DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS, OMAHA DISTRICT
106 SOUTH 15TH STREET
OMAHA NE 68102-1618

AUG 0 2 2006

REPLY TO
ATTENTION OF

Planning, Programs, and Project Management Division

SUBJECT: Transmittal of Final 10 Percent Design Analysis – Lower Yellowstone Project at Intake, Montana

Mr. Dan Jewell, Area Manager
U.S. Bureau of Reclamation
Montana Area Office
P.O. Box 30137
Billings, Montana 59107-0137

Dear Mr. Jewell:

The U.S. Army Corps of Engineers, Omaha District (Corps), recently completed the 10 percent Design Analysis for the Lower Yellowstone Project at Intake, Montana. Enclosed please find three copies of the main report and technical appendices, as well as a CD-ROM that contains an electronic version of the same material.

The 10 percent design effort was completed under contract with your office, and I am pleased to report that we completed the work under budget and will be returning the excess funds later this month. I would also like to express our thanks to the Bureau of Reclamation's Great Plains Region, Montana Area Office, and Technical Engineering Centers, as well as the Lower Yellowstone irrigation districts for their cooperation and dedication to this project as we developed the 10 percent design. Without their support, the 10 percent design would have been less accurate, more expensive, and would not have been completed on time.

The Lower Yellowstone Project is a critical component in the overall recovery strategy for pallid sturgeon in the Missouri River Basin. Working collaboratively, I believe the Bureau of Reclamation and the Corps can and will succeed in modifying the existing diversion dam to successfully address fish passage and fish entrainment issues.

As you may be aware, there is language included in the pending Water Resources Development Act and the Energy and Water Development Appropriations Bill that would authorize the Corps to utilize Missouri River Recovery funds to work with the Bureau of Reclamation on this project. Once authority is received through the passage of one of these Bills, the Corps is prepared to move forward with the Bureau of Reclamation to implement the project in an accelerated manner.

If you have any questions regarding this submittal, please contact Mr. Greg Johnson, Project Manager, at (402) 221-7258.

Sincerely,

Joel R. Cross
Lieutenant Colonel, Corps of Engineers
District Commander

Enclosures

Lower Yellowstone River
Intake Dam Fish Passage and Screening
Preliminary Design Report

July 2006

FINAL REPORT

US Army Corps of Engineers ®
Omaha District

Lower Yellowstone River
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EXECUTIVE SUMMARY

The Bureau of Reclamation's Lower Yellowstone Project and its impacts on the fishery of the Lower Yellowstone River has been the subject of many studies by state and federal resource agencies. These studies indicated that the unscreened intake structure entrains large numbers of fish into the canal system with the diversion flow, and that the diversion dam itself is barrier to upstream migration of many fish species, including the endangered pallid sturgeon. The natural condition of the Yellowstone River and its status as historical habitat to pallid sturgeon affords it unique opportunities that could contribute to assisting with the recovery of that species and developing a better understanding of its behavior and habitat preferences. The U.S. Fish and Wildlife Service even emphasized the importance of the Yellowstone River to pallid sturgeon recovery in both their 1993 Recovery Plan for the Pallid Sturgeon and 2003 Amended Biological Opinion for the Missouri River Master Manual. The Lower Yellowstone Project is key to that effort because of its strategic location, ~75-miles upstream from the Yellowstone-Missouri River confluence and ~165-miles downstream from the next irrigation diversion dam.

During 2005 the Bureau of Reclamation and the Corps of Engineers agreed to collaborate and form a partnership with the U.S. Fish and Wildlife Service, the State of Montana Department of Fish, Wildlife and Parks, and The Nature Conservancy of Montana. The partnership was formalized with the signing of a Memorandum of Understanding in July 2005.

During April 2006 the Bureau of Reclamation contracted with the Omaha District to conduct a Preliminary (10%) Design Analysis and develop detailed cost estimates for the most promising alternatives: (1) retrofitting the existing dam with an engineered rock ramp, and (2) removing the dam and relocating the diversion structure upstream approximately 2.5 miles for gravity diversion without a dam. In addition, the Corps was tasked with (3) reviewing the preliminary design of the v-shaped fish screen and developing a detailed cost estimate. This document presents the results of the Preliminary Design Analysis.

This Preliminary Design Analysis involved a multi-discipline team that completed the following tasks:

- *Fisheries - Reviewed available pallid sturgeon fishery data on the Yellowstone River, fish screen components, and alternative performance for fish passage.*
- *Hydrology – Updated and expanded Yellowstone River flow frequency and flow duration analysis.*
- *Hydraulics – Performed preliminary design analysis of both alternatives to determine size of alternative features, evaluate hydraulic feasibility, and identify design concerns.*
- *Ice Jams and Forces – Collected historic ice information and conducted a preliminary analysis of ice forces and rock ramp stability.*

- *Geomorphology – Evaluated historic Yellowstone River geomorphic data and developed recommended river stabilization structures for the upstream diversion relocation alternative.*
- *Geotechnical – Performed geotechnical evaluation, prepared drawings, identified construction concerns in the river environment, and investigated material sources and disposal for each alternative.*
- *Engineering Design – Performed structural computations and prepared drawings for each alternative including design of a concrete cap for the existing dam to resist ice forces and design of several structures for the upstream diversion.*
- *Cost Engineering – Developed cost estimates of each alternative.*

Recommendation

The purpose of this study was to develop a refined analysis with an updated cost estimate, investigate elements of alternative feasibility, and evaluate alternative performance with respect to successful fish passage. The fish screen will meet the objective of preventing entrainment of pallid sturgeon from all age classes, but due to a lack of detailed data, still has many unresolved questions that will need to be addressed in further design phases. Both the engineered rock ramp and the dam removal combined with moving the diversion upstream appear capable of meeting fish passage objectives while retaining the irrigation diversion capability. Both fish passage alternatives have unresolved issues that will also need to be addressed in further design phases, such as what slope to use on the rock ramp or where to dispose of excavated material for the new canal in the relocate diversion upstream. This study presents only the preliminary design data and cost estimates and is not intended to be a decision document or make recommendations on a final course of action. A summary of the detailed cost estimates that were developed for the three alternatives is presented in the following table.

Summarized Cost Estimates for Alternatives

Alternatives	*Estimated Cost [$ millions]*
Fish Screen Facility	*9.66*
Rock Ramp	
5% Slope [Guernsey quarry]	*12.53*
5% Slope [Local quarry]	*11.18*
5% Slope [Concrete formed stone]	*17.29*
2% Slope [Guernsey quarry]	*15.73*
2% Slope [Local quarry]	*14.38*
2% Slope [Concrete formed stone]	*20.38*
Relocate Diversion Upstream	*43.15*

INTRODUCTION

Project Location and Description

The Lower Yellowstone Project is a 100-year old irrigation project consisting of an irrigation diversion dam (known as Intake Dam), a gated intake structure at the inlet to the Main Canal, the Thomas Point Pumping Plant, a 72-mile Main Canal, 225-miles of laterals, and 118-miles of drains that deliver irrigation water to approximately 52,000 acres of agricultural land in eastern Montana and western North Dakota. The Intake Dam is located approximately 75-miles upstream from the mouth of the Yellowstone River or 17-miles northeast of Glendive, Montana (Figure 1 and Figure 2). The Lower Yellowstone Project was authorized under the Reclamation Act of 1902 and construction began in 1905 with the first water deliveries for irrigation in 1909. The economic impact of the project is estimated at approximately $30 million annually.

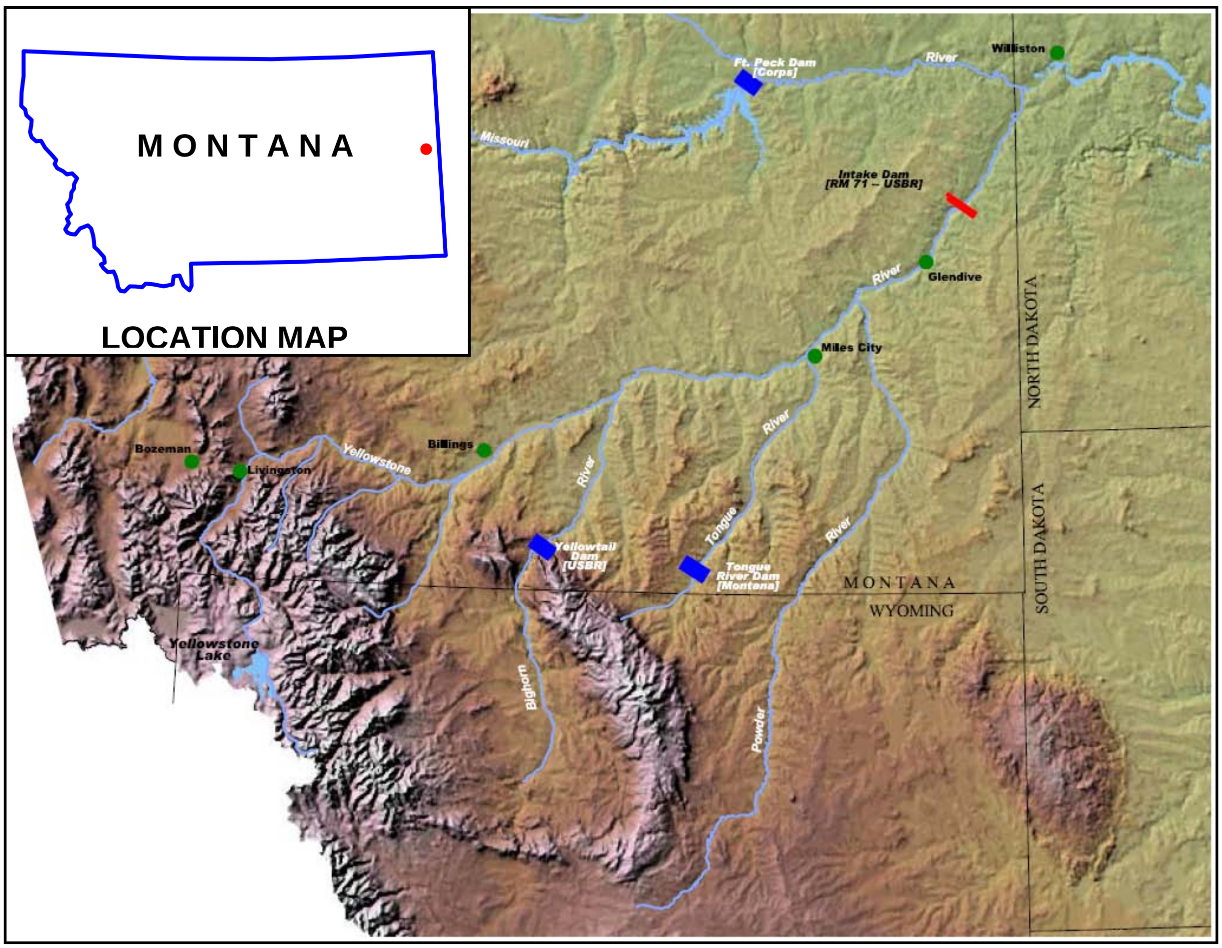


Figure 1. Project Location Map

Project Issues

The Lower Yellowstone Project and its impacts on the fishery of the Lower Yellowstone River has been the subject of several studies by state and federal resource agencies. These studies indicated that the unscreened intake structure entrains large numbers of fish into the canal system with the diversion flow, and that the diversion dam itself is at least a partial barrier to upstream migration of many fish species and likely a complete barrier to some fish species. The State of Montana Department of Fish, Wildlife and Parks (FWP) and U.S. Fish and Wildlife Service (Service) have both expressed concern about the impacts the Lower Yellowstone Project has on the fishery of the Lower Yellowstone River, with special emphasis on the impacts related to the endangered pallid sturgeon. In 1993 the Service published its Pallid Sturgeon Recovery Plan which described the historic range of pallid sturgeon as encompassing the entire Missouri River from Great Falls, Montana to the mouth, the Middle and Lower Mississippi River, and the Lower Yellowstone River from the Bighorn River confluence to the mouth (1993). If successful fish passage could be established at the Intake Diversion Dam it would restore access to the majority (165 additional river miles, 235 total river miles) of the species' historic range on the Lower Yellowstone River including the important confluence areas at the mouths of the Powder and Tongue Rivers.

Project Team and Approach

Reclamation has been working for several years to address the issues with the Lower Yellowstone Project. Their efforts produced two separate Fish Protection and Passage Concept Study Reports (original Concept Report, January 2000, and revised Concept II Report, April 2005) with recommended project modifications for addressing the issues. Both studies recommended a rock fishway be constructed on the right abutment of the dam for upstream fish passage, and an automated screen structure be constructed within the upper end of the Main Canal for entrainment. In both instances feedback from the regulatory agencies, namely the Service and FWP, expressed strong reservations about the effectiveness of the fishway in attracting and passing native fish, especially pallid sturgeon, over the dam. Neither agency expressed strong reservations about the proposed fish screen structure, but did have some minor comments related to design elements.

During the spring of 2005, Reclamation contacted the Omaha District (Corps) to discuss opportunities to work together on addressing endangered pallid sturgeon issues related to the Lower Yellowstone Project. Reclamation and the Corps had successfully worked together in 2001 to evaluate several fish passage alternatives for the project. In addition, Reclamation and the Corps were successfully partnering on multi-agency work elsewhere in Montana, and the two agencies had recently signed a Memorandum of Understanding at the headquarters level indicating a willingness and need to work together on challenging water resources issues. And finally, Reclamation and the Corps were both active members in pallid sturgeon recovery efforts with many stakeholders and were willing to combine forces on the Lower Yellowstone Project to share lessons learned and collaborate to find a solution.

Reclamation and the Corps decided that they should expand the membership in an effort to partner with other agencies and stakeholders and invited the Service, the State of Montana, The Nature Conservancy of Montana (TNC), and the Lower Yellowstone Irrigation Districts to join the team. In July 2005 the multi-agency and stakeholder team signed a memorandum of understanding (MOU) to work together collaboratively towards a solution to the issues at the Lower Yellowstone Project. The signatories on the MOU were FWP, Service, The Nature Conservancy (TNC), Reclamation, and the Corps. The Lower Yellowstone Irrigation Districts chose not to sign the MOU, but have still been very active participants and invaluable partners throughout the entire process.

Subsequent to the signing of the MOU, the team began working together in search of cost effective and reliable solutions the issues. In the summer of 2005 the team met in Billings for a week-long Value Planning Study to brainstorm and develop fish passage alternatives so that they could be evaluated and screened against performance criteria which were considered to be critical to the project. The Value Planning Study results were published in a report dated August 10, 2005. The Value Planning Study recommended a rock ramp, a pumping plant, and moving the diversion upstream as the most promising alternatives for further evaluation.

In April 2006 the Corps was hired by Reclamation to conduct a preliminary (10%) design analysis and develop detailed cost estimates for the most promising alternatives: (1) retrofitting the existing dam with an engineered rock ramp, and (2) removing the dam and relocating the diversion structure upstream approximately 2.5 miles for gravity diversion without a dam. In addition, the Corps was tasked with (3) reviewing the preliminary design of the v-shaped fish screen and developing a detailed cost estimate. In addition, a cursory review of a pumping plant alternative was conducted, but it was concluded that the pumping plant was going to be very expensive and would not be acceptable to many local and regional entities.

BASELINE EXISTING CONDITIONS

This section of the report is dedicated to presenting baseline design information which is universal to all of the alternatives. This data reflects the existing physical and biological condition of the Yellowstone River and project site.

General Project Description

An overview aerial photo of the Lower Yellowstone Project site is shown in Figure 2, including the critical infrastructure to the project and other pertinent features in the same vicinity. Intake Dam itself was originally designed as a 12-foot high rock-filled timber crib weir. The weir spans the entire width of the main channel of the Yellowstone River (approximately 700-feet) and results in raising the water surface profile from 2 to 5-feet depending on the flow in the river (figures 3 and 4 show original design plan and cross section for the project). Over the years, the dam has suffered severe and repeated damage from ice and debris flows during spring runoff. Repairs and maintenance have consisted of periodically replacing the timber on the weir and frequently placing large quantities of rock riprap on the downstream side of the weir. During high flow and ice flows the rock riprap is moved and rolled downstream resulting in the downstream side of the dam resembling a steep rock rapids with rock boulders scattered over an area as wide as the dam and extending downstream from 100 to 300-feet. Figures 4 and 5 show a comparison of the original weir and the current condition.

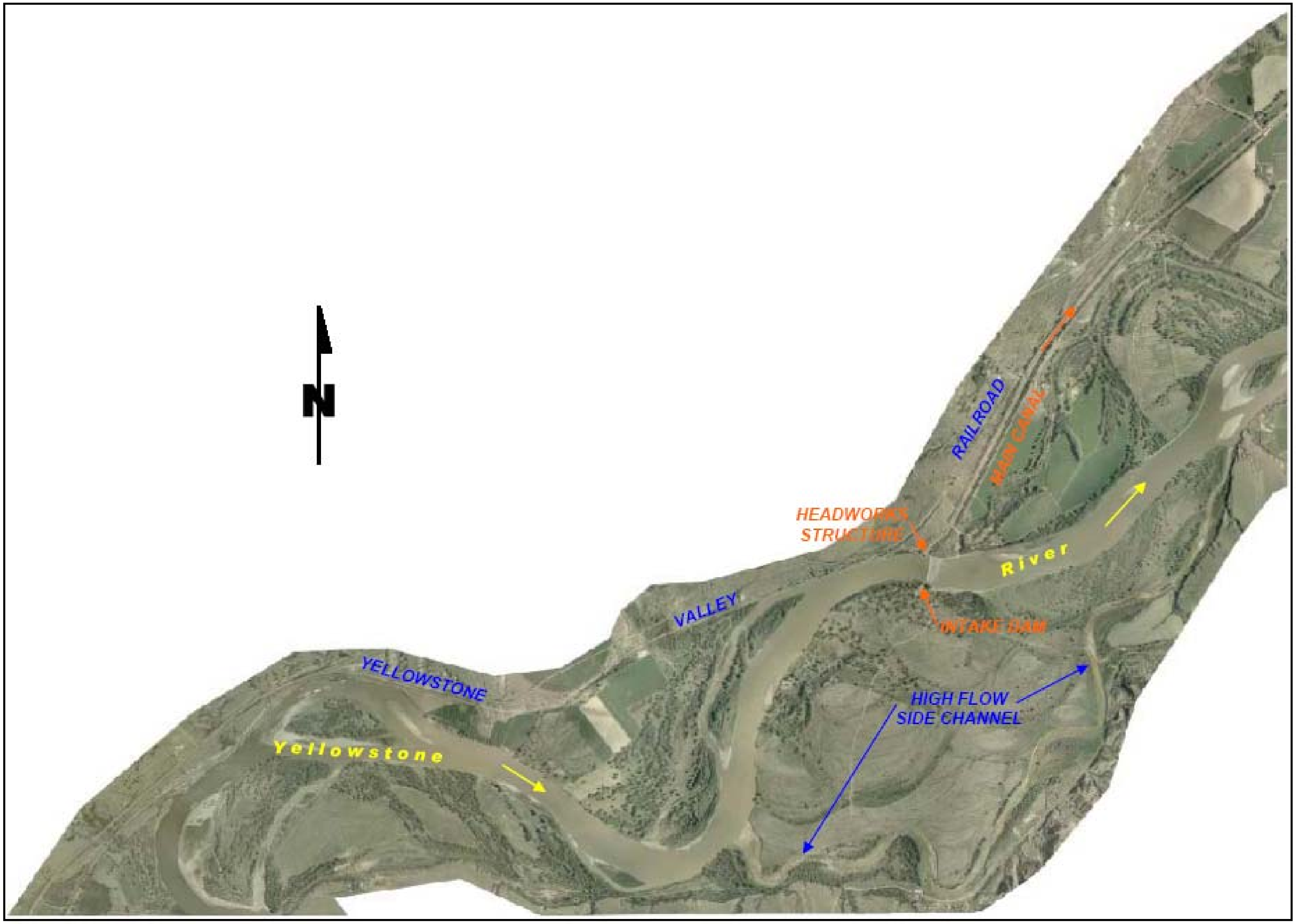


Figure 2. General Overview of Project Site

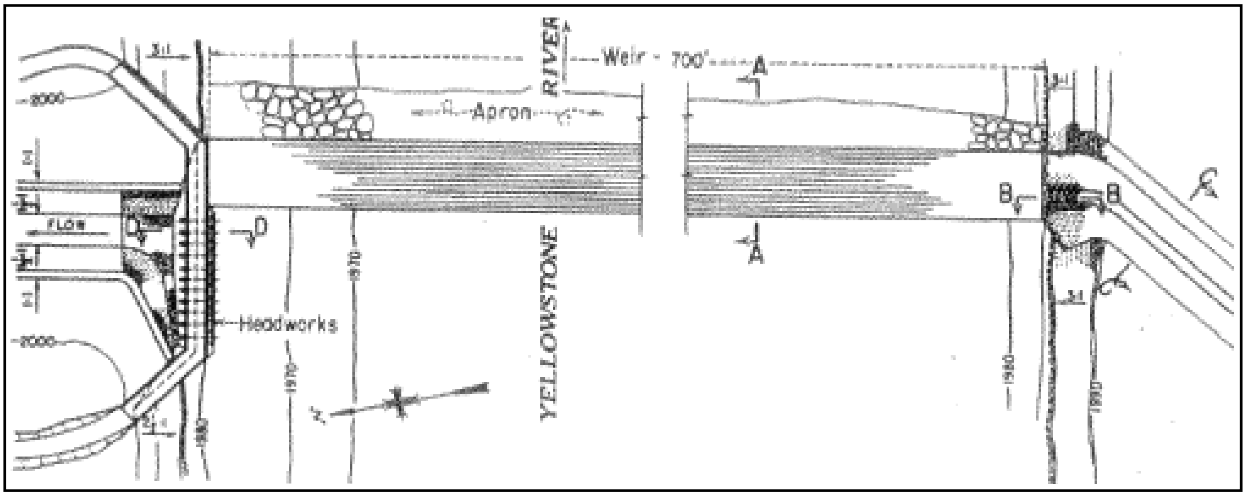


Figure 3. Original Design Plan View of Dam and Headworks (Reclamation)

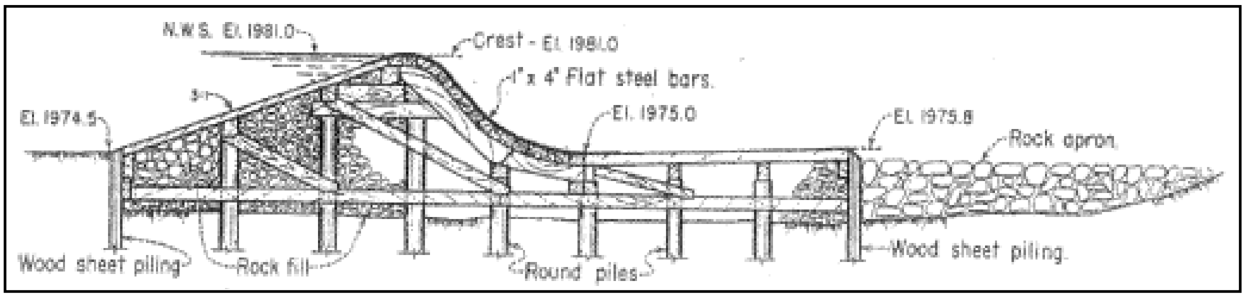


Figure 4. Original Design Cross Section of Dam (Reclamation)

Figure 5. Original Dam/Weir
(Reclamation, circa 1910)

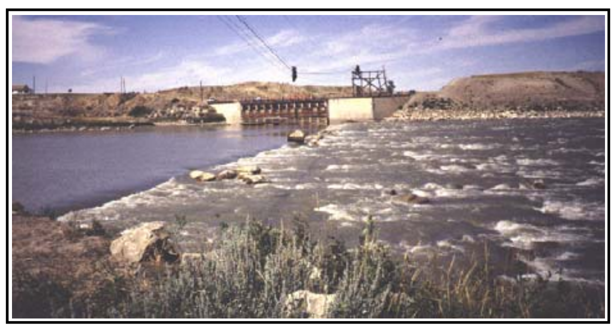


Figure 6. Current Dam/Weir Condition
(Lower Yellowstone Irrigation Districts, circa 1990)

Fisheries

Existing information related to the life cycle requirements of pallid sturgeon (*Scaphirhynchus albus*) are very limited, so it is common practice to extrapolate information from data collected when pallids are captured in the field, laboratory tests, data from surrogate species, or best scientific opinion. Because of these uncertainties, the development of engineered structures to facilitate the migration and spawning of these rare and endangered fish is not an exact science. Therefore, it is important to incorporate flexibility into the designs allowing possible modification and adaptability to accommodate advances in the understanding of sturgeon requirements and behavior that may come from long-term monitoring and other research efforts.

Pallid sturgeon are benthic (bottom dwelling) warm-water fish which are native to large, deep water regimes of the Missouri, Yellowstone, and Mississippi Rivers with some expansion into the lower reaches of major tributaries, such as the Platte and Kansas Rivers. The fish possess poor swimming abilities compared to many other species, such as salmonids. Historically, the majority of fish passage research, literature, guidance, and actual construction has been based on salmonid swimming and jumping capabilities. Fish passage for warm-water fish is a relatively new field by comparison. Some incidental passage of other sturgeon species (*Acipenser spp.*) has been documented on a few fish passage facilities, however, fish passage structures are uncommon within the range of pallid and shovelnose sturgeon, so there is limited opportunity for upstream passage through these structures.

Pallid Sturgeon on the Yellowstone. Pallid sturgeon are known to be present in the Yellowstone River, and, based on telemetry studies conducted by Reclamation and FWP, do appear to be restricted from passage above the Intake Diversion Dam in most years. The last documented capture of a pallid sturgeon above the Intake Dam was in 1991. However, pallid sturgeon are so rare that their capture during sampling is very uncommon. During high flow years, there may be limited fish passage either over the Intake Dam or through the high-flow side channel that runs along the right bank floodplain. Based on captures of adult pallid for brood stock use, pallid sturgeon which are in spawning condition have been found along the Lower Yellowstone River and in the area of the Yellowstone – Missouri River confluence.

The Intake Diversion Canal off the Yellowstone River entrains large quantities of native and non-native fish. Field sampling studies conducted from 1996 through 1998 documented an estimated 383,000 to 810,000 fish were entrained in the canal (Hiebert, 2000). Of the 34 species collected, 25 were native species, including shovelnose sturgeon, paddlefish, and sturgeon chub (a species of concern) but no pallid sturgeon. Even though no pallids were collected during the entrainment study it is still logical to assume that pallid sturgeon may also be entrained within the canal and subsequent studies by Jaeger actually documented entrainment of some telemetered hatchery raised pallids that were restocked above the dam. Little is known about pallid sturgeon (or other native fish) larvae on the Yellowstone above Intake Dam or their entrainment into the canal, since no know larval sampling or drift studies have been conducted on that reach of river or within the canal. However, since larvae tend to drift with the flow, it's logical to suspect that some percentage of larvae probably enter the canal with the diverted flow.

Hydrology

The purpose of the hydrologic analysis was to establish flow-frequency and flow-duration relationships for the Yellowstone River at the Intake Dam site. The amount of flow in the River at any particular time is paramount to understanding the depth and velocity relationships related to design of fish passage at the dam. The two nearest stream gaging stations on the Yellowstone River to the Intake Dam are at Glendive, Montana (approximately 18 miles upstream) and near Sidney, Montana, (approximately 36 miles downstream). The Sidney gage has been in operation more or less continuously since October 1, 1910 (only gap is from October 1, 1931 through September 30, 1934). Even though there is another gage at Glendive that could have been used to fill in the data gaps, a comparison of flows (adjusted for irrigation diversion and lag) indicated that over 40% of the time the flows showed deviations of greater than ±10% (over 65% of the time the deviations were greater than ±5%). Therefore only the data from the Sidney gage was used in the analysis. For more information on the hydrologic analysis see Appendix B.

Flow-Duration Analysis. The daily discharge at Sidney was utilized to develop annual and monthly flow-duration curves for the 1911-1931 and 1934-2005. Flow-duration curves are used to define the percent of time that a given flow is equaled or exceeded, and are not to be used to assess the probability of a given flow occurring. Of note are the effects of reservoir operations, particularly Yellowtail Dam on the Bighorn River, has had on the flow regime of the Lower Yellowstone Basin. Yellowtail Dam is Bureau of Reclamation multi-purpose hydropower dam whose construction was complete in 1966. The reservoir regulates approximately 28% of the Yellowstone River watershed upstream of the Sidney gage. Because of the effects of Yellowtail and other dams it is recommended that the post-Yellowtail Dam period flow-duration curves generally be followed for the design of the project, as this period best represents the expected flow regime for the foreseeable future. The exception would be for flows occurring ~2% or less of the time, as there is still a substantial unregulated area subject to large rainfall and/or snowmelt events upstream of Sidney that could produce large flows. The recommended flow-duration curves are shown in Table 1.

Table 1. Recommended Annual and Monthly Flow-Duration Curves, Sidney, MT, Yellowstone River

Percent Time Flow Exceed or Equaled	Annual	Month: Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
0.01	125,000	119,000	16,000	14,000	17,000	76,000	114,000	118,000	104,000	142,000	112,000	38,000	39,000
0.05	111,000	105,000	16,000	14,000	17,000	53,000	97,000	116,000	104,000	134,000	106,000	38,000	37,800
0.1	95,500	65,700	14,700	13,000	16,200	50,900	93,500	94,000	87,500	127,000	101,000	37,000	33,700
0.2	86,400	32,300	13,300	12,400	15,600	39,700	87,800	84,200	82,900	121,000	85,000	34,000	29,000
0.5	74,700	23,800	12,800	11,800	14,700	35,900	59,300	57,400	61,900	108,000	80,000	28,000	22,000
1	65,500	19,800	12,300	11,300	13,500	22,900	50,500	38,100	53,200	93,000	73,200	25,400	17,900
2	55,700	15,700	11,700	10,500	12,200	18,200	39,500	29,800	48,000	84,600	66,900	22,600	15,300
5	35,700	12,700	11,300	9,310	10,600	14,400	26,900	16,100	35,900	59,900	44,000	16,800	12,800
10	25,500	11,700	10,900	8,790	9,450	11,600	17,500	14,500	31,100	54,700	37,500	13,800	11,500
15	18,400	11,100	10,400	8,490	8,750	10,100	14,400	13,500	27,000	49,900	33,500	12,400	10,500
20	14,700	10,700	10,100	8,290	8,140	9,460	12,800	12,500	23,300	46,200	30,300	11,500	9,710
30	11,300	9,940	9,480	7,930	7,510	8,660	10,900	10,500	19,400	40,500	26,300	9,890	8,780
40	9,600	9,380	8,710	7,490	7,170	7,970	9,670	9,160	16,900	35,400	21,800	8,230	7,820
50	8,460	8,710	8,080	7,100	6,600	7,400	8,720	8,470	14,800	30,700	17,100	7,080	6,660
60	7,570	7,740	7,210	6,560	6,130	6,530	8,110	7,830	13,200	26,800	14,100	6,010	5,710
70	6,650	6,730	6,650	5,680	5,420	5,900	7,100	7,050	11,500	22,700	11,100	4,810	4,970
80	5,640	6,010	5,590	5,020	4,800	4,910	6,230	6,130	9,770	18,700	7,780	3,980	4,320
85	5,100	5,580	5,140	4,580	4,400	4,710	5,880	5,800	8,450	16,900	6,700	3,490	3,910
90	4,530	5,120	4,790	4,210	4,110	4,490	5,160	5,470	7,560	14,900	5,730	2,710	3,600
95	3,800	4,360	4,160	3,520	3,210	4,180	4,200	5,000	6,230	12,400	4,930	1,770	3,060
98	2,850	4,040	3,140	2,730	2,470	3,440	3,310	4,180	5,260	10,000	3,910	1,470	2,330
99	2,130	3,710	2,230	2,130	2,160	2,990	3,110	3,850	4,530	8,570	3,590	1,390	2,020
99.5	1,720	3,500	1,860	1,940	1,850	2,770	2,900	3,560	2,900	7,730	3,130	1,330	1,610
99.8	1,430	3,490	1,600	1,840	1,360	2,570	2,630	3,020	2,380	7,090	2,460	1,260	1,460
99.9	1,300	3,480	1,510	1,780	1,030	2,500	2,510	2,970	2,030	6,530	2,370	1,220	1,410
99.95	1,190	3,480	1,480	1,730	970	2,480	2,480	2,950	1,980	6,500	2,310	1,190	1,380
99.99	1,030	3,470	1,450	1,650	890	2,450	2,450	2,900	1,940	6,480	2,190	1,130	1,320

Note: Discharges in columns above in cubic feet per second (cfs)

Flow Frequency Analysis. The instantaneous peak flows from the USGS gage near Sidney, Montana were used to develop the flow frequency relationships. Data for water years 1911-1931 and 1934-2004 were input into the flow frequency program HEC-FFA to compute the annual flow-frequency relationship for the period of record. A Regional Skew value of 0.05 was used, as obtained from Figure 14-1 of Bulletin #17B. Table 2 and Figure 7 display the results of the analysis; showing computed and expected probabilities, as well as upper and lower confidence limits.

Table 2. Flow-Frequency, Instantaneous Annual Peaks, Yellowstone River, Sidney, MT

Percent Chance Exceedance	Discharge, cfs Computed Probability	Expected Probability	5% Confidence Limit	95% Confidence Limit
0.2	192,400	198,800	230,800	166,200
0.5	172,300	176,700	203,800	150,300
1	156,900	160,200	183,600	138,100
2	141,400	143,700	163,500	125,600
5	120,600	121,800	136,900	108,500
10	104,200	104,900	116,600	94,800
20	86,900	87,200	95,600	79,900
50	60,400	60,400	65,300	56,000
80	41,200	41,000	44,800	37,400
90	33,400	33,100	36,800	29,800
95	28,000	27,600	31,200	24,500
99	19,800	19,300	22,800	16,700

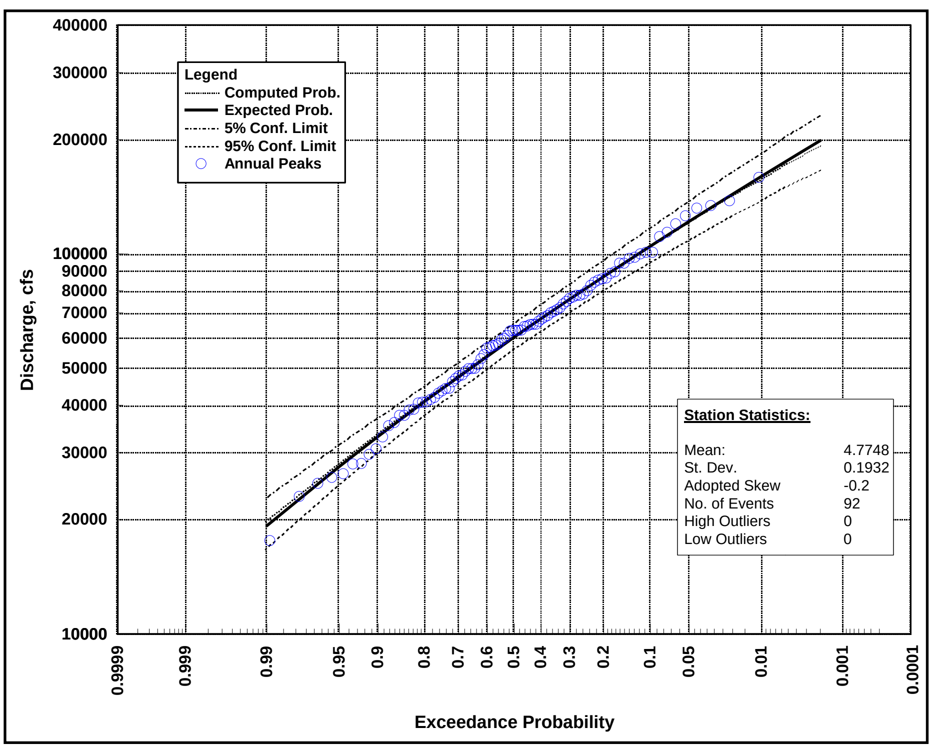


Figure 7. Flow Frequency Relationship (annual instantaneous peak)

In addition, a flow frequency analysis was conducted for the maximum mean daily flow for each month. Again HEC-FFA was used to compute the relationship using a regional skew of 0.05 as obtained from Bulletin #17B. Table 3 contains a summary of the results, with the expected probability shown for each month. It should be noted that flows are often estimated during the ice-cover and ice-breakup period, so computed monthly flow statistics may be adversely affected by the flow estimates in these periods.

It is important to keep in mind the usefulness of each set of hydrologic data. The flow-duration curves should be utilized for meeting design purposes that consider the range of flow that is met a certain percent of the time (e.g., fish passage may require flow velocities between X and Y feet per second for Z percent of the time). The use of the annual or monthly flow-duration curves would be driven by the design parameter to be met. The flow-frequency curves should be utilized for meeting design purposes that require consideration of the likelihood of a given flow being exceeded in a given year (e.g., ice loading on fish passage structures). Use of the annual or monthly flow-frequency curves should be given weight as to which provides the more critical flow condition, as well as suitability to the design feature desired.

Table 3. Summary of Expected Probability and Statistics for Annual and Monthly Flow-Frequency Curves

Percent Chance Exceedance	Annual	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.2	199,000	22,100	93,000	277,000	179,000	121,000	163,000	162,000	56,200	55,900	66,900	17,500	15,500
0.5	177,000	19,500	66,500	195,000	125,000	103,000	147,000	141,000	48,100	44,500	49,700	16,300	14,600
1	160,000	17,600	51,200	148,000	94,000	90,500	135,000	126,,000	42,300	37,200	39,500	15,300	13,800
2	144,000	15,700	39,100	111,000	70,400	79,000	122,000	111,000	36,800	30,800	31,300	14,300	13,000
5	122,000	13,400	26,900	73,100	47,200	64,900	105,000	90,400	29,800	23,500	22,800	12,900	11,900
10	105,000	11,600	19,800	51,700	34,200	55,000	91,900	74,700	24,700	18,700	17,700	11,800	10,900
20	87,200	9,840	14,100	34,700	24,000	45,400	77,400	58,600	19,600	14,500	13,500	10,500	9,850
50	60,400	7,220	8,080	17,300	13,500	32,200	54,500	35,300	12,600	9,210	8,830	8,380	7,940
80	41,000	5,340	5,120	9,330	8,560	23,500	37,300	20,000	7,980	6,150	6,440	6,590	6,280
90	33,100	4,570	4,180	6,930	7,040	20,200	30,100	14,400	6,240	5,070	5,670	5,780	5,510
95	27,600	4,030	3,600	5,490	6,,110	17,800	25,000	10,900	5,070	4,360	5,190	5,160	4,920
99	19,300	3,170	2,830	3,650	4,910	14,300	17,300	6,110	3,400	3,340	4,580	4,130	3,920
Mean	4.7748	3.8611	3.938	4.2615	4.1658	4.5167	4.7274	4.5292	4.0966	3.9789	3.9786	3.9194	3.8941
Standard Deviation	0.1932	0.1561	0.2651	0.3381	0.2707	0.1684	0.1874	0.2766	0.2302	0.2201	0.1981	0.1195	0.1152
Computed Skew	-0.2486	0.0883	1.0778	0.5622	1.0904	0.3418	-0.3685	-0.5715	-0.0703	0.545	2.0861	-0.235	-0.3707
Adopted Skew	-0.2	0.1	0.7	0.4	0.8	0.3	-0.3	-0.4	-0.1	0.4	1	-0.2	-0.3
High Outliers	0	0	1	0	2	0	0	0	0	0	1	0	0
Low Outliers	0	0	0	0	0	1	0	0	1	0	0	0	0

Hydraulics

The purpose of the hydraulic analysis was to develop preliminary hydraulic design information for two alternatives, (1) reconfiguring the existing Intake Dam into an engineered rock ramp or (2) relocating the intake diversion upstream to a location where gravity diversion would not require a dam. The rock ramp alternative would use the existing canal and intake structure and incorporate a sloping rock ramp on the downstream side of the existing structure. Moving the diversion upstream would require installation of a new canal diversion structure at an upstream location and removal of the existing intake diversion dam.

Hydraulic modeling for the design analysis was accomplished using the standard hydraulic modeling software HEC-RAS, version 3.1.3. A new hydraulic model was constructed using recently acquired LIDAR topographic survey data for Dawson County. The LIDAR topographic data was a product collected for the Yellowstone River Corridor Study for the purpose of conducting hydraulic modeling in support of cumulative effects analysis. The datum for the topographic data is Montana State Plane NAD 83 (North American Datum of 1983) and NAVD 88 (North American Vertical Datum of 1988). The topographic data was collected in the fall of 2004 during very low flow conditions (approximately 3,000-4,000 cfs), but did not include any data below the water surface of the river (bathymetry). In order to simulate the missing data an artificial trapezoidal channel about 300 feet wide and 2-3 feet deep was added to the channel cross-sections.

The resulting HEC-RAS model was calibrated to match output from the previous modeling that had been performed by Reclamation in support of the Concept and Concept II Reports since no additional calibration data was available. The HEC-RAS model uses a Manning roughness value of 0.035 for channel regions and 0.050 for overbank regions. The roughness parameters established for the model were based on field observations, engineering judgment, and calibration data, and the resulting values compare favorably to the previous modeling effort. Figure 8 shows the results of the calibrated HEC-RAS model. The crest of the existing Intake Dam is modeled as a broad crested weir 15-foot wide with a discharge coefficient of 2.7. The crest varies in elevation from and elevation of 1987 feet at the Intake to the canal to an elevation of 1989 feet at station 700 (right abutment).

Model Results. The existing condition model was compared to the previous model results with reasonable agreement for similar flow (Figure 9). Comparison shows that the new model has a slightly lower water surface at the higher flow rates. Some of this difference is attributable to the inclusion of the right bank chute and floodplain in the cross section geometry to allow full floodplain flow. The right bank chute allows flow to bypass Intake Dam and access the floodplain. The chute exits the Yellowstone River about 9,500 ft upstream of the dam near station 375+00. The chute re-enters the Yellowstone River about 8,500 feet downstream of the dam near station 195+00. Total chute length is about 24,500 feet. Flow area was not added to the chute as the channel was not flowing at the time of the survey. The chute channel section has a 100 – 200 foot bottom width. At the time of the site visit (23-24 May 2006), the Yellowstone River at Glendive USGS gage flow varied from 26,600 to 29,600. Chute flow seemed to initiate at about that level. During the time of the site visit, estimated chute flow was about 300 - 400 cfs. The estimated flow split is shown in Table 4.

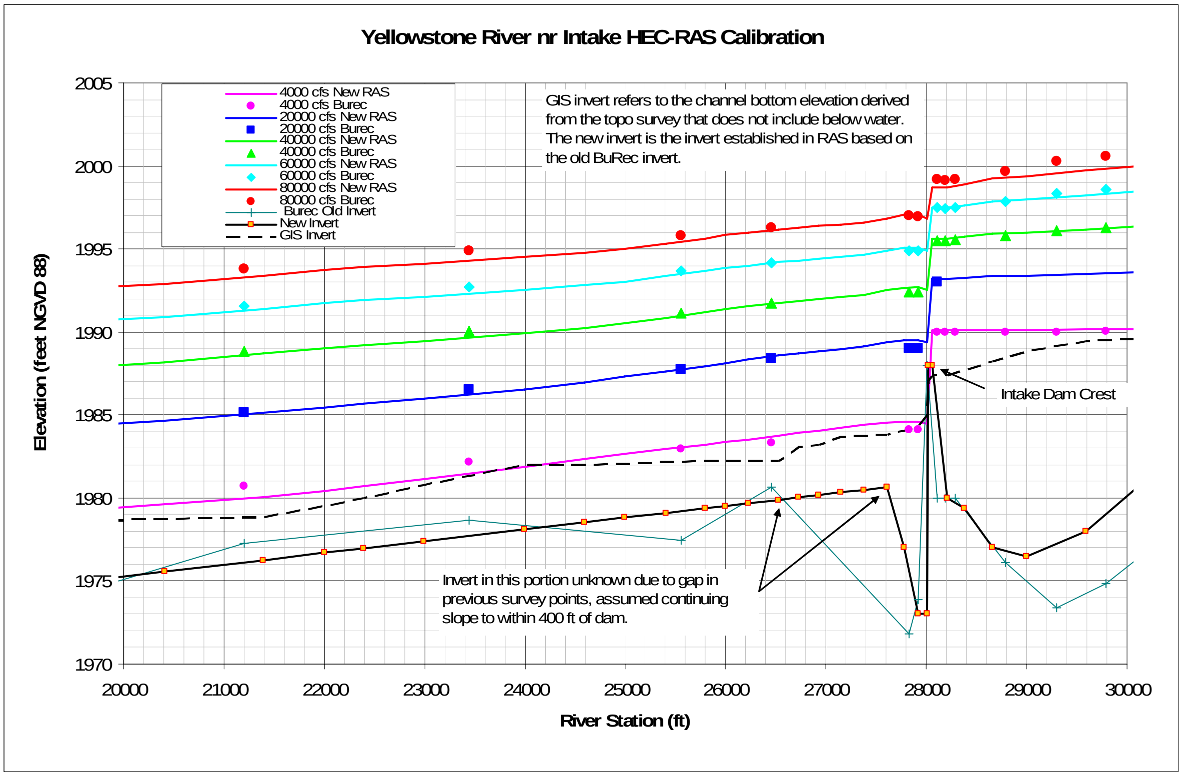


Figure 8. HEC-RAS Model Calibration (New HEC-RAS Model -vs- Reclamation Model)

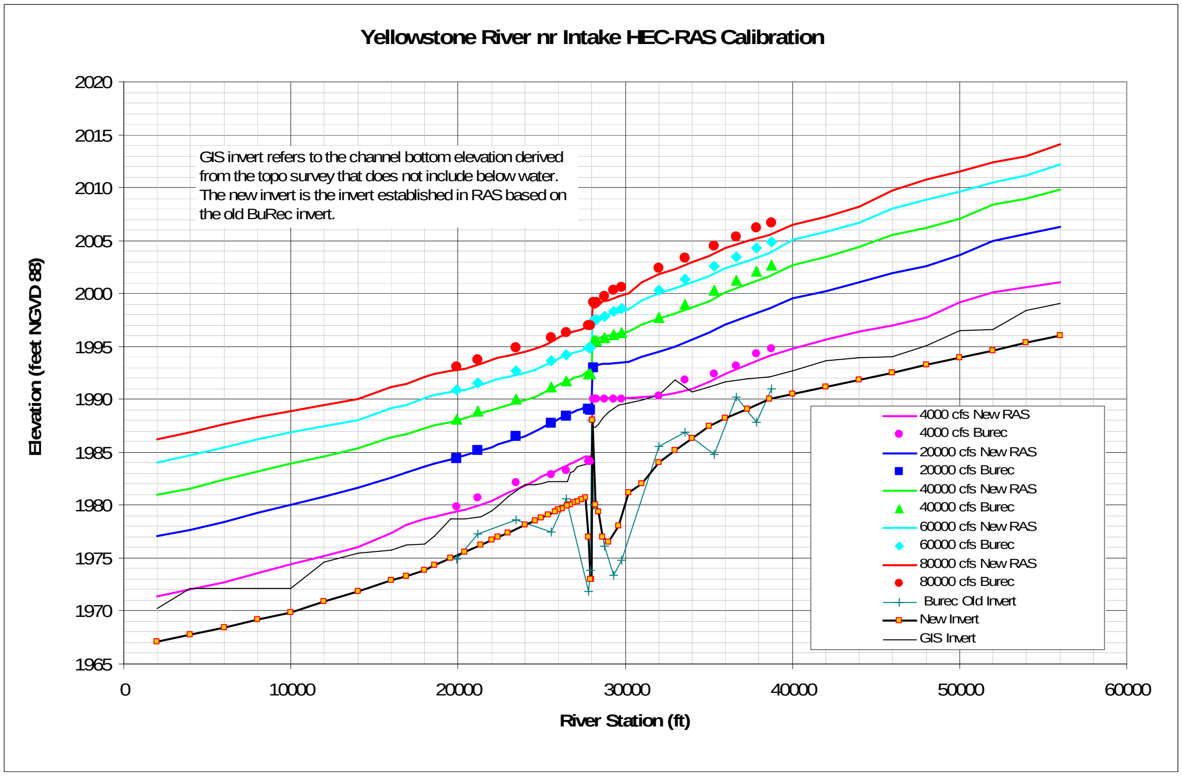


Figure 9. Profile Comparison (New HEC-RAS Model -vs- Reclamation Model)

Table 4. Yellowstone River vs. Right Bank Chute Flow Split			
Total Flow (cfs)	Yellowstone River (cfs)	Chute Flow (cfs)	Chute % of Total Flow
12,000	12,000	0	0.0%
20,000	19,952	48	0.2%
28,000	27,588	412	1.5%
40,000	38,278	1,722	4.5%
60,000	55,413	4,587	8.3%
80,000	73,083	6,917	9.5%
100,000	91,264	8,736	9.6%
120,000	109,311	10,689	9.8%
140,000	127,313	12,687	10.0%
160,000	145,156	14,844	10.2%

Note: Right bank chute flow at low flows is only an approximation since the survey did not include the Yellowstone River invert. The tabulated Yellowstone River flow includes overbank flow above the channel capacity. Within the model, this varies from about 5 to 15 percent of the total flow above 80,000 cfs.

In addition to the chute, the right bank floodplain has capacity to convey floodwaters during high flow events. The right bank floodplain is slightly higher than the chute but still provides floodplain relief. As illustrated in the aerial photo of the floodplain shows several old channel alignment scars. The minimum elevation at which floodplain flow initiates is about elevation 2002 feet upstream of the dam. At the 100-year event, computed flow in the floodplain is about 10,000 cfs with another 15,000 cfs in the right bank chute. It may be possible to excavate a portion of the right bank floodplain to enhance flow bypass of the Intake Dam and reduce the unit discharge across the face of the dam. Figure 10 shows the resulting rating curves upstream and downstream from the dam.

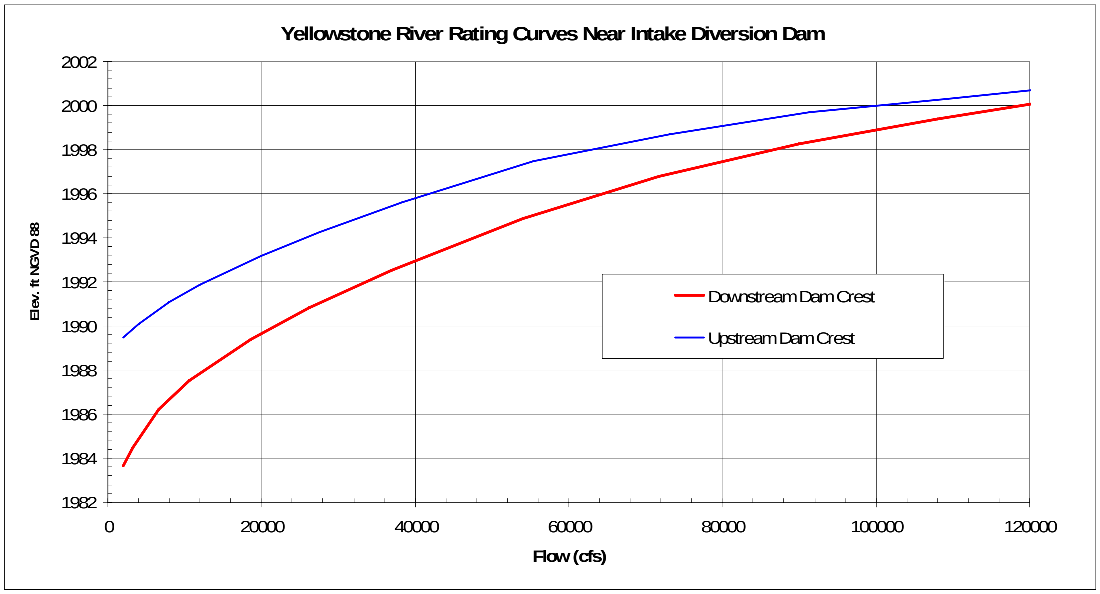


Figure 10. Intake Diversion Dam Rating Curves

<u>Ice Flows and Jams.</u> The Lower Yellowstone River, from Hysham, Montana to the mouth regularly experiences ice jams some of which can be very severe. Ice jam formation on the Yellowstone is a result of several factors, such as decreasing slope of the river from upstream to downstream, transition from warmer to colder air temperatures as one moves in the downstream direction, and contributions of ice from tributaries to the south such as the Bighorn, Tongue and the Power Rivers which typically release their ice before the Yellowstone. Some of the most severe of these jams occur at Glendive, 17 miles upstream of the Intake Dam. Jams are also reported at Intake, MT and the vicinity immediately upstream and downstream of Intake Dam. Table 5 shows the documented occurrence of ice jams from the Corps of Engineers Cold Regions Research and Engineering Laboratory.

Table 5. Historic Ice Jam Locations and Dates on the Lower Yellowstone River

Location	Dates
Hysham	2003
Forsyth	1996
Hathaway	1996 1998
Miles City	1881 1882 1897 1929 1943 1944 1971 1979 1996
Kinsley	1944 1971 1979 1996
Fallon	1996
Cedar Creek	1899 1994 1996
Glendive	1894 1889 1899 1936 1943 1959 1962 1969 1994 1996 1998 2003
Intake	1994
Richland Co. Line	1994 2003
Elk Island	1996
Savage	1943 1996
Sydney	1943 1950 1969 1994 1996 2003
Fairview	1943 1996

Ice thicknesses at the Intake Dam location are estimated to average about 16-inches with a maximum ice thickness of about 21-inches. Based on these ice thickness estimates, to prevent ice related damage to the Intake Dam the recommended required D_{50} rock size for capping the structure is about 4-5 feet. The best estimate of the size of the actual rock size currently used on the structure is 3 feet or less, which explains the need for regular replacement of rock on the structure due to ice pushing rocks off the dam weir during ice flow events.

Geomorphology

There is only a limited amount of historic channel cross-section data available for the Yellowstone River in the vicinity of Intake Dam. However, an evaluation of historic gage measurement data from the Sidney gage, which was collected by the U.S. Geological Survey (USGS) from 1967 to 2006 showed little trend data, suggesting that the river in this reach has likely adjusted to the long term presence of the diversion structure and has now reached a relatively stable geometry. Figure 11 illustrates the long-term data plots for the measured gage heights at the Sidney gage. For the reach upstream from the dam to the City of Glendive, little information is available about historic channel geometry. However, it is probable that the channel in this reach has achieved relative stability based on the fact that the dam has been in place for over 100 years and is not an imposing structure. It is likely that the upstream channel has silted in and formed a delta of sediment.

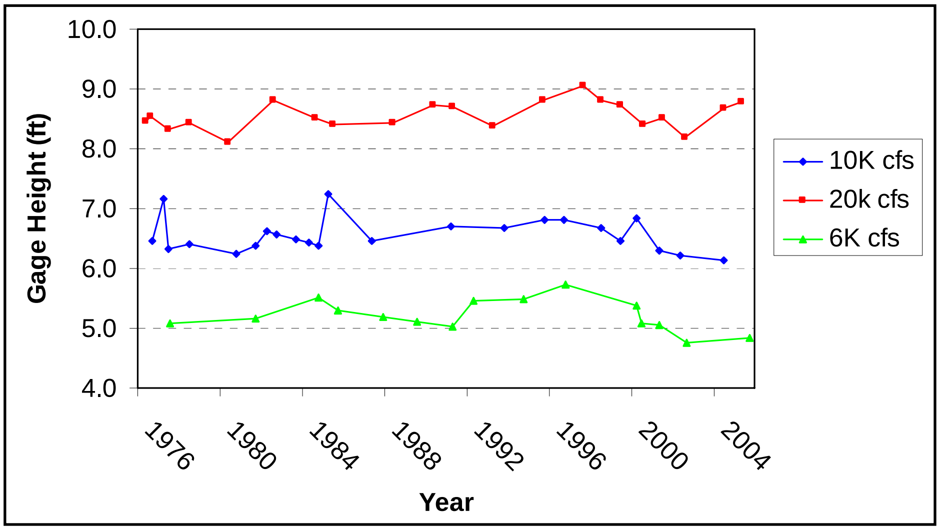


Figure 11. Measured Gage Heights for Yellowstone River at Sidney, Montana

PRELIMINARY FISH SCREEN DESIGN

This section of the report is dedicated to presenting the preliminary design information for a proposed fish screen in the Main Canal which is intended to address the fish entrainment issues. Reclamation has developed the preliminary design for this fish screen as part of the Concept II Report, and the scope of effort in this analysis only included a technical review of the preliminary design and the development of a detailed engineering cost estimate.

Reclamation Design Description

The Bureau of Reclamation Technical Service Center (TSC) developed a conceptual design for a recommended fish screen in 2005. The recommended screen was a flat plate "V" screen structure located on the Main Canal approximately 500 feet downstream from the headworks structure. The fish screen structure includes a trash rack at the inlet to the screen structure, the screens themselves, an automated brush cleaning system, a radial gate check structure, and a fish bypass pipe. The fish screen structure layout and typical sections are shown in figures 12 through 14 show conceptual design drawings for the structure.

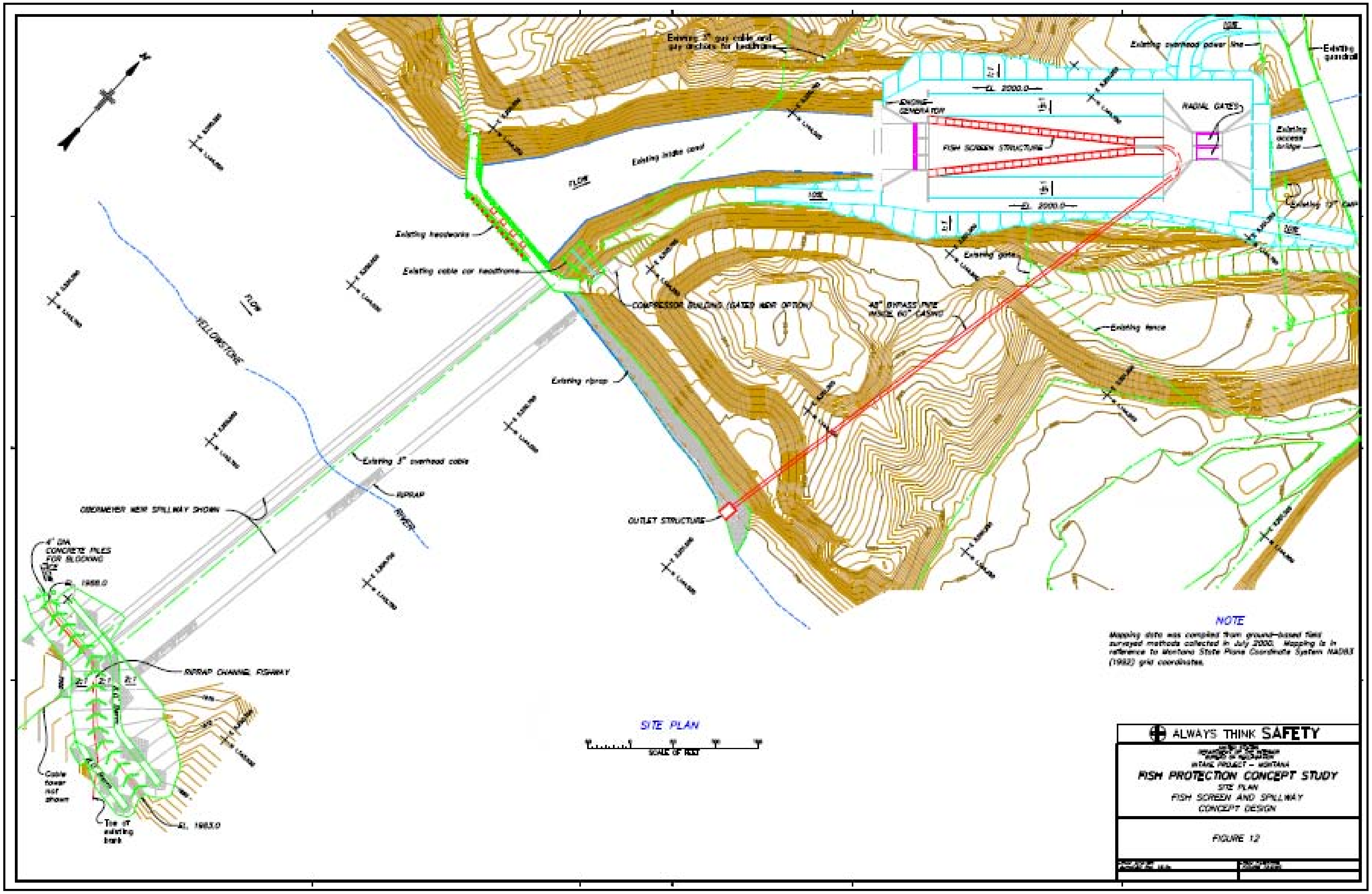


Figure 12. Fish Screen Site Plan

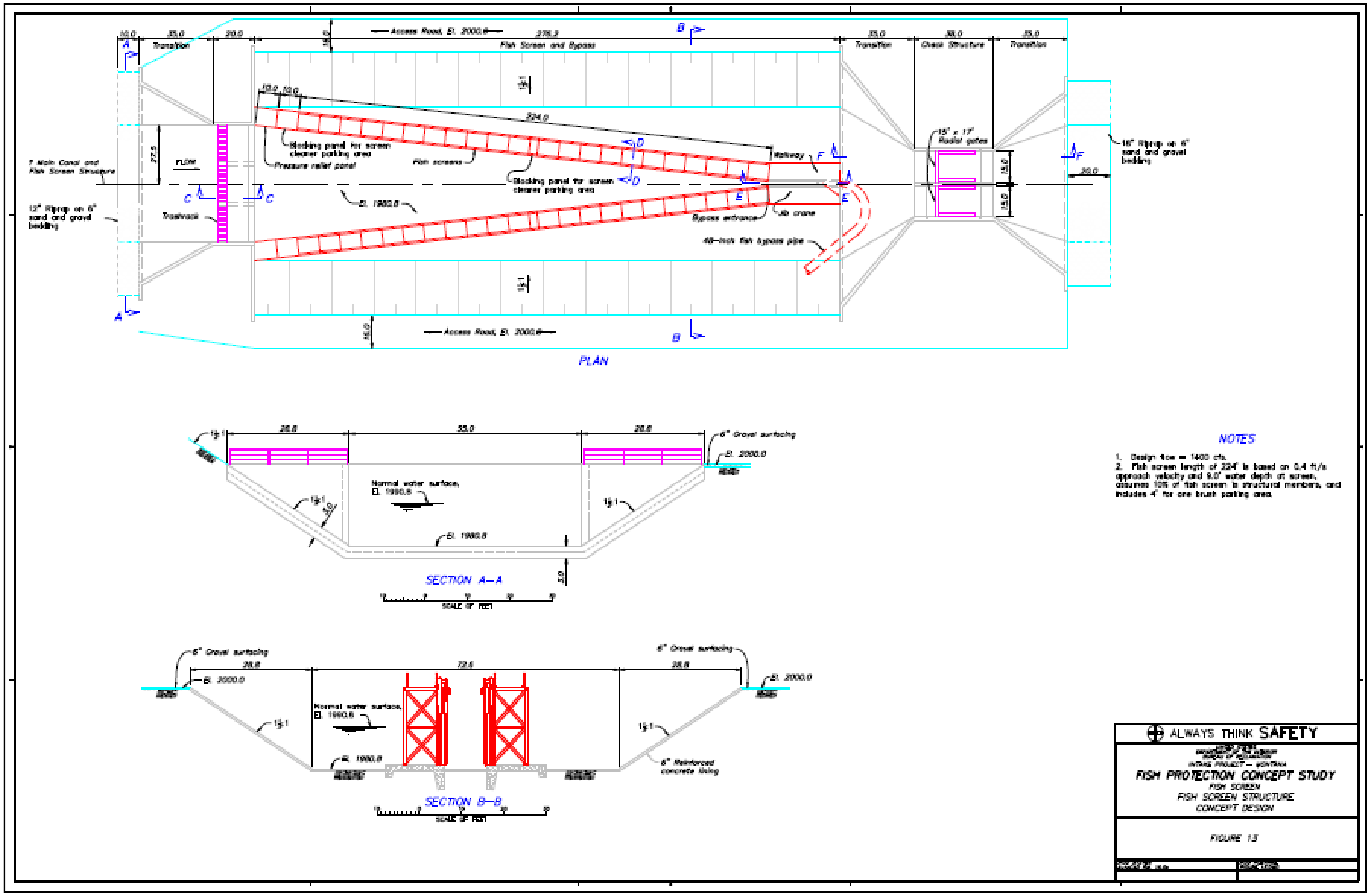
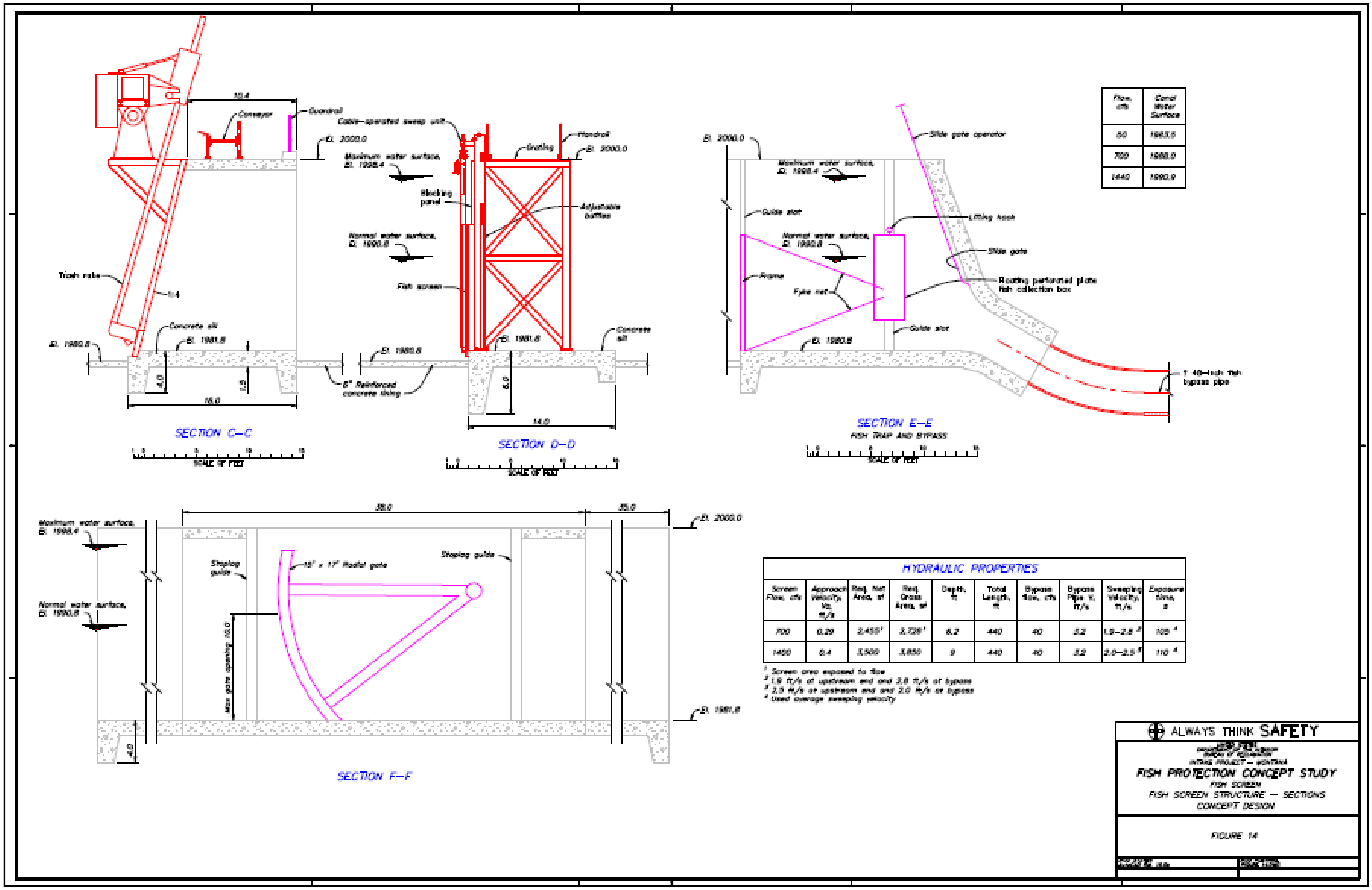


Figure 13. Fish Screen Structure Detail Plan and Cross Sections



Flow, cfs	Canal Water Surface
50	1983.5
700	1988.0
1440	1990.9

HYDRAULIC PROPERTIES

Screen Flow, cfs	Approach Velocity, Va, ft/s	Req. Net Area, sf	Req. Gross Area, sf	Depth, ft	Total Length, ft	Bypass flow, cfs	Bypass Pipe V, ft/s	Sweeping Velocity, ft/s	Exposure time, s
700	0.29	2,455[1]	2,728[1]	6.2	440	40	3.2	1.9–2.8 [2]	105 [4]
1400	0.4	3,500	3,850	9	440	40	3.2	2.0–2.5 [3]	110 [4]

[1] Screen area exposed to flow
[2] 1.9 ft/s at upstream end and 2.8 ft/s at bypass
[3] 2.5 ft/s at upstream end and 2.0 ft/s at bypass
[4] Used average sweeping velocity

Figure 14. Fish Screen Structure Detailed Cross Sections

Listed below is a description of some of the features associated with the fish screen structure:

Trash Rack. The trash rack structure is designed to pass a flow of 1,440 cfs at a 2.5 fps unit velocity. The trash rack is designed with 8-inch spacing between bars, but the bars will stop 2-feet above the structure invert. This design will allow for fish passage through the bars and into the screening structure, but should prevent passage of most large debris. The trash rack structure will also include an integrated hydraulic trash rake and conveyor, for automated cleaning.

Fish screen structure. The fish screen structure is designed to pass a flow of 1,400 cfs with an approach velocity of 0.4 fps and a sweeping velocity that varies from 2.5 fps at the upstream end of the screen to 2.0 fps at the downstream end. The design approach velocity of 0.4 fps was used to reduce the risk that weak swimming fish would be impinged on the screens. The screen structure is designed to have a 12-inch concrete sill to accommodate sediment deposition without affecting the screen operation and to increase the opportunity for benthic fish to pass through the structure without being exposed to the screens. Under the recommended design configuration the total length of the screen structure is estimated to be 440-feet.

The screens themselves will be 1.75-millimeter slot opening stainless steel wedge wire (profile bar) screens which have an open area of about 40-percent. The screens will be in 10-feet-wide by 10-feet-high sections. Above the screens will be 10-foot-wide by 8-foot-high blocking panels to extend to the top of the fish screen structure. The screens and blocking panels will be bolted to the structure to reduce construction cost and because maintenance activities can be performed during the non-irrigation season. The design incorporates a mobile crane capable of lifting 3,000-pounds at a 50-foot-reach would be required to remove and replace the screens and baffles.

The baffles for the recommended design will consist of a pair of perforated plates that will work together to produce adjustable orifices through which the water will flow. The bottom plate will be mounted in a fixed position and the second plate will slide vertically to adjust the orifice openings. The purpose of the baffles is to provide increased resistance to flow in areas of the screen where approach velocities are high and lower resistance to flow in areas where the approach velocities are low, thus resulting in a uniform distribution of flow through the screen. The screens and baffles together are expected to cause about 0.5-foot of head loss though the structure.

Fish Bypass. The purpose of the fish bypass is to return the screened fish and debris to the river in a safe manner. The recommended design of the fish bypass consists of an entrance structure, a bypass pipe, and an exit structure. The entrance structure is a 2-foot-wide rectangular channel at the downstream end of the "V" screens. The entrance structure transitions to a 48-inch-diameter concrete pipe that extends approximately 700-feet back to the river through a high bluff between the canal and river. The bypass pipe would outlet into the river about 350-550-feet downstream of the dam or new diversion structure depending on the final configuration. The fish bypass is designed to carry about 40 cfs of flow at full canal flow and will have an integrated fish trap for monitoring the effectiveness of the fish screen.

Check Structure. The check structure is necessary to raise the water surface in the screen structure to provide sufficient head for the fish bypass to function properly when the Yellowstone River is flowing at moderate to high flows (>28,000 cfs). The check structure consists of two 15-foot-wide by 17-foot-high radial gates that will be adjusted as necessary to maintain sufficient head for the fish bypass to operate when river flows are between 28,000 and 80,000 cfs. Flows above 80,000 cfs are relatively rare (1 out of 10 years) and short-lived (<6 days typical) and it is anticipated that the fish will just reside in the screening structure until the river drops again.

Screen Cleaning System. The screen structure will incorporate two cable operated brush cleaning systems to remove small debris from the screens. The brushes are designed to clean the screens during both upstream and downstream movement and debris dislodged from the screens is then carried back to the river through the fish bypass. It is anticipated that operation will be periodic on an as-needed basis, but continuous operation may be required during periods of high algae or perhaps during fall leaf off. In addition, an air blower system is included with the brushes that will be capable of loosening sediment near the invert of the fish screens so the sediment can be suspended in the water and carried downstream through the fish bypass and to the river.

Ice Concerns. Since the irrigation canal is only operated from April through September it is anticipated that the fish screen structure will not encounter ice conditions and ice should not be an issue.

Cost Estimate. TSC developed a revised conceptual cost estimate for their fish screen structure during winter 2005, and came up with an estimated construction cost of $11.1 million, with approximately $3,000 in annual operations & maintenance and energy costs.

Design Analysis Technical Review

For this design analysis the Corps was tasked with conducting an independent technical review of the conceptual fish screen structure design and to develop an independent detailed engineering construction cost estimate. This section of the report presents some of the pre-eminent technical review questions and issues that are expressed by each of the technical disciplines. These comments are not intended to be critical of the design presented by the TSC, but rather are offered as a list of things for the team to consider as the project moves forward into NEPA and more intensive and refined design efforts. A more thorough presentation is included in each of the individual appendixes.

Fisheries Criteria. The purpose of the fish screen structure is to prevent pallid sturgeon from entering the main canal. The critical design criteria to consider in designing the fish screen are the swimming ability and size of the fish being screened. Table 6 summarizes known data on swim speeds for pallid sturgeon broken down by three categories:

Burst Speeds - speeds which can be maintained for 15 seconds or less;
Prolonged Speeds - speeds which can be maintained for up to 200 minutes; and
Sustained Speeds - speeds which can be maintained for more than 200 minutes (Tunink, 1977).

For both pallid and shovelnose sturgeon the prolonged swim speed can be estimated at approximately 1 fps per foot of length of the fish (Kynard et al, 2002). Table 6 presents data on approximate sizes of different age classes of pallid sturgeon.

Table 6. Pallid Sturgeon Swimming Abilities

Age Class	Prolonged Speed	Burst Speed	Comments
Larvae	N/A	N/A	They “swim” up and down within the water column, not “swim” against flows
Fry (> 60 mm)	No information		
Juveniles	1.0 – 1.6 fps (a) 0.9 – 2.0 body lengths / second (b) 0.6 – 0.9 body lengths / second (d)	1.8 – 2.3 fps (a) 2.3 fps (b)	
Adults (mid-sized)	<4 fps (c) 2.1 – 3.8 fps (a)		
Adults (design-sized)[1]	5 fps (calculated)	7.5 fps (calculated)	Based on one body length per second for sustained speeds and 1.5 x sustained for burst speeds

a. Hoover and Kilgore; b. Adams et al., 1999; c. White and Mefford, 2002; d. Kynard et al, 2002

Table 7. Pallid Sturgeon Size by Age Classification

Age Class	Width	Height	Length	Comments
Larvae	2.0 – 2.5 mm (a)	2.0 – 2.5 mm (a)		Cylindrical in cross-section
Fry			Up to 60 mm	
Juveniles				
Adults (mid-sized)	Approx. 8 inches (a)		Up to 3 feet	20 – 30 pounds
Adults (design size)	9 – 10 inches (a)	8 – 9 inches (a)	Up to 5 feet (approx.)	50 - 70 pounds

a. Hoover and Kilgore; b. Adams et al., 1999; c. White and Mefford, 2002; d. Kynard et al, 2002

<u>Screen Structure Component Comments.</u> For purposes of organization design comments are organized by the various components making up the fish screen structure.

Trash rack Summary - The conceptual design calls for a trash rack with spacing of 8-inches which corresponds to the spacing between the railroad timbers that make up the trash rack on the exterior of the headworks structure. The trash rack is designed for a maximum velocity of 2.5 fps under full canal flow which should be low enough to allow any stray large fish to swim away from the trash rack without getting impinged or stuck in it. Smaller fish (incl. juvenile, fry and larval pallids) will pass through the trash and into the fish screen structure.

Trash rack Comments - Observations to consider during future design efforts:

- If an occasional large fish is able to enter the canal perhaps due to damage on the railroad timber trash rack providing larger openings, the current design does not have a provision to allow those fish to get back to the river. Since the trash rack is located approximately 500-feet downstream from the headworks structure there is a possibility that some fish could be trapped in that section of the Main Canal. Based on the velocity and turbulence within the diversion pipes it seems unlikely that those fish would be able to swim back through the pipes to the river. Recommend incorporating monitoring of the upper end of the Main Canal as part of the long-term monitoring program to determine whether this risk is truly a problem.
- Since the 8-inch spacing on the trash rack would not allow a 5-foot-long, 50-pound pallid into the fish screen facility consideration should be given to designing for a smaller 3-foot-long, 30-pound pallid which may lead to a smaller fish bypass pipe and some reduced cost.
- Consider evaluating possible placement of the trash rack structure on the river side of the headworks structure with ice protection provided by the railroad timbers.

Screen Structure Summary - The conceptual fish screen is a "V" configuration with two screens approximately 224 feet in length. The conceptual design screen material is stainless steel wedge wire with a 1.75 millimeter opening (0.069 inches) and a 40% open area. Most screening installations on dams, irrigation intakes, or pumps conform with National Marine Fisheries Service (NMFS) criteria for salmon fry (2.38-millimeters or 3/32-inch) or clean water act regulatory criteria (6.35-millimeters or 1/4-inch). The proposed 1.75-millimeter screen is designed to prevent pallid sturgeon larvae from entering the Main Canal, but there are no known design criteria for screening for pallid sturgeon larvae, nor are there criteria established by NMFS for the screening of salmon larvae.

There are very few facilities currently using small mesh screens, so information regarding their feasibility for both fry/larvae protection and for use within a turbid river system is very limited. Table 8 summarizes data for facilities known to use small screens and provides some comparison information.

Table 8. Comparison of Screens at Select Facilities				
Project	**Mesh Size**	**Facility Type**	**Location**	**Screen Target**
Intake Canal (proposed)	1.75 mm (inch)	Open Canal	Montana	Pallid sturgeon larvae
Leeburg Dam	1.75 mm		Oregon	Salmon fry
Walterville Dam	1.75 mm		Oregon	Salmon fry
Roosevelt CD	2.38 mm (3/32 inch)	Floating Pump	Montana, North Dakota	Debris / twigs
Various	6.35 mm (1/4 inch) (current Regulatory standard)	Intake Pumps (municipal and irrigation)	Missouri R, Mississippi R, Atchafalaya R watersheds	Pallid fingerlings

Screen Structure Comments - Observations to consider during future design efforts:

- Since little is known about pallid sturgeon larvae/fry swimming abilities consideration should be given to developing a scale model of the screen and testing how the larvae/fry respond. If the fish become impinged and/or are damaged by the brush system then consideration should be given to using a larger mesh and incorporating larval sampling to capture and entrained fish and return them to the river.
- In the absence of any other data recommend that NMFS criteria of 2.38-millimeters should be used for the screen material. This option would still be considered experimental since the standard screen size for regulatory purposes throughout the Missouri, Mississippi, and Atchafalaya River Systems (to protect endangered pallid sturgeon) is 6.35-millimeters, and the fact that the NMFS criteria was developed for salmonids not warm water fish, specifically pallid sturgeon. Using a larger screen size may reduce the overall length and cost of the screen structure.
- Consideration of larvae/fry and their mortality/injury rate in the canal may need to be should be included in the USFWS Biological Opinion as this would likely be categorized as "take."
- The approach velocity of 0.4 fps (NMFS criteria for salmon fry) may be too high if we extrapolate the swimming strength using the 1 fps per foot of fish length rule-of-thumb. A 25-millimeter-long pallid sturgeon fry would be able to swim about 0.08 fps (four times slower than salmon fry). Consideration should be given to conducting fry swim studies to better understand swimming strength or change the design to accommodate an approach velocity of about 0.1 fps.
- The conceptual design states that sediment deposition is reportedly negligible in the reach of the canal. However, given the small mesh size and alteration of hydraulics through the screen area, additional sediment evaluation is recommended to estimate removal requirements and potential screen blockage issues.
- To reduce risk of injury to impinged fish consideration should be given to the use of backside sprayers in lieu of the brushes.

- An open channel fish bypass should be considered in lieu of the pipe, and further evaluation of methods to reduce velocities within the fish bypass is recommended.
- The elevation of the top of the fish bypass pipe is about equal to water surface for 5,000 cfs flow (low flow) on the Yellowstone River downstream of the dam. Therefore, the bypass pipe will nearly always be filled with backwater from the Yellowstone River, even during low flow winter months. Freeze damage to the bypass structure may be an issue if provisions are not included to dewater the pipe. Slide gate closure would prevent backup into the canal.
- If the rock ramp alternative and fish screen are implemented together consideration of the optimal location for the fish screen should be evaluated.

Design Analysis Detailed Construction Cost Estimate. As part of this design analysis a detailed engineering construction cost estimate was developed which estimated the cost of the fish screen structure and associated facilities at $9.66 million including a 20% contingency.

PRELIMINARY ROCK RAMP DESIGN

This section of the report is dedicated to presenting the preliminary (10%) design information for an engineered rock ramp that would be retrofitted onto the existing Intake Dam to provide maximum opportunity for fish passage by pallid sturgeon and other native fish. A rock ramp is a flat-sloped, engineered rock riffle that is sometimes designed to mimic naturally occurring riffles elsewhere on a river system. Rock ramps have recently gained popularity as a preferred method of providing fish passage over low-head structures because of their relatively simple design, flexibility to adapt to a wide array of applications, relatively low maintenance, and natural appearance and function. Rock ramps are also being developed to address boating safety concerns with some traditional drop structures and even have been designed as recreational canoe/kayak/rafting runs. For purposes of this report the rock ramp design elements for each technical discipline are presented separately. A more thorough presentation is included in each of the individual appendixes.

Fishery Design

The most critical design element for the fish passage design is flow depth and velocity and how that relates to the swimming strength of the target species. For the Lower Yellowstone Project the target species is the endangered pallid sturgeon which has been documented to have a maximum burst speed of approximately 7.5 to 8 fps. However, fish swim studies conducted by Kynard in 2002 demonstrated that adult pallid sturgeon in flume tests could maintain position and swim forward "with apparently very little effort at flow velocities of 6 fps," so the upper limit of their burst speed may not be fully understood. For purposes of this design the target flow velocity of 7.5 fps was used for design conditions.

The fishery input into the rock ramp design is incorporated into the information presented later in the Hydraulic Design section. However, there are some additional observations to consider during future design efforts:

- Further evaluation of velocities and turbulence in the rock ramp should be conducted to provide assurance that the design velocities are being met at least 80% of the time (USFWS). Physical modeling may be required to overcome model shortcomings.
- Flow velocities at the crest of the dam appear to be the area of greatest concern, so additional measures to reduce velocities on that section of the ramp should be considered.
- Consider collecting topographic, flow, and velocity data for multiple natural "rocky runs" where pallid sturgeon are known to successfully negotiate and incorporate findings into the rock ramp design.
- Consider opening up the high flow side channel to enhance opportunities for pallid sturgeon to utilize that channel for upstream and downstream migration.

Hydraulic Design

The hydraulic design of the rock ramp evaluated a series of slopes and drop heights were tried with the ramp in an attempt to minimize peak flow velocity and the corresponding rock size required to resist that velocity. Slopes of 5%, 3.33%, and 2% were all evaluated. For each slope the ramp slope is achieved through alternating steps (drops) and pools. Drop heights of 0.5 ft and 1 ft were evaluated to see how they affected the resulting velocities. Research into existing, installed ramps on the Red River of the North and guidance developed by Luther Aadland of the Minnesota DNR (Buesing, 2006 and Breining, 2003) found that a 1 ft drop is common among those ramps. Compared to the Intake Diversion Dam application, the Red River ramps exhibit similar structural heights with slightly lower unit discharges. Design and analysis results are summarized as follows:

Top of Ramp. Current elevation varies from 1987 to 1989; the new dam crest is designed with a top elevation of 1989. Placing the dam crest at 1989 will provide sufficient head for the existing intake structure.

Toe of Ramp. Current elevation is estimated at about 1980 (based on the old channel surveys of limited detail in the near dam vicinity). A tie-in slope of 3H on 1V or similar for rock ramp stability should be used to reach the bottom of the scour hole located downstream of the dam. According to the old survey data, the elevation 1980 isn't reached until about 400 ft from the dam.

Ramp Shape. The rock ramp will be "U" shaped, although unbalanced to maintain the main flow channel along the irrigation intake bank. The ramp shape should be optimized to provide the maximum depth-velocity diversity in detailed design. Due to the width of the river, it is anticipated that a significant portion of the center ramp will be relatively flat (parallel to the upstream face of the dam). Because of the "U" shape the width of the ramp center at the bottom of the slope will be approximately 550-feet. A conceptual ramp layout is illustrated in Figure 15. Ramp details for comparison between different configurations of slope and drops are shown in Figure 16 and Table 9.

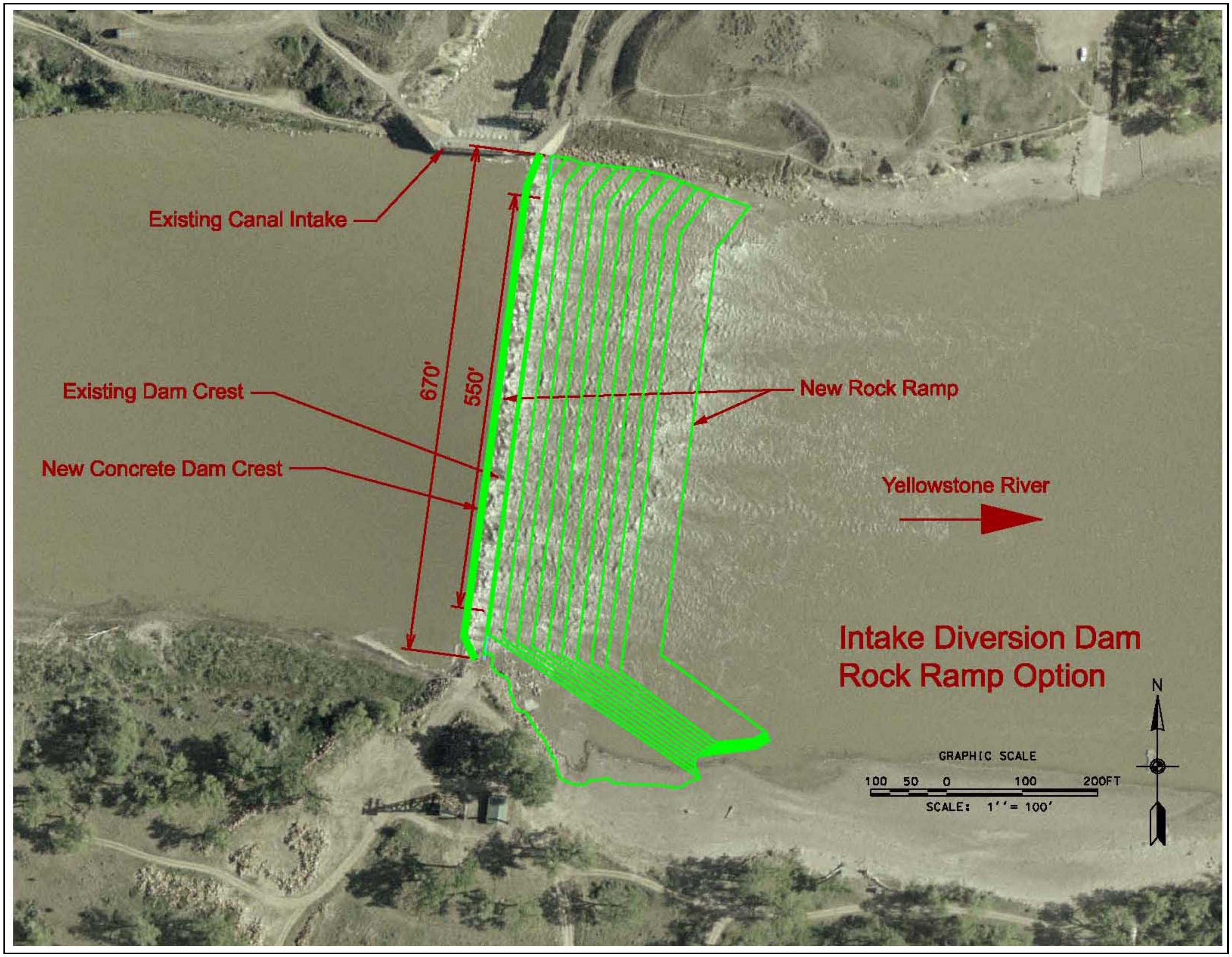


Figure 15. Conceptual Layout of Rock Ramp

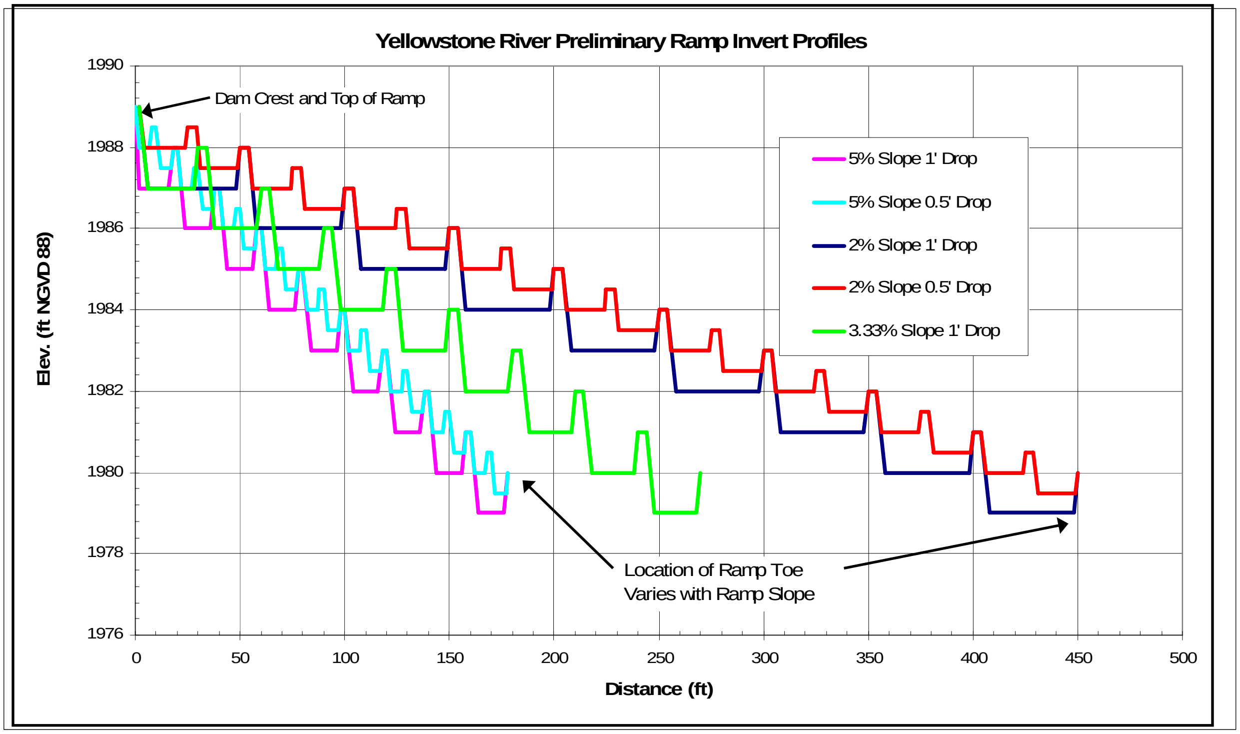


Figure 16. Yellowstone River Preliminary Ramp Invert Profiles

Table 9. Ramp Layout for Various Slopes

Alternative	Ramp Length (ft)	Length Between Steps (ft)	Number of Boulder Rows
5% Slope, 1 ft drop	180	20	9
5% Slope, 0.5 ft drop	180	10	18
3.33% Slope, 1 ft drop	270	30	9
2% Slope, 1 ft drop	450	20	9
2% Slope, 0.5 ft drop	450	10	18

Ramp Steps. The steps on the rock ramp would be formed by large boulders with gaps in between to allow the passage of pallid sturgeon through the openings. A 4’ minimum diameter boulder is placed to form each “step” in the ramp profile. The boulders are not solid but would block the bulk of the flow. The boulder would be 0.5’ to 1’ above grade on the upstream side. The boulder crown would be 1’ to 2’ above the grade of the downstream pool. Smaller "riprap" rock would be placed to form the flat pool between each of the steps. A conceptual layout of the ramp plan and profiles for the boulder placement is shown in Figure 17.

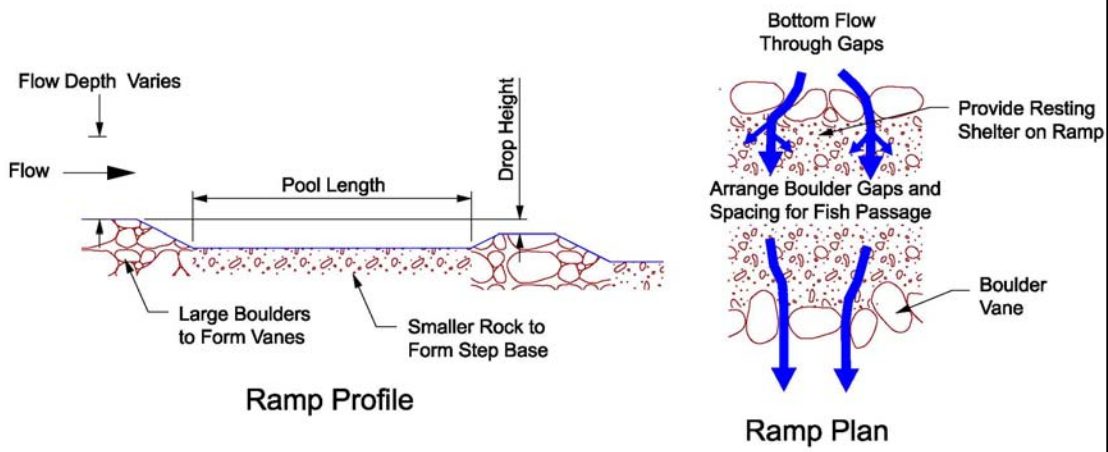


Figure 17. Conceptual Ramp View

Hydraulic Modeling. The HEC-RAS model was used to approximate hydraulic conditions within the rock ramp for various slopes and compute flow velocity. Due to the fact that HEC-RAS is a one-dimensional model, accurately evaluating the flow turbulence and velocity variation in both the horizontal and vertical directions is not possible. However, the HEC-RAS model can be used to produce reasonable estimates of average velocity and depth on the ramp and is suitable for use with comparing ramp conditions for various geometries.

Model Roughness - The rock ramp is expected to have higher roughness values compared to the existing channel due to the rock size and turbulence within the ramp flow. However, overestimating the roughness will cause the model to underestimate the flow velocity. Consequently, ramp stability would be overestimated. Guidance available relates rock size to roughness using the Strickler method (USACE, 1994, eq. 5-2). Computations determined a roughness value of 0.036 for 24 inch D_{100} and 0.042 for 48 inch D_{100} size rock. Since lower roughness values will result in the maximum velocity, a roughness value of 0.036 was used for the entire ramp.

Model Geometry - For the 10% design, no grading plans were developed to reflect the proposed ramp configuration. Therefore, the rock ramp was modeled with HEC-RAS by using a channel modification option, but because the option is limited to simple channels, the center channel section of 560 feet in width was assumed to represent the entire ramp. This bottom width was selected as reasonable for the existing site reflect flow area and concentration on the ramp.

Model Results - The results from the HEC-RAS model were used to evaluate the maximum rock size required for stability and to evaluate the velocities for fish passage. In summary the velocities for all ramp configurations were very similar with some apparent advantages to the 1-foot drop over the 0.5-foot drop due to only half of the number of steps resulting in fewer turbulence zones. The dam begins to become "drowned out" at flow rates higher than about 100,000 cfs where high tailwater results in reducing the flow velocities on the ramp. HEC-RAS output plots of computed water surface elevations and computed average velocities for the 5% slope with 1 ft drop are shown in Figures 18 and 19 respectively. Based on the computations a rock size of 3 to

4-feet is recommended for the ramp boulders (especially in the upper sections of the ramp where velocities are the highest). Constructing the entire ramp from 4 foot boulders is probably cost prohibitive. Based on the analysis, a rock size of 2 feet is recommended for the remainder of the ramp. Using the Ishbash method and HEC-RAS computed velocity; the determined D50 rock size for 100,000 cfs flow was about 1.5 feet. Stability for the 2 foot diameter rock is questionable for flow events in excess of 60,000 cfs. Future efforts will revise the rock size required for stability. However, it is likely that entire ramp stability for events in the critical flow range before the ramp is submerged (roughly greater than 80,000 - 100,000 cfs) is not feasible without using rock approaching 3 foot diameter for the entire upper portion of the ramp. For reference purposes, 100,000 cfs is about a 10-year event.

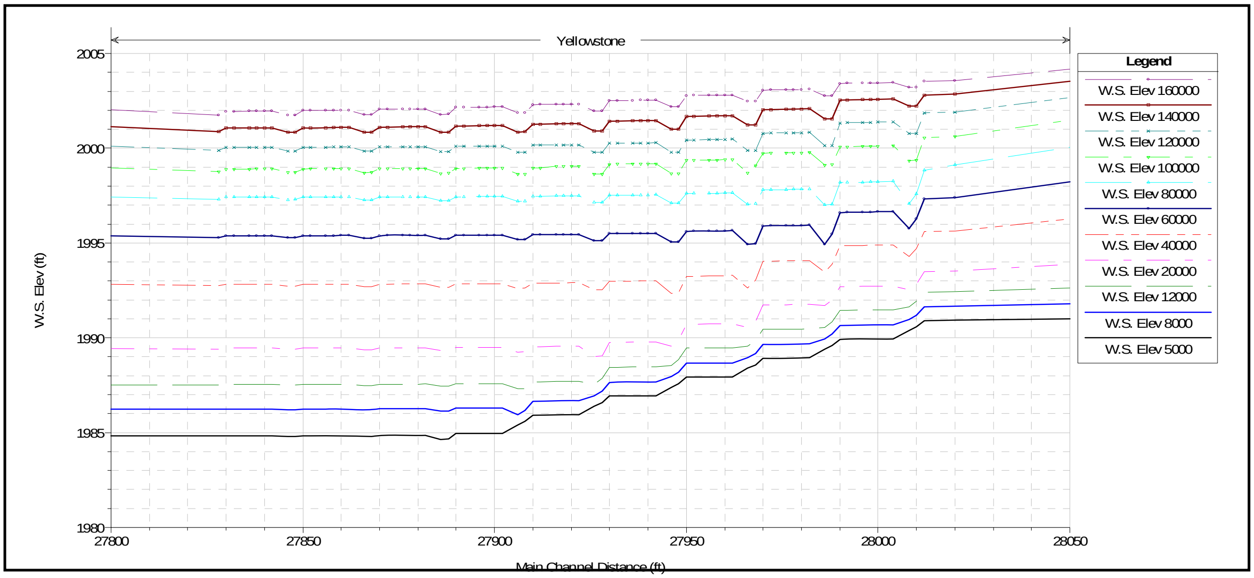


Figure 18. Computed Water Surface Elevation – 5% Slope Ramp, 1' Drop

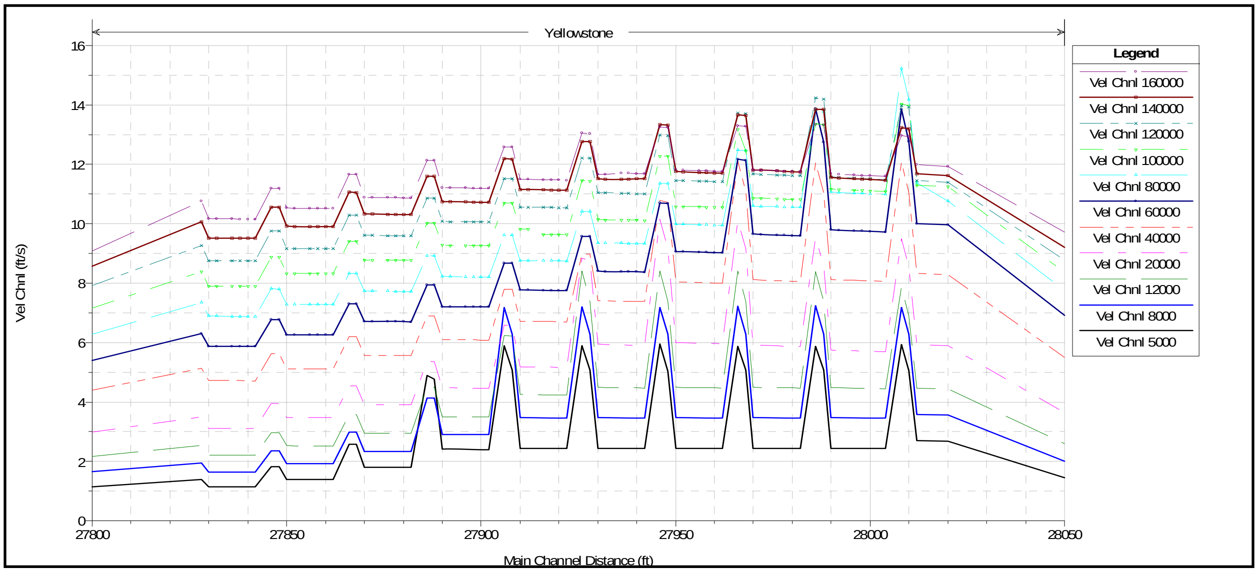


Figure 19. Computed Flow Velocity – 5% Slope Ramp, 1' Drop

Ice Stability/Resistance. Based on the ice analysis conducted by CRREL (Appendix D), the boulder size required for ice stability is estimated to be in the range of 4-6 ft diameter. This correlates fairly well with dam project history, where the larger rocks placed are on the order of 3 ft diameter. As the maintenance record shows, the dam crest riprap has been moved by ice and high flow conditions. Use of natural rock boulders or a simulated rock formed with concrete will provide stability for the boulder steps. In between the steps, loose rock riprap of a much smaller diameter is proposed. Ice damage may occur to portions of the ramp with the smaller rock. In addition the dam face and crest will be capped with reinforced concrete to provide added durability to the structure.

Design Analysis Detailed Construction Cost Estimate. As part of this design analysis a range of detailed engineering construction cost estimates were developed for the rock ramp alternative. These estimates all include a 20% contingency and are summarized in Table 10. The estimates do not include the cost of the fish screen which was evaluated separately.

Table 10. Construction Cost Estimates for Rock Ramp Alternative with Various Slopes and Construction Materials

Alternative	Estimated Cost [$ millions]
5% Slope [Guernsey large stone, local riprap]	12.53
5% Slope [Local large stone, local riprap]	11.18
5% Slope [Concrete fab-form]	17.29
2% Slope [Guernsey large stone, local riprap]	15.73
2% Slope [Local large stone, local riprap]	14.38
2% Slope [Concrete fab-form]	20.38

PRELIMINARY RELOCATE DIVERSION UPSTREAM DESIGN

This section of the report is dedicated to presenting the preliminary (10%) design information for removing the existing Intake Dam and relocating the diversion point upstream to a location where water can be diverted from the river without the need for a dam. Removing the existing dam would restore the river channel to a near natural condition which would likely translate into providing the ideal situation for upstream fish migration by pallid sturgeon and other native fish. Relocating the diversion structure upstream would require the construction of approximately 2-miles of new canal to connect the diversion point to the existing canal. For purposes of this report the relocate diversion upstream design elements for each technical discipline are presented separately. A more thorough presentation is included in each of the individual appendixes.

Fishery Design

Removal of the existing Intake Dam and restoring the river to a near natural condition is the ideal situation from a fishery perspective. The new diversion and canal would include a fish screen structure as discussed earlier and the same comments would apply for potential consideration.

Hydraulic Design

The hydraulic design of the relocate diversion upstream alternative evaluated hydraulic diversion capacities under low flow conditions and development of a design alignment and profile for the new 2-mile section of canal from the new diversion point to the existing canal. Since diversion of flow would need to be achieved without the aid of a dam a larger headworks structure (17, 5-foot-diameter pipes instead of the existing 11) would be required and would incorporate a drop structure immediately downstream of the diversion pipes to establish full flow depth in the canal. In addition the alignment for the new canal would need to cross the Yellowstone Valley Railroad at two locations (utilizing inverted siphons consisting of 5, 96-inch-diameter concrete pipes per siphon) because the rail line runs right along the bank of the river prohibiting construction of a canal in that vicinity. At the downstream end of the new canal a drop structure would be necessary in order to tie into the invert of the existing canal. In addition, the canal excavation would require excavating a vertical cut of over 60-feet for a long section of the canal because of the topography on the landward side of the rail line. Plan views of project features and the new canal alignment are illustrated in Figure 20. Design and analysis results are summarized as follows:

Hydraulic Modeling. The HEC-RAS model was used to approximate hydraulic conditions for the new upstream diversion, size the structures associated with the diversion and new canal, and design the proposed canal segment. Design of project features was based on attaining the canal diversion flow of 1,400 cfs at the minimum Yellowstone River flow of 5,000 cfs. As discussed in the *Existing Conditions HEC-RAS Model* section, the available survey data did not include Yellowstone River bed topography. Therefore, the accuracy of the computed Yellowstone River stage-flow relationship at the proposed diversion site is limited.

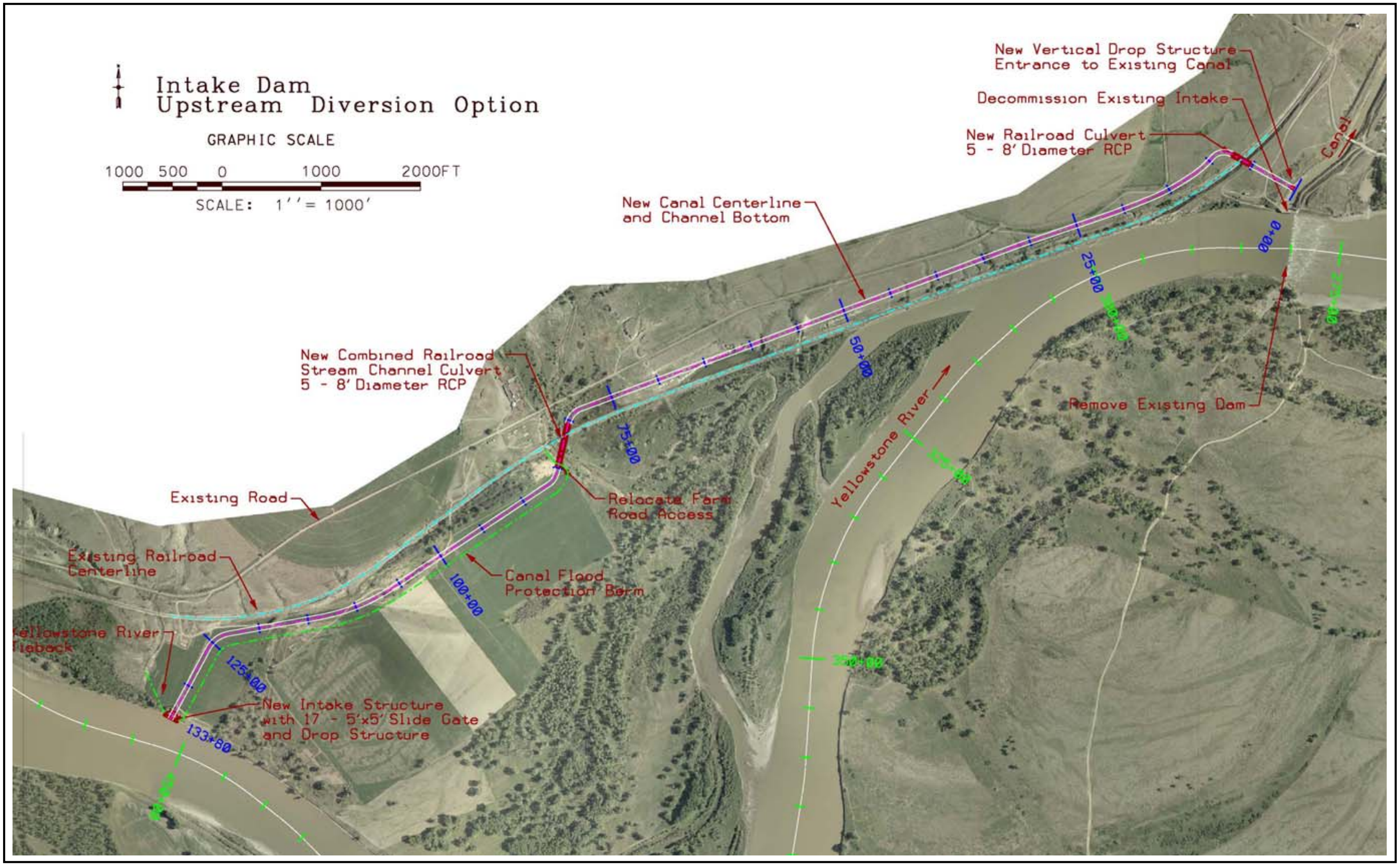


Figure 20. Plan View of Relocate Diversion Upstream Alternative

Model Roughness - The HEC-RAS model uses a Manning roughness value of 0.030 for the channel region of the new canal. The roughness parameters established for the model were similar to the previous modeling effort. Since the canal flow level is relatively constant, vegetation growth should be minimal and a roughness value lower than the Yellowstone River is expected.

Intake Dam - The new model assumes that the existing dam would be removed entirely. Complete removal may not be preferable due to concerns with erosion, bank stabilization, and impact to the Yellowstone River. However, complete removal of the dam causes lower upstream water surface elevations for evaluating diversion capability, and produces conservative cost estimates, so complete removal was assumed for this analysis.

***NOTE**: Dam removal would almost certainly result in some bed and bank erosion upstream of the existing structure. Such erosion was NOT included in this analysis. Detailed analysis would be required to evaluate erosion potential due to dam removal and also optimize dam removal.*

New Canal. The new canal will have a 50 foot bottom width and 2H on 1V side slopes at a longitudinal slope of 0.00013 ft/ft. The canal includes a 16 foot wide maintenance access road located 12 feet vertically above the canal invert. On the opposite bank, the side slope length is over 60 feet for a substantial length. The canal will include a mid-slope channel and berm to intercept side slope flow and prevent slope erosion. Canal invert elevation was designed to allow diversion from the Yellowstone River at 5,000 cfs. Excavation quantities estimated along the proposed alignment are as follows:

New Canal Excavation – 3,720,000 cubic yards of cut.
New Canal Length – 13,400 feet including all structures.

The new canal profile is illustrated in Figure 21. Typical sections are illustrated in Figures 22 and 23.

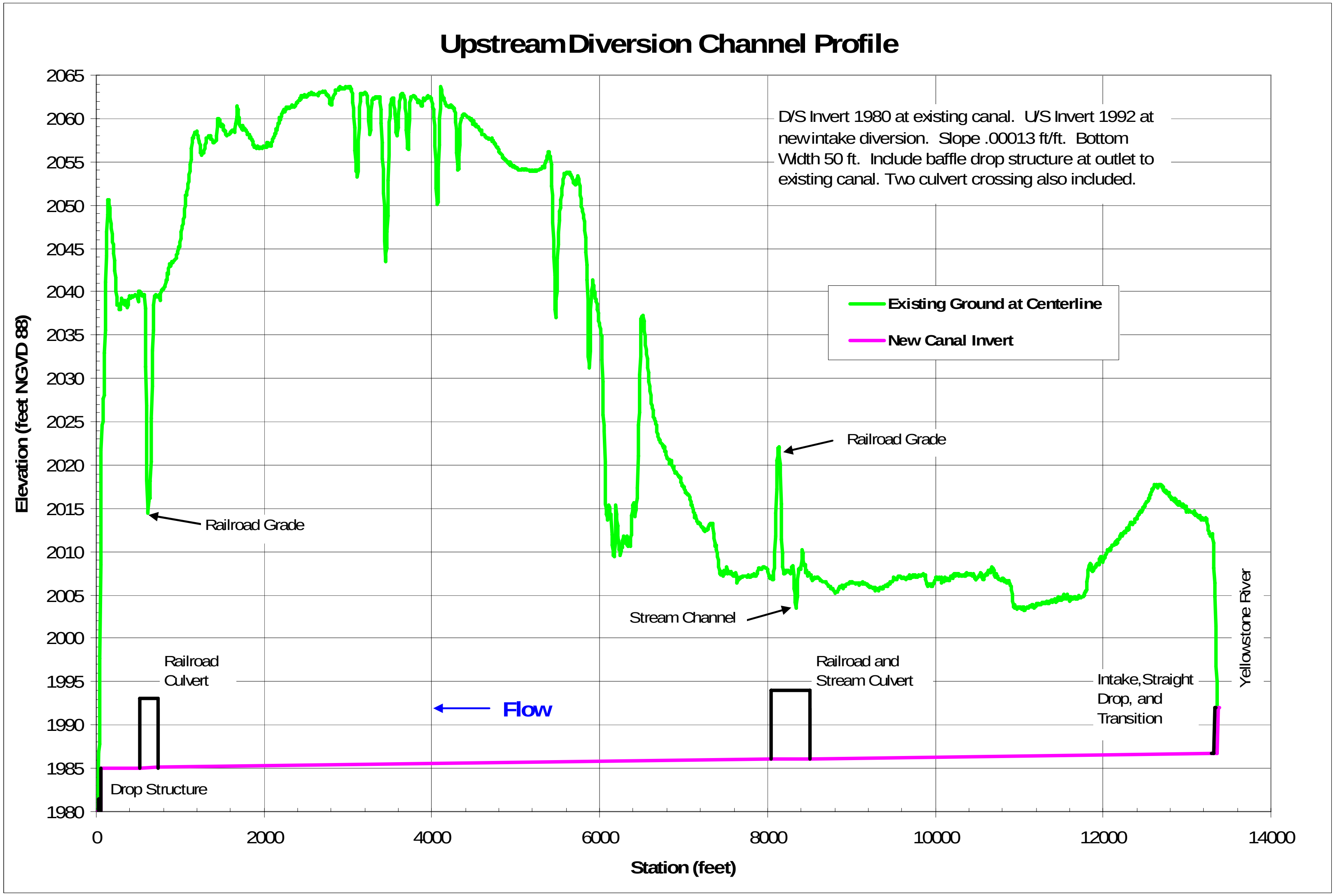


Figure 21. Profile of New Canal for Relocate Diversion Upstream Alternative

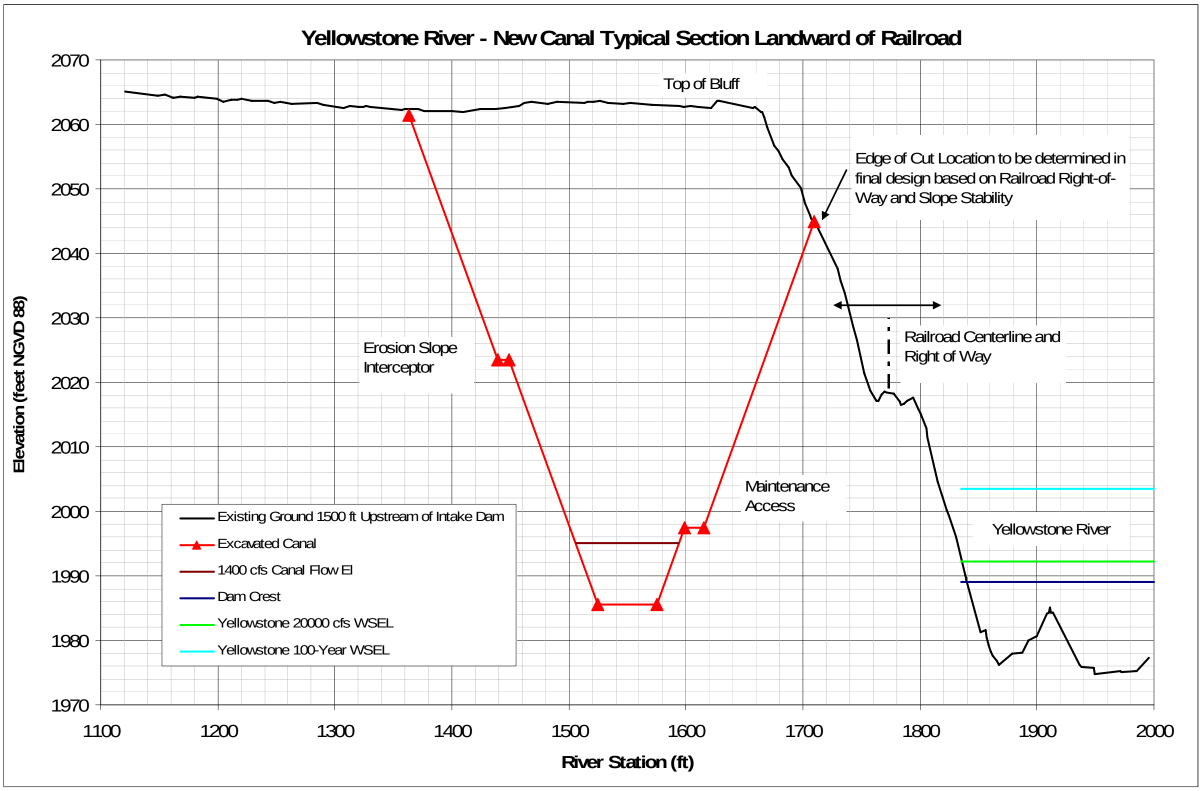


Figure 22. Typical Cross Section of New Canal

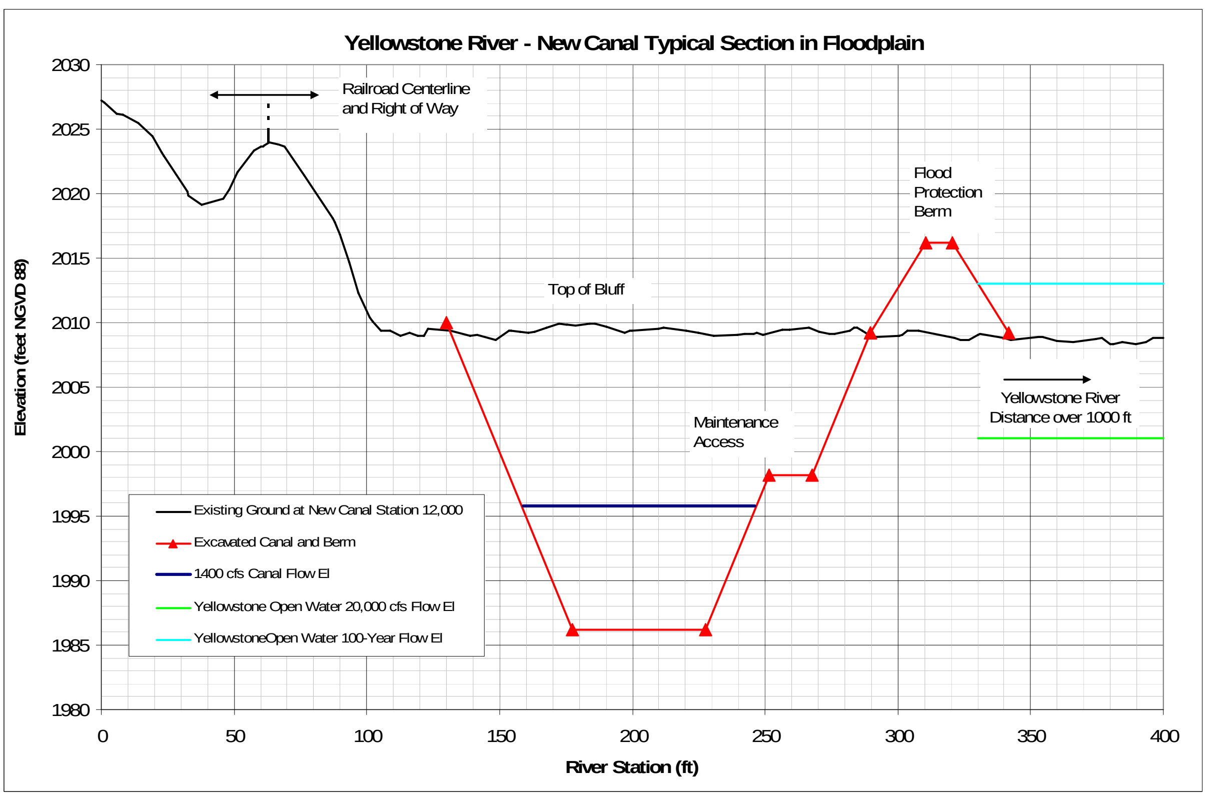


Figure 23. Typical Cross Section of New Canal in Floodplain

New Intake. A new intake headworks structure would be required on the Yellowstone River at the upstream diversion location. The new structure is rectangular with 17 gates. Assuming 4 feet between each gate, the total length perpendicular to the river is about 159-feet (including 5-feet on each side of the outside gate structure). Top of structure is elevation 2016 (about 4 feet above existing ground elevation). The new intake structure will include a straight drop structure on the downstream side of about 3.3-feet vertical drop. Gate invert elevation is assumed as elevation 1992 based on approximate channel bottom elevation. No channel survey data was available, so elevations are approximate based on available information.

Railroad and Tributary Crossings. Two crossings of the railroad are required with the new canal alignment. For the conceptual design, the maximize culvert size beneath the railroad was assumed to be an 8 foot diameter. This assumption is based on boring/jacking limits as stated in the structural appendix.

NOTE: Replacement of the five 8' diameter culverts is possible with a single large siphon. Preliminary analysis indicated that the siphon diameter would be about 20'. Due to concerns with construction beneath the railroad, this option was not pursued for the conceptual design. Detailed design should investigate this option.

Drop Structures. The first drop structure, located just downstream of the new intake structure, is a SAF straight drop structure following criteria illustrated in HDC Sheet 623 - 624-1 (WES, 1988). The structure width matches the intake width with vertical sidewalls. Downstream of the structure, a transition is required from the drop to the new canal. The structure would include grading to avoid using wing walls through the transition.

The second drop structure, from the new canal to the existing canal, is a SAF straight drop structure following criteria illustrated in HDC 624. A baffled chute structure may be preferable for this location, may cost less, and should be evaluated in future design. The structure is rectangular with a 35' bottom width. Structure width is reduced from canal bottom width to lower upstream flow velocities and canal erosion potential. Structure width will be revised in future design. Rock riprap is included upstream of the structure for 25 feet and downstream of the structure for 20 feet. The drop height is 8 feet. The structure length was estimated based on roughly following the criteria developed for SAF basins in HDC Sheet 623 - 624-1 (WES, 1988).

Floodplain Protection Berms. Two berms are required to prevent Yellowstone River flooding from damaging the canal, one parallel to the canal and the second berm ties off to high ground upstream of the intake. The location of both berms is illustrated Figure 18. The preliminary design berm height was estimated based on the 100-year open water elevation of 2013 in the vicinity of the new diversion. Ice affected stages were not evaluated. If an additional 3 feet is included for freeboard, the top of berm elevation is roughly 5 – 7 feet above existing grade. The berm is parallel to the canal and is installed from the new intake structure at the Yellowstone River downstream to the railroad culvert crossing at station 82+70. Downstream of this location, the canal is protected by the railroad embankment. The berm will be earth only, not designed to resist ice forces as it is remote from the river. Construction of the berm will place significant fill within the floodplain and will probably impact Yellowstone River flood elevations and floodway. Mitigation for construction of the berm will probably be required.

A second berm is required upstream of the intake structure to protect the canal from direct flooding and ice jam attack. In order to accommodate rock riprap protection for this berm, a top width of 15 feet is required. The berm proceeds from the structure northwest toward an existing high knoll over a distance of about 470 feet. Quantities are estimated as follows:

Canal Floodplain Protection Berm Length – 5,220 feet.
Canal Floodplain Protection Berm Fill – 24,200 cubic yards (average 125 sq ft per ft berm)
Yellowstone Upstream Berm Length – 470 feet
Yellowstone Upstream Berm Fill – 1600 cubic yards (average 92 sq ft per ft berm)

Rock Riprap. Rock riprap is included at several locations where localized turbulence may occur. Rock riprap is also required to protect the canal from ice jam flooding on the Yellowstone River along the Yellowstone River upstream berm.

In the vicinity of structures for erosion protection due to turbulence, all rock riprap is a 12" layer thickness. Rock riprap is also included on the Yellowstone River flood protection berm located upstream of the intake structure. This berm serves to protect the canal from open water and ice jam flooding. The downstream side of the canal is assumed to be protected from ice jam action as it is located away from the Yellowstone River and in the flow shadow of the upstream flood protection berm and intake structure. Rock for this location is a 4 foot layer thickness to resist the ice forces.

Geomorphology

The geomorphology design of the relocate diversion upstream alternative evaluated potential for channel bed scour at the location of the removed dam and bank stabilization requirements associated with the new diversion structure. Design and analysis results are summarized as follows:

Bed Scour after Removal of the Existing Dam. Removal of the intake diversion will result in an increased sediment transport capacity in this reach. For a constant flow and sediment size, the ratio of the natural river slope to the backwater slope caused by the dam provides an estimate of the increased sediment transport capacity after dam removal. Using this ratio and the water surface profiles from the TSC original hydraulic model produced estimated six-fold, three-fold, and double increases in sediment transport capacity for flows of 5,000 cfs, 15,000 cfs, and 20,000 cfs respectively. In addition, some channel widening can be expected to occur in the vicinity of the dam based on comparison with channel widths upstream and downstream of that location. After dam removal, the reach near the dam will likely conform to the planform geometry of the upstream and downstream reaches.

Channel Stabilization at the Upstream Diversion Location. Since diversion of flow would need to be accomplished without the aid of a dam even under low flow conditions it is recommended that a dike field be constructed on the right bank in the vicinity of the new diversion headworks structure to prevent migration of the right bank landward and keep the thalweg along the left bank. This design includes a series of four to six dike structures along the right bank. The average velocity in the vicinity of the diversion will likely increase somewhat, but since the dikes are submerged at about a 2-year flood velocity impacts at higher discharges are expected to be incremental. The design would induce sediment deposition in the structure dike field. Some ice damage may occur to these structures, and routine maintenance should be anticipated to maintain full diversion capability. The preliminary design also calls for 2 to 3 rock sills constructed in the bed of the river to prevent downcutting of the channel which would adversely affect the hydraulic diversion capacity of the headworks structure. These rock sills will be constructed at the same grade as the current channel bed and should have little or no impact on flow velocities in that area. Revetment is necessary along the left bank in order to prevent erosion that may occur as the velocity increases. A plan view of the project features and typical sections are illustrated in Figures 24 through 26.

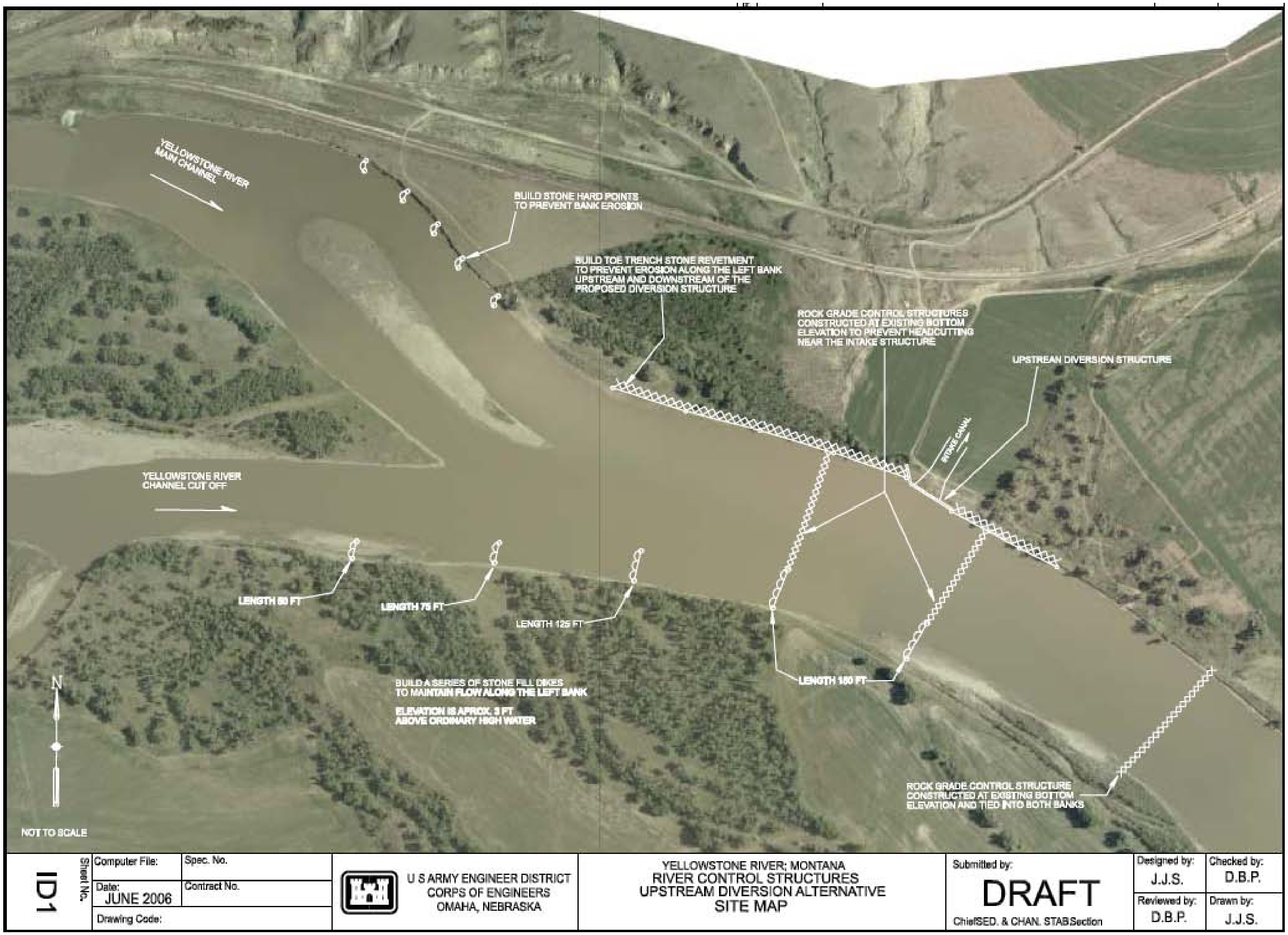


Figure 24. Plan View of Channel Stabilization Structures at Upstream Diversion Location

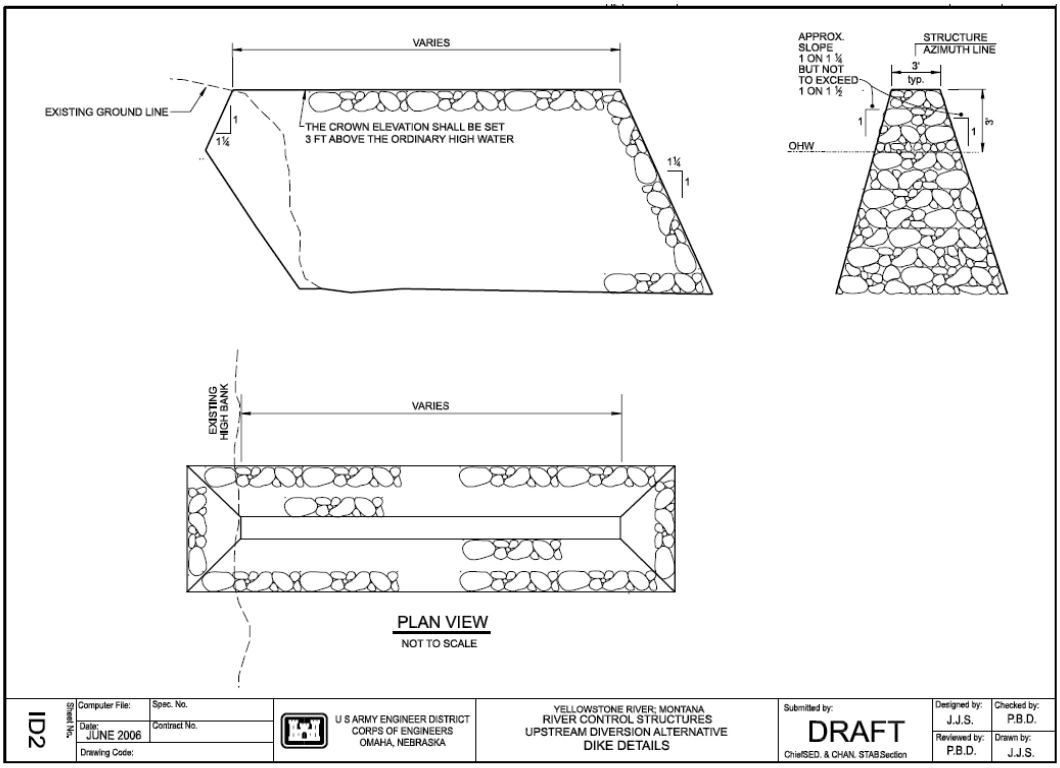


Figure 25. Typical Cross Sections and Details for Rock Dikes

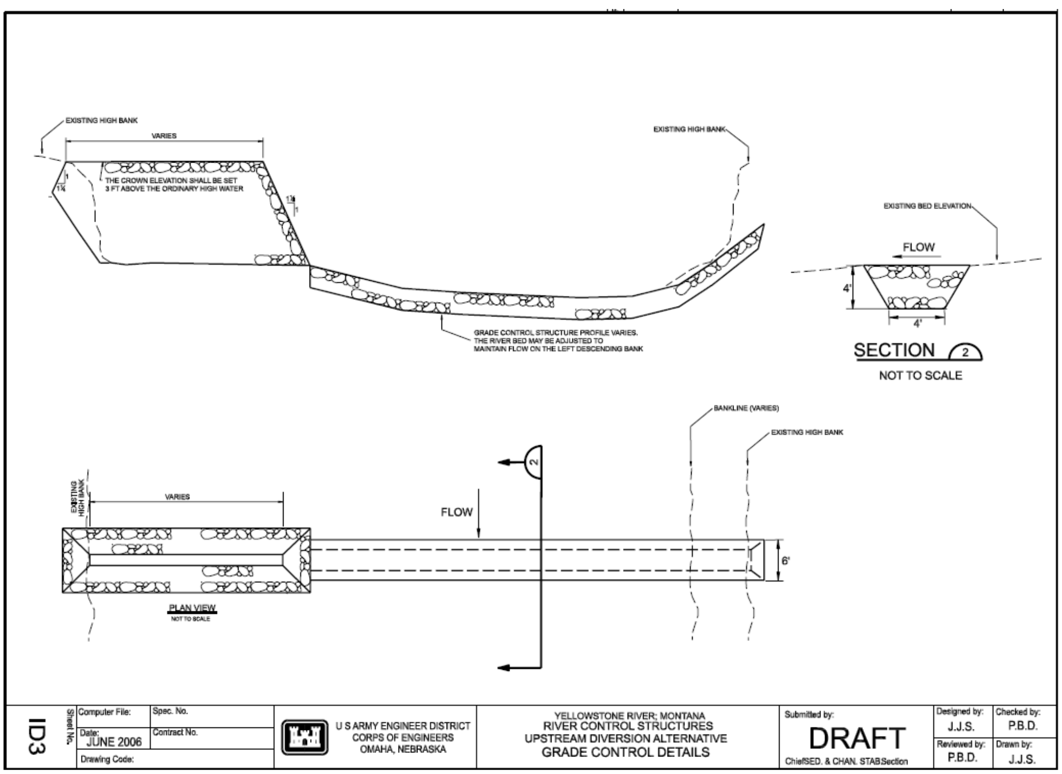


Figure 26. Typical Cross Sections and Details for Rock Sills

Design Analysis Detailed Construction Cost Estimate. As part of this design analysis a detailed engineering construction cost estimate was developed for the relocate diversion upstream alternative which estimated the cost of the facilities at $43.15 million including a 20% contingency. The estimate does not include the cost of the fish screen which was evaluated separately. It also does not include the cost of real estate required for the project which could contribute to the overall cost.

Copy furnished:

Mr. Brent Esplin, Project Manager
U.S. Bureau of Reclamation
Montana Area Office
P.O. Box 30137
Billings, Montana 59107-0137

Appendix A
Fisheries

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

US Army Corps
of Engineers ®
Omaha District

INTAKE DAM FISH PASSAGE AND SCREEN

Design Analysis - Fisheries Review and Recommendations

1. Background

Details regarding adult pallid sturgeon life history criteria are very limited, so specific pallid sturgeon requirements often need to be extrapolated from data collected when pallids are captured in the field, laboratory tests, data from surrogate species, or best scientific opinion. Even less is known about the needs of larval, fry, and juvenile pallid sturgeon. Additionally, little has been documented regarding spawning cues for the pallid sturgeon, the frequency of spawning, the exact timeframes related to spawning or spawning locations, although hypotheses abound. Because of these uncertainties, the development of engineered structures to facilitate the migration and spawning of these rare fish is not an exact science. Therefore, to the extent feasible, **designs should be flexible to allow for modification or adjustment as more information becomes available from post-construction monitoring and sturgeon use of the structures**, as well as other ongoing research.

1.1 What we know.

Pallid sturgeon (*Scaphirynchus albus*) are known to be present in the Yellowstone River, and, based on telemetry studies, do appear to be restricted from passage above the Intake Diversion Dam in most years. Based on captures of adult pallid for brood stock use, pallid sturgeon can be found in spawning condition within the lower Yellowstone River and the Yellowstone – Missouri River confluence area (Bramblett, R.G. 1996.; King and Wilson, 2002; Backes et al, 1994). During high flow years, there limited fish passage is possible either over the Intake Dam or through a connected high-flow side channel. Backes et al (1994) tagged 1373 shovelnose sturgeon below the Intake Dam. Three pallids were also netted below the dam. Nets were later used above the Intake Dam, recapturing 695 shovelnose, including five that were tagged below Intake Dam. No pallids were netted above the dam. In another study by Helfrich et al (1999), of the 4080 fish marked downstream from Intake Dam, 17 (four species) were recaptured above the dam. Although the dam didn't seem to restrict the distribution pattern for some strong-swimming fishes, weaker-swimming fishes, such as sauger, walleye, shovelnose sturgeon, and paddlefish did show restricted distribution patterns and were likely restricted by the series of Yellowstone River dams (Helfrich et al, 1999).

Pallid sturgeon are bottom-dwelling warm-water fish with poor swimming abilities compared to salmonids. The majority of fish passage literature and guidance is based on salmonid swimming and jumping capabilities. Fish passage for warm-water fish is still a relatively new field by comparison, although several structures have been constructed within the United States and Europe. Incidental passage of sturgeon species (*Acipenser spp.*) have been documented in some fish passage facilities. The literature has no record of fish passage by *Scaphirhynchus* sturgeon species, but fish passage structures are uncommon within the range of pallid and shovelnose sturgeon, so the opportunity for upstream passage through structures (other than locks) is lacking.

The Intake Diversion Canal off the Yellowstone River entrains large quantities of native and non-native fish. In field sampling, a total of 8374 fish were netted from 2 – 4 of the 11 canal intakes during the 1997 irrigation season, and 4529 fish in 1998 (Hiebert et al, 2000). Based on extrapolations of this data for all the canal intakes across the entire irrigation season, annual entrainment in the canal was estimated to range from 382,609 to 809,820 fish from 1996 through 1998. Of the 34 species collected, 25 were native species. Species entrained varied by year, by season, by time of day, and by discharge (higher entrainment during lower Yellowstone River flows). Although shovelnose sturgeon and paddlefish were collected, no pallid sturgeon were collected during this study. Sturgeon chub, a species of concern, was within the top 6 fish species entrained within the canal. No estimate was made on how the numbers of fish entrained in the canal related to the numbers of fish in the Yellowstone River, i.e. what percentage of the native fish population was being affected.

1.2 What we don't know / what we can infer

We don't have any data indicating that *Scaphirynchus spp*. utilize fish passage structures because none have been built in areas of anticipated pallid sturgeon or shovelnose sturgeon migration. Studies by Mefford and White (2002) indicate that shovelnose sturgeon can utilize certain fish passage facilities in the laboratory (rock ramp, denil, etc). Sturgeon successfully negotiated the range of velocities tested (0.8 – 6.0 fps) over all substrates (smooth, fine sand, coarse sand, gravel, and cobble). Flow orientation and attraction became strong at 2 fps and remained strong at higher velocities. Movement success declined with increasing velocities, from 81 – 87% at 4 fps to 47% at 6 fps. We can infer that they would act similarly in the wild.

We have some limited information on adult pallid sturgeon swim capabilities, although there is less information on juvenile swim capabilities, and no information on pallid fry swim capabilities.

With regard to fish entrainment within the canal, the lack of pallid sturgeon data in the Hiebert (2000) study does not necessarily mean that pallid sturgeon do not become entrained. Pallid sturgeon are rarely collected in any sampling effort, and since shovelnose sturgeon were collected, it's logical to assume that pallid sturgeon may also be entrained within the canal. The study by Jaeger et al (2---) indicated that 3 of 21 hatchery-raised telemetered pallid sturgeon were relocated within the Intake Canal, however the study leaves the reader with unanswered questions. For instance, pallid sturgeon were released in three locations upstream from the canal – were the pallids captured in the canal released from the same location? Were these fished tracked into the canal to determine if they ended up in a canal arm, or followed the canal all the way to the Missouri River? **Additional studies are needed to further gain insight into the entrainment potential of pallid sturgeon, including tracking of the sturgeon to determine their ultimate fate in the canal system (or if they end up in the Missouri River).**

Since larval drift studies were not done in the canal, we also don't know whether larvae enter the canal and become entrained, although since larvae tend to drift with the flow, it's logical to suspect that some percentage of native larvae do enter the canal. How important that percentage is (e.g. 5%, 40%, or 80% of the larval drift?) to the overall native fish community is unknown due to the current lack of data. Currently, capture of pallid sturgeon above the intake dam is rare, with the last pallid being captured in 1991. Therefore, entrainment of juvenile or larval pallid sturgeon in the intake canal is not currently an issue, but this could become an issue should fish passage at intake dam be successfully constructed. If larvae sampling is done in the canal, we could assume that if shovelnose sturgeon larvae are found within the canal, then pallid larvae will also be similarly entrained (once pallid passage above intake is assured) since shovelnose and pallid sturgeon have similar innate patterns of migration.

To better estimate the percentage of pallid sturgeon larvae entering the canal, we could identify suspected pallid spawning sites upstream from the canal (potential sites that pallids could access once passage beyond Intake is implemented). Then, using a combination of distance of the spawning sites from the canal and the thalweg velocity, we could calculate if pallid larvae would still be in the free-flowing state as they approach the canal. If so, we could estimate (by modeling or other) what percentage of thalweg flows would enter the canal. The larvae may be floating in the river thalweg (Boyd Kynard, personal communication), but this area of research is just emerging and others suggest that the larvae "spin out" into side eddies along the thalweg. Based on information from the larval drift study, the majority (87%) of larvae drift near the bottom in velocities of 0.34 – 0.37 meters per second (1.1 to 1.2 fps). Entrainment of larvae in side eddies does occur, but results suggest that nearly all larvae re-enter the drift and were transported downstream (Braaten et al, 2004). The study also showed that the mean drift velocity of larval sturgeon was significantly slower than the mean water velocity, probably due to a combination of bottom drifting (slower flows) and occasional spin-out into an eddy. The assumption would be that the percentage of thalweg flows entering the canal would equate to the percentage of larvae entering the canal (during the appropriate seasonal timeframe). Then we could document 1) if larvae entrainment in the canal is a problem or not, and 2) if so, how large of a problem larvae entrainment is.

2. Fish Screen Review

This review is based on the fish screen criteria presented in the April 2005 Concept II Bureau of Reclamation Design report.

After consideration of options for the fish screen at the Intake Dam Canal site, the Bureau of Reclamation (BOR) proposed an off-river fish barrier system located within the canal approximately 500 feet from the headworks of the Intake Canal. The fish screen system consists of several components: a trash rack, a fixed "v-shaped" screen, two fish screen "sweeps" to clean the fish screen, adjustable baffles to produce a uniform flow, a fish bypass pipeline back to the river, a downstream check structure, and pressure relief panels to allow water flow and prevent structure failure should the fish screens plug. The report recommends a screen material with a 1.75 mm slot opening stainless steel wedge wire with about a 40% open area.

Each component needs to take into account the varius swimming capabilities of all pallid sturgeon life stages for the system to work to prevent pallid sturgeon impingement or entrainment.

Based on data collected from 1996 – 1998, the stonecat, flathead chub, and the sturgeon chub are the most frequently entrained adult fish in the unscreened canal (Hiebert et al, 2000), although this study captured 34 species of fish through the intake gates of the canal.

2.1 Design Assumptions

Fisheries assumptions regarding objectives of the fish screen[1]:
1. Pallid sturgeon is the target species (sets minimum criteria) however screening of other fish is also desired.
2.. Ideally, all age classes of pallids should be screened, if practical (feasibility, cost, etc).
3. The downstream bypass pipe should be sized to pass adult pallid sturgeon (assume 50 pounds, 5 feet long)
4. Predator protection is necessary at the outfall pipe.
5. Open bypass channel is preferred to a pipe for access reasons.
6. Designing for pallid sturgeon will accommodate other fish species.

2.2 Relevant Pallid Sturgeon Criteria

According to the 1995 Juvenile Fish Screen Criteria developed by the National Marine Fisheries Service (NMFS), the swimming ability of a fish is a primary consideration in the development of a successful fish screen facility. Swim speeds can be split into three categories: burst speeds which are maintained for 15 seconds or less, prolonged speeds which have a duration of up to 200 minutes, and sustained speeds which can be maintained for more than 200 minutes (Tunink, 1977). For the purposes of fish passage, burst speeds and prolonged speeds are the most important. Table ___ summarizes the swimming information currently available for the target species, pallid sturgeon (in **bold**). Information on the surrogate species, the shovelnose sturgeon, is not in bold. Based on comparative studies in laminar flows, rate of travel (body lengths per second) was not significantly different between shovelnose and pallid sturgeon (Kynard et al, 2002). The swim rate was one body length per second for both species. For turbulent flows, pallid sturgeon were slightly stronger swimmers than shovelnose sturgeon.

[1] Bob Dach (FWS Region 6) e-mail to Greg Johnson dated Aril 14, 2006 provided guidance for the screen, return flow pipe, and the rock ramp.

Table 1
Scaphirhynchus Swim Capabilities

Age Class	Prolonged Speed	Burst Speed	Comments
Larvae	N/A	N/A	They "swim" up and down within the water column, not "swim" against flows
Fry (> 60 mm)	No information		
Juveniles	**1.0 – 1.6 fps (a)** **0.9 – 2.0 body lengths / second (d)** 0.6 – 0.9 body lengths / second (d)	**2.3 fps (b)** **1.8 – 2.3 fps (a)**	Calculated burst speed based on 3.5 body lengths / second (b)
Adults (mid-sized)	<4 fps (c) **2.1 – 3.8 fps (a)**	**6 fps (calculated)**	1.5 x sustained for burst speeds
Adults (design-sized)[2]	**5 fps (calculated)**	**7.5 fps (calculated)**	Based on one body length per second for sustained speeds and 1.5 x sustained for burst speeds

a. Hoover and Kilgore
b. Adams et al., 1999
c. White and Mefford, 2002
d. Kynard et al, 2002

Laboratory tests with pallid sturgeon larvae indicate that they may drift / swim relatively high in the water column after day 7, at or above 150 cm above the bottom, and possibly even at the river's surface (Kynard et al, 2002; Kynard et al, 2004). Dr. Kynard has anecdotal accounts of seeing free embryos and early feeding larvae swim upstream for short distances in the laboratory (estimated 6 – 20 cm) in bottom currents of 2 to 3 cm/sec (0.07 – 0.1 fps) (Kynard, personal communication).

In addition to the swim capabilities of pallid sturgeon, size is a consideration with regard to the trash rack openings, screen mesh size, and bypass pipe diameter. Table 2 describes the estimated size range for various life stages of pallid sturgeon.

[2] The design size provided by the FWS for the return flow pipe is for pallid sturgeon 50 pounds and 5 feet long.

Table 2
Pallid Sturgeon Size by Age

Age Class	Width	Height	Length	Comments
Larvae	2.0 – 2.5 mm (a)	2.0 – 2.5 mm (a)	10 – 12 mm (b)	Cylindrical in cross-section
Fry			Up to 60 mm	
Juveniles			> 60 mm	
Adults (mid-sized)	Approx. 8 inches (a)		Up to 3 feet	20 – 30 pounds
Adults (design size)	9 – 10 inches (a)	8 – 9 inches (a)	Up to 5 feet (approx.)	50 - 70 pounds

(a) Herb Bollig, personal communication
(b) Braaten et al, 2004

The smallest fry that the Bureau of Reclamation usually screens for are 25 – 30 mm long, usually salmonids (Brent Mefford, BOR).

There's a relationship between sustained swim speeds and body size which can be used as a rule-of-thumb in the absence of additional data. Viedler (1993) proposed that fish can move one body length per second. Based on data from Adams et al (1999) for pallid sturgeon that were 7.8 inches long (fork length), the burst speed was 2.3 fps or 3.5 body lengths per second (for juvenile fish).

2.3 Analysis of Fish Screen Components

2.3.1 Trash Rack

2.3.1.1 Trash Rack Design

The trash rack is designed to pass a flow of 1400 cfs with a velocity of 2.5 fps. Bar spacing is proposed to be 8 inches apart to prevent the passage of large debris.

Fish diameter is based on body shape, and the pallid sturgeon has a flattened or compressed body shape. Other relevant body shapes for large fish would include the paddlefish (rounded) and the catfish (intermediate between the paddlefish and the sturgeon). Since many fish are also longitudinally tapered on either end, it's possible that the front of a fish could fit through the 8 inch spacing, while the middle of the fish is too wide. Catfish and paddlefish are smooth-bodied and could work their way in or out of the trash rack should this happen. A catfish or paddlefish large enough to wedge themselves in an 8 inch trash rack should have enough swim strength to be able to get themselves out of or through the trash rack should this occur. However, paddlefish, unlike catfish, have a limited ability to navigate backwards, or in reverse, based on experience netting adult paddlefish (Herb Bollig, personal communication).

A pallid sturgeon may be more than 8 inches across laterally, but since sturgeon are flattened in shape, this same fish may be considerably less high, so could still potentially enter the fish screen facility through the trash rack by rotating. Measurements taken at the Gavins Point Fish Hatchery indicate that adult pallid sturgeon weighing 50 pounds have a maximum cross-sectional width range of 9 to 10 inches, and a maximum cross-sectional height of 8 or 9 inches (Herb Bollig, personal communication). Therefore, a fish of this size would not pass through the trash rack, and also would have the swim capability to avoid entering the trash rack, since large fish can resist the incoming velocities into the trash rack.

The largest pallid sturgeon that would potentially enter through the trash rack would be between 20 – 30 pounds. Fish of this size, based on measurements taken at the Gavins Point Fish Hatchery, would be less than the 8 inch spacing of the trash rack bars.

Therefore, we propose that the design criterion for the return flow pipe be reduced to accommodate a pallid sturgeon of a lesser size (e.g. 30 pounds) than the 50 pound size originally proposed.

From a behavioral perspective, normally one would not expect a pallid sturgeon to enter the Intake Canal in the first place since adult pallid sturgeon are large river obligate species. Pallid sturgeon were not collected in secondary channels in studies of the Missouri River above Fort Peck Reservoir (Gerrity, 2005), so may prefer to remain in the Yellowstone River rather than enter into the canal. Shovelnose sturgeon have been collected in the Intake Canal, however, so there is still the possibility that pallid sturgeon could also use the canal under certain circumstances, or at various lesser-known life stages.

The largest fish captured in the Intake Canal during studies in the late 1990's (Hiebert et al) was the bigmouth buffalo, with the largest being 4.2 Kg (9.25 pounds). Other large fish collected include the blue sucker (4.08 Kg or 9 pounds), common carp (4.07 Kg or 9 pounds), smallmouth buffalo (2.94 Kg or 6.5 pounds), shovelnose sturgeon (2.74 Kg or 6 pounds), and channel catfish (2.36 Kg or 5.2 pounds). Based on the sizes of fish captured in this study effort, all would fit within the 8 inch distance between the trash rack bars. This makes sense since the spacing on the railroad ties protecting the intake gates are spaced 8 inches apart.

However, a 37 pound paddlefish was caught in the canal (Brad Schmitz, personal communication). Since there aren't really any long-term resident fish in the canal, it's likely that the railroad ties have been damaged by ice over the years, resulting in gaps potentially much larger than the 8 inch spacing since the railroad ties are probably damaged, leaving larger gaps (Jerry Nypen, personal communication).

The 8 inch spacing of the trash rack bars will not impinge small fish, so if they approach close enough to the structure to swim into it, they will continue on into the fish screen structure. Larger fish, such as catfish, paddlefish, buffalo, and sturgeon will be capable of resisting the velocities near the trash rack, so they too will not become impinged but will either swim through the trash rack (unless they are too large) or will choose to avoid the structure and return to the river. The behavior of very large fish should prevent them from entering the Intake Canal, and very large fish were not collected past the intake gates during previous fisheries studies.

Currently the only entrainment information we have is for juvenile and adult fish that are not pallid sturgeon, not larval fishes. A screen wouldn't be needed to prevent this sized fish from entering the canal, but rather a more closely spaced trash rack could serve that purpose. The trash rack could be placed immediately on the canal side of the existing railroad tie intake structure. A trash rack with 1 to 2 inch spacing would not prevent larval fish from entering the canal.

Evaluate the potential for smaller trash rack spacing to keep juvenile and adult fish out of the canal, in lieu of a fish screen. Determine the best location for this type of structure, if pursued.

2.3.1.2 Trash Rack Location

Currently the design shows the trash rack immediately upstream from the fish screen, which is located approximately 500 into the canal itself. Therefore, fish that pass through the gates would not encounter the trash rack until 500 feet later. In theory, since the spaces between the railroad-tie-protected intake gates are also 8 inches apart, no fish greater than this size should enter the canal, so smaller fish should pass through and along the fish screen into the fish bypass canal. However, it's more likely that the railroad ties have been damaged by ice, as evidenced by very large (37 pound) paddlefish being caught within the canal (Brad Schmitz, personal communication). Due to dangerous conditions riverward of the gates, it will be difficult to verify that there are no openings larger than 8 inches within the gates, and also to prevent further deterioration of the railroad ties over time due to ice.

With the probability of larger openings in the railroad ties leading into the canal, larger fish could enter the canal and be stopped by the trash rack. If this is the case, there is no existing bypass for these larger fish to return to the Yellowstone River, nor would these larger fish be able to pass through the 8 inch openings of the trash rack to access the fish bypass pipe.

Post-construction monitoring should be done to document whether larger fish become trapped within the 500 foot stretch of canal just off the Yellowstone River.

Possible solutions include an external metal trash rack riverward of the railroad ties, including a large fish bypass prior to the trash rack, or periodically netting and relocating any large fish trapped within that 500 feet of canal (probably the least expensive).

2.3.2 Fish Screen and Associated Parts

The fish screen itself has several parts associated with it. The concrete sill upon which the fish screen will sit is 12-inches high. The height of the sill relates to the structure's ability to tolerate sediment deposition, so a low sill will likely pass more benthic fish, but could increase operational costs. The length of each screen (based on the drawings) is 224 feet. Screen mesh will be stainless steel wedge wire with a 1.75 mm opening (0.069 inches) and a 40% open area.

These same permit conditions also require mesh opening sizes no greater than ¼ inch (6.35 mm) to prevent entrainment of pallid sturgeon fingerlings. NMFS mesh screen criteria for salmon fry is no more than 3/32 inches (2.38 mm) and a minimum 27% open area. The proposed mesh opening of 1.75 mm is likely included to address concerns about passing pallid sturgeon larvae into the canal. Currently, there are no known screening criteria for the prevention of pallid sturgeon larvae entrainment or entrapment, nor are there mesh sizes recommended by NMFS for the prevention of salmon larvae entrainment.

There aren't very many facilities currently using small mesh screens, so there is limited information available regarding the feasibility on the use of such screens, both for fry / larvae protection and for practical use within a turbid river system.

Table 3
Screened Facilities

Project	Mesh Size	Facility Type	Location	Screen Target
Intake Canal (proposed)	1.75 mm (inch)	Open Canal	Montana	Pallid sturgeon larvae
Leeburg Dam	1.75 mm		Oregon	Salmon fry (a)
Walterville Dam	1.75 mm		Oregon	Salmon fry (a)
Roosevelt CD	2.38 mm (3/32 inch)	Floating Pump	Montana, North Dakota	Debris / twigs (b)
Redlands Water and Power Canal	2.38 mm (3/32 inch)	Open canal	Colorado	Razorback sucker hatchery fish (3/4" to 1" size) (c)
Various	6.35 mm (1/4 inch) (current Regulatory standard)	Intake Pumps (municipal and irrigation)	Missouri R, Mississippi R, Atchafalaya R watersheds	Pallid fingerlings (d)

(a) Tim Downy, personal communication
(b) Dick Iverson, personal communication
(c) Kevin Moran, personal communication
(d) Matt Wray, Larry Hartzog, Alan Steinle, personal communication

Of the examples in Table 3, only the ¼ inch screen was specifically targeted for pallid sturgeon. The Colorado example targets a smaller life stage of another federally listed fish, the razorback sucker, using 3/32 inch mesh screen. The screen size is smaller than needed for that life stage, however smaller screen mesh and slower velocities are thought to keep the debris load on the screen reduced (Kevin Moran, personal communication; Brent Mefford, personal communication). At this facility, they have also reduced the approach velocity from 0.5 fps to between 0.2 and 0.3 fps for debris control. The screen is on the Gunnison River which has a very high debris load. No monitoring of the targeted fish species is being done at this facility to ensure adequate movement along the screen and into the bypass system.

Because the response of pallid larvae (or any larvae) to the wedgewire fish screen an unknown, it would be useful to construct a model screen with the proposed water velocities and test the ability of larvae to navigate along the screen into the collection tube prior to full-scale construction.

This test would also be able to determine if larvae become impinged through or up against the screen. This data would provide additional information upon which to base the final mesh screen openings for the fish screen.

The pallid sturgeon experts will need to determine whether including larvae-sized screen mesh will, in fact, prevent losses of pallid sturgeon larvae in the Intake Canal, assuming that larvae entrainment within the canal is a problem. If larvae are screened, only to become impinged and mechanically injured, then perhaps there would be a greater value in sizing the mesh to allow the larvae to pass through into the canal, and then collect and relocate them through larval sampling. Alternatively, larvae floating in the central water current may not encounter the screen and may very well pass through the system and into the bypass canal. Those along the screen, however, may be lost and should be accounted for by "take" provisions in the USFWS' Biological Opinion for the fish screen. The net value of the screen to prevent entrapment of pallid sturgeon in the canal would still be positive, even if "take" provisions are needed.

In the absence of additional data on larval swim capabilities, or a working model of the screen to see if sturgeon larvae can navigate along the screen and into a bypass pipe, we recommend using a screen mesh consistent with NOAA guidance for fry (2.38 mm) at the smallest. Even a mesh this size should be considered an experiment, since it's smaller than the ¼ inch (6.38 mm) mesh required for new intakes, and the criteria were developed for salmonids, so monitoring should be done to ensure that non-salmonid fish are not being impinged on the screen.

2.3.3 Screen Velocities

The screen itself will have an approach velocity of 0.4 fps and a sweeping velocity of 2.5 fps at the upstream end and 2.0 fps at the downstream end. All velocities are for the design flows of 1400 fps.

The approach velocity of 0.4 fps is equal to that proposed by NMFS for prevention of entrainment of salmon fry (NMFS 1995) but is more restrictive than the 0.5 fps generally required for screening of juvenile pallid sturgeon on pumped water intake pipes within the Omaha, St. Louis, and New Orleans District Regulatory offices of the Corps (various Corps Regulatory offices, personal communication). The NMFS criteria for salmon fry may be too strong for pallid sturgeon fry. If we assume that the swim speed for adult and juvenile sturgeon (one body length per second) is also appropriate for fry 25 mm long, then the calculated sustained swim speed for fry would be 0.08 fps, or less than 0.1 fps, which would be four times slower than what's used for salmon fry.

Since laboratory studies of pallid sturgeon larvae indicate that while they can swim vertically within the water column, they float near the bottom of the main river current (thalweg) until several days old, and only then have some (very weak) directional movement (Kynard et al, 2002; Kynard et al 2004). Larvae will have minimal ability to control their movement when confronted with the screen. If the approach velocity is a steady flow, they may still be impinged against the screen, then swept along the screen as it is cleaned. If the approach velocity isn't steady, it's possible that the larvae may become temporarily impinged, then released, then impinged, then released, and as such float and "swim" their way down the screen with the current. The wedgewire design facilitates a "bounce and swim" response in fish fry which may or may not be utilized by pallid larvae. The larvae may behave similar to small floating debris with little directional movement. If, however, the approach velocity IS steady (more likely), then the larvae in the pelagic stages may become impinged on the screen until swept downstream by the cleaning brushes, although the very localized zero velocity within the wedgewire may be enough to prevent impingement. The mechanical sweeping action would likely be injurious to the larvae, if impinged.

NOAA recommends that design velocities should be such that it doesn't allow for any contact of fish with the screen; any contact is considered a "take" (Brian Nordland, personal communication). Usually design velocities are established based on swim capabilities of the targeted fish / age group. There is no data on the swim velocities that can be tolerated by larval pallid sturgeon, so an evaluation of the velocities for this age class can't be done without additional studies.

Swim studies of pallid sturgeon fry would provide information on the velocities that this size class can tolerate without impingement. Until such studies are done, the approach velocity should be assumed to be less than 0.1 fps.

The data for juvenile pallid sturgeon (Table 1) is in the vicinity of 1 fps, which is greater than the design approach velocities of 0.4 fps, so fish of this sized should be able to avoid impingement.

2.3.4 Cleaning Mechanism

The proposed cleaning mechanism is a brush cleaner that would brush away debris from the interior portion of the V-shaped screen. However, NOAA recommends that brush cleaners be avoided if there are weak swimmers near the screen. A backside spray cleaner is recommended instead (Brian Nordland, NOAA, personal communication).

Backside spray cleaners should be used instead of brush cleaners to avoid impinging weak swimmers along the screens.

2.3.5 Bypass Pipe and Outfall

A 50-pound 5-foot long sturgeon would not pass through the 8-inch openings of the trash rack (see discussion in Section 2.3.1), which is the design size provided by the FWS for the fish bypass pipe. Since a fish of this size won't pass through the trash rack, the bypass pipe does not need to accommodate a fish this large and can be sized accordingly. The largest sturgeon likely to pass through the trash rack, past the screen, and into the bypass canal is estimated to be 30 pounds.

The proposed design is for a pipe, not an open canal as recommended by the FWS. It is unclear in the report why a pipe was selected over the open canal. An open canal would facilitate collection of fish that pass into the bypass. However, open canals can also result in increased predation of fish by fish-eating birds and mammals, and even local fisherman if the canal is not covered with a screen or otherwise monitored.

Consideration should be given to an open canal for the bypass return pipe, and to reducing the diameter of the pipe, unless there is an engineering or maintenance reason for a pipe this size.

Based on Corps' calculations at the stated design flow of 40 cfs, the pipe will be flowing full with backwater from the Yellowstone River. The flow velocity will be based on the pipe area of 12.6 sq. ft. or about 3.2 fps maximum velocity. This velocity would be acceptable for pallid sturgeon and other native fishes. With a head differential of 5.8 feet (1990.8 - 1985), the maximum flow rate in the pipe is about 120 cfs. Therefore, to limit flows to 40 cfs, the slide gate would need to be lowered. At 120 cfs, the max vel would be 9.6 ft/sec. This would exceed swim capabilities for most native warm-water fishes.

Further analysis should be done on how to limit flow velocities within the bypass pipe during maximum flow rates.

3. Rock Ramp Design Review

This is an evaluation of the Corps' rock ramp design downstream from the Intake Dam.

Fisheries design assumptions regarding objectives of the rock ramp:
1. The rock ramp should be designed for sub-adult pallid sturgeon, with consideration to burbot, small cyprinids, and other species.
2. The rock ramp should function year-round ideally, but should be 100% effective from February to August.
3. The rock ramp should be 100% effective during the targeted timeframe in 75% of the years (allows from drought and flooding).

The rock ramp will be designed primarily for pallid sturgeon, however the design criteria indicate that other species, including smaller cyprinids, should also be able to pass through the rock ramp.

The Corps evaluated various slopes (5%, 3.33%, 2%) and drop elevations (1 ft, 0.5 ft) through modeling. There was no significant velocity difference among the three slopes; the lower the slope, the longer the ramp structure. A minimum 4 foot diameter boulder was used for the ramp stones, based on feedback from CRREL on ice stability. Pallid sturgeon favor laminar flows over turbulent flows, so further study should be done on the turbulence associated with the drop heights in order to make a firm recommendation, but it appears as though the 0.5 drop may have more turbulence since it results in twice as many drops along the length of the rock ramp.

Evaluate turbulence associated with rock ramp designs and elevation drops.

Velocities were modeled for each combination of slope and drop elevation for the 100-year, 10-year, and average discharge. Based on the output of this modeling, fish passage through the ramp during the 100-year and 10-year discharges may not be possible. However, the design criteria is for fish passage during 80% of the water years. During the normal flows (20,000 cfs), flow velocities are such that fish passage may be possible, with the exception of the extreme upstream end of the ramp where the rock dam currently sits. All flow velocities at the dam exceed the burst speed for large adult pallid sturgeon. Based on lab tests with shovelnose sturgeon, White and Mefford (2002) recommended flow velocities between 3.0 and 4.0 fps.

Evaluate ways to reduce the flow velocities at the upstream end of the rock ramp to less than 7.5 fps maximum.

There are two theories for designing fish passage facilities: (1) design for the targeted species or group (which is what was done for the Corps' design), and (2) design based on comparable river conditions. Theory (2) is based on the concept that if the fish can navigate the free-flowing river, those conditions (or more accurately that range of conditions) should be adequate for the fish that inhabit the river. Since the Yellowstone River supports a wide range of fish species and life stages, targeting one species or even one group of species may not be as reliable for overall success as mimicking flows in an unblocked section of the Yellowstone River downstream from the Intake Dam.

Currently, we don't have data available on a "rocky run" section of river that pallid sturgeon currently navigate through. However, if such information was available and utilized as the primary design criteria instead of the limited swim velocity data available for the pallid sturgeon, we could mimic an already-functioning "rock ramp" area and have a greater confidence level of successfully passing pallid sturgeon and other native fishes. Design criteria from existing rocky runs could include maximum velocity, percent of run area within other velocity ranges, depth categories, rock spacing, etc.

Identify several existing "rocky runs" on the Yellowstone or Missouri Rivers to use as design models for the rock ramp. Selected runs should already successfully pass pallid sturgeon and other targeted native fish, such as the paddlefish.

Without the benefit of knowing what velocities, depths, and rock spacing are naturally present, the rock ramp design currently has velocities at the crest that exceed burst speeds for adult pallid sturgeon at all flows (based on pallid swim information in Table 1).

Little is known about juvenile pallid sturgeon, however the rock design should result in slack water resting areas downstream from the boulders where smaller fish can rest and "burst" between. Bottom velocities will likely be less due to riverbed roughness, so these bottom-dwelling fish should be able to move upstream along the bottom, utilizing resting areas behind boulders as needed.

The face of the dam will be raised up to one foot (Corps modification) in order to strengthen the dam for ice resistance. The "higher" end of the dam will be on the opposite end from the canal, with the "lower" end (and thalweg) on the canal side of the river. Raising the face could impede downstream migration of fish during low flows.

Provide a low-water fish passage option by excavating the side channel along the Yellowstone River.

Although the attraction flows from the channel wouldn't be drawing in many fish, some would use the channel for upstream movement, and more may use it for downstream movement during low flows.

4. Canal Intake Relocation Review

Fisheries design assumptions regarding objectives of the canal intake relocation:
1. The relocated canal[3] should be designed for sub-adult pallid sturgeon, with consideration to burbot, small cyprinids, and other species.

[3] No specific guidance was received on the relocated canal, so rock ramp guidance is assumed.

2. The relocated canal should function year-round ideally, but should be 100% effective from February to August.
3. The relocated canal should be 100% effective during the targeted timeframe in 75% of the years (allows from drought and flooding).

This review is based on the Corps' development of a new canal alternative that would connect with the existing Intake canal. Currently the canal is an open ditch over two miles long. A screen would be included, although the location of the screen could be varied; at the opening of the new canal, at the juncture of the new canal and old canal, or at the first major side canal.

Flows within the canal are not expected to be detrimental to fish.

Consideration should be given to darken the entrance to the canal, either by piping or construction a large box culvert, in order to discourage fish movement into the canal from the river.

5. Conclusions and Recommendations

5.1 General Recommendations

Although these recommendations may not be specifically related to the two fish pass alternatives or screening structure, they are other items to consider during final design and the NEPA process.

- **Develop clear guidance on the purpose and need for each structure based on specific details on the species and life stages that are being targeted.**
- **Based on the targeted species and life stages, develop success criteria that are measurable so monitoring can be done to determine if the criteria are being met. This includes timeframes for success.**
- **Pre-construction monitoring should be done as part of the measurement of success.**
- **Develop a fall-back plan if success criteria are not being met.**
- **Evaluate the data in hand to quantify if there is a problem, especially with regard to the screening of the canal. Gather more data if sufficient data isn't available.**
- **Alternatives that include dam removal are preferable for pallid sturgeon than those that leave the dam in place.**
- **Since no documentation exists regarding entrainment of larval fish in the canal, the screen construction could be delayed until such data is available.**
- **Very little data exists on the potential for pallid sturgeon entrainment within the canal. Adults and juvenile entrainment could be addressed through use of more closely spaced trash rack openings.**
- **Some sort of "decision tree" or "best value" analysis should be done for the various alternatives based on the potential for pallid sturgeon to pass**

upstream the dam, to successfully spawn, and for larval pallids to drift downstream over the dam (or into the canal).

5.2 Recommended Studies

- **Additional studies are needed to further gain insight into the entrainment potential of pallid sturgeon, including tracking through the canals to determine ultimate fate.**

- **Post-construction monitoring should be done to document whether larger fish become trapped within the 500 foot stretch of canal just off the Yellowstone River.**
- **Because the response of pallid larvae (or any larvae) to the wedgewire fish screen an unknown, it would be useful to construct a model screen with the proposed water velocities and test the ability of larvae to navigate along the screen into the collection tube prior to full-scale construction.**
- **In the absence of additional data on larval swim capabilities, or a working model of the screen to see if sturgeon larvae can navigate along the screen and into a bypass pipe, we recommend using a screen mesh consistent with NOAA guidance (2.38 mm) at the smallest. Even a mesh this size should be considered an experiment, since it's smaller than the ¼ inch (6.38 mm) mesh required for new intakes, and the criteria were developed for salmonids, so monitoring should be done to ensure that non-salmonid fish are not being impinged on the screen.**
- **Swim studies of pallid sturgeon fry would provide information on the velocities that this size class can tolerate without impingement.**
- **Further analysis should be done on how to limit flow velocities within the bypass pipe during maximum flow rates.**
- **Evaluate turbulence associated with rock ramp designs and elevation drops.**
- **Evaluate ways to reduce the flow velocities at the upstream end of the rock ramp to less than 7.5 fps maximum.**

5.3 Recommended Design Criteria

- **Designs should be flexible to allow for modification or adjustment as more information becomes available from post-construction monitoring and sturgeon use of the structures.**
- D**esign criterion for the return flow pipe should be reduced to accommodate a pallid sturgeon of a lesser size (e.g. 30 pounds) than the 50 pound size originally proposed.**

- **Evaluate the potential for smaller trash rack spacing to keep juvenile and adult fish out of the canal, in lieu of a fish screen. Determine the best location for this type of structure, if pursued.**
- **Until larval swim studies are done, the approach velocity near the screen should be assumed to be less than 0.1 fps.**
- **Backside spray cleaners should be used instead of brush cleaners to avoid impinging weak swimmers along the screens.**
- **Consideration should be given to an open canal for the bypass return pipe, and to reducing the diameter of the pipe, unless there is an engineering or maintenance reason for a pipe this size.**
- **Identify several existing "rocky runs" on the Yellowstone or Missouri Rivers to use as design models for the rock ramp. Selected runs should already successfully pass pallid sturgeon and other targeted native fish, such as the paddlefish.**
- **Provide a low-water fish passage option by excavating the side channel along the Yellowstone River.**
- **Consideration should be given to darken the entrance to the canal, either by piping or construction a large box culvert, in order to discourage fish movement into the canal from the river.**

6. References

Adams, S.R., J.J. Hoover, K.J. Kilgore. 1999. Swimming endurance of juvenile pallid sturgeon *Scaphirhynchus albus*. Copeia 1999: 802-807.

Backes, K., W. M. Gardner, P.A. Stewart. 1994. Lower Yellowstone River Sturgeon Study IV and Missouri River Pallid Sturgeon Creel Survey. State of Montana Dept of Fish Wildlife and Parks, Miles City MT; U.S. Bureau of Reclamation, Billings MT.

Braaten, P.J., D.B. Fuller, T. Brandt. 2004. Drift dynamics of larval shovelnose sturgeon in a side channel of the upper Missouri River in Eastern Montana. USGS CERC, Montana Fish Wildlife and Parks, USGS WRD, Fort Peck, Montana. Prepared for the Omaha District Corps of Engineers.

Bramblett, R.G. 1996. Habitat and movement of pallid and shovelnose sturgeon in the Yellowstone and Missouri Rivers, Montana and North Dakota. PhD Dissertation, Montana State University, Bozeman, MT.

Helfrich, L.A.; C. Liston, S. Hiebert, M. Albers. 1999. Rivers 7(1): 21-32.

Hiebert, S. R. Wydoski, T. Parks. 2000. Fish entrainment at the Lower Yellowstone Diversion Dam, Intake Canal, Montana, 1996 – 1998. U.S. Department of Interior, Bureau of Reclamation, Denver Office, Montana Area Office; Montana Fish Wildlife and Parks; Lower Yellowstone Irrigation Project. April 2000.

Jaeger, M., G. Jordan, S. Camp. 2005. Assessment of the Suitability of the Yellowstone River for Pallid Sturgeon Restoration Efforts. Montana Dept or Fish, Wildlfie and Parks, U.S. Fish and Wildlife Service, U.S. Bureau of Reclamation. Upper Basin Pallid Workgroup 2004 Annual Report, May 2005.

King, W., R.H. Wilson. 2003. Movements and habitat preferences of adult post-spawn pallid sturgeon. 2002 Progress Report. USFWS, Bismarck, ND.

Kynard, B., E. Parker, T. Parker (in press). Effect of water temperature, water velocity, and fish swimming height on dispersal of Missouri River pallid sturgeon early life intervals: a laboratory study. USGS Biological Resources Division, S.O. Conte Anadromous Fish Research Center, Turner Falls, MA.

Kynard, B., E. Parker, D. Pugh, T. Parker. 2004. Swimming height and the effect of velocity on dispersal of free embryo and larval pallid sturgeon: An experimental study. . USGS Biological Resources Division, S.O. Conte Anadromous Fish Research Center, Turner Falls, MA. A report for the U.S. Army Corps of Engineers, Omaha District.

Kynard, B., D. Pugh, E. Henyey, T. Parker. 2002. Preliminary comparisons of pallid and shovelnose sturgeon for swimming ability and use of fish passage structures. . USGS Biological Resources Division, S.O. Conte Anadromous Fish Research Center, Turner Falls, MA. A report for the U.S. Army Corps of Engineers, Omaha District.

Kynard, B., E. Henyey, M. Horgan. 2001. Ontogenetic behavior, migration, and social behavior of pallid sturgeon, *Scaphirynchus albus*, and shovelnose sturgeon, *S. platorynchus* with notes on the adaptive significance of body color. USGS Biological Resources Division, S.O. Conte Anadromous Fish Research Center, Turner Falls, MA.

National Marine Fisheries Service. 1995. Juvenile fish screen criteria. Environmental and Technical Services Division, Portland, OR.

U.S. Dept of Interior Bureau of Reclamation. 2005. Appendix A: Intake Diversion Dam Fish Protection and Passage Concept Study Report II, Lower Yellowstone Project, Yellowstone River, Montana. Bureau of Reclamation Technical Service Center, Denver CO. April 2004.

7. Personal Communication and Contacts

Ames, Mike. Agra Industries (makes screen mesh). (701) 572-0767.

Bollig, Herb. Hatchery specialist with the USFWS Gavins Point Hatchery, Yankton, SD. (605) 665-3352.

Dach, Bob. Biologist with USFWS Region 6, Denver, CO (303) 236-4264.

Downy, Tim. Biologist with Eugene Water and Electric, Eugene, OR. (541) 344-6311.

Jordan, George. Fisheries Biologist with USFWS, Montana. Pallid sturgeon Recovery Team coordinator (406) 247-7365.

Hartzog, Larry. Fisheries Biologist with USACE New Orleans District Regulatory office, New Orleans, LA. (504) 862-2524.

Iverson, Dick. NRCS, Montana office. (406) 433-2103, ext 126.

Keevin, Dr. Tom. Fisheries Biologist with USACE St Louis District Planning office, St. Louis, MO. (314) 331-8462.

Kynard, Boyd. Fish researcher with the USGS S.O. Conte Anadromous Fish Research Center, Turner Falls, MA. drboyd@forwild.umass.edu

Lydick, Steve. Fisheries Biologist with USFWS Nebraska Field Office, Grand Island, NE. (308) 382-6468.

Mattelin, Buzz. Director, Roosevelt Conservation District, Montana. (406) 786-3286.

Mefford, Brent, PE. Fish passage engineer with the US Bureau of Reclamation Technical Lab, Denver, CO. (406) 445-2149.

Moran, Kevin. Civil engineer with the US Bureau of Reclamation Grand Junction office. (970) 248-0635.

Muggli, Roger. Manager, Tongue and Yellowstone River Irrigation District. (406) 232-5312.

Nordland, Brian. Fisheries Biologist with NOAA, Lacey, WA. (360) 534-9338.

Nypen, Jerry. Lower Yellowstone Irrigation District, Montana.

Ryan, Jeff. Montana Dept of Environmental Quality via e-mail to Alan Steinle.

Schmitz, Brad. Fisheries Biologist with Montana Game Fish and Parks

Schwartz, Dave. Manager, Buffalo Rapids Irrigation District. djs@midrivers.com

Smith, Dave. Fisheries Biologist with USACE ERDC lab (601) 634-4267.

Steinle, Alan. Environmental Resources Specialist for USACE, Chief Montana State Office, Regulatory, Helena, MT. (406) 441-1375.

Wray, Matt. Environmental Resources Specialist with USACE Omaha District Regulatory, Nebraska Office, Omaha, NE (402) 896-0896

Appendix B
Hydrology

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

US Army Corps
of Engineers ®
Omaha District

Lower Yellowstone River
Intake Dam Fish Passage
10% Design Analysis
Hydrology Appendix B

Flow Duration and Flow Frequency Curves near Project Site

Study Purpose. The purpose of this study is to update and develop flow duration curves for the Yellowstone River at Sidney, Montana to assist in the development of pertinent hydraulic data for assessing the feasibility of a fish passage structure at the Lower Yellowstone Diversion Dam near Intake, Montana. Flow-frequency curves will also be developed.

Background Information. The Lower Yellowstone Diversion Dam (hereafter referred to as Intake Dam) and Main Canal are part of the Lower Yellowstone Project located near Intake, Montana at 47°17´N, 104°32´W, or in the NW¼ Sec. 36, T. 18 N., R. 56 E., with the canal headworks along the left bank of the Yellowstone River. The canal provides irrigation waters to approximately 54,000 acres along the left bank of the Yellowstone River valley from the diversion point downstream to the Missouri River, where the canal returns excess waters at a point approximately 1¾ miles upstream of the Yellowstone-Missouri confluence. Principal crops grown include small grains, alfalfa and other hay crops, pasture, silage, beans, and sugar beets. The canal project also supplies municipal water to the town of Savage, Montana. The canal is approximately 72 miles long, with 225 miles of laterals and 118 miles of drains. The diversion dam is just downstream of the canal headworks and is a rockfilled timber crib weir about 12 feet in height. The U.S. Bureau of Reclamation began construction on the project in 1905, with water first made available during the 1909 growing season.

A Biological Assessment of the Operation of the Intake Dam concluded that the current configuration and operation of the dam is blocking the upstream migration of the pallid sturgeon. Design of a fish passage structure is therefore desired to assist in the recovery of the pallid sturgeon.

Study Data. The nearest stream gaging station on the Yellowstone River to the Intake Dam is near Sidney, Montana, approximately 36 miles downstream. The period of record with mean daily flows for the Sidney gage covers October 1, 1910 – December 31, 1931 and October 1, 1933 – present. The Sidney gage was located several miles downstream of Intake Dam from 1910-1931. The gage has subsequently been at several locations near its present location 2.5 miles downstream of Sidney since 1933. The USGS concurrently published monthly diversions to the Lower Yellowstone Canal until September 1987; these monthly values are tabulated in Table B-1.1.

A gage at Glendive, Montana, approximately 18 miles upstream of the Intake Dam, was in operation from October 1, 1897 – September 30, 1902 (monthly values only), March 16, 1903 – December 31, 1910, October 1, 1931 – September 30, 1934, and October 1, 2002 – present. The U.S. Geological Survey (USGS) considers the Sidney record plus the Lower Yellowstone Canal diversions to be equivalent to the Glendive record, so the Glendive record could be used to extend the Sidney gage record. However, since the canal diversion records are monthly values, the daily means would only be approximate.

Streamflow data for both the Sidney and Glendive gages were retrieved from the USGS website through use of a HEC-DSSVue (ver. 1.2.10) script that automatically places USGS data into an appropriate DSS file. Monthly values of canal diversions were manually entered using HEC-DSSVue. The monthly values were then distributed to daily values using the built-in math functions of HEC-DSSVue.

The mean annual flow and annual maximum and minimum daily flows for Sidney gage are shown in Figures B-1 to B-3 below, as well as in Tables B-1.2 to B-1.4. The annual data includes Glendive data from 1898-1910 and 1932 and 1933, with the appropriate flow volume for Lower Yellowstone Canal subtracted out in 1909-1910 and 1932-1933.

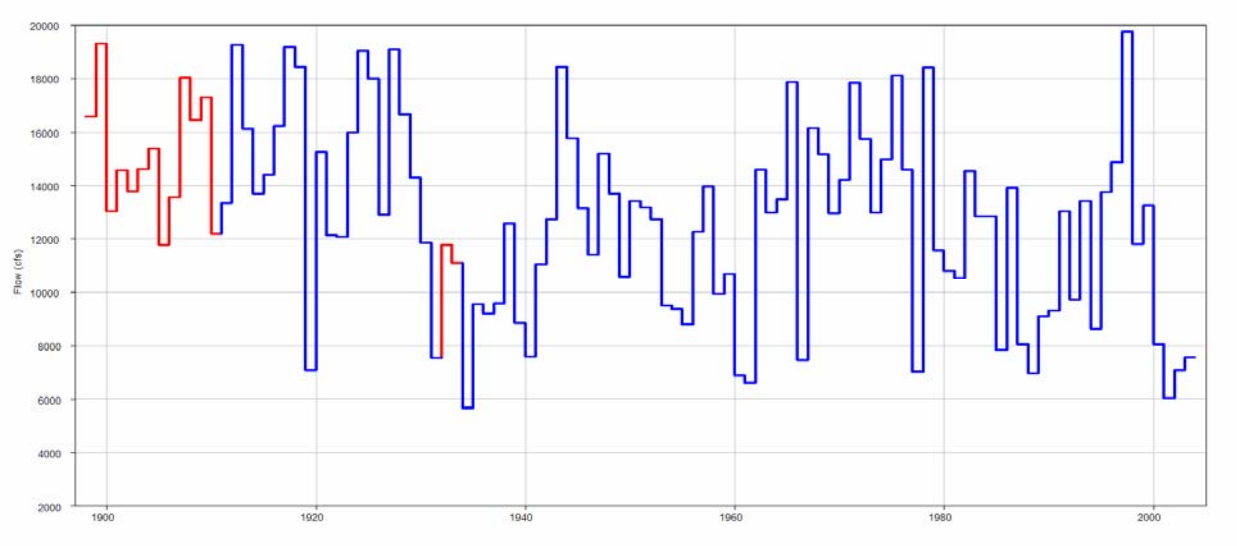


Figure B-1. Sidney, MT (in blue) and Glendive, MT (with Intake Dam flows subtracted, in red) Mean Annual Discharge, Calendar Year 1898-2004

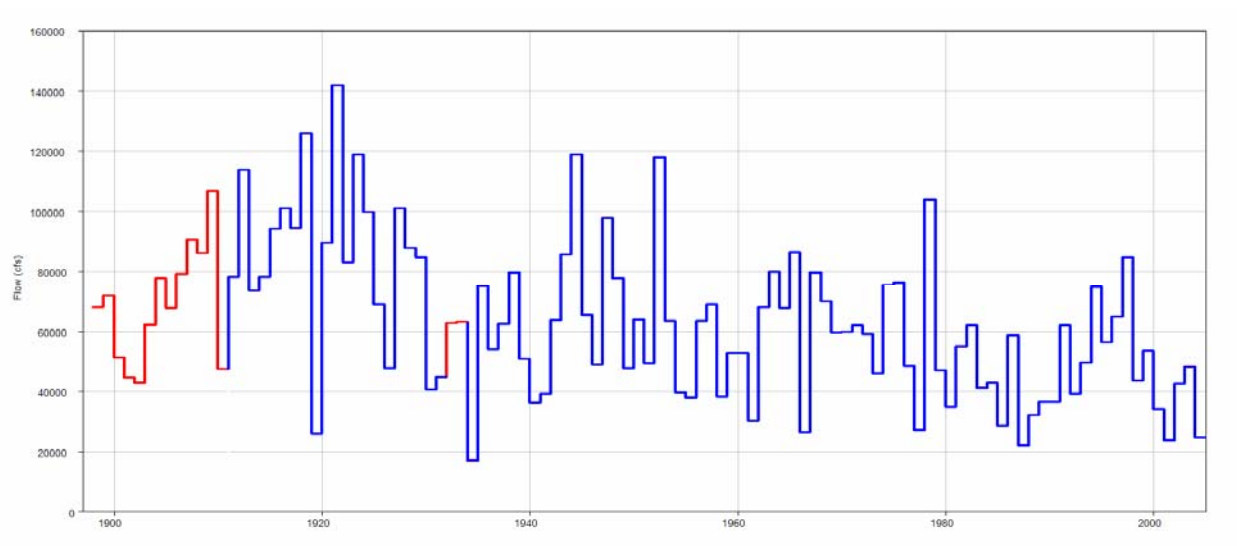


Figure B-2. Sidney, MT (in blue) and Glendive, MT (with Intake Dam flows subtracted, in red) Annual Maximum Daily Discharge, Calendar Year 1898-2004

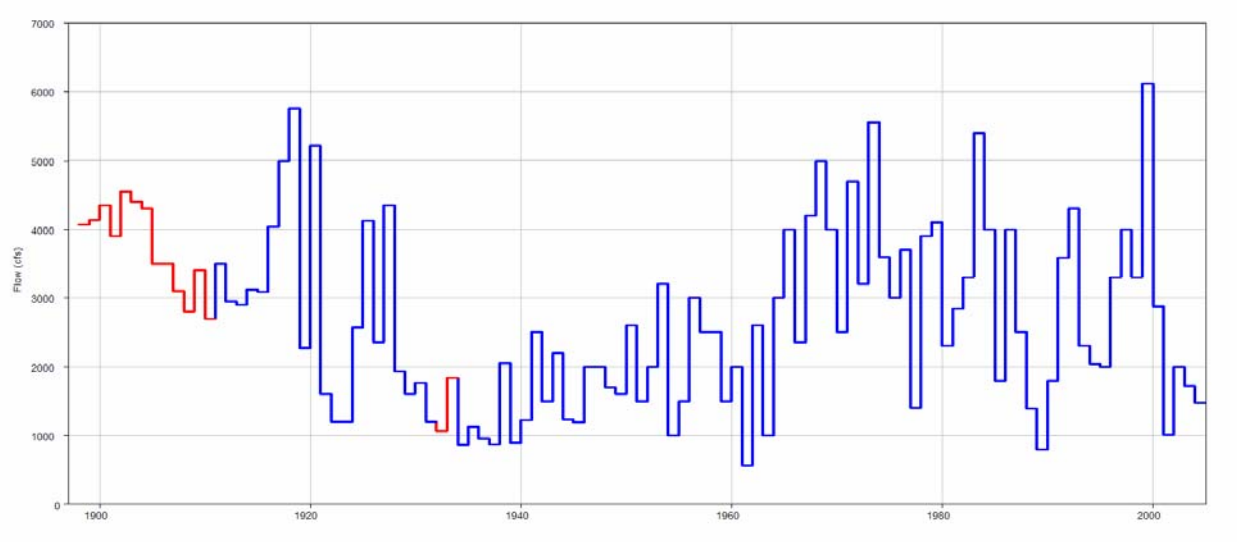


Figure B-3. Sidney, MT (in blue) and Glendive, MT (with Intake Dam flows subtracted, in red) Annual Minimum Daily Discharge, Calendar Year 1898-2004

A simple screening was done to assess the value in using Glendive daily values to extend the Sidney period of record for the flow-duration analysis. The daily flows for the periods of overlapping records at the 2 gages was compared, with the Glendive flows corrected for Canal withdrawals and lagged a day. Of the 1280 days of overlapping records, 803 days had Sidney daily flows within ±10% of the Glendive flows (with only 467 days within ±5%), while 389 days had flows more than 10% lower than the Glendive flows and the remaining days more than 10% greater than the Glendive flows. With nearly 40% of the overlapping record showing differences of more than 10% and nearly 65% more than 5% difference, it was decided that the Glendive records would not add substantially to the Sidney record for the daily flow-duration (or flow-frequency) curves.

Monthly Statistics and Flow-Duration Analysis. The daily discharge at Sidney was utilized to develop an annual and monthly flow-duration curves for the 1911-1931 and 1934-2005 Water Years (plus flow for October through December 1931). Figure B-4 below depicts the typical flow pattern for the year, showing the 5th, 10th, 25th, 50th, 75th, 90th, and 95th daily flow percentiles for the period of record. As can be seen, flow is relatively constant with the exception of a spring rise in mid-March through mid-April (which occurs in about 50% of the years), and a second, larger rise which occurs starting in early May, peaking in late June and receding by early August. The first rise is generally driven by plains snowmelt and rainfall, while the second rise is primarily driven by mountain snowmelt and augments by summer rainfall. HEC-DSSVue was used to compute the various percentiles of flow and retrieve the monthly mean flows over the period of record. as well as maximum and minimum flow values for each month. Tables B-1.5 through B-1.7 contain the mean monthly flows, as well as the monthly maximum and minimum daily flow for each year, respectively.

One thing that can be observed from the data is that the annual mean flow appears to show a downward trend, especially if a 5-year moving average of flow is used (as shown in Figure B-5), while the various months show differing trends over the period of record, with the winter months showing an increase and the summer months showing a decrease (see Figures B-6 and B-7 for typical examples). While this may intuitively seem due to irrigation diversions and reservoir operation, with higher summer flows diverted or held in storage and winter flows augmented with reservoir releases, the trends are not pronounced enough to determine if indeed flows have been impacted through irrigation and reservoir operation or if the trends may be due to various climatic factors, or just coincidence.

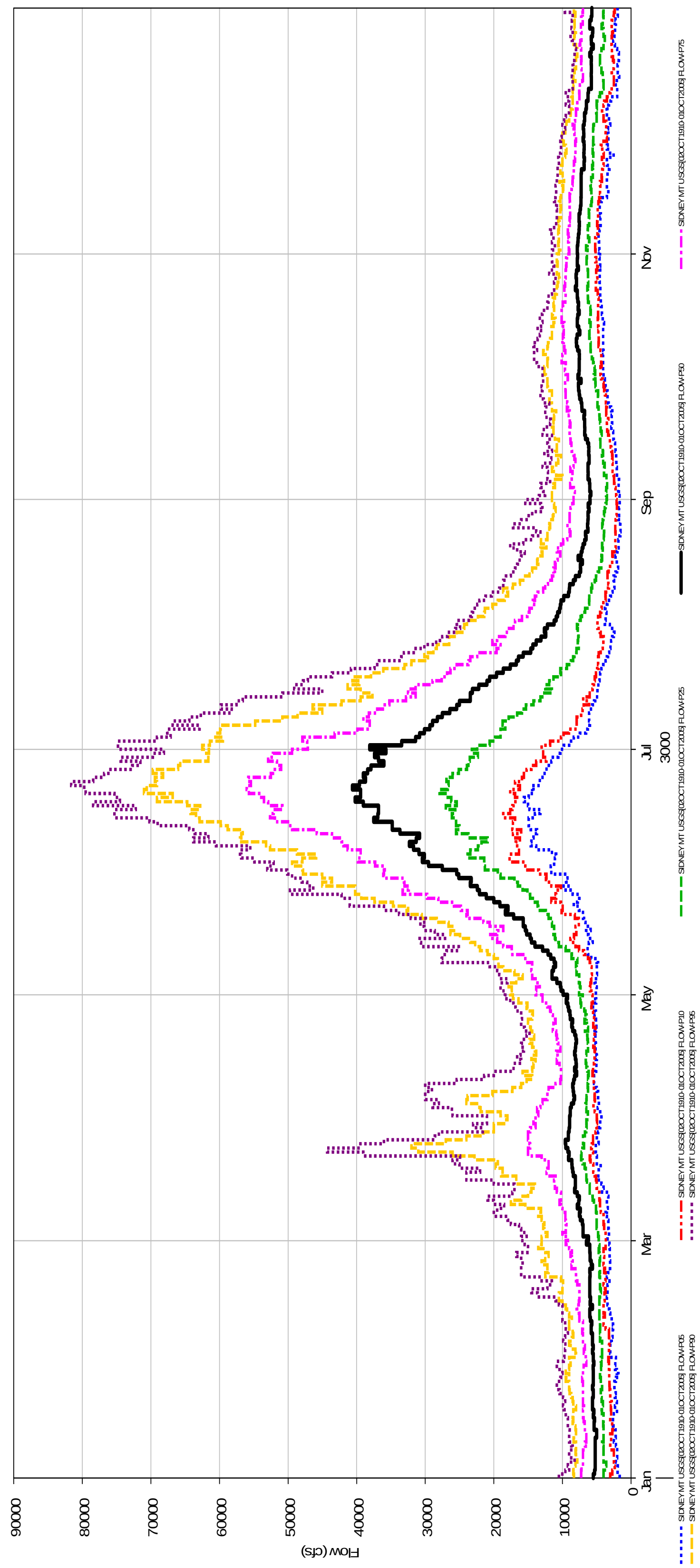


Figure B-4. 5th, 10th, 25th, 50th, 75th, 90th and 95th Daily Flow Percentiles for Period of Record, Sidney, MT, Yellowstone River

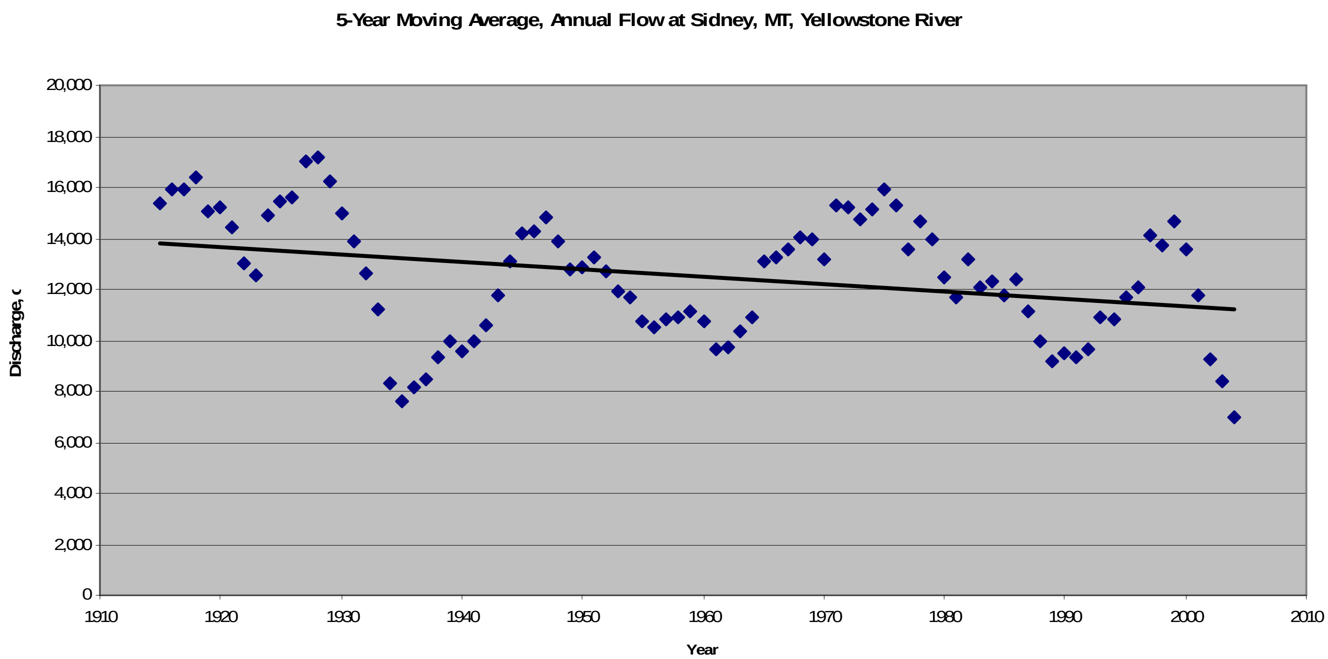


Figure B-5. 5-Year Moving Average Annual Flows at Sidney, MT, Yellowstone River

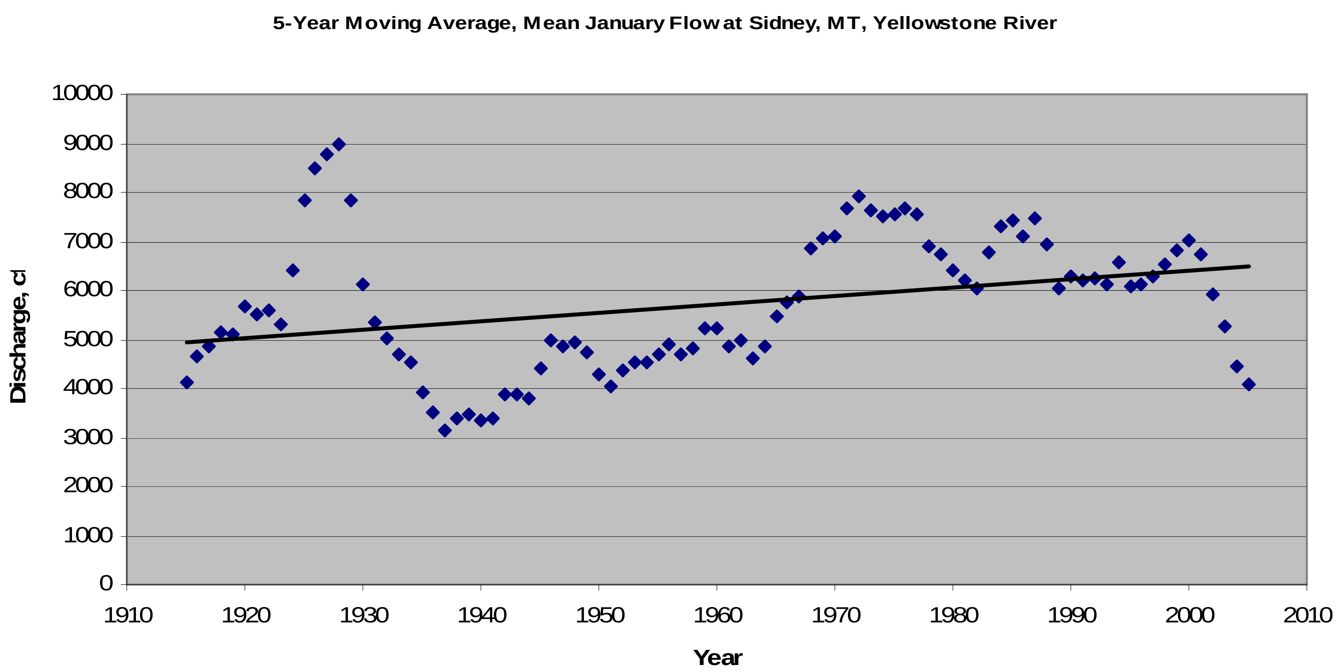


Figure B-6. 5-Year Moving Average Mean January Flows at Sidney, MT, Yellowstone River

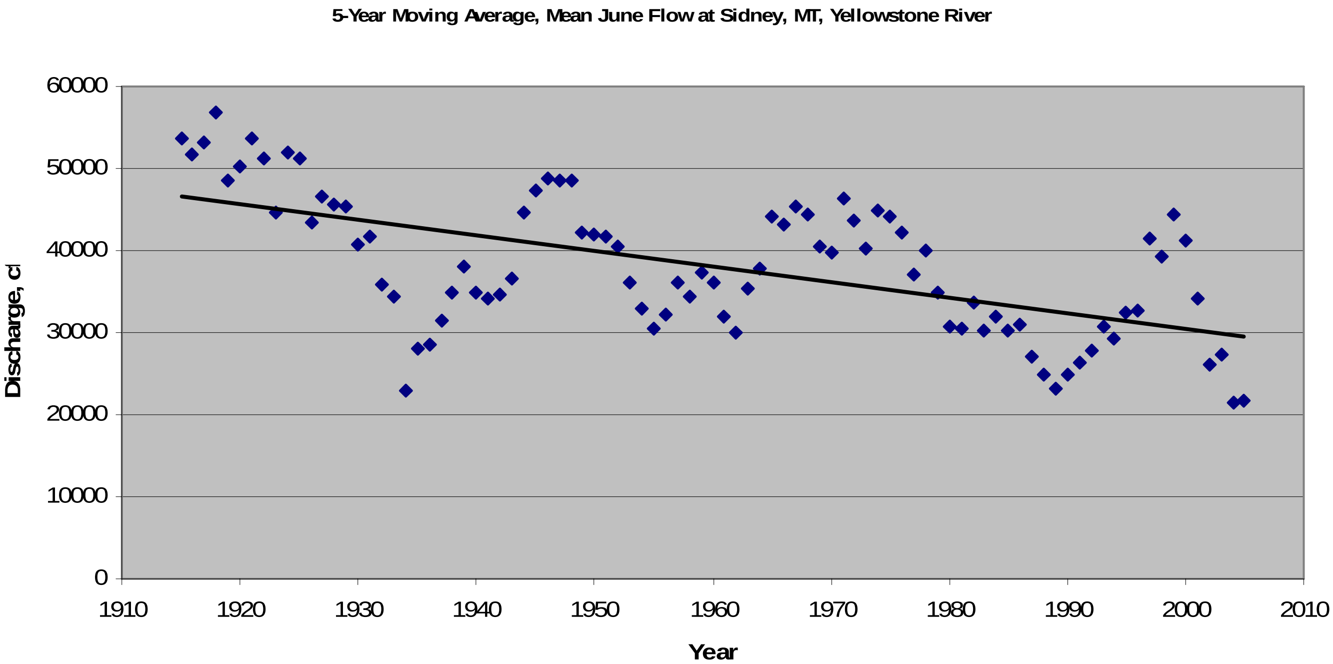


Figure B-7. 5-Year Moving Average Mean June Flows at Sidney, MT, Yellowstone River

The statistical analysis program HEC-STATS was utilized to compute the various flow-duration relationships. The following table contains a summary of all the flow-duration relationships derived, while figures of the relationships are attached as Figures B-8 to B-20. It should be noted that flow-duration curves are used to define the percent of time that a given flow is equaled or exceeded, and are not to be used to assess the probability of a given flow occurring.

Given the trend in flows over time noted above, flow-duration analysis was also performed over the various months since the completion of Yellowtail Dam in 1967 to see if any significant differences exist. Results are tabulated in Table B-2 below, and the resulting relationships are also shown in Figures B-8 to B-20.

As can be seen in the tables above, the computed flows generally correspond in a relative way to the flows shown in Figure B-4, with the notable exception of the month of October for flows that are exceeded 1% of the time or less in Table B-1. This is due to the occurrence of a large rainfall event that was centered near Savageton, WY and produced 17.1 inches of rain between September 27 and October 1, 1923, with most of the rain falling on the first two days of the storm event.

Table B-1. Annual and Monthly Flow-Duration Curves, Period of Record, Sidney, MT, Yellowstone River

Percent Time Flow Exceed or Equaled	Annual	Month: Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
0.01	125000	119000	16000	14000	17000	76000	114000	118000	104000	142000	112000	38000	39000
0.05	111000	105000	15000	14000	17000	53000	97000	116000	86000	134000	106000	38000	37000
0.1	95500	65700	13800	12900	16200	50900	93500	94000	82100	127000	101000	37000	33000
0.2	86400	32300	13300	12200	15600	39500	87800	84200	75100	121000	85000	34000	29000
0.5	74700	23800	12800	11500	14700	27500	59300	57400	61900	108000	80000	28000	22000
1	65500	19800	12300	11000	13500	22500	50500	38100	53200	93000	73200	25400	17900
2	55700	15700	11700	10200	12100	18200	39500	29800	48000	84600	66900	22600	15300
5	40100	12900	10900	9090	10100	13500	25900	19200	38800	72400	55700	19200	13000
10	28000	11400	10200	8480	8470	10400	18600	15700	31800	62900	42800	15700	11400
15	20500	10600	9670	8160	7760	9240	15500	14000	27400	57200	37400	13500	10200
20	15900	10100	9030	7860	7350	8510	13200	12800	24000	53300	33800	12100	9500
30	11200	9220	8250	7250	6660	7440	10800	10600	20200	46100	28100	10300	8470
40	9200	8470	7740	6690	6090	6580	9390	9300	17400	40400	24000	8820	7590
50	8010	7730	7230	6160	5430	5950	8350	8440	15000	35300	19800	7560	6560
60	7030	6930	6760	5420	5070	5280	7580	7740	13200	30700	16100	6320	5700
70	6150	6330	6230	4870	4520	4850	6530	6980	11500	26800	12800	5300	4940
80	5210	5690	5530	4210	3990	4420	5850	6200	9500	22100	9020	4150	4020
85	4730	5320	5090	3710	3560	4220	5310	5870	8370	19600	7630	3680	3630
90	4170	4840	4710	3130	3130	3880	4820	5500	7390	16600	6040	2960	3190
95	3270	4310	3980	2380	2320	3180	4160	5120	6070	13300	4580	2030	2620
98	2310	3950	3030	1720	1690	2560	3330	4110	5110	10300	2860	1510	1920
99	1820	3690	2190	1440	1440	2190	3170	3530	3200	8880	2260	1300	1590
99.5	1530	3520	1880	1210	1270	1990	2990	2690	2150	7580	1920	1140	1470
99.8	1280	3460	1670	1020	1110	1880	2640	2030	1500	6750	1520	1010	1390
99.9	1130	3430	1580	960	1030	1820	2360	1920	1120	6490	1450	960	1350
99.95	1030	3410	1520	910	980	1770	2080	1840	960	6440	1400	920	1310
99.99	850	3370	1460	830	910	1670	1880	1710	790	6360	1320	840	1240

Discharges in columns above in cubic feet per second (cfs)

Table B-2. Annual and Monthly Flow-Duration Curves, Post-Yellowtail Dam (Oct. 1967-Sept. 2005), Sidney, MT, Yellowstone River

Percent Time Flow Exceed or Equaled	Annual	Month											
		Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
0.01	87700	29800	16000	13000	15000	50000	75000	26100	104000	84900	76300	26800	37800
0.05	81300	29800	16000	13000	15000	50000	75000	26100	104000	84900	76300	26800	37800
0.1	76800	29700	14700	13000	15000	44500	68400	26100	87500	84900	76200	26800	33700
0.2	70500	23600	13300	12400	15000	39700	63100	22100	82900	83700	74500	26200	23800
0.5	60500	18800	12400	11800	14200	35900	53600	20700	51300	80200	67800	24700	21000
1	55200	14900	12000	11300	13200	22900	47800	19700	48200	74700	59400	22800	16700
2	47500	13700	11700	10500	12200	16900	39200	18100	45200	66400	53800	20500	14300
5	35700	12700	11300	9310	10600	14400	26900	16100	35900	59900	44000	16800	12800
10	25500	11700	10900	8790	9450	11600	17500	14500	31100	54700	37500	13800	11500
15	18400	11100	10400	8490	8750	10100	14400	13500	27000	49900	33500	12400	10500
20	14700	10700	10100	8290	8140	9460	12800	12500	23300	46200	30300	11500	9710
30	11300	9940	9480	7930	7510	8660	10900	10500	19400	40500	26300	9890	8780
40	9600	9380	8710	7490	7170	7970	9670	9160	16900	35400	21800	8230	7820
50	8460	8710	8080	7100	6600	7400	8720	8470	14800	30700	17100	7080	6660
60	7570	7740	7210	6560	6130	6530	8110	7830	13200	26800	14100	6010	5710
70	6650	6730	6650	5680	5420	5900	7100	7050	11500	22700	11100	4810	4970
80	5640	6010	5590	5020	4800	4910	6230	6130	9770	18700	7780	3980	4320
85	5100	5580	5140	4580	4400	4710	5880	5800	8450	16900	6700	3490	3910
90	4530	5120	4790	4210	4110	4490	5160	5470	7560	14900	5730	2710	3600
95	3800	4360	4160	3520	3210	4180	4200	5000	6230	12400	4930	1770	3060
98	2850	4040	3140	2730	2470	3440	3310	4180	5260	10000	3910	1470	2330
99	2130	3710	2230	2130	2160	2990	3110	3850	4530	8570	3590	1390	2020
99.5	1720	3500	1860	1940	1850	2770	2900	3560	2900	7730	3130	1330	1610
99.8	1430	3490	1600	1840	1360	2570	2630	3020	2380	7090	2460	1260	1460
99.9	1300	3480	1510	1780	1030	2500	2510	2970	2030	6530	2370	1220	1410
99.95	1190	3480	1480	1730	970	2480	2480	2950	1980	6500	2310	1190	1380
99.99	1030	3470	1450	1650	890	2450	2450	2900	1940	6480	2190	1130	1320

Discharges in columns above in cubic feet per second (cfs)

Table B-3. Recommended Annual and Monthly Flow-Duration Curves, Sidney, MT, Yellowstone River

Percent Time Flow Exceed or Equaled	Annual	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
		Month											
0.01	125000	119000	16000	14000	17000	76000	114000	118000	104000	142000	112000	38000	39000
0.05	111000	105000	16000	14000	17000	53000	97000	116000	104000	134000	106000	38000	37800
0.1	95500	65700	14700	13000	16200	50900	93500	94000	87500	127000	101000	37000	33700
0.2	86400	32300	13300	12400	15600	39700	87800	84200	82900	121000	85000	34000	29000
0.5	74700	23800	12800	11800	14700	35900	59300	57400	61900	108000	80000	28000	22000
1	65500	19800	12300	11300	13500	22900	50500	38100	53200	93000	73200	25400	17900
2	55700	15700	11700	10500	12200	18200	39500	29800	48000	84600	66900	22600	15300
5	35700	12700	11300	9310	10600	14400	26900	16100	35900	59900	44000	16800	12800
10	25500	11700	10900	8790	9450	11600	17500	14500	31100	54700	37500	13800	11500
15	18400	11100	10400	8490	8750	10100	14400	13500	27000	49900	33500	12400	10500
20	14700	10700	10100	8290	8140	9460	12800	12500	23300	46200	30300	11500	9710
30	11300	9940	9480	7930	7510	8660	10900	10500	19400	40500	26300	9890	8780
40	9600	9380	8710	7490	7170	7970	9670	9160	16900	35400	21800	8230	7820
50	8460	8710	8080	7100	6600	7400	8720	8470	14800	30700	17100	7080	6660
60	7570	7740	7210	6560	6130	6530	8110	7830	13200	26800	14100	6010	5710
70	6650	6730	6650	5680	5420	5900	7100	7050	11500	22700	11100	4810	4970
80	5640	6010	5590	5020	4800	4910	6230	6130	9770	18700	7780	3980	4320
85	5100	5580	5140	4580	4400	4710	5880	5800	8450	16900	6700	3490	3910
90	4530	5120	4790	4210	4110	4490	5160	5470	7560	14900	5730	2710	3600
95	3800	4360	4160	3520	3210	4180	4200	5000	6230	12400	4930	1770	3060
98	2850	4040	3140	2730	2470	3440	3310	4180	5260	10000	3910	1470	2330
99	2130	3710	2230	2130	2160	2990	3110	3850	4530	8570	3590	1390	2020
99.5	1720	3500	1860	1940	1850	2770	2900	3560	2900	7730	3130	1330	1610
99.8	1430	3490	1600	1840	1360	2570	2630	3020	2380	7090	2460	1260	1460
99.9	1300	3480	1510	1780	1030	2500	2510	2970	2030	6530	2370	1220	1410
99.95	1190	3480	1480	1730	970	2480	2480	2950	1980	6500	2310	1190	1380
99.99	1030	3470	1450	1650	890	2450	2450	2900	1940	6480	2190	1130	1320

Discharges in columns above in cubic feet per second (cfs)

It can also be noted that the post-Yellowtail Dam period has different flow-duration relationships in most months, sometimes with rather significant differences of over 40%. These differences are not unexpected, as Yellowtail Dam regulates approximately 28% of the basin upstream of Sidney (as well as numerous other smaller reservoirs upstream of Sidney), and reservoir operations can greatly alter the flow regime. It is therefore recommended that the post-Yellowtail Dam period flow-duration curves generally be followed in making design decisions for hydraulic performance of the various project features, as this period best represents the expected flow regime for the foreseeable future. The exception would be for flows occurring ~2% or less of the time, as there is still a substantial unregulated area subject to large rainfall and/or snowmelt events upstream of Sidney that could produce large flows. The recommended flow-duration curves are shown in Table 3 above.

Flow Frequency Analysis. Instantaneous peak flows were retrieved from the USGS website for the Sidney gage and input to an HEC-FFA input file. Data for water years 1911-1931 and 1934-2004 were available for download – a maximum instantaneous peak for WY2005 was not available, even though the daily mean data is available. The flow frequency program HEC-FFA was used to compute the annual flow-frequency relationship for the Sidney gage for the period of record. A Regional Skew value of 0.05 was used, as obtained from Figure 14-1 of Bulletin #17B. Table B-4 below contains the results of the analysis; showing computed and expected probabilities, as well as upper and lower confidence limits. Results are shown graphically in Figure B-21.

Table B-4. Flow-Frequency, Instantaneous Annual Peaks, Yellowstone River, Sidney, MT

	Discharge, cfs			
Percent Chance Exceedance	Computed Probability	Expected Probability	5% Confidence Limit	95% Confidence Limit
0.2	192,400	198,800	230,800	166,200
0.5	172,300	176,700	203,800	150,300
1	156,900	160,200	183,600	138,100
2	141,400	143,700	163,500	125,600
5	120,600	121,800	136,900	108,500
10	104,200	104,900	116,600	94,800
20	86,900	87,200	95,600	79,900
50	60,400	60,400	65,300	56,000
80	41,200	41,000	44,800	37,400
90	33,400	33,100	36,800	29,800
95	28,000	27,600	31,200	24,500
99	19,800	19,300	22,800	16,700

The maximum mean daily monthly flows were tabulated using HEC-DSSVue. These values were then input to the appropriate HEC-FFA input files, and HEC-FFA was used to compute the flow-frequency relationship of the maximum mean daily for each month. A regional skew value of 0.05 was used, even though the value as obtained from Bulletin #17B was based upon annual peaks; if further refinement of the regional skew value used for each month is desired, additional analyis would need to be considered.

Several months contained flows that were computed as either high or low outliers, so the appropriate adjustments were made to the statistics and frequency curves based upon their presence. Table B-5 below contains a summary of the results, with the expected probability shown for each month. Figures B-22 to B-33 graphically represent the computed and expected probability. It should be noted that flows are often estimated during the ice-cover and ice-breakup period, so computed monthly flow statistics may be adversely affected by the flow estimates in these periods.

The post-Yellowtail Dam period does show a trend towards lower peak annual and (some) peak monthly flows. However, due to the shorter period covered, the confidence limits cover a wider range of flows. Additionally, a true flow-frequency analysis of this more heavily regulated period would call for a far more rigorous methodology than called for in this study. Therefore, the adopted flow-frequency curves are those mentioned in the preceding paragraphs.

Summary of Results. Flow-duration curves were derived for the Yellowstone River at Sidney, Montana gage over the period of record, as well as the period since the completion of Yellowtail Dam, for the annual flow series, as well as each monthly flow series. The recommended flow-duration curves for design purposes are found in Table 3 of this document.

Flow-frequency curves were derived for the Yellowstone River at Sidney, Montana gage over the period of record available for the instantaneous annual peak, as well as the maximum mean daily flow for each month. The recommended curves for design purposes are found in Table B-5 of this report.

It is important to keep in mind the usefulness of each set of curves. The flow-duration curves should be utilized for meeting design purposes that consider the range of flow that is met a certain percent of the time (e.g., fish passage may require flow velocities between X and Y feet per second for Z percent of the time). The use of the annual or monthly flow-duration curves would be driven by the design parameter to be met. The flow-frequency curves should be utilized for meeting design purposes that require consideration of the likelihood of a given flow being exceeded in a given year (e.g., ice loading on fish passage structures). Use of the annual or monthly flow-frequency curves should be given weight as to which provides the more critical flow condition, as well as suitability to the design feature desired.

Examples of additional hydrologic analysis that may provide useful for final design of the project may include: number of years in which a given discharge is met, number of days in any given year that a given discharge is met, number of consecutive days in which a given discharge is met, rate of rise or fall in daily discharge during certain seasons, etc.

Table B-5. Summary of Expected Probability and Statistics for Annual and Monthly Flow-Frequency Curves

Percent Chance Exceedance	Annual	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.2	199000	22100	93000	277000	179000	121000	163000	162000	56200	55900	66900	17500	15500
0.5	177000	19500	66500	195000	125000	103000	147000	141000	48100	44500	49700	16300	14600
1	160000	17600	51200	148000	94000	90500	135000	126000	42300	37200	39500	15300	13800
2	144000	15700	39100	111000	70400	79000	122000	111000	36800	30800	31300	14300	13000
5	122000	13400	26900	73100	47200	64900	105000	90400	29800	23500	22800	12900	11900
10	105000	11600	19800	51700	34200	55000	91900	74700	24700	18700	17700	11800	10900
20	87200	9840	14100	34700	24000	45400	77400	58600	19600	14500	13500	10500	9850
50	60400	7220	8080	17300	13500	32200	54500	35300	12600	9210	8830	8380	7940
80	41000	5340	5120	9330	8560	23500	37300	20000	7980	6150	6440	6590	6280
90	33100	4570	4180	6930	7040	20200	30100	14400	6240	5070	5670	5780	5510
95	27600	4030	3600	5490	6110	17800	25000	10900	5070	4360	5190	5160	4920
99	19300	3170	2830	3650	4910	14300	17300	6110	3400	3340	4580	4130	3920
Mean	4.7748	3.8611	3.938	4.2615	4.1658	4.5167	4.7274	4.5292	4.0966	3.9789	3.9786	3.9194	3.8941
Standard Deviation	0.1932	0.1561	0.2651	0.3381	0.2707	0.1684	0.1874	0.2766	0.2302	0.2201	0.1981	0.1195	0.1152
Computed Skew	-0.2486	0.0883	1.0778	0.5622	1.0904	0.3418	-0.3685	-0.5715	-0.0703	0.545	2.0861	-0.235	-0.3707
Adopted Skew	-0.2	0.1	0.7	0.4	0.8	0.3	-0.3	-0.4	-0.1	0.4	1	-0.2	-0.3
High Outliers	0	0	1	0	2	0	0	0	0	0	1	0	0
Low Outliers	0	0	0	0	0	1	0	0	1	0	0	0	0

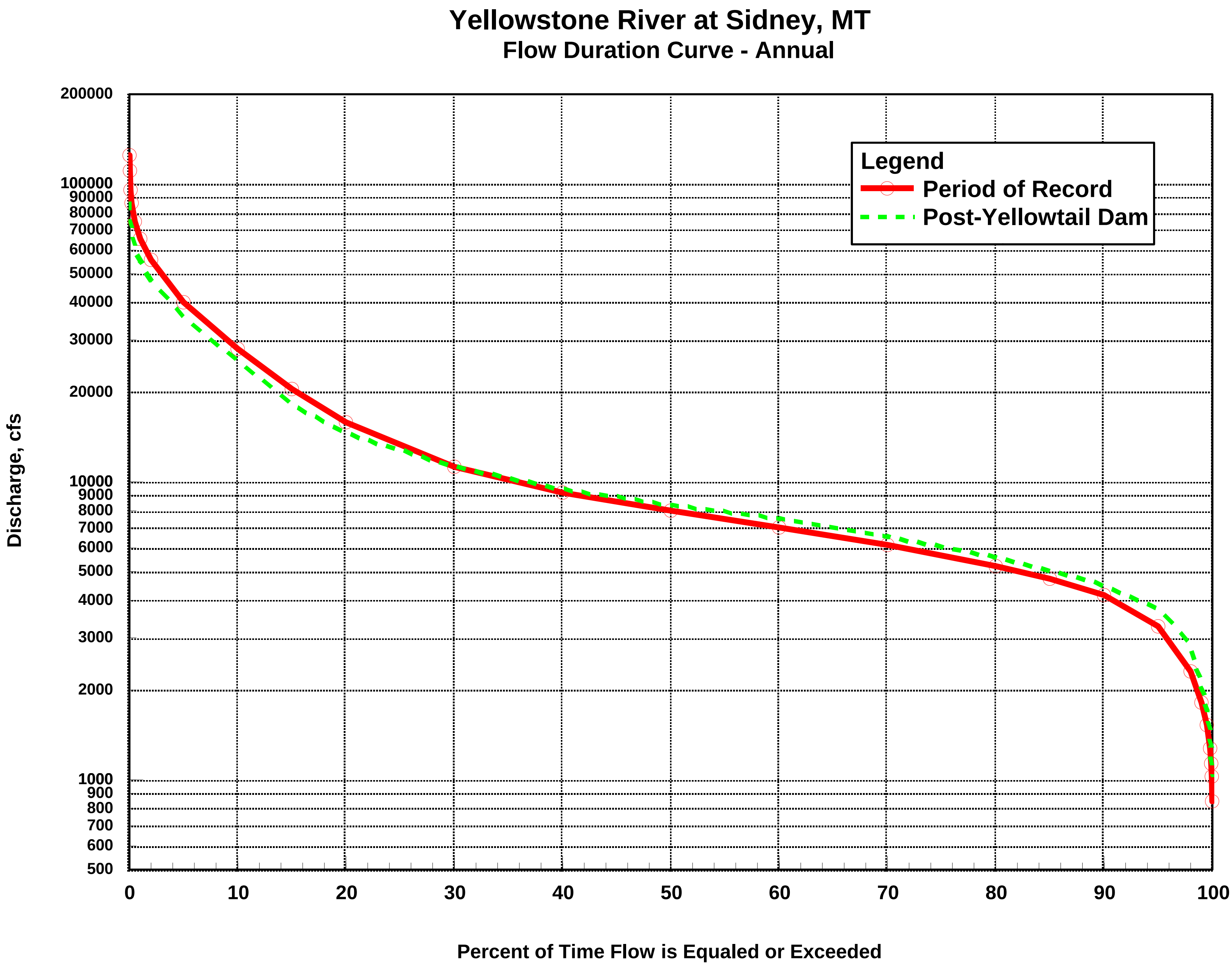


Figure B-8. Annual Flow-Duration Curves

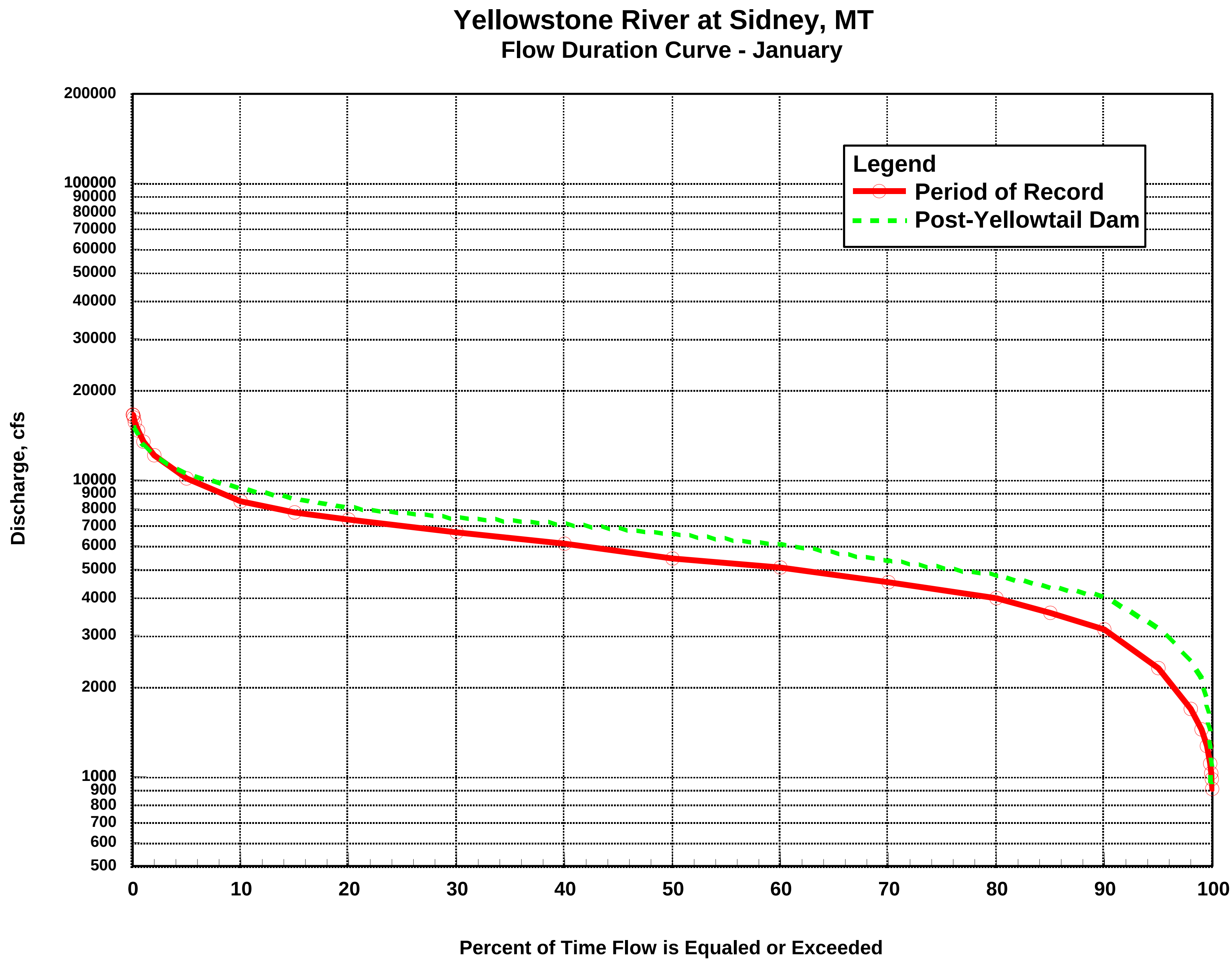


Figure B-9. January Flow-Duration Curves

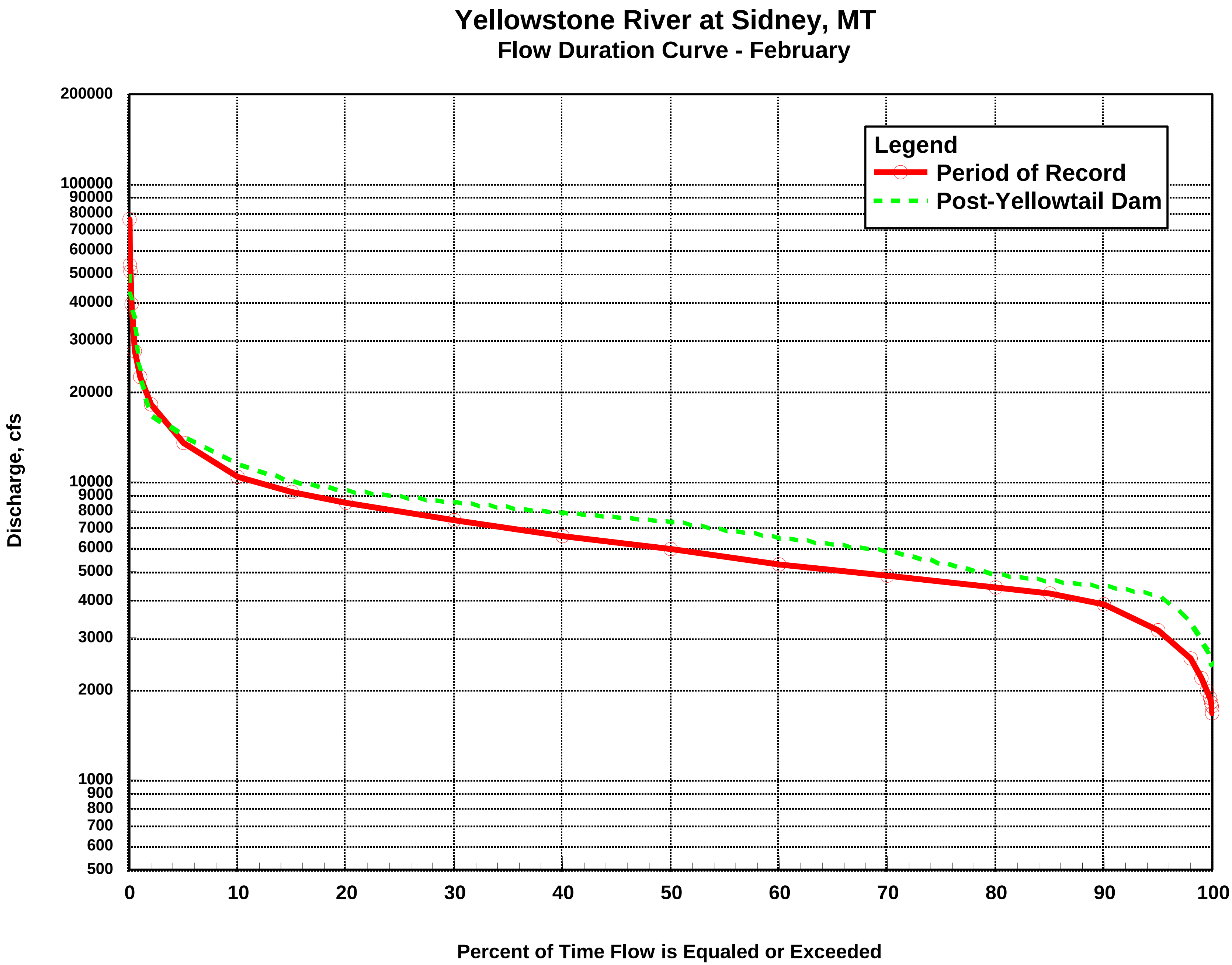


Figure B-10. February Flow-Duration Curves

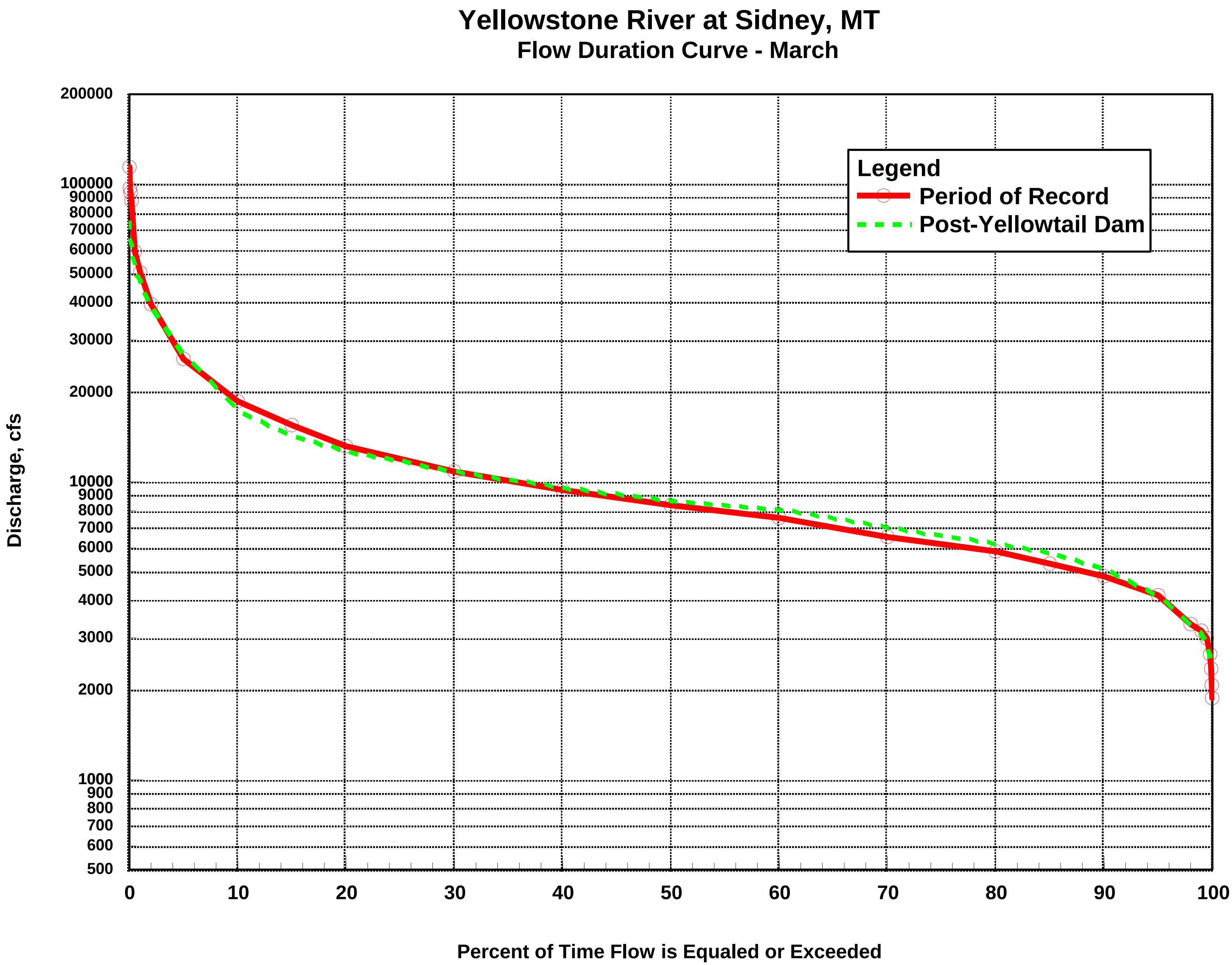


Figure B-11. March Flow-Duration Curves

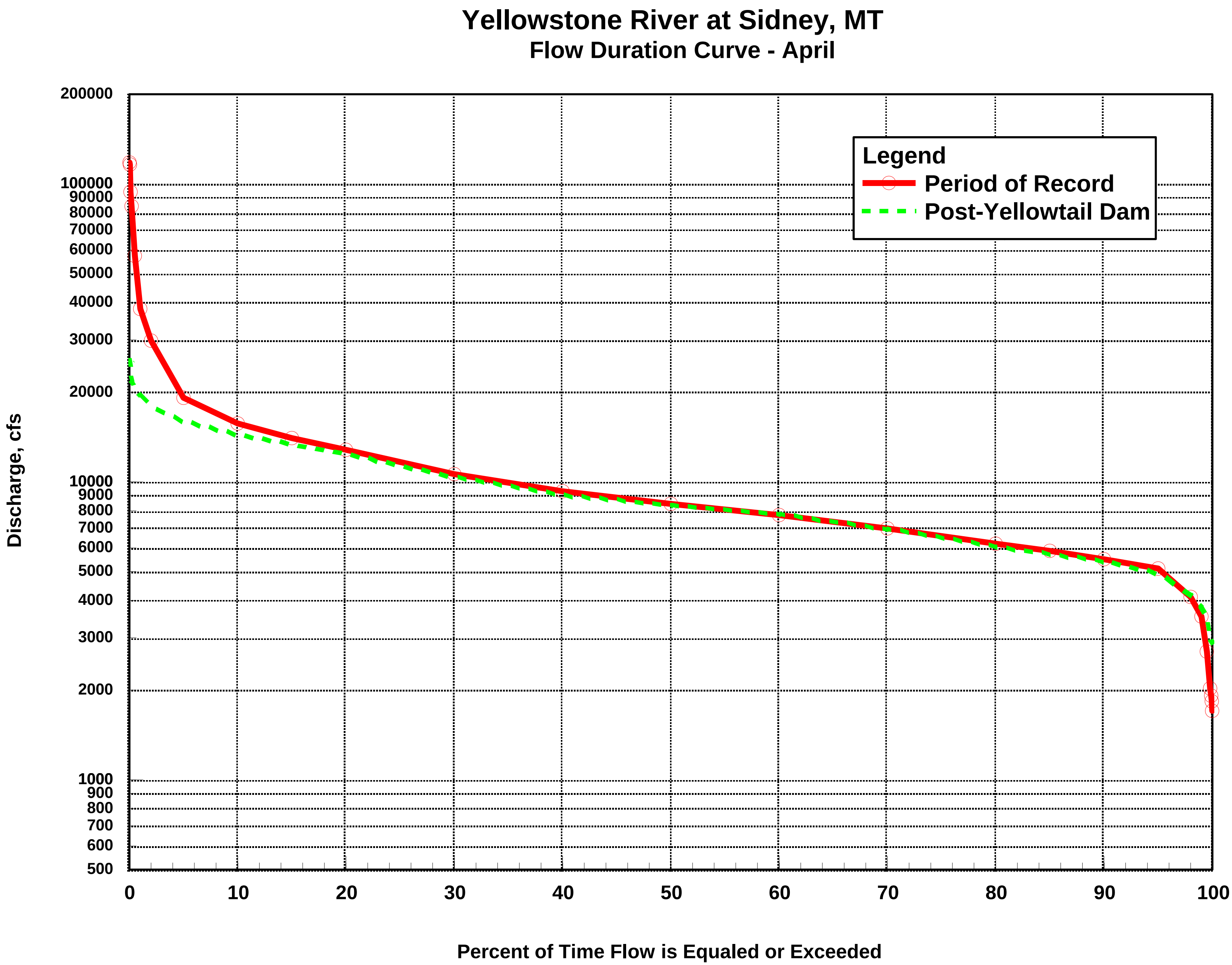


Figure B-12. April Flow-Duration Curves

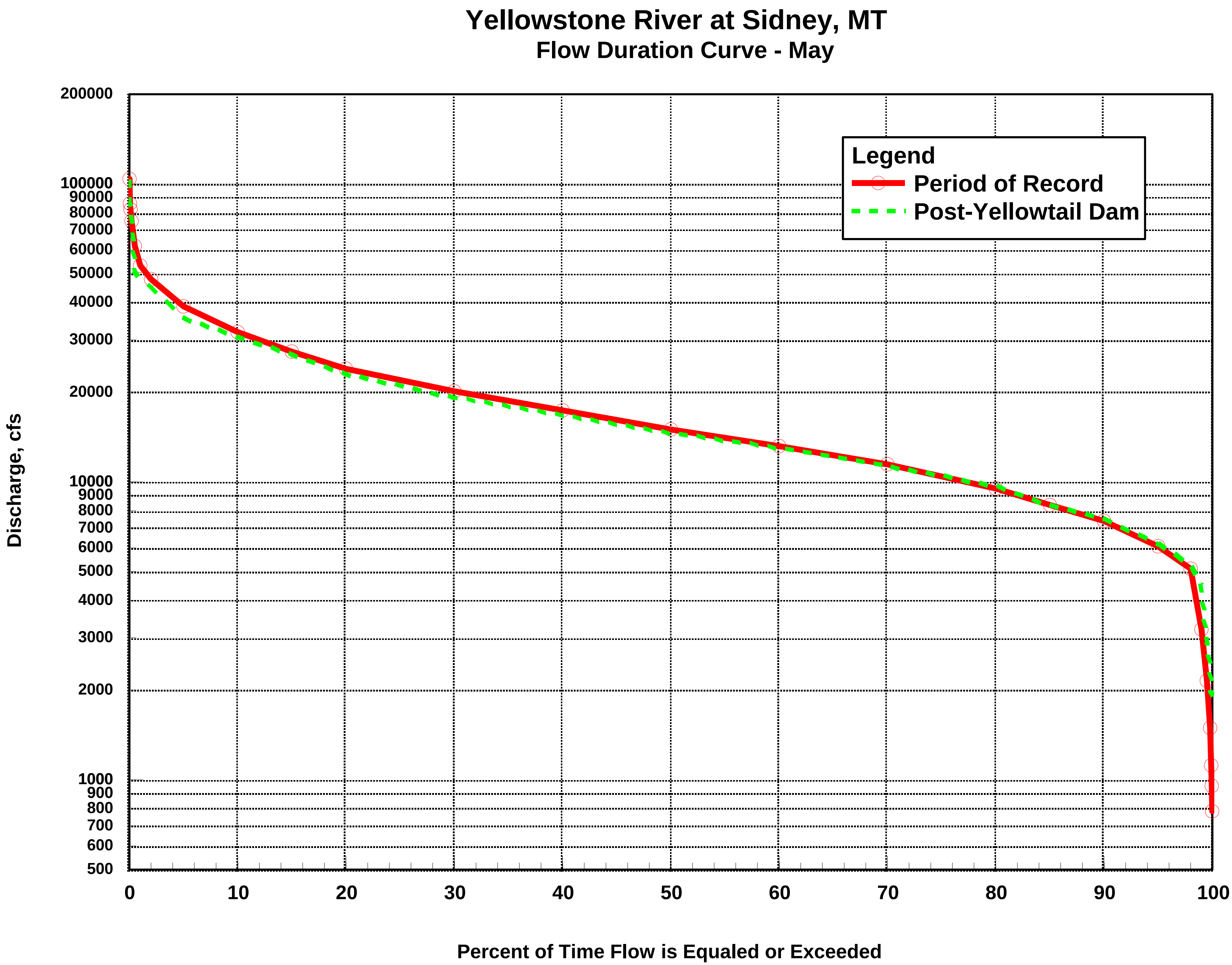


Figure B-13. May Flow-Duration Curves

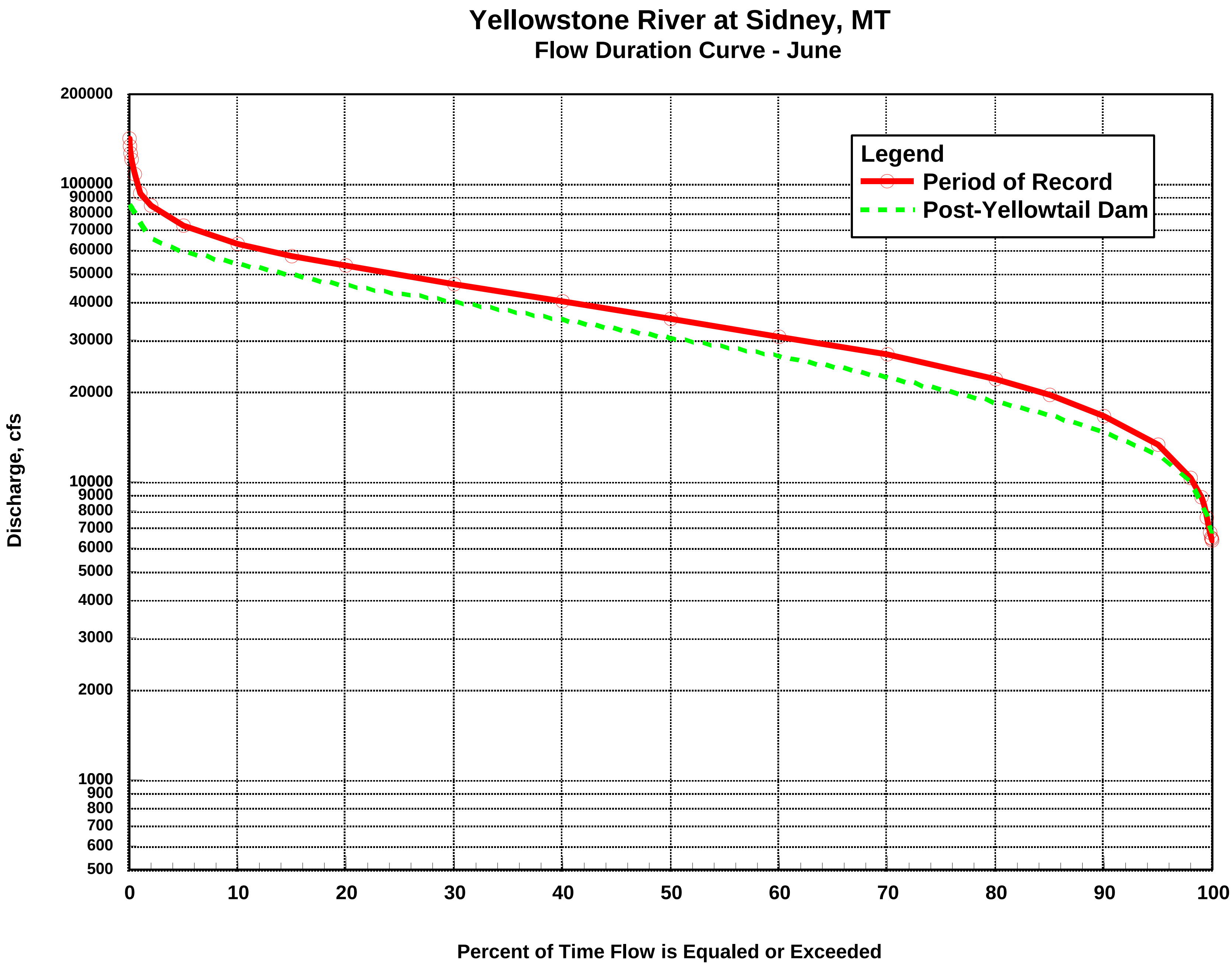


Figure B-14. June Flow-Duration Curves

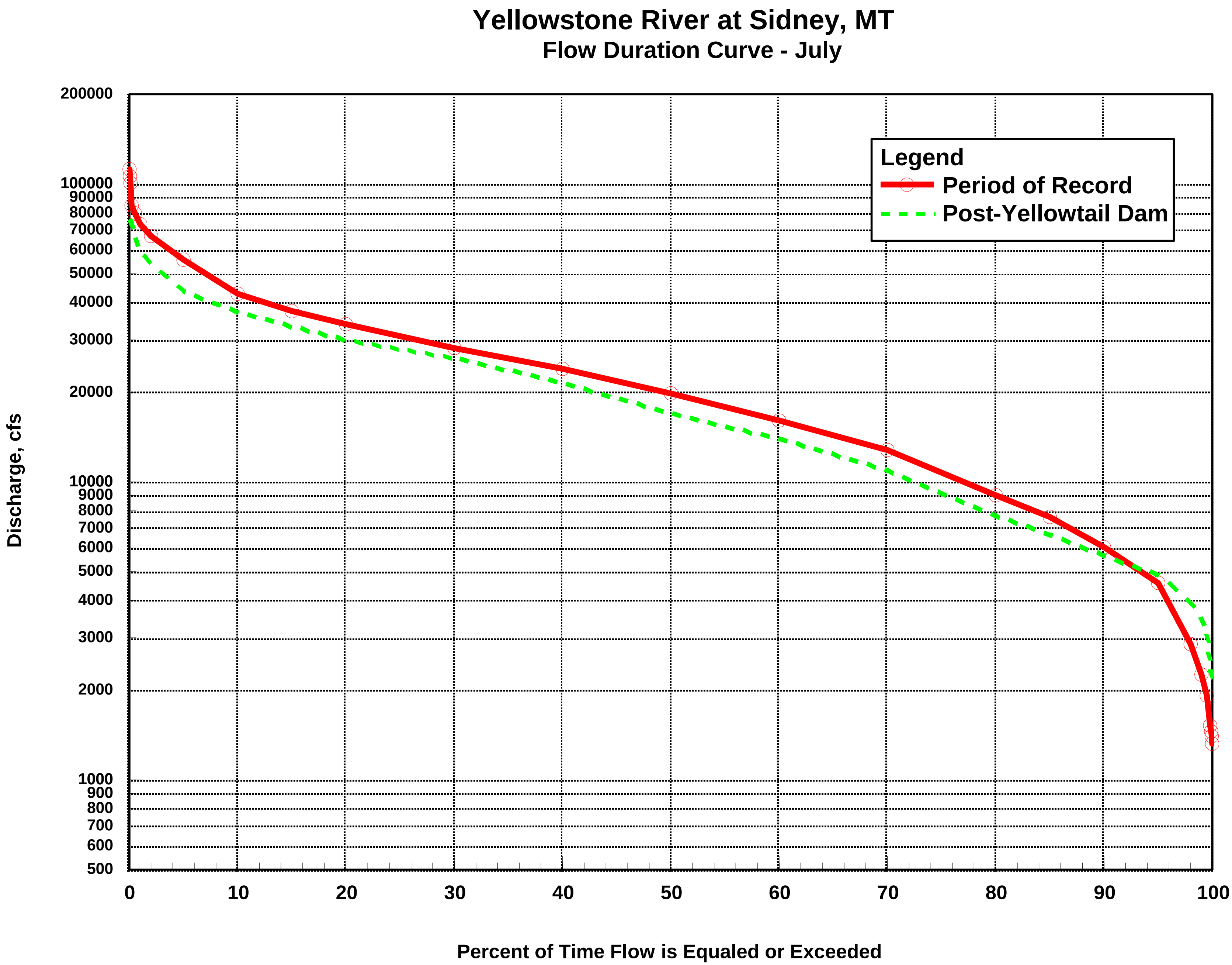


Figure B-15. Flow-Duration Curves

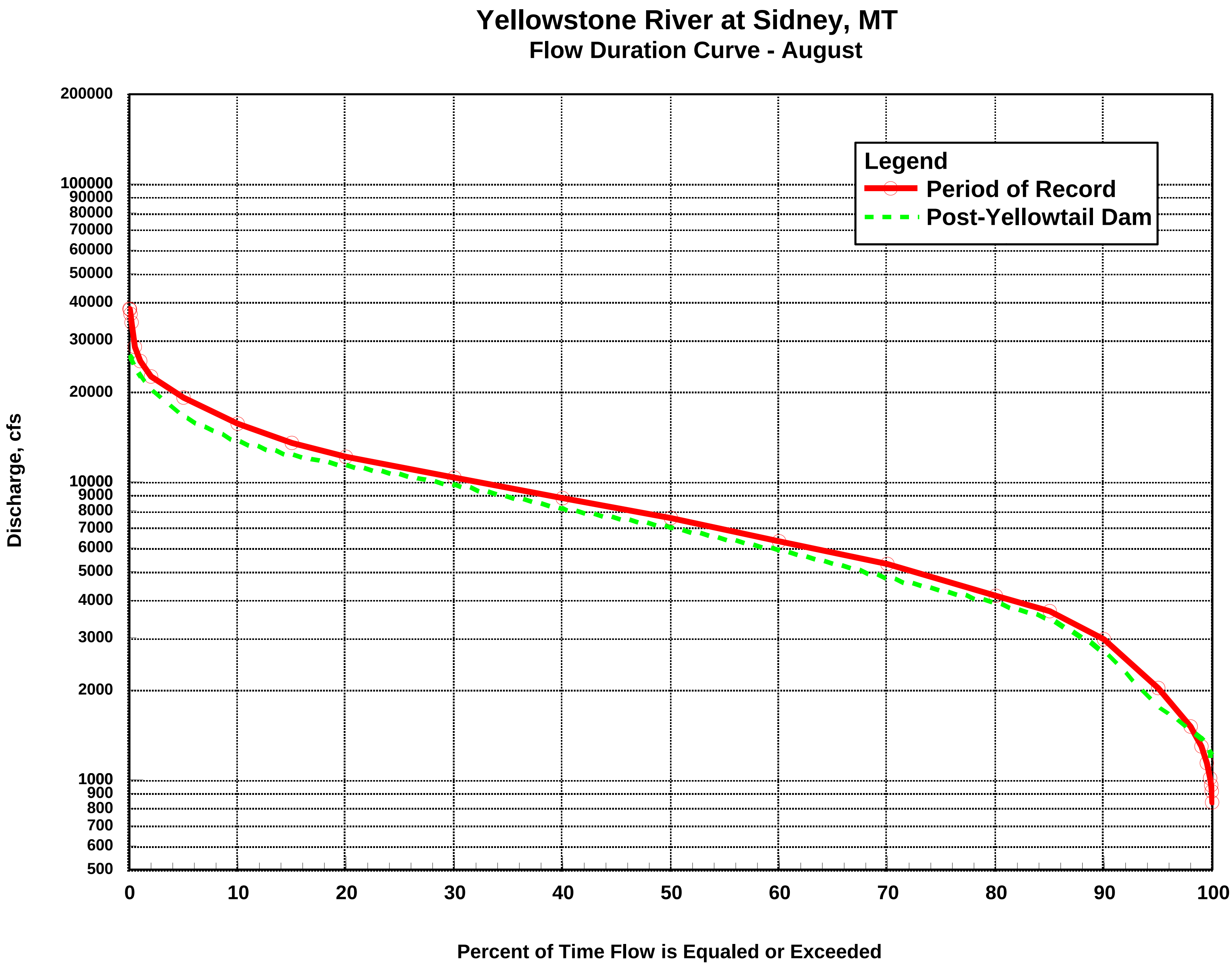


Figure B-16. August Flow-Duration Curves

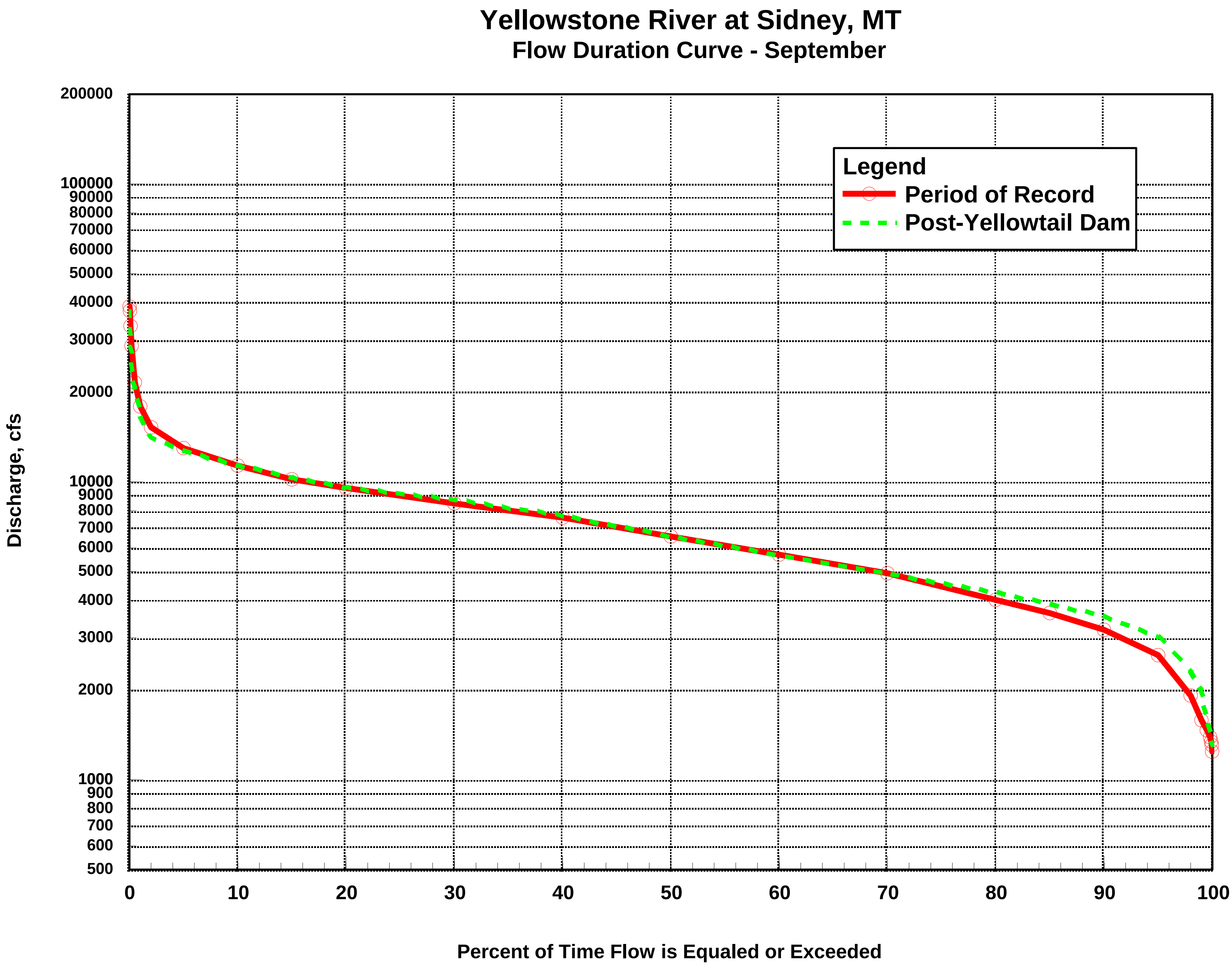


Figure B-17. September Flow-Duration Curves

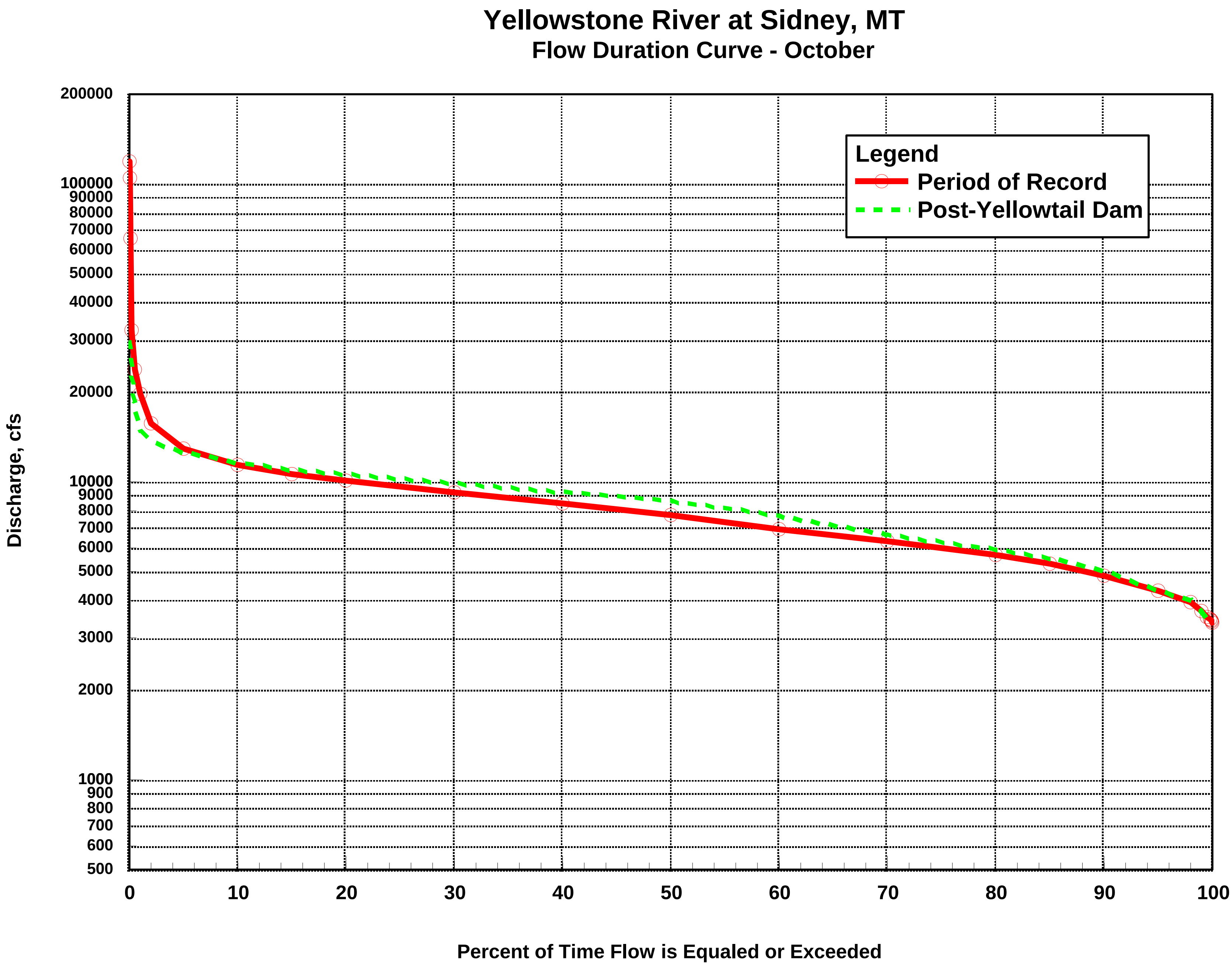


Figure B-18. October Flow-Duration Curves

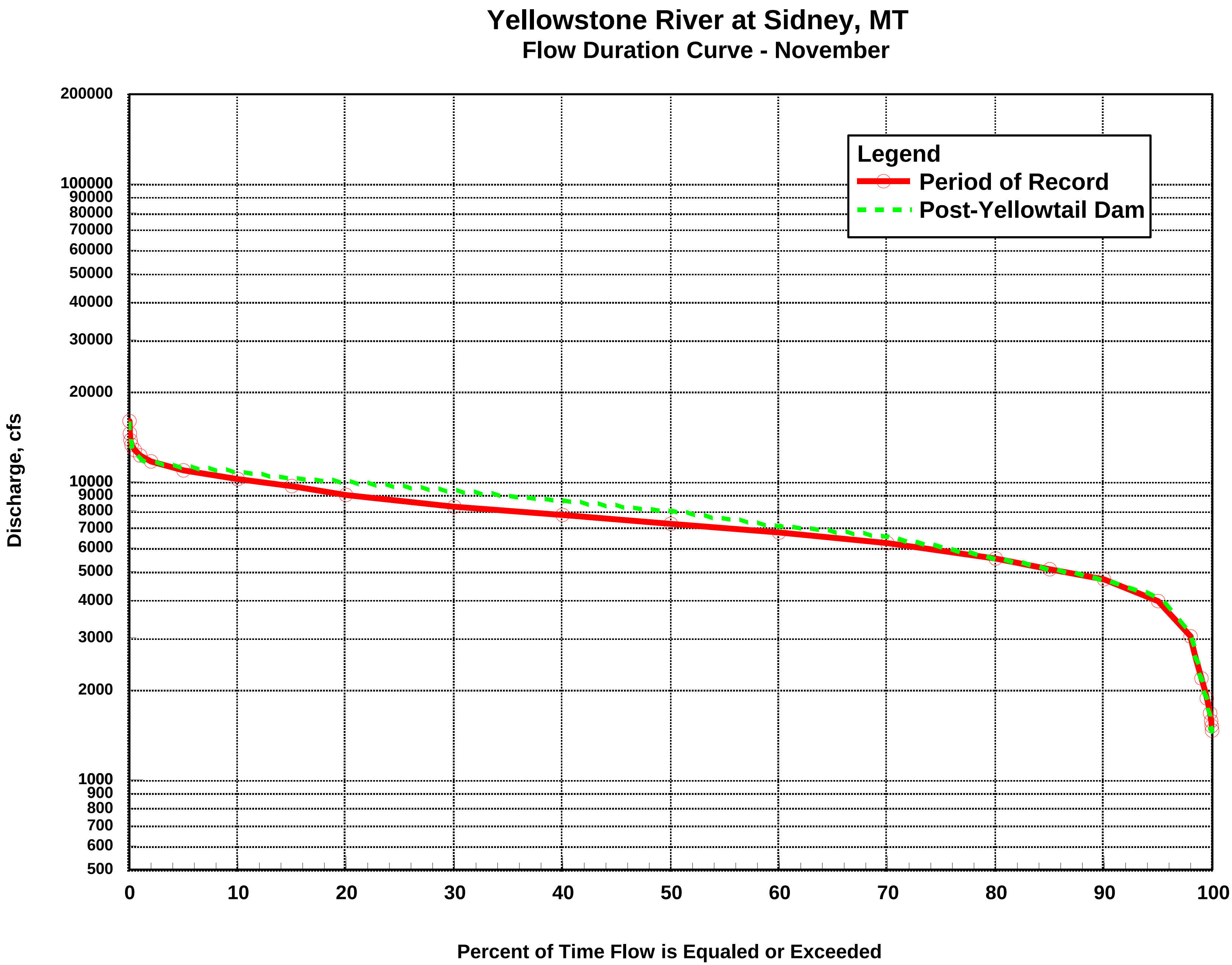


Figure B-19. November Flow-Duration Curves

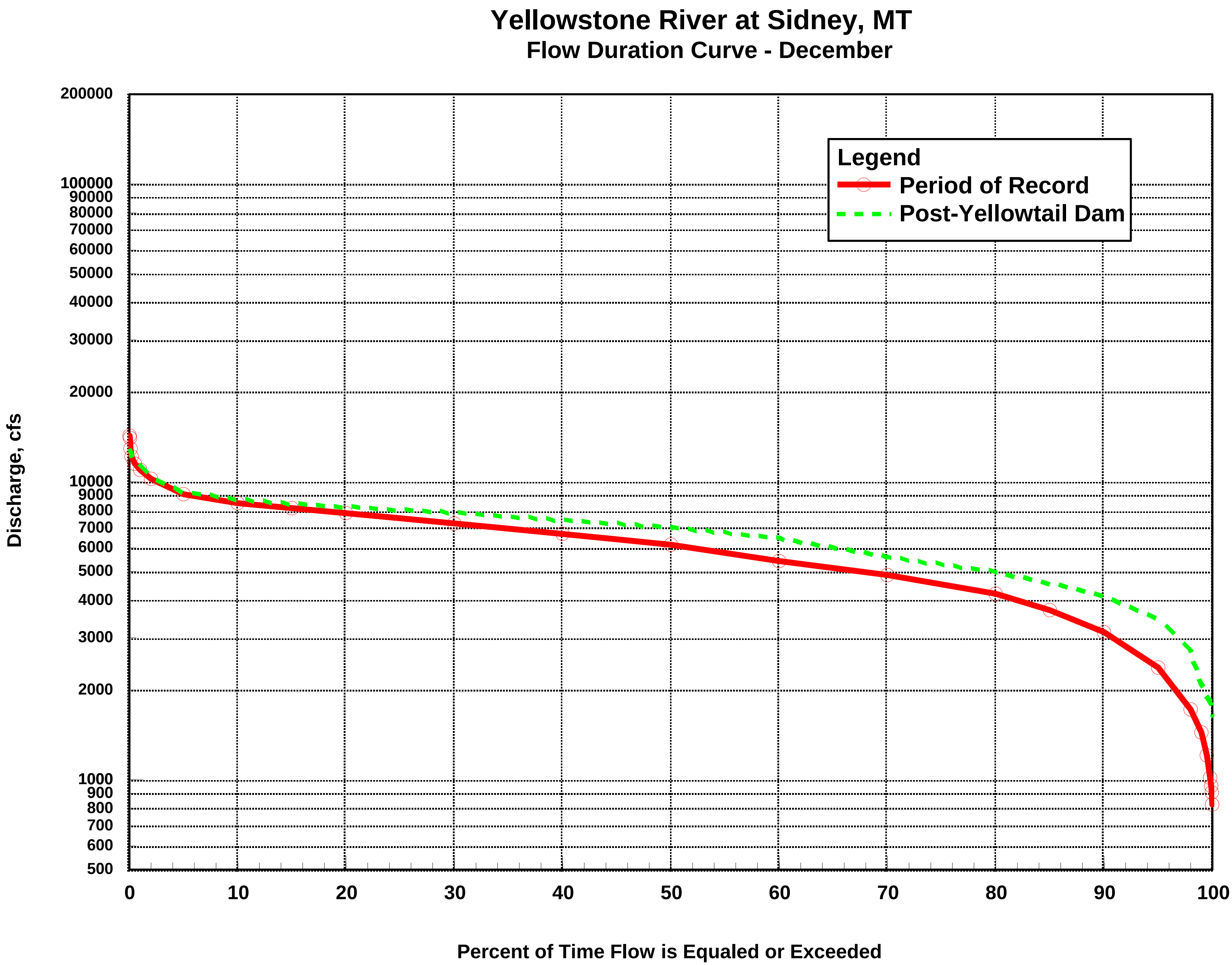


Figure B-20. December Flow-Duration Curves

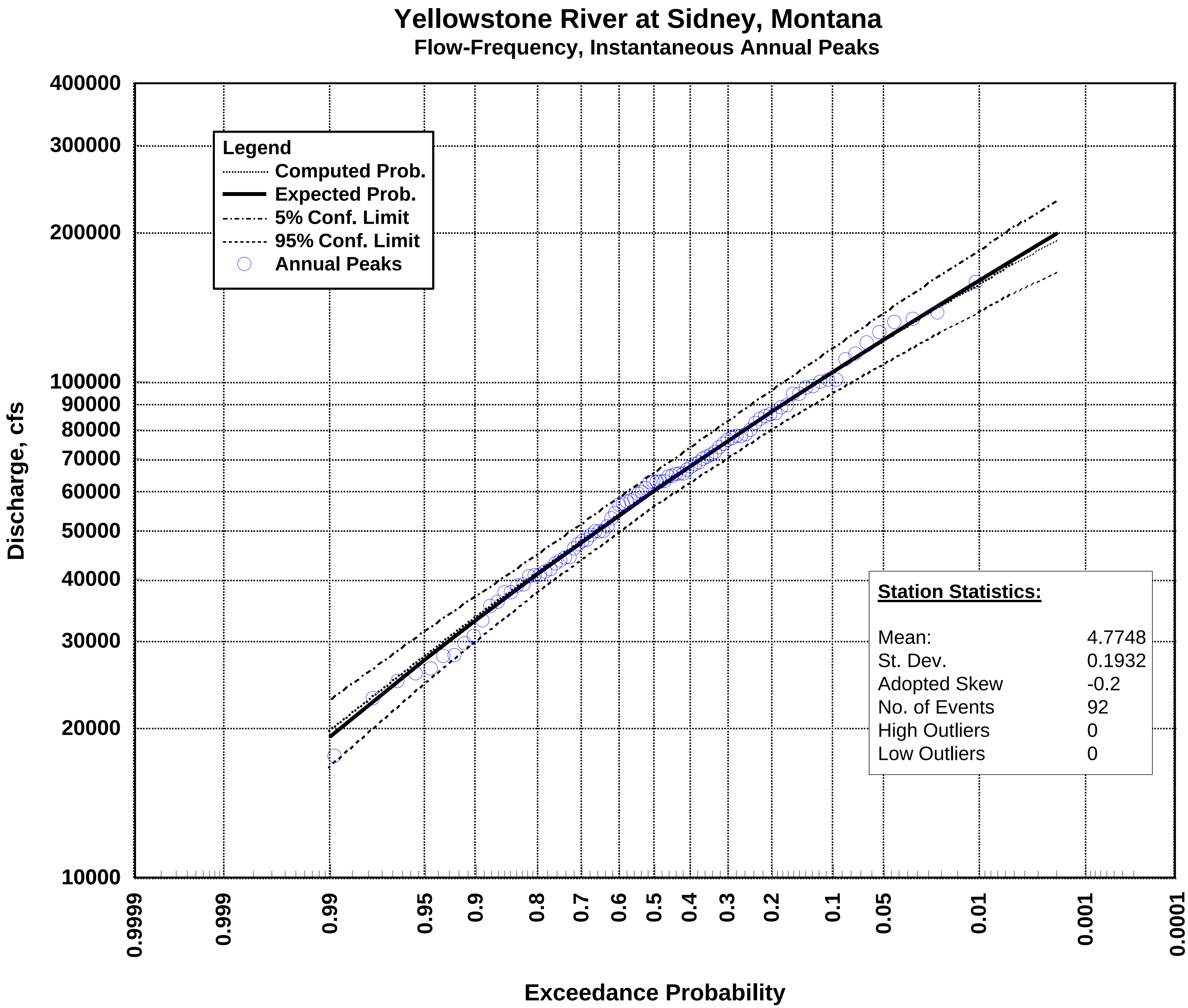


Figure B-21. Annual Flow-Frequency Curve

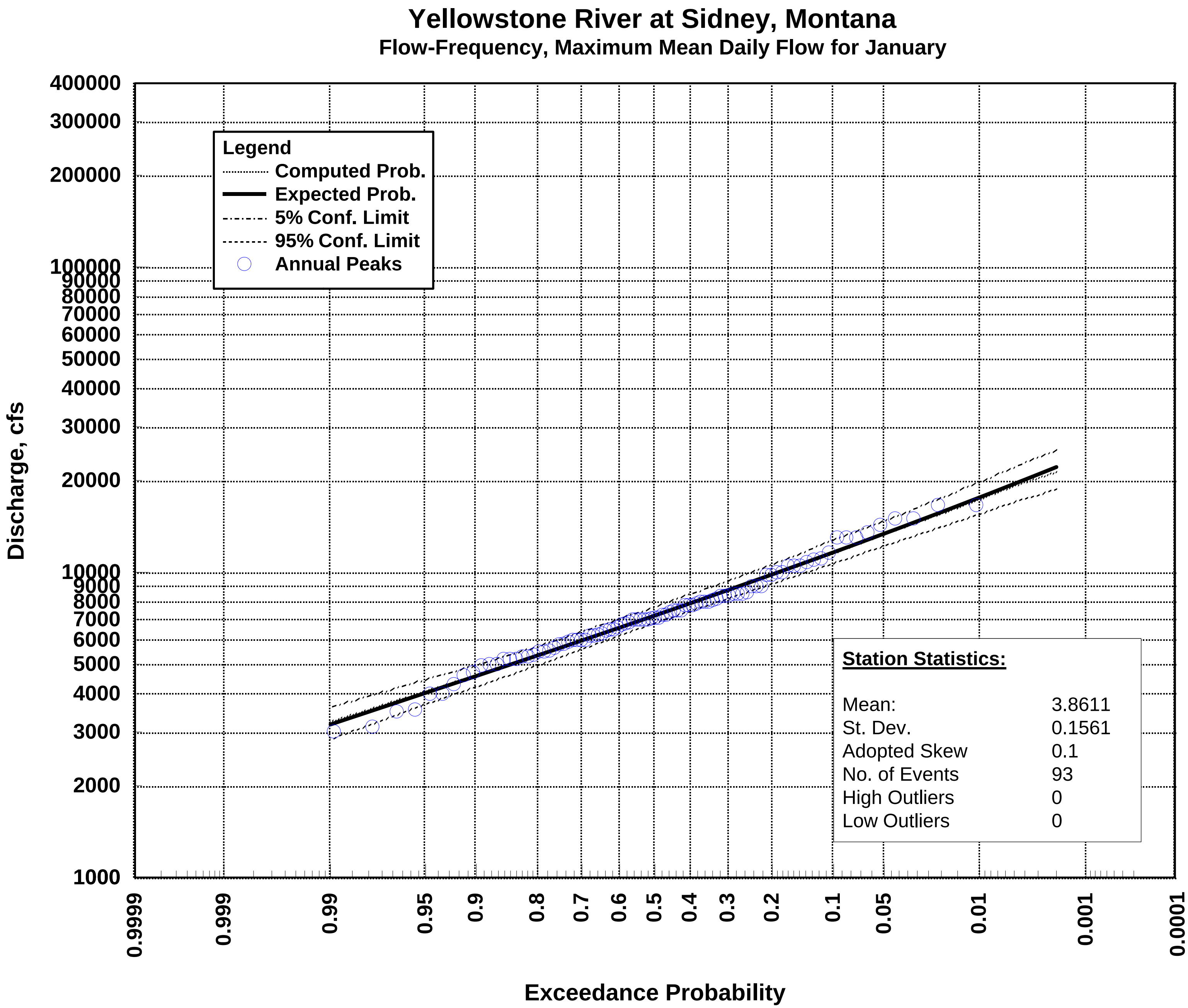


Figure B-22. Flow-Frequency Curve for January

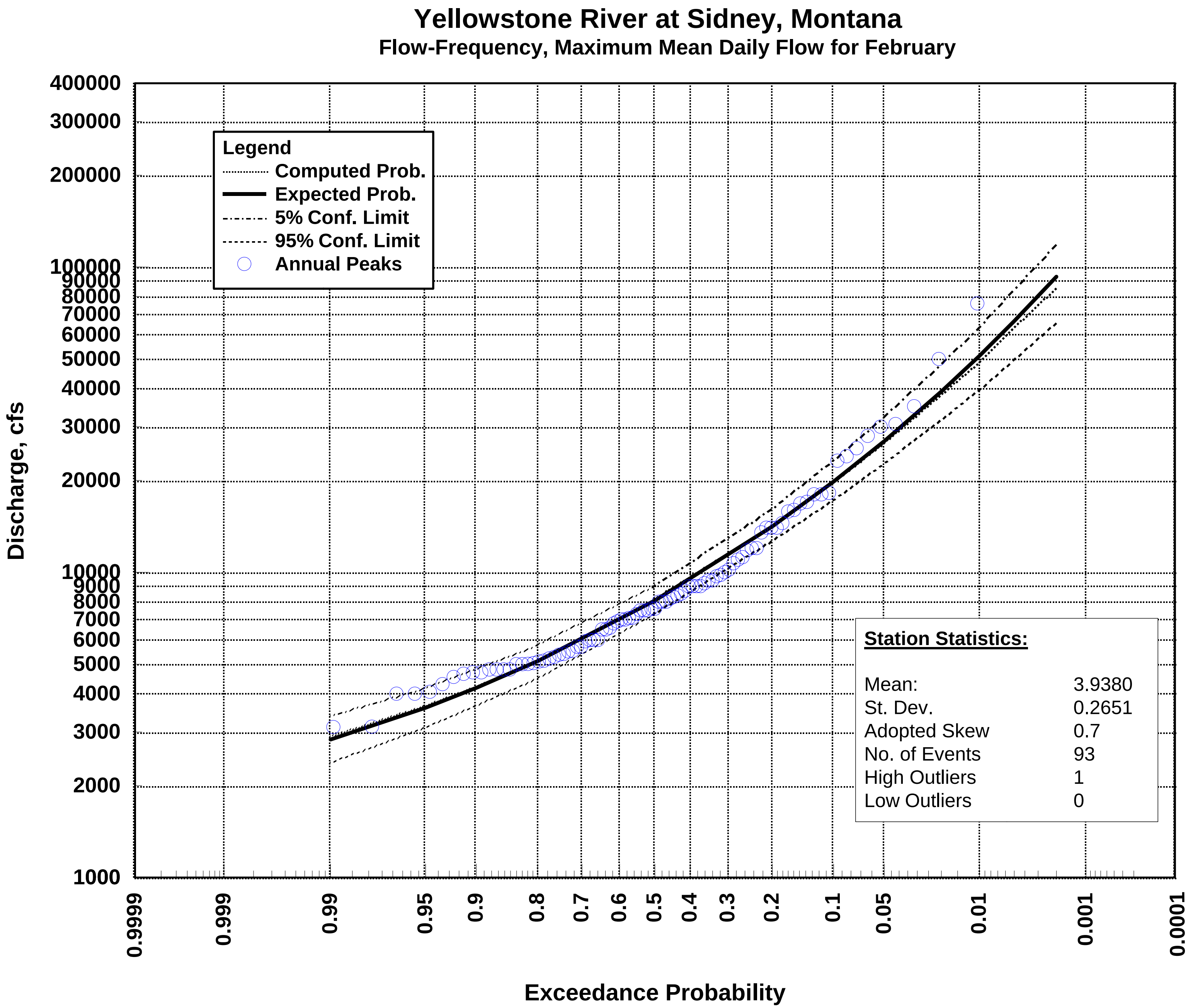


Figure B-23. Flow-Frequency Curve for February

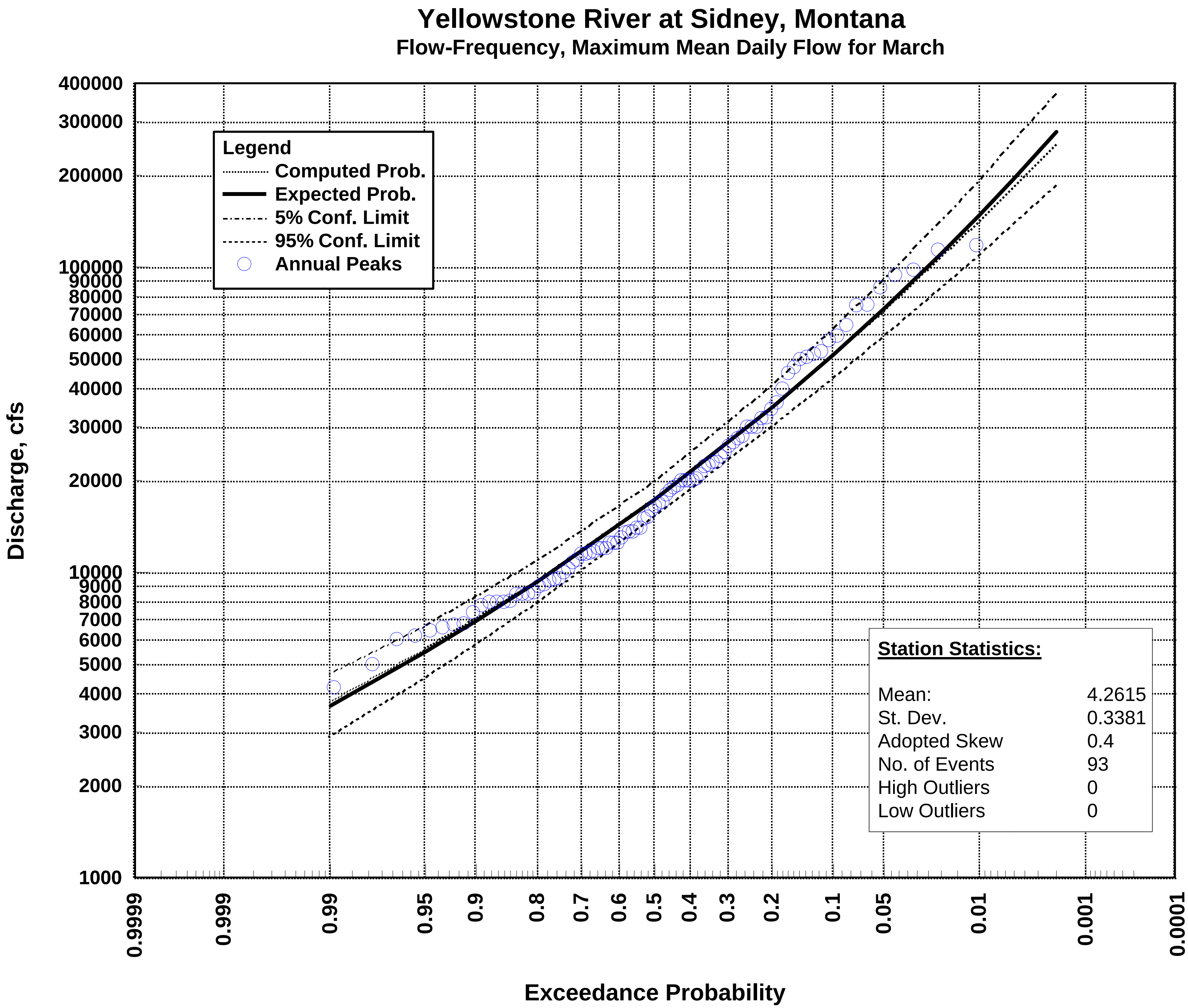


Figure B-24. Flow-Frequency for March

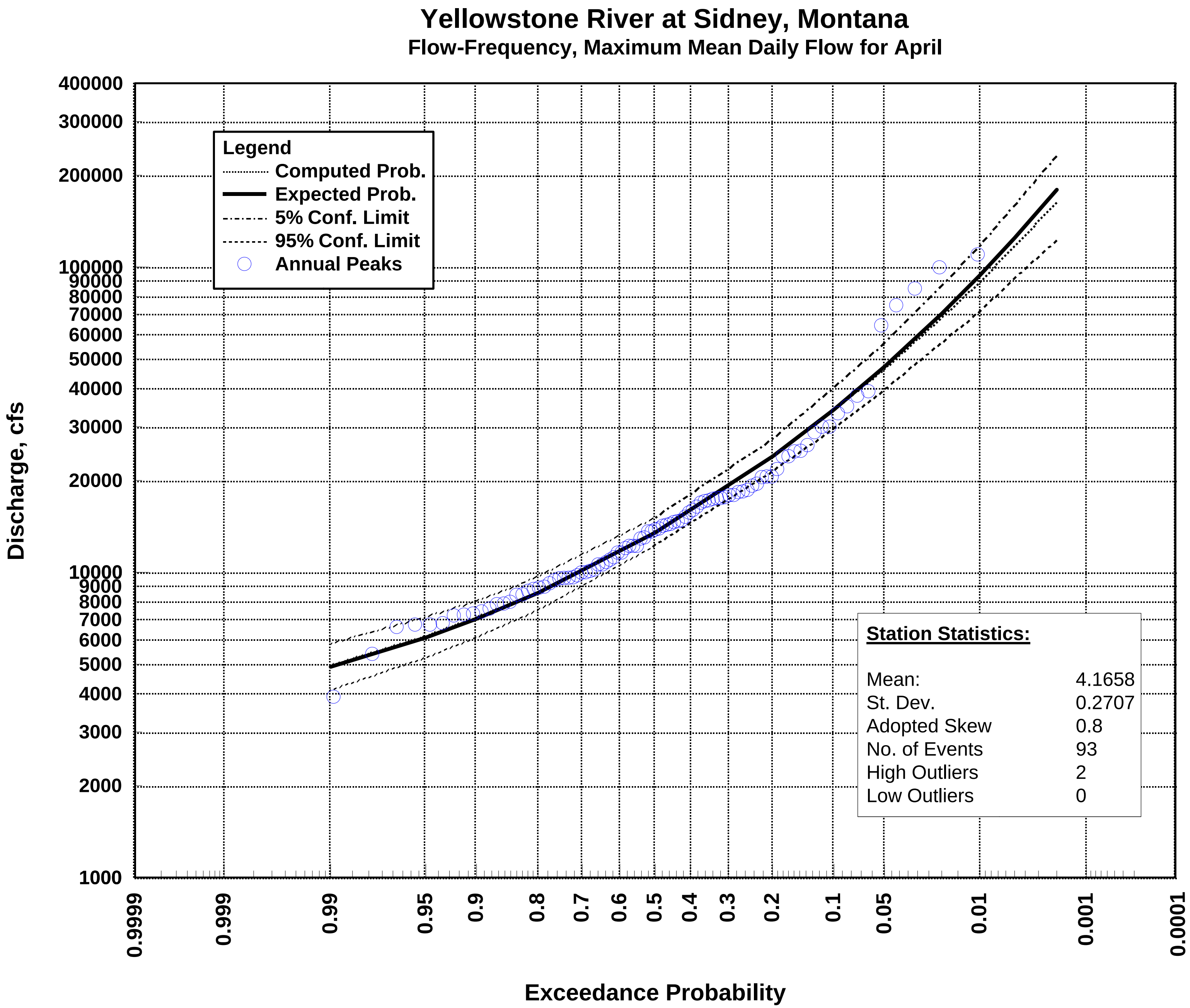


Figure B-25. Flow-Frequency for April

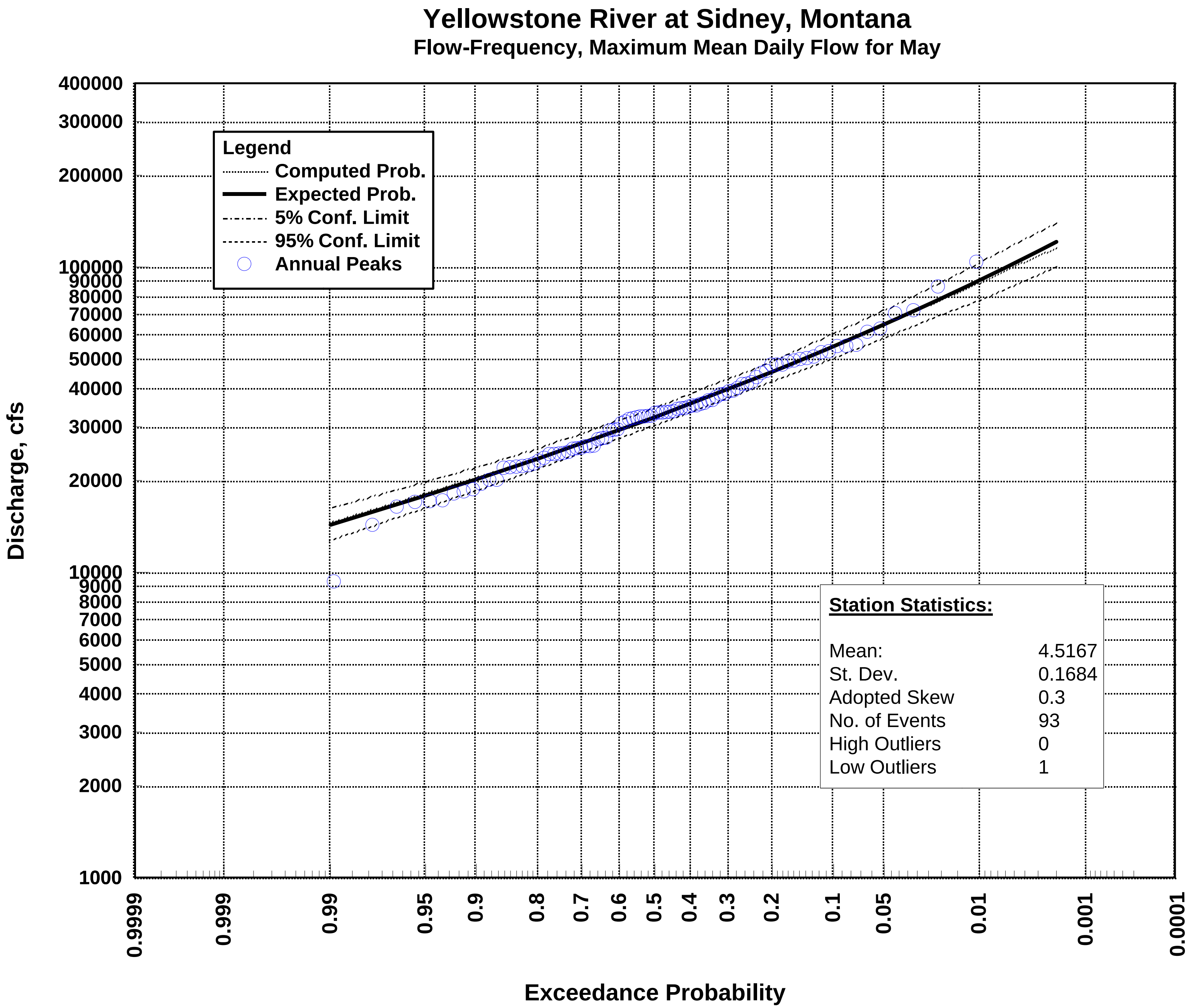


Figure B-26. Flow-Frequency for May

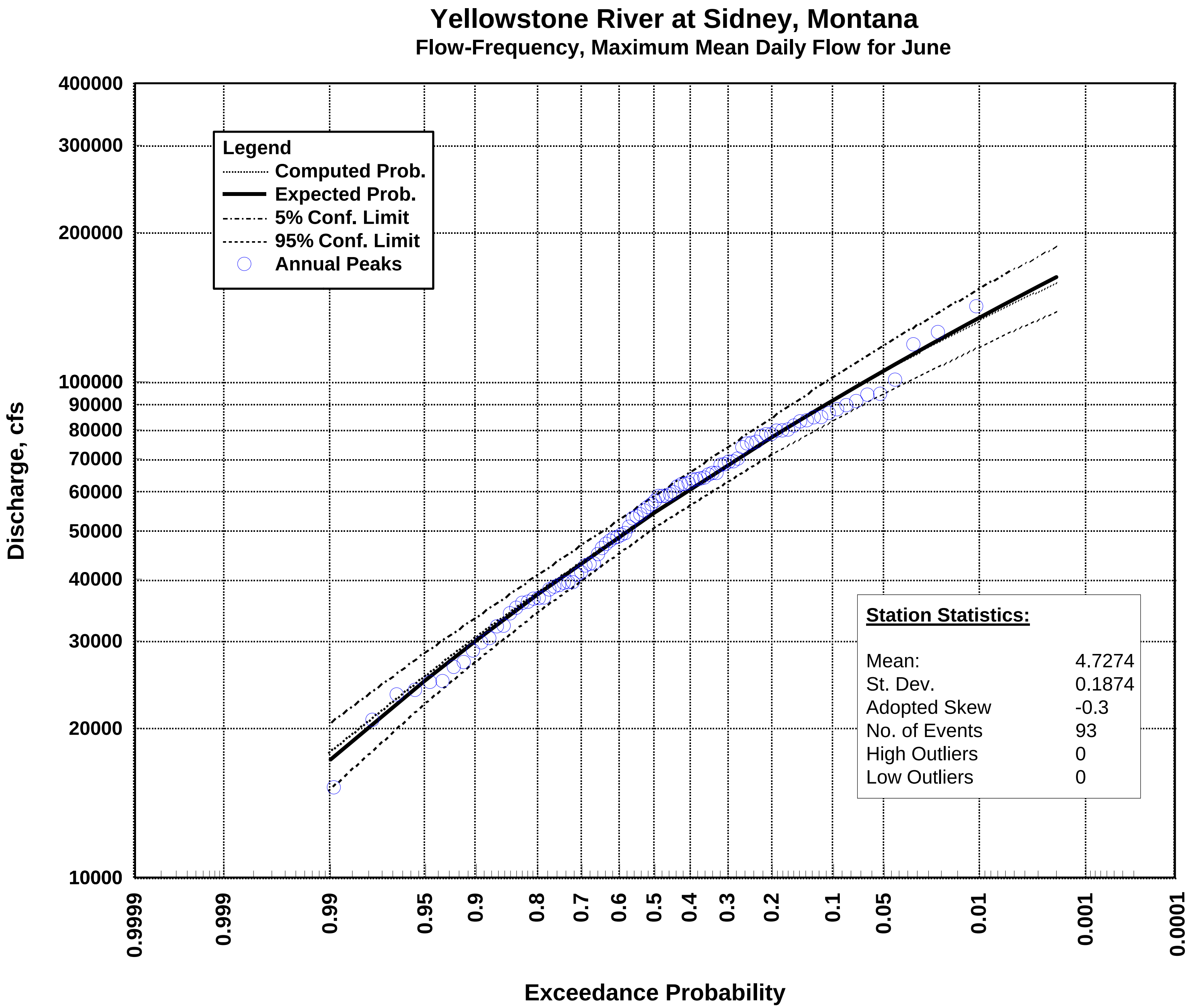


Figure B-27. Flow-Frequency for June

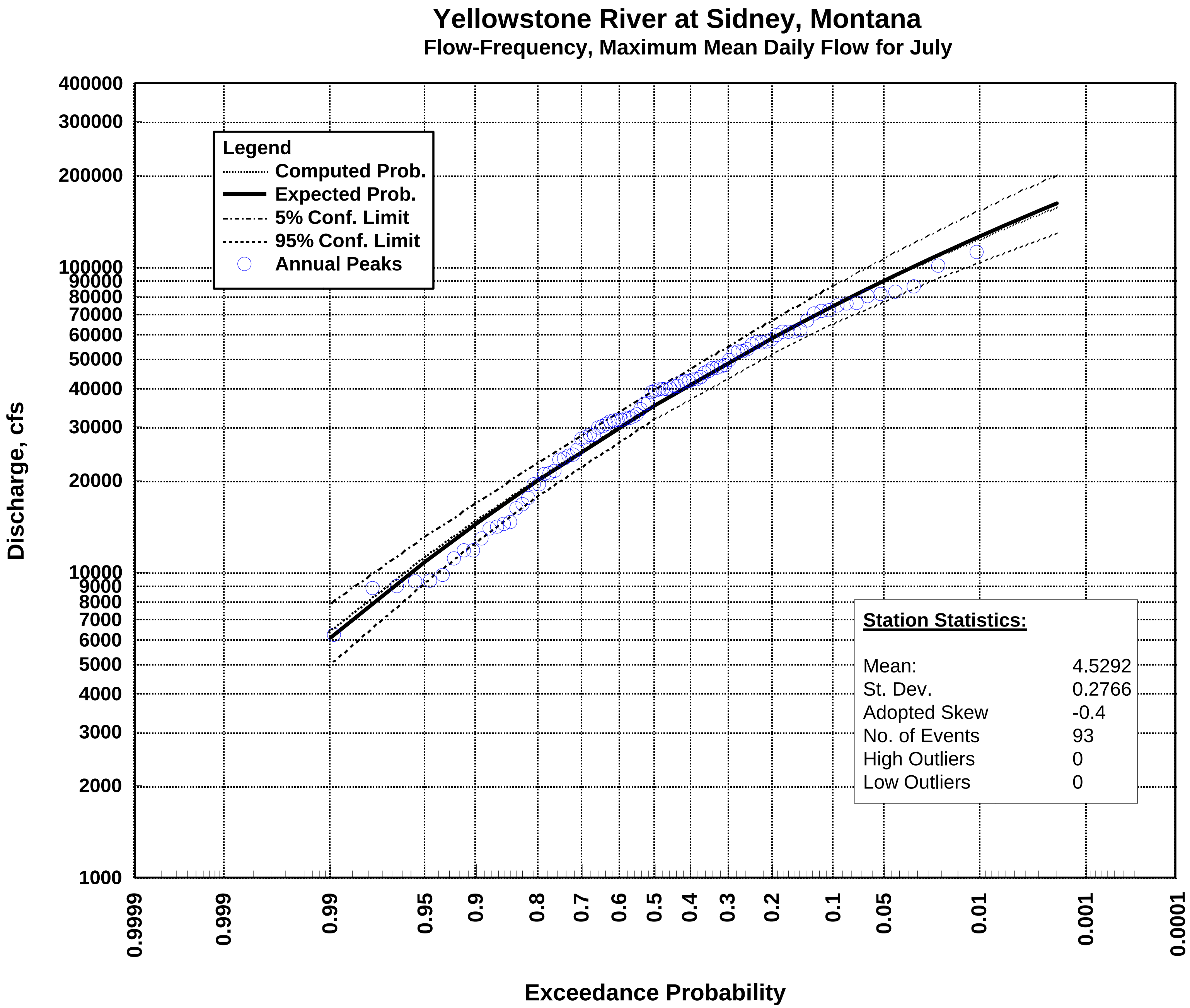


Figure B-28. Flow-Frequency for July

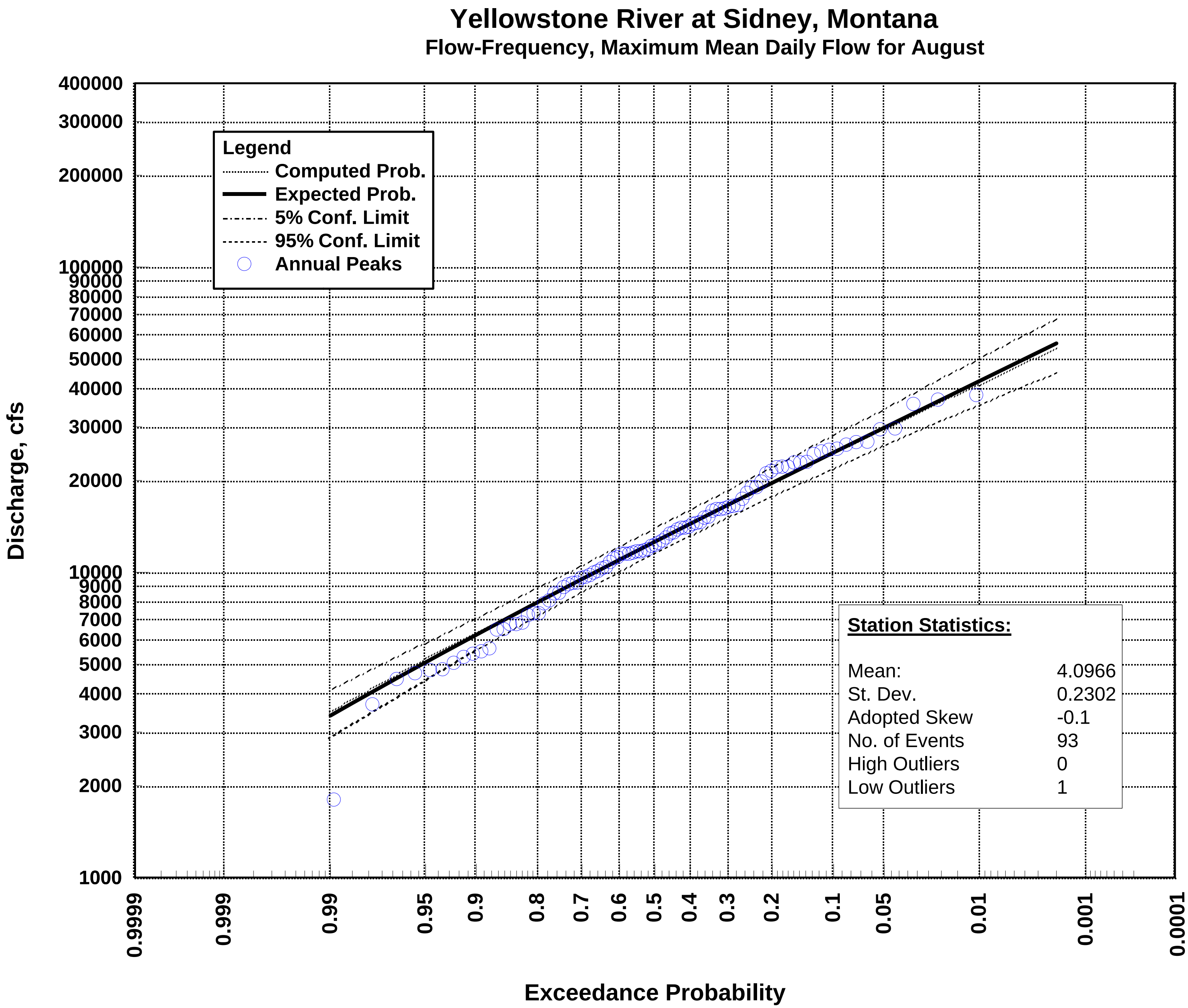


Figure B-29. Flow-Frequency for August

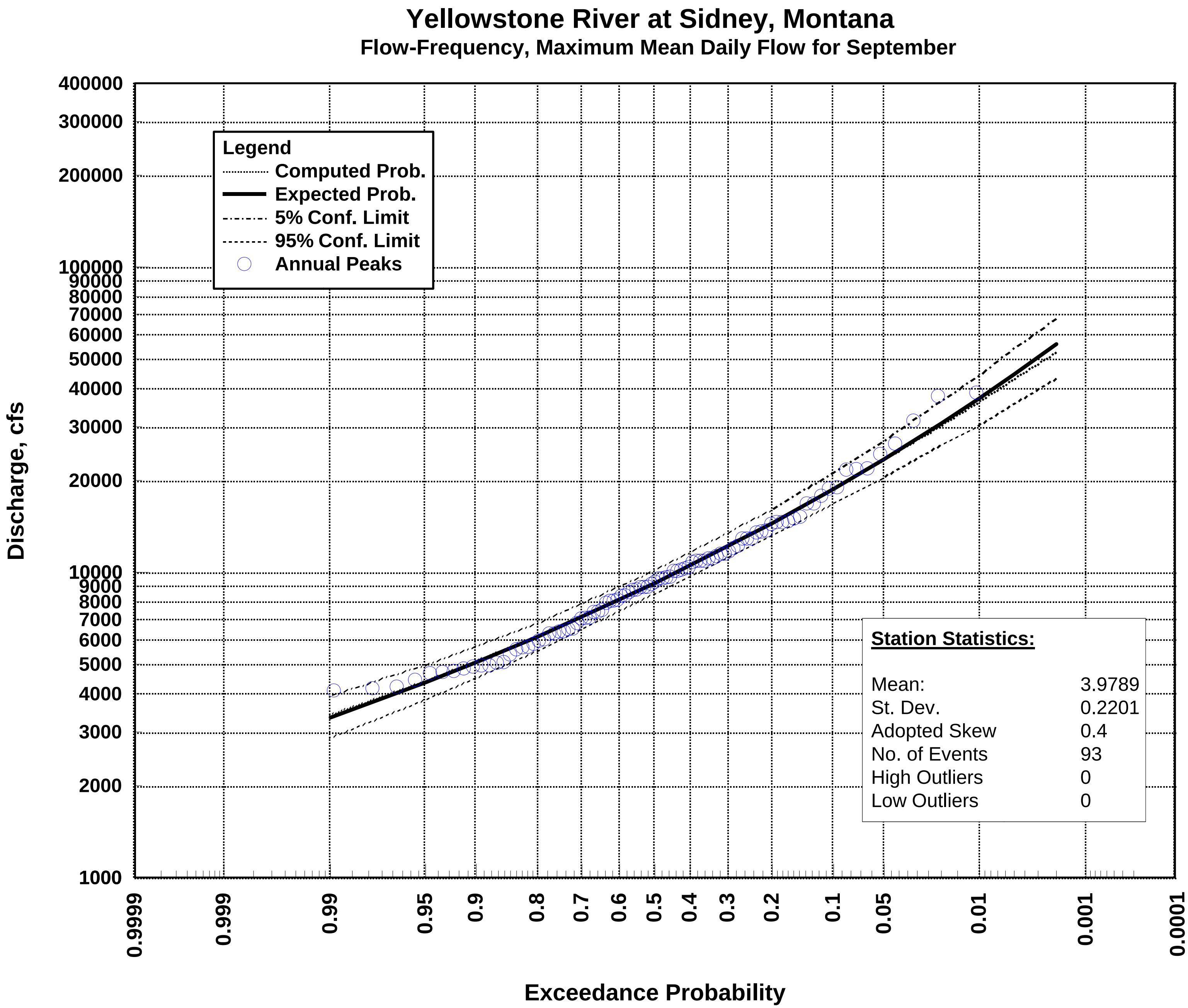


Figure B-30. Flow-Frequency for September

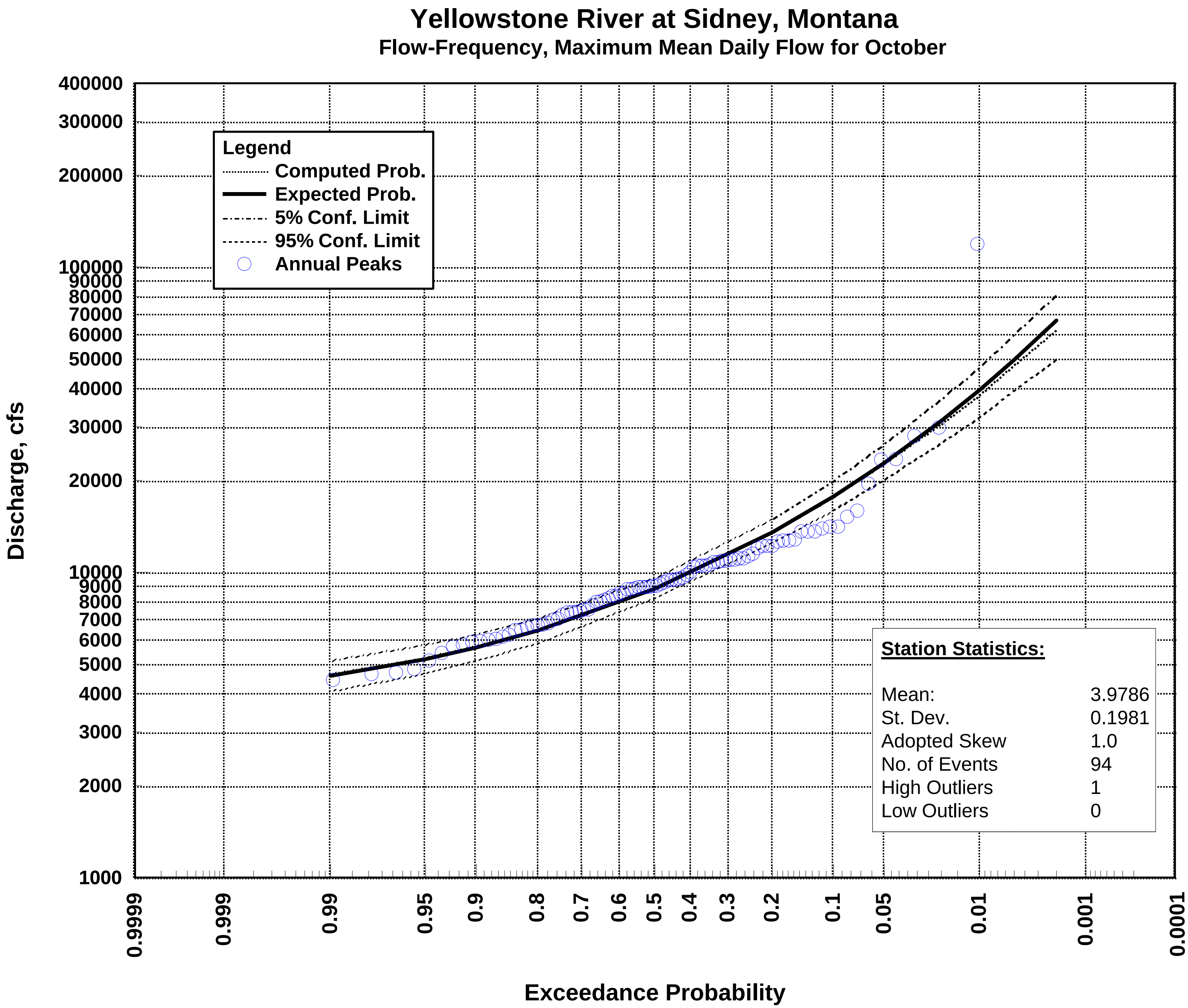


Figure B-31. Flow-Frequency for October

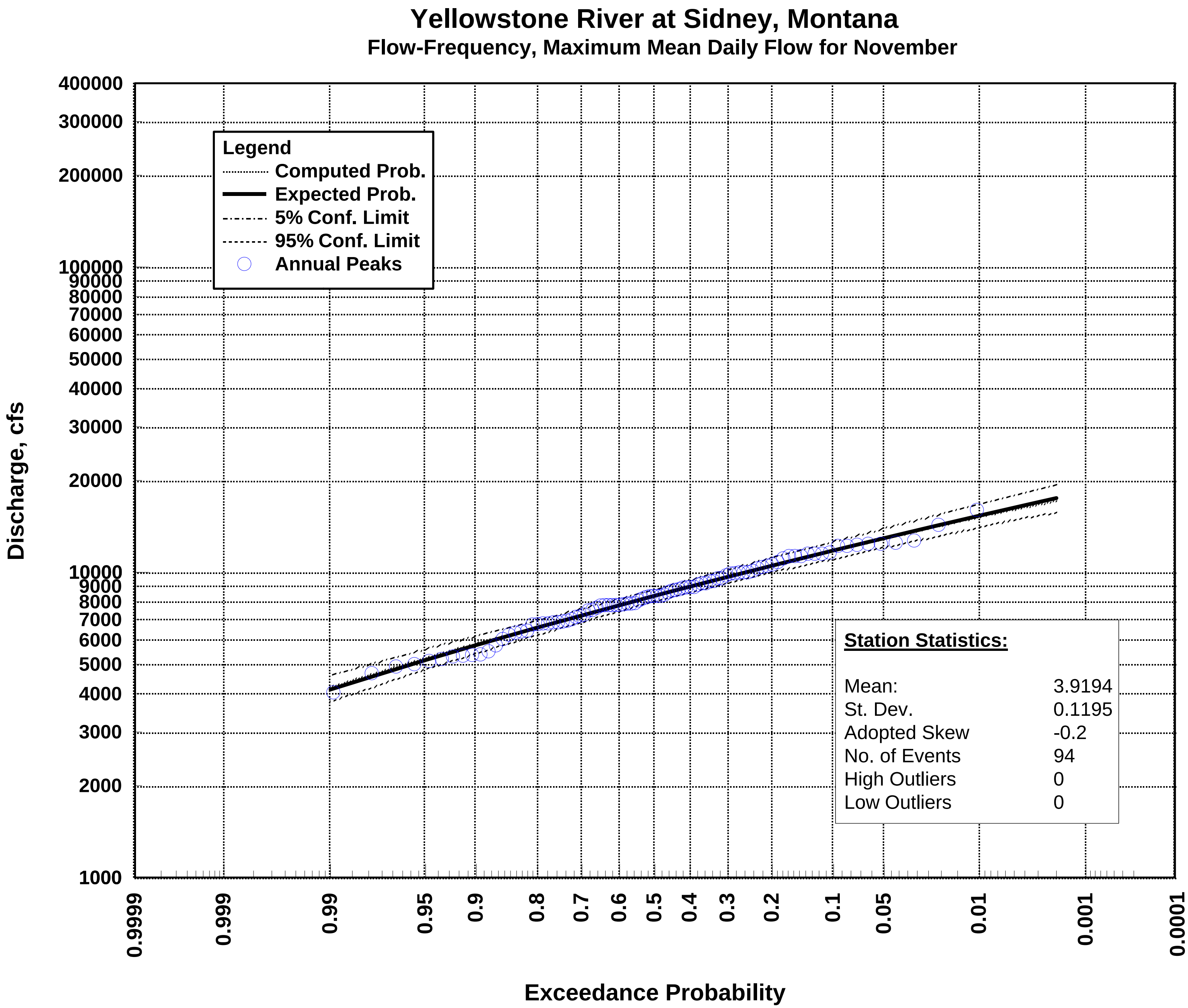


Figure B-32. Flow-Frequnecy for November

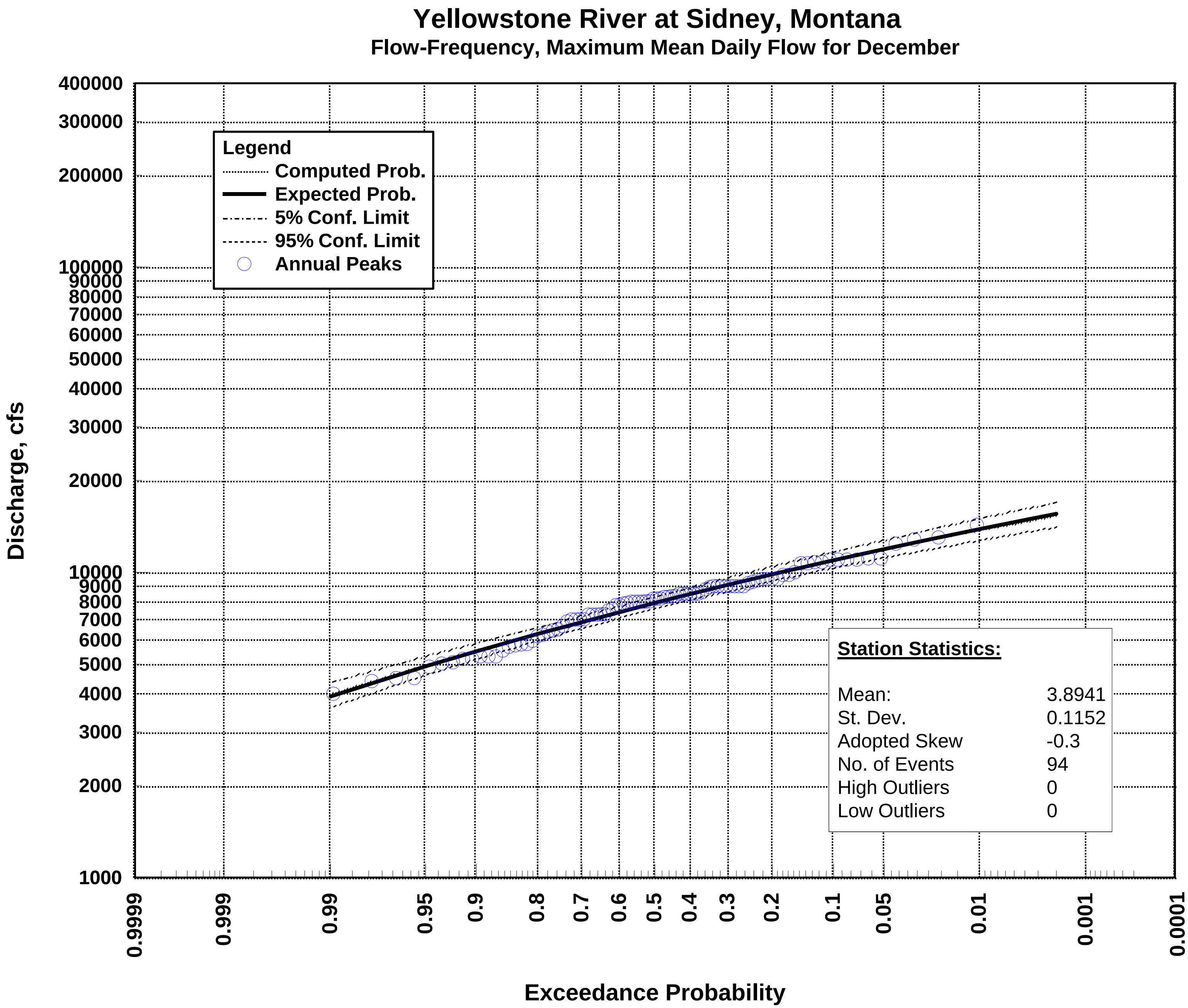


Figure B-33. Flow-Frequency for December

Appendix B-1

Tables of Annual and Monthly Flow Values

Table B-1.1. Monthly Diversions to Lower Yellowstone Canal near Intake, MT, acre-feet

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1909					1,400	11,600	17,500	12,400	9,800	3,300	0	0
1910	0	0	0	0	3,910	11,100	16,400	6,780	11,200	1,720	0	0
1911	0	0	0	0	6,950	12,700	23,900	5,660	2,260	1,570	0	0
1912	0	0	0	0	0	5,430	9,560	1,720	347	349	0	0
1913	0	0	0	0	0	12,900	8,660	4,920	2,610	980	0	0
1914	0	0	0	0	2,530	6,230	9,280	5,000	740	1,480	0	0
1915	0	0	0	192	8,890	9,930	13,700	5,720	1,090	742	0	0
1916	0	0	0	0	0	6,080	11,300	6,260	3,550	0	0	0
1917	0	0	0	0	2,700	13,400	21,700	16,500	5,920	0	0	0
1918	0	0	0	0	3,690	19,900	23,700	4,200	0	0	0	0
1919	0	0	0	0	0	18,500	26,000	15,700	9,850	0	0	0
1920	0	0	0	0	0	15,800	16,800	9,720	5,100	0	0	0
1921	0	0	0	0	1,950	19,700	19,500	16,000	7,860	0	0	0
1922	0	0	0	0	0	10,100	18,100	14,900	6,220	0	0	0
1923	0	0	0	0	1,890	20,800	30,000	23,700	13,000	0	0	0
1924	0	0	0	0	3,400	16,000	21,000	28,200	12,500	0	0	0
1925	0	0	0	0	17,600	10,100	24,600	31,700	21,200	2,090	0	0
1926	0	0	0	0	26,600	18,800	28,000	27,000	20,200	3,600	0	0
1927	0	0	0	0	0	7,430	24,000	22,300	10,000	0	0	0
1928	0	0	0	0	27,800	26,400	15,200	24,000	22,000	19,000	0	0
1929	0	0	0	0	16,000	13,500	40,000	35,900	22,500	12,200	8,000	0
1930	0	0	0	1,150	13,200	31,500	40,600	28,100	23,100	17,400	0	0
1931	0	0	0	12,100	38,000	40,100	39,700	42,600	36,600	18,500	0	0
1932	0	0	0	0	24,500	23,700	40,100	38,600	34,500	14,500	0	0
1933	0	0	0	0	13,800	31,300	45,300	45,000	31,600	19,790	0	0
1934	0	0	0	2,690	39,460	31,800	48,670	46,940	35,570	11,000	0	0
1935	0	0	0	4,460	12,120	22,720	43,700	46,340	33,560	18,160	0	0
1936	0	0	0	0	24,860	56,800	57,840	62,750	44,780	21,020	0	0
1937	0	0	0	8,720	59,690	47,470	61,430	63,500	55,210	8,720	0	0
1938	0	0	0	0	29,280	40,280	42,550	49,550	40,890	10,370	0	0
1939	0	0	0	5,110	53,250	15,770	57,710	50,670	51,480	6,650	0	0
1940	0	0	0	0	22,620	45,340	70,670	59,620	54,040	5,430	0	0
1941	0	0	0	0	31,350	35,550	63,690	68,540	28,950	0	0	0

Table B-1.1. Monthly Diversions to Lower Yellowstone Canal near Intake, MT, acre-feet

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1942	0	0	0	0	40,160	46,400	74,500	70,690	56,340	0	0	0
1943	0	0	0	0	37,270	49,150	63,660	66,130	62,180	15,220	0	0
1944	0	0	0	0	50,930	30,610	54,290	64,890	64,300	2,230	0	0
1945	0	0	0	0	60,050	61,860	70,290	73,950	68,980	12,430	0	0
1946	0	0	0	18,570	64,340	64,560	56,120	57,880	51,850	12,560	0	0
1947	0	0	0	0	52,860	50,470	68,300	67,120	54,490	16,120	0	0
1948	0	0	0	4,310	61,710	63,560	57,930	61,890	57,350	25,370	0	0
1949	0	0	0	4,130	60,370	62,400	65,780	67,600	66,680	30,750	0	0
1950	0	0	0	0	25,870	50,400	60,150	65,300	48,620	0	0	0
1951	0	0	0	0	43,160	52,190	64,980	62,950	49,080	17,510	0	0
1952	0	0	0	0	60,360	63,920	66,340	50,140	54,150	31,370	0	0
1953	0	0	0	17,370	44,580	29,600	56,030	62,400	55,170	42,970	0	0
1954	0	0	0	0	26,450	59,600	66,940	61,620	54,810	0	0	0
1955	0	0	0	0	21,310	55,690	56,520	51,220	47,280	10,180	0	0
1956	0	0	0	10,840	65,440	66,540	70,020	63,170	59,920	14,960	0	0
1957	0	0	0	0	46,410	54,660	68,630	64,310	63,500	17,860	0	0
1958	0	0	0	0	64,220	58,740	64,090	62,310	58,640	30,460	0	0
1959	0	0	0	0	57,580	61,120	63,140	60,410	55,040	16,580	800	800
1960	800	0	0	0	41,630	61,030	60,640	62,400	58,110	35,390	9,020	12,830
1961	8,600	0	0	18,730	52,320	61,340	59,580	56,640	66,100	30,310	12,900	11,590
1962	7,130	4,600	0	0	56,560	59,130	66,660	68,620	66,320	17,430	6,900	7,130
1963	7,130	4,600	0	0	16,040	62,440	68,010	59,720	63,370	22,620	7,200	7,440
1964	7,440	6,960	3,600	7,610	65,800	50,210	70,950	70,690	66,650	15,670	6,600	6,820
1965	6,820	2,200	0	0	19,440	52,960	64,690	72,600	64,530	3,630	4,940	5,120
1966	3,960	0	0	18,340	68,770	69,240	80,110	63,220	58,380	8,570	4,950	5,120
1967	3,960	3,560	3,560	0	35,000	64,960	71,560	72,030	65,400	3,960	3,810	3,960
1968	3,960	3,560	0	0	70,920	68,740	74,990	72,390	69,020	3,050	3,810	3,960
1969	3,960	3,560	381	0	45,050	73,490	74,090	79,430	74,230	18,290	3,810	3,960
1970	3,960	3,560	3,960	0	640	65,600	81,180	81,200	69,590	3,050	3,810	3,960
1971	3,960	2,300	0	0	37,800	70,050	80,470	82,700	59,610	3,050	3,810	3,960
1972	3,960	1,910	0	0	26,430	64,720	78,660	79,380	70,690	3,180	3,810	3,960
1973	3,960	2,410	0	0	64,080	72,010	79,510	77,350	59,290	0	0	0
1974	0	0	0	0	66,600	69,310	81,250	79,270	49,810	0	0	0

Table B-1.1. Monthly Diversions to Lower Yellowstone Canal near Intake, MT, acre-feet

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1975	0	0	0	0	7,010	65,940	75,450	79,840	62,390	0	0	0
1976	0	0	0	0	62,010	74,960	83,440	82,730	71,970	0	0	0
1977	0	0	0	3,790	13,980	9,840	14,700	13,000	663	0	0	0
1978	0	0	0	0	57,290	61,240	71,950	76,180	49,450	0	0	0
1979	0	0	0	0	4,680	18,580	20,840	16,120	12,610	0	0	0
1980	0	0	0	0	77,940	74,310	80,160	70,610	49,670	0	0	0
1981	0	0	0	3,910	10,220	9,260	14,720	13,170	10,830	0	0	0
1982	0	0	0	0	152	2,300	12,300	13,000	4,650	0	0	0
1983	0	0	0	0	52,200	61,000	60,800	63,600	49,900	0	0	0
1984	0	0	0	13,920	74,040	69,510	76,630	78,710	59,960	0	0	0
1985	0	0	0	0	57,400	55,500	56,500	56,600	45,900	0	0	0
1986	0	0	0	0	67,200	72,800	75,300	81,200	51,600	0	0	0
1987	0	0	0	0	57,020	69,800	79,430	70,530	61,530			

Table B-1.2. Mean Annual Discharge by Calendar Year

Year	Discharge, cfs	Year	Discharge, cfs	Year	Discharge, cfs
1898	16600[†]	1934	5630	1970	14220
1899	19320[†]	1935	9570	1971	17850
1900	13030[†]	1936	9200	1972	15750
1901	14570[†]	1937	9570	1973	13000
1902	13790[†]	1938	12590	1974	14980
1903	14630[†]	1939	8860	1975	18120
1904	15390[†]	1940	7590	1976	14610
1905	11790[†]	1941	11050	1977	7030
1906	13570[†]	1942	12750	1978	18410
1907	18030[†]	1943	18430	1979	11580
1908	16460[†]	1944	15760	1980	10820
1909	17300[††]	1945	13160	1981	10540
1910	12200[††]	1946	11400	1982	14550
1911	13340	1947	15200	1983	12850
1912	19240	1948	13710	1984	12850
1913	16120	1949	10610	1985	7850
1914	13710	1950	13430	1986	13920
1915	14400	1951	13190	1987	8050
1916	16250	1952	12740	1988	6970
1917	19170	1953	9510	1989	9100
1918	18430	1954	9370	1990	9320
1919	7090	1955	8800	1991	13050
1920	15260	1956	12270	1992	9720
1921	12160	1957	13970	1993	13430
1922	12100	1958	9940	1994	8640
1923	15990	1959	10710	1995	13750
1924	19040	1960	6890	1996	14870
1925	18020	1961	6630	1997	19750
1926	12920	1962	14590	1998	11810
1927	19090	1963	12990	1999	13250
1928	16670	1964	13480	2000	8060
1929	14310	1965	17870	2001	6050
1930	11880	1966	7460	2002	7080
1931	7540	1967	16160	2003	7560
1932	11780[††]	1968	15170		
1933	11100[††]	1969	12960		

[†] Glendive Flow

[††] Glendive Flow, with Lower Yellowstone Canal Flows Subtracted

Table B-1.3. Maximum Daily Discharge by Calendar Year

Year	Discharge, cfs	Year	Discharge, cfs	Year	Discharge, cfs
1898	68100[†]	1934	17100	1970	59800
1899	72000[†]	1935	75100	1971	62100
1900	51400[†]	1936	54000	1972	59300
1901	44700[†]	1937	62500	1973	46200
1902	42900[†]	1938	79800	1974	75700
1903	62300[†]	1939	51000	1975	76300
1904	77900[†]	1940	36500	1976	48600
1905	67800[†]	1941	39400	1977	27200
1906	79400[†]	1942	63800	1978	104000
1907	90600[†]	1943	85800	1979	47000
1908	86400[†]	1944	119000	1980	35000
1909	106800[††]	1945	65500	1981	55000
1910	47600[††]	1946	49100	1982	62200
1911	78400	1947	98000	1983	41300
1912	114000	1948	77800	1984	43000
1913	73800	1949	47800	1985	28700
1914	78400	1950	64100	1986	58800
1915	94200	1951	49500	1987	22100
1916	101000	1952	118000	1988	32200
1917	94600	1953	63600	1989	36700
1918	126000	1954	39800	1990	36600
1919	25900	1955	38100	1991	62200
1920	89700	1956	63500	1992	39300
1921	142000	1957	69000	1993	49600
1922	83200	1958	38300	1994	75000
1923	119000	1959	53000	1995	56600
1924	99800	1960	53000	1996	65000
1925	69100	1961	30400	1997	84900
1926	47900	1962	68200	1998	43600
1927	101000	1963	80100	1999	53500
1928	88000	1964	68000	2000	34100
1929	84800	1965	86500	2001	23900
1930	40800	1966	26600	2002	42600
1931	44900	1967	79700	2003	48400
1932	62700[††]	1968	70000	2004	24900
1933	63200[††]	1969	59600		

[†] Glendive Flow
[††] Glendive Flow, with Lower Yellowstone Canal Flows Subtracted (if needed)

Table B-1.4. Minimum Daily Discharge by Calendar Year

Year	Discharge, cfs	Year	Discharge, cfs	Year	Discharge, cfs
1898	4070[†]	1934	860	1970	2500
1899	4140[†]	1935	1130	1971	4700
1900	4350[†]	1936	950	1972	3200
1901	3900[†]	1937	870	1973	5560
1902	4550[†]	1938	2050	1974	3600
1903	4400[†]	1939	900	1975	3000
1904	4300[†]	1940	1220	1976	3700
1905	3500[†]	1941	2500	1977	1400
1906	3500[†]	1942	1500	1978	3900
1907	3100[†]	1943	2200	1979	4100
1908	2800[†]	1944	1230	1980	2300
1909	4000[††]	1945	1190	1981	2840
1910	2700[††]	1946	2000	1982	3300
1911	3500	1947	2000	1983	5400
1912	2950	1948	1700	1984	4000
1913	2900	1949	1600	1985	1800
1914	3120	1950	2600	1986	4000
1915	3090	1951	1500	1987	2500
1916	4040	1952	2000	1988	1390
1917	5000	1953	3200	1989	800
1918	5760	1954	1000	1990	1800
1919	2270	1955	1500	1991	3590
1920	5220	1956	3000	1992	4310
1921	1600	1957	2500	1993	2300
1922	1200	1958	2500	1994	2040
1923	1200	1959	1500	1995	2000
1924	2570	1960	2000	1996	3300
1925	4120	1961	570	1997	4000
1926	2360	1962	2600	1998	3300
1927	4350	1963	1000	1999	6120
1928	1930	1964	3000	2000	2880
1929	1600	1965	4000	2001	1010
1930	1760	1966	2360	2002	2000
1931	1200	1967	4200	2003	1720
1932	1060[††]	1968	5000	2004	1480
1933	1840[††]	1969	4000		

[†] Glendive Flow

[††] Glendive Flow, with Lower Yellowstone Canal Flows Subtracted (if needed)

Table B-1.5. Mean Monthly Discharges, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1910										5790	5510	4500
1911	3500	4000	9270	5900	13180	55830	27450	15460	9900	6680	4760	4000
1912	4000	4000	17000	22580	21070	54710	45390	20470	13430	13640	9450	4950
1913	5070	4560	9830	17460	22290	59180	28650	18740	8210	7500	6340	5370
1914	5000	4500	7990	9320	29420	53180	21400	9010	6370	7790	6500	3800
1915	3120	3120	5550	8580	16640	45500	34360	17820	12930	11420	6710	6470
1916	6000	6000	20000	9300	17420	46380	47320	15090	6790	7480	6880	5750
1917	5000	5000	7800	24200	26330	61910	55000	13380	10020	7600	6650	6580
1918	6500	7000	19000	11600	19360	77280	33660	12750	8790	9990	7730	7370
1919	4980	4800	10490	10560	13340	11580	3310	3530	3770	4930	7390	6320
1920	5910	5220	10650	9480	23920	53510	37470	10910	6810	6280	6560	6180
1921	5200	5940	8820	5750	16100	64560	17900	5830	4190	3730	3700	4500
1922	5370	5040	10300	9090	17830	49240	18240	9740	5750	4850	5270	4470
1923	5000	5000	8000	9560	17410	44610	30080	12810	8690	29130	12150	8710
1924	10600	14450	16550	39160	35040	47890	27270	7910	6390	9450	7940	6330
1925	13110	15350	18430	13360	24920	49230	34000	11870	8550	10210	9090	8070
1926	8450	9340	10170	11170	29120	26260	18580	8590	8640	8840	7390	8180
1927	6650	8830	11560	9020	28320	65210	39130	19450	13050	9660	9860	8000
1928	6000	6000	13500	8590	38100	40060	45060	12730	8120	7890	7560	5580
1929	4890	4340	21240	14870	22550	46470	22850	7020	7220	7810	6230	5800
1930	4570	11920	11820	10550	14940	26190	14800	14110	9200	10460	8110	6100
1931	4580	5370	5210	5910	12520	30460	4740	4800	2720	5240	3920	5260
1932												
1933										5210	5800	3370
1934	4470	4670	5660	7110	12690	12230	3680	2510	2390	4780	4200	3130
1935	2760	4430	5250	5460	9760	41560	24250	5610	3340	4210	4520	3710
1936	3250	2700	11080	8320	19430	31890	10170	5840	3480	5220	5510	3480
1937	2090	2850	6790	6800	13100	39810	22460	3730	3390	6120	4530	3090
1938	4370	3410	9330	6220	15700	49400	31060	7820	6340	6500	6520	4130
1939	4820	3000	12000	7120	17330	27100	11430	4630	3240	5390	5860	4050
1940	2190	3840	5440	6350	12930	26770	8530	2600	2760	9470	4920	5330
1941	3480	4320	5880	6910	16790	27830	9830	9760	16000	14460	9700	7550
1942	4540	5750	13400	11570	22560	42460	21140	6390	5390	7330	7660	4620
1943	4400	14250	19860	18360	18820	58160	46410	13360	8020	6530	7450	5680
1944	4310	4510	15330	12350	20640	67690	32880	7520	6210	7020	6950	3960
1945	5320	5490	11740	7220	12860	40480	34670	10580	8550	9070	7300	4240

Table B-1.5. Mean Monthly Discharges, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1946	6410	5410	10440	9210	14780	34720	17980	4680	8960	10090	8080	5970
1947	3750	5300	18370	12010	28430	41340	31820	10150	6630	9180	8380	6310
1948	4870	6030	12410	11050	19060	58260	24550	7500	4260	6560	7100	3170
1949	3250	4170	11670	10500	18970	35850	14850	3870	5260	7460	7300	4010
1950	3180	5510	10400	15260	12720	39020	30540	10980	8340	10170	7720	7060
1951	5210	6050	9000	12510	17710	34040	27000	15540	9900	8730	8300	3960
1952	5350	7000	10140	23530	25790	34850	15840	8050	5510	6220	5880	4970
1953	5720	5940	6620	5790	9260	36660	15560	6860	4060	5510	6740	5580
1954	3180	7130	5210	8020	17490	20050	20240	7670	5070	6020	6510	5750
1955	3930	4620	5590	12690	13900	26870	14200	4740	3020	5450	5040	5570
1956	6280	5990	12730	9940	20390	42760	15040	6960	6240	6500	8380	6230
1957	4300	4740	8310	7550	18570	54510	30900	7120	7560	8490	8790	6690
1958	6490	6010	8600	6820	17120	27950	12760	6660	5810	7370	7420	6180
1959	5130	4340	18770	7410	10390	34290	17750	4730	4690	7960	6340	6410
1960	4030	5910	13720	6950	7280	20460	5690	3440	2990	4580	4790	3020
1961	4310	4710	5190	2820	5410	21990	4970	1600	7560	8780	7840	4580
1962	4850	10980	11800	11960	20550	45800	27990	9930	8360	9050	7650	6340
1963	4640	10810	9150	6610	19830	54360	19810	4780	7260	6700	7380	5150
1964	6390	6120	6150	9170	18710	46430	33120	8920	7810	6810	7060	5170
1965	7160	7950	9320	20340	18540	52300	47580	14740	11010	11970	6460	6700
1966	5790	6000	9650	5800	10990	17570	8660	3870	3670	5840	5720	5860
1967	5350	6330	8050	7380	14740	56540	49710	10770	8280	9490	9060	7770
1968	9670	10220	14420	9130	12000	49370	24100	12570	12640	10590	10220	7490
1969	7440	8090	17950	13330	19040	27050	25410	7200	5770	8240	7970	7610
1970	7300	9740	9180	7200	23000	48010	26900	7260	7380	8560	8320	7850
1971	8520	17750	19000	14870	21660	50480	29380	9870	10060	15410	10750	6850
1972	6580	10430	25980	13290	19240	43300	19370	11110	9880	11690	11440	6830
1973	8420	8860	10340	10860	23670	32620	13910	6530	12230	10890	10420	7240
1974	6700	8040	9060	11910	15490	49710	32060	10740	8750	9470	10030	7850
1975	7490	5980	12210	12820	29100	45000	48640	16250	9720	9790	9800	9590
1976	9110	10780	10650	11570	27120	40580	22770	9400	7320	9450	9130	7500
1977	6040	7750	6100	7450	10530	17100	5360	3710	5300	6720	3970	4590
1978	5060	7210	21840	12940	34600	47590	37660	14240	12100	10240	8210	8350
1979	6010	7030	23090	14930	16050	24100	15160	7470	5860	5850	5830	7290
1980	5790	7620	9760	9570	16700	24900	14020	6790	8320	10520	8720	7210
1981	8120	7570	5840	4230	17370	38650	14780	5160	3750	6780	7380	6950
1982	5190	9740	11710	8810	13490	33440	36530	13470	10450	12180	10940	8350
1983	8870	10430	9660	8390	12230	30160	27110	9950	7950	11170	10680	7560

Table B-1.5. Mean Monthly Discharges, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1984	8540	8430	8080	8450	20240	32550	24070	9970	7670	9850	9760	6480
1985	6420	6500	10400	8940	10410	15970	5730	6480	5490	7470	4660	5710
1986	6550	6950	14260	10160	16890	42680	19370	8040	12930	10740	10150	8260
1987	6970	7140	8120	8270	13550	14450	8350	5910	6130	5960	6740	5020
1988	6160	6590	6980	5990	15560	19210	4810	1600	3360	4350	4750	4450
1989	4180	4180	9940	8660	18970	23660	11430	4910	4830	5740	6630	5780
1990	7500	6270	8020	9350	11650	24750	15320	5850	4810	6450	6580	5260
1991	6290	7590	6440	7680	22160	49240	19640	5160	8120	8180	8150	8080
1992	7020	6290	6080	7860	15690	21770	19420	6400	5660	6720	7250	6420
1993	5590	4410	9150	7610	21240	33780	31320	13480	7810	9780	8080	8100
1994	6470	5810	17070	9270	19270	16270	5950	2510	3310	6470	5520	5450
1995	5080	7000	6670	6070	20060	41160	33540	10550	8660	10110	8490	7210
1996	6430	11780	12590	14500	21680	50270	25300	7870	6390	8150	7280	6630
1997	7870	13520	14420	15360	28840	65270	32360	18190	12100	12070	9830	7320
1998	6810	8790	8440	10830	14470	23220	24750	10530	7650	10170	9050	6810
1999	7960	8410	8640	8630	19080	42430	21970	10150	8980	8610	7450	6740
2000	5990	6220	6350	6230	13000	24540	9920	3680	4320	6590	5540	4560
2001	4950	4550	6970	5740	9820	15010	7660	2210	2990	4090	4630	3970
2002	3820	4200	3240	5630	10060	25830	9840	4120	4350	5150	5040	3750
2003	3640	4440	9130	6300	11540	28200	8470	2650	3200	4260	4100	4820
2004	3930	4270	5510	5340	5920	13360	9260	2960	4420	6070	6010	5290
2005	4080	4700	4400	4830	17630	26070	14070	4140	3750			

Table B-1.6. Maximum Daily Discharge by Month, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1910										5790	5510	4500
1911	3500	4000	13600	6740	27500	78400	42200	24400	24400	7800	5760	4000
1912	4000	4000	114000	64400	39400	81600	112000	36800	16800	19500	12200	8940
1913	8360	6000	19000	30000	70600	73800	45000	35600	10200	9540	6740	5760
1914	5300	4700	10800	13600	62900	78400	35600	11500	12200	8940	7800	5300
1915	3120	3120	11500	12200	24400	94200	72200	38100	26400	23400	7800	7260
1916	6000	6000	94200	14400	24400	101000	83000	29600	8940	8360	8940	7800
1917	5000	5000	7800	85000	47900	94600	86400	25400	11500	8940	7800	7260
1918	6500	7000	75300	16000	25400	126000	61400	19000	12900	11500	8360	8360
1919	6740	4800	18600	18600	25900	23400	6240	6490	4440	6740	8360	7800
1920	6240	5220	15000	15200	47900	89700	66800	16400	8080	7260	7260	6180
1921	5200	5940	15200	6740	41500	142000	40800	8940	5080	4440	4040	6240
1922	5370	5040	32000	12200	43600	83200	32000	18200	7530	6240	6240	7260
1923	7260	5530	20000	24900	49400	57500	42900	22900	31400	119000	14300	10800
1924	16600	30600	32200	99800	61400	75300	47200	12200	8370	12200	9550	11100
1925	16600	23200	36000	20600	55200	69100	61400	15100	11500	10800	12200	14300
1926	14300	15800	13600	20600	47900	36000	31600	17400	13600	12200	9550	12800
1927	8370	18200	22300	13600	45000	91400	101000	26700	19000	13600	11500	10800
1928	7100	7000	26700	10800	86400	88000	81600	21500	10800	8950	7810	7270
1929	6000	5280	50800	33200	72200	84800	40800	9660	8400	9020	7780	12400
1930	11000	25500	16800	17600	18400	40800	21000	29400	12900	13600	8900	8300
1931	6860	6580	6040	7220	32000	44900	11800	11500	6540	5950	5380	7570
1932												
1933										6000	6800	5700
1934	5660	7080	12500	11600	17100	15200	8870	7340	4840	5700	4670	5020
1935	5540	5370	8480	7900	22400	75100	53000	10300	4670	4640	6430	5540
1936	5200	4060	20300	14700	35200	54000	17500	9780	4750	6470	9200	5300
1937	3010	3110	10300	9410	26000	62500	47700	9250	6510	9040	5200	5200
1938	5920	4540	19300	9200	35600	79800	71800	14500	17800	8800	8190	6380
1939	5830	5010	34300	9550	22700	51000	21100	6760	4170	6050	6340	5070
1940	3550	4300	8060	9980	22300	36500	19400	6840	5970	28000	6800	9200
1941	4950	4650	8600	11200	34300	39400	19400	16100	38800	23500	11300	10700
1942	7800	6900	24800	17200	55600	63800	31600	10800	8610	8520	9360	9000
1943	7000	76000	85800	39200	29400	83600	75900	24900	10400	7550	8440	7930
1944	6440	5130	57600	37900	52600	119000	80300	14600	13500	8210	7650	6440
1945	7060	7500	26000	9620	19500	65500	61500	15900	11100	9800	8750	8200

Table B-1.6. Maximum Daily Discharge by Month, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1946	11100	11200	18000	16400	24500	49100	31400	8090	18800	15900	9980	8500
1947	5500	7500	98000	19500	50300	65500	53800	14000	9150	10400	10000	10000
1948	7000	14000	21000	16900	50900	77800	39900	11600	7940	7380	7860	5820
1949	4700	5100	22500	17500	32500	47800	23400	6510	7050	9600	8590	7000
1950	4000	8000	20000	35000	22200	64100	46700	16500	11700	11100	9360	8500
1951	7250	9200	24000	30000	35200	49500	31200	21100	14600	10500	10500	9000
1952	7000	12000	118000	110000	38400	58800	24100	10100	6410	7400	6850	6500
1953	7900	7500	10000	7260	17200	63600	24300	11200	5680	9160	7010	6660
1954	6000	9000	8000	28800	36600	38900	39800	13400	9620	6810	7140	7000
1955	5500	6500	8500	24000	23700	38100	30100	9590	5700	5920	7500	11000
1956	10500	8400	27500	12900	55100	63500	25200	9230	8780	8030	9200	8500
1957	7500	5500	11600	14600	33700	69000	56700	11700	8960	9260	10300	8030
1958	8500	13500	12500	12000	38300	35800	27400	11800	6820	11100	8340	8000
1959	7000	5700	50000	8700	17000	53000	39400	7240	8130	8850	11500	9500
1960	6500	6800	53000	9580	14300	29800	11800	5280	4910	6150	5120	4400
1961	5000	5700	6600	3910	25900	30400	9010	4810	16800	10900	8800	6900
1962	6600	30000	28000	20500	41400	68200	52400	14100	9670	10600	8080	8020
1963	8600	28000	16000	7600	33400	80100	39700	8520	10900	8520	8370	8500
1964	7800	7300	8500	24000	33600	68000	59900	22100	15200	7460	7900	8500
1965	8200	16000	13500	75000	24800	86500	74800	22200	13700	14100	9900	8000
1966	8500	8300	23000	10600	23200	26600	14400	5510	5380	6600	6400	9000
1967	6300	7600	9400	8460	34900	79700	70400	19800	12900	10900	11300	11100
1968	13000	17000	23000	10000	18100	70000	46800	25200	14700	12600	11300	9310
1969	11600	9400	59600	21800	32600	47100	40000	11700	7110	9440	8920	8800
1970	13000	11800	11000	10100	53000	59800	57900	12700	9540	9410	8950	8600
1971	13500	50000	40000	26100	31200	62100	55800	16200	14400	29800	12300	10700
1972	9000	16800	52000	14800	31800	59300	28200	15200	11300	12700	12400	9500
1973	10500	10700	14000	18400	36700	46200	30600	8560	21900	12700	12700	8200
1974	13000	9700	12500	17900	32400	75700	57000	14000	9550	10500	16000	9000
1975	15000	8000	20000	17900	41900	61500	76300	26800	12000	11300	10600	13000
1976	15000	14500	14000	14200	39200	48600	35900	16600	9310	9990	9920	8000
1977	9800	8200	6740	10200	18700	27200	9380	4470	7090	8360	5330	8300
1978	6000	9000	64600	25000	104000	58800	52900	22900	21800	12000	10800	9400
1979	7100	8700	47000	17100	32200	32100	27700	11100	7430	6700	6760	9800
1980	7800	14000	12000	12200	33500	35000	23600	11500	11100	13600	9760	11000
1981	10500	12000	8000	5400	45700	55000	32400	9980	4950	7990	7820	7900
1982	7000	18000	17000	11000	29300	55900	62200	22200	14600	15200	12400	11000
1983	10000	14000	11500	8900	24600	41300	38900	16100	10100	12200	11600	9000

Table B-1.6. Maximum Daily Discharge by Month, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1984	10800	9400	9130	11600	33300	43000	41900	13800	10100	10500	10400	9000
1985	8000	9000	12000	10600	25600	28700	9330	11900	7410	8120	6940	8000
1986	8000	35000	45000	13900	27300	58800	28200	13000	37800	13900	11500	9500
1987	8000	8000	9560	9770	22100	20800	12900	9130	8060	6710	6860	7010
1988	9000	9000	8000	7320	29200	32200	9800	1800	5050	4690	5300	5200
1989	5800	5000	30000	18300	30800	36700	16200	6760	5830	6490	7520	11000
1990	9000	11000	12000	15700	16400	36600	31900	7910	5590	6990	6900	7500
1991	7500	8500	7400	14300	50000	62200	45700	7340	21700	9450	10000	9000
1992	7400	7100	6450	13800	22000	39300	29800	10400	6300	7410	7900	8500
1993	7500	6000	13000	8810	40200	39400	49600	26200	8750	10800	11100	9500
1994	8500	7600	75000	17400	33400	24800	11100	3690	4220	11000	6040	7000
1995	6200	10000	16500	7860	34500	56600	42300	19000	10900	11000	9110	9800
1996	8400	18000	30000	17500	35900	65000	42800	12500	8950	8800	8700	9000
1997	10000	24000	20000	19200	49000	84900	56700	23000	15000	14100	12500	8340
1998	8100	10200	11700	13000	20100	38600	43600	14400	10300	12800	10100	8180
1999	9800	9800	9500	9590	37600	53500	34200	13500	10900	8960	7920	7290
2000	6800	6500	6800	8970	27600	34100	16700	4780	6400	7040	8290	5800
2001	5200	4700	9000	6620	20100	23900	14100	12300	4710	4820	4910	4500
2002	4600	4800	4200	8000	25500	42600	21500	5630	4960	5440	5360	4900
2003	4300	4800	30000	8430	32400	48400	13900	5050	4100	5140	5000	5300
2004	5300	4800	6200	6810	9330	24900	14600	4670	6300	7550	7180	5960
2005	6200	5400	5000	7440	34500	42900	32900	5410	6030			

Table B-1.7. Minimum Daily Discharge by Month, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1910										5790	5510	4500
1911	3500	4000	6740	5080	6740	29600	18600	9850	6740	6000	4500	4000
1912	4000	4000	4000	10200	9540	27500	21400	11500	11500	9540	7800	2950
1913	3660	2900	3600	10200	9540	43600	14400	7800	6740	6240	5760	5300
1914	4700	4300	6000	6740	10800	34300	10200	5760	4860	6240	5760	3120
1915	3120	3120	3090	6240	10200	16800	18600	10800	8940	7260	5760	5760
1916	6000	6000	6000	7800	12200	18600	21400	7800	6240	6240	4040	4440
1917	5000	5000	7800	10800	13600	30800	27500	9540	8940	6740	6240	6240
1918	6500	7000	7000	9540	10200	21400	20400	7800	7800	8360	7260	5760
1919	3300	4800	5760	4860	5300	6240	2270	2270	2780	3660	6740	5760
1920	5760	5220	6000	7800	9850	25900	16800	6740	5760	5300	5760	6180
1921	5200	5940	5760	5300	4440	39400	8080	4440	3660	3300	2530	1600
1922	5370	5040	5000	6240	7260	28600	10800	5760	4860	4040	4860	1200
1923	1200	4800	5000	6740	6740	28600	17300	7530	5300	13900	10200	6750
1924	6250	9550	9550	17000	15800	34900	12800	5280	4810	7810	5760	2570
1925	9550	9550	6250	7810	10800	29600	15800	8370	7270	8950	7810	4120
1926	2360	6750	7270	6500	16600	15100	8370	5760	6000	8090	3900	3010
1927	4350	6250	8370	6750	15800	29600	19800	15100	10200	7270	7270	6000
1928	5200	5000	5000	7810	9550	28600	21500	8370	7270	7270	6750	1930
1929	3900	3680	7010	9660	10300	31000	10300	5080	5080	6330	3760	1600
1930	1760	5320	7430	7140	10800	17600	8300	5780	7140	7430	7140	3760
1931	3290	4750	3540	4000	3100	12800	1200	1670	1370	4830	1520	2760
1932												
1933										4700	5400	1810
1934	1930	1510	1670	5000	10100	8280	1440	1400	1390	4240	3370	860
1935	1130	3200	2500	3730	5020	20300	11100	3260	2600	3620	1740	1640
1936	1390	1970	4230	4060	11800	18300	4840	3080	2660	4420	3990	950
1937	870	2430	3100	4700	5000	20600	6510	1560	1750	5380	3290	1120
1938	2050	2320	4560	5000	6830	35600	15400	4740	4000	3950	2900	2210
1939	3100	1690	4580	5830	8300	19000	4040	2920	2520	4500	5070	900
1940	1380	2190	3790	4720	6910	16500	3790	1410	1220	5260	1700	2300
1941	2500	3500	3250	5010	7790	16900	5100	4010	10800	11400	6000	3630
1942	1700	4200	4520	7330	11300	27300	10300	3310	3310	6480	6000	1500
1943	2200	4500	5500	12900	14900	33800	24100	7090	6680	5770	6760	4000
1944	2400	3780	4300	5060	6280	37000	11300	3870	4230	6070	3440	1230
1945	1190	4000	4700	5770	6590	20800	17500	8380	5800	8480	5500	1400

Table B-1.7. Minimum Daily Discharge by Month, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1946	4000	3700	6850	6790	11200	22300	8530	2610	4240	7400	6500	2000
1947	2000	3800	4700	9480	10200	23400	15600	6100	4550	8600	6900	4000
1948	3800	3200	5100	7350	7100	38400	9650	3450	2940	6000	6000	1700
1949	1800	2800	5300	8640	10300	25600	6560	2510	2680	4970	6800	1600
1950	2600	3700	6000	8310	8220	20100	14000	6950	5030	8390	5000	3000
1951	3000	2100	4000	7410	10000	21100	18700	10100	7250	8000	7000	1500
1952	4000	5000	4900	10400	16600	18400	9340	5030	4730	4850	2000	2000
1953	3200	4500	4000	5050	5660	22400	7480	3630	3280	4000	6200	4500
1954	1000	5800	2300	5720	6950	15100	8920	4660	3100	5000	6050	3400
1955	3000	2970	2720	8000	7710	15000	7710	2610	2270	4900	1500	3500
1956	4500	4100	7000	9100	8540	25600	8360	5100	4620	5020	7310	3000
1957	2500	3200	5700	5190	5850	30100	12700	5140	5710	7800	7950	5500
1958	4500	3500	7000	5850	6500	22600	6370	4730	4630	5620	6500	2500
1959	3800	3400	4600	6580	7130	9770	7750	3470	3290	7520	1500	5000
1960	2000	4000	3000	5580	5160	6790	2830	2230	2120	3630	4400	2000
1961	3000	2600	3820	1510	570	10300	2180	720	1130	7680	5780	700
1962	2600	5300	3400	8220	9990	26900	12800	6170	6520	7880	7210	2800
1963	1200	4400	6900	6230	10900	33000	8620	3440	3820	6080	6260	1000
1964	4500	5000	4000	6920	10800	23500	11900	5160	5700	6270	6000	3000
1965	6200	5300	6000	8000	12600	18100	22800	8310	8100	10200	5300	4000
1966	3700	4200	4700	4980	4380	11400	5540	2520	2360	4820	4690	3600
1967	4200	5200	7000	5440	7660	31500	22600	5200	5680	8000	5600	5000
1968	6200	7800	9700	8160	7380	15400	12600	7610	11300	9410	9540	5000
1969	4000	6700	9200	9840	12500	19200	12400	5060	4950	6400	7140	6200
1970	2500	8400	7200	5510	9800	36400	12400	4060	4180	6580	7400	7000
1971	5600	8000	9600	12100	15100	37800	17000	5840	6950	8620	9970	4700
1972	4000	5300	13700	12100	12500	23700	12300	9250	9010	9700	9820	3200
1973	5800	6800	7990	7750	16000	24900	6760	5560	7390	7870	8400	5600
1974	3600	5200	8000	9600	12600	31700	14500	9690	8200	8680	8290	6500
1975	3000	4000	7800	7600	14300	23700	24200	12400	8920	8890	7600	6500
1976	3700	6000	6600	7570	14100	30800	11900	6310	5890	8850	6850	6000
1977	3200	6600	5800	5550	5510	7600	3350	3160	4400	5510	1400	1650
1978	4200	5400	8400	8500	12700	36400	24000	10600	8040	7700	3900	6200
1979	5000	5800	8300	12500	11100	16700	8940	5940	5160	5190	4800	4100
1980	2300	4500	7100	7840	10100	16700	7870	5080	5460	8750	7200	5000
1981	7000	4220	4920	2840	4670	27100	7100	2860	3200	5170	6600	3400
1982	3300	5000	8200	7800	9480	17700	22000	10400	7730	9240	9000	6500
1983	7000	7500	8690	7030	6750	23100	16900	7670	6650	9010	9600	5400

Table B-1.7. Minimum Daily Discharge by Month, Period of Record, Sidney, MT, Yellowstone River

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1984	6000	8000	7400	7280	10300	20800	13700	6910	6730	7660	9100	4000
1985	4500	4500	9110	7720	6250	9640	4410	3330	3840	6410	1800	3500
1986	5000	4000	8410	8010	11600	26600	13600	6280	6690	8330	8500	7500
1987	5500	6500	7000	6700	9210	7500	5310	4150	5400	5370	6620	2500
1988	3500	4500	6000	5330	5620	10800	1770	1390	1890	4100	4350	1500
1989	800	2800	3500	6160	10500	15100	6780	3840	4210	4870	5800	2500
1990	5400	3500	6240	6200	9140	15000	7750	4290	4400	5060	5500	1800
1991	5000	6500	5500	5380	7060	29500	7730	3590	3630	7170	5500	7300
1992	5800	5600	5490	5470	8140	11600	11200	4310	4480	6070	6530	4500
1993	2300	2500	4000	6710	7150	25800	20600	9140	6900	8090	3300	7400
1994	3000	3500	5600	7170	11200	8460	3700	2040	2170	4020	4000	3500
1995	2900	3000	2500	5320	6100	20900	20200	7260	6460	8110	7620	2000
1996	3500	4000	5400	11900	11900	36700	12600	4850	4900	7030	3300	4000
1997	4000	8000	12000	13100	16100	38000	22600	15100	10500	9910	7750	6340
1998	5000	7500	5160	10200	9150	15700	10500	7380	5960	8020	8110	3300
1999	6800	7500	8000	7070	8240	33800	12500	7170	7440	7690	6980	6120
2000	4850	5800	5860	5330	5720	17900	4880	2880	3160	5870	3210	3000
2001	4500	3800	4300	5000	5110	8820	5090	1010	1220	3450	3300	3000
2002	2300	2900	2700	4400	4890	14200	5740	2810	3390	4570	4600	2000
2003	2800	3400	2400	4990	7400	13600	4550	1720	1850	3850	2500	3800
2004	2100	2400	4800	3110	1890	6470	4760	1480	2890	5530	5540	4200
2005	1500	4200	4230	3920	4800	19200	5110	3420	3010			

Appendix C
Hydraulics

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

US Army Corps
of Engineers ®
Omaha District

Hydraulic Analysis Appendix C
Intake Diversion Dam, Yellowstone River
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1. INTRODUCTION

This appendix describes the performed hydraulic analysis to conduct the preliminary design. Analysis procedures, assumptions, and results are presented. Numerous past studies have been performed. This study provides additional modeling to further investigate two of the alternatives.

1.1 STUDY PURPOSE

The purpose of the hydraulic analysis was to develop preliminary hydraulic design information for two alternatives, (1) reconfiguring the existing Intake Dam into an engineered rock ramp or (2) relocating the intake diversion upstream to a location where gravity diversion would not require a dam. The rock ramp alternative would use the existing canal and intake structure and incorporate a sloping rock ramp on the downstream side of the existing diversion dam. Moving the diversion upstream would require installation of a new canal intake structure at an upstream location and removal of the existing intake diversion dam.

1.2 STUDY SCOPE

Analysis was performed at a conceptual level to examine alternative feasibility and refine cost estimates. The lack of updated Yellowstone River survey data in the vicinity of the Intake Diversion Dam and the relocated upstream diversion intake site limit the performed analysis. Future detailed design analysis is required to further define project features and thoroughly evaluate alternative feasibility.

1.3 PAST STUDIES

This study has a narrow hydraulic scope that relies on previous evaluations. Numerous past studies have been performed to evaluate many different alternatives for providing fish passage at the Intake Diversion Dam. A few of the recent studies with additional information include the *Intake Diversion Dam, Yellowstone River, Montana, Fish Protection and Passage Concept Study Report* (Bureau of Reclamation, 2000), the *Lower Yellowstone River Intake Dam Fish Passage Alternatives Analysis* (USACE, 2002), the *Intake Diversion Dam, Fish Protection and Passage Concept Study Report II* (Bureau of Reclamation, 2004), and the *Draft Biological Assessment: Future Operation of the Lower Yellowstone Project with Proposed Conservation Measures* (Bureau of Reclamation, 2005).

2. INTAKE DAM EXISTING CONDITIONS

Evaluation and analysis was performed to review and update existing conditions. A plan view of the Intake Diversion Dam and Yellowstone River vicinity is shown in Plate 1.

2.1 INTAKE DAM

Intake Dam was originally constructed as a rock-filled timber crib weir with a height of 12 feet. The dam spans across the Yellowstone River channel for a width of 700 feet. The dam extends about 135 feet longitudinally along the channel and consists of a 1 vertical on 2 horizontal (1:2) upstream slope, a 15-foot wide crest, and a varying degree downstream slope. Since the construction of the dam, the structure has required frequent repair to maintain the upstream Yellowstone River water surface elevation required to for irrigation flow diversion. In the current condition, the dam crest elevation varies as ice and flood flows progressively displace riprap material from the crest. Updated survey data of the dam crest and vicinity was not available. Previous survey data indicated a range of 2 feet across the crest from elevation 1987 to 1989 feet. Current practice is to maintain the rock crest a minimum of one foot above the wooden structure to provide enough head for the maximum diversion rate of 1,400 cfs.

2.2 DAM MAINTENANCE

Significant repair has occurred several times following major flood and/or ice events. Over the years, large quantities of rock have been added to the dam to replace rock displaced by the river. Major structure repair has also occurred with the last occurrence in the 1970's. A cable way that crosses the Yellowstone River along the crest of the dam is used to replace shifted rock and maintain the crest elevation. Rock extends downstream of the dam in a scattered rock rubble field of varying length that is over 300 feet on the left bank (north, intake structure side) to about 150 feet on the right bank. On an as-needed basis, 300 to 1200 cubic yards of large quarried rock is placed to maintain the dam crest (Bureau of Reclamation, 2005, pg. 7). The maintenance is usually annually, with the degree required variable with conditions. Drought and mild winters reduce crest damage caused by flood flows and ice damage. Using the cableway, the largest rock that can be placed is about 1 cu yd or about a 3' by 3' boulder. Typical practice is to take rock with a quarry run gradation, place the large rocks across the crest, and then use smaller rocks to fill in around the large rock. Rock is often taken from a nearby quarry with quality that varies from durable to fractured. A photo from the site that was taken during a low period in the early 2000's illustrates the rock crest and downstream rubble field in Figure 1. Figure 2 illustrates the replacement of dam crest rock.

Figure 1 – Intake Diversion Dam at Low Flow

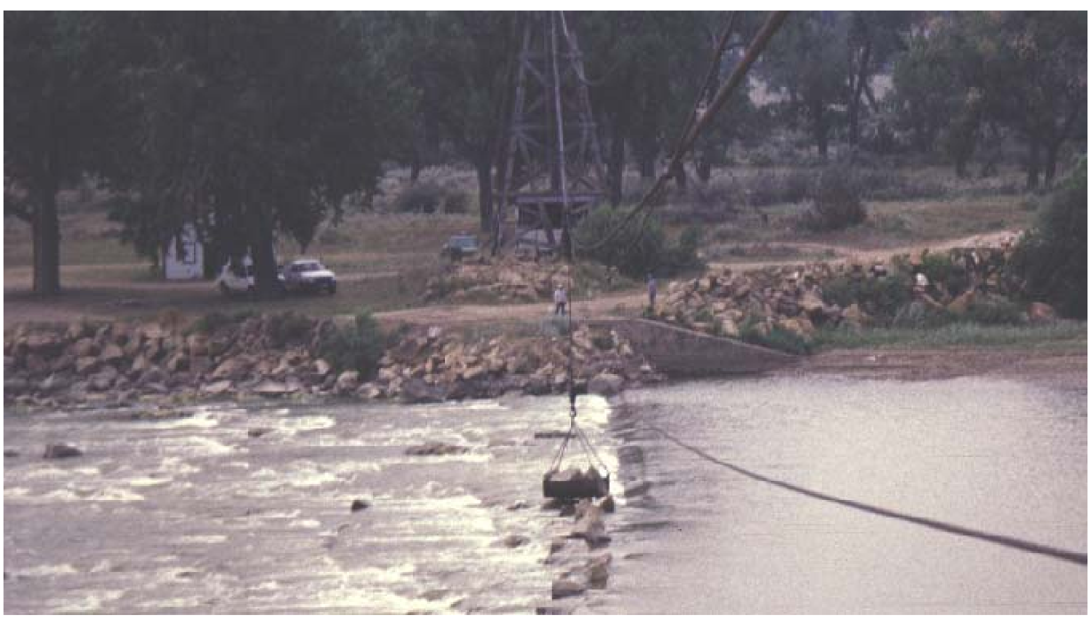


Figure 2 – Intake Diversion Dam Replacing Crest Rock

2.3 YELLOWSTONE RIVER HYDROLOGY

Yellowstone River flow values were evaluated during this study and are reported in the Hydrology Appendix B. Flow frequency and flow duration analysis considered both the Sydney and Glendive gage record and examined the impact of Yellowtail Dam on results. Refer to the Hydrology Appendix B for a complete discussion of analysis methods and results. Significant values used in this analysis are as follows:

Table 1 – Yellowstone River Hydrologic Design Data

Instantaneous Annual Peak Flow[1]	100-Year	160,200	
	10-Year	104,900	
Flow Duration (Percent Time Flow is Equaled or Exceeded)[2]	July	August	September
5000 cfs	>90	<70	<70
4000 cfs	>98	>80	<85

1 Hydrology Appendix B, Table 4.
2 Hydrology Appendix B, Table 3.

2.4 EXISTING CONDITIONS HEC-RAS MODEL

An HEC-RAS model was available from the previous study entitled *Intake Diversion Dam, Yellowstone River, Montana, Fish Protection and Passage Concept Study Report* (Bureau of Reclamation, 2000). A new model was constructed using the available Lidar survey data in order to add additional sections to refine the model in the ramp vicinity, model the right bank chute flow, and estimate Yellowstone River flow elevations at the upstream diversion location. The newly constructed HEC-RAS model was calibrated to match results from the previous modeling effort (Bureau of Reclamation, 2000) as no additional calibration data was available.

Survey Data

Lidar topographic data of the site was also available that was previously collected for the Yellowstone River Corridor Study. The constructed HEC-RAS model used the latest topography for the channel banks and floodplain. However, the Lidar topography didn't include any below water elevations with the minimum elevation near the waters edge elevation. Yellowstone River bed topography is not available. At the time of the Lidar survey in September 2004, the Yellowstone River flow was about 3,000 to 4,000 cfs. The channel improvement option was used in HEC-RAS to add flow area that was roughly dimensioned as a trapezoidal section about 300 feet wide and 2 – 3 feet below the minimum survey elevation. The Lidar survey data used in the HEC-RAS model is in the following coordinate system:

Horizontal: Montana State Plane NAD 83
Vertical: NAVD 1988

HEC-RAS Model Version
The Hydrologic Engineering Center (HEC) in Davis, California, developed HEC-RAS to calculate water surface profiles for uniform, steady-state flow using the standard step method and for unsteady one-dimensional flow simulation. Microcomputer version 3.1.3, released in May 2005, was used for this study.

Model Stationing
An arbitrary model stationing was established when constructing the new HEC-RAS model. Model extents begin at station 0 approximately 28,000 feet downstream of the existing Intake Diversion Dam.

Model Roughness
The HEC-RAS model uses a Manning roughness value of 0.035 for channel regions and 0.050 for overbank regions. The roughness parameters established for the model were similar to the previous modeling effort.

Intake Dam Crest
The Intake Diversion Dam crest was modeled within HEC-RAS using the inline weir option. Sufficient Yellowstone River stage-flow data is not available to calibrate the weir parameters. Modeling parameters are as follows within the HEC-RAS model:

Table 2
Intake Dam Crest HEC-RAS Data

HEC-RAS Parameters	Weir Crest (Station, Elevation)	
Yellowstone River Crest Station 280+22	0	1987
Discharge Coefficient – 2.7	30	1987
Width – 15 feet	130	1988
	430	1989
	700	1989

Model Results
The new existing condition model was compared to the previous model results with reasonable agreement for similar flow (Plate C-2). Comparison illustrates that the model results are reasonably similar. Comparison shows that the new model has a slightly lower water surface at the higher flow rates. Some of this difference is attributable to the inclusion of the right bank chute and also expanding the Yellowstone River section geometry to full floodplain width.

2.5 RIGHT BANK CHUTE

The new HEC-RAS model includes the overbank flow area and the right bank chute. The right bank chute allows flow to bypass Intake Dam and access the southern floodplain. The chute exits the Yellowstone River about 9,500 ft upstream of the dam near station 375+00. The chute re-enters the Yellowstone River about 8,500 feet downstream of the dam near station 195+00. Total chute length is about 24,500 feet. Flow area was not added to the chute as the channel was not flowing at the time of the Lidar survey. The chute channel section has a 100 – 200 foot bottom width. At the time of the site visit (23-24 May 2006), the Yellowstone River at Glendive USGS gage flow varied from 26,600 to 29,600. Chute flow seemed to initiate at about that level. During the time of the site visit, estimated chute flow was about 300 - 400 cfs.

The initial model included an upstream chute invert elevation of 1995.0. Initial HEC-RAS computations determined a chute flow of about 960 cfs with a Yellowstone River total flow of 28,000 cfs. Based on the site observation of the flow split, the invert elevation of the right bank chute cross section located just downstream at the Yellowstone River junction was raised by two feet to an elevation of 1997. As a result, the HEC-RAS model computed right bank chute was reduced to just over 400 cfs. Given the accuracy of the Lidar data set and lack of below water survey information, the adjustment seemed reasonable. The HEC-RAS estimated flow split is shown in Table 3.

Table 3
Yellowstone River vs. Right Bank Chute Flow Split

Total Flow (cfs)	Yellowstone River (cfs)	Chute Flow (cfs)	Chute % of Total Flow
12,000	12,000	0	0.0%
20,000	19,952	48	0.2%
28,000	27,588	412	1.5%
40,000	38,278	1,722	4.5%
60,000	55,413	4,587	8.3%
80,000	73,083	6,917	9.5%
100,000	91,264	8,736	9.6%
120,000	109,311	10,689	9.8%
140,000	127,313	12,687	10.0%
160,000	145,156	14,844	10.2%

Note: Right bank chute flow at low flows is only an approximation since the survey did not include the Yellowstone River invert. The tabulated Yellowstone River flow includes overbank flow above the channel capacity. Within the RAS model, this varies from about 5 to 15 percent of the total flow above 80,000 cfs.

2.6 RIGHT BANK FLOODPLAIN

The new HEC-RAS model includes full width cross sections that span the right bank. At the Intake Diversion Dam location, the left bank is very high and does not allow any overbank flow. For extreme events, the right bank chute begins to flow at around 25,000 to 30,000 cfs. The remainder of the right bank floodplain is slightly higher but still provides floodplain relief. The aerial photo of the floodplain illustrates several old channel alignment scars. The minimum elevation at which floodplain flow initiates is about elevation 2000 to 2002 ft NGVD upstream of the dam. At the 100-year event, computed flow in the floodplain is about 10,000 cfs with another 15,000 cfs in the right bank chute. It may be possible to excavate a portion of the right bank floodplain to enhance flow bypass of the Intake Diversion Dam.

2.7 INTAKE DAM WATER SURFACE IMPACT

The new HEC-RAS model was used to compute rating curves upstream and downstream of the dam. The rating curves illustrate that the dam still impacts flow elevations for events greater than the 10-year peak flow rate of 104,900 cfs. Results are plotted in figure 3.

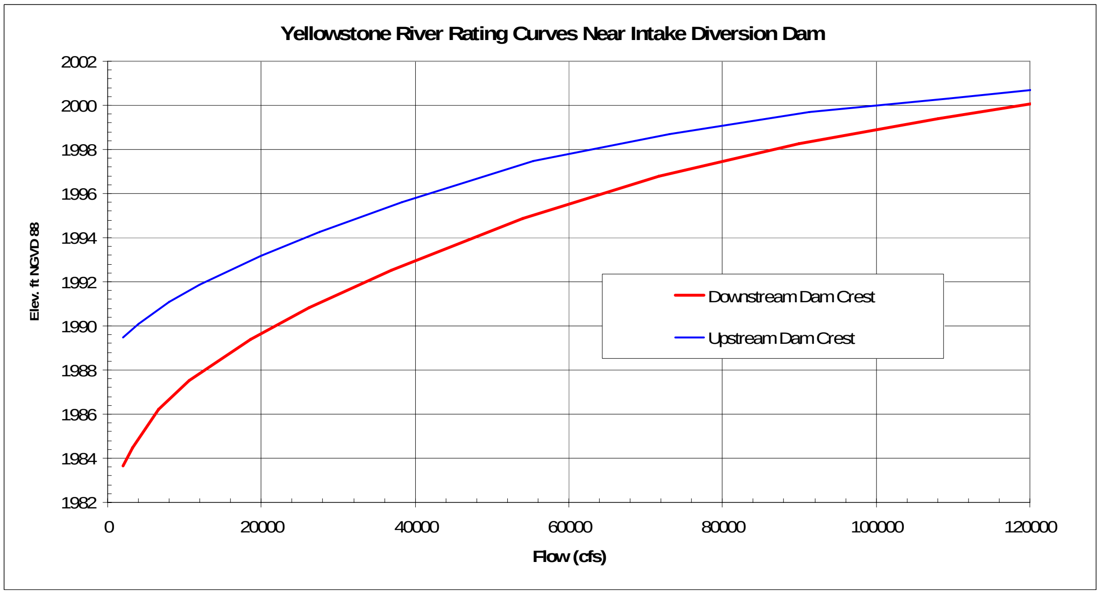


Figure 3. Intake Diversion Dam Rating Curves

3. ROCK RAMP

The rock ramp alternative consists of constructing a rock ramp downstream of the existing Intake Diversion Dam structure. Constructing the rock ramp will maintain the existing Yellowstone River stage-flow relationship such that diversion with the same canal intake is feasible. The ramp is constructed by adding material on the downstream side of the existing structure. Highlights of the ramp project and analysis are as follows:

- Install concrete cap on existing dam and maintain existing intake for diversion.
- Construct sloping rock ramp downstream of the dam crest.
- Design ramp to be suitable for fish passage with diverse flow depths and velocities. Evaluate both 2% and 5% ramp slope. Review boulder spacing and configuration.
- Review ice impacts to the ramp stability.
- Ramp design is conceptual with sufficient detail to evaluate feasibility and to prepare cost estimates.
- Review available ramp design criteria and site specifics to develop guidance for refined design.
- Numerous examples of rock ramps are available. However, an installation on a duplicate river to the Yellowstone River with similar flow, unit discharge, drop height, sediment transport, substrate, section, slope, and other physical parameters was not located.

3.1 RAMP LAYOUT.

A series of slopes and drop heights were tried with the ramp in an attempt to minimize peak flow velocity and the corresponding rock size. Slopes of 5%, 3.33%, and 2% were all evaluated. Drop heights of 0.5 ft and 1 ft were also checked. Installed ramps on the Red River of the North and guidance developed by Luther Aadland of the Minnesota DNR (Buesing, 2006 and Breining, 2003) used a 1 foot drop. Compared to the Intake Diversion Dam application, the Red River ramps are a similar drop height and slightly lower unit discharge. Design and analysis results are summarized as follows:

Top of Ramp – Current elevation varies from 1987 to 1989, assume new dam crest is at elevation 1989. Placing the dam crest at 1989 will provide sufficient head for the existing intake structure.
NOTE: To facilitate fish passage and maintain flow distribution, an uneven crest with possibly natural rock set in the crest concrete is probably required. These details will be determined in final design.

Toe of Ramp – Elevation 1980 (based on the old channel surveys of limited detail in the near dam vicinity). A tie-in slope of 3H on 1V or similar for rock ramp stability should be used to reach the bottom of the scour hole located downstream of the dam. According to the old survey data, the elevation 1980 isn't reached until about 400 ft from the dam.

Approximate Ramp Center Bottom Width – 550 feet

Ramp Shape - Ramp is "U" shaped, although unbalanced to maintain the main flow channel along the irrigation intake bank. The ramp shape should be optimized to provide the maximum depth-velocity diversity in detailed design. Due to the width of the river, it is anticipated that a significant portion of the center ramp will be relatively flat. A conceptual ramp layout is illustrated in Plate C-3. Ramp details are shown in Table 4. A typical ramp profile is shown in Figure 4.

Table 4.
Ramp Layout for Various Slopes

Alternative	Ramp Length (ft)	Length Between Steps (ft)	Number of Boulder Rows
5% Slope, 1 ft drop	180	20	9
5% Slope, 0.5 ft drop	180	10	18
3.33% Slope, 1 ft drop	270	30	9
2% Slope, 1 ft drop	450	20	9
2% Slope, 0.5 ft drop	450	10	18

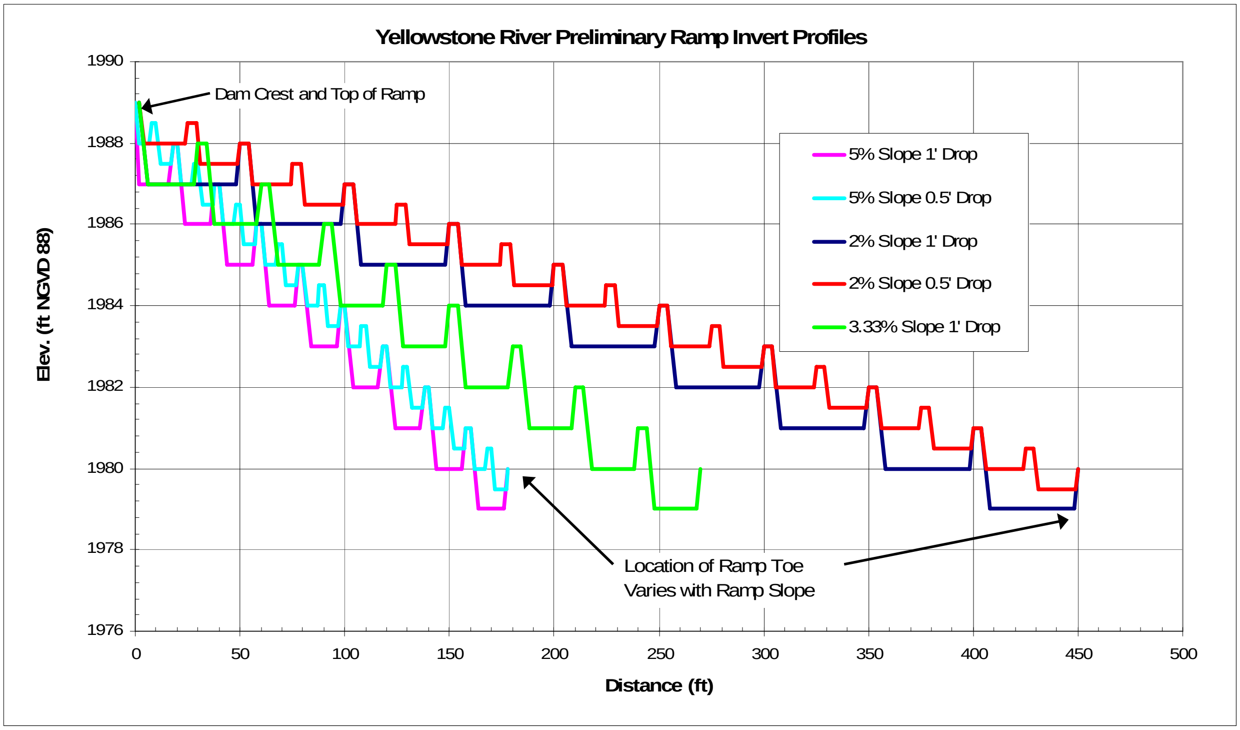


Figure 4. Yellowstone River Preliminary Ramp Invert Profiles.

3.2 RAMP STEP AND BOULDER LAYOUT GUIDANCE.

A 4’ minimum diameter boulder is placed to form each “step” in the above profile. The boulders are not solid but would block the bulk of the flow. The boulders would be offset a little from each other to allow fish passage between boulders and give staggered resting pools. The boulder would be 0.5’ to 1’ above grade on the upstream side. The boulder crown would be 1’ to 2’ above the grade of the downstream pool. A conceptual layout of the ramp boulders is shown in Figure 5.

- The ramp is formed by constructing a series of steps. Large boulders form perpendicular vanes that are used to anchor the steps, smaller rock is used to form the base of the steps.
- Fish passage is achieved passing through the large boulders vanes.
- Gaps in the boulder are staggered and variable to achieve velocity diversity for a range of flows.
- The large boulders should protrude about one foot above the ramp slope where the boulder vanes are perpendicular to the channel centerline.
- The large boulders should protrude two feet above the slope at the channel edges and transition between the two.
- Along a vane, the boulders at the channel edge should be two feet higher in actual elevation than the boulders perpendicular to the channel centerline. This will require that the base rockfill also have a limited transverse slope.
- Ramp boulder anchoring must be sufficient to resist ice forces and 100-year event flow forces.

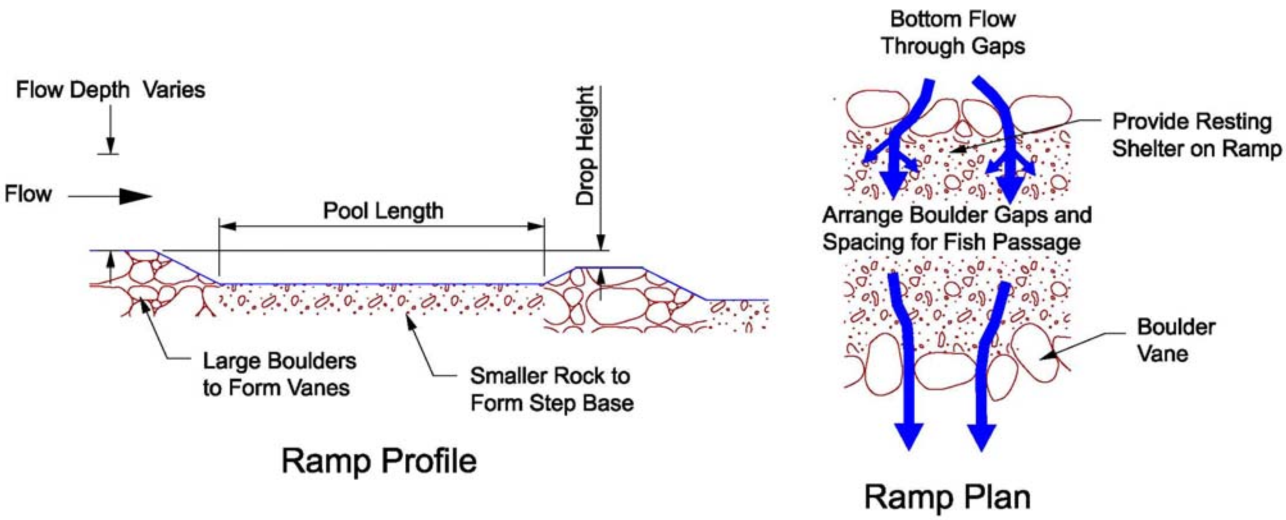


Figure 5. Conceptual Ramp View.

3.3 HEC-RAS ROCK RAMP MODEL

The existing condition HEC-RAS model was used to add a rock ramp and compute flow velocity. Since HEC-RAS is a one dimensional model, accurately evaluating the flow turbulence and velocity variation in both the horizontal and vertical directions is not possible. However, the HEC-RAS model can be used to produce reasonable estimates of average velocity and depth on the ramp and is suitable for use with comparing ramp conditions for various geometries.

Model Roughness

The rock ramp is expected to have higher roughness values compared to the existing channel due to the rock size and turbulence within the ramp flow. However, overestimating the roughness will cause the model to underestimate the flow velocity. Consequently, ramp stability would be overestimated. Guidance available relates rock size to roughness using the Strickler method (USACE, 1994, eq. 5-2).

Computations determined a roughness value of 0.036 for 24 inch D_{100} and 0.042 for 48 inch D_{100} size rock. Since lower roughness values will result in the maximum velocity, a conservatively low roughness value of 0.036 was used for the entire ramp.

Model Geometry

Grading plans were not available for the proposed ramp configuration. Therefore, the channel modification option was used within HEC-RAS to generate different slope alternatives. The channel improvement option is limited to simple channels, so the complex shape of the ramp could not be completely modeled. For the conceptual analysis, a center channel section of 560 feet, compared to an existing dam width of 700 feet, was assumed. This bottom width was selected as reasonable for the existing site to reflect flow area and concentration on the ramp.

3.4 HEC-RAS MODEL RESULTS

Computation results from the HEC-RAS model were used to evaluate the maximum rock size required for stability. Interpretation of computed results is summarized as follows:

1) Results showed only a small change between the different alternatives when comparing velocity at similar ramp elevation location. Modeling the ramp with HEC-RAS may be of limited accuracy for absolute values but relative comparison between locations should be useful. An HEC-RAS output plot of computed water surface elevation for the 5% slope with 1 ft drop is shown in Figure 6. Computed velocity range is shown in Figure 7 for the 5% slope with 1 ft drop.

2) Results did show that the 1 foot step drop appears to be a little superior to the 0.5 ft drop. The smaller drop has similar velocities at the step compared to the 1 foot drop. However, the smaller drop has short resting areas with twice as many turbulent zones over the ramp length.

3) Computations determined that ramp velocity peaks for flow rates of 80,000 to 100,000 cfs. For larger flow events, tailwater conditions reduce computed flow velocity. Computations determined that critical depth occurs at the ramp crest for all flows below 80,000 cfs.

4) Reducing the ramp slope from 5% to 2% had a marginal effect on average flow velocity with a decrease of less than 1 ft/sec. From a fish passage aspect, the flatter slope serves to lengthen the high velocity and turbulent zone and may not be preferable. However, the flatter slope may indicate a wider range vertical velocity distribution that corresponds to a lower near bottom velocity within the ramp.

5) Computed rock size decreases in the direction of flow down the ramp. It is necessary to provide a concrete cap on the existing structure for upper ramp stability, this will also help with ice forces.

6) The maximum velocity is located at the crest of each boulder row. The minimum velocity occurs within the pool section located between the boulder steps.

7) Flow velocity difference between the two slopes is lower than expected. The ramp slope reduction from 5% to 2% would be expected to cause some decrease in ramp velocity and turbulence. Differences at the higher flows would probably be much greater but the impact of the floodplain and chute flow offsets the slope change. At the lower flows, although the ramp invert slope is changing, the energy grade slope is very similar between the different ramp slopes. Figure 8 compares the ramp velocity profile at 100,000 cfs for various ramp geometries. The plot illustrates the difference between the 1 ft and the 0.5 ft drop heights. Figure 9 compares the relative velocity difference on the ramp for the 5% slope and 2% slope at different flows using the 1’ drop height. Both figures must be interpreted with caution.

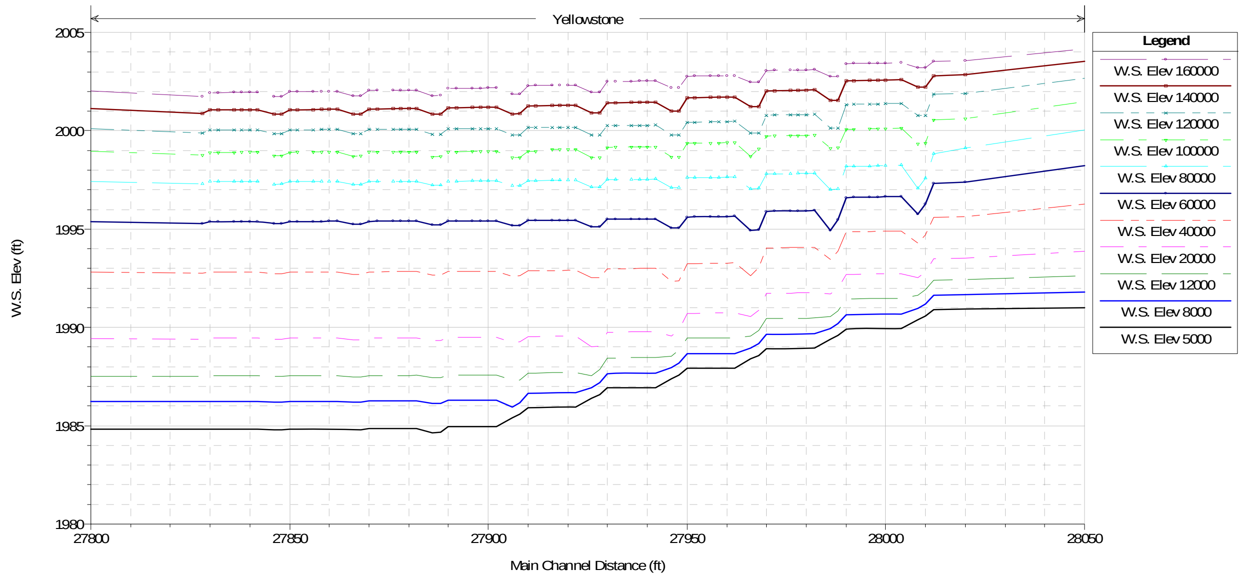


Figure 6. Computed Water Surface Elevation – 5% Slope Ramp, 1’ Drop

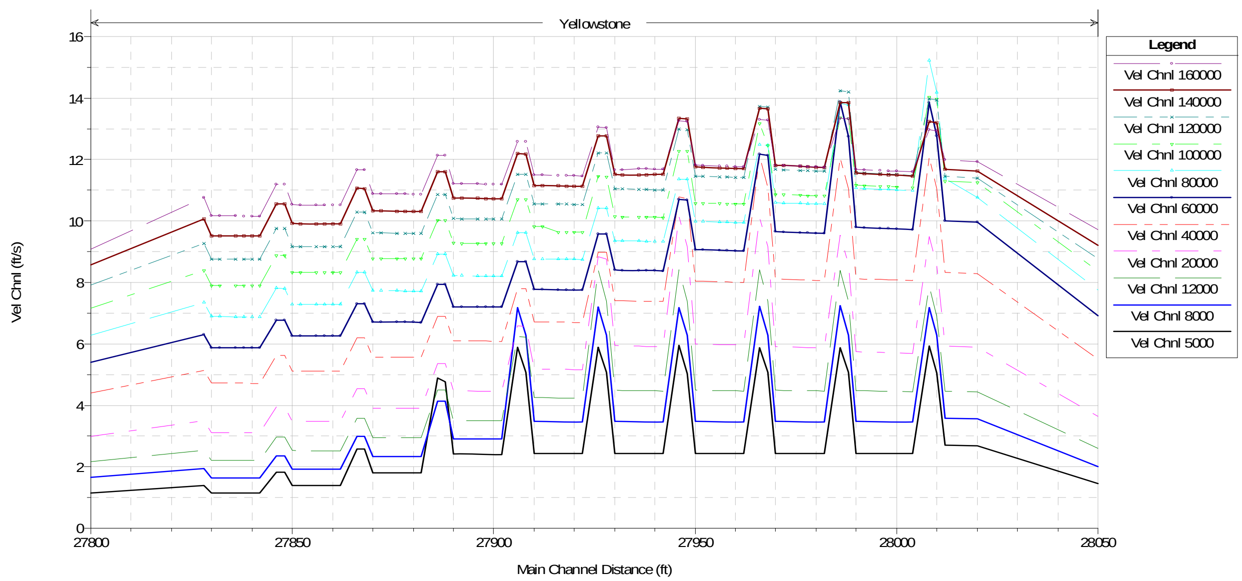


Figure 7. Computed Flow Velocity – 5% Slope Ramp, 1’ Drop

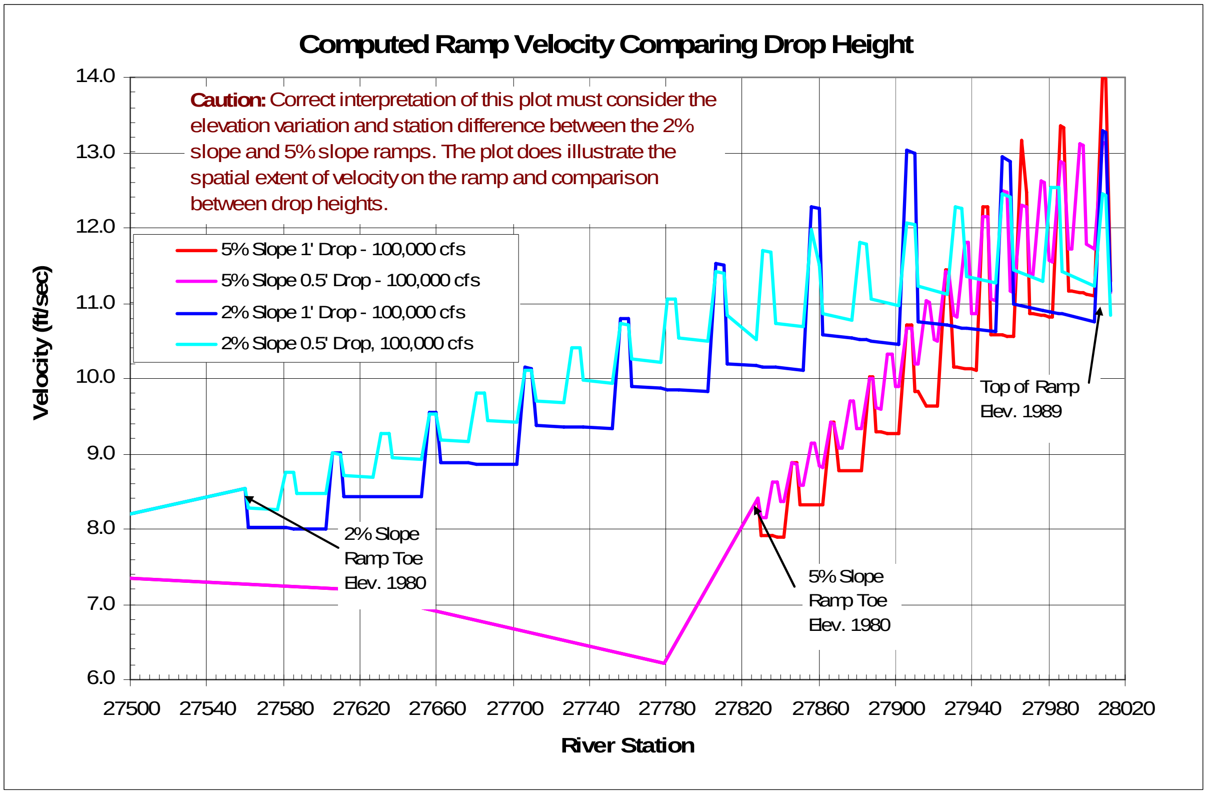


Figure 8. Computed Flow Velocity Comparing Drop Height

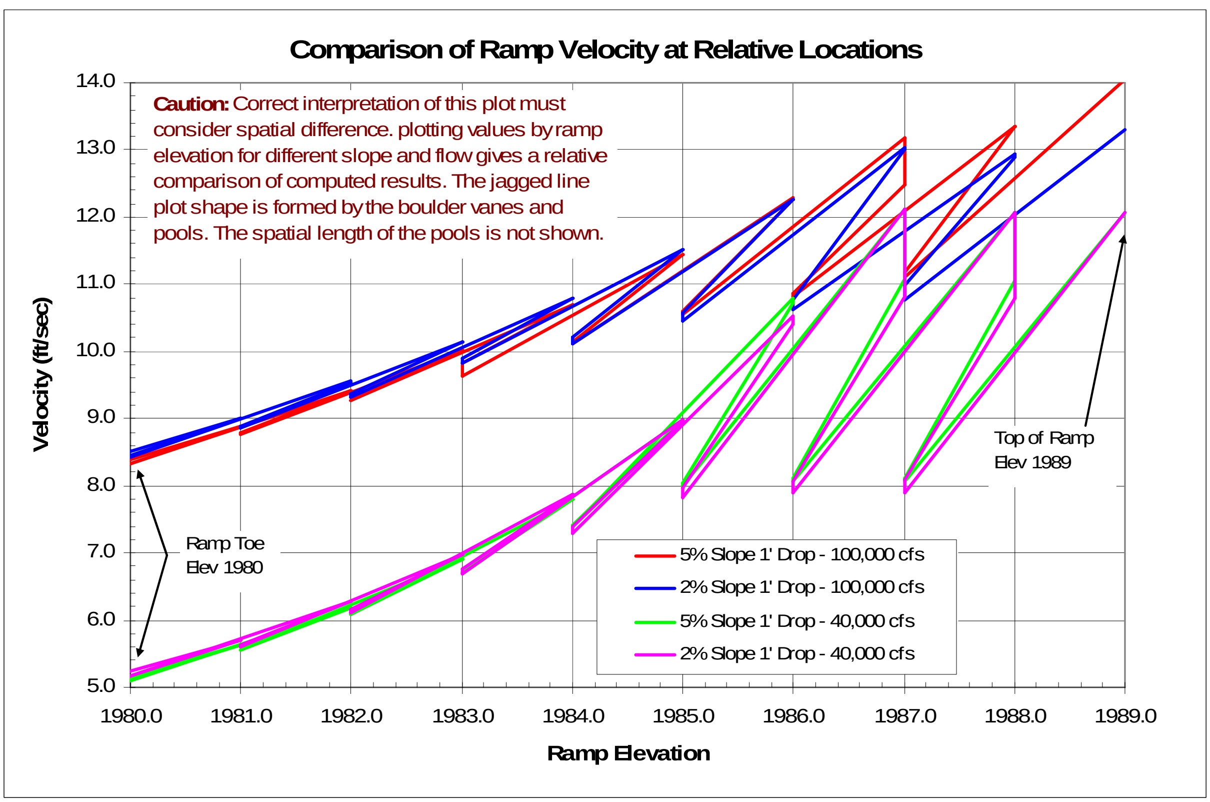


Figure 9. Comparison of Ramp Velocity at Relative Locations

NOTE: All computed velocities are average velocities. Actual velocity will vary considerably both horizontally across the ramp and vertically within the water column.

3.5 STABLE ROCK SIZE FOR RAMP

Stable rock size was evaluated using ramp flow velocities computed with the HEC-RAS model. A comparison of results from different flow events and locations on the ramp is shown in the below tables. The critical threshold for the initiation of motion is often expressed as critical shear stress which relates the initiation of material movement to material size, flow depth, and slope. Additional empirical methods for evaluating material movement are also available.

For the conceptual analysis, stable rock size was computed using the flow velocity and the turbulent method presented by Ishbash on HDC Sheet 712-1 (WES, 1988). Additional computations were performed using the steep slope riprap equation in EM 1110-2-1601 (USACE, 1994, eq. 3-5). Stable rock size computations demonstrate that very large rock is required for ramp stability. In addition, it is doubtful that all rock on the ramp will be stable for extreme events. Results from the Ishbash computation method using HEC-RAS results for a range of flows and ramp slope are shown in Table 5.

Table 5. HEC-RAS Rock Ramp Stability Computations.

Yellowstone River - 160,000 cfs Total Flow

Ramp Position		5% Slope, 1 ft drop	5% Slope, 0.5 ft drop	3.33% Slope, 1 ft drop	2% Slope, 1 ft drop	2% Slope, 0.5 ft drop
Top	Comp. Veloc. (ft/sec)	13.0	12.6	12.8	12.4	11.9
	Stable Rock Size D_{50} (ft)	*2.14*	*2.03*	*2.10*	*1.96*	*1.81*
Elev 1988 Step Center	Comp. Veloc. (ft/sec)	11.7	12.0	11.6	11.4	11.5
	Stable Rock Size D_{50} (ft)	*1.74*	*1.83*	*1.72*	*1.66*	*1.67*
Elev 1988 Boulder	Comp. Veloc. (ft/sec)	13.3	12.7	13.0	12.7	12.1
	Stable Rock Size D_{50} (ft)	*2.27*	*2.07*	*2.16*	*2.05*	*1.87*
Elev 1987 Step Center	Comp. Veloc. (ft/sec)	11.8	12.0	11.6	11.5	11.6
	Stable Rock Size D_{50} (ft)	*1.78*	*1.85*	*1.73*	*1.69*	*1.71*
Elev 1987 Boulder	Comp. Veloc. (ft/sec)	13.3	12.9	13.2	13.0	12.3
	Stable Rock Size D_{50} (ft)	*2.26*	*2.12*	*2.23*	*2.15*	*1.94*
Elev 1986 Step Center	Comp. Veloc. (ft/sec)	11.8	12.2	11.8	11.6	11.8
	Stable Rock Size D_{50} (ft)	*1.78*	*1.88*	*1.77*	*1.72*	*1.77*
Elev 1986 Boulder	Comp. Veloc. (ft/sec)	13.3	12.8	13.2	13.0	12.6
	Stable Rock Size D_{50} (ft)	*2.25*	*2.10*	*2.23*	*2.15*	*2.02*
Elev 1985 Step Center	Comp. Veloc. (ft/sec)	11.7	11.9	11.7	11.6	11.8
	Stable Rock Size D_{50} (ft)	*1.74*	*1.82*	*1.74*	*1.72*	*1.78*
Elev 1985 Boulder	Comp. Veloc. (ft/sec)	13.1	12.7	13.1	13.0	12.5
	Stable Rock Size D_{50} (ft)	*2.17*	*2.06*	*2.17*	*2.16*	*2.00*
Elev 1984 Step Center	Comp. Veloc. (ft/sec)	11.5	11.8	11.6	11.5	11.8
	Stable Rock Size D_{50} (ft)	*1.69*	*1.79*	*1.70*	*1.70*	*1.77*
Elev 1984 Boulder	Comp. Veloc. (ft/sec)	12.6	12.5	12.7	12.7	12.5
	Stable Rock Size D_{50} (ft)	*2.02*	*1.99*	*2.06*	*2.07*	*1.99*

Table 5. HEC-RAS Rock Ramp Stability Computations (Continued).

Yellowstone River - 100,000 cfs Total Flow

Ramp Position		5% Slope, 1 ft drop	5% Slope, 0.5 ft drop	3.33% Slope, 1 ft drop	2% Slope, 1 ft drop	2% Slope, 0.5 ft drop
Top	Comp. Veloc. (ft/sec)	14.0	13.2	13.8	13.3	12.5
	Stable Rock Size D_{50} (ft)	*2.52*	*2.21*	*2.45*	*2.26*	*1.98*
Elev 1988 Step Center	Comp. Veloc. (ft/sec)	11.2	11.7	11.1	10.9	11.3
	Stable Rock Size D_{50} (ft)	*1.59*	*1.75*	*1.57*	*1.52*	*1.63*
Elev 1988 Boulder	Comp. Veloc. (ft/sec)	13.4	12.9	13.3	13.0	12.5
	Stable Rock Size D_{50} (ft)	*2.28*	*2.12*	*2.25*	*2.14*	*1.98*
Elev 1987 Step Center	Comp. Veloc. (ft/sec)	10.9	11.4	10.8	10.7	11.1
	Stable Rock Size D_{50} (ft)	*1.50*	*1.65*	*1.50*	*1.46*	*1.58*
Elev 1987 Boulder	Comp. Veloc. (ft/sec)	13.2	12.3	13.2	13.0	12.1
	Stable Rock Size D_{50} (ft)	*2.21*	*1.93*	*2.22*	*2.17*	*1.86*
Elev 1986 Step Center	Comp. Veloc. (ft/sec)	10.6	11.0	10.6	10.5	10.8
	Stable Rock Size D_{50} (ft)	*1.43*	*1.55*	*1.43*	*1.42*	*1.48*
Elev 1986 Boulder	Comp. Veloc. (ft/sec)	12.3	12.2	12.3	12.3	12.0
	Stable Rock Size D_{50} (ft)	*1.93*	*1.89*	*1.94*	*1.92*	*1.83*
Elev 1985 Step Center	Comp. Veloc. (ft/sec)	10.1	10.8	10.2	10.2	10.5
	Stable Rock Size D_{50} (ft)	*1.31*	*1.49*	*1.32*	*1.32*	*1.41*
Elev 1985 Boulder	Comp. Veloc. (ft/sec)	11.5	11.4	11.5	11.5	11.4
	Stable Rock Size D_{50} (ft)	*1.67*	*1.66*	*1.69*	*1.69*	*1.66*
Elev 1984 Step Center	Comp. Veloc. (ft/sec)	9.8	10.2	9.9	9.9	10.2
	Stable Rock Size D_{50} (ft)	*1.23*	*1.33*	*1.24*	*1.24*	*1.33*
Elev 1984 Boulder	Comp. Veloc. (ft/sec)	10.7	10.7	10.8	10.8	10.7
	Stable Rock Size D_{50} (ft)	*1.46*	*1.45*	*1.48*	*1.49*	*1.47*

Yellowstone River - 60,000 cfs Total Flow

Ramp Position		5% Slope, 1 ft drop	5% Slope, 0.5 ft drop	3.33% Slope, 1 ft drop	2% Slope, 1 ft drop	2% Slope, 0.5 ft drop
Top	Comp. Veloc. (ft/sec)	13.9	13.1	13.9	13.9	12.3
	Stable Rock Size D_{50} (ft)	*2.45*	*2.18*	*2.45*	*2.45*	*1.93*
Elev 1988 Step Center	Comp. Veloc. (ft/sec)	9.8	10.7	9.7	9.7	10.3
	Stable Rock Size D_{50} (ft)	*1.22*	*1.45*	*1.21*	*1.20*	*1.35*
Elev 1988 Boulder	Comp. Veloc. (ft/sec)	13.9	12.7	13.8	13.2	12.0
	Stable Rock Size D_{50} (ft)	*2.46*	*2.06*	*2.42*	*2.21*	*1.83*
Elev 1987 Step Center	Comp. Veloc. (ft/sec)	9.6	10.3	9.6	9.5	9.9
	Stable Rock Size D_{50} (ft)	*1.19*	*1.35*	*1.17*	*1.14*	*1.26*
Elev 1987 Boulder	Comp. Veloc. (ft/sec)	12.2	11.6	12.0	11.8	11.2
	Stable Rock Size D_{50} (ft)	*1.89*	*1.73*	*1.85*	*1.78*	*1.61*
Elev 1986 Step Center	Comp. Veloc. (ft/sec)	9.1	9.6	9.0	9.0	9.4
	Stable Rock Size D_{50} (ft)	*1.05*	*1.17*	*1.04*	*1.03*	*1.13*
Elev 1986 Boulder	Comp. Veloc. (ft/sec)	10.7	10.5	10.7	10.6	10.3
	Stable Rock Size D_{50} (ft)	*1.46*	*1.41*	*1.45*	*1.44*	*1.36*
Elev 1985 Step Center	Comp. Veloc. (ft/sec)	8.4	8.8	8.4	8.4	8.8
	Stable Rock Size D_{50} (ft)	*0.90*	*1.00*	*0.90*	*0.90*	*0.98*
Elev 1985 Boulder	Comp. Veloc. (ft/sec)	9.6	9.5	9.6	9.6	9.5
	Stable Rock Size D_{50} (ft)	*1.17*	*1.15*	*1.17*	*1.18*	*1.14*
Elev 1984 Step Center	Comp. Veloc. (ft/sec)	7.8	8.1	7.8	7.8	8.2
	Stable Rock Size D_{50} (ft)	*0.77*	*0.85*	*0.77*	*0.78*	*0.85*
Elev 1984 Boulder	Comp. Veloc. (ft/sec)	8.7	8.6	8.7	8.8	8.7
	Stable Rock Size D_{50} (ft)	*0.96*	*0.95*	*0.97*	*0.98*	*0.96*

A second method was also used to evaluate stable rock size for the conceptual analysis. This method uses the steep slope equation presented within EM 1110-2-1601 (USACE, 1994, eq. 3-5). This method computes stable rock size based on unit discharge and slope. Results from those computations are shown in Table 6.

Table 6. Steep Slope Rock Ramp Stability Computations.

Design Flow (cfs)	Bottom width	Unit q (cfs/ft)	Flow Factor q * 1.25	Design Slope (ft/ft)	COE D_{30} (ft)	Est. D_{50} (ft)
60,000	550	109	136	0.050	3.08	3.70
100,000	550	182	227	0.050	4.33	5.19
160,000	550	291	364	0.050	5.92	7.11
60,000	550	109	136	0.020	1.85	2.22
100,000	550	182	227	0.020	2.60	3.12
160,000	550	291	364	0.020	3.56	4.27
60,000	700	86	107	0.050	2.62	3.15
100,000	700	143	179	0.050	3.69	4.42
160,000	700	229	286	0.050	5.04	6.05
60,000	700	86	107	0.020	1.58	1.89
100,000	700	143	179	0.020	2.22	2.66
160,000	700	229	286	0.020	3.03	3.64

Rock size computed with the steep slope equation determined an even larger rock required for stability than the Ishbash method using the HEC-RAS velocity. Results are tabulated for both 5% and 2% slope and two bottom widths. It should be noted that the steep slope method ignores the energy dissipation provided by the individual steps on the ramp.

Recommended Rock Size

Based on the computation results, a rock size in excess of 4 ft is recommended for the ramp boulders. Constructing the entire ramp from 4 foot boulders is probably cost prohibitive. Computed rock size is based on average HEC-RAS model average flow velocity and a vertical velocity distribution is expected within the pool section of the ramp. Based on the analysis, a rock size of 2 feet is recommended for the remainder of the ramp. Using the Ishbash method and HEC-RAS computed velocity, the determined D_{50} rock size for 100,000 cfs flow was about 1.5 feet. Stability for the 2 foot diameter rock is questionable for flow events in excess of 60,000 cfs. Future efforts will revise the rock size required for stability. However, it is likely that entire ramp stability for events in the critical flow range before the ramp begins to submerge (roughly flow greater than 80,000 - 100,000 cfs) is not feasible without using rock approaching 3 foot diameter for the entire upper portion of the ramp. Referring to the flow frequency analysis, 100,000 cfs is approximately a 10-year event.

3.6 RAMP ICE STABILITY

Based on the ice analysis conducted by CRREL (App. D), the boulder size required for ice stability is estimated to be in the range of 4-6 ft diameter. This correlates fairly well with dam project history, where the larger rocks placed are on the order of 3 ft diameter. As the maintenance record shows, the dam crest riprap has been moved by ice and high flow conditions. Use of natural rock boulders or a simulated rock formed with concrete will provide stability for the boulder steps. In between the steps, loose rock riprap of a much smaller diameter is proposed. Ice damage may occur to portions of the ramp with the smaller rock. Ramp ice stability is summarized as follows:

- CRREL analysis determined a boulder size required for stability of 4-6 ft diameter.

- Natural rock boulders or simulated rock boulders will be placed to form the boulder steps that conform to the recommended size.
- A concrete cap will be placed on the existing dam to resist ice forces. The cap is required to resist ice loads as calculated in the structural analysis contained within Appendix G.
- Ice forces on the ramp will be mitigated by the upstream concrete crest. The crest should break up the ice floe into much smaller fragments.
- Smaller rock will be used to form the ramp bottom between the boulders. The smaller rock is integral to achieving fish passage. Some ice damage may occur to this portion of the ramp. The boulder rows should serve to shield the bottom portion of the ramp. In addition, flow depths that transport the ice sheets should keep the ice above the ramp bottom and reduce damage potential.

3.7 RAMP FISH PASSAGE RELATED TO RELEVANT PALLID STURGEON SWIM GUIDANCE.

Design swim guidance for the Pallid Sturgeon is available from documented laboratory testing. Test results were reviewed for conclusions specific to rock ramp navigability. The report *Assessment of Behavior and Swimming Ability of Yellowstone River Sturgeon for Design of Fish Passage Devices, White and Mefford, 2002* (USACE, 2002, App. A) included tests conducted to evaluate the behavioral response of adult shovelnose sturgeon to velocity, substrate, horizontal turbulence, vertical turbulence, and three prototype fishways. Relevant conclusions are:

- Sturgeon successfully negotiated the range of average velocities tested (0.8-6.0 ft/sec) over all substrates (smooth, fine sand, coarse sand, gravel, and cobble).
- As substrate increased, movement success declines but small sample size precluded definitive conclusions.
- Sturgeon were able to negotiate horizontal and vertical eddies. However, larger eddies tended to cause delays. Asymmetrical eddies were also noted to be problematic for passage.
- Fishway tests indicated that the rock fishway passage success was much improved.

A second report *Preliminary Comparison of Pallid and Shovelnose Sturgeon for Swimming Ability and Use of Fish Passage Structure, Kynard, 2002* (USACE, 2002, App. B) gathered information in an experimental flume on the swimming ability and behavior of pallid sturgeon in two different flow regimes, laminar and a complex turbulent flow created by a structure. Relevant conclusions are:

- Pallid sturgeon demonstrated the swimming ability to navigate complex currents in a side-baffle fish ladder at 6% slope and should be able to swim upstream in complex flows in other passage situation, like rock ramps, as along as velocities are appropriate.
- Pallid and shovelnose sturgeon swam through the side-baffle section off the bottom, passing quickly through 65 cm/sec (2.1 ft/sec) velocity in only 1-2 sec using about two tailbeats/sec.
- Current velocity in fish ladders or rock ramps that enable fish to swim in the prolonged mode, and do not require the burst swim mode, seems preferable for these species.

Relevant pallid sturgeon fish passage criteria from previous studies ((USACE, 2002, App. A, B) does not include maximum velocity criteria. However, the computed average velocities determined with the HEC-RAS model are high enough to be concerning for flows in excess of 40,000 cfs for the upper portion of the ramp. Interpretation of results should consider that the ramp geometry is variable and these results are not reflected in the HEC-RAS computations. Flow duration data (Hydrology Appendix B, Table 3) indicates that the percent of time that a flow of 40,000 cfs is equaled or exceeded in June is about 30%. For all other months, the percent of time decreases to about 5% or less.

While velocity and turbulence in the ramp center may be excessive for high flows, the sloping ramp and U shape should provide a portion of the ramp that is amenable to fish passage. In addition, computed velocities are average. Actual velocity will vary both horizontally across the ramp and vertically within the water column. Future design will determine velocity variation within the ramp. Given the computed high velocities for the upper portion of the ramp, it seems probable that fish passage success for high flow events may be less than desirable.

Comparing the computed results for the 0.5' drop height and the 1.0' drop height, it appears that the larger drop height is preferable. Average velocity at the boulder crest is similar for both drop heights. From a fish passage aspect, the flatter slope doubles the number of drop and serves to lengthen the high velocity and turbulent zone and may not be preferable.

The required rock size for ramp stability is in the 2 – 3 foot diameter range with greater than 4 foot diameter rock used for the boulder vanes. Previous studies indicated that substrate may impact fish passage success. It is probable that Yellowstone River sediment load will naturally fill the rock ramp voids with small cobbles and normal bed load. The smaller material will become mobile during large flows.

3.8 FUTURE DESIGN EFFORT

Future design efforts are required to further evaluate ramp components. Recommended design components include a two-dimensional computational model and a physical model. The preliminary concept is that the physical model would be constructed in a flume to represent a short ramp width. Variable slopes would be evaluated in the flume. Boulder placement adjustment could also be checked. The physical model would illustrate velocity variation/depth down the ramp both horizontally and vertically. The two-dimensional model would be constructed of the entire ramp area and include a segment upstream and downstream of the ramp. A two-dimensional model would be constructed of both existing and refined conditions. The two-dimensional model would illustrate depth averaged velocity magnitude and direction throughout the area. The physical and numerical modeling results can be used to refine ramp design features. Specific items are as follows:

- Use the physical model to calibrate depth averaged two-dimensional model results. Results from both models can be used jointly to develop a comprehensive view of flow parameters and verify results.
- Construct the two-dimensional model of both existing and ramp conditions. Compare results to verify fish passage improvement.
- Compare ramp velocities with relevant pallid sturgeon swim guidance. Evaluate substrate effect on ramp fish passage success. Revise ramp geometry as necessary to optimize fish passage.
- Evaluate velocity magnitude distribution through the ramp.
- Evaluate flow velocity through the boulder gaps to aid with design.
- Design the ramp crest with a variable elevation and structure to facilitate fish passage.
- Evaluate the impact of step length and drop height on ramp velocity/depth.
- Evaluate ramp curvature and transverse slope on the boulder vanes.
- Evaluate for a range of flow conditions to optimize boulder placement for pools and gaps.
- Evaluate flow parameters at the intake structure.
- Evaluate flow parameters at the base of the ramp and energy dissipation requirements.
- Revise ramp rock stability estimate based on computed results. Ramp geometry and rock components may require revision to provide required stability.

3.9 RAMP SUMMARY

The analysis performed to date is a conceptual design to evaluate feasibility of the rock ramp. The analysis compared basic ramp geometry and identified several design restrictions that need to be further evaluated. Results are summarized as follows:

- All computed velocities are average velocities. Actual velocity will vary considerably both horizontally across the ramp and vertically within the water column.
- Relevant pallid sturgeon guidance indicated that sturgeon successfully negotiated the range of average velocities tested from 0.8 to 6.0 ft/sec. Computation results determined average flow velocities at the crest in excess of 10 ft/sec for flows in excess of 40,000 cfs. Proper design of the resting pools and boulder vanes are critical to optimize fish passage success. Given the computed high velocities for the upper portion of the ramp, it seems probable that fish passage success for high flow events may be less than desirable. Flow duration data indicates that these flows are equaled or exceeded in June about 30% of the time.
- Results determined high velocities at the ramp crest. To facilitate fish passage and maintain flow distribution, an uneven crest with possibly natural rock set in the crest concrete is probably required. These details will be determined in final design.
- The performed analysis used HEC-RAS to evaluate ramp flow parameters. Both a physical and two-dimensional model is recommended for future design efforts to determine ramp geometry.
- Analysis evaluated ramps with a slope between 2% and 5%. Computed flow velocities in the HEC-RAS model did not vary that much for the different slopes. It is expected that a refined analysis using both physical and two-dimensional models would better illustrate the ramp stability and fish passage advantages between different ramp slopes and pool length. Due to the change in ramp size, a significant cost difference is expected for the two slopes.
- Large rock is required to provide ramp stability. Previous studies indicated that substrate may impact ramp fish passage success.
- Given the large boulder size, it may be more economical to use the 1' drop and a flatter ramp slope (longer ramp). This would reduce the number of boulder rows. The ramp slope could be adjusted to optimize for the boulder size that is economically available and also refined to consider constructability.
- Computed results show that the 100-yr event requires a very large rock size with a D_{50} of over 2.6 ft for the 100-year event at the top of the ramp. Below elevation 1984, the stable rock size has a D_{50} of 1.1 ft. This still equates to a D_{100} of 24 to 27 inches. For the 10-year event, the lower half of the chute has a stable rock size of about 18 – 21 inches. Future efforts will revise the rock size required for stability. However, it is likely that entire ramp stability for larger flow events is not feasible.
- A boulder size of 4-6 feet is required on the ramp. The base rock D_{100} of less than 24 inches for the ramp is recommended. Analysis determined that this rock is likely not stable for events greater than 10-year near the top of the ramp. The interface of the concrete cap and ramp was not evaluated with HEC-RAS and will provide stability in this location.
- A concrete cap provides stability for the upper portion of the ramp and protects against ice damage. To facilitate fish passage and maintain flow distribution, an uneven crest with possibly natural rock set in the crest concrete is probably required.
- The right bank floodplain currently conveys a limited amount of flow. An excavated bypass on the right bank could be used to reduce the ramp unit discharge for extreme events and reduce the maximum ramp flow to enhance ramp stability.

4. UPSTREAM DIVERSION OPTION

Analysis was performed to evaluate the feasibility of moving the intake upstream from the present location. The existing diversion dam would be removed to a suitable level. Highlights of the proposed project are as follows:

- Remove existing dam to a suitable level and decommission existing intake.
- Install channel stability and stabilization structures within the Yellowstone River as shown in Appendix D of this report.
- Dam removal impacts and potential effect on the upstream diversion option have not been evaluated.

- The alignment includes two crossings beneath the railroad and a tributary crossing. Combine the tributary crossing and the upstream railroad crossing into a single crossing to save funds. The pipe length for the single crossing is longer but an entrance and exit is eliminated.
- Canal alignment was selected to minimize impact to the railroad right-of-way. A second alignment, that was placed on the river side of the existing railroad, was evaluated but abandoned.
- Canal bottom width of 50 feet with 2H on 1V side slopes. The VE study assumed 1.5H on 1V. However, due to the cut depth and slope length the side slope was flattened for stability. Also, a mid slope bench was added to reduce slope length for erosion and a maintenance access road on one side.
- Canal longitudinal slope of 0.00013 ft/ft. Maximum flow depth is about 10 feet. With 2 feet of freeboard, this gives a minimum canal depth of about 12 ft.
- Canal length of over 13,000 feet.
- Increase the number of gates at the new intake to divert 1400 cfs at minimum Yellowstone flow of 5,000 cfs. The existing headgate structure has 11 5-ft diameter sluice gates at a bottom elevation of about 1985 ft. The existing Yellowstone River invert at the diversion point is about elevation 1990 to 1992 (difficult to estimate from the available topo). Due to sedimentation and maintenance concerns, it is preferable to locate the headgate invert at or above the channel invert. The Yellowstone River flow elevation at 5,000 cfs is about elevation 1996. In order to divert 1,400 cfs, the headgate structure size would have to be increased from 11 to 17 gates.
- Construct a straight drop structure downstream of the inlet to reduce the canal tailwater depth and accommodate the required diversion of 1400 cfs at minimum Yellowstone flow rates. Canal invert elevation at the gate was selected as 1986.7 to be low enough to allow 1,400 cfs flow diversion combined with a canal flow depth of 10 ft.
- Construct an 8' high drop structure at the new canal entrance to the old canal. A baffled chute may be preferable for this location and should be evaluated in future design.
- Construct Yellowstone River floodplain protection berms to protect against canal flooding. Significant floodplain fill will impact Yellowstone River flood elevations that may require mitigation.
- Remove and replace farm access roads at several locations to accommodate the new structures and canal.

Plan views of project features and the new canal alignment are illustrated in Plate C-4.

4.1 PROFILE TABULATION.

Elev.	Station	
1992.0	133+80	New Intake at Yellowstone River with 17 5'x5' sluice gates.
1992.0	133+70	D/S end of new Intake, upstream crest of straight drop structure.
1986.73	133+70	U/S toe of straight drop structure.
1986.72	133+55	D/S end of straight drop structure, transition to trapezoidal earth channel.
1986.72	132+05	End of transition to trapezoidal channel, 50' bottom width, 2H on 1V sideslopes. Canal slope of 0.00013 ft/ft. Compound channel with side maintenance berm and midslope erosion berm.
1985.0	0+50	U/S end of drop structure, 35 ft bottom width rectangular, vertical drop of eight feet. Roughly follows SAF basin outlet design.
1977.0	0+22	D/S end of drop structure with end sill. Transition to existing canal.
1977.0	0+00	Centerline of existing canal.

4.2 HEC-RAS MODEL

An HEC-RAS model was constructed to determine the geometry of the upstream diversion channel and structures. The existing condition model was modified to include a reach for the new canal. Features within HEC-RAS were used to size the new intake number of gates and the culvert structure geometry for the new railroad crossings. Design of project features was based on attaining the canal diversion flow of 1,400 cfs at the minimum Yellowstone River flow of 5,000 cfs. As discussed in the *Existing Conditions HEC-RAS Model* section, the available survey data did not include Yellowstone River bed topography. Therefore, the accuracy of the computed Yellowstone River stage-flow relationship at the proposed diversion site is limited.

Model Roughness

The HEC-RAS model uses a Manning roughness value of 0.030 for the channel region of the new canal. The roughness parameters established for the model were similar to the previous modeling effort. Since the canal flow level is relatively constant, vegetation growth should be minimal and a roughness value lower than the Yellowstone River is expected.

Structure Modeling

All proposed structures were modeled with HEC-RAS for the conceptual analysis. The intake gate structure was modeled to consist of similar dimension gates (5’x5’) as the existing condition intake. Notable parameters used in the model for all structures are included in Table 7.

Table 7 – Upstream Diversion Option HEC-RAS Data

New Intake Structure	Modeled as sluice gates, discharge coefficient of 0.6 Orifice coefficient of 0.8, Head exponent of 0.5.
Culverts	Circular concrete pipe, square edge with headwall, entrance loss coefficient 0.5, exit loss coefficient 1.0, pipe roughness of 0.012.

Intake Dam

The new model assumed that the existing dam would be removed entirely. Complete removal may not be preferable due to concerns with erosion, bank stabilization, and impact to the Yellowstone River. However, complete removal results in a lower upstream water surface elevation for evaluating diversion capability.

> **NOTE**: Dam removal would almost certainly result in some bed and bank erosion upstream of the existing structure. Such erosion was NOT included in this analysis. Detailed analysis would be required to evaluate erosion potential due to dam removal and also optimize dam removal.

4.3 HEC-RAS MODEL RESULTS

The new model was used to evaluate diversion capability, canal capacity, and structure design. All upstream diversion components were sized to provide the required diversion capacity of 1,400 cfs when the Yellowstone River total flow is a low as 5,000 cfs. Different combinations of canal bottom width, drop structure height, and intake structure gates are also possible. For the design flow rate of 1,400 cfs, the computed normal canal flow depth is estimated as less than 10 feet. Proposed structures affect the flow depth with a minimum depth of less than 8 feet in the reach upstream of the canal exit drop structure. The Yellowstone River rating curve and canal flow depth are shown in Figure 10.

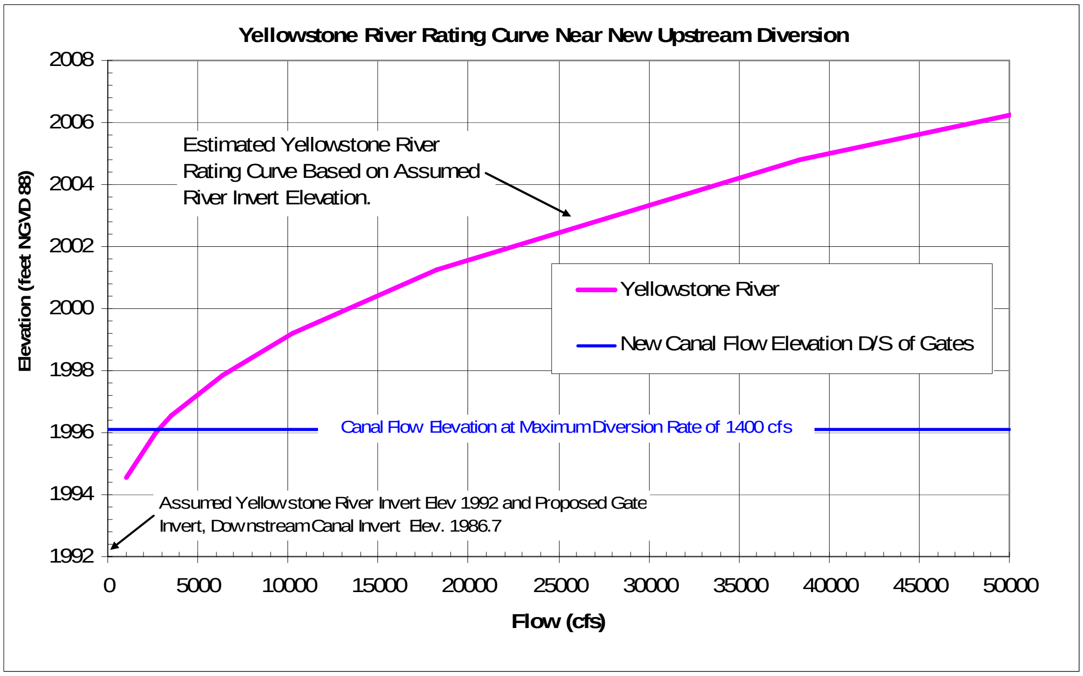


Figure 10. Yellowstone River Rating Curve Near Upstream Diversion

4.4 New Canal Alignment and Geometry

The canal alignment was selected based on site constraints. The existing railroad grade is very near the Yellowstone River for the first several thousand feet upstream of the existing dam. An alignment was considered that was on the river side of the existing track. However, this alignment required about 250 – 300 feet of encroachment into the river for about 5,000 feet. This encroachment would require a berm armored with large riprap to protect the canal from flooding and ice damage. The selected alignment requires two railroad crossings but eliminates the substantial river fill.

The new canal will have a 50 foot bottom width and 2H on 1V sideslopes at a slope of 0.00013 ft/ft. The canal includes a 16 foot wide maintenance access road located 12 feet vertically above the canal invert. On the opposite bank, the cut depth is over 60 feet with a large sideslope length for about 6,000 ft of canal length. The canal will include a midslope channel and berm to intercept sideslope flow and prevent slope erosion. Canal invert elevation was designed to allow diversion from the Yellowstone River at 5,000 cfs. Excavation quantities estimated along the proposed alignment are as follows:

New Canal Excavation – 3,720,000 cubic yards of cut.
New Canal Length – 13,400 feet including all structures.

The new canal profile is illustrated in Plate C-5. Typical sections are illustrated in Plates C-6 and C-7.

4.5 New Intake.

A new intake is required on the Yellowstone River. The new structure is rectangular with 17 gates. Assuming 4 feet between each gate, the total length perpendicular to the river is about 159 ft including 5 feet each side of the outside gate structure. Top of structure is elevation 2016 that is about 4 - 6 feet above existing ground elevation and 3 feet above the estimate 100-year Yellowstone River flow elevation. The new intake structure will include a straight drop structure on the downstream side of about 3.3 feet vertical drop. Gate invert elevation is assumed as elevation 1992 based on approximate channel bottom elevation. No channel survey data was available, elevations are approximate based on available information. A schematic of the structure is included in Plate C-8.

Note: Locating the new intake elevation near the river bottom provides additional head at low flow and reduces the number of gates. However, sediment load at higher Yellowstone River flow levels will be an issue. Future design will consider the incorporation of several bi-fold or top lowering gates to alleviate sediment during periods of higher Yellowstone River flow diversion.

4.6 RAILROAD AND TRIBUTARY CROSSINGS

Two crossings of the railroad are required with the new canal alignment. For the conceptual design, the maximize culvert size beneath the railroad was assumed to be an 8 foot diameter. This assumption is based on boring/jacking limits as stated in the structural appendix.

Elev. Station (Centerline)
1986.0682+70 Bore/jack culvert beneath railroad. Continue culverts beneath stream and farm road. Five culverts each 8' diameter RCP, length of about 460 feet, with concrete headwall. Culvert installed from station 80+40 to 85+00. Total culvert length of 460 feet, match canal slope of .00013 ft/ft. Rock riprap is included 10 feet upstream and downstream of the structure. A schematic of the structure is included in Plate C-9 and the profile is illustrated in Plate C-10.

Elev. Station (Centerline)
1985.086+30 Bore/jack culvert beneath railroad. Five culverts each 8' diameter RCP, center line length of about 360 ft with concrete headwall. Culvert installed from station 4+50 to 8+10. Top of rail about 2016, existing ground slopes upward to about 2038 at edge of culvert. Rock riprap is included 10 feet upstream and downstream of the structure. A schematic of the structure is included in Plate C-11 and the profile is illustrated in Plate C-12.

NOTE: Replacement of the five 8' diameter culverts is possible with a single large siphon. Preliminary analysis indicated that the siphon diameter would be about 20'. Due to concerns with construction beneath the railroad, this option was not pursued for the conceptual design. Detailed design should investigate this option.

4.7 DROP STRUCTURES

Station 133+50. The drop structure downstream of the new intake structure is a SAF straight drop structure following criteria illustrated in HDC Sheet 623 - 624-1 (WES, 1988). The structure width matches the intake width with vertical sidewalls. Downstream of the structure, a transition is required from the drop to the new canal. The structure would include grading to avoid using wingwalls through the transition. Rock riprap is included 10 feet downstream of the end sill. The drop structure is required to transition from the gate invert to the downstream canal invert.

Station 0+50. The drop structure from the new canal to the existing canal is a SAF straight drop structure following criteria illustrated in HDC 624. A baffled chute structure may be preferable for this location, may be less cost, and should be evaluated in future design. The structure is rectangular with a 35' bottom width. Structure width is reduced from canal bottom width to lower upstream flow velocities and canal erosion potential. Structure width will be revised in future design. Rock riprap is included upstream of the structure for 25 feet and downstream of the structure for 20 feet. The drop height is 8 feet.

The structure length of both basins was estimated based on roughly following the criteria developed for SAF basins in HDC Sheet 623 - 624-1 (WES, 1988). Estimated tailwater elevation at both locations is not within the optimum range and energy dissipation is expected to be less than desirable. Basin design will be refined in future analysis.

4.8 FLOODPLAIN PROTECTION BERMS

Two berms are required to prevent Yellowstone River flooding from damaging the canal. One berm is parallel to the canal and the second berm ties off to high ground upstream of the intake. The location of both berms is illustrated in Plate C-4 and a cross section of the floodplain berm is shown in Plate C-7.

Conceptual design berm height was estimated based on the 100-year open water elevation of 2013 in the vicinity of the new diversion. Ice affected stages were not evaluated. If an additional 3 feet is included for freeboard, the top of berm elevation is roughly 5 – 7 feet above existing grade. The berm is parallel to the canal and is installed from the new intake structure at the Yellowstone River downstream to the railroad culvert crossing at station 82+70. Downstream of this location, the canal is protected by the railroad embankment. The berm will be earth only, not designed to resist ice forces as it is remote from the river. Construction of the berm will place significant fill within the floodplain and will probably impact Yellowstone River flood elevations and floodway. Mitigation for the berm will probably be required.

The typical cross section for the berm is 5 feet above the existing grade with 3H on 1V sideslopes and a ten foot top width. A second berm is required upstream of the intake structure to protect the canal from direct flooding and ice jam attack. In order to accommodate rock riprap protection for this berm, a top width of 15 feet is required. The berm proceeds from the structure northwest toward an existing high knoll over a distance of about 470 feet. Quantities are estimated as follows:

Canal Floodplain Protection Berm Length – 5,220 feet.
Canal Floodplain Protection Berm Fill – 24,200 cubic yards (average 125 sq ft per ft berm)
Yellowstone Upstream Berm Length – 470 feet
Yellowstone Upstream Berm Fill – 1600 cubic yards (average 92 sq ft per ft berm)

4.9 ROCK RIPRAP

Rock riprap is included at several locations where localized turbulence may occur. Rock riprap is also required to protect the canal from ice jam flooding on the Yellowstone River along the Yellowstone River upstream berm.

In the vicinity of structures for erosion protection due to turbulence, all rock riprap is a 12” layer thickness. Rock riprap is also included on the Yellowstone River flood protection berm located upstream of the intake structure. This berm serves to protect the canal from open water and ice jam flooding. The downstream side of the canal is assumed to be protected from ice jam action as it is located away from the Yellowstone River and in the flow shadow of the upstream flood protection berm and intake structure. Rock for this location is a 4 foot layer thickness to resist the ice forces.

Downstream of Intake drop structure for a distance of 10 ft, Station 133+15 to 133+05
Upstream and Downstream of Culvert Crossings for a distance of 10 feet (station 85+10 to 85+00, station 80+40 to 80+30, station 7+50 to station 7+40, and station 5+20 to 5+10).
Upstream of drop structure for a distance of 25 feet, station 50 to station 75.
Downstream of drop structure for a distance of 20 feet, station 22 to station 02, wrap around the existing canal banks for a distance of 10 feet each direction. Rock riprap placement is summarized in Table 8.

Table 8. Rock Riprap for Upstream Diversion Summary

Location	Station	Layer Thickness
Downstream of Intake Drop for 10 feet	Station 133+15 to 133+05	12”
Upstream and Downstream of Culvert for 10 feet	Station 85+10 to 85+00 Station 80+40 to 80+30 Station 7+50 to 7+40 Station 5+20 to 5+10	12”
Upstream and downstream of drop structure for 25 and 20 feet	Station 75 to 50 Station 22 to 2 (wrap on existing canal banks as needed)	12”
Yellowstone protection berm	NA – 470 foot length	48”

4.10 FUTURE DESIGN EFFORT

Future design effort for the upstream diversion is required to define Yellowstone River stage-flow rating at the proposed diversion site. River channel surveys are required to provide an accurate model and evaluate the feasibility of the proposed structure. In addition, different combinations of canal bottom width, drop structure height, and intake structure gates should be evaluated to optimize the minimum cost design.

4.11 UPSTREAM DIVERSION SUMMARY

A conceptual analysis was conducted to evaluate the feasibility of moving the intake upstream from the present location. The existing diversion dam would be removed to a suitable level. An HEC-RAS model was constructed of the new upstream diversion to verify that the required diversion rate of 1,400 cfs could be achieved with the designed components. Results are summarized as follows:

- Major components of the upstream diversion includes an excavated canal, a downstream drop structure, a railroad crossing, a combined railroad and stream crossing, flood protection berms, a new intake structure with drop, removal of the existing dam and decommissioning the intake structure, and Yellowstone River stabilization structures.
- Design of project features was performed with an HEC-RAS model. Computations are based on assumed river channel elevations. Design is suitable for a conceptual level only.
- Locating the new intake elevation near the river bottom provides additional head at low flow and reduces the number of gates. However, sediment load at higher Yellowstone River flow levels will be an issue. Future design will consider the incorporation of several bi-fold or top lowering gates to alleviate sediment during periods of higher Yellowstone River flow diversion.
- Canal flow depth is less than 10 feet with a normal velocity of less than 2.5 ft/sec.
- The drop structure at the new canal junction with the existing canal is required to meet grades. The structure has a width narrower than the upstream canal to reduce flow velocity. Future design will further evaluate canal erosion potential and structure size.
- Different combinations of canal width and structure size are possible to meet the required diversion rate. These combinations were not investigated due to the conceptual nature of the design.
- A single siphon could be used in place of the five 8’ diameter culverts. Due to concerns with construction beneath the railroad, this option was not pursued for the conceptual design.
- Floodplain berm construction will probably impact Yellowstone River flood elevations and floodway. Design and cost for any mitigation was not included in the conceptual analysis and should be addressed in future design.

5. FISH SCREEN HYDRAULICS

Both alternatives require a fish screen installed within the irrigation canal downstream of the intake to return entrained fish from the canal back to the Yellowstone River. Fish screen design was briefly evaluated to determine any hydraulic items that should be further evaluated in future design. In addition, fish screen head loss was evaluated with respect to screen mesh size. Screen design data was reported in the Concept II report (USBR, 2004, App. A pg. 13-16). Reported significant features includes a V configuration screen with a total length of 440 feet, design flow rate of 1400 cfs, approach velocity of 0.4 ft/sec, sweeping velocity of 2 to 2.5 ft/sec, 1.75 mm slot opening stainless steel wedge wire with about 40% open area, a 12 inch sill height of the screen above the invert, a bypass return structure that includes 700 feet of 48" diameter pipe and slide gate, a check structure with two radial gates to raise canal head and prevent Yellowstone River backflow through the bypass pipe, and an estimated head loss through the baffles and fish screen of less than 0.5 feet.

- The Concept II report (Bureau, 2004, App. A pf. 14) states that sediment deposition is reportedly negligible in the reach of the canal. However, given the small mesh size and alteration of hydraulics through the screen area, additional sediment evaluation is recommended to estimate removal requirements and potential screen blockage issues.
- The crown of the bypass culvert is about equal to the 5,000 cfs Yellowstone River flow elevation. Therefore, the bypass pipe will nearly always be filled with backwater from the Yellowstone River, even during winter months. Freeze damage to the bypass structure may be an issue if provisions are not included to dewater the pipe. Slide gate closure would prevent backup into the canal.
- The proposed screen size of 1.75 mm slot opening may be smaller than required and feasible for reliable operation. As head loss and flow velocity through the screen decrease with open area, it may be possible to reduce the total screen length by using a larger opening.
- With regard to the fish screen bypass pipe, existing elevations in the proposed screen location prevent the effective use of an open channel due to high cut depths. An alternative alignment is possible that would provide a combination of open channel and about 300 feet of pipe length.

6. REFERENCES

Buesing, A., 2006. *Safety and Fish Passage for Low-Head Dams,* Proceedings Joint Federal Interagency Conference, April 2006.

Bureau of Reclamation, 2005. *Draft Biological Assessment: Future Operation of the Lower Yellowstone Project with Proposed Conservation Measures*, Montana Area Office, Billings Montana.

Bureau of Reclamation, 2004. *Intake Diversion Dam, Fish Protection and Passage Concept Study Report II,* Bureau of Reclamation, Technical Service Center, Denver, Colorado.

Bureau of Reclamation, 2000. *Intake Diversion Dam, Yellowstone River, Montana, Fish Protection and Passage Concept Study Report*, Bureau of Reclamation, Technical Service Center, Denver, Colorado.

Breining, G., 2003. *Rapid Changes on the Red River,* Minnesota Department of Natural Resources, Minnesota Conservation Volunteer, Nov-Dec 2003.

USACE, 2002. *Lower Yellowstone River Intake Dam Fish Passage Alternatives Analysis,* prepared by U.S. Army Corps of Engineers, Omaha District.

USACE, 1994. *Hydraulic Design of Flood Control Channels, EM 1110-2-1601, Change 1*, Department of the Army, U.S. Army Corps of Engineers, Washington, D.C.

WES, 1988. Hydraulic Design Criteria Separates, Waterways Experiment Station, Transmitted June 1988.

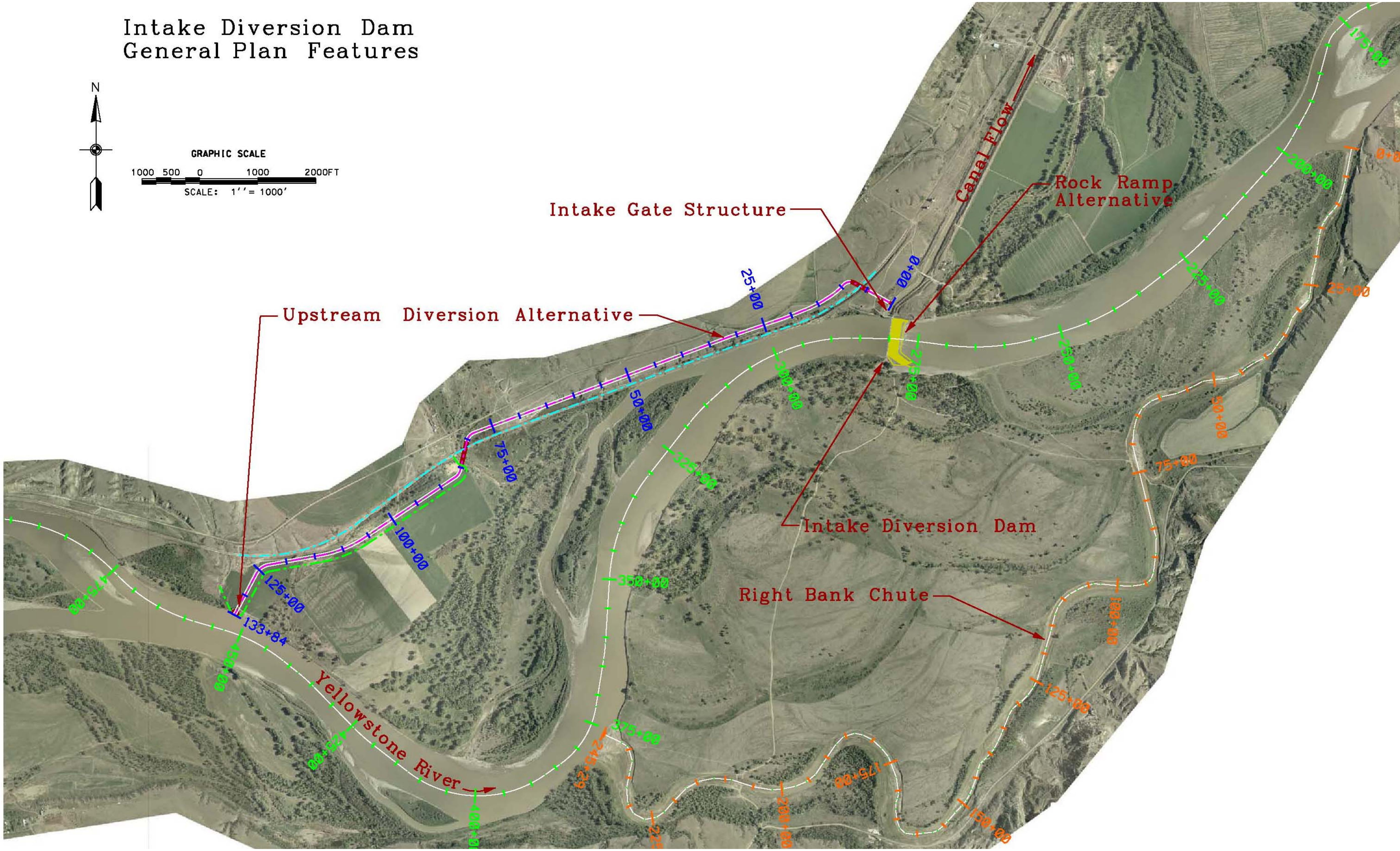
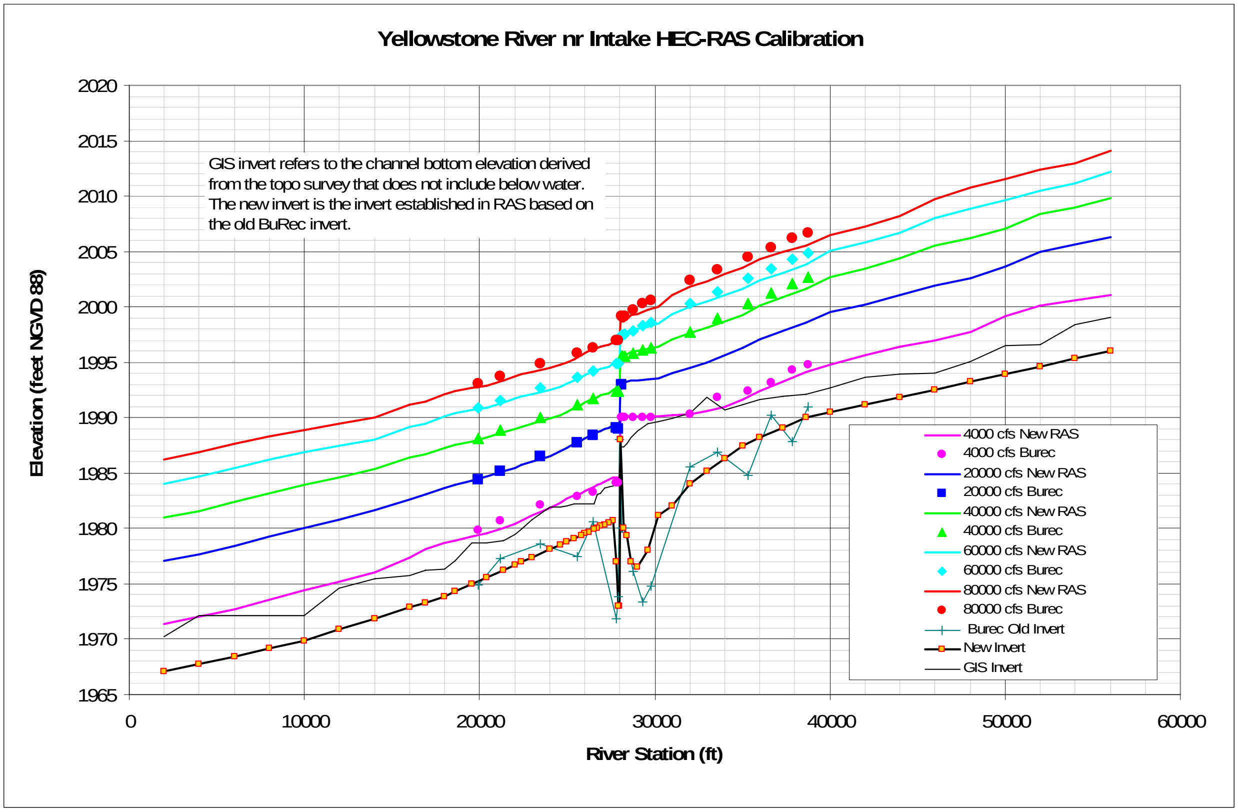


Plate C- 1. Intake Diversion Dam General Plan Features.



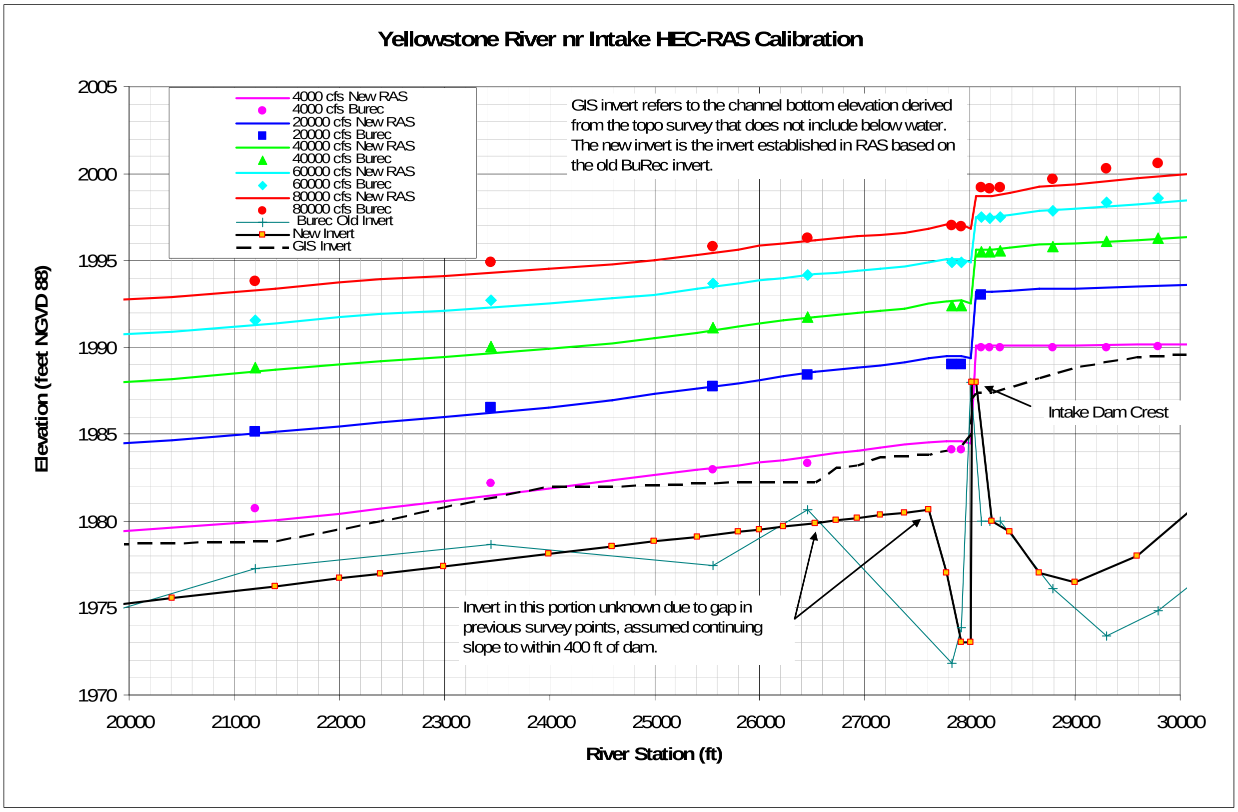


Plate C- 2. HEC-RAS Calibration Computed Profiles.

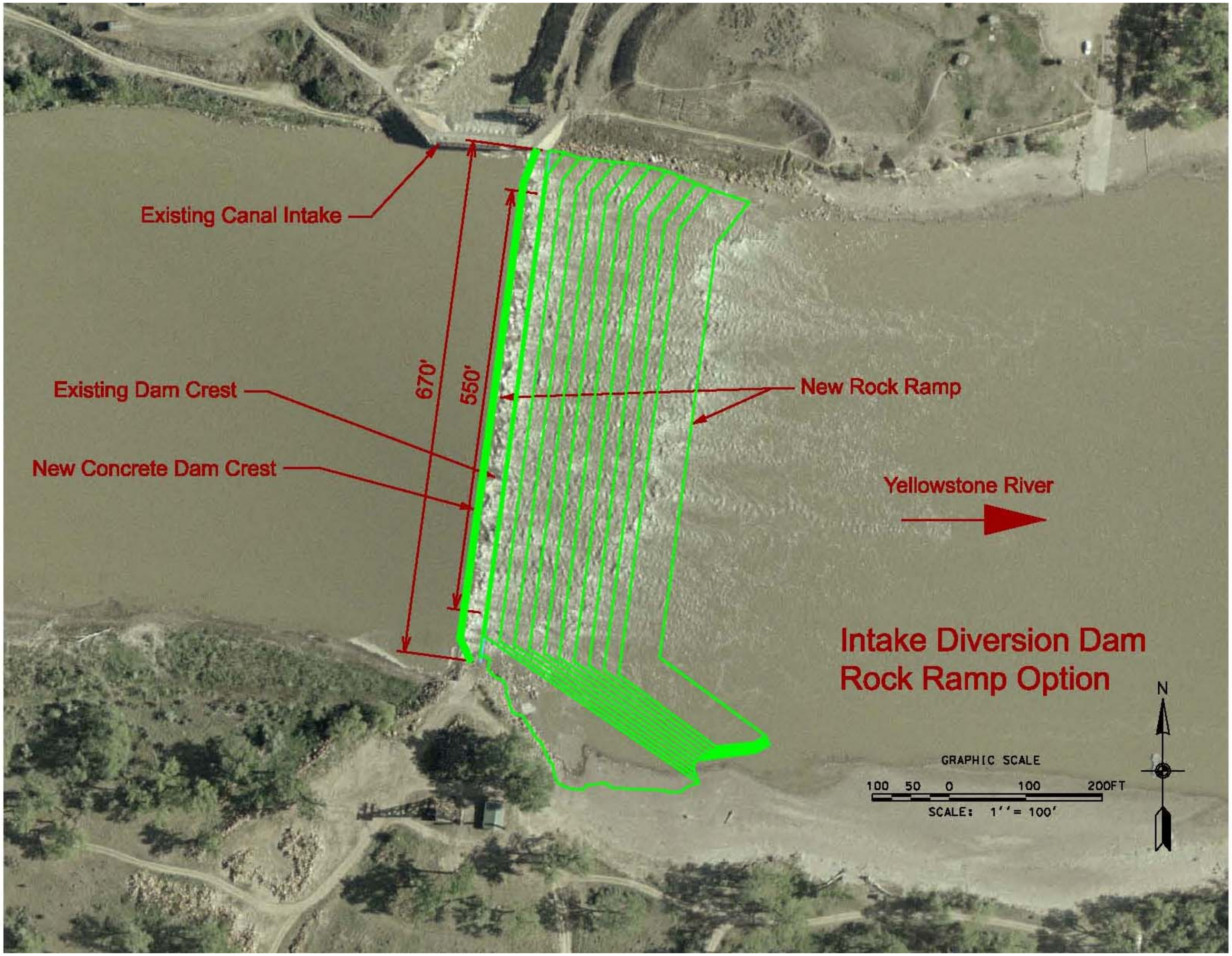


Plate C- 3. Conceptual View Intake Diversion Dam Rock Ramp Option.

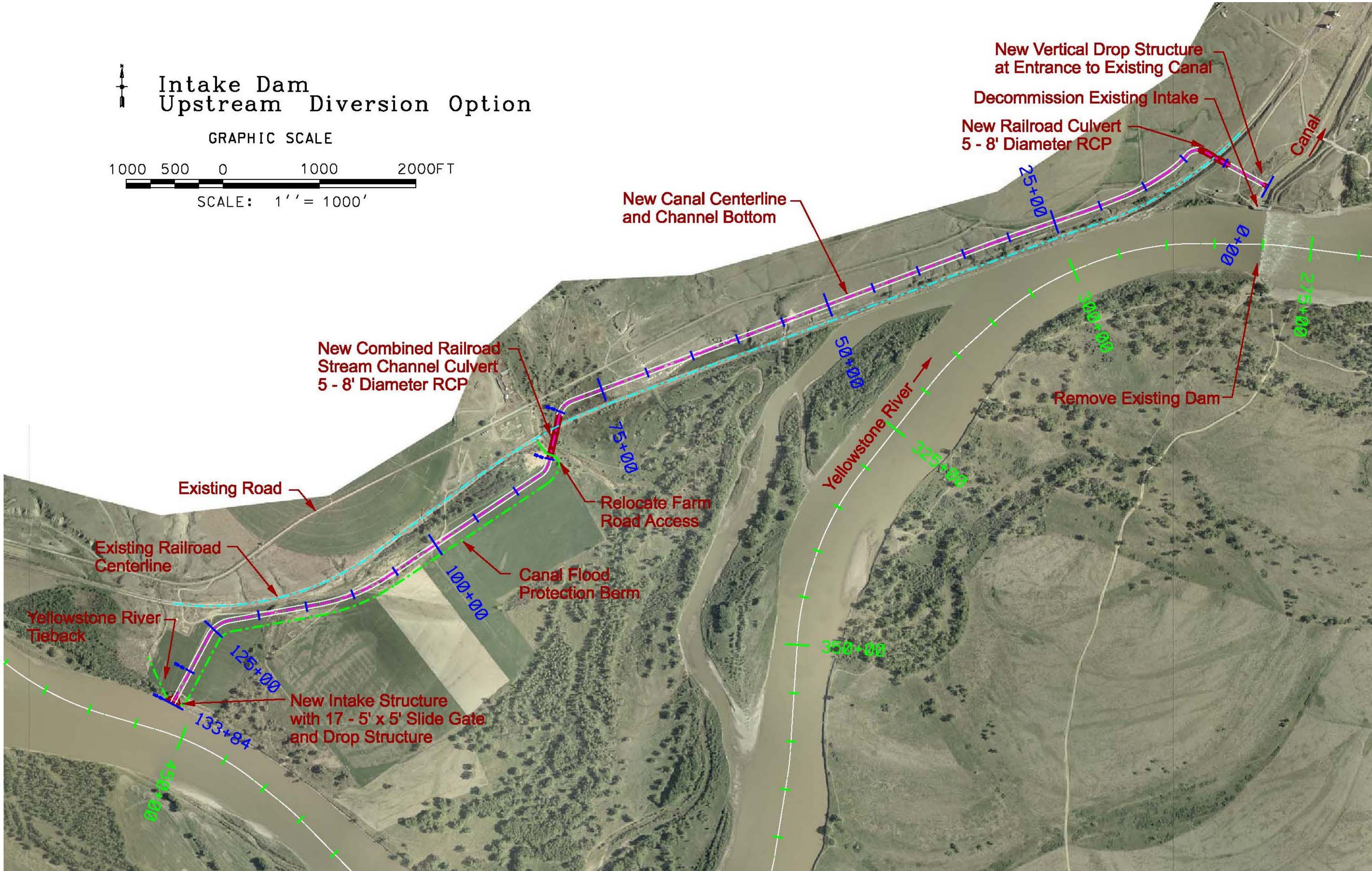


Plate C- 4. Upstream Diversion Option Plan View

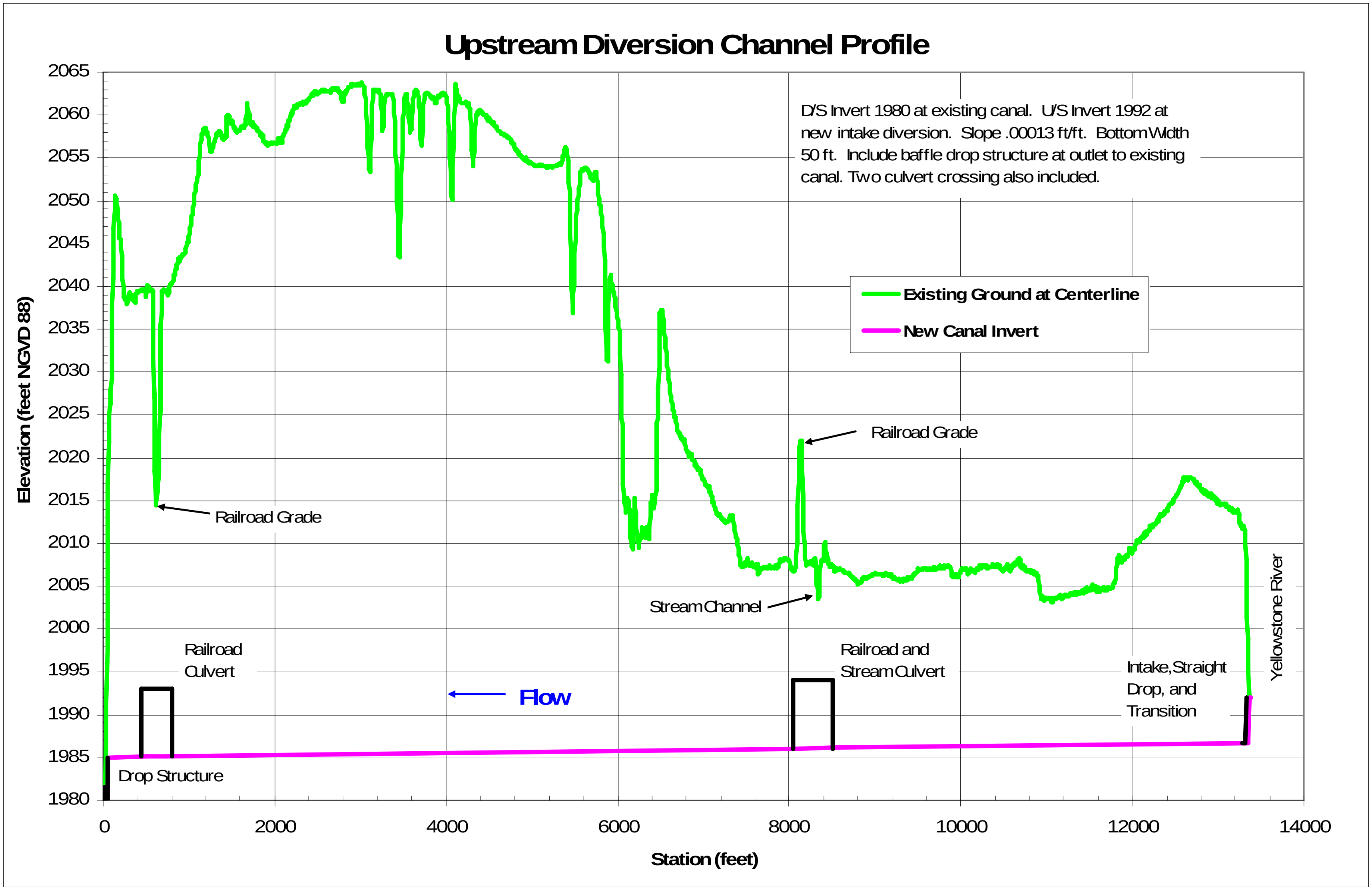


Plate C- 5. Upstream Diversion Channel Profile

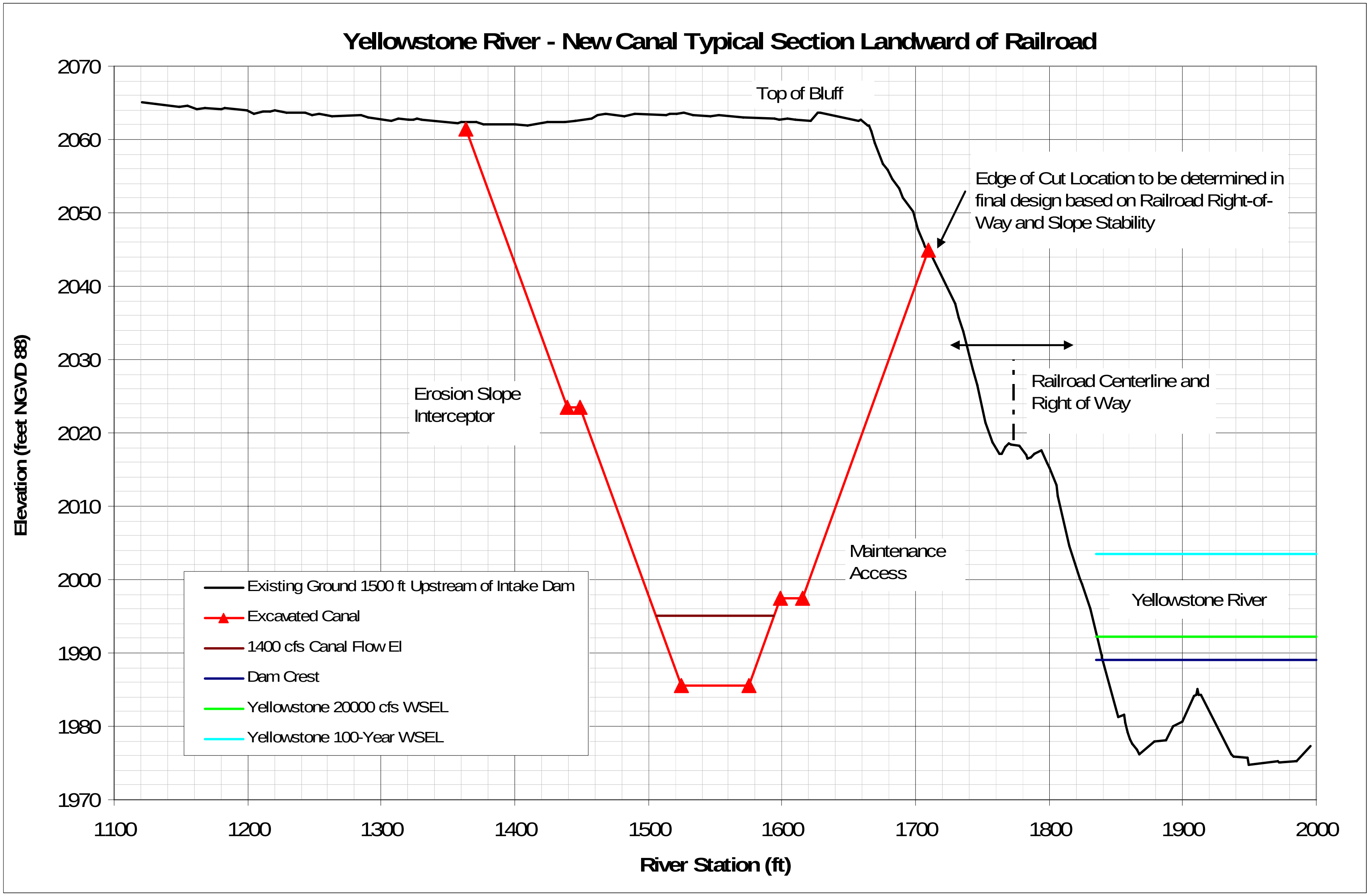


Plate C- 6. Typical Canal Section Landward of Railroad.

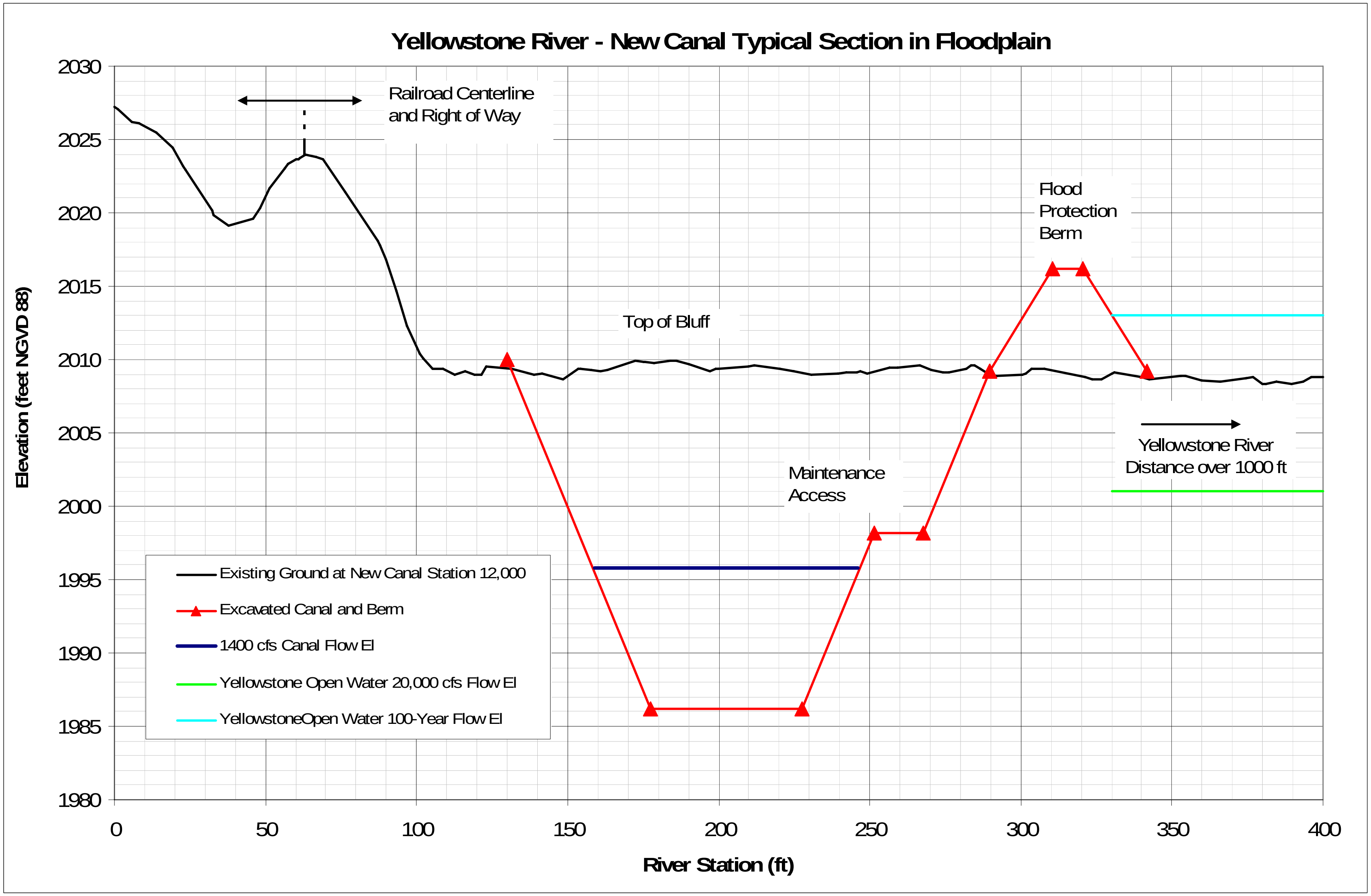


Plate C- 7. Typical Canal Section in Floodplain.

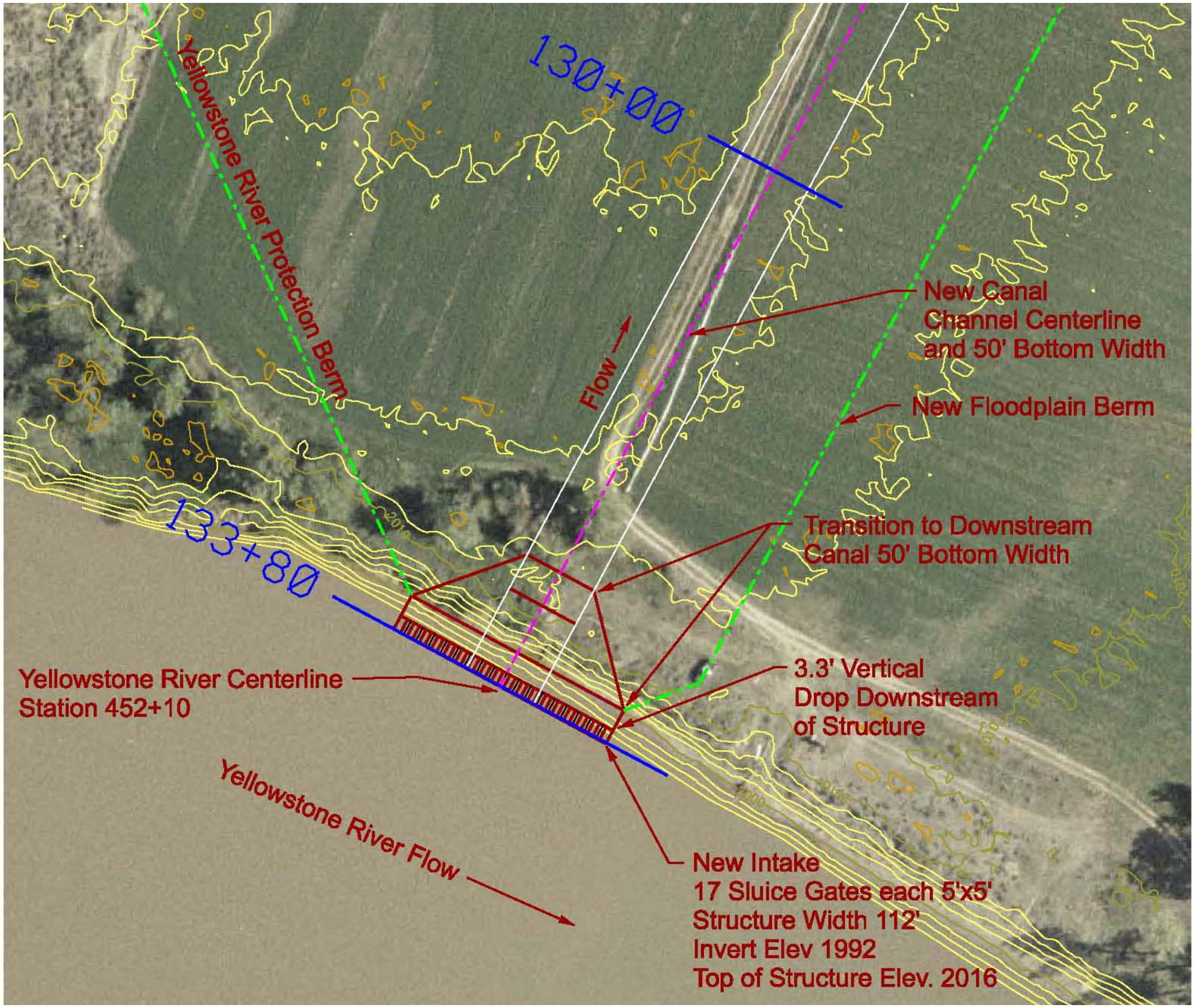


Plate C- 8. New Intake Structure on Yellowstone River

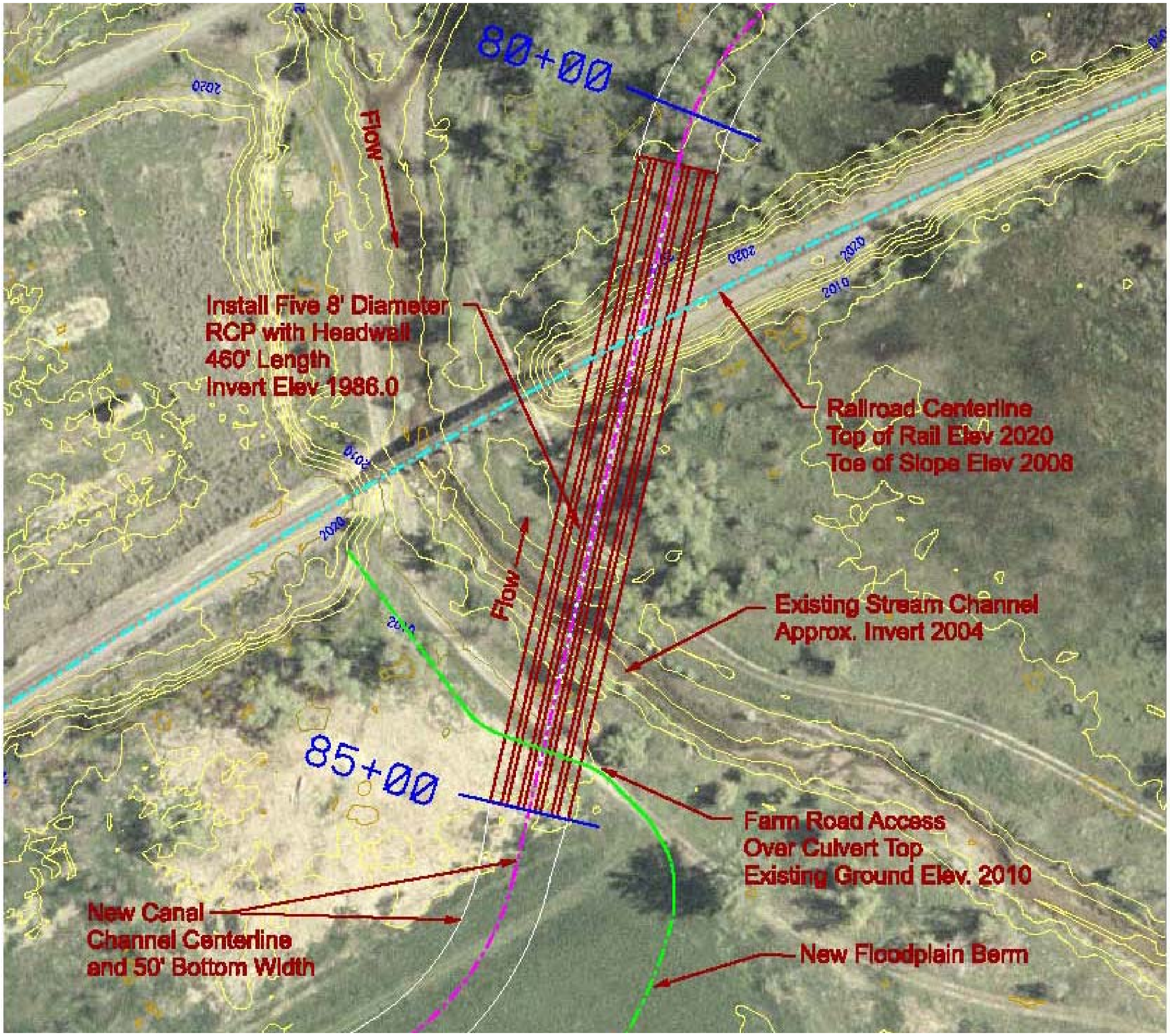


Plate C- 9. New Combined Stream and Railroad Culvert Crossing, Station 82+70.

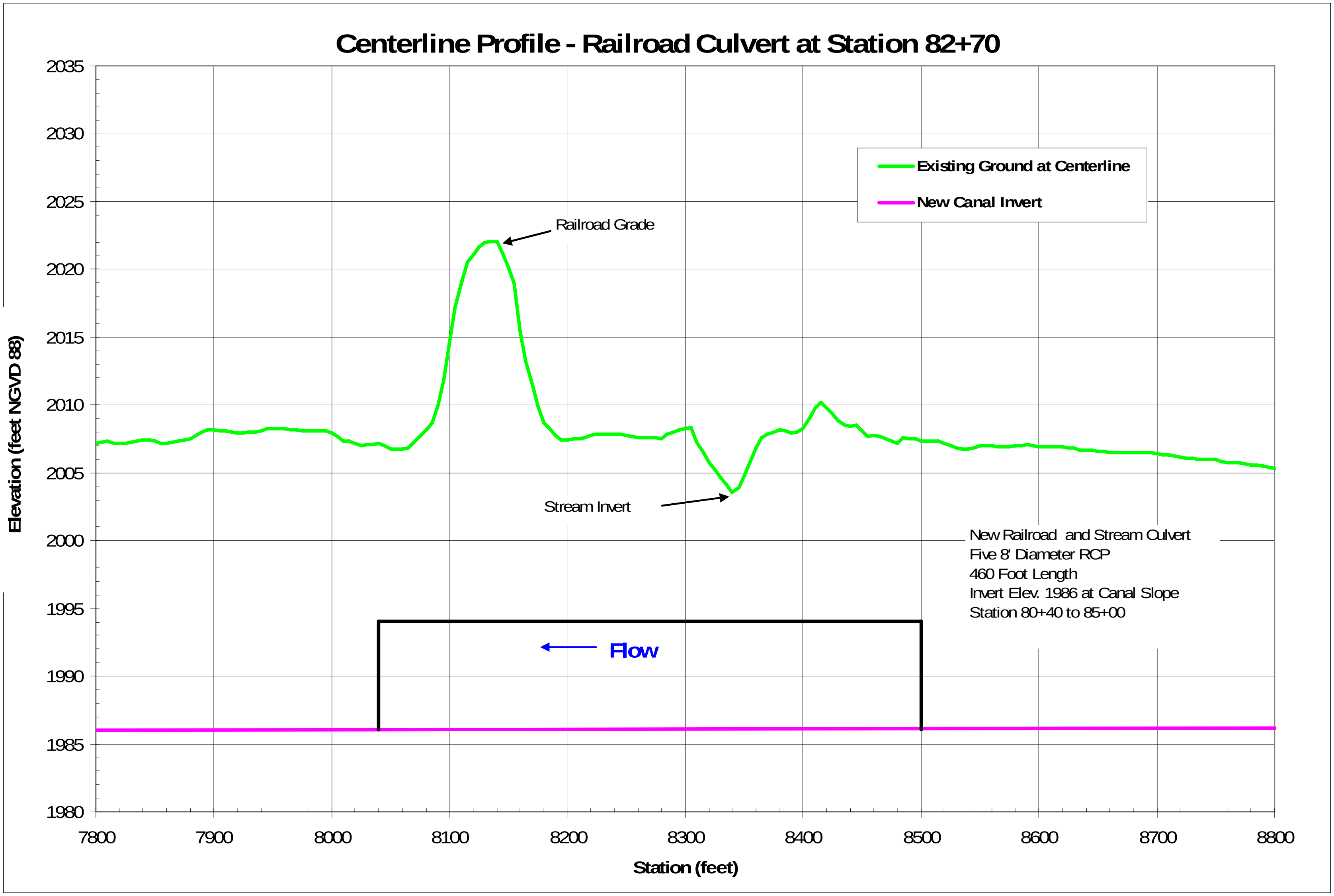


Plate C- 10. Centerline Profile – Railroad Culvert at Station 82+70

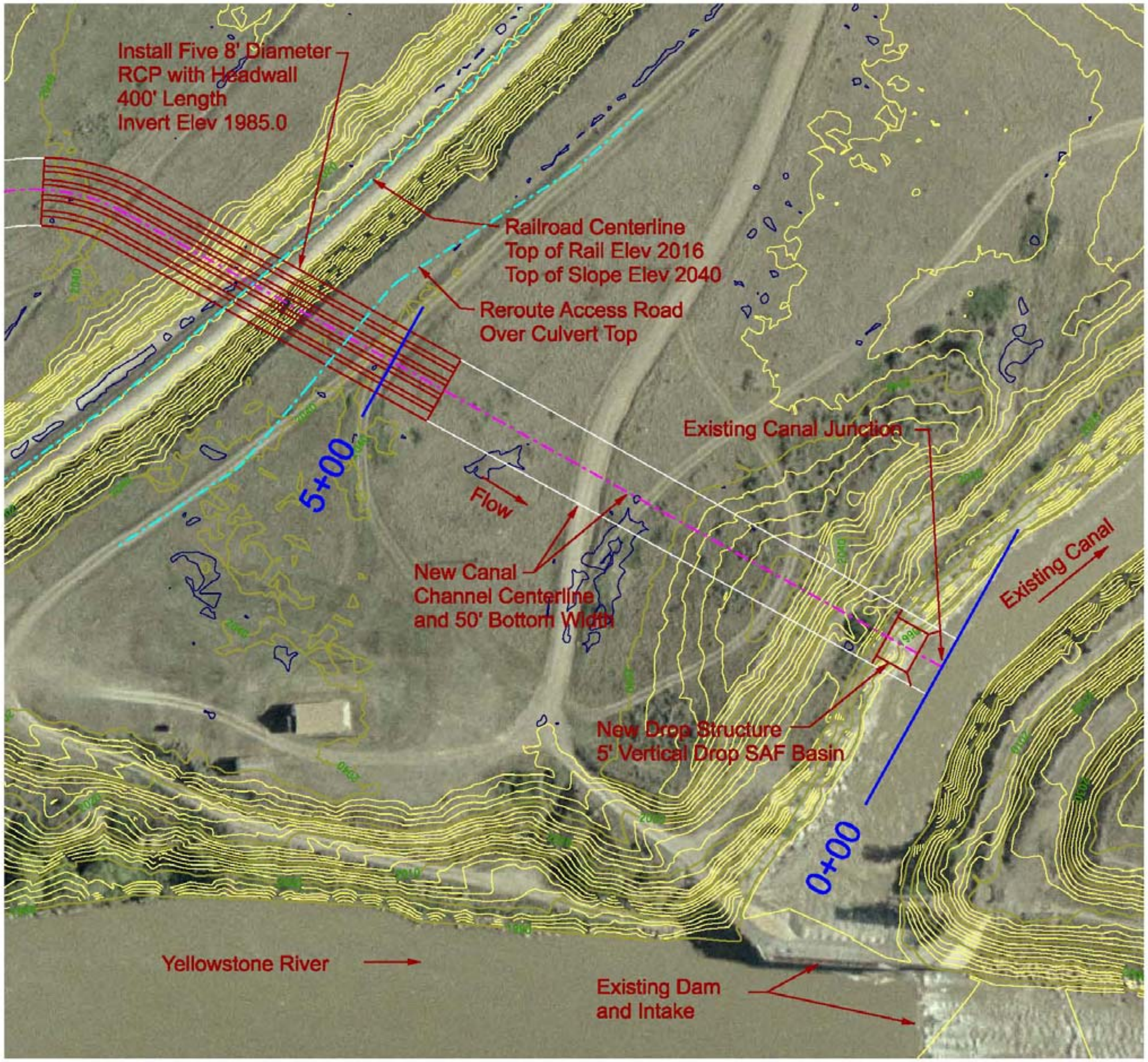


Plate C- 11. New Railroad Culvert Crossing, Station 6+30.

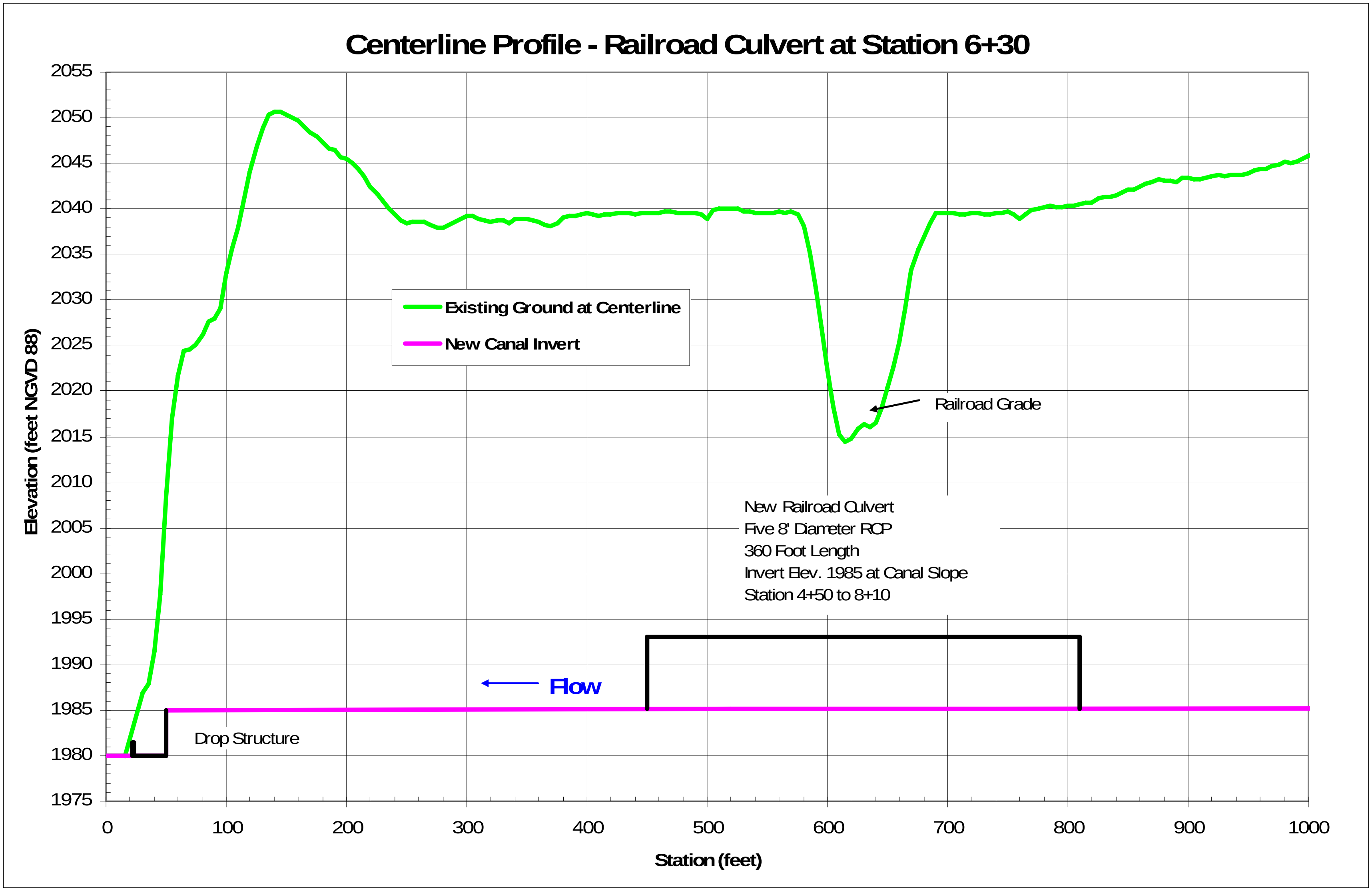


Plate C- 12. Centerline Profile – Railroad Culvert at Station 6+30

Appendix D
Ice Jams and Forces

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

US Army Corps
of Engineers ®
Omaha District

15 May 2006

Ice forces on Intake Dam, lower Yellowstone River: Ten percent design

Robert Haehnel
Andrew Tuthill
US Army Corps of Engineers
Engineer Research and Development Center / Cold Regions Research and Engineering Laboratory
72 Lyme Road, Hanover, NH 03755

Introduction

The Intake Dam on the Lower Yellowstone River, Montana (located approximately 17 miles downstream from Glendive) is regularly subjected to impacts by ice floes. These impacts remove rocks from the dam and carry the rocks downstream. Regular maintenance of the dam is required to replace the lost rock. This dam also is an obstruction in the river, preventing passage of sturgeon by this site. It is proposed that this dam be modified to allow fish passage by putting a long gradually sloped rock ramp on the downstream side of the dam. These rock ramps are starting to be widely used since they closely replicate naturally occurring pools and riffles. The downstream slope of the ramp is gradually sloped (e.g. 1:10 to 1:30) to allow fish passage, and strewn with boulders to provide shelter and resting places for migrating fish.

Based on an initial literature review we found that despite the increased use of rock ramp structures it appears there has been limited consideration of the effects of ice on them. Calkins et al. (1989) studied the ice effects on installed habitat enhancement structures on the Bingo Brook in Vermont. Part of the restoration project included boulders, up to 2.5 ft in size, placed on the streambed to provide shelter for fish. Due to mild break-up conditions during the period of study no conclusive findings could be drawn. Yet, the potential damage can be extensive. For example, Doyle (1988) reported that riprap (D_{50}=1.6 to 2 ft, maximum size = 2.5 to 3 ft) that was placed on seven river bends on the Nicola River in British Columbia, Canada, was partially or completely removed after a 1984 ice run. Eyewitness accounts indicate that this erosion was the result of ice floes impinging on the embankments during the ice run.

Even in cases where ice effects are considered in advance (e.g. Lower Churchill River, Manitoba, Canada; Korbaylo and Shumilak 1999) there has been little design guidance available to address the anticipated ice problems. As a result, the approach used by Korbaylo and Shumilak (1999) was to accept that annual damage would necessitate regular maintenance, so stockpiles of replacement rock were placed on either shore of the structure for future use.

Omaha District has requested the help of ERDC/CRREL (Engineer Research and Development Center/Cold Regions Research and Engineering Laboratory) to provide engineering input on ice forces for the Intake Dam and proposed rock ramp, as well as recommendations for making the dam and ramp resistant to seasonal damage due to ice floes impacting the structure. This report provides an initial assessment of the ice effects on the Intake Dam structure. Estimates of the rock size needed to prevent ice damage to the dam are provided. Estimates of the ice forces acting on the dam and boulders resting on the rock ramp are also provided. Suggested measures to reduce ice damage to the structure are given. As part of this initial effort a literature search on ice effects on riverine rock structures was initiated. Initial findings from that search are included in this report and Bibliography.

Lower Yellowstone Ice Regime and Ice Events

The lower Yellowstone River experiences severe ice events on a regular basis. These include freeze-up ice jams, dynamic breakup ice jams, and ice jam floods. In terms of damage to property and structures, the breakup ice runs and ice jams are the most important and are therefore the focus of this section.

Between Billings and Sidney, the Yellowstone channel is typically wide and shallow with relatively high water velocities, conditions favoring rapid heat loss, and the formation of thick frazil ice covers. Table 1 shows important characteristics of the river that relate to ice and ice jams. These include average bed slope and long-term average discharge at USGS Gage locations. Figure 1 plots maximum accumulated freezing degree-days, *maxAFDD*, for about the last 55 winters at locations along the lower Yellowstone. The *maxAFDD* is much like heating degrees-days used to determine heating requirements, however, in this case the number of degree-days below zero are accumulated to give an indication of the maximum ice thickness h using the formula (USACE 2002):

$$h = \alpha\sqrt{maxAFDD} \qquad (1)$$

Although eq. (1) typically applies to thermal ice growth rather than ice cover formation due to accumulated frazil, it still provides a basis for ice thickness comparison from site to site. Using a coefficient of $\alpha = 0.4$, average ice thicknesses are estimated. The data indicate that even the average winters in the lower Yellowstone Basin are sufficiently cold to produce the ice volumes needed to cause severe breakup ice events.

Table 1. Summary data for the lower Yellowstone River from Billings to Sydney, MT.

	Distance (miles)	Average Channel Slope	Long-term Average Discharge (cfs)	Average Net-Maximum AFDD (F)	Calculated Ice Thickness (inches)
Billings			7940	764	11.6
	145	0.0010			
Miles City			12,600	1344	14.7
	78	0.0007			
Glendive			16,800	1688	16.4
	119	0.00016			
Sydney			12,100	1769	16.8

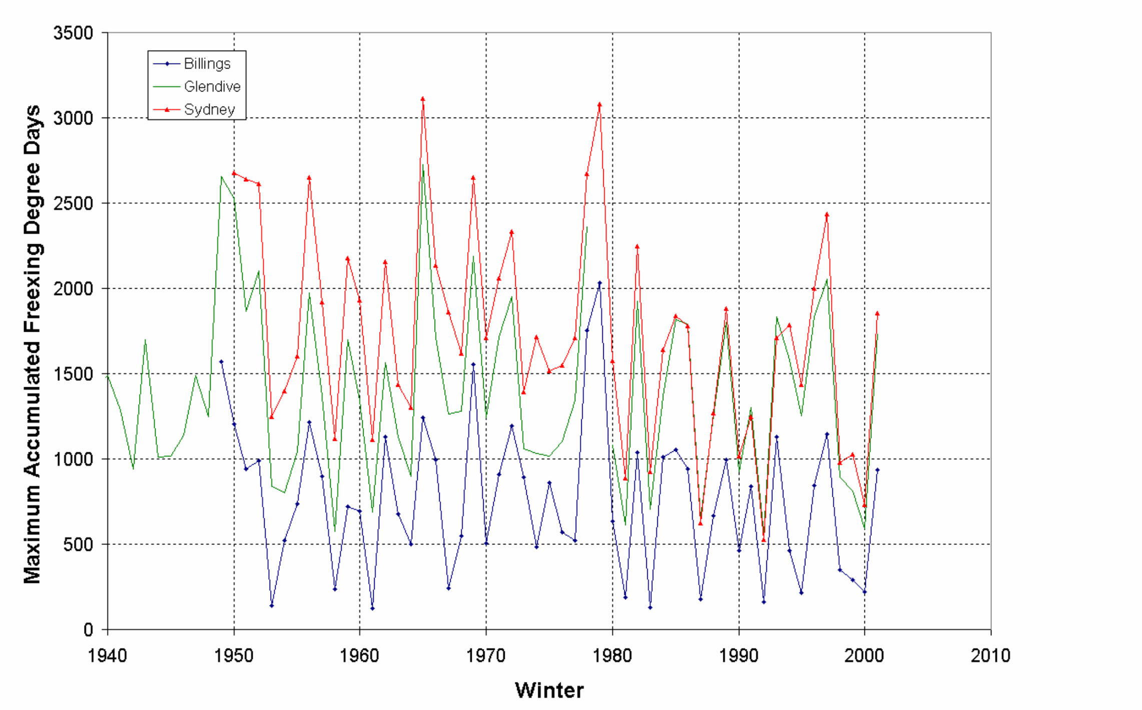


Figure 1. Maximum accumulated freezing degree-days at locations along the lower Yellowstone.

Several other factors contribute to the severity of breakup ice events on the lower Yellowstone. The first is the gradual decrease in channel slope, in the downstream direction, which reduces ice conveyance capacity while increasing the likelihood of ice jam formation.

The second factor is the transition from warmer to colder air temperatures as one moves in the downstream direction, shown by the increasing *maxAFDD* values shown in Table 1 and Figure 1. The increase in *maxAFDD* in the downstream direction means that the breakup front encounters thicker stronger ice as it moves downstream. As a result, breakup runs, that are triggered by Chinook-related warming in the mountainous headwater regions to the southwest often stall as they encounter the colder conditions and thicker ice covers on the lower Yellowstone. The tributaries to the south such as the Bighorn, Tongue and the Power Rivers also release their ice before the Yellowstone, triggering ice runs and ice jams in the confluence areas and downstream.

A common scenario is for a tributary, or faster-flowing section of the mainstem river, to release its ice that then jams at a downstream obstruction such as a bend, an island, or simply a section of river with a stronger ice cover. The jam serves as a temporary dam, backing up flow and pushing ice floes and rubble out into the floodplain areas. Once sufficient head develops, the ice jam often fails, sending a surge of ice and water downstream to jam at the next location. Often much of the floodplain ice remains behind in the form of shear walls, and some of these have been observed to be greater than 20 ft in height on the lower Yellowstone River.

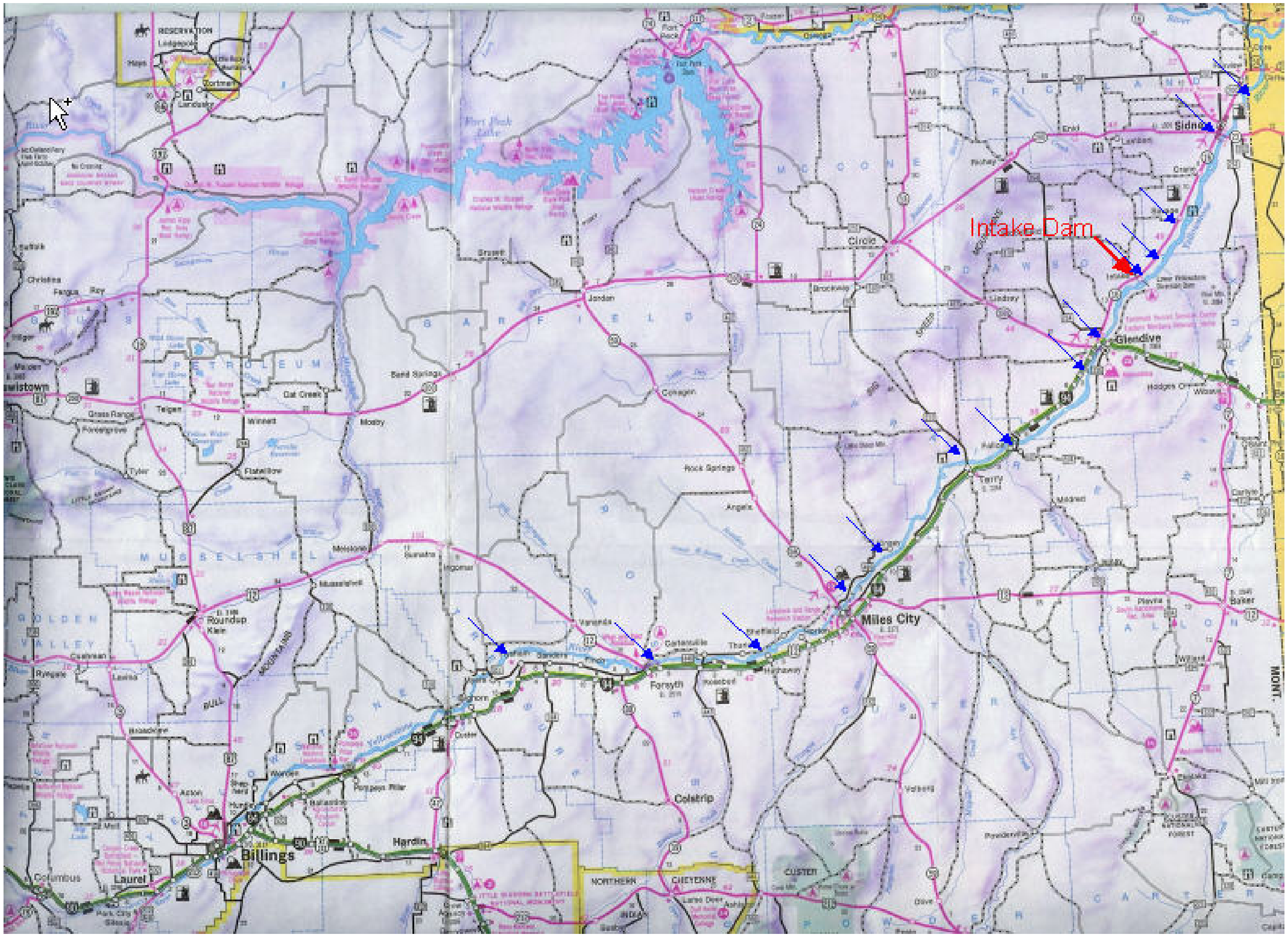


Figure 2. Map of lower Yellowstone River showing common ice jam locations (blue arrows).

The lower Yellowstone has a long history of ice jams and ice jam floods. Table 2, compiled from the CRREL Ice Jam Data Base (IJDB) (http://www.crrel.usace.army.mil/ierd/ijdb/) and a 1997 trip report by Tuthill, lists historic breakup ice jam locations and dates and Figure 2 shows a map with ice jam locations. Based on the winter of 1996, which contains the greatest number of IJDB entries, the downstream progression of the ice breakup from Forsyth to Sidney takes about 6 days. Figure 3 shows a similar lag in the discharge hydrographs for Miles City and Sydney; accumulated freezing degree-day curves are included to illustrate the increasing coldness in the downstream direction.

Not surprisingly, the greatest numbers of ice jams are reported at the most densely settled locations of Miles City, Glendive and Sydney. Jams that occur above and below these locations are of concern since the failure of an upstream jam can release an unexpected surge of ice and water on people and property, similar to a dam-break. Though not quite as drastic, jams that form downstream of settled areas could quickly back up flow and flood people out.

At Miles City, 7 of the 8 major floods since 1882 resulted from ice jams. A 24,000-ft-long dike that protects 2000 acres of land in Miles City has suffered damage during many of these ice events, as well as the large open water flood in 1997. Typically the Tongue River releases its ice into the Yellowstone, which then jams 2 miles below Miles City at Buffalo Rapids. When this jam fails the ice from it is usually added to a larger jam that forms nearly every year 10 miles downstream at the Tusler Bridge at Kinsley. On average, the Tusler Bridge jam extends upstream to within 3 miles of Miles City and may remain in place for weeks. Upstream ice jams at Hathaway, Forsyth and Hysham are a concern to emergency responders, as they might release and add to the jams at Miles City. Over the years, efforts have been made to break the jams, the most colorful being the use of large bombers in March of 1944.

Table 2. Historic ice jam locations and dates, lower Yellowstone River.

Location	*Dates*
Hysham	2003
Forsyth	2/71996
Hathaway	2/7/1996 1998
Miles City	1881 1882 1897 1929 1943 1944 1971 1979 2/8/1996
Kinsley	1944 1971 1979 1996
Fallon	2/9/1996
Cedar Creek	1899 1994 1996
Glendive	1894 1889 1899 1936 1943 1959 1962 1969 1994 2/11/1996 1998 2003
Intake	1994
Richland Co. Line	1994 2003
Elk Island	2/16/1996
Savage	1943 2/13/1996
Sydney	1943 1950 1969 1994 2/13/1996 2003
Fairview	1943 2/12/1996

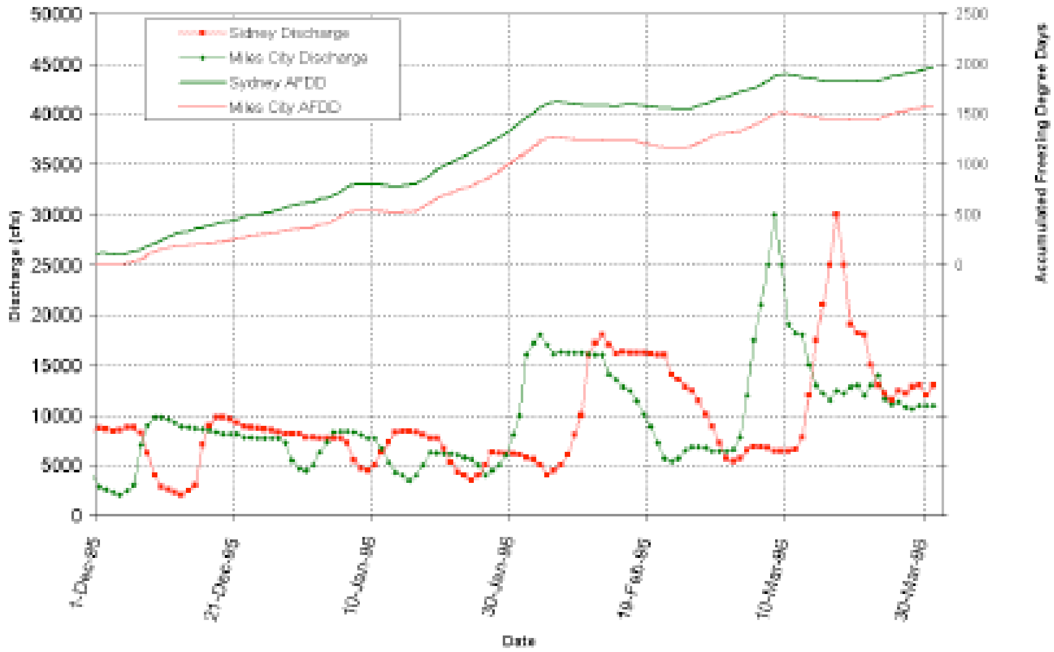


Figure 3. Discharge and accumulated freezing degree-day data for the Yellowstone River at Miles City and Sydney, MT.

The ice jams at Glendive have resulted in fatalities on several occasions. For example, in 1894, three men died at Glendive trying to escape ice jam floodwaters and in 1912, an ice jam flood took the lives of the Sullivan family in their farmhouse on the northwest side of the river. A number of factors contribute to Glendive's reputation for the worst ice jam floods on the Yellowstone. First is the downstream-progressing nature of breakup on the Yellowstone. As the upstream jams fail, adding their ice to the downstream one, the jams tend to increase in size, in spite of significant ice losses to deposition along the banks and ice melting.

River morphology and structures cause the ice to stop at Glendive. Just upstream of the city is a sharp bend and a railroad bridge whose piers are askew to the present flow direction of the river (the Yellowstone experiences much channel shifting). Jams at this location brought floodwaters to within 0.5 ft of the crest of the west Glendive Dike in 1969 and 1994, at an estimated rate of rise of 6 ft per hour. Downstream of the railroad Bridge, two additional bridges impede ice passage past the city. Jams also occur downstream of the Interstate 90 Bridge in a flat-lying reach with a mid-channel island.

The Omaha District of the Corps of Engineers completed a study in 2002 that recommended construction of a flow relief channel to alleviate the recurring ice jam flooding at Glendive.

Between Glendive and Sidney, ice jams and ice jam flooding have been reported at Intake, the Richmond County line area, Elk Island and Savage. Of the 33 major floods

Between Glendive and Sidney, ice jams and ice jam flooding have been reported at Intake, the Richmond County line area, Elk Island and Savage. Of the 33 major floods that have occurred at Sidney between 1882 and 1960, 16 have reportedly involved ice jams. On average these jams occur about 12-15 hours after the release of the jams at Glendive, provided the ice run does not stall at in-between locations. An important observation is that the Intake Dam must experience significant ice runs on a fairly regular basis. Based on inspection of the river channel geometry, one would expect the ice from Glendive to stall in the bends and islands above Intake, then release to pass over the dam and stall again in the bends and islands in the county line area, and so on, past Savage and Elk Island downstream to Sidney.

Ice jams in the vicinity of Sidney and Fairview result in part from the proximity of the Missouri River confluence about 30 river miles downstream. This section of the Missouri, which lies just upstream of Lake Sakakawea, typically retains its ice several weeks longer than the lower Yellowstone.

Ice related damage to the Intake Dam

Presently the Intake Dam is an 8 to 10-ft-high, 500-ft-wide, rock-capped timber structure. The upstream slope is 1V:2H; the downstream slope is 1V:10H. The crest is 15 feet wide (Omaha District, 2002). We consider three possible modes of ice related damage to the Intake Dam:

1. Removal of rock from the dam
2. Damage to the dam structure due to ice forces
3. Removal of boulders from the proposed rock ramp structure

These are discussed below.

Removal of rock from the dam - The effects of ice on riprap or rock structures was studied by Sodhi et al. (1996), Sodhi et al. (1997) and Sodhi and Donnelly (1999). In these works model test were performed to determine the conditions leading to failure of riprap embankments under the influence of ice floes striking the riparian protection. Their findings show that to prevent failure of a riprap embankment the mean rock size (D_{50}) needs to be 2-3 times the ice thickness.

Thus, to determine the size of rock needed to prevent erosion of the Intake Dam by ice an estimate of the ice thickness is required. This was done using eq. (1) ($\alpha = 0.4$) and historical temperature data from Glendive, MT, which is 17 miles upstream of Intake Dam. This is summarized in Table 3.

From this analysis we find the mean ice thickness is about 16 in., the maximum ice thickness is about 21 in. and the minimum ice thickness is approximately 10 in. These values are typical for the northern tier of the United States. Also, summarized in Table 1 is the maximum D_{50} rock size required to resist ice damage. The rock size required to resist damage by the maximum estimated ice thickness is $D_{50} \approx 6'$. To resist damage by the minimum ice thickness, the rock size would still need to be at least 2.5'.

Table 3. Summary statistics for AFDD and ice thickness at Glendive, MT for the period of record from 1894-2001. Based on the estimated ice thickness, a maximum rock size required to prevent damage the Intake Dam is estimated. The ice thickness is computed using eq. (1), and the maximum rock size required to resist damage due to ice is 3 times the ice thickness (Sodhi and Donnelly 1999).

	AFDD (°F-Days)	Ice Thickness (in)	Maximum D_{50} rock size (in)
Maximum	2859	21	64
90 Percentile	2500	20	60
75 Percentile	2054	18	54
Average	1688	16	48
25 Percentile	1277	14	42
10 Percentile	936.5	12	36
Minimum	733.5	10	30

At this point it is not certain what D_{50} is for the rock on the Intake Dam. However, the cableway used to place rock on the structure can only support placement of rock that is about 1 cubic yard in size (about 3' square)[1]. Comparing this with Table 1 we find that rock of this size is capable of resisting damage by ice thickness corresponding to the 10th percentile and below. This comparison helps explain the regular maintenance required on the Intake Dam to replace rock that is removed almost annually due to the spring ice run. The Bureau of Reclamation (2005) indicates that 300 to 1200 cu. yds. of large quarried rock are regularly place on the Intake Dam during these maintenance efforts.

To prevent removal of rock from the structure several approaches can be taken. The first is to place larger rock on the dam during the regular maintenance. From Table 1 it appears rock as large as 4 – 5' would be required to protect the Intake Dam against damage by the estimated average or maximum ice thickness seen at this site. Though this size rock may seem excessive, Doyle (1988, 1992) reports during an ice runs on the Nicola River even boulders as large as 1.2 to 2 m (4 to 6.5 ft) were removed from revetment structures and the stream bed and transported tens of meters downstream.

The advantage to this approach is the rock need only be dumped on the structure. No care is required to key in the rock. However, unless the cable way is upgraded or an alternative means of delivering rock to the site is provided, this may not be feasible.

A second approach is to make the structure as smooth as possible to promote the ice riding up and over the structure, minimizing interaction of ice with individual rocks that protrude above the surrounding grade. This can be done with "smoothly graded size distributions that are well keyed-in, or … even hand placed" (Wuebben 1995). Guidance

[1] Email communication with Dan Pridal, Omaha District Corps of Engineers, 26 April 2006.

for the D_{50} rock size needed for this approach is not available. However, it appears that this approach is already being used at Intake Dam with some success reported[2].

A third approach is to cap the dam structure with concrete or pre-assembled concrete mats. This relatively smooth surface will also allow the ice to ride-up and over the structure. The principle concern with this approach is damage to the concrete due to the forces associated with ice impacting the structure. The next section addresses this issue.

Ice forces on the dam structure – Direct impact of ice flows on the dam structure can damage either the concrete or armor stone face. Thus, the structure needs to be designed to withstand these forces. AASHTO (1998) provides guidance on estimating the forces of ice floes striking a bridge pier or other solid structure. Either momentum of the floe or failure of the ice limits the maximum ice force applied to the structure. AASHTO (1998) accounts for both of these aspects. First the ice force associated with ice failure is computed using

$$F_c = \left(\frac{5h}{w} + 1\right)^{0.5} phw \qquad (2)$$

$$F_b = \left(\frac{0.5}{\tan(\phi - 15)}\right) ph^2 \qquad \text{for } \phi > 15^\circ \qquad (3)$$

where F_c is the force exerted on the structure when the ice fails in crushing and F_b is the force if the ice fails in bending. Also, w is the length of the ice-structure line of interaction; if the structure is narrow (e.g. a pier), w is the width of the structure. If the structure is wide, w is the approximate diameter of the floe. p is the effective pressure the ice can exert on the structure (an indication of the ice strength) and ϕ is the angle between the structure face and vertical.

The lesser of the two forces, F_c or F_b, is the design force for the structure. If $\phi < 15°$ then F_b is not computed and it is assumed that the ice only fails in crushing so F_c is used.

If an ice floe is small the momentum of the floe is not sufficient to cause the ice to fail on impact. In this case it is the momentum of the floe that determines the impact force. AASHTO (1998) accounts for this by applying a load reduction factor, K_t, to the design load computed from either eqs. (2) or (3) above. The load reduction factor, as shown in Table 4, is a function of A/h^2, where A is the plan area of the floe. AASHTO (1998) stipulates that K_t of 0.5 is the minimum value that can be used.

The Yellowstone River near the Intake Dam is approximately 600 ft wide. A reasonable maximum floe size is 2/3 the river width or 400 ft. From Table 3 we find the maximum ice thickness is about 21 in. A typical value of p for a springtime ice run is about 110 psi; the slope of the upstream face of the structure is $\phi = \tan^{-1}(2/1) = 63°$. Using these as input values for eqs. (2) and (3) we find

$F_c = 9500$ kips

[2] Email communication with Dan Pridal, Omaha District Corps of Engineers, 27 April 2006.

F_b = 22.5 kips

For a floe of this size $A/h^2 > 1000$, thus $K_t = 1.0$ and the design load that the structure needs to withstand is 22.5 kips.

Table 4. Load reduction factors to account for small floes (AASHTO 1998).

A/h^2	Load reduction factor, K_t
1000	1.0
500	0.9
200	0.7
100	0.6
50	0.5

Removal of boulders from the rock ramp - The proposed rock ramp structure for allowing fish passage over the Intake Dam requires lengthening the downstream slope to support a shallower grade of 2 – 5%. Also along the grade boulders, approximately 5' in size would be placed to provide pools and shelter for the fish. Figure 4 shows a conceptual sketch of what the downstream ramp would look like.

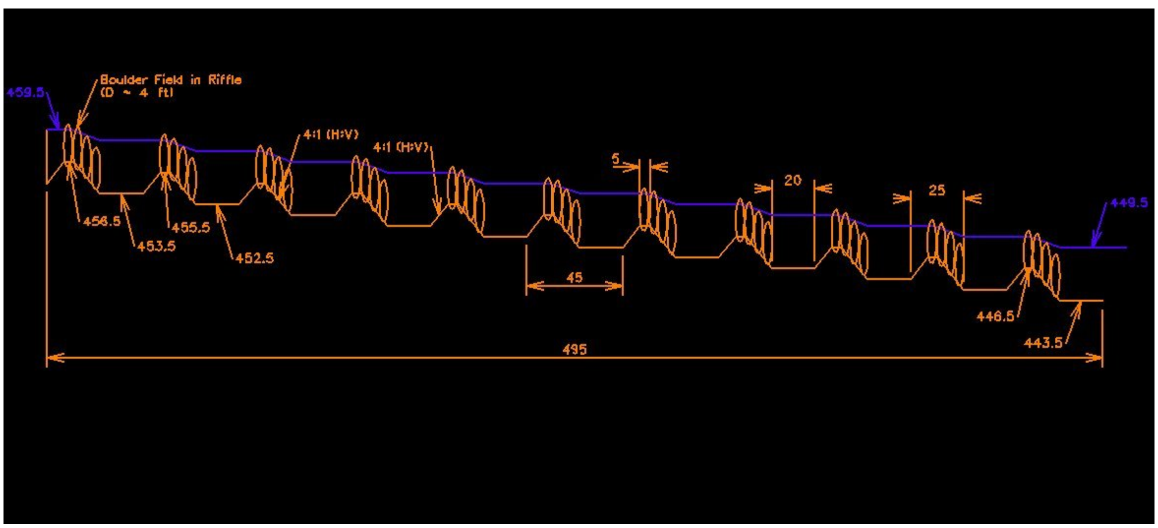


Figure 4. Conceptual drawing of down-stream rock ramp proposed for the Intake Dam. This drawing is the actual design proposed for Lock and Dam 22, St. Paul District, U.S. Army Corps of Engineers.

Once ice clears the top of the weir it can interact with the protruding boulders on the ramp. Though Sodhi and Donnelly (1999) indicate that 5' rock is big enough to resist damage by ice when it is used as bank protection, the boulders shown in Figure 1 are resting on grade, as isolated features, rather then lying in the bed. As such the boulders are exposed to an overturning moment as they are hit by the oncoming ice floes, rather than being pushed up out of the bed as the ice interacts with the bed surface. As a result the design guidance put forward by Sodhi and Donnelly (1999) does not apply, and a different approach is required.

To analyze this situation we consider a single boulder resting on a flat bed (Fig. 5), and determine the minimum force required to start the boulder in motion. For simplicity, in this initial study we consider the boulder to be a 5' cube. The point at which the boulder just starts to rotate about point O, is when the $\Sigma M_O = 0$, where M_O are the moments about point zero. The forces acting on the boulder are the gravitational force, *mg*, and the force of the floe, *F*; *m* is the mass of the stone and *g* is the gravitational constant. The moment arms for each of these forces are indicated in Figure 2.

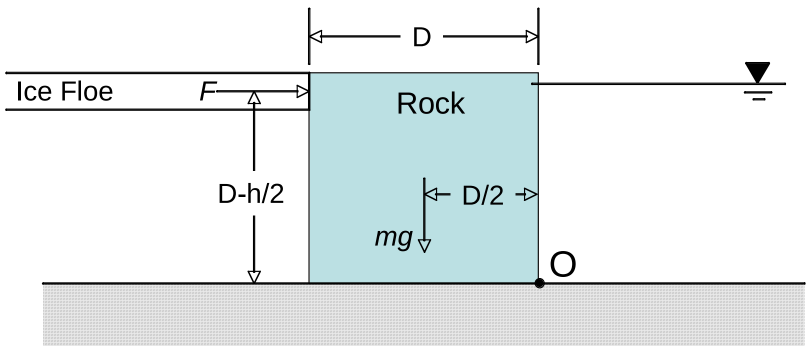


Figure 5. Force diagram of a cubic boulder being pushed by an ice floe.

Assuming the density of the stone is approximately 2500kg/m^3 the mass, *m*, is 8850 kg. Adjusted for buoyancy, the submerged weight of the stone is 51.7kN. The ice force is computed from eq. (2)—the cube has a flat face, so $\phi < 15°$ and F_b does not apply. Using $w = D = 5'$, $h = 20$ in., and $p = 110$psi, $F_c = 240$ kips. However, this value does not yet take into account the load reduction factor. If we assume the ice floes striking the boulders are the same size as the ice floes the hit the front of structure (i.e. 2/3 of the river width or 400 ft), then no load reduction factor is applied. Yet, it is quite often that the floes break into smaller pieces as they bend going over a weir. The size of these pieces is roughly the same size as the characteristic length of the ice sheet (see USACE 2002). For freshwater river ice the characteristic length is approximately 15 to 20 times the thickness of the ice. Given the above data, that corresponds to a floe size of approximately 30 ft in diameter, or $A/h^2 = 113$ and $K_t \approx 0.6$. Applying this gives $F_c = 140$ kips.

Yet from the moment balance

$$F\left(D - \frac{h}{2}\right) = mg\frac{D}{2} \qquad (4)$$

the minimum ice force required to start rolling the boulder is 7.0 kips. The force the ice can exert on a boulder is about 20 times greater than the resisting force of gravity.

In concept one could place the boulders closely together so they share the load of an oncoming floe. Unfortunately, over a distance of 30 ft the best one can hope is for the ice to contact 6 boulders all at the same time (i.e. flat edge of ice striking a line of boulders that are abutted right next to each other with no spacing between them). What is more likely is that the flow would be round or elliptical and simultaneous contact with two or three boulders is more likely. In either scenario it appears that the resistance force of multiple boulders would not be enough to prevent the ice from rolling them down the slope without additional anchoring.

We recommend this be further studied in the 90% design phase so that a better estimate of the ice loads on the protruding boulders can be determined using available laboratory data of ice forces on the proposed Cazenovia Creek ice control structure (ICS) (near Buffalo, NY). Also, methods of anchoring large boulders at the Hardwick, VT ICS may need to be considered in the follow on work.

Conclusions

This initial study shows that the lower Yellowstone River, from Hysham to Fairview, MT regularly experience ice jams. Some of the most severe of these jams occur at Glendive, 17 miles upstream of the Intake Dam. Jams are also reported at Intake, MT and the vicinity immediately upstream and downstream of Intake Dam.

The estimated thickness of the ice at and around Intake Dam is on average about 16 in; the maximum ice thickness is estimated at 21 in. Based on these ice thickness estimates, to prevent ice related damage to the Intake Dam the recommended required D_{50} rock size for capping the structure is about 4-5'. The best estimate of the actual rock size used on the structure is 3' or less, which explains the need for regular replacement of rock on the structure due to ice pushing rocks off the dam weir.

Suggested methods for preventing damage to the weir are
1. Increase the rock size on the structure to D_{50} = 4 to 5'
2. Key the rocks into the face to provide a smooth surface for the ice to ride up and over
3. Cap the structure with poured concrete or concrete mats.

No design guidance is available for preventing ice related damage to structures built using the second method. The latter two of these methods must be designed to withstand ice impact forces estimated at 23 kips.

The design of the downstream rock ramp includes putting boulders that are approximately 5' in size and spaced apart, to provide pools and shelter for the migrating fish. Initial estimates of the ice forces striking these protruding boulders indicate that the oncoming ice will readily roll them down the ramp. Further work is needed to confirm that the estimated ice forces striking boulders is accurate. Methods for anchoring the boulders against these loads need to be explored.

Acknowledgements
The authors would like to thank Carrie Vuyovitch for providing the AFDD data for Glendive, MT.

References and Selected Bibliography

AASHTO (1998) *LRFD Bridge design specifications*. American Association of State Highway and Transportation Officials, Washinton, DC.

Bureau of Reclamation (2005) Draft biological assessment: Future operation of the lower Yellowstone Project with proposed conservation measures, Montana Area Office, U.S. Department of the Interior, Billings Montana.

Calkins, D.J., L.W. Gatto and B.E. Brockett (1989) “Field assessment of fisheries habitat-enhancement structures in the Bingo Brook, Vermont, after the spring 1983 ice run.” Proceedings of the 23rd International Association of Hydraulic Sciences Congress, Ottawa Ontario, 22 August.

Doyle, P.F. (1988) Damage resulting from a sudden river ice breakup. Canadian Journal of Civil Engineering. **15**:4, 609-615.

Doyle, P.F. (1992) Documentation of severe mid-winter 1991 break-up on the Nicola River and its tributaries. British Columbia, Regional Water Management, Kamloops.

Highway (1989) Pre-assembled mats placed between tides. *Highway & Heavy Construction.* **132**:11, 81.

Korbaylo, B. W. and B.E. Shumilak (1999) A case study: Lower Churchill River water level enhancement weir project. River ice management with a changing climate: dealing with extreme events. Proceedings of the 10th workshop on river ice, Winnipeg, Manitoba, Canada, 8-11 June. Canadian Geophysical Union Hydrology Section, 135-149.

Omaha District (2002) Lower Yellowstone River Intake Dam Fish Passage Alternatives Analysis, U.S. Army Corps of Engineers, Omaha, NE.

Sodhi, D.S., S.L. Borland and J.S. Stanley (1996) Ice action on rip-rap: Small-scale tests, *CRREL Report 96-12*, U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, NH.

Sodhi, D.S., S. Borland, J.S. Stanley and C.J. Donnelly (1997) Ice effects on rip-rap: small scale tests, *Energy and water: sustainable development*, Proceedings of the 27th Source Congress if the International Association for Hydraulic Research (IAHR), San Francisco, CA, Aug 10-15, American Society of Civil Engineers, New York, NY, p. 162-167.

Sodhi, D.S. and C.J.Donnelly (1999) Ice effects on rip-rap: Model tests, *Putting Research into Practice,* Proceedings of the 10th ASCE Cold Regions Engineering Conference, Lincoln, NH, Aug. 16-19, American Society of Civil Engineers, Reston, VA, p. 824-837.

USACE (2002) Ice Engineering, Engineer Manual EM 1110-2-1612, U.S. Army Corps of Engineers. http://www.usace.army.mil/inet/usace-docs/eng-manuals/em1110-2-1612/toc.htm

Wuebben, J.L. (1995) Ice effects on riprap. *River, Coastal and Shoreline Protection: Erosion Control Using Riprap and Armourstone*, John Wiley.

Appendix E
Geomorphology

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

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Lower Yellowstone River
Intake Dam Fish Passage
10% Design Analysis
Geomorphology Appendix E

1. Study Purpose

The purpose of this study is to provide qualitative estimates of the types of river adjustments that may be expected if the existing irrigation diversion structure was removed. Some flow data and geometry data were used to provide insight into the channel stability of this reach. Additionally, some preliminary designs were prepared for the upstream diversion option to limit lateral migration of the channel and ensure adequate water surface elevation is available for a new irrigation intake.

2. Expected Changes Due to Dam Removal

It is likely that channel adjustments will occur if the existing structure is removed and replaced with a new diversion upstream. If the structure is removed or lowered, the energy slope across the sediment delta will increase. There will then be a much greater capacity for the river to transport the sediment that has accumulated behind the dam. The current bathymetry behind the dam and the sediment gradation is not known. Additional survey data will be required to quantify the degradation that is possible if the structure is lowered or removed.

Complete removal of the structure is likely to result in headcutting that migrates upstream until it reaches another hard point or is limited by armoring of the bed material. Coarse sediment is typically deposited first, on the most upstream portion of the backwater created by the dam. Gradually finer material is deposited closer to the dam as velocity decreases. One headcut starts at the dam where the deposition consists of primarily fine materials. The second is at the delta formed at the upstream end of the impoundment. Channel widening will also occur as bank heights increase along the newly exposed stream are scoured and become unstable.

Since the intake dam for this project has been in place for over 100 years, the structure is submerged during high flows, and the Yellowstone has a large variability in flows, it is very possible that the delta has completely filled behind the dam. Headcutting at the irrigation diversion structure could be prevented by lowering the dam to the existing channel elevation. The dam may also be lowered only a sufficient amount to allow for fish passage. This would not allow for as much of the upstream delta to be released downstream or for the sediment carrying capacity of the river to increase as much as it could with complete dam removal. Another option is to removal only a portion of the dam laterally across the river. This would result in water flowing through a constriction which would greatly increase the velocity that would not be beneficial to fish passage. Scour at the location of partial dam removal and instability of the remaining portion of the dam would occur. Since this would lead to instability and not aid fish passage upstream, this option is not recommended.

The Yellowstone River location is also known to experience large amounts of ice buildup during the winter months with significant ice flow and jam potential during spring breakup. If the dam captures most of the ice and reduces the amount of ice built up downstream, ice buildup may be more frequent and severe in the downstream reach after the dam is removed. Ice may continue to buildup at the former dam location, and degradation that occurs near the dam location could increase due to higher velocities under the ice that is not limited by the presence of the dam.

3. Intake Diversion Data Analysis

Specific gage plots for the Yellowstone River at the Sidney, MT gage (41.9 miles downstream) were prepared to determine if there is any substantial channel aggradation or degradation near the irrigation diversion. Measured data for this site was available from the USGS sporadically from 1967 to 1976 and approximately once per month from 1976 to 2006. Three flows of 6000, 10000, and 20000 cfs were plotted, shown in Figure 1. Exact values of these flows were not always present, so gage height values were interpolated or extrapolated based on measured values close to the flow of interest. Some obvious outliers were removed that may have been influenced by ice or bed changes due to recent high flows.

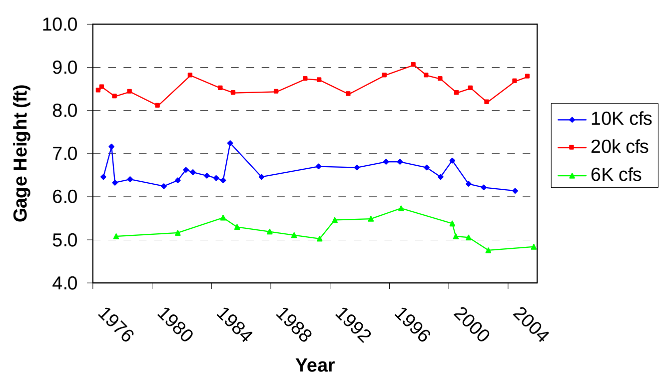

Figure 1. Yellowstone River at Sidney, MT Specific Gage

No trend is apparent from the gage height values at any of the discharges plotted. The river in this reach has likely adjusted to the long term presence of the diversion structure and has now reached a stable geometry. Only very limited data is available at the closest upstream (17 miles) gage at Glendive, MT.

Removal of the intake diversion will result in an increased sediment transport capacity in this reach. For a constant flow and sediment size, the ratio of the natural river slope to the backwater slope caused by the dam provides an estimate of the increased sediment transport capacity after dam removal. Water surface profiles generated by the previously developed Bureau of Reclamation RAS model were used to estimate the increased transport capacity at the intake dam site. At 5000 cfs, there is a six-fold increase, at 15000 cfs a three-fold increase, and at 20000 cfs, the transport capacity doubles. In reality, the sediment deposit behind the dam is not homogeneous, but varies as different sediment sizes were deposited by various flows. Nevertheless, the coarser sediment is likely to be on top of the deposit and will be moved while the slope is the highest in the removed dam. The Yellowstone will have a much greater capacity to transport sediment until a stable geometry is reached if the existing dam is removed or lowered.

Cross-section data, collected in 1976, from the dam at river mile 71.1 downstream to river mile 69.4 was also available. This was compared to the geometry used for the USBR RAS model. Average bed elevations were calculated using both data sets but little conclusions can be gained because the RAS cross-sections and the 1976 cross-sections are not at the same location.

Some channel widening is expected to occur in the vicinity of the dam. River widths upstream, near the dam, and downstream of the dam were measured on aerial photography. The channel just upstream of the dam is approximately 16% narrower than the upstream reach, and 10% narrower than the downstream reach. After dam removal, the reach near the dam will likely conform to the planform geometry of the upstream and downstream reaches.

4. Upstream Diversion Channel Stabilization

One of the main concerns for relocating the diversion to an upstream location is the uncertainty of maintaining the channel against the left bank, especially during low flows. A preliminary design has been made to prevent migration of the right bank landward and keep the thalweg along the left bank. This design includes a series of four to six dike structures along the right bank. The average velocity in the vicinity of the diversion will increase as it is constricted and this will also prevent sediment deposition near the headworks. The design would induce sediment deposition in the structure dike field. Some ice damage may occur to these structures, and routine maintenance should be anticipated to maintain full diversion capability. Revetment is necessary along the left bank in order to prevent erosion that may occur as the velocity increases.

An additional option to the design are several rock hard points along the left bank upstream of the revetment. There appears to be bank erosion in this area seen on the aerial photographs, but this should be confirmed during a site visit if this option is selected. Additional water could flow through the left channel if the river is overly constricted near the dikes and a backwater is created.

Additional survey data will be needed to determine exact placement and calculate rock quantities. The structure elevations will be two to three feet above the ordinary high water (OHW) which will also be determined through a site visit and survey data. If this option is selected, additional RAS modeling will be needed to verify that the structure height is adequate to maintain enough head at the intake for the flows required.

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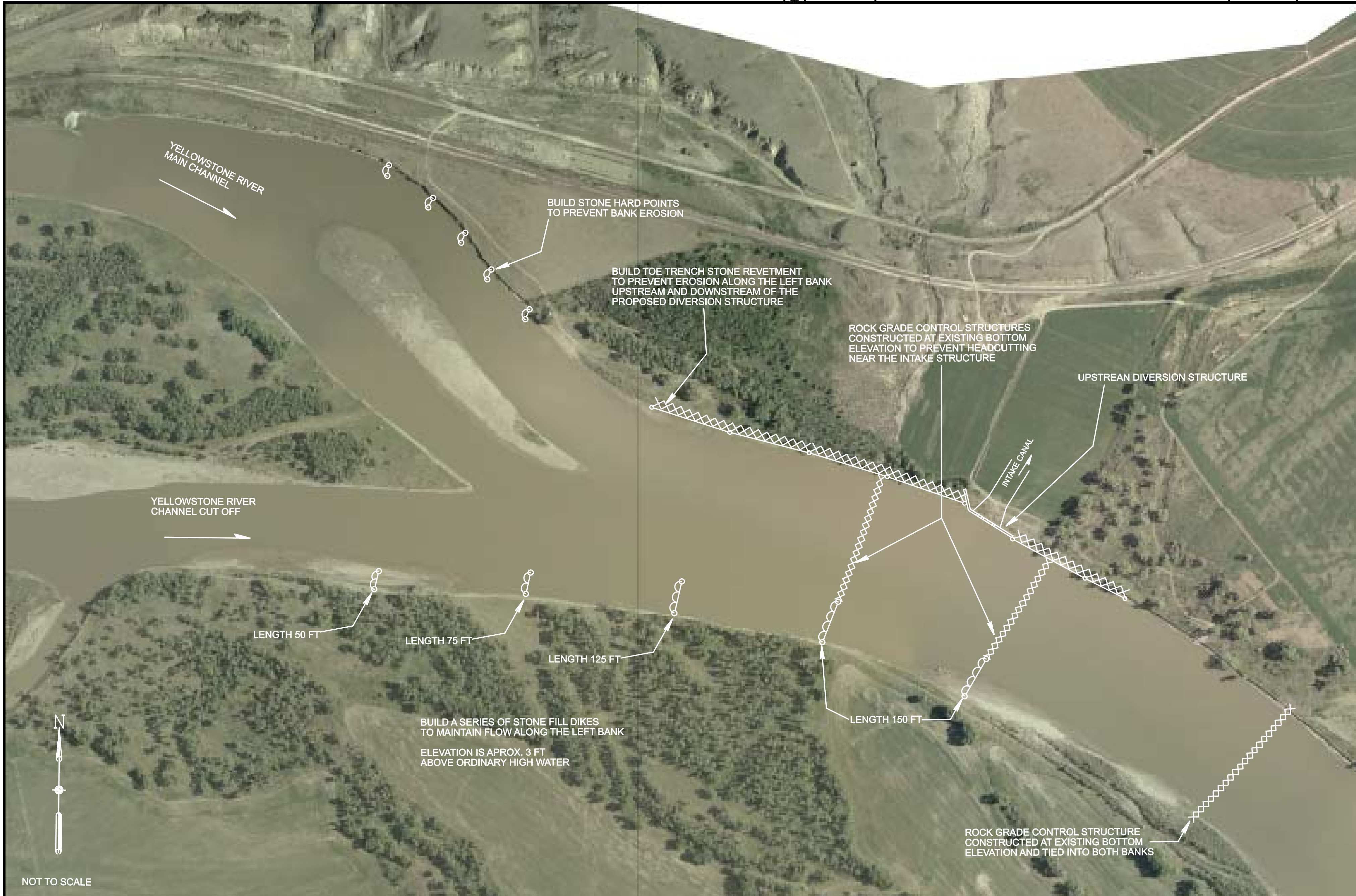

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ID1	Date: JUNE 2006	Contract No.				Reviewed by: D.B.P.	Drawn by: J.J.S.
	Drawing Code:						

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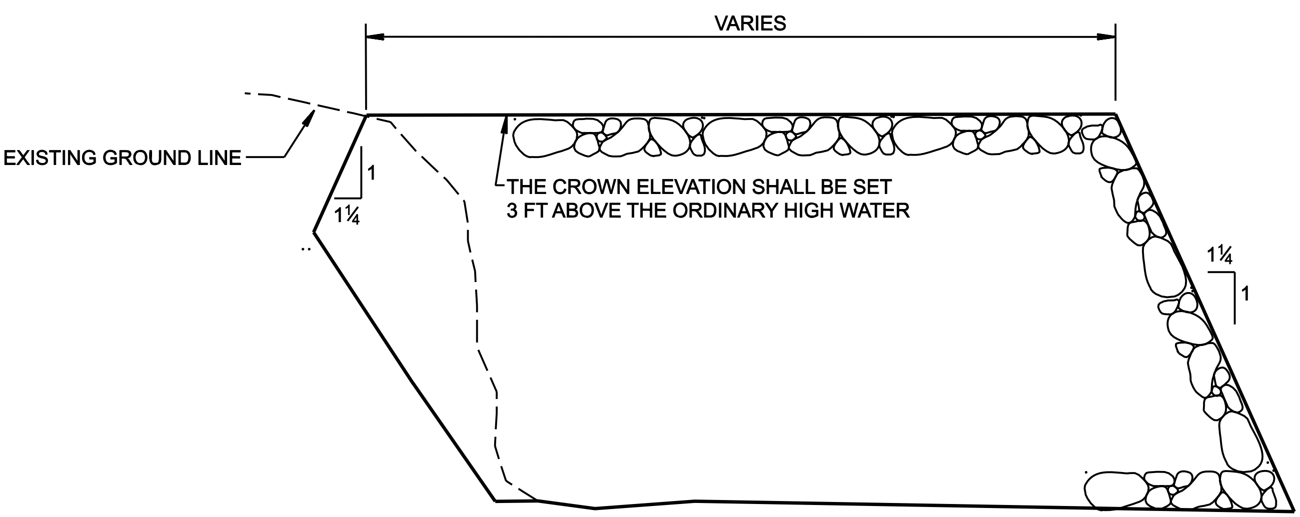

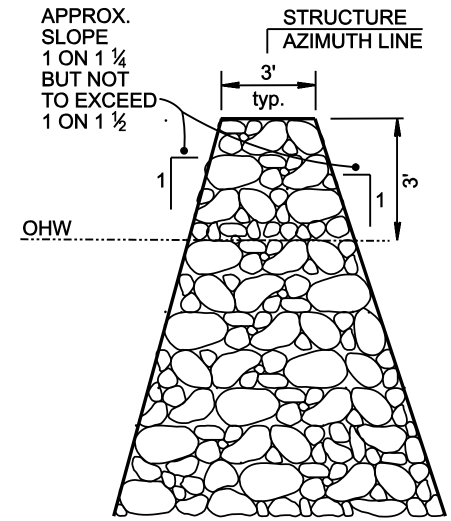

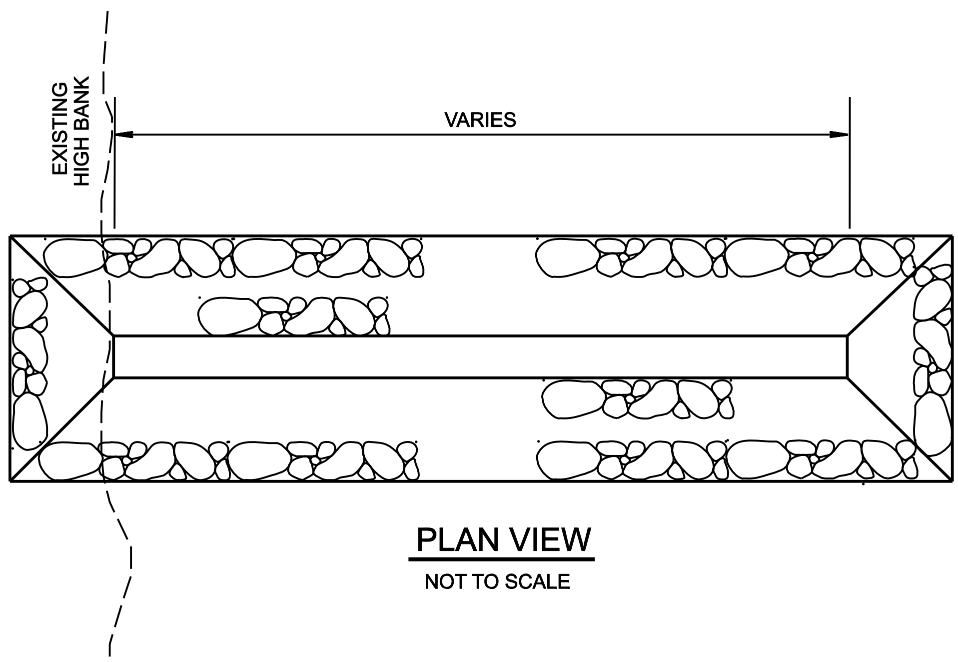

PLAN VIEW

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Sheet No. ID2	Computer File:	Spec. No.	U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS OMAHA, NEBRASKA	YELLOWSTONE RIVER; MONTANA RIVER CONTROL STRUCTURES UPSTREAM DIVERSION ALTERNATIVE DIKE DETAILS
	Date: JUNE 2006	Contract No.		
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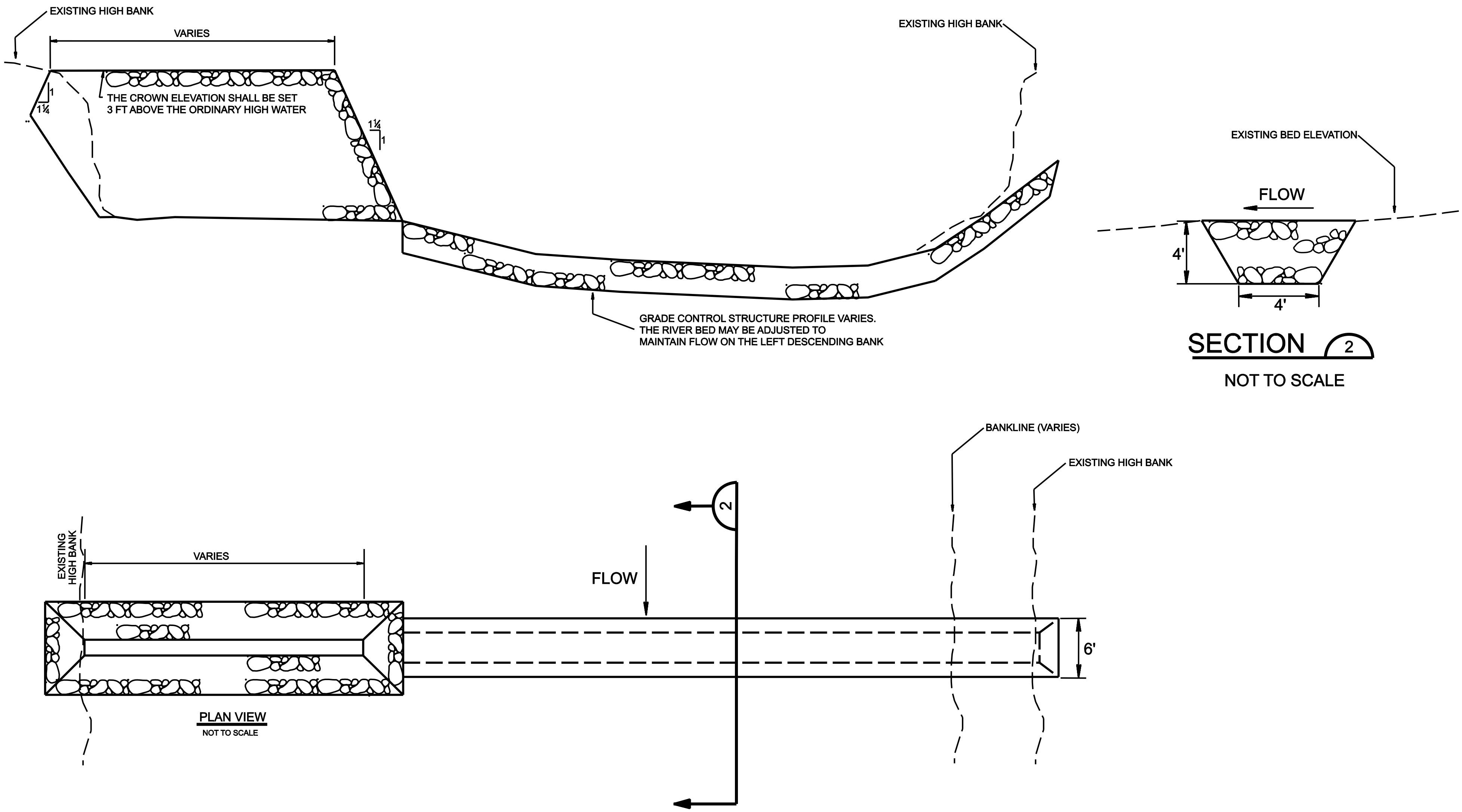
Submitted by: DRAFT ChiefSED. & CHAN. STABSection	Designed by: J.J.S.	Checked by: P.B.D.
	Reviewed by: P.B.D.	Drawn by: J.J.S.



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Designed by: J.J.S.	Checked by: P.B.D.
Reviewed by: P.B.D.	Drawn by: J.J.S.

Appendix F
Geotechnical Design

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

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Geotechnical

1. Geology and Soils. The following geology and soils information presented in paragraphs 1.1. and 1.2. was obtained from the Bureau of Reclamations Concept Study Report II.

1.1. General. The Intake Diversion Dam, here after referred to as Intake Dam, is situated along the northeast Bank of the Cedar Creek Anticline, a major structural feature in southeastern Montana. Cretaceous strata, exposed along the axis of this northwest-southeast trending (northwest plunging) anticline, dip gently to the northeast and are overlain by Paleocene sedimentary strata of the Fort Union Formation in the Intake area. Here, the Yellowstone River has incised an approximately 2-mile-wide channel into the surrounding upland. The Fort Union Formation constitutes bedrock in the area and consists of an alternating sequence of clay shales, siltstones, sandstones, lignitic shales and lignite. Because of the terrestrial-type deposition, the beds interfinger and grade both laterally and vertically. The stratigraphic section varies from location to location and correlation between points is unpredictable. Permeability of the various strata varies greatly due to the varying degree of compaction and cementation. The high erodibility of Fort Union material on steep, unprotected slopes gives rise to badland type topography along the walls of the Yellowstone River valley.

Weathered bedrock is soft and has soil properties. Unweathered bedrock materials have both rock- and soil-like characteristics. Exceptions are lenticular bodies of moderately cemented, moderately hard sandstone locally present within the Fort Union. Also, thicker lignite beds have burned back from their outcrops and overlying shales have been baked and fused to form moderately hard material locally referred to as clinker. These vary in both thickness and lateral extent. Beds of variable thickness of lignitic shale to lignite occur throughout the Fort Union Formation.

Several terrace levels, cut into the Fort Union Formation and overlain with gravel, are recognized along the valley. These range in age from Pleistocene to Holocene (recent) and occur from 14 to as high as 420 feet above the present river level. The younger terraces which range from 14 to 90 feet above the river underlie most of the Intake Dam area. The gravel terrace occurring in the floodplain is generally blanketed with fine-grained soils.

1.2. Fish Screen, Drop Outlet Structure and Rock Ramp. The fish screen structure, located within the Main Canal, will be founded on bedrock of the Fort Union Formation. The fish bypass, extending hm the downstream end of the screen to the Yellowstone River downstream of the Intake Dam, a distance of approximately 700 feet, will be excavated in bedrock of the Fort Union Formation. Overburden is up to about 55 feet thick above the bypass invert.

Surficial deposits consisting of alluvial, colluvial, eolian and terrace deposits of Quaternary age generally mantle the bedrock and occur along the upper portion of the

canal prism. Surficial deposits consisting of material excavated from the canal and placed in waste banks is present on both sides of the canal. Also, fill material has been placed along the river bank downstream of the Intake Dam to provide slope protection. Depending on the direction of the bypass alignment, the slope protection material may be encountered at the bypass outlet. Surficial deposits will have no significant design or construction considerations for the fish screen and, depending on designs and construction methods, no to minor considerations for the fish bypass.
Shales, siltstones, uncemented sandstones, lignitic shales and lignite of the Fort Union Formation generally are rippable with modern equipment and excavated by common methods. Cemented sandstones and concretions within the Fort Union can not be ripped and, if encountered, may require drilling and blasting to remove from excavations. It should be anticipated bedrock will bulk about 27 percent if excavated and dumped. It will probably bulk 10 to 15 percent after being excavated and compacted.

The siltstones, uncemented sandstones, lignitic shale and lignite are all quite erodible. However, the shales and cemented sandstones will retard (but not eliminate) erosion. There is a potential of encountering methane gas within the lignific shales and lignite beds.

Stability of bedrock materials within the fish barrier and bypass excavations is not expected to be a significant problem. Shallow excavations in bedrock will be stable on 1/2:l slopes. Permanent excavations should be laid back on 1:1 slopes.

Bedrock materials below the weathered zone (upper 5 to 10 feet) likely will have sufficient bearing capacity to support the fish barrier and bypass pipeline. However, lignitic shales and lignite are fractured, soft, low in density and readily air slake. If these materials are encountered within the excavations, they should be overexcavated and replaced with compacted backfill to preclude problems with deformation. Also, shales exposed within the excavations will likely air slake rapidly and freshly exposed surfaces should be protected before being covered with concrete or compacted backfill.

Groundwater is believed to be tributary to the Yellowstone River with the water table occurring at or above the river. Perched groundwater may occur in surficial deposits just above the bedrock contact and also in sandstone units and fractured lignite beds within the bedrock.

The shales and siltstones are generally impervious. The sandstones are semipervious and will weep water. The lignite beds are fractured, low in density and semipervious to pervious. Lignite beds encountered within the screen or bypass excavations should be expected to pass water rapidly.

The dam across the center section and right abutment is founded on Quaternary alluvial deposits. Alluvial deposits are shown to extend across the floodplain (Torrey and Kohout, 1956) and mapped by McKenna, et al (1994) to vary between 20 and 50 feet thick in the vicinity of the Intake Dam. However, a small, isolated exposure of bedrock of the Fort

Union Formation appears to outcrop locally along the right (south) bank of the river downstream of the dam.

Preconstruction drill hole information indicate alluvial deposits within the area of the present river channel consist of sand and gravel. Although not noted on the logs, cobble-size material is also present within the coarse-grained materials. These coarse-grained soils are continuous across the floodplain but, outside the river channel, including the right abutment, are overlain with fine-grained soils (silts and clays).

Fill material was placed on the right abutment to divert river flows around and support the right abutment concrete wall. These materials consist of a varying percentage of boulders and cobbles in a matrix of fine- and coarse-grained soils. The dimensions and configuration of the fill material is uncertain but maximum thickness is believed to be about 20 feet adjacent to the right abutment concrete wall based on design drawings.

The right abutment fill material may contain boulders up to 3 feet maximum size. Drill hole data suggest the bedrock surface occurs at approximate elevation 1960 feet along the fishway and bedrock is not expected to be encountered. The coarse-grained alluvial deposits are rounded and consist of sand, gravel, and cobbles, up to about 6-inch-maximum size with lesser amounts of cohesive and cohesionless fines. These materials are stable on 2-1/2:1 slopes. The fine-grained alluvial deposits and fill material are stable an 2:l slopes if seepage is not occurring. If seepage occurs in these materials, remedial measures may be required to prevent internal erosion and slope instability including flattening the cut slopes.

2. Construction of Alternatives. Paragraphs 2.1. and 2.2. deal with the construction specifics of the two alternatives studied.

2.1. Upstream Diversion Alternative.

The upstream diversion alternative is described in detail in the Hydraulics appendix. A plan view of this alternative is shown on Sheets F-101 and F-102. A major feature is the gate structure. The gate structure will require pile foundations because of it's location relative to the river bed and the pressure placed upon the soil. Dewatering with sumps and well points will be needed. This will done in conjunction with an earthen cofferdam.

Another feature is the outlet drop structure located at the irrigation canal. The structure is founded in the canal side slope and will require a cofferdam to construct it. An earthen cofferdam is not feasible as it would completely block the canal, therefore a sheet pile cofferdam would be used.

The 3,720,000 cubic yards of canal excavation will be a mixture of soil and stone, and soft weathered (easily rippable) bedrock. The material would be excavated and hauled using scrapers, and excavated with backhoes, and loaded on trucks and hauled to the disposal site. The disposal site was not located but assumed to be within a distance of 2 miles. Real estate related costs would be incurred but were not estimated.

The upstream diversion alternative would also include the degrading of the existing Yellowstone River dam. This would require the construction of a riprap protected/earthen cofferdam. The cofferdam would be constructed to close-off approximately half of the Yellowstone flows. The existing dam planking, timbers and rock would be removed to the elevation and cross section as shown on Sheet F-106. The cofferdam would be removed and the other half constructed, and the other half of the existing dam removed. This construction operation involving the cofferdam will require some ramping on both banks of the Yellowstone River. The cofferdam riprap will be reused for each half and the earthen fill will not. It is assumed some of the earthen material will not be recovered completely due to the moisture content and foundation fill will be allowed to wash into the river.

2.2. Rock Ramp Alternative.

The rock ramp alternative is described in detail in the Hydraulics appendix. A plan view of this alternative is shown on Sheet F-103. The rock ramp alternative options (various slopes and drops) involve large amounts of riprap. The Hydraulic appendix provides details on the rock ramp design. The input received from CRREL (see Appendix D) provided guidance that blocks in the 4' to 6' diameter range should be used to prevent movement from ice forces. These forces would apply to the large fish blocks needed as resting spots for the fish to continue their trip up the ramp. The blocks protruding above the rest of the riprap (18"-24" riprap) would be subject to attack from the ice. The blocks are currently designed to be located at the ends of the steps in of the ramp in a zig-zag pattern. Future designs may be clusters or groups of blocks of various diameters. The irrigation district has performed riprap maintenance on the structure over the past 80 years, and a considerable amount of stone has been deposited at the dam and has migrated due to ice jams to just downstream of the dam. The actual quantity of riprap required may be significantly less than the calculated amount due the assumption of a flat river bottom. In actuality there may be a significant amount of existing riprap in the river. The 5% slope/1' drop rock ramp would require 45,530 tons of the 18"-24" riprap and 20,700 tons (1755 pieces) of the 4'-6' diameter stone. The 2% slope/1' drop rock ramp would require 109,140 tons of the 18"-24" riprap and 20,700 tons (1755 pieces) of the 4'-6' diameter stone. The number of tons of the large stone did not change because the ramp options are the same drop.

The rock ramp alternative would also include the reconstruction of the crest of the existing Yellowstone River dam. This would require the construction of a riprap protected/earthen cofferdam. The cofferdam would be constructed to close-off approximately half of the Yellowstone flows. The cofferdam would be a U-shaped structure to completely isolate the construction area. The portion parallel with the flow would require the removal of some existing riprap to minimize seepage into the construction area. The crest would be hardened with one of two options of a concrete structure. The existing dam planking, timbers and rock would be removed to the elevation and cross section as shown on Sheet F-106. The existing wood piling, wood sheet piling and rock would provide a excellent base for the new crest structure. The wood should be in good condition if it has been continuously covered with water. The cofferdam would be removed and the other half constructed, and the other half of the existing dam removed. This construction operation involving the cofferdam will require some ramping on both banks of the Yellowstone River. The cofferdam riprap will be reused for each half and the earthen fill will not. It is assumed some of the earthen material will not be recovered completely due to the moisture content and will wash into the river. The existing sheet pile of Intake Dam should prevent excessive underseepage flows from entering the work area. Pumps would be utilized to minimize water in the construction area, the rock should provide a good base for tracked equipment. The contractor may also place the riprap by erecting a overhead cable system as used by the irrigation district for placing maintenance riprap. The concrete crest structure would require forming and concrete pumping. This construction operation is shown on Sheet F-104.

2.2.1. Precast Concrete Blocks. The precast concrete blocks option was used as an option due to the lack of riprap sources in the area. The blocks would be formed using fabric-form bags to give the blocks a rounded (stone-like) shape. The bags would be placed in a wooden form for adequate shape and dimensioning. <u>The blocks could be either used as a replacement for the large stone or used with driven H-pile to add integrity to the overall structure.</u> The H-pile would have a maximum length of 15 feet. The second option was used for the design. Due to the number of pieces, a concrete batch plant would probably be established near the site. The existing river bottom cleared of rock, the H-piles driven, and the cured concrete blocks placed over the piles (using a cylindrical opening formed in the blocks). The remainder of the ramp (steps) would consist of 18”-24” diameter stone. The ramp profile the H-pile and precast concrete blocks is shown on Sheet F-105. Details of the precast concrete block system is shown on Sheet F-107. The quantity of concrete required for the precast blocks would be 9100 cubic yards.

2.2.2. Quarried Stone. The large stone at the steps would be buried within the smaller riprap and the top of the stone protruding above them. The detail for quarry stone option is shown on Sheet F-107.

Quarry locations able to supply the 4’-6’ diameter stone were investigated. No quarries in North Dakota were located. Several quarries in Canada were located but they are in the East Central part of Manitoba (considerable distance from the site).

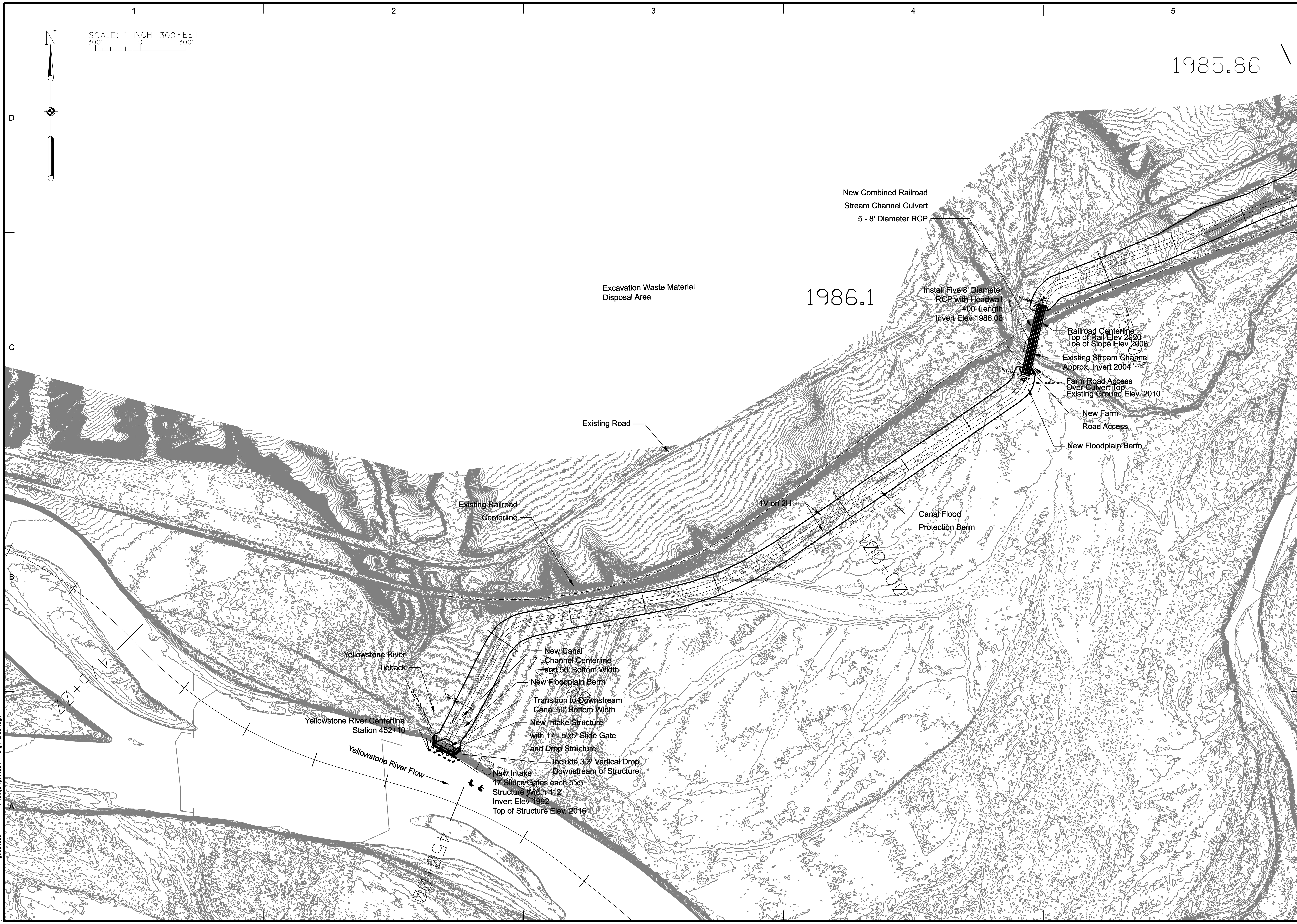
Quarries in Montana and Wyoming are the most promising. The irrigation district quarries their own stone from a site directly southeast of the site, see Sheet F-108 for the location relative to the project. They have been using this source for the maintenance of Intake Dam. This site may not be capable of producing the large stone. Commercial quarries exist in Glendive and Warren. Contacts at both of these sites did not seem confident in producing the number of pieces (1700+) of the 4’-6’ diameter stone. The best source for the large stone is in Guernsey Wyoming, this quarry has produced large stone for a number of projects and is good quality stone.

The Guernsey quarry would require a 3-month lead time to produce the stone. It would be transported to Glendive by the Burlington Northern Santa Fe Railway using gondola cars. It would then be trucked to the site on both sides of the site. A concern is with the availability of gondola cars, a limited number would be available. It takes approximately 5-7 days to transport the stone from Guernsey to Glendive.

The 18’-24” diameter riprap could be obtained from Glendive, the irrigation district quarry, Warren or Guernsey.

For this design, the Guernsey quarry would be used for the 4’-6’ diameter stone, and the irrigation district site would be used for the 18”-24” riprap, this would be the best and most feasible combination. The irrigation district quarry may require additional State of Montana documentation and permitting with the Department of Environmental Quality.

3. Future Studies. A soil investigation will be required for all project structures prior to initiated final designs. The irrigation district quarry site will be investigated and mapped by a geologist to confirm the production potential.



US ARMY CORPS
OF ENGINEERS
OMAHA DISTRICT

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U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

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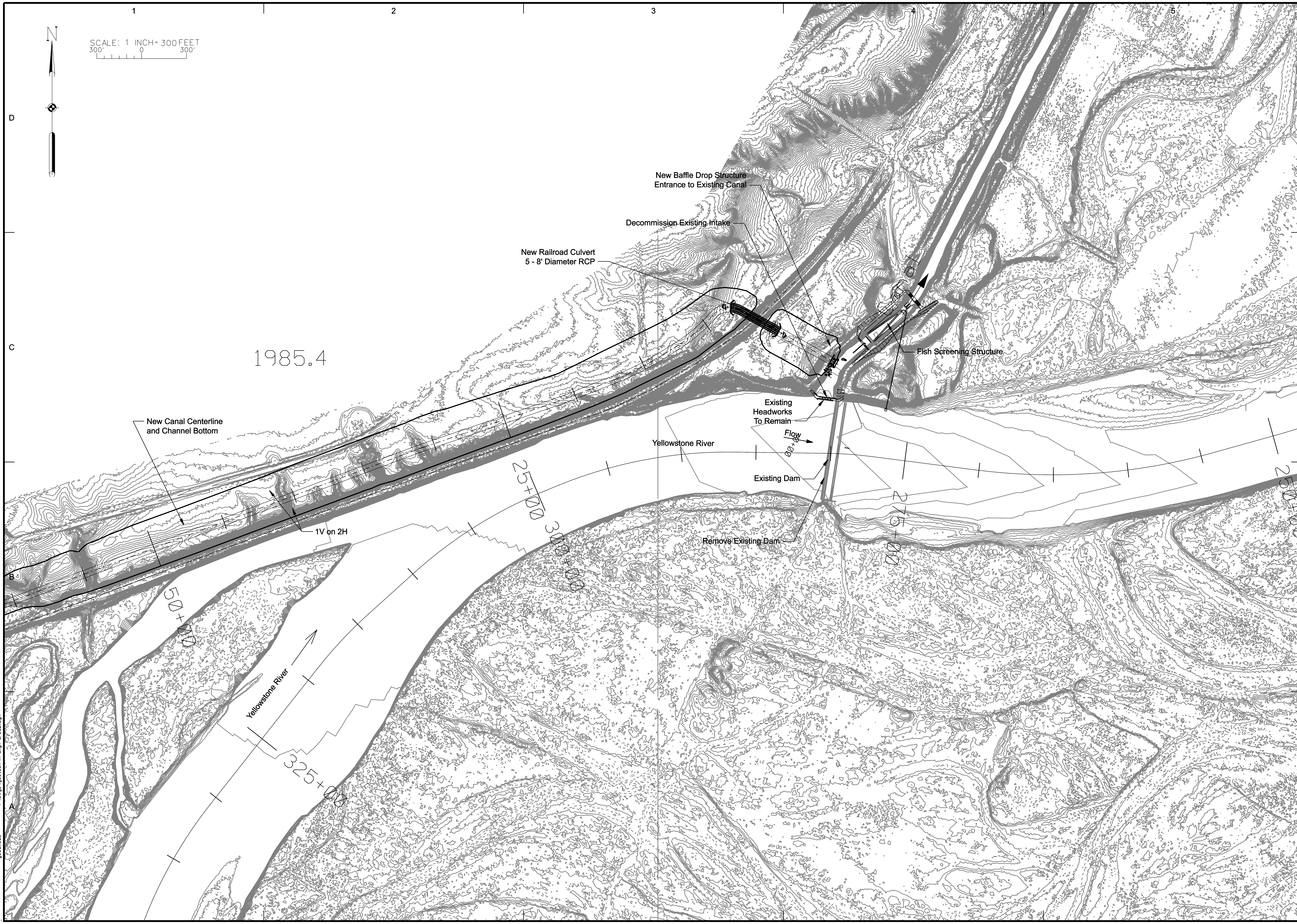

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UPSTREAM DIVERSION
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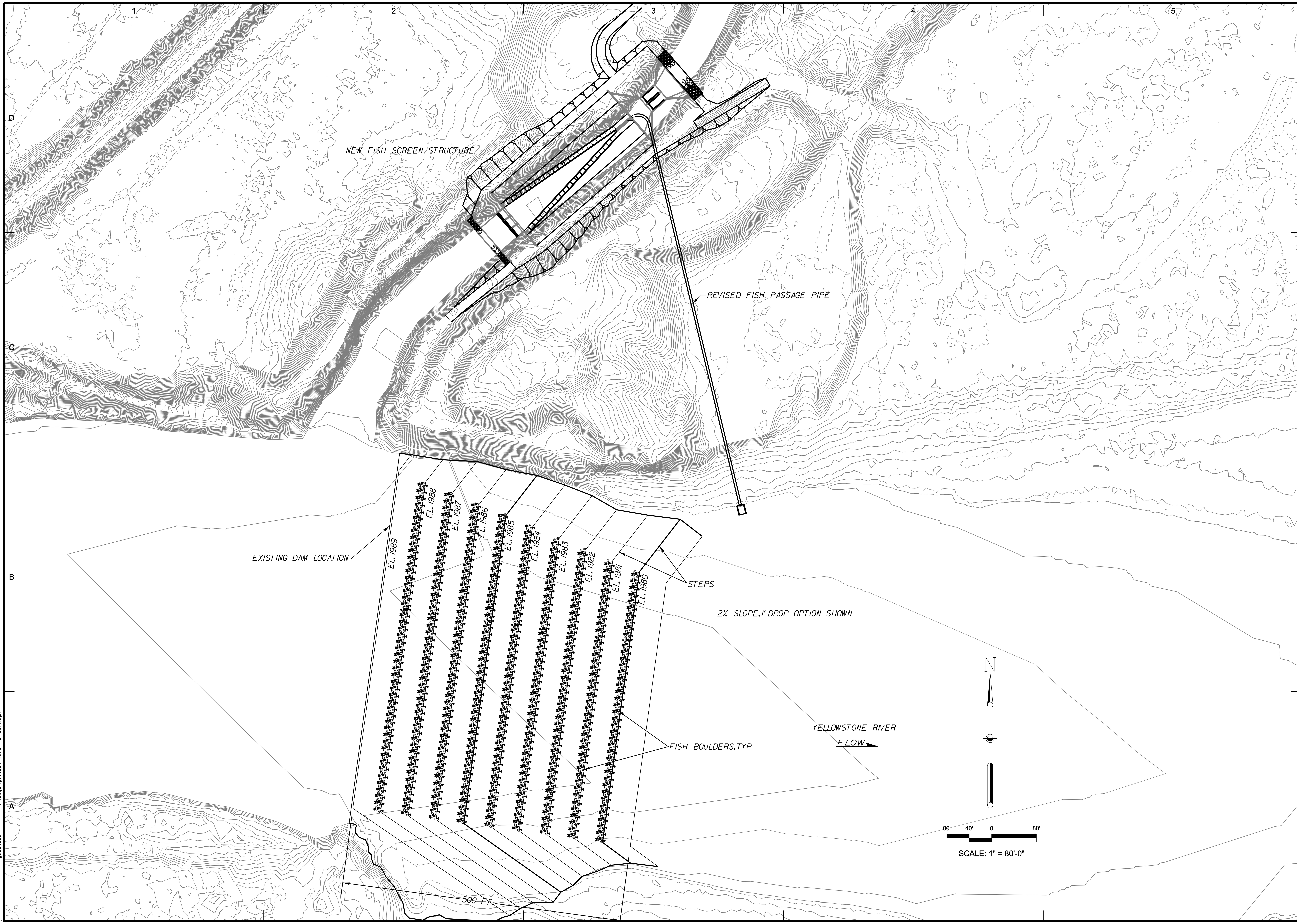

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US ARMY CORPS OF ENGINEERS
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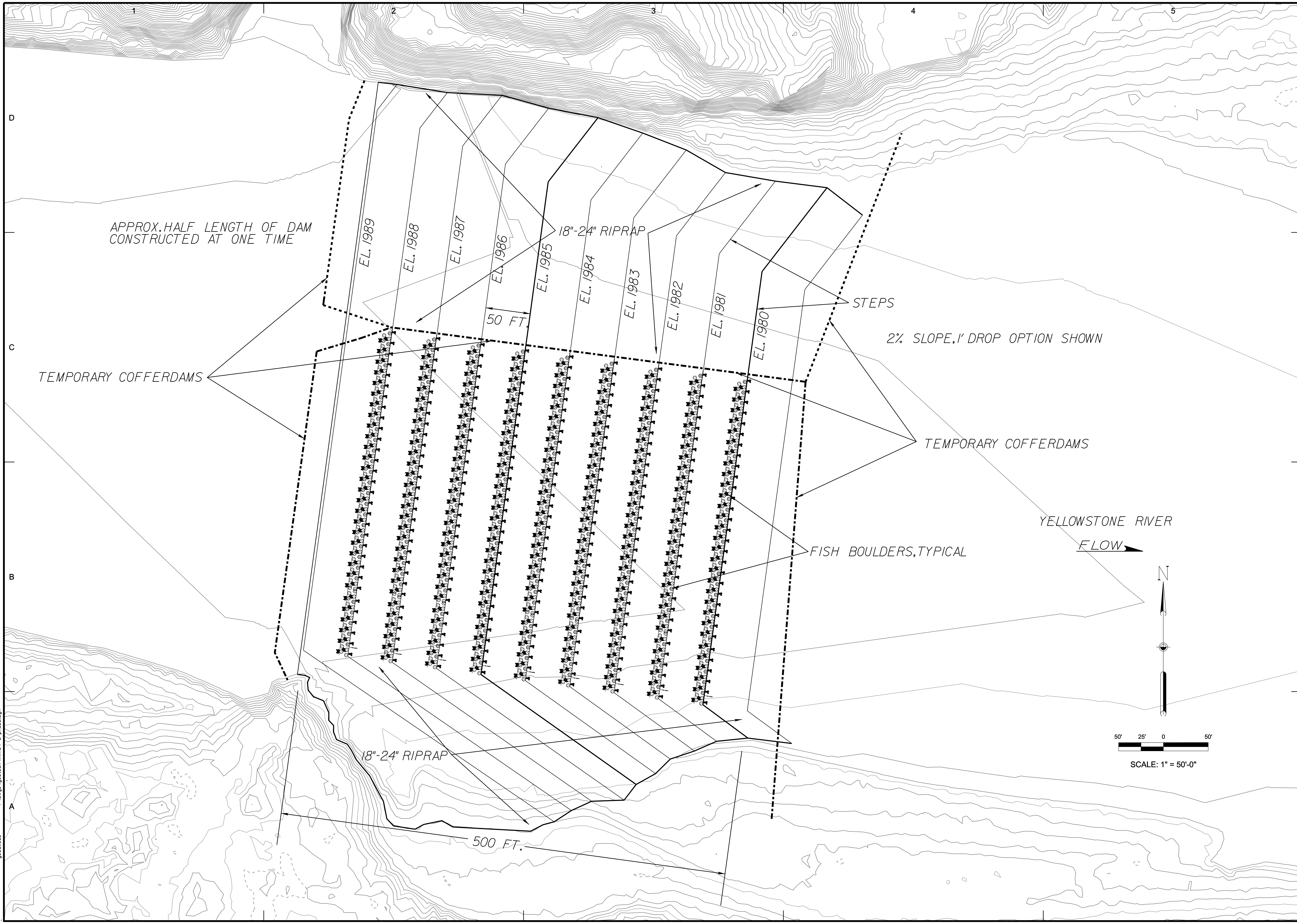
CORPS OF ENGINEERS

DESIGNED BY: X		DATE: MMM YYYY
DWN BY: X	CKD BY: X	SOLICITATION NO.: W9128F-0X-X-00XX
SUBMITTED BY: X		CONTRACT NO.: W9128F-0X-C-00XX
FILE NAME: mm04-G-idam.dgn		FILE NUMBER: X
SIZE: 33.1X22	PLOT SCALE: 0.08 ft / IN.	PLOT DATE: 7/12/2006

Y
Y
Y
Y

ROCK RAMP ALTERNATIVE
GENERAL PLAN

SHEET IDENTIFICATION NUMBER
F-103



US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

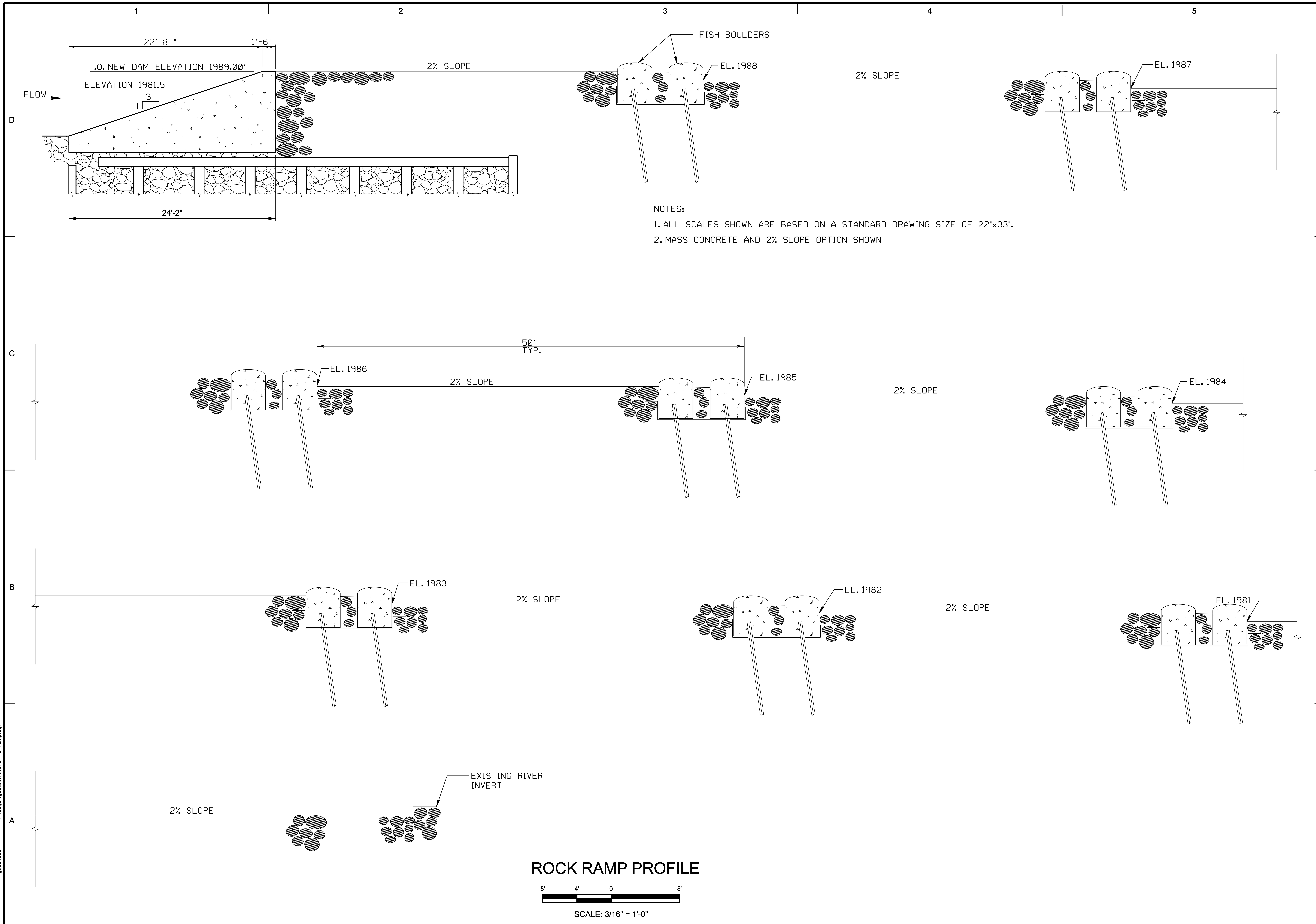
DESIGNED BY: X		DATE: MMM YYYY
DWN BY: X	CKD BY: X	SOLICITATION NO.: W9128F-0X-X-00XX
SUBMITTED BY: X		CONTRACT NO.: W9128F-0X-C-00XX
FILE NAME: mm04-G-phase.dgn		FILE NUMBER: mm04-g-phase.dgn
SIZE: 33.1X22	PLOT SCALE: 0.08 ft / IN.	PLOT DATE: 7/12/2006

Y
Y
Y
Y

CONSTRUCTION PHASING OF CONCRETE RAMP

SHEET IDENTIFICATION NUMBER
F-104

I:\dwgs\geotech\mm04-G-phase.dgn
g6edxcos



NOTES:

1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"x33".
2. MASS CONCRETE AND 2% SLOPE OPTION SHOWN

ROCK RAMP PROFILE

SCALE: 3/16" = 1'-0"

US ARMY CORPS
OF ENGINEERS
OMAHA DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

DESIGNED BY: X
DWN BY: X
CKD BY: X
SUBMITTED BY: X
FILE NAME: mm04-G-ramp.dgn
SIZE: 33.1X22
PLOT SCALE: 0:1 ft:in / IN.
DATE: MMM YYYY
SOLICITATION NO.: W9128F-0X-X-00XX
CONTRACT NO.: W9128F-0X-C-00XX
FILE NUMBER: X
PLOT DATE: 7/12/2006

Y
Y
Y
Y

ROCK RAMP PROFILE
2% SLOPE OPTION

SHEET IDENTIFICATION NUMBER
F-105

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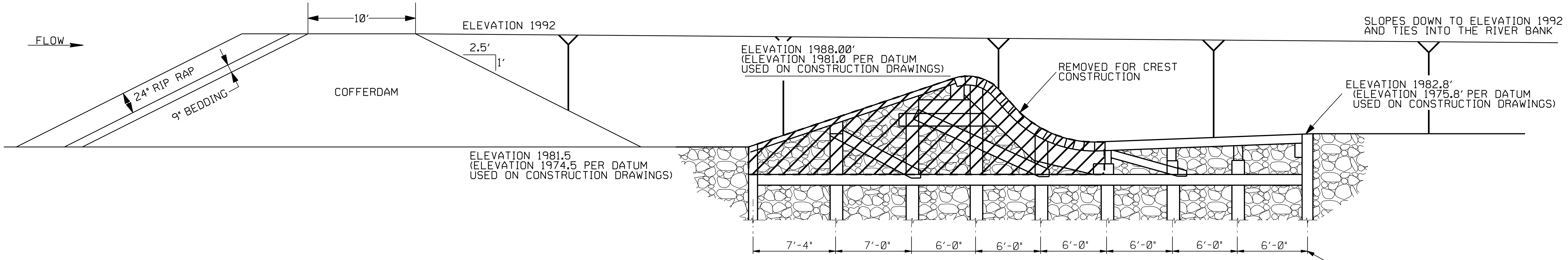
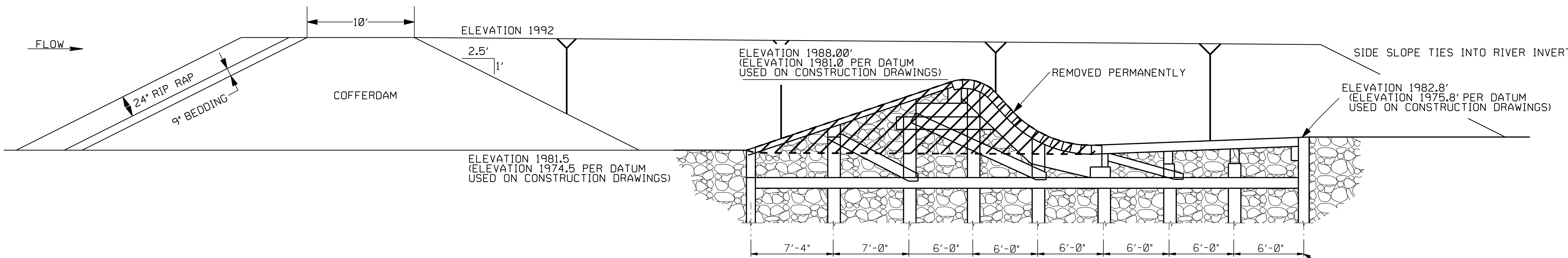
g6edxcos



EXISTING TIMBER DAM - REMOVAL SECTION FOR RECONSTRUCTION ON CREST (R0CK RAMP ALTERNATIVE)

8' 4' 0 8'

SCALE: 3/16" = 1'-0"



EXISTING TIMBER DAM - PERMANENT REMOVAL SECTION (UPSTREAM DIVERSION ALTERNATIVE)

8' 4' 0 8'

SCALE: 3/16" = 1'-0"

NOTES:

1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"x33".

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

MARK	DESCRIPTION	DATE	APPR

MARK	DESCRIPTION	DATE	APPR

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

DESIGNED BY: L.E.P.
DWN BY: L.E.P.
CKD BY:
SUBMITTED BY: B.N.H.
FILE NAME: MM04-S-104.DGN
SIZE: 33 X 22
PLOT SCALE: 0:1 ft/in / IN.
DATE: MMM YYYY
SOLICITATION NO.: W9128F-0X-X-00XX
CONTRACT NO.: W9128F-0X-C-00XX
FILE NUMBER: X
PLOT DATE: 7/12/2006

Y Y Y Y

SECTIONS
COFFER DAM AND EXISTING DAM
ALTERATIONS

SHEET IDENTIFICATION NUMBER
F-106

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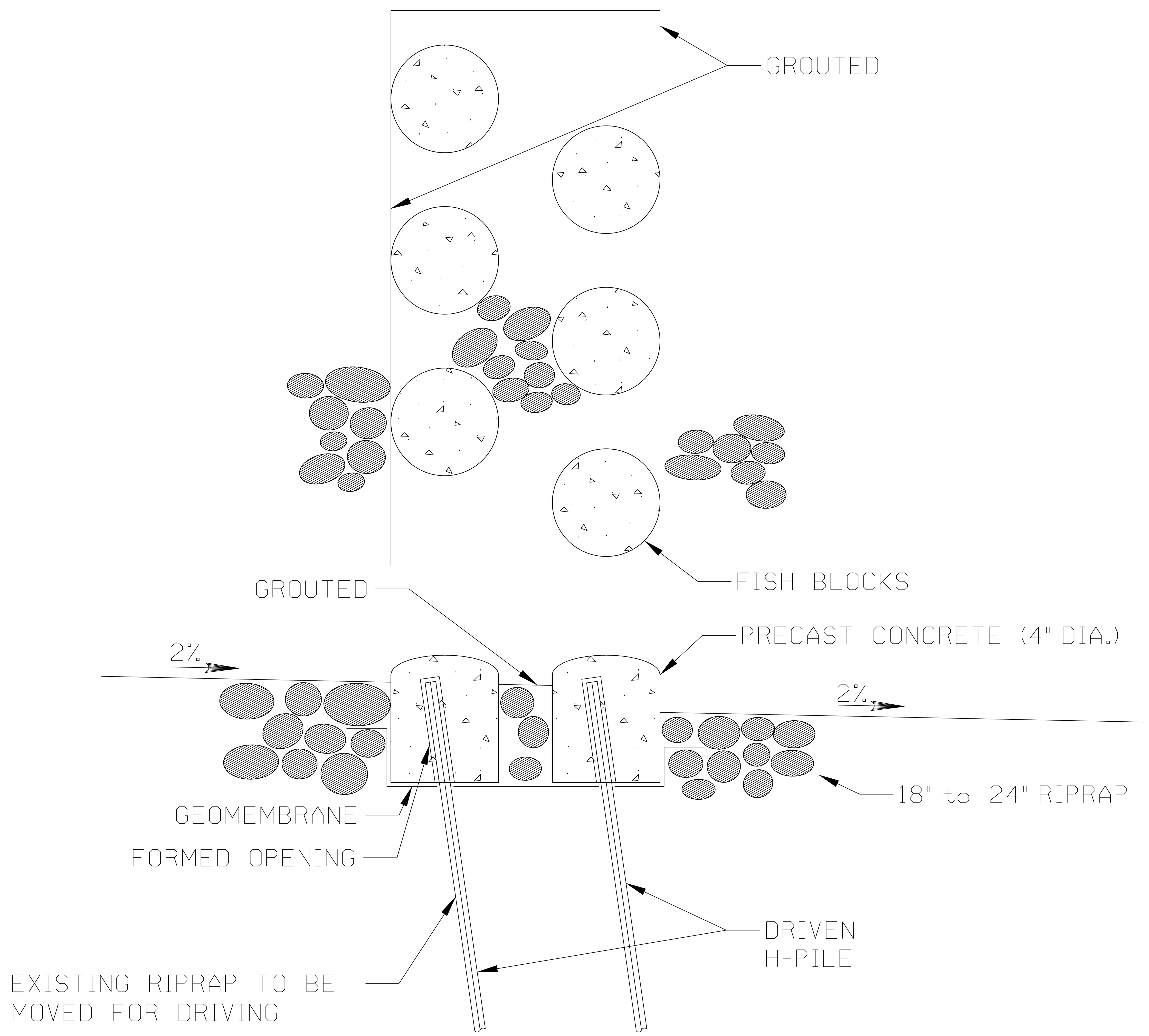
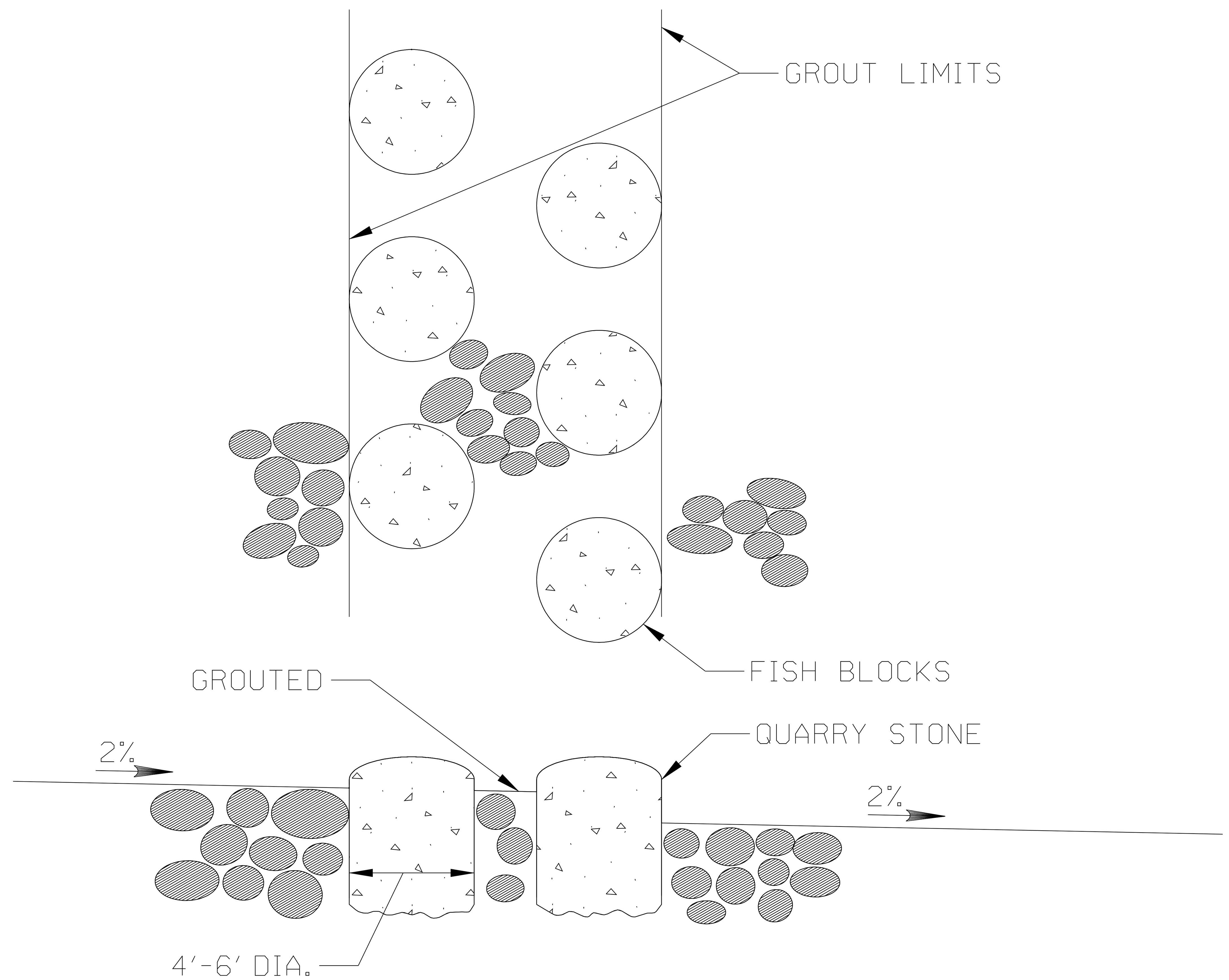
g6edxcos



PRECAST CONCRETE
PLAN AND PROFILE



QUARRY STONE
PLAN AND PROFILE

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

DESIGNED BY: X
DWN BY: X
CKD BY: X
SUBMITTED BY: X
DATE: MMM YYYY
SOLICITATION NO.: W9128F-0X-X-00XX
CONTRACT NO.: W9128F-0X-C-00XX
FILE NAME: terry_detail.dgn
FILE NUMBER: X
SIZE: 33.1X22
PLOT SCALE: 0.1 ft/in / IN.
PLOT DATE: 7/12/2006

Y
Y
Y
Y

TYPICAL DETAILS
FISH BLOCKS AT STEPS

SHEET IDENTIFICATION NUMBER
F-107

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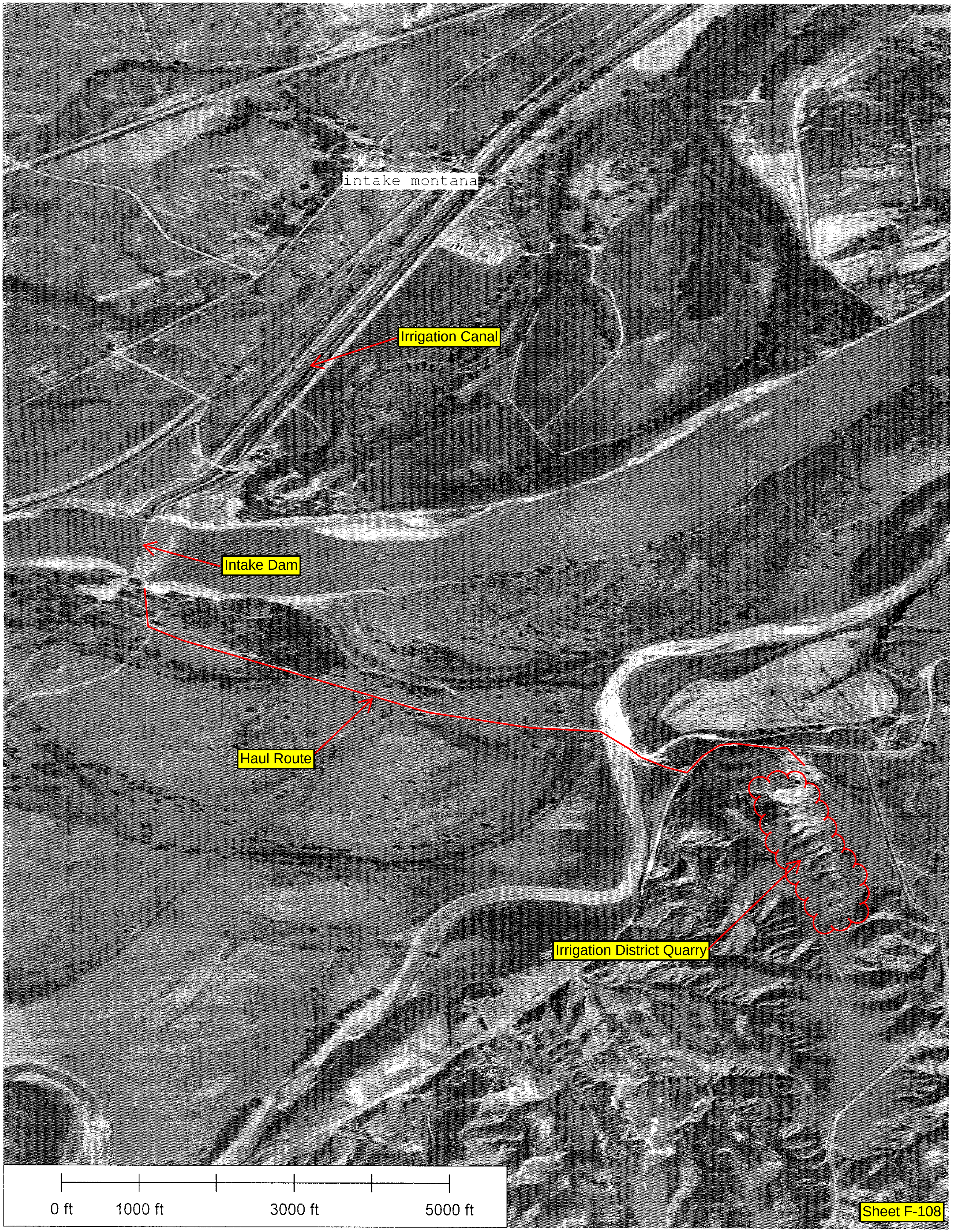
g6edxcos



intake montana
Irrigation Canal
Intake Dam
Haul Route
Irrigation District Quarry
0 ft
1000 ft
3000 ft
5000 ft
Sheet F-108

Appendix G
Engineering Design

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

US Army Corps
of Engineers ®
Omaha District

1 STRUCTURAL

1.1 DESIGN CRITERIA

The following references were used in preparing the structural design:

American Concrete Institute (ACI) Publications

ACI 318-05 Building Code Requirements for Structural Concrete (2005)

American Concrete Pipe Association Publications

Concrete Pipe Design Manual (1987)

American Institute of Steel Construction (AISC) Publications

Steel Construction Manual (2005)

Corps of Engineers, **Engineer Research and Development Center (ERDC)** Computer-Aided Structural Engineering (CASE) Programs

CTWALL (Analysis and Design of Retaining and Flood Walls, 1992)

Corps of Engineers, Engineer Manuals

EM 1110-2-1612 Ice Engineering (2002)

EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures (1992)

EM 1110-2-2502 Retaining and Flood Walls (1989)

EM 1110-2-2902 Conduits, Culverts and Pipes (1998)

EM 1110-2-2906 Design of Pile Foundations (1991)

1.2 DESIGN LOADS

1.2.1 Ice Loads

Ice loads were calculated in accordance with EM 1110-2-1612 assuming an ice sheet thickness of 24 inches, and ice sheet compressive strength of 250 psi on upstream side of dam, and 100 psi on downstream side of dam due to partial breakup of the ice sheet after passing over the dam. The assumed ice load on the concrete dam is the force required to push a sheet of ice up and over the dam, including friction. If the dam must be assumed to be frozen into a large sheet of moving ice, piles beneath the dam would be required to resist the force of the ice.

1.2.2 Seismic Loads

At the project site, the maximum considered earthquake (MCE) short-period spectral acceleration is Ss = 0.12g, and one-second spectral acceleration is S1 = 0.035g. For these accelerations, seismic loads usually do not control design of retaining walls. Therefore, seismic loads were not calculated for this preliminary design, but must be checked for final design of these structures.

1.2.3 Assumed Foundation Design Parameters

Design frost depth = 3.5 feet below finish grade;

Allowable excess soil bearing pressure = 2,000 psf;

Lateral earth pressure coefficients: Ka = 0.5, Kp = 2.0;

Steel piles are assumed to be end-bearing on competent rock.

1.3 STRUCTURAL MATERIALS

Concrete, ACI 318, f'c = 4,000 psi at 28 days;

Concrete Reinforcement, deformed bars conforming to ASTM A 615 Grade 60;

Structural Steel, ASTM A 36;

Reinforced Concrete Pipe (RCP), ASTM C 76, Class to be determined for earth pressures encountered.

1.4 DESCRIPTION OF STRUCTURES

1.4.1 DAM CREST

The existing dam crest is founded on timber piles with timber framework and rock infill and timber facing and steel bars for resistance to ice abrasion. Since construction, damage from ice has been repeatedly repaired by addition of rock to the dam crest. It is proposed to replace the existing dam with a concrete dam. The proposed concrete dam crest was designed to resist the force generated by a moving sheet of ice being force up the sloping face of the dam with friction. If it must be assumed that the dam could be frozen into a moving sheet of ice, piles would be required to resist the resulting ice force.

1.4.2 ROCK RAMP

It is proposed to place boulders approximately 4 to 5 feet in diameter on the ramp with 1 foot of projection above the surrounding fill composed of smaller rock rip rap. Calculation of forces from a moving ice sheet indicate that the passive pressures on the downstream side of the boulder combined with friction against surrounding soil would not be sufficient to resist the force of the ice. However, if the ice sheet is broken into smaller pieces after passing over the dam, pieces of ice would exert much smaller forces and the boulders would be stable.

1.4.3 FISH SCREEN STRUCTURE

The fish screen is a reinforced concrete floodwall founded on steel piles. The intakes consist of 17 openings, each 5 feet in diameter with cast iron sluice gate and galvanized steel trash rack.

1.4.4 OUTLET STRUCTURE

The outlet structure is a reinforced concrete retaining wall with angled wing walls provided at the end of the reinforced concrete pipe bypass.

1.4.5 GATED INTAKE STRUCTURE

The gated intake structure is a reinforced concrete floodwall founded on steel piles. The wall has 17 openings each 5 foot in diameter with cast iron sluice gate and galvanized steel trash rack.

1.4.6 PIPE CULVERTS AND HEADWALL

Five 96-inch diameter reinforced concrete pipe (rcp) culverts can be placed in open excavations for part of their lengths, but would be jacked beneath the railway crossings. The Concrete Pipe Design Manual states that concrete pipe as small as 18-inch diameter and as large as 132-inch diameter have been installed by jacking. Other references suggest 108-inch diameter as approaching the upper limit of practical size for pipe jacking. Reinforced concrete headwalls with angled wing walls are provided at each end of the culverts.

1.4.7 DROP BASIN

The drop basing is reinforced concrete retaining walls with a bottom slab having an 8 foot drop and floor blocks and end sill. A steel sheet pile wall is provided beneath the downstream edge of the slab to cut off seepage and scour.

EM 1110-2-1612
30 Oct 02

6-6. Canadian and American Codes

a. To estimate dynamic ice force F on bridge piers resulting from moving ice, CSA (2000) and AASHTO (1994) codes specify the following:

F = lesser of the F_c or F_b for $D/h<6$

and

$F = F_c$ for $D/h>6$

where

F_c = $C_a pDh$ (horizontal force in when ice floes fail by crushing over full width of the pier)

F_b = $C_n ph^2$ (horizontal force in when ice floes fail in bending against a sloping pier)

D = the pier width

h = the ice thickness

C_a = $(5h/D+1)^{0.5}$ (to account for the aspect ratio effect found in small-scale identation tests)

C_n = $0.5\tan(\alpha+15°)$

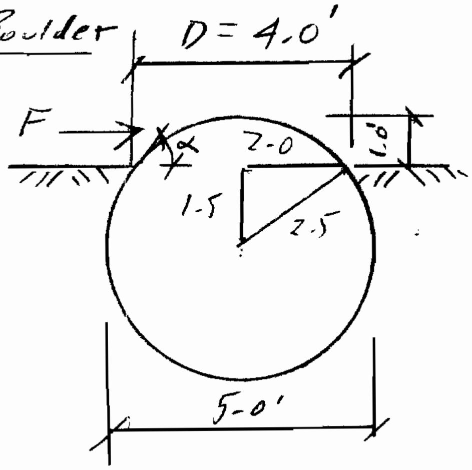
α = slope of the pier from the downstream horizontal (<75°)

p = effective ice crushing pressure for which following values have been recommended.

0.7 MPa (101.5 psi)	Ice breaks up at melting temperature and is somewhat disintegrated.
1.1 MPa (159.5 psi)	Ice breaks up or moves at melting temperature, but the ice moves in large floes and is internally sound.
1.5 MPa (217.5 psi)	Ice breaks up or moves at temperatures considerably below its melting point. Even higher pressures are recommended for ice temperatures 2 or 3°C (35.6 or 37.4°F) below melting temperatures.

b. Further, these codes recommend reducing the dynamic ice force F by 50% of the values derived above for piers in small streams where it is unlikely to encounter large-size floes.

Ice Force on Boulder, $D = 4.0'$



Assume ice $h = 1.0$ (maximum that impacts boulder)

$p = 101.5$ psi (somewhat disintegrated after passing over dam crest)

$D/h = 4.0/1.0 < 6 \Rightarrow$ use lesser of F_c or F_b

$$\alpha = \cos^{-1}\left(\frac{1.5}{2.5}\right) = 53.13°$$

$$F_c = \left(\frac{5(1.0)}{4.0} + 1\right)^{0.5}(101.5\ \text{lb/in}^2)\left(\frac{144\ \text{in}^2/\text{ft}^2}{1000\ \text{lb/k}}\right)(4.0)(1.0) = 87.7^K$$

$$F_b = 0.5\tan(53.13 + 15)(101.5)\left(\frac{144}{1000}\right)(1.0)^2 = 18.2^K \leftarrow \text{controls}$$

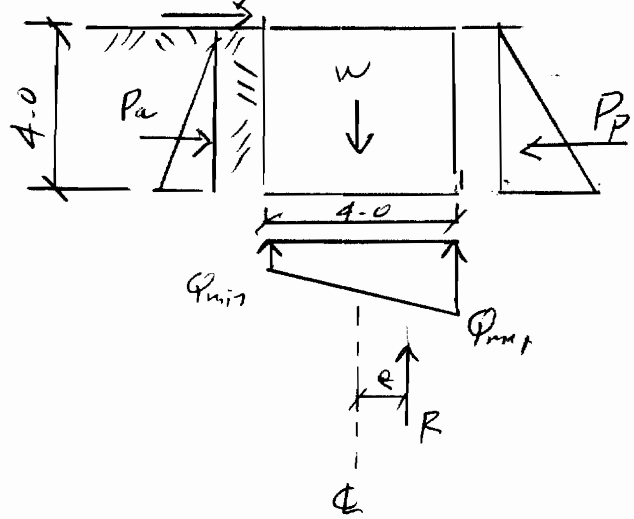
OMAHA DISTRICT	COMPUTATION SHEET		CORPS OF ENGINEERS
PROJECT Yellowstone River		SHEET NO. 3	OF 24
ITEM		BY LEP	DATE June 06
		CHKD. BY	DATE

Ice Force on Boulder (cont)

$$Volume = \frac{4}{3}\pi(2.5)^3 = 65.45\ ft^3$$

Equivalent cube $b = \sqrt[3]{65.45} = 4.03\ ft$, say $4.0\ ft$

Ice $18.2^K / 4.0 = 4.55^{K/f}$



Assume:

boulder $\gamma = 150\ pcf$

Soil $\gamma_s = 120\ pcf$

$K_a = 0.40$, $K_p = 2.50$ } $\phi = 25^\circ$

friction $\mu = 0.50$

$$\text{buoyant } W = \frac{65.45(0.15 - 0.0624)}{4.0} = 1.43\ K/f$$

$$P_a = K_a(\gamma_s - \gamma_w)h^2/2 = 0.4(0.12 - 0.0624)(4.0)^2/2 = 0.18\ K/F$$

$$P_p = 2.5(0.12 - 0.0624)(4.0)^2/2 = 1.15\ K/f$$

$$\text{Sliding F.S.} = \frac{1.15 + 0.5(1.43)}{4.55 + 0.18} = 0.40 < 1.5\ \underline{NG}$$

$$\text{Overturning F.S.} = \frac{1.43(4.0/2) + (1.15 - 0.18)(4.0/3)}{4.55(4.0)} = 0.23 < 1.5\ \underline{NG}$$

Sliding & Overturning OK if ice force $\leq 0.69\ K/f$

$\Rightarrow F_b \leq 4.0(0.69) = 2.769\ K/F$

$\Rightarrow \alpha \leq 5.75^\circ$ occurs 1/8" below top of sphere

OMAHA DISTRICT	COMPUTATION SHEET		CORPS OF ENGINEERS
PROJECT Yellowstone River		SHEET NO. 4	OF 24
ITEM		BY LEP	DATE June 06
		CHKD. BY	DATE

Ice Force on Dam Crest

Assume vertical face to determine upper limit on ice force. In reality, sloped face will push ice upward to pass over the crest, and reduce horizontal force.

$$D/h = 682/2 > 6 \Rightarrow F_b \text{ is not applicable}$$

assume ice $h = 2.0$ ft

$p = 217.5$ psi

$$F_c = \underbrace{\left(\frac{5(2.0)}{682} + 1\right)^{0.5}}_{\approx 1.0 \text{ for width effect}} \left(217.5 \text{ lb/in}^2\right)\left(\frac{144 \text{ in}^2/\text{ft}^2}{1000 \text{ lb/k}}\right)(2.0)(1.0) = 62.6 \text{ k/ft}$$

calculate force per foot width

c. Bending Failure.

(1) *Sloping Structure.* When a floating ice sheet moves against an upward or downward sloping structure, the sheet is pushed either up or down, and breaks by bending into blocks. As the ice sheet continues to be pushed up or down, the broken slabs are further broken into slabs that are typically 4 to 8 times the ice thickness. The force on the structure is limited by the amount required to fail the ice sheet in bending and to overcome the weight and frictional forces of the broken ice blocks. If the structure is narrow, the broken pieces of ice may be able to go around the structure. For wide structures, the broken pieces of ice either ride up to clear over the top of the structure or forms an ice rubble mound. Procedures to estimate ice forces on sloping and conical structures are given in textbooks (Ashton 1986, Cammaert and Muggerdige 1988, Sanderson 1988).

(*a*) API (1995) gives equations for determining the ice forces on a sloping structure, where the broken ice pieces are assumed to ride up the sloping surface and fall off into the water on the other side. Figure 6-6 shows forces during an interaction of a floating ice sheet of thickness h being pushed against a wide sloping surface at an angle α with the horizontal. If the ice blocks are lifted up a height z along the sloping surface, the weight of the broken ice sheet on the sloping surface has a magnitude per unit width of $W=\rho ghz/\sin\alpha$, where ρg is the specific weight of ice, and h is the ice thickness. The normal force per unit width on the surface is $N=W\cos\alpha$, and the tangential force along the surface is μN, where μ is the coefficient of friction between the surface and the ice. As shown in Figure 6-6, the force T acting between the broken ice on the sloping surface and the top of the floating ice sheet has a magnitude per unit width of $T=W(\sin\alpha + \mu\cos\alpha)$.

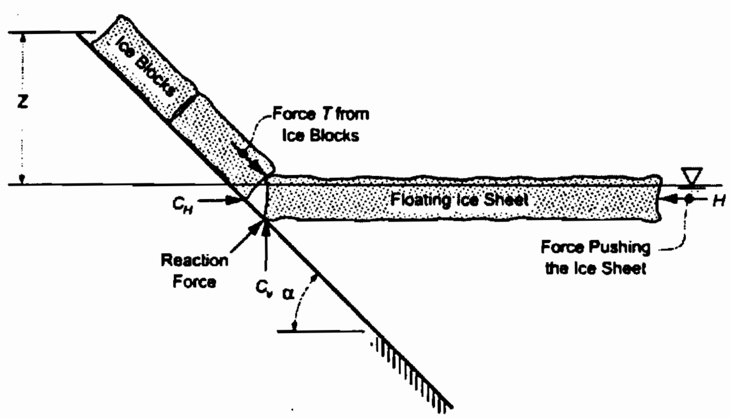


Figure 6-6. forces during an interaction of a floating ice sheet of thickness *h* being pushed against a wide sloping surface at an angle α with the horizontal.

(*b*) The reaction force (Figure 6-6) acting on the contact between the sloping structure and the advancing ice sheet has components C_H and C_V in the horizontal and vertical directions, respectively. The total horizontal force per unit width is given by $C_H + T\cos\alpha$. As the structure pushes the advancing ice sheet up, the vertical force acting at the end of the ice sheet has a magnitude per unit width equal to $C_V - T\sin\alpha$.

(*c*) Under the assumption that there is no moment acting on the floating ice sheet, the vertical force component C_V per unit width required to break the floating ice sheet and push it up is given by:

$$C_V = \frac{\sigma h^2 + 6\ell e^{-\frac{\pi}{4}} T\sin\alpha + Th\cos\alpha}{6\ell e^{-\frac{\pi}{4}} - h\tan(\alpha + \arctan\mu)}, \qquad (6\text{-}14)$$

where

σ = flexural strength of ice sheet
h = ice thickness
α = angle between the sloping surface and the horizontal
ℓ = $[Eh^3/\{12(1-\nu^2)\rho_w g\}]^{1/4}$ (the characteristic length of floating ice sheet)
E = effective elastic modulus of ice
ν = Poisson's ratio of ice
$\rho_w g$ = specific weight of ice.

For typical bending rates, the effective elastic modulus of freshwater ice is in the range of 1–3 GPa (1.45×10^5 to 4.35×10^5 psi), and Poisson's ratio is about 1/3. The range of the coefficient of friction between ice and a structure is between 0.1 for freshly coated surfaces and 0.5 for rusty, rough surfaces. There are, at present, no guidelines available for the coefficient of friction on rough surfaces or on riprap protected surfaces.

(*d*) The horizontal force C_H per unit width from the structure on the ice sheet is

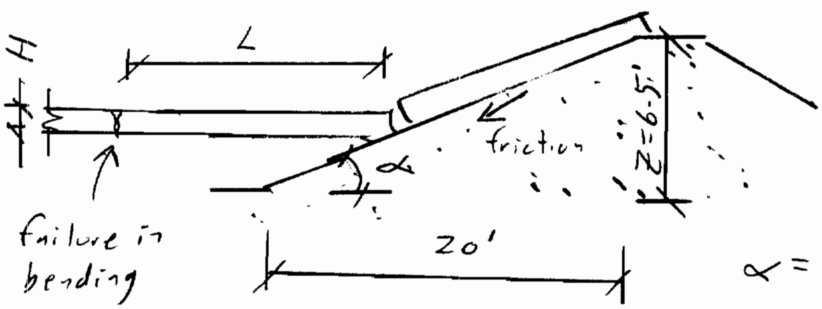
$$C_H = C_V \tan(\alpha + \arctan\mu) \qquad (6\text{-}15)$$

The total force H per unit width generated during the interaction to break the ice sheet at a distance away from the contact zone and to push the broken ice block along the sloping surface is given by:

$$H = C_H + T\cos\alpha$$

OMAHA DISTRICT	COMPUTATION SHEET	CORPS OF ENGINEERS	
PROJECT		SHEET NO. 7	OF 24
ITEM		BY LEP	DATE June 06
		CHKD. BY	DATE

Ice Bending Failure on Sloping Dam Crest



$$\alpha = \tan^{-1}\left(\frac{6.5}{20}\right) = 18^\circ$$

Assumed

Ice flexural strength $\sigma = 250$ psi [Ref: API, "Recommended Practice ... Structures for Arctic Conditions]

Ice poisson's ratio, $\nu = 0.33$

Ice modulus of elasticity $E = 300{,}000$ psi

Ice sheet thickness, $H = 24$ in [Ref: CRREL Report]

Ice density $\gamma = 60$ pcf, Ice-concrete friction $\mu = 0.50$

Calculated Forces

$$W = \frac{\gamma H Z}{\sin\alpha} = \frac{(60\ \mathrm{lb/ft^3})(2.0\ \mathrm{ft})(6.5\ \mathrm{ft})}{\sin\alpha} = 2524\ \mathrm{plf}$$

$$T = W(\sin\alpha + \mu\cos\alpha) = 2524[\sin(18) + 0.5\cos(18)] = 1980\ \mathrm{plf}$$

$$L = \left[\frac{Eh^3}{12(1-\nu^2)\gamma_w}\right]^{0.25} = \left[\frac{(300{,}000\ \mathrm{lb/in^2})(24\ \mathrm{in})^3}{12[1-(0.33)^2](62.4\ \mathrm{lb/ft^3})(12\ \mathrm{in/ft})}\right]^{0.25} = 26.8\ \mathrm{ft}$$

$$C_V = \frac{\sigma H^2 + 6Le^{-\pi/4} T \sin\alpha + TH\cos\alpha}{6Le^{-\pi/4} - H\tan(\alpha + \tan^{-1}(\mu))}$$

$$e^{-\pi/4} = 0.456$$

$$C_V = \frac{(250\,\text{lb/in}^2)(24\,\text{in})^2 + 6(26.8\,\text{ft})(0.456)(1980\,\text{lb/ft})\sin(18) + (1980\,\text{lb/ft})(2.0\,\text{ft})\cos(18)}{6(26.8\,\text{ft})(0.456) - (2.0\,\text{ft})\tan[18 + \tan^{-1}(0.5)]}$$

$$C_V = \frac{144{,}000\,\text{lb} + 44{,}864\,\text{lb} + 3{,}766\,\text{lb}}{73.32\,\text{ft} - 1.97\,\text{ft}} = 2699.8\,\text{lb/ft},$$

say $2700\,\text{lb/ft}$

$$C_H = C_V \tan(\alpha + \tan^{-1}\mu) = 2700\tan[18 + \tan^{-1}(0.5)] = 2659\,\text{lb/ft}$$

$$H = C_H + T\cos\alpha = 2659 + 1980\cos(18) = 4{,}542\,\text{lb/ft}$$

$$V = C_V - T\sin\alpha = 2700 - 1980\sin(18) = 2{,}088\,\text{lb/ft}$$

9/24

****************** Echoprint of Input Data ******************

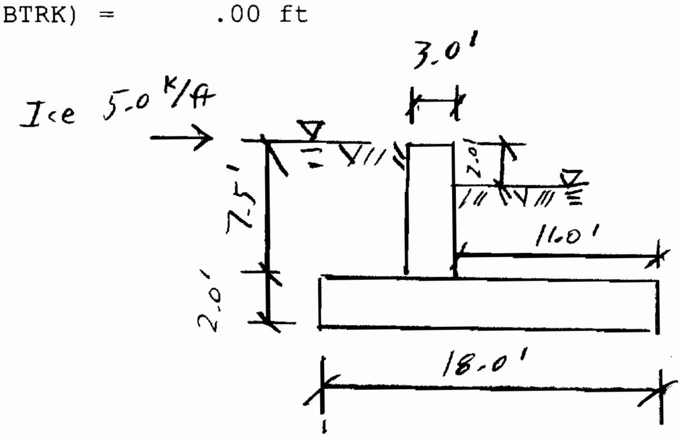
Date: **/06/27 Time: 11.32.44

Structural geometry data:

```
  Elevation of top of stem (ELTS)           =  1989.00 ft
  Height of stem (HTS)                      =     7.50 ft
  Thickness top of stem (TTS)               =     3.00 ft
  Thickness bottom of stem (TBS)            =     3.00 ft
  Dist. of batter at bot. of stem (TBSR)=        .00 ft
  Depth of heel (THEEL)                     =     2.00 ft
  Distance of batter for heel (BTRH)        =      .00 ft
  Depth of toe (TTOE)                       =     2.00 ft
  Width of toe (TWIDTH)                     =    11.00 ft
  Distance of batter for toe (BTRT)         =      .00 ft
  Width of base (BWIDTH)                    =    18.00 ft
  Depth of key (HK)                         =      .00 ft
  Width of bottom of key (TK)               =      .00 ft
  Dist. of batter at bot. of key (BTRK) =          .00 ft
```

Structure coordinates:

x (ft)	y (ft)
.00	1979.50
.00	1981.50
4.00	1981.50
4.00	1989.00
7.00	1989.00
7.00	1981.50
18.00	1981.50
18.00	1979.50



Dam

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
30.00	.000	.120	.125	.00	1989.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)

1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1496.00	1989.00
2	4.00	1989.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
30.00	.000	.120	.125	1987.00	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	7.00	1987.00
2	507.00	1987.00

Foundation property data:
phi for soil-structure interface = 30.00 (deg)
c for soil-structure interface = .000 (ksf)
phi for soil-soil interface = 30.00 (deg)
c for soil-soil interface = .000 (ksf)

Water data:
Driving side elevation = 1989.00 ft
Resisting side elevation = 1989.00 ft
Unit weight of water = .0624 kcf
Seepage pressures computed are hydrostatic.

Horizontal line load data:

Elevation (ft)	Force (kips)
1989.00	5.00 ← Ice

Minimum required factors of safety:
Sliding FS = 1.33
Overturning = 75.00% base in compression
Flotation FS = 1.30

Crack options:
- Crack depth is to be calculated
- Computed cracks *will* be filled with water

11/24

```
Strength mobilization factor =    .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

*Do* iterate in overturning analysis.



***** Summary of Results *****

***************    *** Satisfied ***
* Overturning *    Required base in comp. =  75.00 %
***************    Actual base in comp.   =  83.97 %
                   Overturning ratio      =   1.35

Xr (measured from toe) =   5.04 ft
Resultant ratio        =  .2799
Stem ratio             =  .6111
Base pressure at x=  15.11 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.4277 ksf

***********    *** Satisfied ***
* Sliding *    Min. Required =   1.33
***********    Actual FS     =   1.88

*************    *** Satisfied ***
* Flotation *    Min. Required =   1.30
*************    Actual FS     =   3.96


********************** Output Results **********************


Date: **/06/27                               Time: 11.32.44


***************************
**  Overturning Results  **
***************************

Solution converged in  2 iterations.

SMF used to calculate K's =    .6667
Alpha for the SMF          = -55.7255
Calculated earth pressure coefficients:
  Driving side at rest K     =    .4714
  Driving side at rest Kc    =    .6866
  Resisting side at rest K  =    .5000
  Resisting side at rest Kc =    .7071
  At-rest K's for resisting side calculated.

Depth of cracking =      .00 ft
```

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
1989.00	.0000
1979.50	.5928

Earth pressures:

Elevation (ft)	Pressure (ksf)
1989.00	.0000
1979.50	.2804

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
1989.00	.0000
1987.00	.1248
1979.50	.5928

Earth pressures:

Elevation (ft)	Pressure (ksf)
1987.00	.0000
1979.50	.2348

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
.00	.5928
2.89	.5928
18.00	.5928

** Forces and moments **

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight...........	8.775		-10.35	-90.79
Structure, driving side:				
Moist soil.................	.000		.00	.00
Saturated soil.............	3.750		-16.00	-60.00
Water above structure......	.000		.00	.00
Water above soil...........	.000		.00	.00

External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads......		5.000	9.50	47.50
Structure, resisting side:				
Moist soil.................	.000		.00	.00
Saturated soil.............	7.563		-5.50	-41.59
Water above structure......	.000		.00	.00
Water above soil...........	1.373		-5.50	-7.55
Driving side:				
Effective earth loads......		1.332	3.21	4.27
Shear (due to delta).......	.000		.00	.00
Horiz. surcharge effects...		.000	.00	.00
Water loads................		2.816	3.19	8.97
Resisting side:				
Effective earth loads......		-.880	2.54	-2.24
Water loads................		-2.816	3.19	-8.97
Foundation:				
Vertical force on base.....	-10.790		-5.04	54.36
Shear on base..............		-5.451	.00	.00
Uplift.....................	-10.670		-9.00	96.03
** Statics Check ** SUMS =	.000	.000		.00

```
Angle of base        =      .00 degrees
Normal force on base =   10.790 kips
Shear force on base  =    5.451 kips
Max. available shear force =    6.230 kips

Base pressure at x=  15.11 ft from toe =    .0000 ksf
Base pressure at toe                   =   1.4277 ksf

Xr (measured from toe) =     5.04 ft
Resultant ratio        =    .2799
Stem ratio             =    .6111
Base in compression    =    83.97 %
Overturning ratio      =     1.35

Volume of concrete =     2.17 cubic yds/ft of wall
```

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

```
***********************
**  Sliding Results  **
***********************
```

Solution converged. Summation of forces = 0.

```
           Horizontal   Vertical
```

Wedge Number	Loads (kips)	Loads (kips)
1	.000	.000
2	4.875	1.373
3	.000	1.258

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.0000	.5928		
2			.0000	.5928
2			2.8853	.5928
2			18.0000	.5928
3	.1248	.5928		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 1979.50 ft
Point 2 (right), x = 18.00 ft, y = 1979.50 ft

Depth of cracking = .00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	-53.712	11.786	4.142	11.786	3.493
2	.000	18.000	20.088	18.000	10.670
3	36.656	12.563	4.724	12.563	4.507

Wedge number	Net force (kips)
1	-4.358
2	-1.559
3	5.917
SUM =	.000

```
+----------------------------+
| Factor of safety =   1.878 |
+----------------------------+
```

```
************************
**  Flotation Results  **
************************
```

Loads:

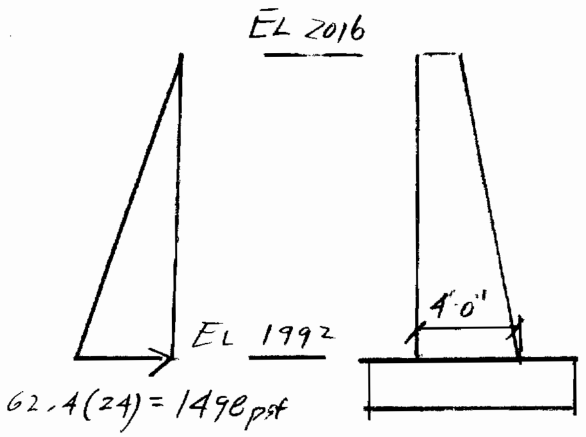
Ws loads:
Structure .... 8.775 kips

```
  Moist soil on driving ..................      .000 kips
  Saturated soil on driving (submerged)....    1.878 kips
  Moist soil on resisting ................      .000 kips
  Saturated soil on resisting (submerged)..    3.787 kips
                               Sum .........   14.440 kips
Wc loads:
  No Wc loads present for retaining wall ..     .000 kips
Wg loads:
  Water above structure on driving side ...     .000 kips
  Water above soil on driving side ........     .000 kips
  Water below soil on driving side ........    1.872 kips
  Water above structure on resisting side .     .000 kips
  Water above soil on resisting side ......    1.373 kips
  Water below soil on resisting side ......    3.775 kips
                               Sum .........    7.020 kips
Surcharge loads:
  Surcharge loads on structure ...........      .000 kips
Uplift loads:
  Uplift load on base of structure ........   10.670 kips
============================================================

Effective unit wt. of water, driving side   =    .0624 kcf
Effective unit wt. of water, resisting side =    .0624 kcf


      (Ws + Wc + S)
SFf = -------------
        (U - Wg)

+------------------------------+
| Factor of safety =    3.956 |
+------------------------------+
```



$V = 1498(24)/2 = 17{,}971$ plf

$M = 1498(24)^2/6 = 143{,}770$ ft·lb/ft

$V_u = 1.7(17971) = 30{,}550$

$M_u = 1.3(1.7)(143.77) = 318$ k·ft/ft

Base of wall $d \approx 48 - 3 - 1.0 - \frac{1.0}{2} = 43.5$

$\phi V_c = 0.85(2)\sqrt{4000}(12)(43.5) = 56{,}115 > V_u$ OK

Req $R_n = \dfrac{M_u}{\phi b d^2} = \dfrac{318(12{,}000)}{0.9(12)(43.5)^2} = 186.7 \Rightarrow$ Req. $\rho = 0.00320$

$= 11\%\ \rho_{bd}$

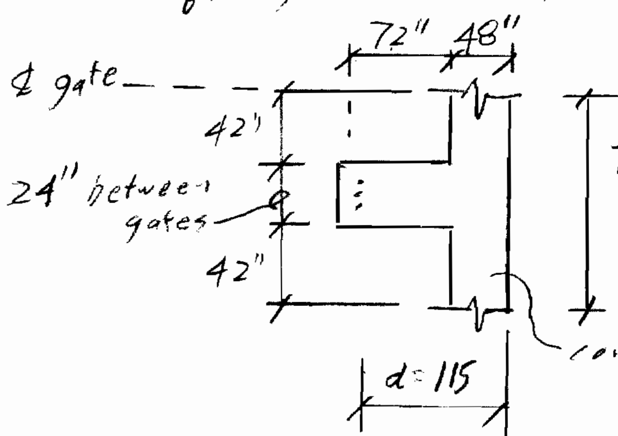
OK

Req. $A_s = 0.00320(12)(43.5) \cong 1.67\ \text{in}^2/\text{ft} \Rightarrow$ #9 @ 7"

$A_s = 1.71$

$\phi M_n = 325$

$> M_u$ OK



$V_u = 9(30{,}550) = 275{,}000^k$

$M_u = 9(318) = 2862$ k·ft

$\phi V_c = 0.85(2)\sqrt{4000}(108)(115) = 1{,}335{,}000 > V_u$ OK

$V_u < 0.5\phi V_c \Rightarrow$ no shear reinf. req'd.

$R_n = \dfrac{2862(12{,}000)}{0.9(108)(115)^2} = 26.7 \Rightarrow$ Req $\rho = 0.001447 < \rho_{min}$

Req $A_s = 0.001447(\frac{4}{3})(108)(115) = 7.40\ \text{in}^2$

$\Rightarrow$ 8 - #9

Gated Diversion Structure

Try Steel Piles HP12x53, assume end bearing @ 20' depth

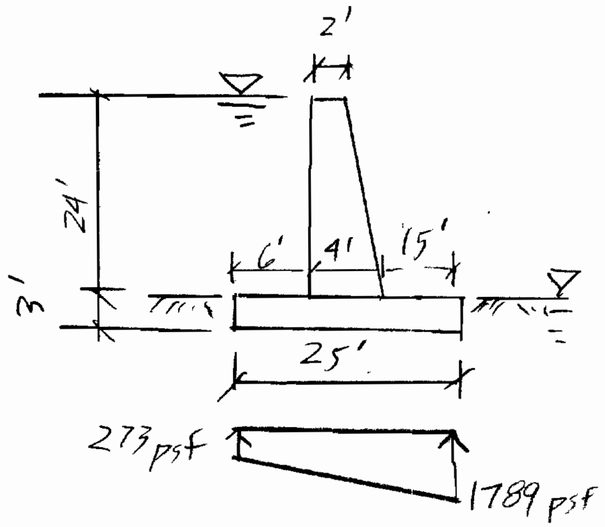
$A = 15.5 in^2$

$r_y = 2.86 in$

$Kl/r = 1.0(20)(12)/2.86 = 83.92 < C_c = 126.1$

$\frac{Kl/r}{C_c} = \frac{83.92}{126.1} = 0.6655 \Rightarrow F_a = 0.414(36) = 14.9 \text{ Ksi}$

$P_{allow} = 14.9(15.5) = 231^K$



program CTWAL

Min trib area per pile $\approx \frac{231,000}{1789} = 129 ft^2$

If 3 rows of piles,

$Sp. = \frac{129(3)}{25} = 15'$ ← use closer spacing for horiz. load to battered piles

****************** Echoprint of Input Data ******************

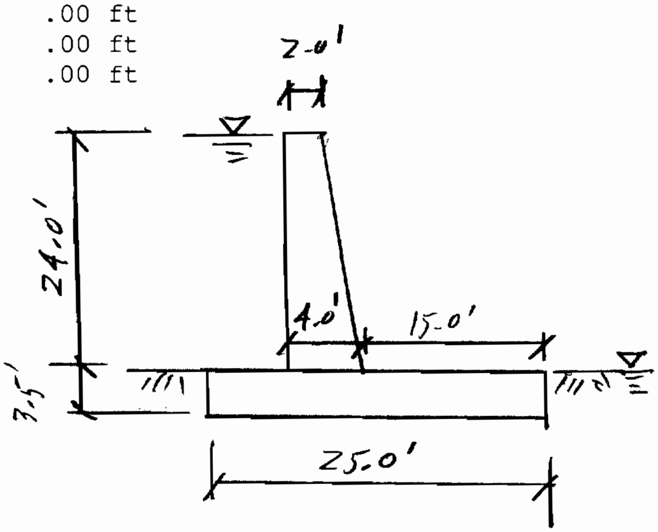
Date: **/06/27 Time: 11.52.05

Structural geometry data:

Elevation of top of stem (ELTS)	=	2016.00 ft
Height of stem (HTS)	=	24.00 ft
Thickness top of stem (TTS)	=	2.00 ft
Thickness bottom of stem (TBS)	=	4.00 ft
Dist. of batter at bot. of stem (TBSR)	=	2.00 ft
Depth of heel (THEEL)	=	3.50 ft
Distance of batter for heel (BTRH)	=	.00 ft
Depth of toe (TTOE)	=	3.50 ft
Width of toe (TWIDTH)	=	15.00 ft
Distance of batter for toe (BTRT)	=	.00 ft
Width of base (BWIDTH)	=	25.00 ft
Depth of key (HK)	=	.00 ft
Width of bottom of key (TK)	=	.00 ft
Dist. of batter at bot. of key (BTRK)	=	.00 ft

Structure coordinates:

x (ft)	y (ft)
.00	1988.50
.00	1992.00
6.00	1992.00
6.00	2016.00
8.00	2016.00
10.00	1992.00
25.00	1992.00
25.00	1988.50



Gated Intake

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Delta (deg)	Elev. soil (ft)
30.00	.000	.120	.125	.00	2016.00

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)

1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1494.00	2016.00
2	6.00	2016.00

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
30.00	.000	.120	.125	1992.00	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	25.00	1992.00
2	525.00	1992.00

Foundation property data:
phi for soil-structure interface = 30.00 (deg)
c for soil-structure interface = .000 (ksf)
phi for soil-soil interface = 30.00 (deg)
c for soil-soil interface = .000 (ksf)

Water data:
Driving side elevation = 2016.00 ft
Resisting side elevation = 1992.00 ft
Unit weight of water = .0624 kcf
Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
Sliding FS = 1.50
Overturning = 100.00% base in compression
Flotation FS = 1.50

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

```
*Do* iterate in overturning analysis.



***** Summary of Results *****

***************    *** Satisfied ***
* Overturning *    Required base in comp. = 100.00 %
***************    Actual base in comp.   = 100.00 %
                   Overturning ratio      =   1.48

Xr (measured from toe) =   9.44 ft
Resultant ratio        =  .3774
Stem ratio             =  .6000
Base pressure at heel =     .2726 ksf
Base pressure at toe  =    1.7888 ksf

*** Warning ***  The maximum available shear along the
base of the structure has been exceeded!

***********    *** Not Satisfied ***
* Sliding *    Min. Required =   1.50
***********    Actual FS     =    .74

  To increase stability try one or a combination
  of the following:
    1.  Increase the base width
    2.  Slope the base of the structure
    3.  Lower the wall base
    4.  Add a key

*************    *** Satisfied ***
* Flotation *    Min. Required =   1.50
*************    Actual FS     =   3.34


********************** Output Results **********************


Date: **/06/27                                 Time: 11.52.05


***************************
**  Overturning Results  **
***************************

Solution converged in  1 iterations.

SMF used to calculate K's =    .6667
Alpha for the SMF          = -55.6919
Calculated earth pressure coefficients:
  Driving side at rest K     =    .4714
  Driving side at rest Kc    =    .6866
  Resisting side at rest K  =    .5000
  Resisting side at rest Kc =    .7071
```

At-rest K's for resisting side calculated.

Depth of cracking = .00 ft

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
2016.00	.0000
1988.50	.9806

Earth pressures:

Elevation (ft)	Pressure (ksf)
2016.00	.0000
1988.50	1.1583

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
1992.00	.0000
1988.50	.3120

Earth pressures:

Elevation (ft)	Pressure (ksf)
1992.00	.0000
1988.50	.0628

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
.00	.9806
25.00	.3120

** Forces and moments **

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight...........	23.925		-14.73	-352.46
Structure, driving side:				
Moist soil.................	.000		.00	.00
Saturated soil.............	18.000		-22.00	-396.00
Water above structure......	.000		.00	.00

Water above soil..........	.000		.00	.00
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads......		.000	.00	.00
Structure, resisting side:				
Moist soil................	.000		.00	.00
Saturated soil............	.000		.00	.00
Water above structure......	.000		.00	.00
Water above soil..........	.000		.00	.00
Driving side:				
Effective earth loads......		15.926	9.16	145.81
Shear (due to delta).......	.000		.00	.00
Horiz. surcharge effects...		.000	.00	.00
Water loads................		13.483	9.16	123.50
Resisting side:				
Effective earth loads......		-.110	1.12	-.12
Water loads................		-.546	1.18	-.65
Foundation:				
Vertical force on base.....	-25.768		-9.44	243.13
Shear on base..............		-28.754	.00	.00
Uplift.....................	-16.157		-14.66	236.79
** Statics Check ** SUMS =	.000	.000		.00

Angle of base = .00 degrees
Normal force on base = 25.768 kips
Shear force on base = 28.754 kips
Max. available shear force = 14.877 kips

*** Warning *** The maximum available shear along the base of the structure has been exceeded!

Base pressure at heel = .2726 ksf
Base pressure at toe = 1.7888 ksf

Xr (measured from toe) = 9.44 ft
Resultant ratio = .3774
Stem ratio = .6000
Base in compression = 100.00 %
Overturning ratio = 1.48

Volume of concrete = 5.91 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	.000	.000
2	.000	.000
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.0000	.9806		
2			.0000	.9806
2			25.0000	.3120
3	.0000	.3120		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 1988.50 ft
Point 2 (right), x = 25.00 ft, y = 1988.50 ft

Depth of cracking = .00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	-64.091	30.573	22.960	30.573	14.990
2	.000	25.000	41.925	25.000	16.157
3	26.245	7.915	1.553	7.915	1.235

Wedge number	Net force (kips)
1	-21.545
2	20.078
3	1.466
SUM =	-.001

```
+----------------------------+
| Factor of safety =    .741 |
+----------------------------+
```

```
************************
**  Flotation Results  **
************************
```

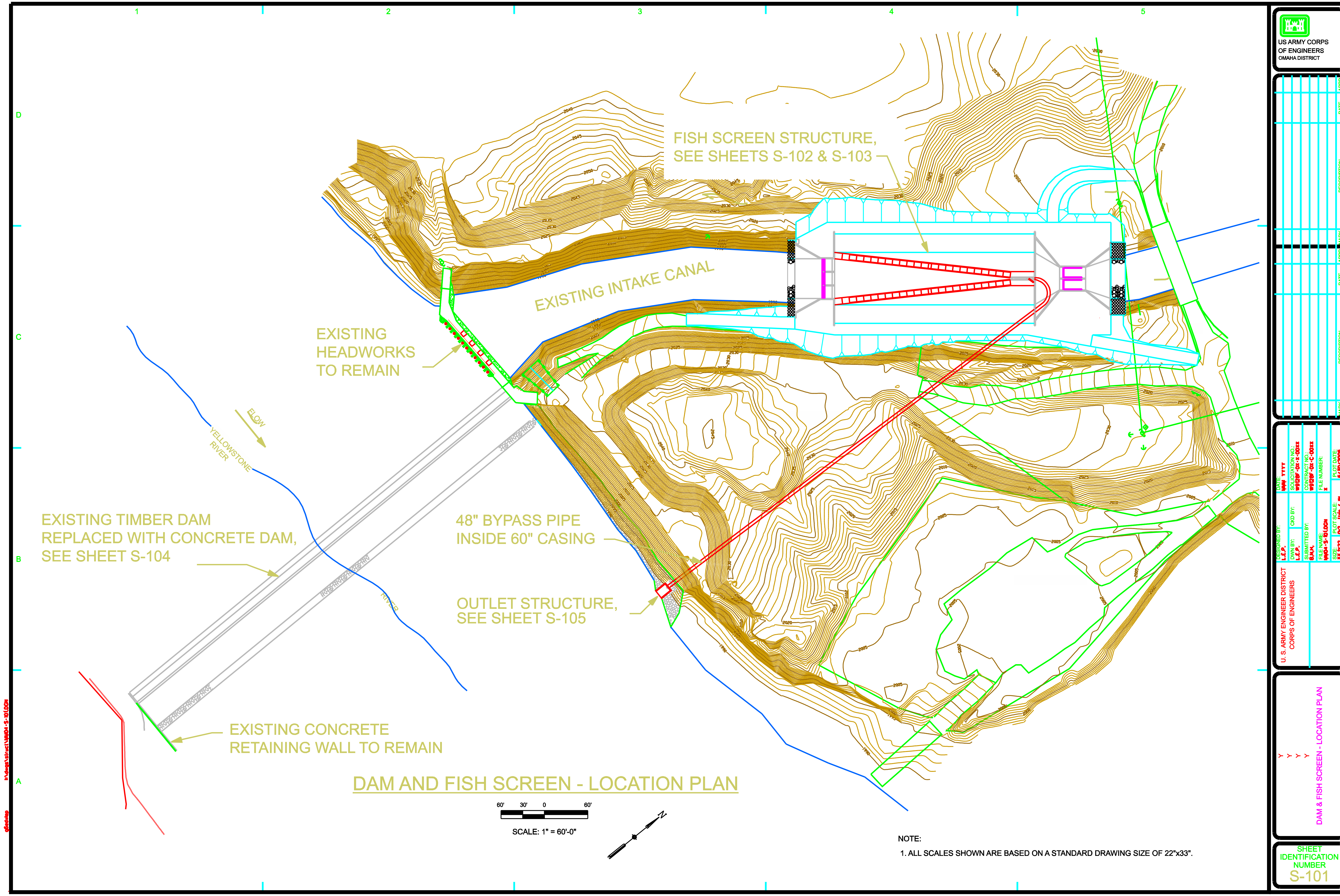
Loads:

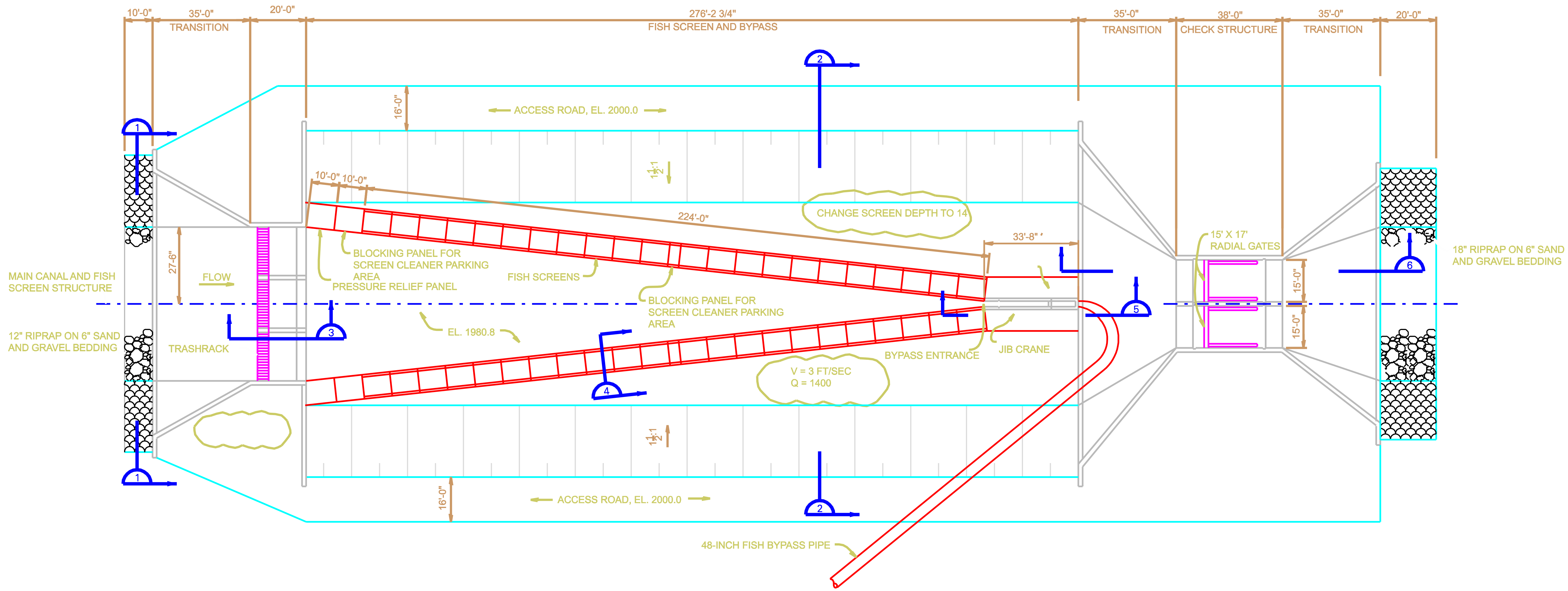
```
==============================================================
Ws loads:
  Structure ...............................   23.925 kips
  Moist soil on driving ..................     .000 kips
  Saturated soil on driving (submerged)....   12.865 kips
  Moist soil on resisting ................     .000 kips
  Saturated soil on resisting (submerged)..     .000 kips
                              Sum .........   36.790 kips
Wc loads:
  No Wc loads present for retaining wall ..     .000 kips
Wg loads:
  Water above structure on driving side ...     .000 kips
  Water above soil on driving side ........     .000 kips
  Water below soil on driving side ........    5.135 kips
  Water above structure on resisting side .     .000 kips
  Water above soil on resisting side ......     .000 kips
  Water below soil on resisting side ......     .000 kips
                              Sum .........    5.135 kips
Surcharge loads:
  Surcharge loads on structure ...........     .000 kips
Uplift loads:
  Uplift load on base of structure ........   16.157 kips
==============================================================

Effective unit wt. of water, driving side   =    .0357 kcf
Effective unit wt. of water, resisting side =    .0891 kcf


      (Ws + Wc + S)
SFf = -------------
        (U - Wg)

+------------------------------+
| Factor of safety =    3.338 |
+------------------------------+
```


US ARMY CORPS
OF ENGINEERS
OMAHA DISTRICT
FISH SCREEN STRUCTURE,
SEE SHEETS S-102 & S-103
EXISTING INTAKE CANAL
EXISTING
HEADWORKS
TO REMAIN
YELLOWSTONE
RIVER
FLOW
EXISTING TIMBER DAM
REPLACED WITH CONCRETE DAM,
SEE SHEET S-104
48" BYPASS PIPE
INSIDE 60" CASING
OUTLET STRUCTURE,
SEE SHEET S-105
EXISTING CONCRETE
RETAINING WALL TO REMAIN
DAM AND FISH SCREEN - LOCATION PLAN
60' 30' 0 60'
SCALE: 1" = 60'-0"
NOTE:
1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"x33".
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
DAM & FISH SCREEN - LOCATION PLAN
SHEET
IDENTIFICATION
NUMBER
S-101



FISH SCREEN STRUCTURE - PLAN

20' 10' 0 20'

SCALE: 1" = 20'-0"

NOTE:

1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"X33".
2. SEE SHEET S-103 FOR SECTIONS OF FISH SCREEN STRUCTURE.

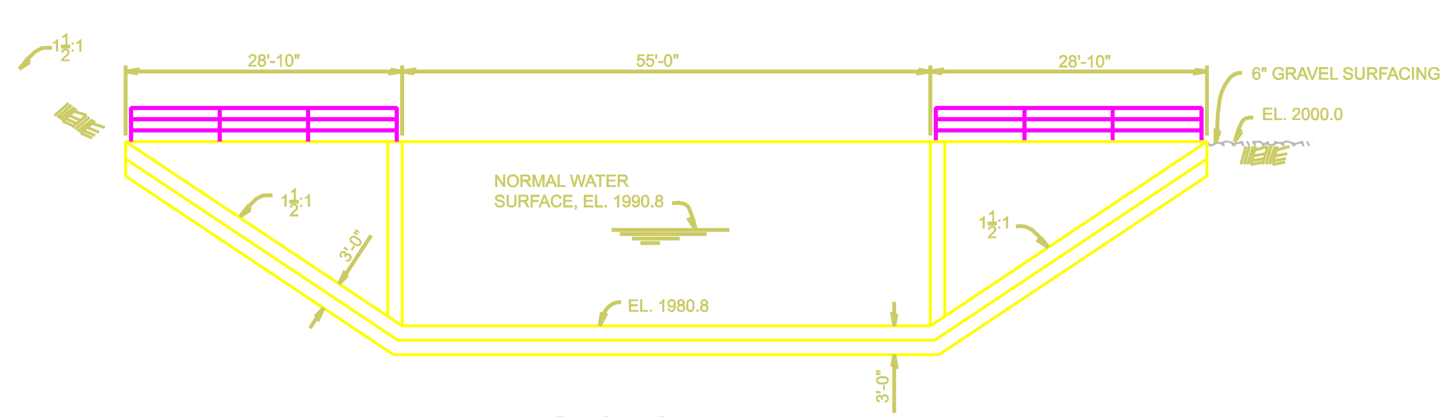
US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

DESIGNED BY: L.E.P. | DWN BY: L.E.P. | CKD BY: | SUBMITTED BY: B.A.H. | FILE NAME: WW04-S-102.DGN | SIZE: 33.1x22 | PLOT SCALE: 0:2 1 in / IN | DATE: MM YYYY | SOLICITATION NO.: W9128F-0X-X-00XX | CONTRACT NO.: W9128F-0X-C-00XX | FILE NUMBER: X | PLOT DATE: 6/30/2006

FISH SCREEN STRUCTURE - PLAN

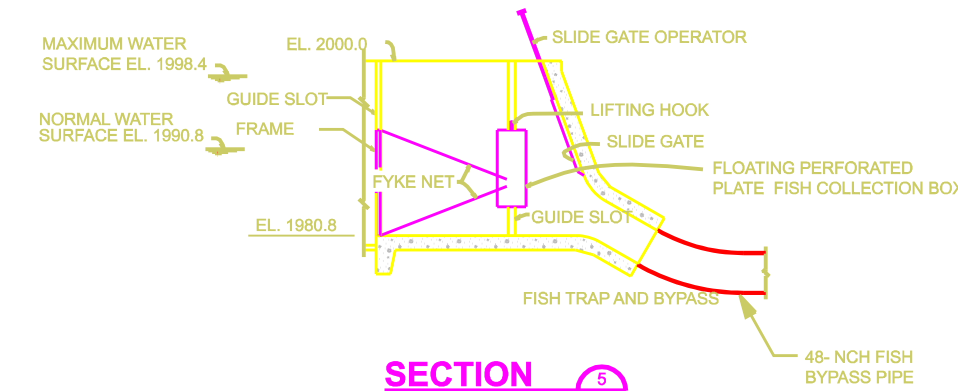
SHEET IDENTIFICATION NUMBER
S-102



SECTION 1

10' 5' 0 10'

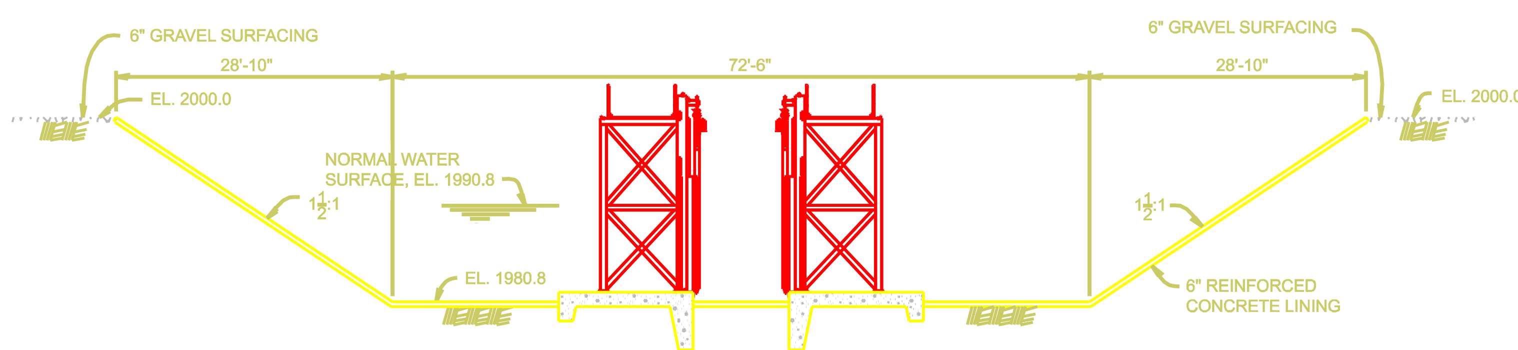
SCALE: 1" = 10'-0"



SECTION 5

10' 5' 0 10'

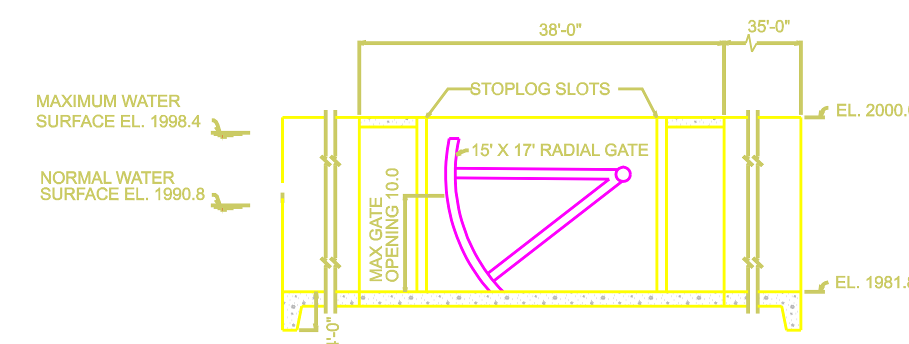
SCALE: 1" = 10'-0"



SECTION 2

10' 5' 0 10'

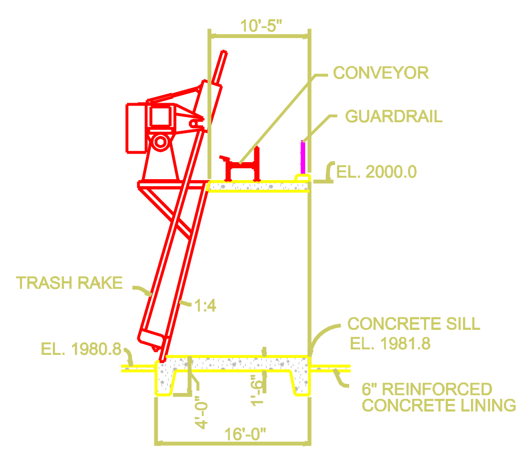
SCALE: 1" = 10'-0"



SECTION 6

10' 5' 0 10'

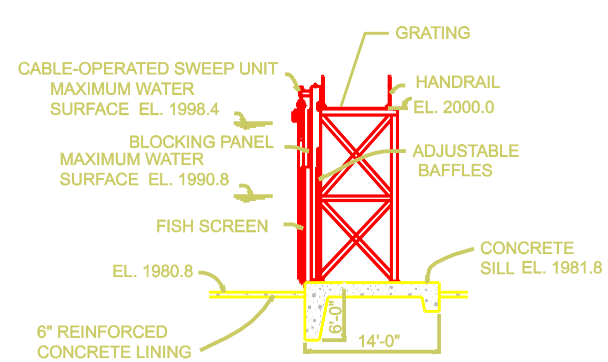
SCALE: 1" = 10'-0"



SECTION 3

10' 5' 0 10'

SCALE: 1" = 10'-0"



SECTION 4

10' 5' 0 10'

SCALE: 1" = 10'-0"

NOTE:

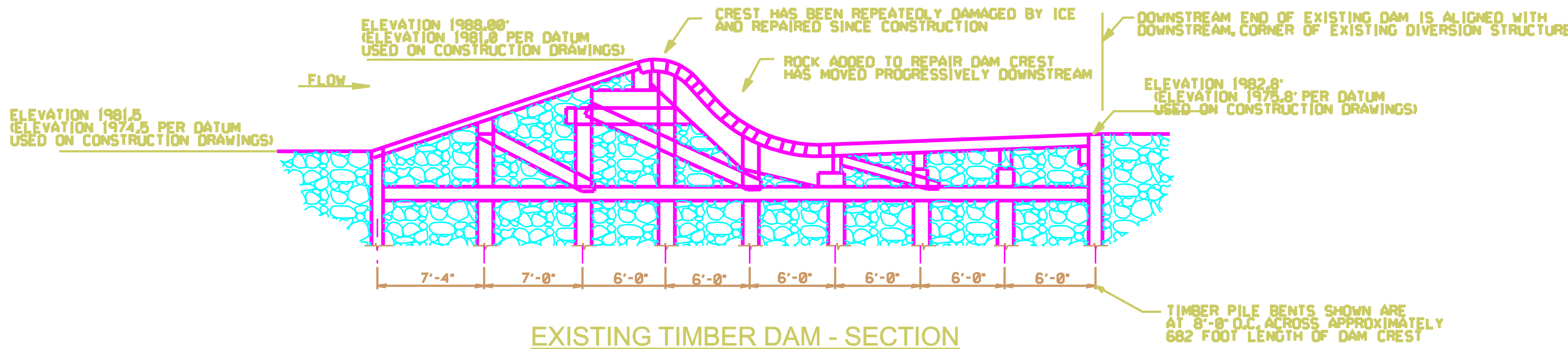
1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"X33".

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

FISH SCREEN STRUCTURE - SECTIONS

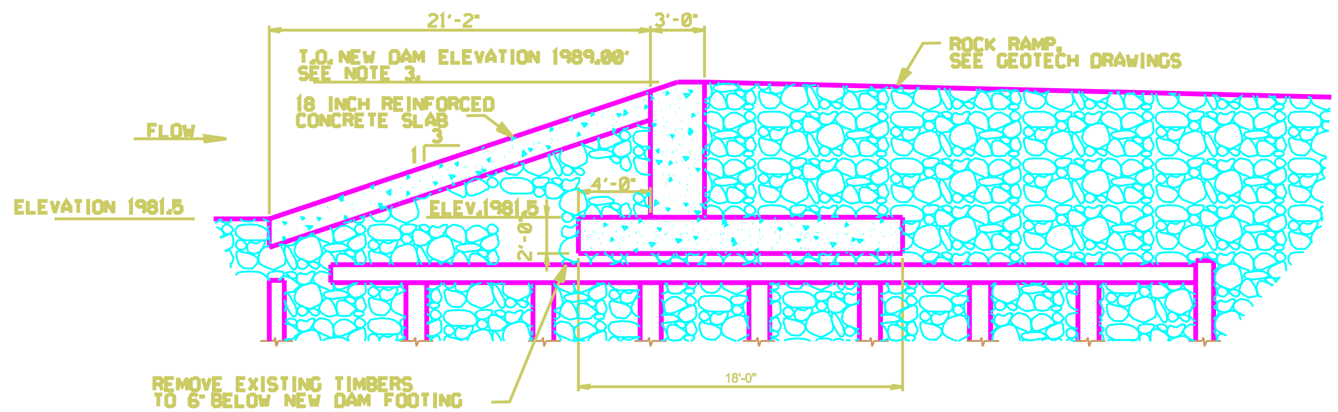
SHEET IDENTIFICATION NUMBER
S-103



EXISTING TIMBER DAM - SECTION

8' 4' 0 8'

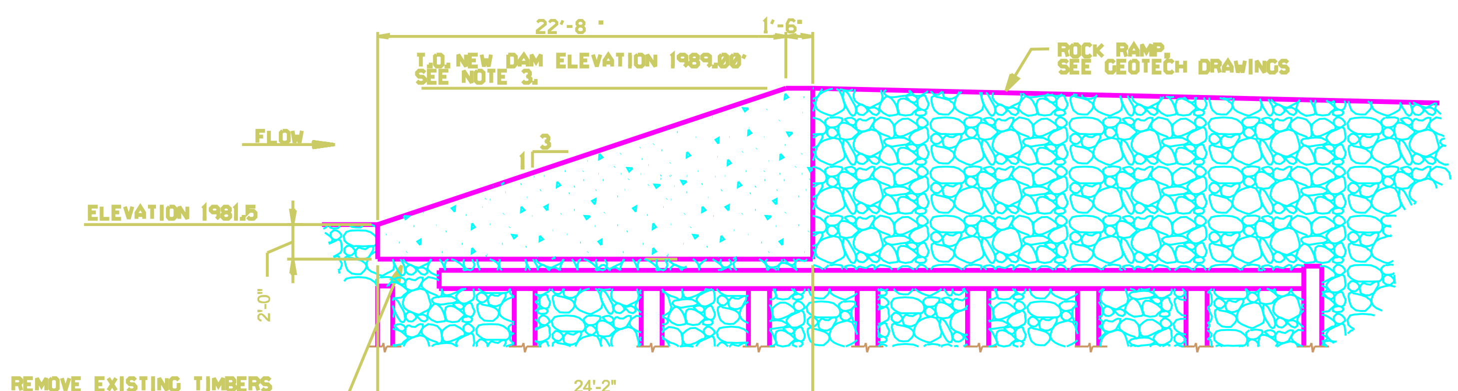
SCALE: 3/16" = 1'-0"



NEW CONCRETE DAM (RETAINING WALL OPTION) - SECTION

8' 4' 0 8'

SCALE: 3/16" = 1'-0"



NEW CONCRET DAM (WEIR OPTION) - SECTION

8' 4' 0 8'

SCALE: 3/16" = 1'-0"

NOTES:

1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"x33".

2. NEW CONCRETE DAM OPTIONS SHOWN WERE DESIGNED FOR FORCE DUE TO SHEET OF ICE BEING FORCED UP THE UPSTREAM SLOPE. IF DAM MUST BE DESIGNED FOR FORCE OF ICE BEING CRUSHED AGAINST A STRUCTURE, PILES WILL BE REQUIRED.

3. THE CREST ELEVATION COULD BE VARIED AND INCLUDE SOME GROUTED ROCK TO ENHANCE FISH PASSAGE.

US ARMY CORPS
OF ENGINEERS
OMAHA DISTRICT

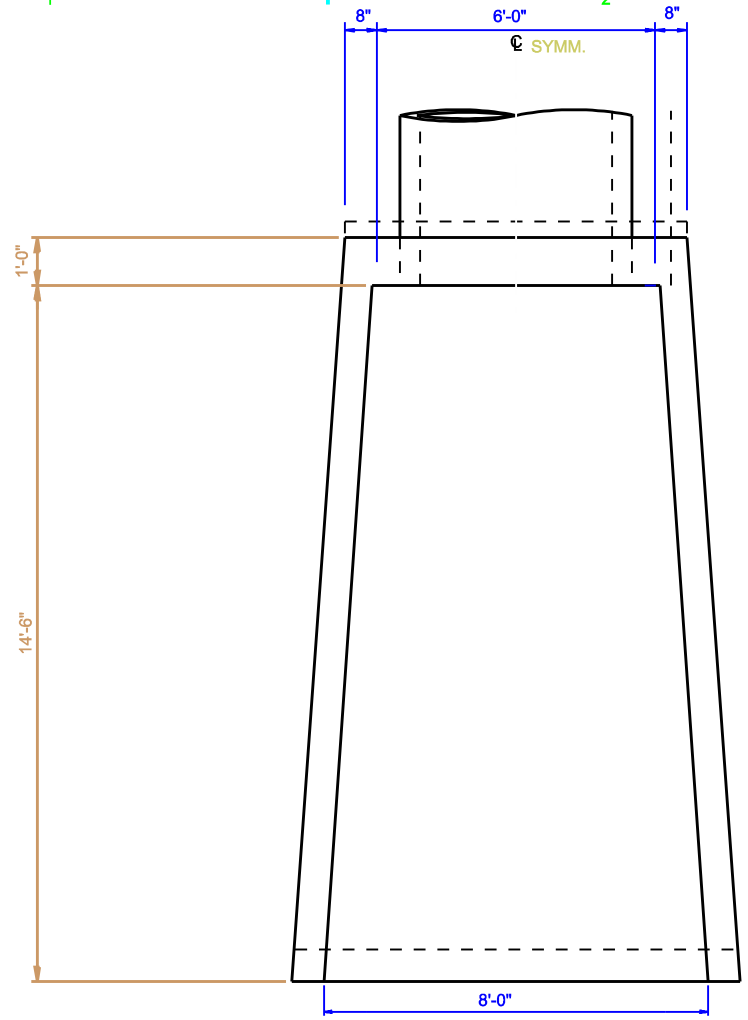
MARK | DESCRIPTION | DATE | APPR.

DESIGNED BY: L.E.P.
DWN BY: L.E.P.
CKD BY:
SUBMITTED BY: B.N.H.
FILE NAME: WW04-S-104.DGN
SIZE: 33.1x22
PLOT SCALE: 0.2 1 in / in.
DATE: MM YYYY
SOLICITATION NO.: W9128F-0X-X-00XX
CONTRACT NO.: W9128F-0X-C-00XX
FILE NUMBER: X
PLOT DATE: 7/12/2006

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

NEW CONCRETE DAM

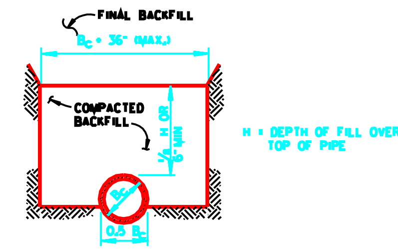
SHEET
IDENTIFICATION
NUMBER
S-104



OUTLET STRUCTURE - PLN

2' 1' 0 2'

SCALE: 1/2" = 1'-0"



CLASS C *

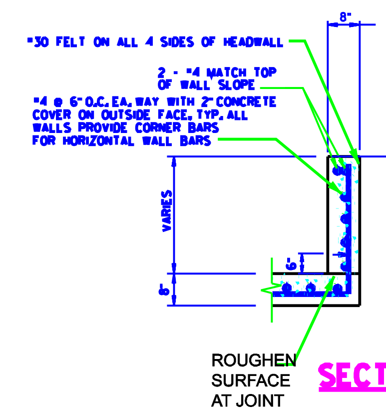
* SEE SPECIFICATION SECTION 02310 FOR MATERIAL AND COMPACTION REQUIREMENTS

BEDDING DETAILS

NO SCALE

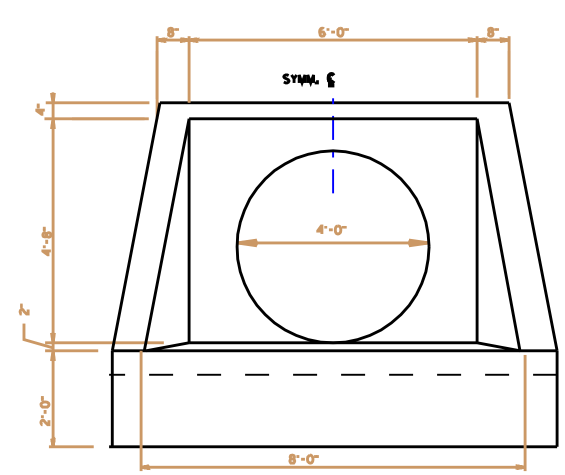
NOTES:

1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"x33".



SECTION 2

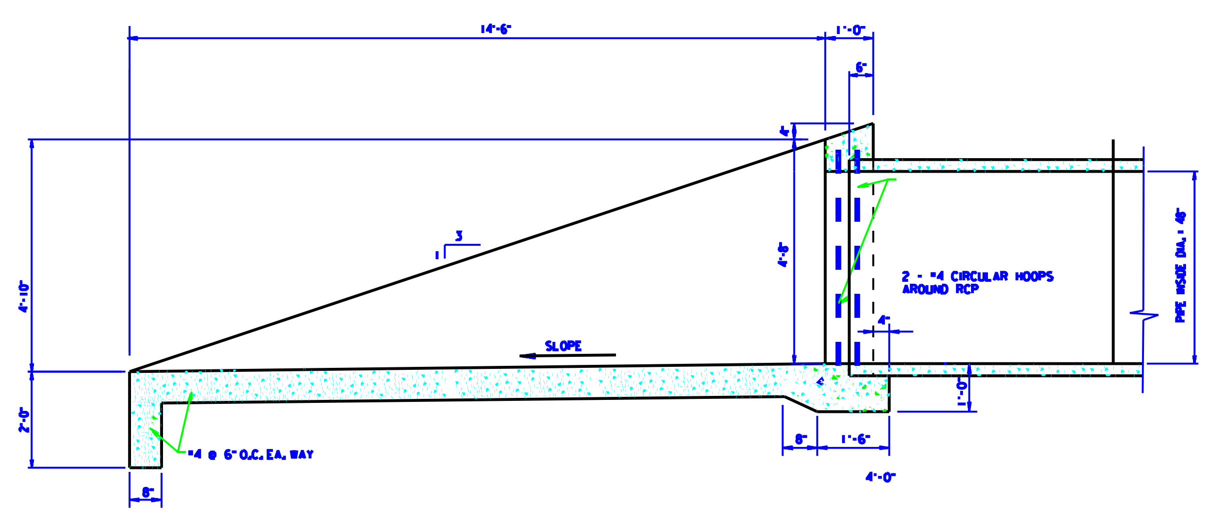
NO SCALE



SECTION 1

2' 1' 0 2'

SCALE: 1/2" = 1'-0"



SECTION 1

2' 1' 0 2'

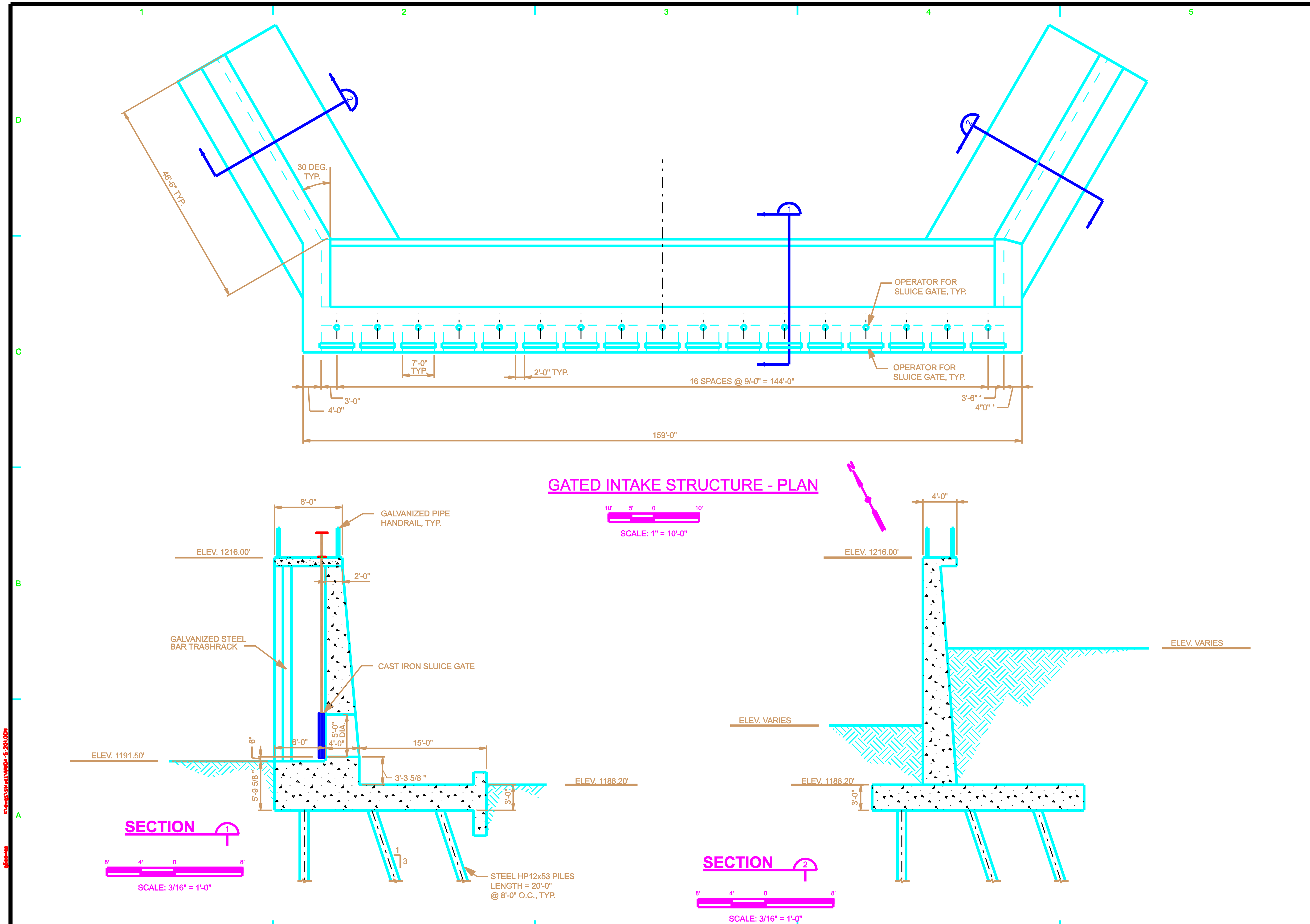
SCALE: 1/2" = 1'-0"

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

OUTLET STRUCTURE

SHEET IDENTIFICATION NUMBER
S-104



GATED INTAKE STRUCTURE - PLAN

SCALE: 1" = 10'-0"

SECTION 1

SCALE: 3/16" = 1'-0"

SECTION 2

SCALE: 3/16" = 1'-0"

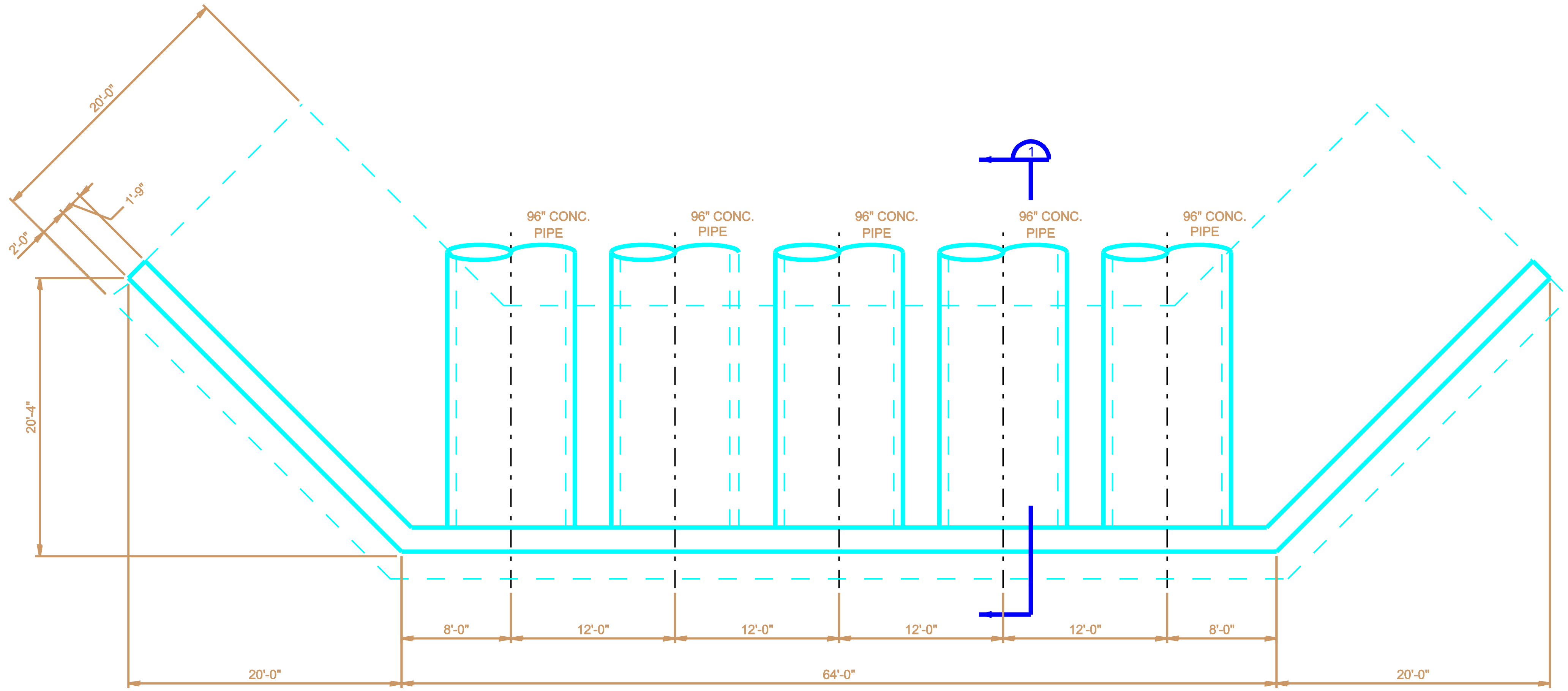
US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

DESIGNED BY: L.E.P.
DWN BY: L.E.P.
CKD BY:
SUBMITTED BY: B.A.H.
DATE: MM YYYY
SOLICITATION NO.: W9128F-0X-X-00XX
CONTRACT NO.: W9128F-0X-C-00XX
FILE NUMBER: X
FILE NAME: WA04-S-201.DGN
SIZE: 33.1x22
PLOT SCALE: 0:2 1 in / in.
PLOT DATE: 6/30/2006

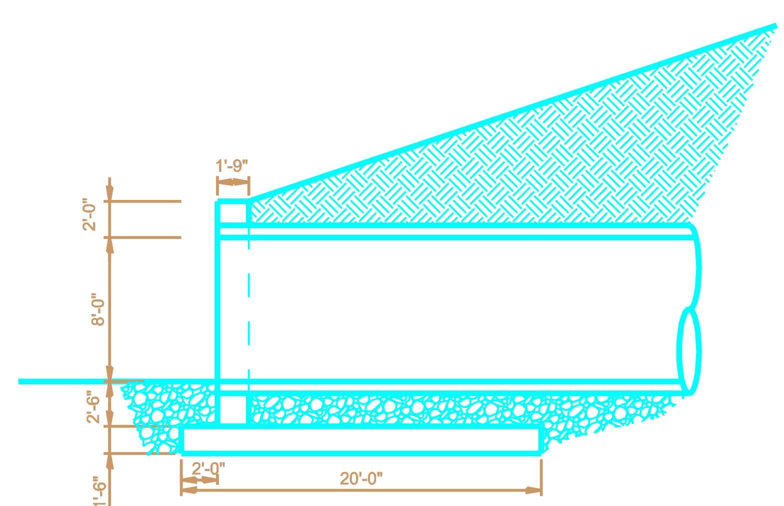
GATED INTAKE STRUCTURE

SHEET IDENTIFICATION NUMBER
S-201



PIPE CULVERT HEADWALL

SCALE: 3/16" = 1'-0"



SECTION

SCALE: 3/16" = 1'-0"

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

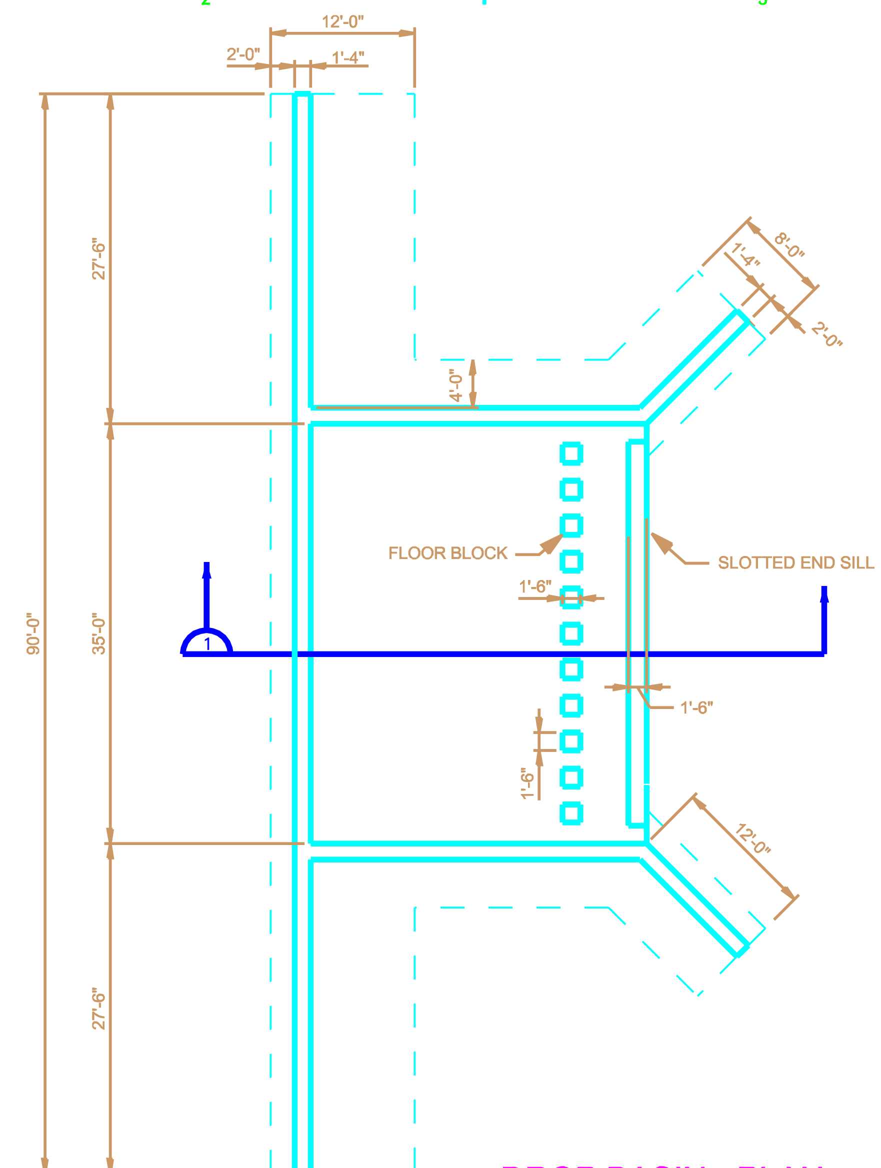
MARK | DESCRIPTION | DATE | APPR

DESIGNED BY: L.E.P.
DWN BY: L.E.P.
CKD BY:
SUBMITTED BY: B.N.H.
FILE NAME: W04-S-301.dgn
SIZE: 33.1x22
PLOT SCALE: 0:2 1in / ft.
DATE: MM YYYY
SOLICITATION NO.: W9128F-0X-X-00XX
CONTRACT NO.: W9128F-0X-C-00XX
FILE NUMBER: x
PLOT DATE: 6/30/2006

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

CULVERT HEADWALL

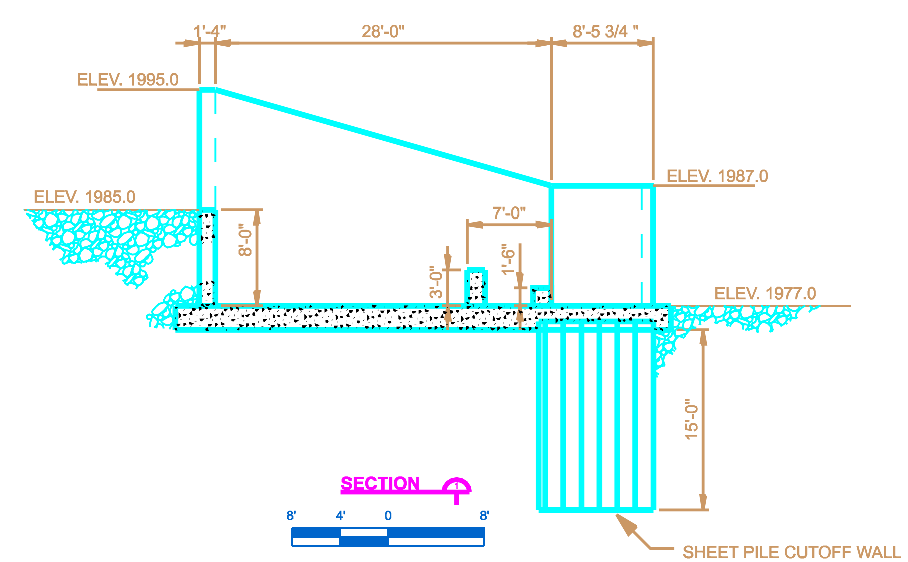
SHEET IDENTIFICATION NUMBER
S-301



DROP BASIN - PLAN

8' 4' 0 8'

SCALE: 1/8" = 1'-0"



SECTION

8' 4' 0 8'

SCALE: 1/8" = 1'-0"

NOTE:

1. ALL SCALES SHOWN ARE BASED ON A STANDARD DRAWING SIZE OF 22"X33".

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

MARK | DESCRIPTION | DATE | APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

DESIGNED BY: L.E.P. | DWN BY: L.E.P. | CKD BY: | SUBMITTED BY: B.N.H. | FILE NAME: WW04-S-401.dgn | SIZE: 33.1x22 | PLOT SCALE: 0.2 1 in / in. | DATE: MM YYYY | SOLICITATION NO.: W9128F-0X-X-00XX | CONTRACT NO.: W9128F-0X-C-00XX | FILE NUMBER: X | PLOT DATE: 7/7/2006

DROP BASIN

SHEET IDENTIFICATION NUMBER

S-401

ELECTRICAL

The electrical utility for the Lower Yellowstone Fish Passage – Intake Diversion Dam was identified as Montana Dakota Utilities (MDU) in Glendive, Montana. Steve Merrill was designated as the point of contact steve.merrill@mdu.com (406) 359-3100 (406-359-3122 electrical) for MDU. Location maps for each option were sent to Steve Merrill of MDU by Joe Chamberlain of USACE-Omaha on May 10th, 2006. The load for the fish screen was identified by Joe Chamberlain of USACE-Omaha as being 1 phase, 20 kW according to input from the Bureau of Reclamation. The following costs were identified for each Option:

- OPTION 1: MDU has existing single phase primary lines within 200 feet of the existing irrigation intake. MDU currently has an existing low voltage service to lighting at the intake structure. There would be no charges from MDU to install a larger single phase transformer and overhead service at this location. If 3 phase is required; costs would be approximately $4000-5,000 for MDU's upgrade of facilities to accommodate a three phase load.
- OPTION 2 – MDU has an existing 3 phase underground primary power line in the vicinity of what the Option 2 site plan shows as 'PVC return pipe'. This existing power line serves an irrigation pump. MDU can provide electrical service to Option 2 as either single phase power or three phase power from this site. A very rough estimate of this construction cost is $10,000. Actual measurements for the power line extension are required to obtain a more accurate cost. This line is served from a substation that is energized only seasonally as needed for the farmers irrigation.

In both options, MDU would provide electrical service to the meter point only. The Corps of Engineers contractor would provide the meter base, breakers and downstream wiring. The Corps of Engineers contract would provide lighting and power for the fish screen sweep unit. This load is assumed to be 20 kW, 1 phase.

Appendix H
Cost Engineering

FINAL REPORT

Lower Yellowstone Project Fish Passage and Screening
Preliminary Design Report
Intake Diversion Dam
July 2006

US Army Corps
of Engineers ®
Omaha District

U.S. Army Corps of Engineers
Project CI 11959: INTAKE DAM & FISH SCREEN
Appendix H

INTAKE DAM & FISH SCREEN
LOWER YELLOWSTONE RIVER, MONTANA

Estimated by	CENWO-ED-C
Designed by	COE - Omaha District
Prepared by	Gary Norenberg
Preparation Date	7/12/2006
Effective Date of Pricing	7/12/2006
Estimated Construction Time	Days

Date	Author	Note
	CENWO-ED-C	ESTIMATE ASSUMPTIONS: 1. Estimate is does not include real estate costs. 2. Engineering and Design - 9% (Misc Contract Cost) 3. Supervision and Administration - 6% (SIOH) 4. Contingencies - 20% 5. Assume random fill is obtained on site at no cost for the material. 6. All excess excavated material to be wasted on-site at no cost to the project. 7. Based on feasibility study design, using 20% contingencies is probably low considering cost of concrete and metal material prices and remoteness ofthe project.

Description	ContractCost	MiscContract	SIOH	Contingency	ProjectCost
Option Summary	76,725,860.49	2,886,495.93	7,319,647.09	17,729,811.84	106,378,871.05
New Concrete Dam Options	5,468,979.76	291,675.93	521,740.67	1,263,771.84	7,582,631.06
Retaining Wall Option	2,618,317.83	138,313.62	249,787.52	605,040.89	3,630,245.31
Weir Option	2,850,661.93	153,362.32	271,953.15	658,730.96	3,952,385.75
Rock Ramp Options, 2% slope	20,007,612.03	697,913.76	1,908,726.19	4,623,358.99	27,740,153.93
Riprap from nearby quarry	4,899,803.01	118,059.56	467,441.21	1,132,246.48	6,793,478.88
Riprap from Guernsey quarry	5,876,271.98	141,587.35	560,596.35	1,357,888.93	8,147,333.57
Conc Fab-form	9,231,537.04	438,266.85	880,688.63	2,133,223.58	12,799,341.47
Rock Ramp Options, 5% slope	13,157,541.14	537,328.02	1,255,229.42	3,040,444.61	18,242,667.64
Riprap from nearby quarry	2,590,734.92	62,423.13	247,156.11	598,667.03	3,592,002.16
Riprap from Guernsey quarry	3,567,203.89	85,950.91	340,311.25	824,309.47	4,945,856.85
Conc Fab-form	6,999,602.33	388,953.97	667,762.06	1,617,468.11	9,704,808.64
Fish Screen Option	6,966,771.09	272,646.95	664,629.96	1,609,881.46	9,659,288.79
Fish Screen Structure	6,966,771.09	272,646.95	664,629.96	1,609,881.46	9,659,288.79
Upstream Diversion Option	31,124,956.46	1,086,931.27	2,969,320.85	7,192,354.94	43,154,129.63
Upstream Diversion Option	31,124,956.46	1,086,931.27	2,969,320.85	7,192,354.94	43,154,129.63

Currency in US dollars

Description	ContractCost	MiscContract	SIOH	Contingency	ProjectCost
Concrete Dam Options	5,468,979.76	291,675.93	521,740.67	1,263,771.84	7,582,631.06
New Concrete Dam Options	5,468,979.76	291,675.93	521,740.67	1,263,771.84	7,582,631.06
Retaining Wall Option	2,618,317.83	138,313.62	249,787.52	605,040.89	3,630,245.31
Diversion of Water	740,267.53	18,027.34	70,621.52	171,061.02	1,026,366.12
Pump of Water	218,721.40	5,460.82	20,866.02	50,542.14	303,252.84
Construct Coffer Dam, 1st half	161,737.69	3,897.03	15,429.78	37,374.34	224,246.07
Remove Coffer Dam, 1st half	117,512.43	2,831.43	11,210.69	27,154.77	162,928.63
Construct Coffer Dam, 2nd half	124,783.59	3,006.63	11,904.35	28,834.99	173,009.95
Remove Coffer Dam, 2nd half	117,512.43	2,831.43	11,210.69	27,154.77	162,928.63
Remove Existing Dam	36,942.47	890.12	3,524.31	8,536.67	51,220.00
Foundation Excavation	4,541.81	109.43	433.29	1,049.52	6,297.13
Wall Footing	594,250.45	38,926.02	56,691.49	137,319.39	823,916.36
Retaining wall	532,552.20	34,884.51	50,805.48	123,062.16	738,372.97
Backfill Headwall	11,409.60	274.91	1,088.48	2,636.53	15,819.19
Rock Riprap	13,136.31	316.52	1,253.20	3,035.54	18,213.23
Concrete Slab	685,217.47	44,884.76	65,369.75	158,340.05	950,040.31
Weir Option	2,850,661.93	153,362.32	271,953.15	658,730.96	3,952,385.75
Diversion of Water	740,267.53	18,027.34	70,621.52	171,061.02	1,026,366.12
Pump of Water	218,721.40	5,460.82	20,866.02	50,542.14	303,252.84
Construct Coffer Dam, 1st half	161,737.69	3,897.03	15,429.78	37,374.34	224,246.07
Remove Coffer Dam, 1st half	117,512.43	2,831.43	11,210.69	27,154.77	162,928.63
Construct Coffer Dam, 2nd half	124,783.59	3,006.63	11,904.35	28,834.99	173,009.95
Remove Coffer Dam, 2nd half	117,512.43	2,831.43	11,210.69	27,154.77	162,928.63
Remove Existing Dam	36,942.47	890.12	3,524.31	8,536.67	51,220.00
Foundation Excavation	4,541.81	109.43	433.29	1,049.52	6,297.13
Concrete Dam	2,040,238.13	133,644.57	194,638.72	471,458.23	2,828,749.36
Backfill	9,795.48	236.02	934.49	2,263.54	13,581.24
Rock Riprap	18,876.51	454.83	1,800.82	4,361.98	26,171.91

Labor ID: LB04NatFD EQ ID: EP03R08 Currency in US dollars TRACES MII Version 2.2

Description	ContractCost	MiscContract	SIOH	Contingency	ProjectCost
Rock Ramp Options, 2%	**20,007,612.03**	**697,913.76**	**1,908,726.19**	**4,623,358.99**	**27,740,153.93**
Rock Ramp Options, 2% slope	**20,007,612.03**	**697,913.76**	**1,908,726.19**	**4,623,358.99**	**27,740,153.93**
Riprap from nearby quarry	**4,899,803.01**	**118,059.56**	**467,441.21**	**1,132,246.48**	**6,793,478.88**
Riprap	**3,964,828.12**	**95,531.57**	**378,244.60**	**916,192.48**	**5,497,154.89**
Large Stone	**934,974.90**	**22,527.99**	**89,196.61**	**216,054.00**	**1,296,324.00**
Riprap from Guernsey quarry	**5,876,271.98**	**141,587.35**	**560,596.35**	**1,357,888.93**	**8,147,333.57**
Riprap from nearby quarry	**3,964,828.12**	**95,531.57**	**378,244.60**	**916,192.48**	**5,497,154.89**
Large stone from Guerney	**1,911,443.86**	**46,055.78**	**182,351.74**	**441,696.45**	**2,650,178.68**
Conc Fab-form	**9,231,537.04**	**438,266.85**	**880,688.63**	**2,133,223.58**	**12,799,341.47**
Conc Castings	**4,306,435.10**	**282,090.45**	**410,833.91**	**995,131.02**	**5,970,786.14**
Pilings	**905,761.72**	**59,331.38**	**86,409.67**	**209,303.42**	**1,255,820.52**
Place Conc	**54,512.09**	**1,313.46**	**5,200.45**	**12,596.65**	**75,579.93**
Riprap	**3,964,828.12**	**95,531.57**	**378,244.60**	**916,192.48**	**5,497,154.89**

Labor ID: LB04NatFD EQ ID: EP03R08 Currency in US dollars TRACES MII Version 2.2

Description	ContractCost	MiscContract	SIOH	Contingency	ProjectCost
Rock Ramp Options 5%	13,157,541.14	537,328.02	1,255,229.42	3,040,444.61	18,242,667.64
Rock Ramp Options, 5% slope	13,157,541.14	537,328.02	1,255,229.42	3,040,444.61	18,242,667.64
Riprap from nearby quarry	2,590,734.92	62,423.13	247,156.11	598,667.03	3,592,002.16
Riprap	1,655,760.03	39,895.14	157,959.51	382,613.03	2,295,678.16
Large Stone	934,974.90	22,527.99	89,196.61	216,054.00	1,296,324.00
Riprap from Guernsey quarry	3,567,203.89	85,950.91	340,311.25	824,309.47	4,945,856.85
Riprap from nearby quarry	1,655,760.03	39,895.14	157,959.51	382,613.03	2,295,678.16
Large stone from Guerney	1,911,443.86	46,055.78	182,351.74	441,696.45	2,650,178.68
Conc Fab-form	6,999,602.33	388,953.97	667,762.06	1,617,468.11	9,704,808.64
Conc Castings	4,416,291.98	289,286.56	421,314.26	1,020,516.75	6,123,100.51
Pilings	903,731.13	59,198.37	86,215.95	208,834.19	1,253,005.14
Place Conc	23,819.19	573.92	2,272.35	5,504.14	33,024.83
Riprap	1,655,760.03	39,895.14	157,959.51	382,613.03	2,295,678.16

Currency in US dollars

Description	ContractCost	MiscContract	SIOH	Contingency	ProjectCost
Fish Screen	**6,966,771.09**	**272,646.95**	**664,629.96**	**1,609,881.46**	**9,659,288.79**
Fish Screen Option	**6,966,771.09**	**272,646.95**	**664,629.96**	**1,609,881.46**	**9,659,288.79**
Fish Screen Structure	**6,966,771.09**	**272,646.95**	**664,629.96**	**1,609,881.46**	**9,659,288.79**
Subsurface Investigation	**42,793.18**	**1,068.42**	**4,082.47**	**9,888.65**	**59,331.89**
Mobilization	**16,096.04**	**401.87**	**1,535.56**	**3,719.47**	**22,316.84**
Diversion of Water	**649,279.34**	**16,210.56**	**61,941.25**	**150,035.47**	**900,212.82**
Pump of Water	**218,721.40**	**5,460.82**	**20,866.02**	**50,542.14**	**303,252.84**
Construct Coffer Dam, 1st half	**114,495.53**	**2,858.61**	**10,922.87**	**26,457.63**	**158,745.76**
Remove Coffer Dam, 1st half	**100,783.44**	**2,516.26**	**9,614.74**	**23,289.04**	**139,734.23**
Construct Coffer Dam, 2nd half	**114,495.53**	**2,858.61**	**10,922.87**	**26,457.63**	**158,745.76**
Remove Coffer Dam, 2nd half	**100,783.44**	**2,516.26**	**9,614.74**	**23,289.04**	**139,734.23**
Stripping	**12,886.87**	**310.51**	**1,229.41**	**2,977.90**	**17,867.38**
Foundation Excavation	**1,224.22**	**29.50**	**116.79**	**282.89**	**1,697.35**
Basin Slab	**1,057,016.21**	**69,239.21**	**100,839.35**	**244,255.31**	**1,465,531.84**
Basin Side Walls	**227,051.62**	**14,872.88**	**21,660.72**	**52,467.09**	**314,802.53**
Radial Gates	**450,850.52**	**29,532.69**	**43,011.14**	**104,182.54**	**625,095.23**
Fish Screen Structure	**3,784,995.90**	**94,500.00**	**361,088.61**	**874,636.85**	**5,247,821.12**
Air Cleaning System	**175.56**	**11.50**	**16.75**	**40.57**	**243.41**
Bypass Pipe	**644,347.99**	**42,207.63**	**61,470.80**	**148,895.93**	**893,375.60**
Riprap, Ramp	**7,949.55**	**191.54**	**758.39**	**1,836.98**	**11,021.89**
Mobilization, year 2	**16,096.04**	**401.87**	**1,535.56**	**3,719.47**	**22,316.84**
Restoration	**56,008.07**	**3,668.77**	**5,343.17**	**12,942.34**	**77,654.06**

Currency in US dollars

Description	ContractCost	MiscContract	SIOH	Contingency	ProjectCost
Upstream Diversion Option	**31,124,956.46**	**1,086,931.27**	**2,969,320.85**	**7,192,354.94**	**43,154,129.63**
Upstream Diversion Option	**31,124,956.46**	**1,086,931.27**	**2,969,320.85**	**7,192,354.94**	**43,154,129.63**
Upstream Diversion Option	**31,124,956.46**	**1,086,931.27**	**2,969,320.85**	**7,192,354.94**	**43,154,129.63**
Subsurface Investigation	**42,793.18**	**1,068.42**	**4,082.47**	**9,888.65**	**59,331.89**
Mobilization	**16,272.63**	**406.28**	**1,552.41**	**3,760.28**	**22,561.68**
Diversion of Water, Upstream	**278,064.22**	**6,890.67**	**26,527.33**	**64,255.08**	**385,530.48**
Construct Coffer Dam	**27,102.15**	**653.02**	**2,585.54**	**6,262.76**	**37,576.59**
Rock Riprap	**2,677.80**	**64.52**	**255.46**	**618.79**	**3,712.72**
Pump of Water	**218,721.40**	**5,460.82**	**20,866.02**	**50,542.14**	**303,252.84**
Remove Coffer Dam	**29,562.87**	**712.31**	**2,820.30**	**6,831.39**	**40,988.33**
Intake Structure	**1,700,719.93**	**110,468.37**	**162,248.68**	**393,002.36**	**2,358,014.16**
Foundation Excavation	**3,066.23**	**73.88**	**292.52**	**708.54**	**4,251.26**
Intake Footing	**627,493.75**	**41,103.60**	**59,862.90**	**145,001.26**	**870,007.54**
Sluice Gates	**423,688.21**	**27,753.44**	**40,419.86**	**97,905.87**	**587,435.23**
Intake Structure	**626,928.00**	**41,066.54**	**59,808.93**	**144,870.52**	**869,223.14**
Backfill Headwall	**14,435.77**	**347.83**	**1,377.17**	**3,335.82**	**20,014.91**
Rock Riprap	**5,107.96**	**123.08**	**487.30**	**1,180.35**	**7,082.09**
Canal	**21,887,940.55**	**546,523.74**	**2,088,109.53**	**5,057,865.30**	**30,347,191.81**
Stripping	**214,028.27**	**5,156.96**	**20,418.30**	**49,457.65**	**296,745.92**
Berms	**61,696.39**	**1,486.56**	**5,885.84**	**14,256.80**	**85,540.80**
Channel Excavation and Haul	**21,150,025.75**	**509,604.74**	**2,017,712.46**	**4,887,347.95**	**29,324,087.70**
Restoration	**462,190.14**	**30,275.49**	**44,092.94**	**106,802.90**	**640,817.38**
Railroad Culverts, Station 82+70	**3,908,519.64**	**254,141.08**	**372,872.77**	**903,180.72**	**5,419,084.32**
Foundation Excavation	**4,188.17**	**100.91**	**399.55**	**967.80**	**5,806.82**
Headwall Footing	**360,715.63**	**23,628.46**	**34,412.27**	**83,354.17**	**500,125.01**
Headwalls	**143,025.92**	**9,368.83**	**13,644.67**	**33,050.43**	**198,302.58**
8' RCP Culvert	**3,359,277.70**	**220,047.47**	**320,475.09**	**776,261.89**	**4,657,571.34**
Backfill Headwall	**33,878.30**	**816.29**	**3,231.99**	**7,828.60**	**46,971.59**
Rock Riprap	**7,433.92**	**179.12**	**709.20**	**1,717.83**	**10,306.98**
Railroad Culverts, Station 6+30	**1,880,157.71**	**121,394.55**	**179,367.05**	**434,466.84**	**2,606,801.07**

Currency in US dollars

Description	ContractCost	MiscContract	SIOH	Contingency	ProjectCost
Foundation Excavation	**1,813.06**	**43.69**	**172.97**	**418.96**	**2,513.77**
Headwall Footing	**318,199.40**	**20,843.46**	**30,356.22**	**73,529.52**	**441,177.11**
Headwalls	**143,025.92**	**9,368.83**	**13,644.67**	**33,050.43**	**198,302.58**
8' RCP Culvert	**1,376,332.36**	**90,155.83**	**131,302.11**	**318,042.88**	**1,908,257.29**
Backfill Headwall	**33,878.30**	**816.29**	**3,231.99**	**7,828.60**	**46,971.59**
Rock Riprap	**6,908.66**	**166.46**	**659.09**	**1,596.45**	**9,578.72**
Diversion of Water, Downstream	**312,983.75**	**9,948.92**	**29,858.65**	**72,324.28**	**433,945.71**
Sheetpile Coffer Dam	**53,535.14**	**3,506.79**	**5,107.25**	**12,370.90**	**74,225.40**
Rock Riprap	**40,727.22**	**981.31**	**3,885.38**	**9,411.25**	**56,467.47**
Pump of Water	**218,721.40**	**5,460.82**	**20,866.02**	**50,542.14**	**303,252.84**
Drop Structure	**252,411.36**	**15,726.93**	**24,080.04**	**58,327.22**	**349,963.30**
Foundation Excavation	**2,082.72**	**50.18**	**198.69**	**481.28**	**2,887.65**
Basin Slab	**140,114.62**	**9,178.12**	**13,366.93**	**32,377.69**	**194,266.11**
Basin Side Walls	**92,805.43**	**6,079.16**	**8,853.64**	**21,445.48**	**128,672.87**
Backfill Headwall	**4,272.28**	**102.94**	**407.58**	**987.24**	**5,923.43**
Rock Riprap	**13,136.31**	**316.52**	**1,253.20**	**3,035.54**	**18,213.23**
Remove Existing Dam	**77,719.09**	**1,872.62**	**7,414.40**	**17,959.33**	**107,755.97**
Rock Grade Control	**767,374.39**	**18,489.70**	**73,207.52**	**177,324.87**	**1,063,949.24**
Excavate rock channel	**21,993.22**	**529.92**	**2,098.15**	**5,082.19**	**30,493.16**
Riprap	**745,381.17**	**17,959.78**	**71,109.36**	**172,242.68**	**1,033,456.08**

Labor ID: LB04NatFD EQ ID: EP03R08 Currency in US dollars TRACES MII Version 2.2