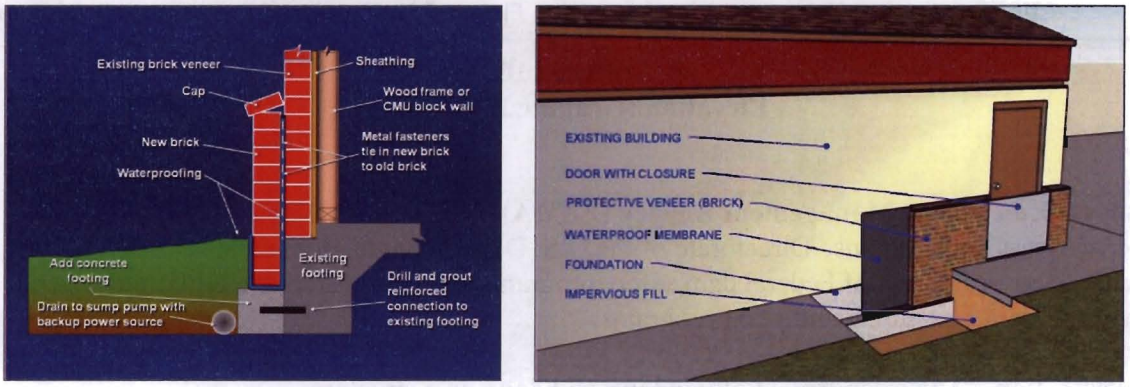

Figure 8
Dry Flood Proofing (Details)

Figure 9
Temporary Dry Flood Proofing (Detail/Example)

Elevation- involves raising the buildings in place so that the structure has a reduction in frequency and/or depth of flooding during high-water events. Elevation can be done on fill, foundation walls, piers, piles, posts or columns. Selection of proper elevation method depends on flood characteristics such as flood depth or velocity. Example of elevation shown in Figure 10.

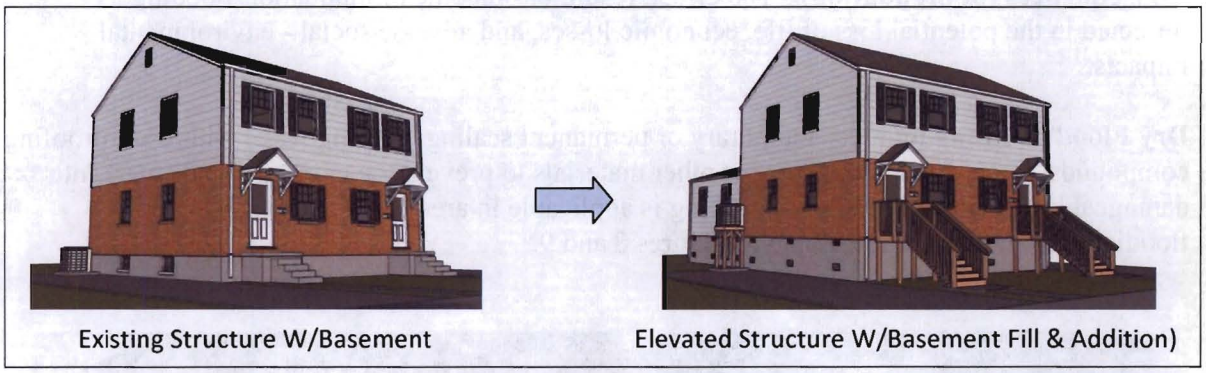

Figure 10
Elevation (Graphic Illustration)

Federal Emergency Management Agency (FEMA) – The agency within the Emergency Preparedness and Response Directorate of the U.S. Department of Homeland Security. FEMA facilitates coordination of Federal dam safety programs and administers the NFIP and several flood mitigation planning and grant programs.

Fenestration – The arrangement of windows and doors in a structure.

FIRM – Flood Insurance Rate Map

Flood - A *flood* is an overflow of water that submerges land or structures which is normally dry.

Flood Insurance provides insurance to assist in recovery from a flood event. Typically not include with homeowners insurance policy.

Flood Louver / Flood Vent – Flood louvers / flood vents are a permanent opening in a wall designed to allow unobstructed passage of water (automatically) in and out of a structure thereby preventing water pressure buildup (hydrostatic pressure) that can damage or destroy foundations and bearing walls.

Flood Risk - The likelihood and consequences that may arise from flood event.

Flood Risk Management – Federal and non-Federal policies and programs for managing flood risk. This includes measures that reduce the flood hazard as well as measures that reduce the exposure and vulnerability of persons and property.

Flood Risk Management Measures - These measures include implementation of reservoirs, detention storage, channels, diversions, levees, interior drainage systems, flood-proofing, levee raising, relocation of buildings/communities, and flood warning and emergency preparedness actions. It also includes policies and programs intended to inform and to influence the decisions made by Federal, state, and local government agencies, individuals, businesses and communities in their choice of flood risk reduction measures and to locate assets in flood plain.

Flood-frequency – A graph, table, or single tabulation showing the relationship of the flood variable of interest (peak flow, peak stage, 3-hour volume, etc.) to the probability of the variable being exceeded in any given year.

Foundation Vents – Foundation vents are permanent openings in foundation walls ventilation and unrestricted passage of air for ventilation of the crawl space. In wet floodproofing applications additional foundation vents may be required to release air pressure changes caused by rising/falling water in confined spaces (crawlspace).

National Flood Insurance Program (NFIP) – Federal program under which flood-prone areas are identified and flood insurance is made available to the owners of the property in participating communities.

Nonstructural Measures – Historically, this term was used to distinguish flood risk reduction measures constructed to reduce the flood hazard (such as reservoirs and levees) from measures that might be directed to reducing consequences.

National Nonstructural Flood Proofing Committee (NFPC) - The National Nonstructural Flood Proofing Committee functions under the general direction of the Chief, Planning Community of Practice, Directorate of Civil Works, and HQUSACE. The objectives of the NFPC are to:

- Promote the development and use of all nonstructural flood risk reduction measures.
- Risk expertise on all aspects of nonstructural flood risk reduction and associated opportunities.
- Disseminate nonstructural flood reduction information
- Partner with Planning Centers of Expertise in all aspects of nonstructural flood risk reduction and associated opportunities.
- Provide leadership in all aspects of floodplain management NFPC Website - http://www.usace.army.mil/Missions/CivilWorks/ProjectPlanning/nfpc.aspx

Project (flood risk reduction) - A flood risk reduction project is made up of one or more flood risk reduction systems which are under the same Congressional or other organizational entity authorization.

Probability (likelihood) – Likelihood is a measure of the chance, or degree of belief that a particular outcome or consequence will occur. A probability provides a quantitative description of the likelihood of occurrence of a particular event.

Relocation involves moving the structure to another location away from flood hazards. Relocation is the most dependable method of protection and provides the benefit of use of the evacuated floodplain.

Return period – Alternate term 'recurrence interval.' The return period is the average time interval, usually expressed in years, between occurrences of an event of a certain magnitude. The return period is often computed as the reciprocal of the annual chance exceedance.

Risk – Measure of the probability and severity of undesirable consequences.

Risk Communication – Risk communication is the open, two-way exchange of information and opinion about hazards and risks leading to a better understanding of the risks and better risk management decisions.

Silver Jackets (SJ) - The Silver Jackets is an innovative program that provides an opportunity to consistently bring together multiple state, federal, and sometimes tribal and local agencies to learn from one another and apply their knowledge to reduce risk. Website - http://www.nfrmp.us/state/

Structural Measures – Historically, this term was used to distinguish flood risk reduction measures constructed to reduce the flood hazard (such as reservoirs and levees) from measures that might be directed to reducing consequences.

Uncertainty – Used to describe any situations without sureness, whether or not described by a probability distribution.

Wet Flood Proofing measures allows floodwater to enter the structure, vulnerable items such as utilities appliances and furnaces are relocated or waterproofed to higher locations. By allowing floodwater to enter the structure hydrostatic forces on the inside and outside of the structure can be equalized reducing the risk of structural damage.