INTAKE DIVERSION DAM MODIFICATION

LOWER YELLOWSTONE PROJECT

PRELIMINARY DRAFT EA-ADDENDUM

APPENDIX A2
ENGINEERING

The following Attachments comprise the Engineering Appendix to the subject EA Addendum.

Intake Diversion Dam Modification

Lower Yellowstone Project, Montana

Intake Fish Bypass Option Evaluation Summary

Attachments

May 2012

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Attachment 1a

Ice Forces on Intake Dam
Lower Yellowstone River, 30% Design

Prepared by:
Andrew Tuthill, Cold Regions Research and Engineering Laboratory (CRREL)

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**Ice Forces on Intake Dam, Lower Yellowstone River:
30 Percent Design**

Andrew M. Tuthill

US Army Corps of Engineers
Cold Regions Research and Engineering Laboratory
72 Lyme Road, Hanover, NH 03755

1. Introduction

Omaha District of the Corps of Engineers (NWO) is redesigning The Yellowstone Intake Dam on the Lower Yellowstone River to include a 1600-ft-long rock ramp for fish passage and a new headworks structure with sixteen 6-ft-diameter circular intakes. The intakes will be screened and the screens removed during the winter season to avoid ice damage. The new dam will have a 1V:3H upstream face will be constructed of concrete. Plans are being developed for a downstream rock ramp with a trapezoidal cross section and a mild slope grading from a 0.002 to 006 slope in the downstream direction. Current plans call for the ramp to be constructed of large rocks with smaller size infill material.

The lower Yellowstone River is subject to heavy ice formation, dynamic ice breakups and ice jams. Ice action and ice forces are therefore a critical design factor, particularly for the rock ramp since this is a new and relatively untested type of structure in an extreme ice environment. In this report, CRREL provides ice force estimates for the headworks, dam and ramp for the 30 percent design being developed by NWO.

2. Previous Studies

In 2006, CRREL provided NWO with ice force estimates for the 10 percent design (Haehnel and Tuthill, 2006). This study included a literature review of past studies and designs related to ice forces on rock structures and armor stone. The study analyzed the Lower Yellowstone River ice regime and historic ice jam events as they related to the 10 percent design. Ice force estimates were developed for a rock dam similar to the existing one with a 1V:2H upstream face and a 1V:10H downstream rock ramp with rows of large boulders to create a pool and riffle sequence. The study addressed removal of rock from the dam, damage to the dam by ice forces and removal of boulders from the proposed rock ramp structure.

Based on the results of physical model tests at CRREL (Sodhi et al. 1996, Sodhi et al., 1997, Sodhi and Donnelly, 1999), for an estimated maximum ice thickness of 21 in, the estimated D_{50} to resist rock movement on the upstream face of the dam would need to be approximately 6 ft. Following the methods prescribed in AASHTO (1998), assuming an effective ice strength of 110 psi , the estimated ice force on the upstream face of the dam resulting from the crushing failure of a 400-ft diameter ice floe was estimated to be 5600 kips or 14 kips/ft. This assumed that the floe would fail along its 400 ft-wide contact with which is unlikely.

Most vulnerable to movement by ice were the 5-ft-diamter boulders protruding from the rock ramp which could experience ice forces as great as 140 kips compared to the estimated 7 kips needed to overturn them.

Omaha District completed a 30 percent design hydraulics study in Sept. 2009 that selected a top of headworks elevation of 2012.5 ft based on estimated maximum ice affected stages at the

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project (NWO, 2009). By their calculations, this elevation provided a 95% assurance of containing the 100-year return interval ice jam event. Based on a review of historic ice events, and maximum accumulated freezing degree days (AFDD), a maximum estimated thermal ice thickness of 31 inches estimated[1]. A solid ice cover of this thickness was used in the HEC-RAS model at the 100-yr open water discharge to estimate maximum ice-affected water levels at the project. The HEC-RAS ice jam routine was also used for a range of winter-season discharges. These ice accumulation thicknesses were much greater than the solid cover thickness. Jam thicknesses ranged from about 10 ft just downstream of the dam to 13 ft upstream for the lower discharges and were between about 7 and 8 ft for the 100-year discharge.

3. Literature Update

The 30% ice force analysis contains a review of pertinent literature not included or post-dating the 2006 10% design ice force report by CRREL. The most important development since 2006 is a somewhat improved understanding of rock placement methods in terms of resisting rock movement due to ice.

Daly et al, (2008) conducted 1:20 scale physical model tests of ice impacts on a rock breakwater for Barrow Alaska. In the tests, a 1V:1.5H ramp supporting armor stone was shoved against a 5-ft-thick 87 psi ice sheet. They found that by selectively placing the rocks so that they interlocked, the embankment's resistance ice damage was significantly increased.

CETN (1985) defines selected placement as the "careful selection and placement of individual armor stones to achieve a higher degree of interlocking". Canfield (1998) gives specifics on selective stone placement saying that stones should be keyed and fitted, maximizing contact on all sides and recommending a minimum of three points of contact for stones within the same layer.

For example, in one tests by Daly et al (2008), randomly-placed 4-ton stone suffered extensive damage, while the same size rock selectively placed experienced little damage. Assuming a roughly spherical shape and a specific gravity of 2.7, a 4-ton rock would be about 3.6 ft in diameter, considerably less than the 5-ft thickness of the sliding ice sheet.

In previous model tests of ice impacts on riprap by Sodhi et al. (1996), Sodhi et al. (1997) and Sodhi and Donnelly (1999), it was concluded that, to prevent rock movement, the mean size (D_{50}) of randomly-placed stone needed to be 2-3 times the ice thickness. This was the basis of the 6-ft rock diameter estimate for the dam in the 10% ice force analysis.

The literature review for the 30 percent design found other pertinent information in terms of ice action on structures. Since it is unlikely that environmental driving forces (current, gravity and wind) are great enough to fail a very large floe or an ice sheet in crushing over a large width, static ice forces due to thermal expansion of the ice sheet typically govern the design of dam faces in northern climates. Morse et al., (2009) gives an ice force range of 6.9 to 10.3 kips/ft used in the design of vertical concrete dam faces in Canada. US Army (1999) gives a slightly higher range of 10-15 kips/ft for ice loadings on dams and rigid structures.

Gerard (1983) describes important river ice processes from ice formation to breakup, focusing on scenarios that produce the greatest ice forces on river structures. In addition to estimates of the ice type, ice thickness and ice strength, an accurate ice force estimate must consider hydraulic

[1] Memo: "Computation of Ice Thickness", provided by NWO.

factors such as discharge, stage and water velocity. Frazil ice accumulations, though typically not as strong as sheet ice, have been observed to mobilize and exert pressures of approximately 1.7 pounds per square ft (psf). The usual design situation occurs when a strong ice cover is lifted and moved by the current to impact a structure. This usually takes place at the onset of dynamic breakups which are common to the lower Yellowstone.

Discussions with NWO and project operators suggest that the breakup process and ice structure interaction are more complex than this however, sometimes consisting of multiple surges were ice floes become stranded in the channel, overbank areas and even on the dam crest and ramp. Subsequent discharge increases and releases of upstream jams, can remobilize this ice causing large, solid floes to impact the dam crest and ramp over a wide range of discharges and water levels. A site visit was made on January 14, 2011 to gather additional information to better understand the breakup process and the ice-structure interaction. Findings from the trip are described in Appendix A.

Historic breakup ice events have been the greatest single cause of damage to Intake Dam over its 100 year history. For example, the breakup of 1911 destroyed much of the wooden apron and lower sheet pile cutoff wall and the ensuing bed erosion caused nearly the dam to fail. Repairs included the replacement of the wooden sheet pile wall with steel sheet piling and the placement of 3800 cubic yards of armor stone on the apron and downstream. Review of early reports indicate that dam designers underestimated ice effects in the initial structure and much effort and expense was required to remediate these design deficiencies. It was originally thought that ice would pass the dam crest at depths of 3 ft or greater and the 9 plus ft of floe above the apron would provide a “water cushion”, preventing ice floes from hitting the wooden apron. The first few ice runs proved this not to be the case as large ice floes coming over the dam at speeds greater than 10 ft/s tended to pile up and pound against the apron and downstream bed protection for hours on end. Unlike open water conditions where bed shear and potential for rock movement increase with discharge and depth, ice gouging of bed material is more prevalent where the ice run occurs at lower flows and depths. Appendix A provides more detail on the early history of ice damages and remedial measures.

It is important to note that the present day structure at Intake bears little resemblance to the 1910 timber dam with its ogee crest and downstream wooden apron. The estimated 115,000 CY of rock fill that has been added over the years has created a downstream rock ramp filling the apron area and eliminating the backroller. Assuming a river width of 700 and an average layer thickness of 3 ft, the ramp would extend about 1500 ft downstream of the dam not all that different in form from the proposed new ramp.

In addition to direct ice impacts and ice gouging, the presence of a an ice jam or rough ice cover can increase bed shear and cause hydraulic scour as a result of decreased flow area, higher near bed water velocity and increased turbulence. This is mentioned since, should a jam form on the ramp, under-ice scour might mobilize the choke gravel placed between the larger stones. Beltaos, 2001 describes the methods of estimating bed shear in the presence of an ice cover.

5. Design Details

The 30% calls for a concrete dam with an upstream slope of 1V:3H. The dam crest is concrete with a trapezoidal cross section with a minimum elevation of 1988.1 ft. For the new downstream rock ramp, a trapezoidal channel with a mild slope (0.002-0.006) is anticipated. The ramp would be constructed of large (D_{100} = 2.5-3.0 ft) rocks with a choke gravel infill material to stabilize the larger rocks. Fig. 1 shows the project plan configuration and bathymetry.

The headworks will have a top elevation of 2012.5 ft. NWO (2009) specifies a target (low flow) water surface elevation of 1991.4 ft, and bottom of headworks elevation of 1981.4. The 6.5 ft diameter fish screens have top and bottom elevations of 1989.9 and 1983.4 ft respectively.

5. Ice-Hydraulic Processes Related to ice Loads on the Project

The ice analysis for the 10% design found the lower Yellowstone River to be subject to heavy ice formation, dynamic ice breakups and ice jams. Because the Yellowstone flows northeastward from warmer to colder climate, the ice breakup progresses downstream in a series of jams and releases, and ice jam severity tends to increase in the downstream direction as the breaking front encounters stronger thicker ice. Numerous ice jams and ice jam floods have occurred upstream of Intake at Glendive and downstream at Sidney (Haehnel and Tuthill, 2006). Jams have also been reported at Intake (Appendix A), in the vicinity of the Richland County Line, Elk Island and Savage. All this suggests that the project reach is subject to the dynamic formation and release of ice jams. The most recent severe ice jam event on the Yellowstone occurred in February 7-13, 1996. Fig. 2. shows Yellowstone River discharge and AFDD for that winter at Miles City and Sidney.

On faster flowing rivers such as the Yellowstone, the predominant ice type is frazil which forms as small particles in supercooled open water reaches. The frazil crystals stick together (flocculate) to form floes that tend to increase in size with distance traveled. The floes may accumulate along the channel sides to form border ice or stall in slack areas or channel obstructions to build an ice cover in the upstream direction. Only where water currents are slow ($\leq$ 1 ft/s) can in situ thermal ice growth be expected. In the 1 to 1-1/4 ft/s velocity range, the frazil floes will accumulate edge-to-edge in a process known as juxtaposition. At higher water velocities, the floes will stack or "shove" into a thicker ice accumulation. The HEC-RAS model contains an ice routine that calculates ice accumulation thickness by these processes for both the freezeup and breakup cases.

Average December-January discharge at Sidney gage is 5800 cfs with a standard deviation of 1680 cfs for the 1910-2009 period. A higher freezeup discharge will cause a thicker freezeup ice accumulation, since the water velocities and shear forces on the ice underside will be greater. An extreme case freezeup discharge is defined as the long term December-January average flow plus two standard deviations or 9160 cfs. Fig. 3 shows HEC-RAS calculated water surface profiles and water velocities for the freezeup discharge range indicating the predominant ice formation mode to be juxtaposition and shoving of frazil floes. Figs. 4 and 5 show HEC-RAS simulated freezeup ice covers in the project reach for discharges of 5800 and 9160 cfs respectively. This suggests that it would be possible for an 8-ft-thick frazil ice mass to release from upstream and impact the project at the onset of breakup. Immediately upstream of the dam the water velocity is low enough to allow the in situ growth of thermal ice, the maximum thickness of which can be calculated from AFDD data. On the rock ramp, the calculated under-ice velocity is sufficiently high to allow ice cover thinning by erosion. In this case the under-ice erosion velocity was set at 5 ft/s.

From review of past ice jam events, is estimated that a late-season, thick ice cover such as the one shown in Fig. 5 will release in the project reach at a discharge of about 20 Kcfs[2]. Fig. 6 shows this pre-release condition. It is also assumed that a breakup ice jam in the project area will release

[2] Review of the early project reports indicates that the ice could release once depth at the dam crest exceeded 3 ft at river flows as low as 9,000 cfs.

at a discharge of about 40 Kcfs[3]. This is based on the Sidney Gage data that give the annual peak on 3/14/1996 of 19.48 ft (instantaneous peak Q = 30 Kcfs) as ice-affected while the 3/6/1994 peak of 24.03 ft (peak Q=75 Kcfs) is listed as open water. In 1994 ice jams were reported at many locations on the lower Yellowstone, but the river may have been clear of ice by the time of the instantaneous peak on 3/6. HEC-RAS was used to calculate breakup ice jam profiles in the vicinity of the project at discharges of 30 and 40 Kcfs (Figs. 7 and 8 respectively).

Once the ice cover releases, it is assumed that the floes and thicker frazil ice masses travel downstream and impact the project at approximately open water surface elevations (WSE). Open water surface and velocity profiles were calculated for discharges of 20, 40, 60, 80 and 100 Kcfs (Fig. 9) These elevations were used to estimate the height range that the ice floes and ice masses impacted the headworks and dam crest. The WSE at dam crest for 20 Kcfs open water flow conditions was approximately 1995 ft (Fig. 10). It is possible that the floes and frazil ice masses could impact the headworks higher elevations under post breakup conditions of increasing discharge.

6. Estimation of Ice Forces

Ice forces and ice pressures were estimated for the dam crest the headworks and diversion inlets and the rock ramp. In the original analysis, it was assumed that the maximum ice forces would occur at the onset of ice breakup when the moving ice is the strongest and floe size the largest as suggested by Gerard (1983). It was also assumed that the lower discharge threshold for breakup on this part of the Yellowstone is about 20 Kcfs and the upper limit before ice jams release is about 40 Kcfs. Information gathered on the Jan. 14, 2011 trip suggest that the discharge range for breakup ice impacting the structure is much larger however, on the order of 9000 to above 100,000 cfs (Appendix A).

Two scenarios are considered. The first was for a large, 30-inch-thick, 167 psi ice floe impacting the headworks and dam crest[4]. The 167 psi (24 KSF) effective ice strength was taken from AASTO (1998) “where ice breakup occurs at melting temperatures but the ice moves in large pieces and is internally sound”. The 30 inch thickness estimate was based on 31-inch maximum ice thickness calculation provided by NWO. The second scenario considered an 8-ft-thick frazil ice mass releasing at 20 Kcfs and impacting the headworks and dam crest at the onset of ice breakup.

In this analysis, it is assumed that the rock ramp will be constructed of large rocks (D_{100} = 2.5-3.0 ft) with smaller size rock and choke gravel in between. The ice force estimates assume a 30-inch-thick 167 psi floe and an 8-ft-thick frazil ice mass impacts the surface formed by the larger rocks.

Movement of the smaller size infill material, because it lies below the ice-boulder contact, was assumed to be less susceptible to displacement by impacts by the large floes, provided the larger rocks are not displaced. Under ice hydraulic scour is a possibility however and shear on the ramp was calculated with a breakup ice jam in place at an assumed pre-release river discharge of 40 Kcfs (Figs. 8 &13).

[3] These ice cover and breakup ice jam release discharges are very approximate and will vary greatly depending on ice thickness and ice strength.

[4] In a conference call with NOW concern was expressed that the 110 psi ice strength “where breakup occurs at melting temperatures and the ice structure is somewhat disintegrated” was un-conservative. The 167 psi ice strength applies to “where breakup of major ice movement occurs at melting temperatures but the ice moves in large pieces and is internally sound.”

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6.1. Ice Forces on Dam Crest

Impact by large solid ice floes

The ice forces due to a large floe impacting the dam crest were estimated following the methods prescribed in AASHTO (1998). The maximum ice force applied to the structure is limited either the momentum of the floe or failure of the ice and the code accounts for both of these aspects. First the ice force associated with ice failure is computed using

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

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

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), h is the ice thickness 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. In cases where the $w/h \geq 6$, or $\phi < 15°$ then F_b is not computed and it is assumed that the ice fails in crushing and F_c is used. In the case of a large floe impacting the dam crest, w >> h so only crushing failure F_c applies.

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 1, is a function of A/h^2, where A is the plan area of the floe. The code stipulates that K_t of 0.5 is the minimum value that can be used.

The upstream face of the Intake Dam is approximately 600 ft wide. A reasonable maximum floe size is 2/3 the river width or about 400 ft (Fig. 11). A maximum sheet ice thickness of 30 in was assumed. A p value of of 167 psi was used for reasons explained above. For a floe of this size $A/h^2 > 1000$, thus $K_t = 1.0$. In the case of ice floes impacting inclined structures, a reduction coefficient can be used. For the 1V : 3H face of the intake dam (horizontal angle = 32°) the reduction coefficient is 0.5. (US Army, 1999).

Using these input values in eq. 2 gives

F_c = 12,200 kips / 400 ft = 31 kips/ft

The load that the dam face must withstand due to the impact of a large solid ice floe would be 12,000 kips or 31 kips/ft, assuming the floe fails along its entire 400 ft width which is extremely unlikely. More likely, the width of interaction and total ice force would be much less. If the floe crushed along half its width, the maximum load on the dam would be about 15 kips/ft which

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corresponds with the high end of the range given in the design literature. It was originally assumed that the top of the floe would be slightly above the 20 Kcfs water surface elevation of 1995 ft and the bottom of the floe at about 1993 ft. Under this scenario, the floe would contact the dam crest only near its edges which are at an elevation of about 1992. In light on the findings of the Jan 2011 field visit, moving ice floes can impact the dam face and abutments (including the headworks) over a much wider range of elevations, say 1985 to 2000 ft which would correspond to a 10-80 Kcfs flow range.

Table 1. 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

Impact by a thick frazil ice mass

From the Literature (Gerard, 1983 and AASHTO, 1998) large frazil ice masses moving against structures can exert shear forces as great as 2 kips/ft^2 (KSF). Classic ice jam theory assumes the coefficient of friction for sliding or jammed ice to be 1, so shear forces and normal forces on the dam face would be roughly equivalent. As in the solid ice floe case, the frazil ice mass would be expected to impact the structure over a wide range of elevations between 1985 and 2000 ft.

6.2. Ice Forces on the Headworks

Impact by large solid ice floes

In the case of the headworks, it was assumed that a 30-inch thick 167 psi floe, 200 ft in diameter impacts the headworks at a 20 º angle (Fig.12). At the 1998 ft water surface elevation, the bottom of the floe will be 1995.7 ft, well above the 1989.4 ft tops of the rectangular openings.

By the same steps outlined in Section 6.1, the impact force of the floe on the headworks at a 20 º angle is 4200 kips. In the case of ice impacting a vertical face ($\phi = 0 < 15°$) AASHTO (1998) says that the failure will occur in crushing rather than in bending. Distributed over 200 ft, this amounts to 21 kips/ft. By the above argument, it is very unlikely that the floe would fail along its entire width and the high end of the design literature of 15 kips/ft is recommended.

Impact by a thick frazil ice mass

An 8-ft-thick frazil ice mass exerting 2 kips/ ft^2 could theoretically exert a force of 16 kips/ft along the headworks. Based on the upper-limit value from the ice force literature, 15 kip/ft is recommended for both the solid floe and frazil ice mass cases. At the 1995 ft water surface elevation, the bottom of 8-ft-thick frazil ice mass would be at about 1988 ft, slightly below the 1989.4 ft tops of the fish screen openings. Accounts by locals indicate that the ice can act on the existing dam at elevations at or above 2000 ft however[5]. It is therefore recommended that the design load on the headworks above the 1995 ft elevation and below the 1988 ft elevation be 5 kips/ft.

[5] Correspondence with Lyle Peterson 2/10/10, Structural Engineer, NWO.

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Need and feasibility of ice protection features for headworks

Structures built in the channel to protect the headworks from ice would experience ice loads at least as great as those calculated for the headworks. These features would likely need to be bottom-founded such as dolphins, sheet pile cells or fixed shear booms. Conventional floating ice boom, with a maximum ice restraint capacity of about 2 kips/ft would not restrain breakup ice on the Yellowstone. Because the protective structures would be out in the channel rather than along the side, they would experience direct rather than oblique ice impacts and higher ice loadings than the headworks. From the above discussion it is estimated that the ice will act on the headworks over a 12-ft height range from about 1985 to 2000 ft (8 to 20 ft above the bed) and the moments on the protective structure would be great.

The most economical solution may be to design the headworks to withstand the estimated ice loads rather than rely on protective structures out in the channel to deflect or absorb the ice impacts.

6. 3. Ice Forces and Riprap Design for Ramp

Analysis of ice forces on the rock ramp and riprap design includes a review of existing design guidance and literature on case studies of similar projects. Although coastal revetments have been designed resist extreme ice ride-up and large riprap has been used to protect revetments and bridge abutments on rivers with dynamic ice runs, no instances were found where a such a large area of river bed has been protected from ice impacts as the proposed Intake rock ramp.

Due to this lack of guidance, the 10% design used the conservative rule of thumb that the average riprap diameter D_{50} should be twice the maximum expected ice thickness leading to a value of about 6 ft Sodhi et al. (1996), Sodhi et al. (1997) and Sodhi and Donnelly (1999). This guidance was based on physical model tests of ice rideup on riprap revetments sloped less than 3H:1V where the maximum damage occurs when the ice sheet pushes between the piled ice rubble and the riprap to dislodge individual rocks. Assuming layer thickness $T = 1.5 \times D_{50}$, the rock blanket would need to be 9 ft-thick. From economic and construction reasons, this design was deemed infeasible.

It was then speculated that the D_{50} could be reduced by selectively placed armor stone as described by Daly et al. 2008. In this 1:20 physical model study, selectively placed 3.6 ft diameter armor stone withstood ride-up of a 5-ft-thick ice sheet on a 1.5H:1V breakwater proposed for Barrow, AK. This approach was deemed unfeasible for the Intake ramp for two reasons. First, the armor stone being uniformly graded, would be less well suited to resist damage by hydraulic shear than conventional riprap which is better graded and more angular, allowing the pieces to interlock. Second, the estimated cost of individually placing large rock over the 700 ft ×1600 ft area was considered prohibitive.

The above cited cases differ from the Intake ramp situation in several ways. First, unlike a revetment, the rock ramp at Intake will be relatively flat [6] and the ice is expected to act more or

[6] The longitudinal slope of the ramp will increase by 0.2% increments of from 0.2% near the crest to 0.6% near the bottom over its 1600 ft length. The channel will be trapezoidal in cross section with a 70-ft-wide thalweg and gentle side slopes ranging from 1 to 3 %.

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less parallel to the ramp surface. Second, the Intake ramp bed protection will be subject to considerable hydraulic shear and lift forces during open water flood flows.

In the Intake ramp case, the ice will interact with the ramp in at least three ways. The first is under-ice hydraulic scour where the ice cover occupies a large portion of the flow area potentially increasing near-bed water velocity, turbulence and bed shear. Ice adhesion to bed material and uplift due to stage increase is also possible. The third and potentially most destructive mode is gouging of the riprap by moving ice during an ice run. This is the process that likely damaged the original dam apron and downstream rock protection in 1911 (Appendix A). Ice-hydraulic shear, ice uplift and ice gouging are considered in this analysis

HEC-RAS Analysis of potential ice impacts to ramp

The hydraulic and ice conditions at in the ramp reach were re-examined to better understand ice structure interaction by the above described modes. From the historical review of ice jams on the lower Yellowstone, it appears that most ice jams occur in the 20-40 Kcfs flow range. Ice runs can occur over a wider range of flows. An interesting note is that the daily average at Sidney for the damaging ice run of 3/9/1911 was only 9000 cfs (Appendix A) but this may be an outlier. Ice jams were simulated in the Intake reach for flows of 9160, 20,000 and 40,000 cfs using HEC-RAS and the proposed rock ramp geometry. Default ice jam parameters were ice jam internal strength = 45°; ice jam porosity = 0.4, under-ice erosion velocity = 5 ft/s. Figures 6.3.1-6.3.15 show ice jam profiles, under-ice water velocities and ice jam cross sections taken at locations on the ramp 110, 744 and 1340 ft below the dam crest.

In the 9160 cfs jam case, the maximum ice thickness is about 10 ft and the highest under ice water velocities about 3.5 ft/s. For the 20,000 cfs jam, ice thickness near the downstream end of the ramp reached 12 ft and maximum velocity was again about 3.5 ft/s. For the 40,000 cfs jam, ice thickness reached 15 ft and maximum water velocity was about 3.3 ft/s. In the 20 and 40 Kcfs cases the right flood plain is mostly inundated which limited stage rise in the main channel. Water velocities with a jam in place on the ramp are relatively mild (≤ 4 ft/s) due the staging effect of the jam on the ramp and downstream. The release of this downstream jam would trigger a rapid drop in upstream stage, high water velocities and potential gouging of the bed as ice on the ramp and the upstream river moved out. This “pulling the plug” phenomenon occurred during the 3/28/1912 ice-out at Intake (Appendix A).

Following jam release, the river +is assumed to return to more-or-less open water flow conditions during the ice run past the project. Figures 6.3.16 and 6.3.17 show water surface profiles and average water velocities for open water discharges of 9160, 20,000, 40,000 and 70,000 cfs. In the 40 and 70 Kcfs cases average water velocities on the ramp are in the 8-10 ft/s range and depths exceed 10 ft in general agreement with the BR model. If the river were conveying a heavy ice run, water depths might be a little higher and average velocities lower for a given discharge, but the HEC-RAS results provide a good approximation of hydraulic ice conditions.

Figures 6.3.18-6.3.22 show cross sections at locations along the ramp 94, 386, 744, 1047 and 1341 ft below the dam crest for the breakup flow range. The blue symbols represent ice floes that draw 2 ft at the 20 Kcfs discharge. This ice thickness and discharge would be fairly common during an ice run on the lower Yellowstone. The figures show that under these conditions, the side slopes of the rock ramp would experience impacts from the moving floes. Several factors would produce more severe conditions in terms of ice gouging. First would be the tendency for the moving ice to bunch up or raft to produce multi-layer thicknesses in portions of the ram area. A 4-ft thick accumulation of moving floes would potentially impact a much larger portion of the

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ramp area at the 20 Kcfs flow level. Second, based on the 1911 breakup, ice may run past the project at discharges lower than 20Kcfs cfs. In this lower discharge case, a 2-ft average floe thickness would potentially impact much of the ramp area than at the assumed 20 Kcfs lower threshold for breakup.

Also important to consider is the duration of the ice runs which can be quite long. During a heavy ice year, much ice must pass the Intake project before the river clears out. For example, the initial 3/12/1911 ice run occurred over a 5.5 hour period followed by a second run from farther upstream that went on from 6 pm to "late into the night". In light of this, ice damages to the bed protection may occur from multiple small hits over a large area rather than a few large ones.

Finally, the hydrograph peak associated with river breakup may be followed by a larger open water peak. If the ramp is damaged by moving ice during the initial crest, the bed protection may be more vulnerable to hydraulic scour during the open water peak that follows.

Under-ice Hydraulic Shear on the Rock Ramp

Under-ice bed shear was calculated for a breakup ice jam at the structure at an assumed maximum discharge of 40 Kcfs (Fig. 13.). It is assumed that discharges in excess of 40 Kcfs will cause the ice jam to release, and open water bed shear calculations would then be appropriate. Total shear τ was estimated from the depth-slope product based HEC-RAS calculated values

$$\tau = \gamma y_{ui} S \tag{3}$$

where γ = the unit weight of water, y_{ui} is the under ice depth and S is the water surface slope. For flow beneath an ice cover the total water shear is distributed between the underside of the ice cover and the river bed. Methods for calculating under-iced shear and bed shear are described in Tuthill et al, (2009). In flow beneath an ice cover, the maximum water velocity typically occurs near the mid depth (within 40-60 percent of the total depth) and the under ice hydraulic radius is divided into ice-affected and bed-affected portions, depending on the roughness of the ice cover and the bed material. As a first-cut estimate we will assume that bed shear accounts for about half of the total shear, average under-ice bed shear for rock ramp = 0.7 psf with a maximum of 2.0 psf, and a minimum of 0.3 psf. (see attached spreadsheet). This shear force is assumed to act on the both the larger rocks and to a lesser extent the smaller infill material in the concavities between the rocks. In the case of the smaller material, a shear force of 2.0 psf could initiate movement of material up to 5 inches in diameter by the the Meyer-Peter criteria for the initiation of motion:

$$\frac{\tau_c}{(S_s - 1)\gamma D} = 0.047 \tag{4}$$

Where τ_c is the critcal shear stress S_s is the specific weight of the rock (assumed 2.65), γ = the unit weight of water (= 62.4 lb/ft^3) and D is the representative diameter of the infill material.

As a comparison, the bed shear resulting from an open water discharge of 70,000 was calculated at about 3 psf , using inputs of S = 0.005 and R = 10 ft. This indicates that bed protection designed to resist movement of under extreme open water conditions will be adequate for hydraulic bed shear with a stationary ice jam on the ramp.

Ice freezup forces and lifting of large rock or infill material.

For the large rocks on the ramp, sheet ice or thick frazil ice masses may adhere to the large rocks particularly near the edges of the ice cover. With the onset of breakup, the water and ice level in the channel will rise. At some point the ice cover will fracture free from the channel edges and move downstream. The concern is that buoyant force of the ice will be great enough to pull rocks from the ramp. Calculations indicate that the submerged weight of the rocks is greater than the buoyant force of the adhered ice, even in the case of an 8-ft-thick frazil ice mass. The adhered ice when it moves will likely remove some of the infill material.

Minimum rock size needed to withstand ice forces on ramp

The approach taken was to sizing the bed protection rock size to resist movement due to ice gouging had three parts. The first step was to size riprap bed protection to withstand extreme open water conditions using method described in US Army (1994) and NCHRP (2006) and applying rules thumb to adjust the design for impacts of ice and debris. The second step was to compare the preliminary riprap design for the Intake ramp to bed protection on others rivers with extreme ice action. A final step was to review hydro-meteorological data associated with historic ice-outs on the lower Yellowstone to estimate the frequency of ice events that could potentially damage the bed protection.

A preliminary bed protection design for extreme open water conditions was developed using methods described in US Army (1994). Worst case open water hydraulic conditions of water velocity V = 12 ft/s and depth d =12 ft were taken from the "Ramp Passage Optimization" Report of the BR physical model study dated 11/4/2010.

Using Eq. 3-3, and the inputs listed in Table 2 gives a D_{30} of 0.77 ft. Assuming a D_{85}/D_{15} ratio of 1.9 and a size distribution similar to those in Table C8.1 of NCHRP (2006) gives a D_{50} of 1.0 ft and a D_{100} of 1.8 ft. In the initial open water case, the recommended minimum factor of safety C_s of 1.1 was used. The minimum thickness T* is the greater of the D_{100} or 1.5 × the D_{50}. Assuming a final blanket thickness of about 4 ft gives a thickness coefficient C_t of 0.8. US Army, (1994) gives the rule of thumb that "for riprap subject to attack by large floating debris (the layer) thickness should be increased by 6-12 in, accompanied by appropriate increase in stone size". This guidance is reiterated in Province of British Columbia (2000). Adding the maximum of 12 inches would increase the layer thickness from 2.7 to 3.7 ft and proportionally, increase the D_{50} to 1.3 ft or 16 in. This rock size distribution is similar to the Class IV riprap described in Table C8.1 of NCHRP (2006) and is not that different to the preliminary riprap design being developed by NWO.

Table 2.

Intake Dam Ramp Riprap Design using EM 1110-2-1601		
Channel Width	600	ft
Radius of curvature	1200	ft
Local flow depth d	12	ft
Average water velocity V	12	ft/s
Unit Wt of water	62.4	lb/ft3
Angle of side slope	3	deg
Side slope correction K1	0.99	
Unit wt of rock	165	lb/ft3
Factor of safety Sf	1.1	
Stability coefficient for incipient failure Cs	0.375	
Vertical velocity distribution coefficient Cv	1.22	
Thickness coefficient Ct	0.8	
Acceleration due to gravity	32.2	ft/s3
D30	0.77	ft
Assumed D85/D15	1.9	
D15	0.7	ft
D50	1.0	ft
D85	1.3	ft
D100	1.8	ft
2*D50	1.9	ft
1.5*D100	2.7	ft
Layer Thickness	2.7	ft
Increased layer thickness due to ice impacts	1	ft
Increased rock size due to ice impacts	1.4	times
Assumed D85/D15	1.9	
D15	0.9	ft
D50	1.3	ft
D85	1.7	ft
D100	2.5	ft
2*D50	2.6	ft
1.5*D100	3.7	ft
Layer Thickness	3.7	ft

Riprap designs at other projects with severe ice action

Several bed and bank protection projects on rivers with extreme ice action are described for comparison to the Intake ramp. The first a guide bank and embankment spur design for the Tanana River Alaska Railroad Crossing at Salcha, AK. The CRREL ice jam database contains numerous reports of severe ice runs and ice jams in this section of the Tanana River, indicating ice action comparable to the Yellowstone at Intake. The guide bank is an angled rock spur that directs flow around the left abutment of the main bridge span. It consists of a raised berm and a flat-lying 4-ft-thick riprap blanket that extends 100 ft out into the main channel. This riprap blanket, by its location will be subject to the full force of ice action in the river. The design developed by HDR (Doeing and Swift, 2009) calls for AKDOT Class III and IV riprap in this area. Assuming AKDOT Class IV riprap is similar to the Class IV riprap described in Table C8.1 of NCHRP (2006), the D_{50} would be 15 inches or 1.3 ft and the D_{100} about 2.2 ft.

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A second location subject to extreme ice impacts is the Penobscot River at Bangor, Maine. Here the river banks are lined with riprap revetments with a D_{50} on the order of 1.5 ft and a D_{100} of nearly 3 ft. The banks slope at about 2.5H: 1:V. Based on general observations of bank protection and rock structures along northern New England rivers with dynamic ice runs, the average riprap size to withstand the ice appears to be roughly equivalent to the maximum ice thicknesses, which falls in the 1.5 – 2 ft range. Well graded riprap bedded in clayey banks appears to survive better than uniformly graded riprap.

Frequency of ice events that could potentially damage the bed protection.

The above analysis suggests that the rock may experience some damage during years with extreme breakup ice runs. This section reviews hydro-meteorological in an effort estimate the frequency of events that could potentially damage the bed protection on the ramp. These data are listed in Table 3. In the report "Computation of Ice Thickness: Yellowstone River at Glendive, MT", NWO provided ice-out dates at Glendive from 1969-2000. Additional ice-out dates were estimated from daily average discharges at Glendive and Sidney for the years of 2001-2008. The LYIP records provided detailed information on the ice breakups of 1910, 1911 and 1912 at Intake (Attachment A). The Years with known ice events on the lower Yellowstone are highlighted in yellow along with dates and daily average discharges at the Sidney Gage. For all these events, with the exception of 1911, daily average discharge Q_b is $\geq$ 25,000 cfs and calculated ice thickness $T_i \geq 20$ in.[7] The list of known ice events is relatively uncertain as many years with severe ice runs may be missing from the record. Searching the list for other years where $Q_b \geq$ 25,000 *and* $T_i \geq 20$ in produced and additional 5 probable event years between 1969 and 2008, bringing the total to 8 for the 39 year period of record. By this reasoning the annual probability of a severe ice event on the lower Yellowstone is 8/49=16% , or a recurrence interval of about 6 years.

[7] Ice thickness is calculated as $0.5 \times \sqrt{\text{maximum net AFDD}}$

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Table 3. Calculated ice thickness and discharge for ice out events on the lower Yellowstone River, 1910-2008.

Year		Calculated	Observed	Estimated	Average Discarge (cfs)	
	Maximum	IceThick.	Iceout Date	Iceout Date	on Iceout Date	
	Net AFDD	(in)	at Glendive	at Glendive	at Glendive	at Sidney
2008	2098	22.9		30-Mar	5,700	
2007	1465	19.1		16-Mar	8,000	
2006	1175	17.1		13-Mar	6,000	
2005	771	13.9		7-Mar	4,850	
2004	967	15.5		20-Mar	5,400	
2003	1413	18.8		17-Mar	29,500	28,000
2002	1364	18.5		12-Apr		5,600
2001	1332	18.2		25-Mar		8,000
2000	1843	21.5	4-Mar			6,600
1999	945	15.4	3-Mar			9,400
1998	1168	17.1	21-Mar			9,750
1997	2211	23.5	20-Mar			16,000
1996	2167	23.3	16-Mar			25,000
1995	1302	18.0	14-Mar			4,500
1994	1650	20.3	5-Mar			40,000
1993	1717	20.7	8-Mar			11,500
1992	484	11.0	28-Feb			6,200
1991	1221	17.5	15-Mar			7,000
1990	929	15.2	5-Mar			9,500
1989	1851	21.5	27-Mar			20,000
1988	1268	17.8	21-Mar			7,500
1987	550	11.7	5-Mar			9,000
1986	1905	21.8	27-Feb			25,000
1985	1891	21.7	24-Mar			10,000
1984	1639	20.2	18-Feb			8,800
1983	846	14.5	19-Feb			13,000
1982	2098	22.9	21-Feb			16,000
1981	759	13.8	22-Feb			10,500
1980	1242	17.6	27-Mar			9,800
1979	2808	26.5	17-Mar			35,000
1978	2396	24.5	21-Mar			39,000
1977	1453	19.1	16-Mar			6,000
1976	1278	17.9	19-Mar			11,500
1975	1158	17.0	11-Apr			13,000
1974	1430	18.9	6-Mar			8,300
1973	1234	17.6	10-Mar			12,000
1972	2190	23.4	12-Mar			39,000
1971	1905	21.8	16-Feb			39,000
1970	1472	19.2	1-Apr			7,000
1969	2448	24.7	20-Mar			25,000
1962	1929	22.0		18-Feb		30,000
1959	1774	21.1		20-Mar		42,000
1943	2138	23.1		21-Feb		50,000
1936	2968	27.2		6-Mar		27,000
1912	2524	25.1	28-Mar			83,200
1911	1811	21.3	3-Mar			9,000
1910	2064	22.7	4-Mar			30,000
			Observed			
			Ice Out Date			
			at Intake			

Discussion

The 30% riprap design for the rock ramp is based on existing design guidance. The total factor of safety is 1.5, 1.1 of which was the initial factor of safety in Eq. 3-3 and 1.4 by adding an additional 12 in to the layer thickness and scaling up the rock size proportionally. Although riprap revetments have been built to survive extreme ice action along northern rivers, no precedent was found for where an entire river channel is protected in a way similar to the proposed rock ramp at Intake. Due to the lack of design guidance specific to the Intake ramp case and the lack of comparable bed protection designs of this scale and cost, the level of confidence in the 30% design is well below 100%.

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The existing design guidance closest to the Intake case is for direct ice rideup on riprap revetments where the recommended D_{50} is twice the maximum ice thickness. From practical and cost standpoints this design is not feasible for the intake ramp.

The 30% design presented in this report calls for a $D_{50} \geq 1.3$ ft, $D_{100} \geq 2.5$ ft and a blanket thickness of at least 4 ft. In support of this less conservative design are several factors. First, the slope of the ramp surface is very small (≤ 0.06% longitudinally and ≤ 3% on the side slopes). Second, the surface of riprap on the ramp will be fairly smooth. Provided the smaller infill material is not removed, this will provide a better sliding surface for ice floes reducing the forces on the bed protection.

Concerns and unknowns remain. Some damage and O&M is to be expected following extreme ice events. A first concern is the potential the cumulative ice impacts of long duration ice runs affecting a large area of the ramp, rather than a fewer large hits over a more limited area. A second concern is the tendency for bunching up and rafting of ice floes that increases the potential severity of ice gouging of the bed. A third is the fact that ice runs can occur at relatively low discharge and stages which, based on the 1911, experience increases the likelihood of ice damages to the ramp.

Table 4. Summary of Estimated Ice Forces

Project Component	*Mechanism*	*Ice Load*
Dam Crest	Direct impact by large ice floes and static forces due to thermal expansion of the cover	15 kips/ft
Headworks 1992.7-1995.2’	Crushing impact by strong ice floes	15 kips/ft
Headworks 1987.6-1995.6’	Stresses from moving frazil ice mass	10 kips/ ft
Headworks › 1995’ and ‹ 1988’	Stresses from moving frazil ice mass	5 kips/ft
Rock Ramp Riprap (assuming 165 pcf stone)	Sliding of thick frazil ice masses	460 psf
	Impact and gouging by large ice floes	$D_{50} \geq 1.3$ ft $D_{100} \geq 2.5$ ft $T \geq 4.0$ ft
Rock Ramp Infill Material	Under ice hydraulic scour,	2.0 psf

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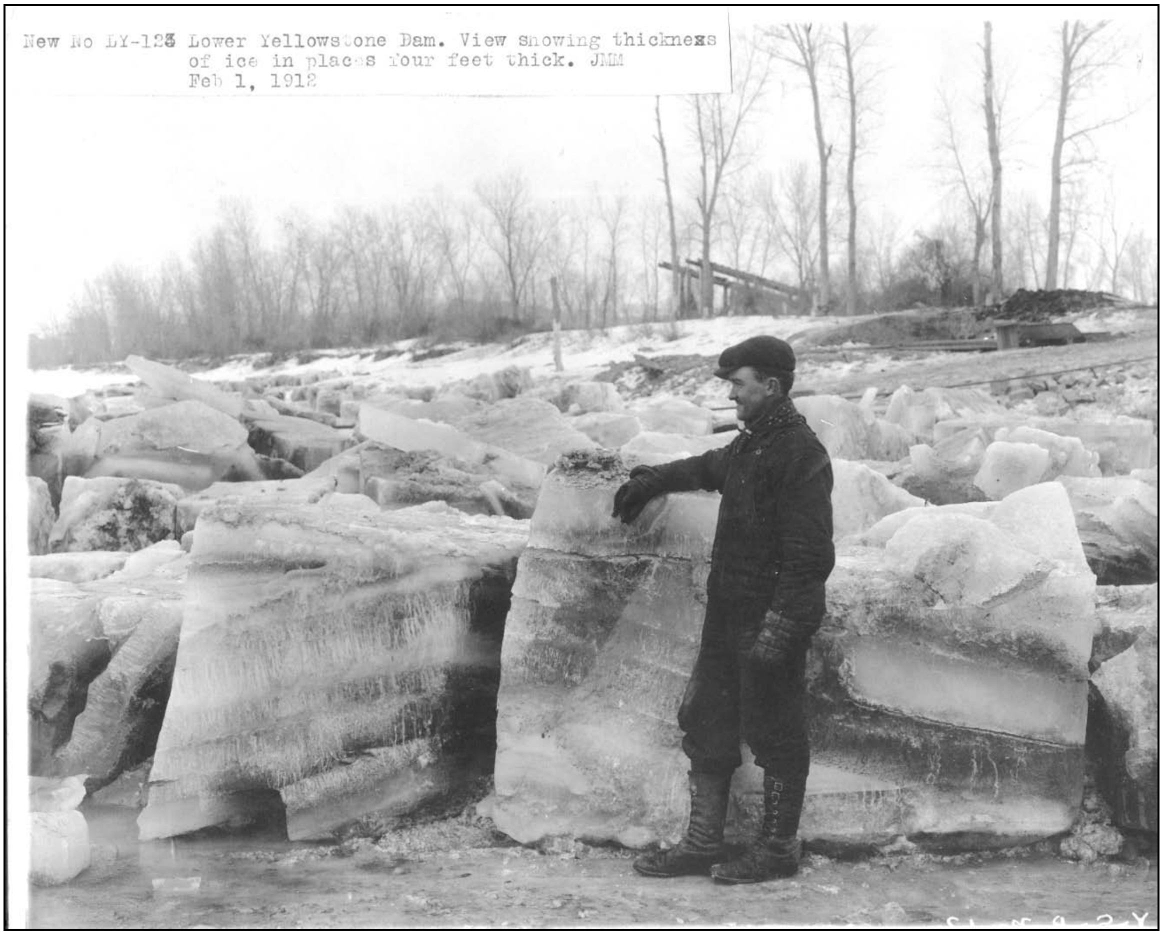
Ice run at newly completed Intake Dam on 3/4/1910



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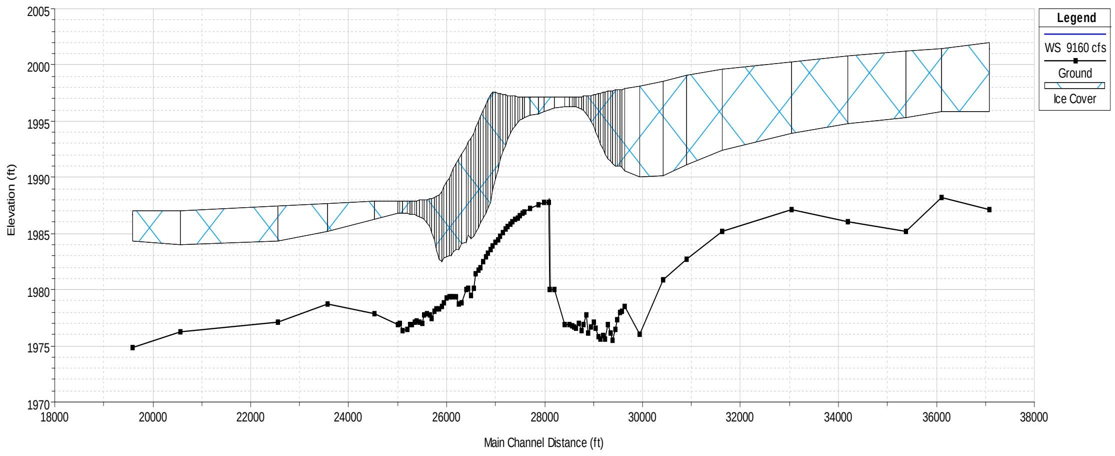

Figure 6.1.1. Ice jam profile. Q=9160 cfs

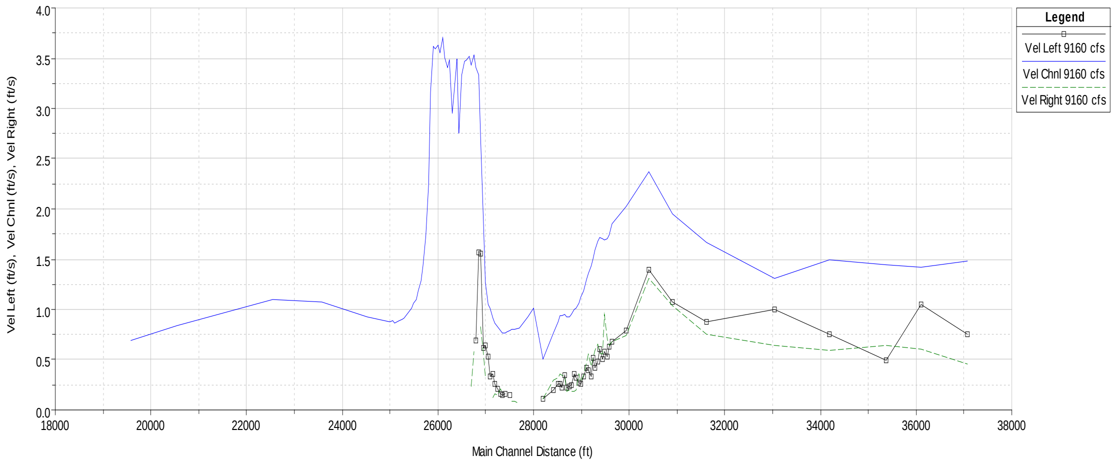

Figure 6.1.2. Water velocity beneath ice jam. Q=9160 cfs.

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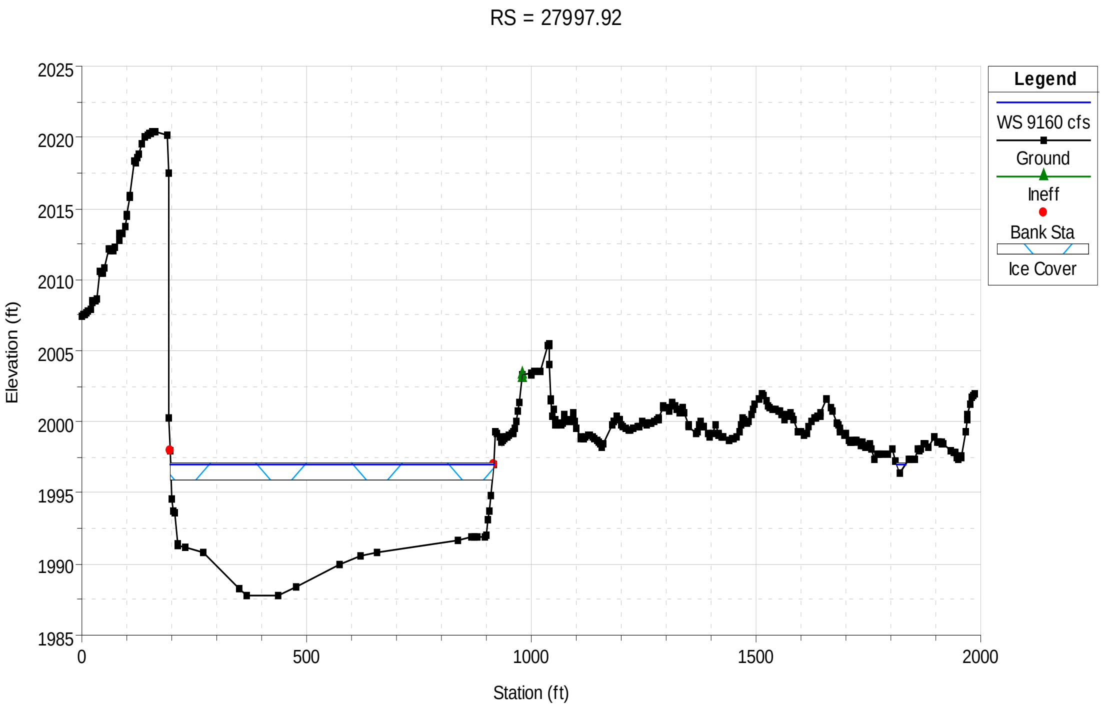

Figure 6.1.3. Cross section of ramp 110 ft below crest with ice jam. Q=9160 .

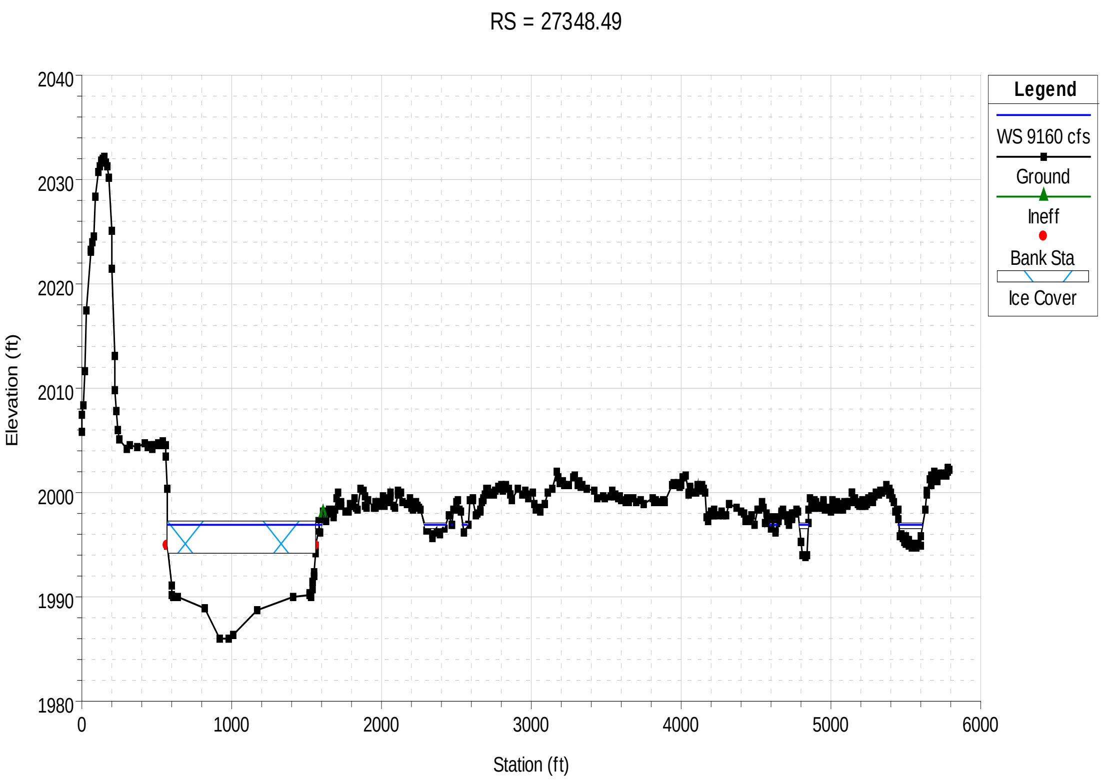

Figure 6.1.4. Cross section of ramp 744 ft below crest with ice jam. Q = 9160.

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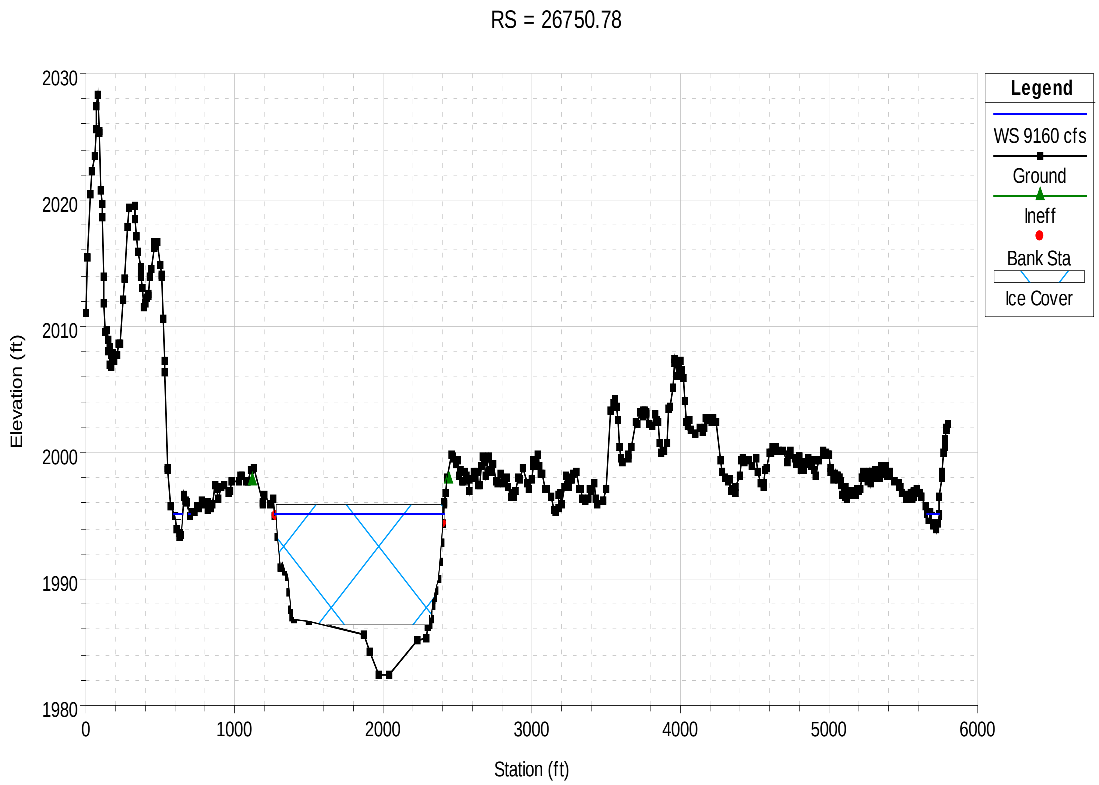

Figure 6.1.5. Cross section of ramp 1340 ft below crest with ice jam. Q = 9160 cfs.

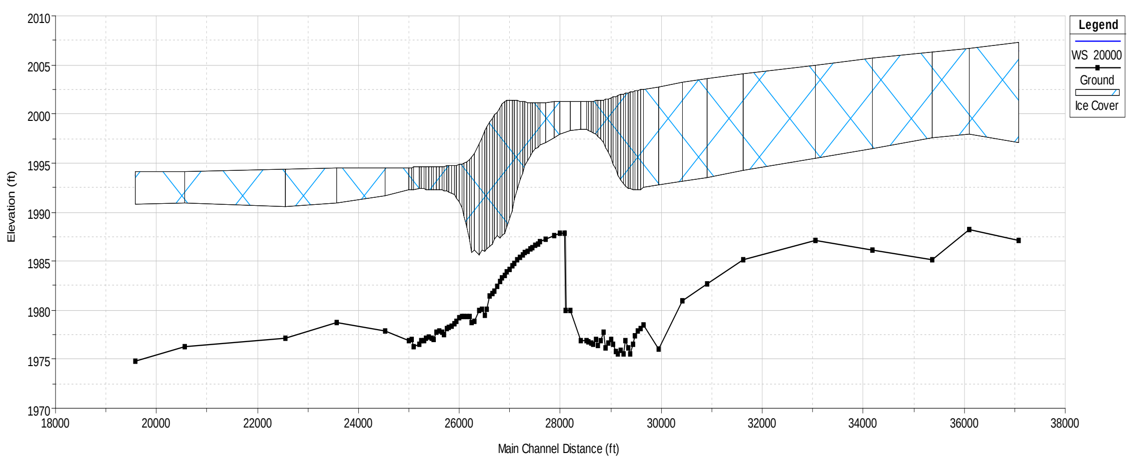

Figure 6.1.6. Ice jam profile. Q=20,000 cfs

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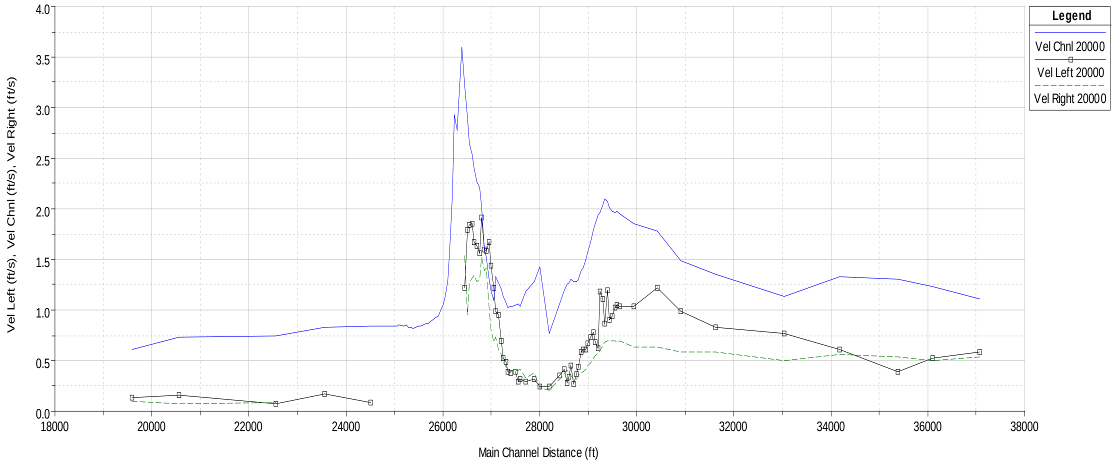

Figure 6.1.7. Water velocity beneath ice jam. Q=20,000 cfs.

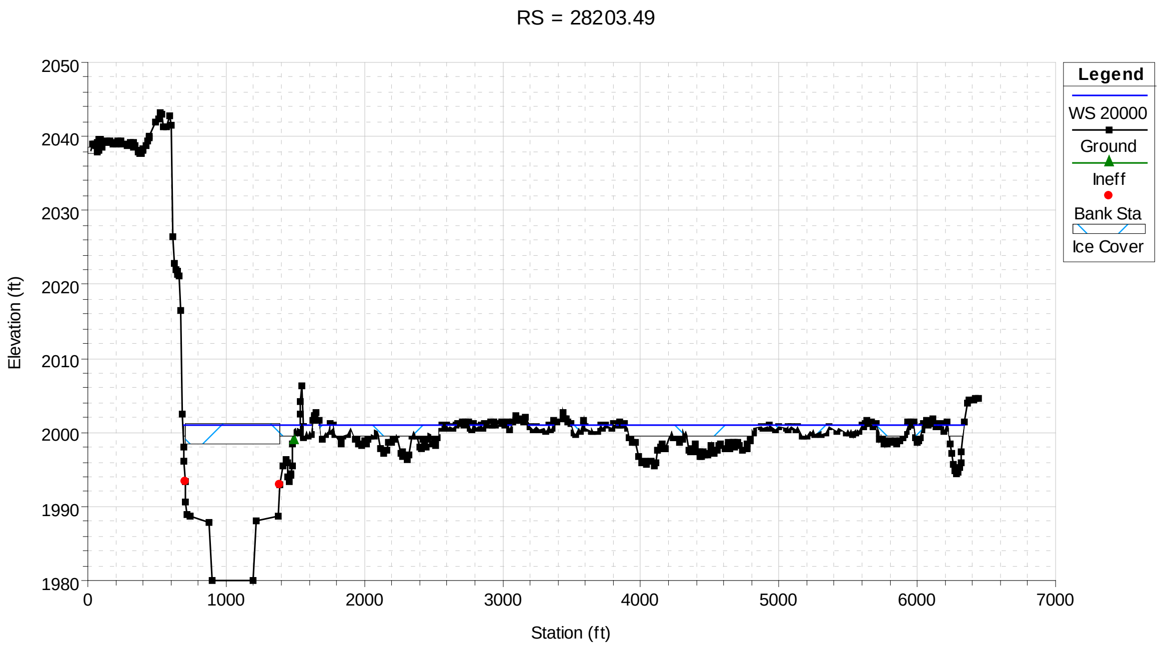

Figure 6.1.8. Cross section of ramp 110 ft below crest with ice jam. Q=20,000 .

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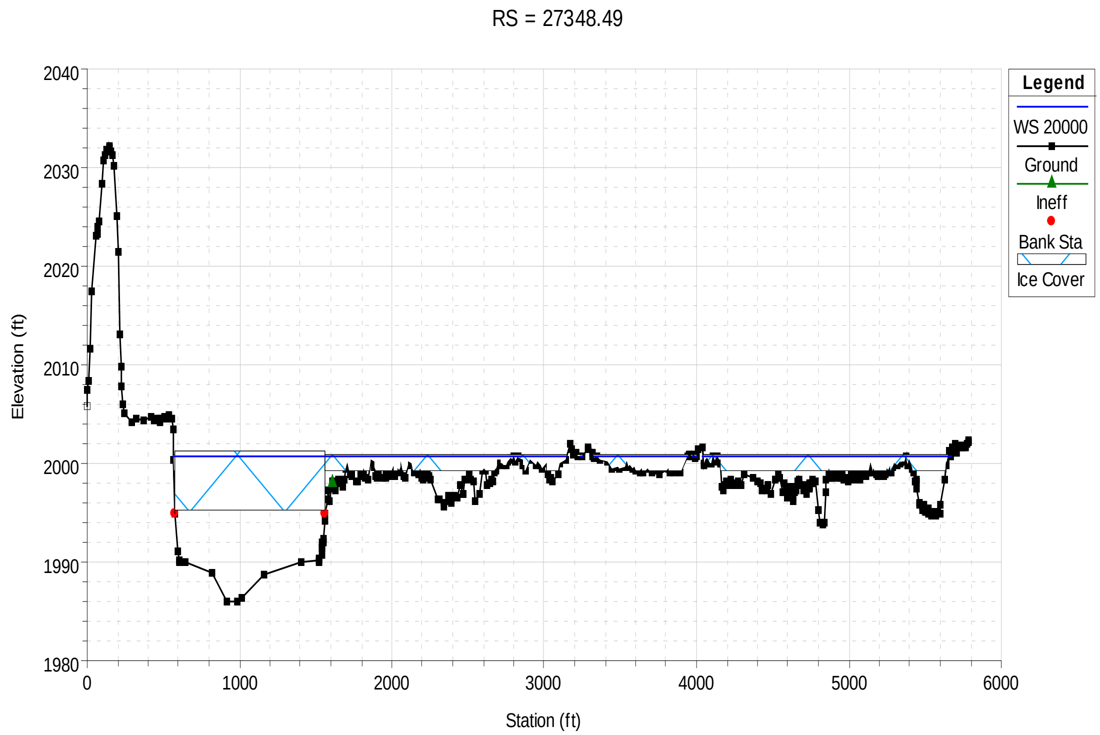

Figure 6.1.9. Cross section of ramp 744 ft below crest with ice jam. Q=20,000.

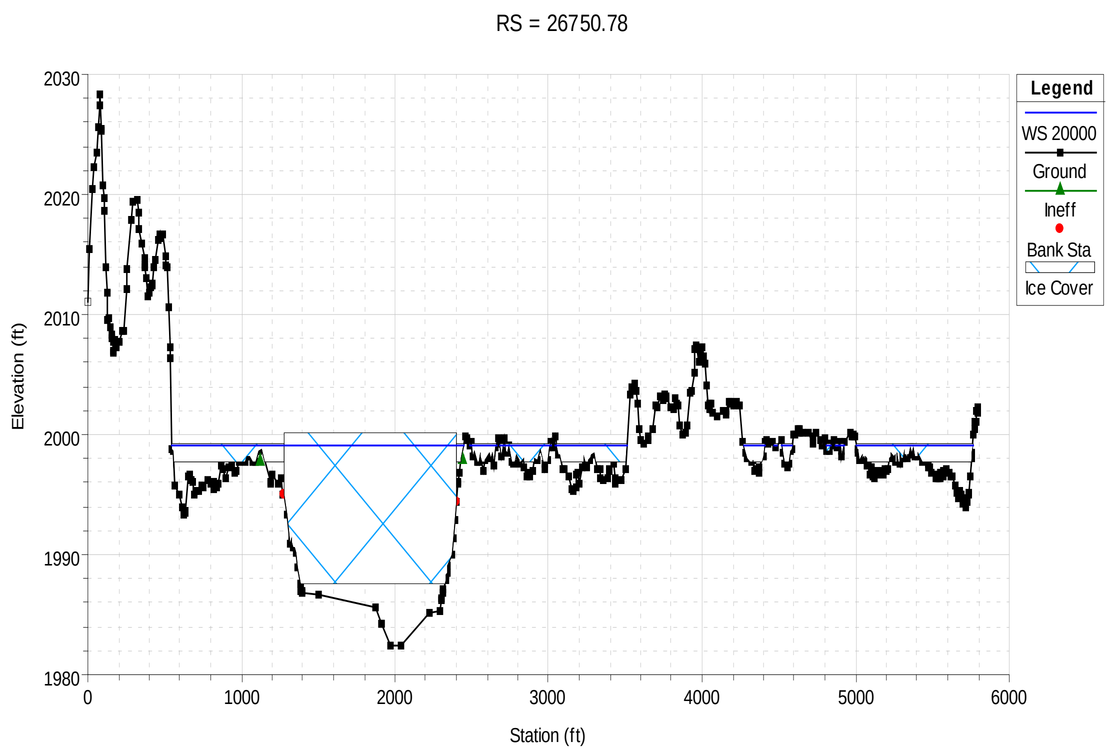

Figure 6.1.10. Cross section of ramp 1340 ft below crest with ice jam. Q=20,000.

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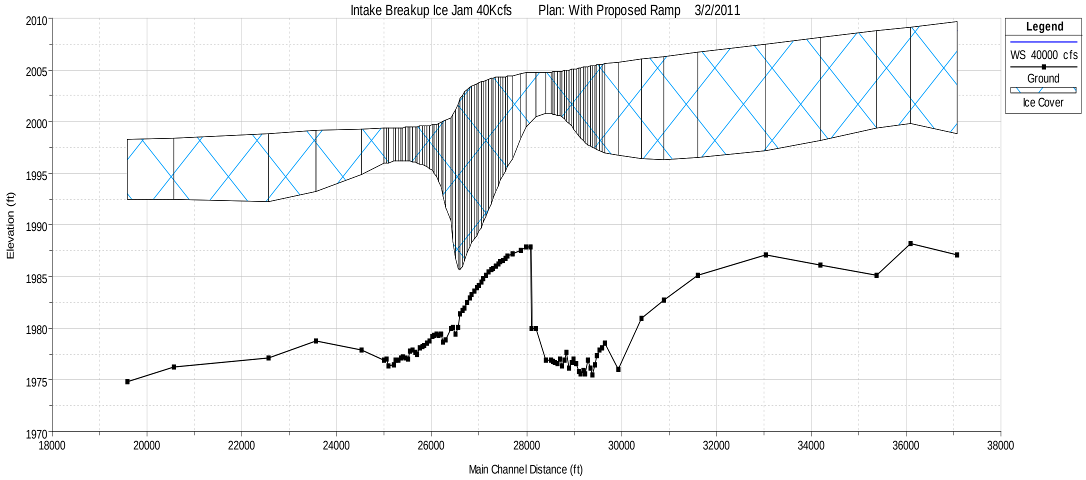

Figure 6.1.11. Ice jam profile. Q=40,000 cfs

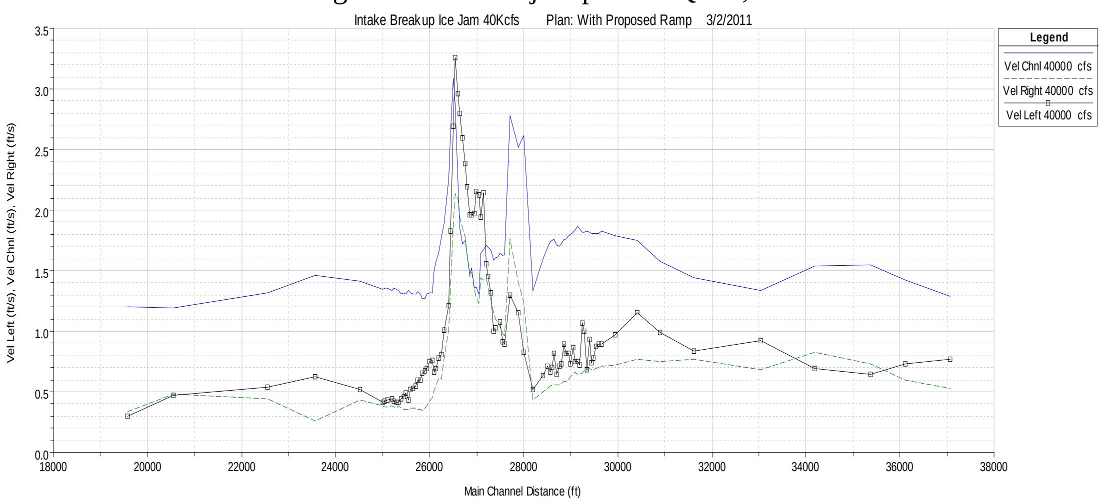

Figure 6.1.12. Water velocity beneath ice jam. Q=40,000 cfs.

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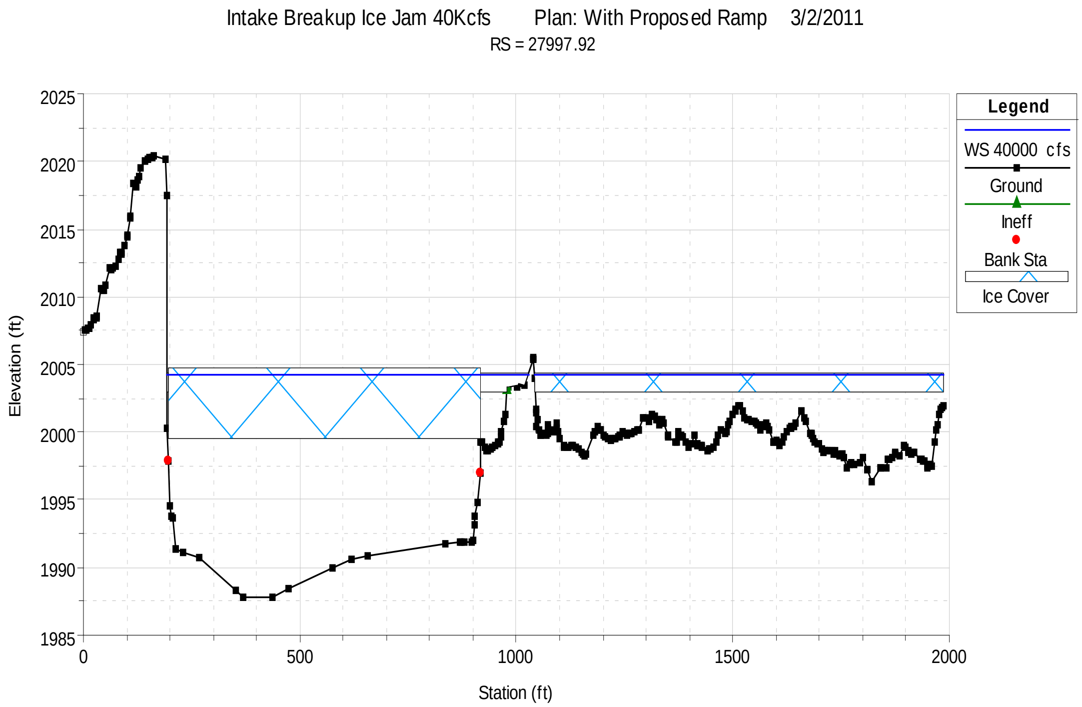

Figure 6.1.13. Cross section of ramp 110 ft below crest with ice jam. Q=40,000 .

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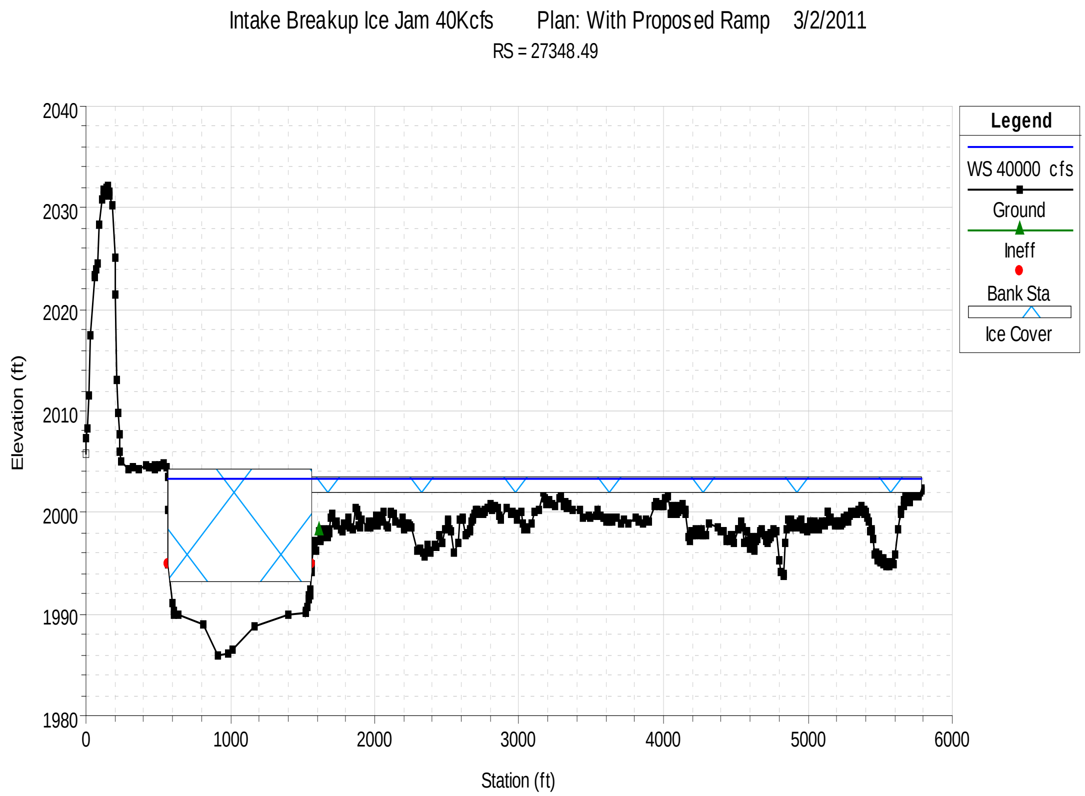

Figure 6.1.14. Cross section of ramp 744 ft below crest with ice jam. Q=40,000 .

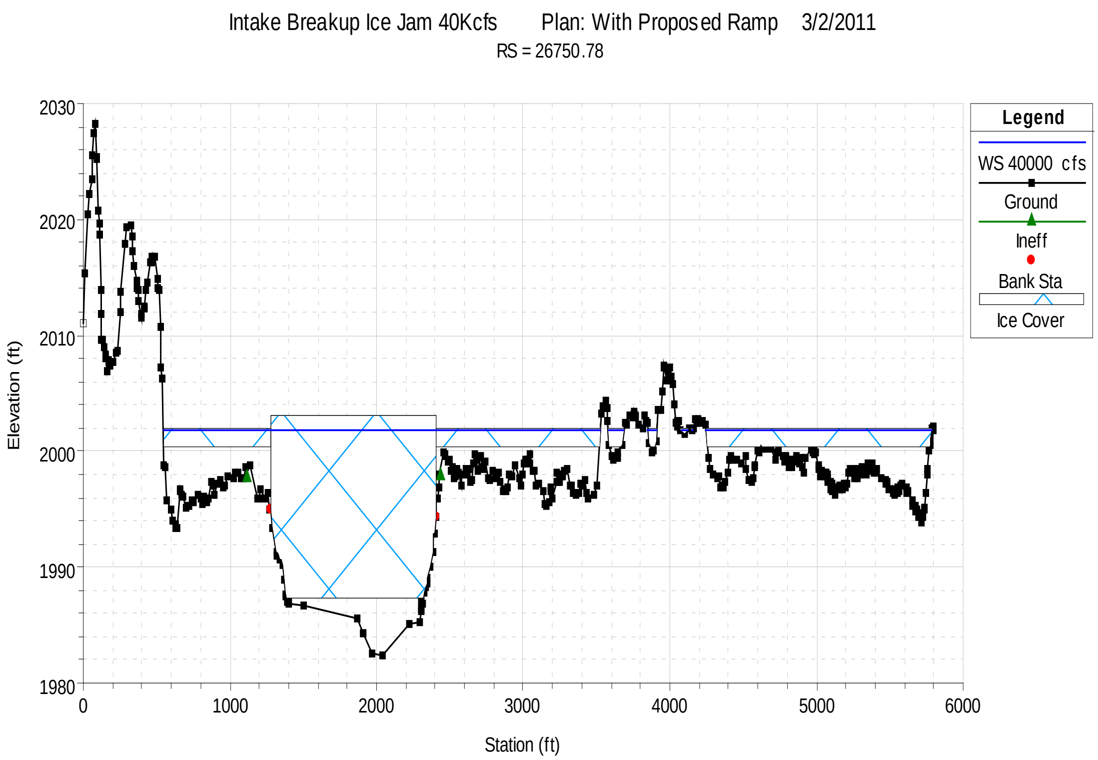

Figure 6.1.15. Cross section of ramp 1340 ft below crest with ice jam. Q=40,000 .

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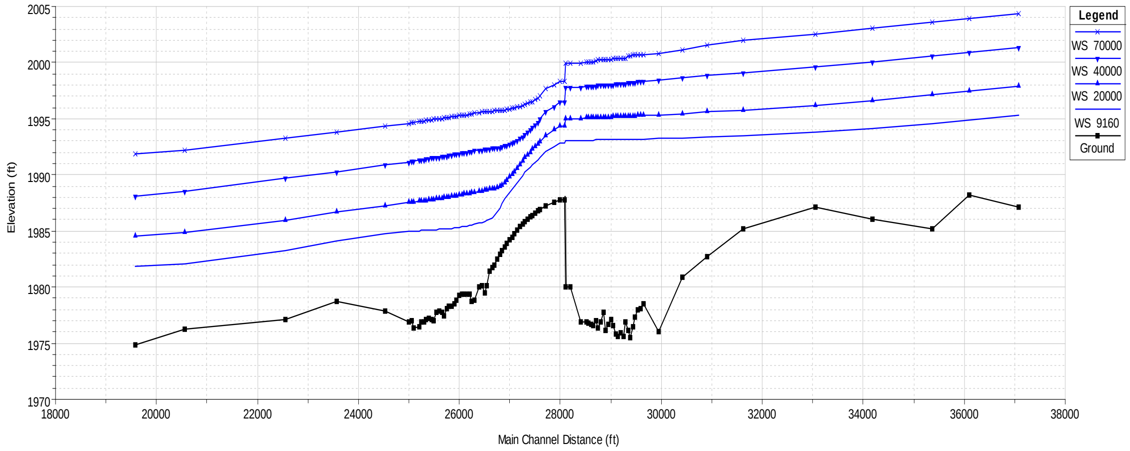

Figure 6.1.16. Open water profiles for Q= 9160, 20,000, 40,000 and 70,000 cfs.

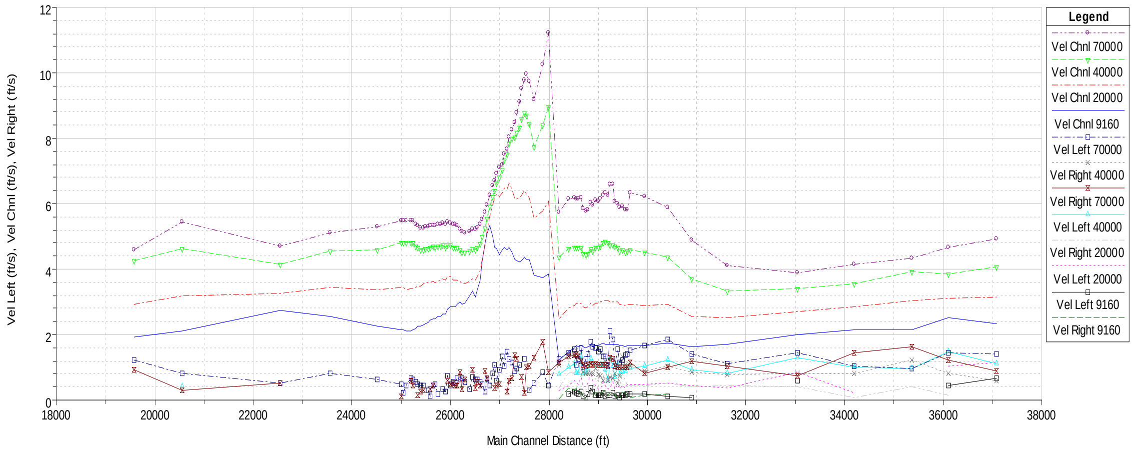

Figure 6.1.17. Average water velocity for Q= 9160, 20,000, 40,000 and 70,000 cfs.

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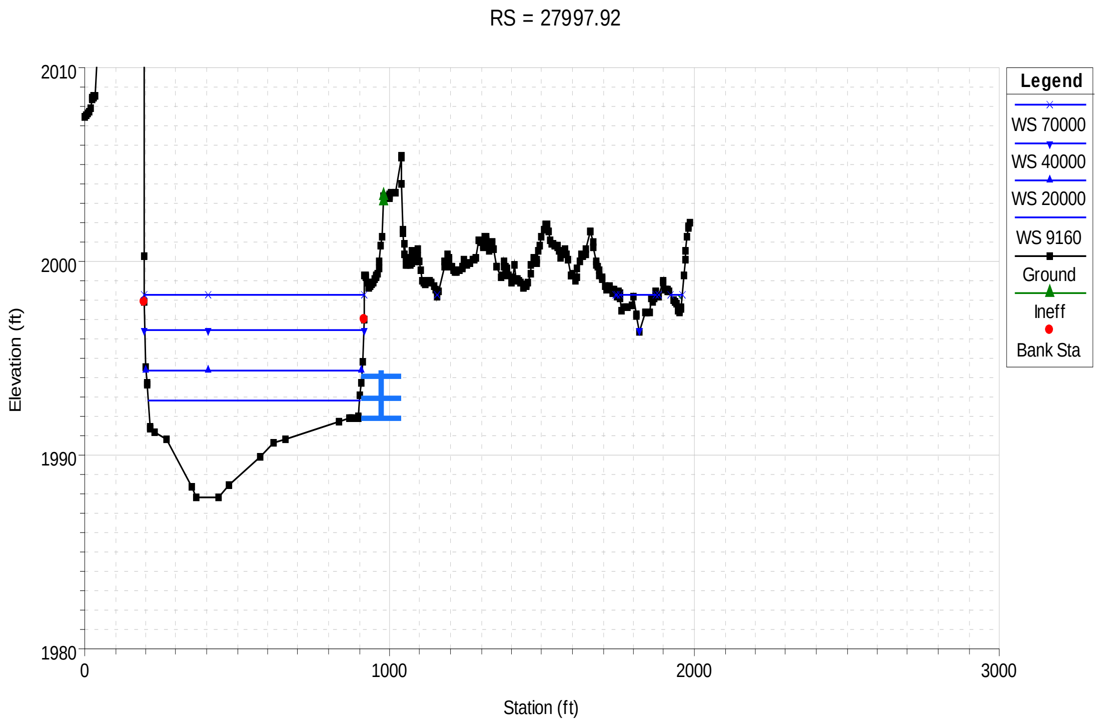

Figure 6.1.18. Cross section 92 ft below crest showing open water levels and 2-ft-thick ice floe.

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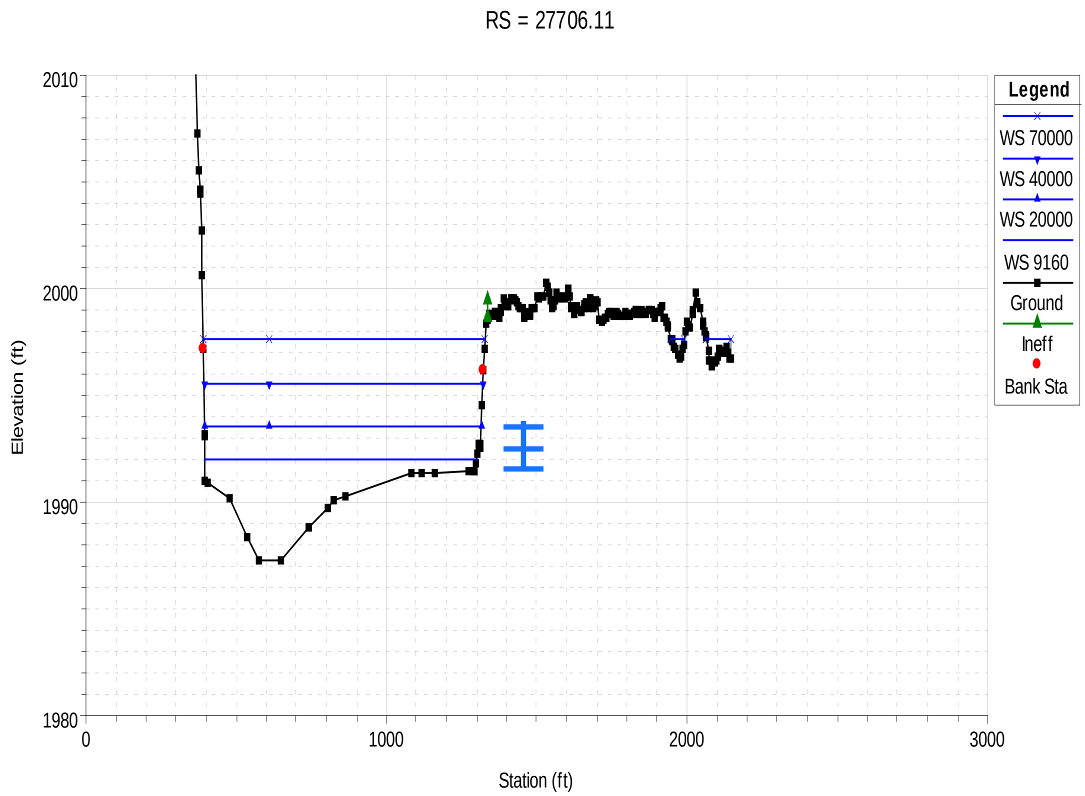

Figure 6.1.19. Cross section 386 ft below crest showing open water levels and 2-ft-thick ice floe.

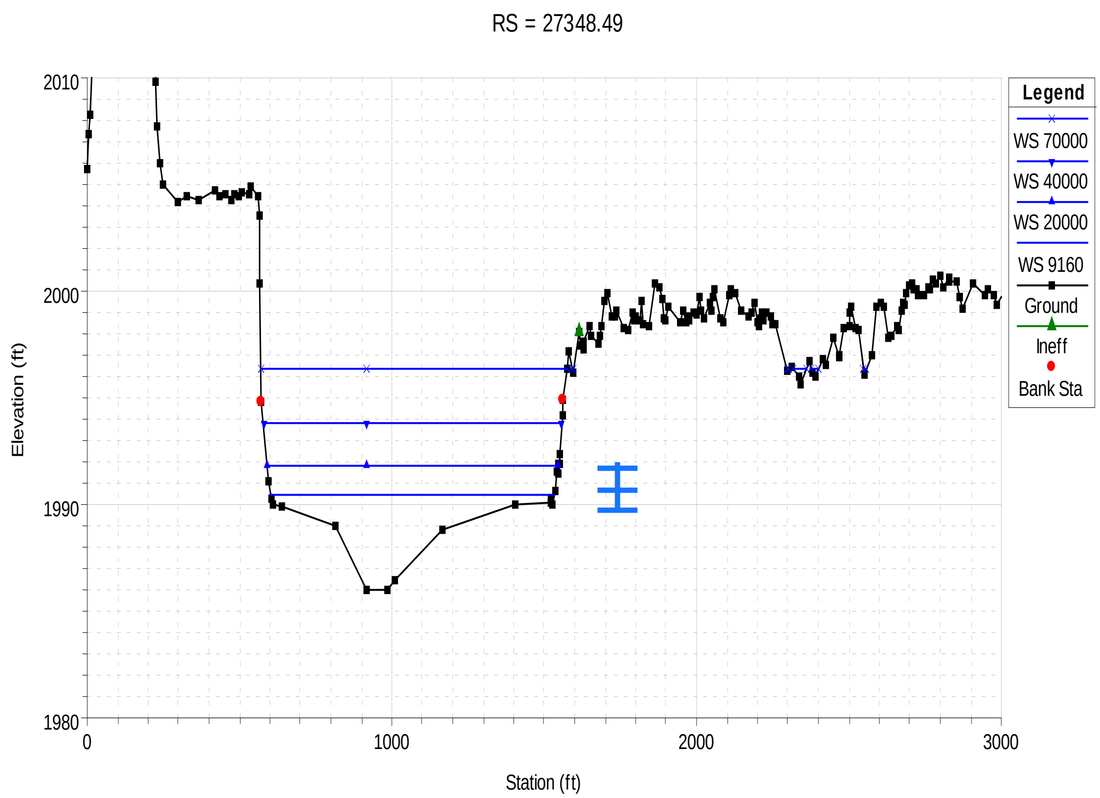

Figure 6.1.20. Cross section 744 ft below crest showing open water levels and 2-ft-thick ice floe.

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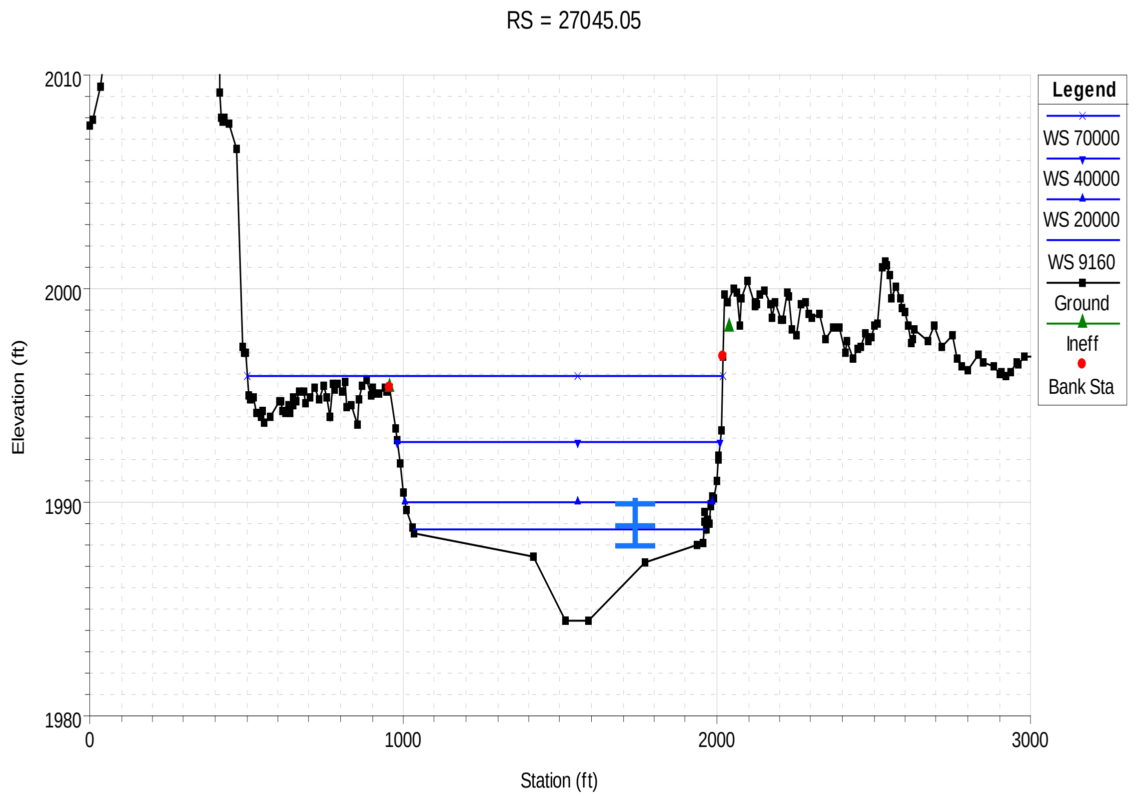

Figure 6.1.21. Cross section 1047 ft below crest showing open water levels and 2-ft-thick ice floe.

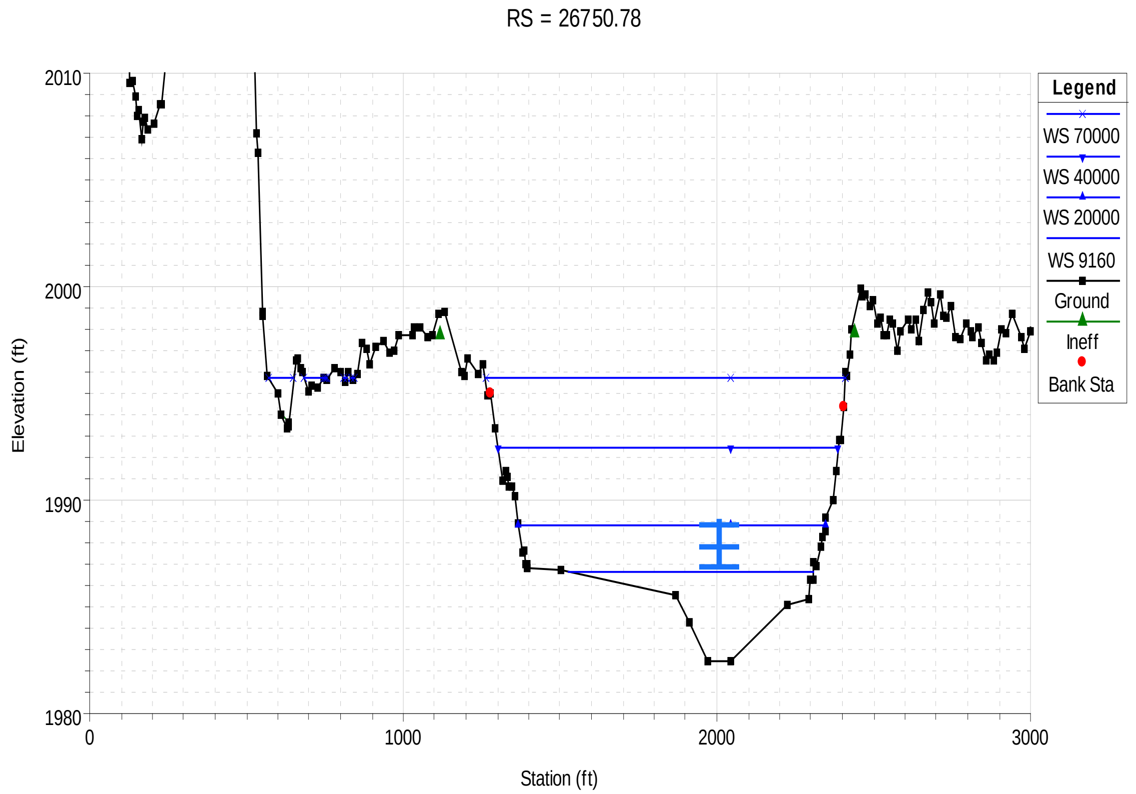

Figure 6.1.22. Cross section 1340 ft below crest showing open water levels and 2-ft-thick ice floe.

Appendix A January 14, 2011 Site Visit to Intake and Findings

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On the morning of Jan, 14, 2011, Andy Tuthill of CRREL met Justin Kuchera, Brad Coutant and Rick Hanson of Reclamation in Glendive, MT. We drove to Intake where Bruce Anderson of USACE, NWO showed us the dam from the north side. (Figure A1). A narrow lead came down the center of the upstream channel widening at the dam crest. In the apron section was several hundred ft of rapids that ran into deeper water below. Armor stone of sizes ranging from 1 to 3 ft were visible along the dam crest and in the rapids. A layer of ice, about 1.5 ft thick, covered portions of the rapids section. The wooden crest of the dam was barely visible under 1-2 ft of water and the downstream ogee section and wooden apron were completely rock covered. Justin remarked that, during a visit to the site last winter, the ice cover had been complete with no open water visible at the dam.

From Intake we drove to the offices of the Lower Yellowstone Irrigation Project in Sidney office at Sidney to meet with Jerry Nypen who has overseen operation at Intake for the last 15 years. Jerry described breakup on the lower Yellowstone as an extremely dynamic and often destructive process. The ice runs result from snowmelt driven runoff and can occur anytime between mid-February and mid-April, but more commonly in the mid-to-late March timeframe. Breakups vary in nature from fairly benign to extremely violent, with discharges ranging from 15,000 to 160,000 cfs.

The ice run he said can at times gouge the rock protection from the apron in a manner similar to driving a D-8 bulldozer downstream along the bed. The moving ice accumulation is not uniform in thickness and ice floes can pile up to scour sections of the dam and apron. Large floes can tilt on end and at times even launch in to the air. Following a large ice run, 30-ft-high shear walls may be left behind along the channel banks and it is not uncommon to see car-sized ice floes stranded on the floodplains.

On an average year, about two weeks are spent placing 300 to 1200 cubic yards (CY) of locally quarried sandstone along the dam crest using the 99-year old cableway (Figure A2). Assuming an average of 700 CY of solid rock are added per year over a period of 98 years, with 40 % voids, this represents a fill volume of about 115,000 CY of fill.

Subsequent passage of ice and floods serve to move much of this material including very large rocks downstream for distances as far as 2 miles. The original shape of the downstream ogee crest and wooden apron are hidden by this riprap whose average diameter is on the order of 1-2 ft. The largest rocks are 5-6 ft in diameter and smaller size fractions also included to provide infill material. The surface of the rock layer was non-uniform with the larger rocks protruding higher than the smaller ones.

Jerry said that the wooden crest of the dam and the downstream apron have been substantially rebuilt at least three times in the last century. Several of the major repairs have followed partial destruction by ice events. He remarked that the aftermath of a large ice event extends valley-wide at that the roar of the ice run can be heard from a mile away from the river. Jerry is not confident that any rock structure can withstand a severe ice run on the lower Yellowstone River without substantial damage.

Jerry provided reports and photographs from the construction of the dam and its first three years of operation from 1910 to 1912. These three years all experienced severe ice runs with the breakup of 1911 causing extensive damage to the project. These ice events and their impacts on the project are summarized below. Much of this information comes from Reclamation Services report “Lower Yellowstone Dam Feature History; March 4, 1910 to May 1, 1912”

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The 1910 ice run over the newly-completed dam which occurred on March 4 was quite severe. (Figure A3). The dam designers had assumed that ice would start passing the dam once the depth at the crest exceeded about 3 ft. and that the 9 ft-depth immediately downstream would provide a protective cushion for ice floes moving over the apron and downstream armor stone. Daily average flow at Glendive on 3/4/1910 was 30,000 cfs and the ice above the dam began to move at the expected depth of 3 ft at the crest. Observers noted that “crowding and jamming” of the ice as it moved over the dam and apron which raised concerns about potential damage to the structure. Soundings the following October found no damages however.

The 1911 ice broke up locally at the dam on March 9 with a depth of 3 ft at the dam crest. General breakup began on March 12 with a depth of 8.6 ft at the dam crest due to a downstream jam which released at 3:30 am. The ice run continued until 9 am with “much pounding of the apron”.   The release of a large jam from 50 miles upstream at Fallon reached Intake at 6 pm and continued until late in the night. Depth at the dam crest was 11.6 ft and “Much pounding of the apron was observed and heard”. No gage records exist for Glendive but daily average flow at Sidney for the March 98-12 is listed at only 9000 cfs.

Soundings taken that April showed serious erosion along and below the lower sheet pile cutoff wall and that much of the loose stone below the sheet piling had been moved downstream. A survey on November 1, 1911 found 500 ft of the wooden apron destroyed with much of the stone filling gone. A 500 ft length of the lower wooden sheet piling had also been broken down and scour “had progressed to such an extent as to render the dam unsafe”. Emergency repairs during the winter of 1911-12 included driving a row of steel sheet piling and placing 3800 CY of large rock above and below the new sheet piling. The rock was quarried locally from both sides of the river and drawn by horse and wagon out a trestle to be placed in the river using a floating derrick. A major part of the operation was cutting the 3-4-ft-thick ice cover to allow movement of the derrick (Figure A4). The cableway was also installed in the winter of 1912 to provide a more efficient means of adding rock to the apron in the future. The cost of repairs was $65,000, a sizeable sum considering the total cost of the dam had been $190,000.

The 1912 ice run was again severe, starting on March 28 at a daily average flow at Sidney of 83,200 cfs. A jam on a downstream bar (Figure A5) delayed the upstream ice release until depth at the dam crest had reached 4.8 ft. During the run, depth at the crest fluctuated between 9 and 14 ft, with ice velocities were estimated at 10-11 ft/s, peaking as high as 15 ft/s, “large cakes of ice were seen to strike the apron and sometimes…jump 10 ft above the water”. Similar to the previous year, a late night lull preceded the arrival of a second large ice run from Fallon that “pitched the ice on end as it passed over the dam” and caused “great pounding” to the apron (Figure A6).

Soundings taken in April 1913 found little serious damage to the structure. Engineers credited this in part to the higher discharges and water levels over the dam and apron. The ice did erode the crest timbers to a depth of 3-4 inches and the new steel sheet piling reportedly withstood the ice pretty well.

Review of these early reports indicates that the impact of ice was initially underestimated in the design of the Intake Dam. Following the events of 1910-1912, ice became the dominant issue in terms of engineering and construction activities to protect the dam.

It appears that discharge and water depth are major factors in terms of potential damage to the dam and apron during breakup. Of these three well documented events the 1911 event had the lowest flows and water levels and by far the greatest damages to the structure.

Respectfully Submitted:

Andrew M. Tuthill, P. E.
U S Army Cold Regions Research and Engineering Laboratory
72 Lyme Rd.
Hanover, NH 03755
603-646-4225 office
603-643-3354 home
603-306-6699 cell

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Figure A1. View of Intake Dam from the north on January 14, 2011. Flow is right to left.

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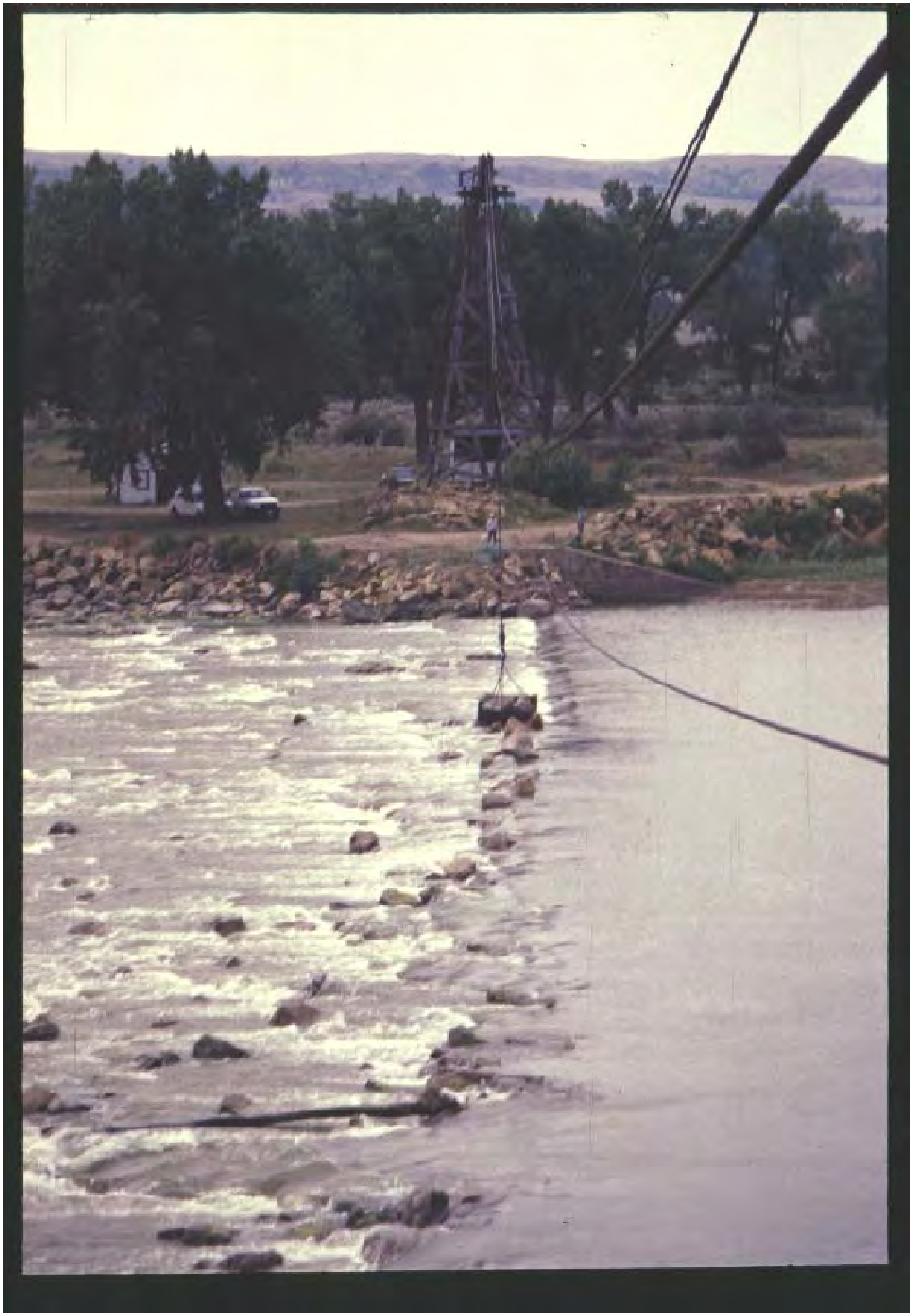

Figure A2. Adding armor stone to the dam apron using the cableway

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Figure A3. Ice moving over the newly completed dam on March 4, 1910.

Figure A4. Clearing ice from the river to repair the apron Feb. 1912.

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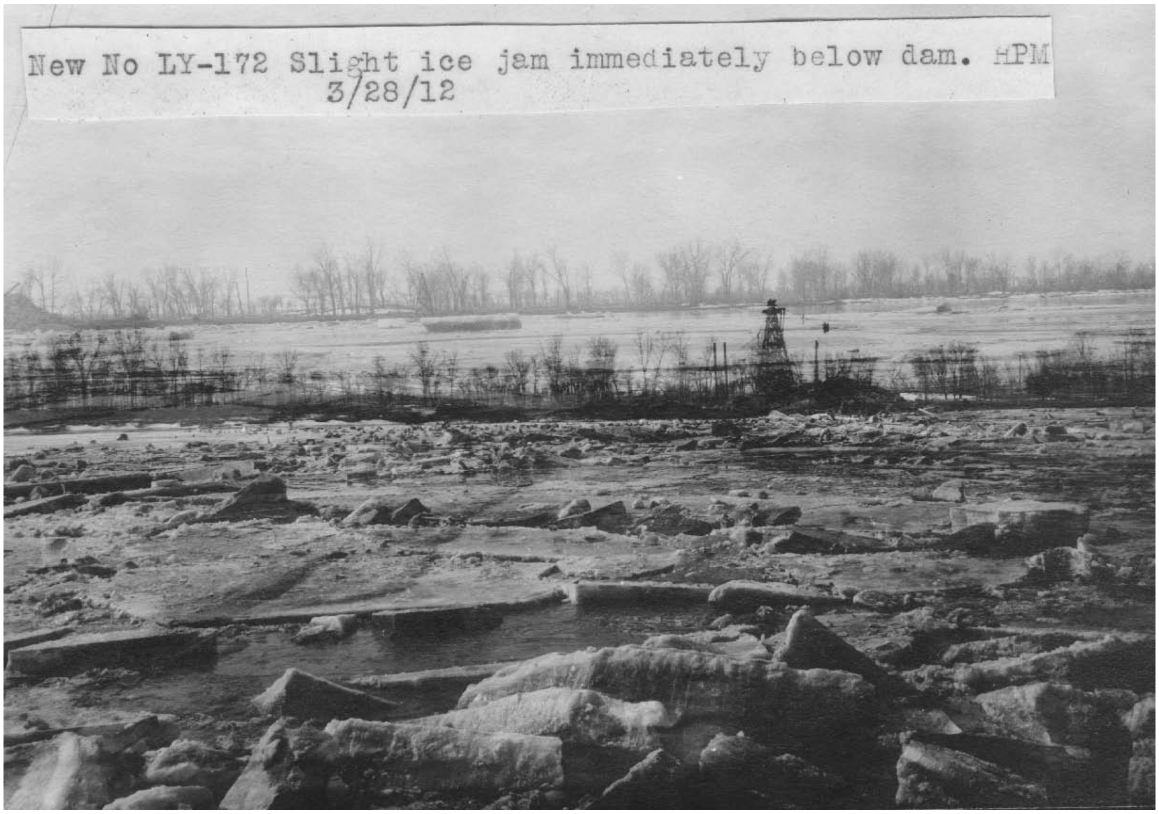

Figure A5. Jam below the Intake Dam at the onset of the March 28, 1912 breakup.

Figure A6. Second ice run during March 28, 1912.

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Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Intake Fish Passage Option Evaluation Summary
May 2012

Attachment 1b

Ice Forces-Intake Bypass Channel
Lower Yellowstone River, 30% Design

Prepared by:
Andrew Tuthill, Cold Regions Research and Engineering Laboratory (CRREL)

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Evaluation of Ice Impacts on Fish Bypass Channel at Intake Dam, Lower Yellowstone River

Andrew M. Tuthill
Meredith L. Carr

US Army Corps of Engineers
Cold Regions Research and Engineering Laboratory
72 Lyme Road, Hanover, NH 03755

February 12, 2012

1. Introduction

The Lower Yellowstone Project at Intake is a Bureau of Reclamation irrigation project located on the Yellowstone River approximately 70 miles upstream from the confluence with the Missouri River. The project consists of a low-head diversion dam, a diversion headworks structure, and an irrigation canal system to deliver water to approximately 53,000 acres in Eastern Montana and Western North Dakota.

The diversion dam is a known barrier to native fish migration including endangered pallid sturgeon. The canal has been documented to entrain many thousands of fish during diversion operations (April through September). Bureau of Reclamation has an obligation, under the Endangered Species Act, to modify the structure or the operation of this facility to address pallid sturgeon concerns raised by USFWS and the Montana Department of Fish, Wildlife, and Parks. The Corps has been working with the Bureau of Reclamation to develop plans to construct a new headworks with screens and also provide fish passage. Two fish passage alternatives under consideration consist of a full river width rock ramp at an average slope of 0.5 to 1% and a bypass channel of 10,000 to 15,000 feet in length that would provide habitat similar to existing natural chutes. Figure 1 shows the preferred alignment of the bypass channel and its structural components.

Omaha District requested ERDC/CRREL[1] to provide engineering design guidance related to ice on the bypass channel and associated structures. This effort follows previous work by CRREL in 2011 that estimated ice forces on the intake dam and the new headworks structure and provided ice related design guidance for the rock ramp.

2. Design Background

The new headworks structure is currently under construction and will be in service for the 2012 irrigation season. A preliminary diversion dam and rock ramp fish passage concept design was completed in spring 2010. The next engineering phase identified unacceptable cost escalation associated with the rock ramp design however. This led to consideration of additional fish passage alternatives during preparation of a Decision Document (USACE March 2011a). One alternative is the construction of a bypass channel. The preliminary design assessed performance based on bypass channel geometry and hydraulic conditions needed for fish passage (USACE March2011b). An updated design

[1]Engineer Research and Development Center/Cold Regions Research and Engineering Laboratory

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(USACE January 2012) provided greater detail on various project components with slight revisions to channel geometry. The ice design information presented in this report will be incorporated in a 30% concept level design due in April 2012.

The ice force design effort draws on previous ice analyses done in 10% and 30% designs of the intake headworks, new dam and rock ramp as well as HEC-RAS modeling of hydraulic and ice processes in the preferred bypass channel and adjacent river.

3. Approach

This study began with a review of previous design efforts for the bypass channel (USACE March 2011a) as well as earlier ice analyses associated the design of the headworks, diversion dam and rock ramp. (USACE, March 2011a and January 2012). The previous literature review of related ice issues will be updated to include information on ice processes associated with bypass channels and chutes.

Ice-interaction was analyzed for the following project components:

a. Upstream control structure at bypass channel inlet (referred to as "exit" from fish perspective)

b. Channel plug where bypass channel diverges from path of existing natural chute.

c. Riprap at bypass channel bends for lateral stability

d. Vertical grade control structures along bypass channel

e. Downstream vertical control structure (referred to as "entrance" from fish perspective) where bypass channel re-enters Yellowstone River below dam.

f. Lateral stability structure along Yellowstone R. below bypass channel outlet.

g. New dam

h. Flow augmentation weir parallel to the Yellowstone River right bank immediately upstream of the diversion dam. This weir would provide additional attractive flow to the bypass channel entrance downstream of the dam during high flow events.

These structures are shown on Figure 1 and described in USACE, March 2011a and USACE January 2012.

For the purpose of design for ice forces, a worst case ice formation, breakup, jam and release scenario was developed similar to the approach in previous ice design analyses. Figure 2 shows the hydrograph for the Yellowstone River at Miles City and Sidney for the winter of 1996 which had the most severe ice jamming in recent history. The ice scenario starts with a hydraulically thickened ice cover forming during the early winter at flows in the 8-10 Kcfs range that remains in place until mid-March-early April when flow increases to an assumed breakup level of about 20 Kcfs. A large ice jam is assumed to form downstream of the Intake Dam as it has historically. As discharge continues to increase the jam in the main river channel forces flow and ice into the right overbank and bypass channel. It is assumed that the ice cover breaks up and formas a smaller jam in the bypass channel. At about 40 Kcfs the jams are assumed to release and the flow impounded by the jam surges downstream in the river channel and floodplain area leaving behind high shear walls and large ice pieces in the right overbank area. Figure 3 shows the aftermath of such an event which occurred in March of 1912.

This process of ice formation, breakup and subsequent ice run was modeled using HEC-RAS and the resulting water surface and ice jam profiles used to evaluate ice-structure interaction. For concrete structures such as weirs, design ice forces and heights of ice structure interaction are estimated based on established bridge design codes such as AASHTO (1998) and ice loading guidance found in the Ice Engineering Manual (US Army, 1999). In this project, the main type of ice interaction will be with riprap structures where the lack of theoretical guidance relating ice forces to rock stability necessitates a more empirical approach such increasing the riprap layer thickness.

The design approach for the riprap structures followed an approach similar to the one used in the design of the rock ramp (USACE, 2011a). The rule of thumb taken from lab tests by Sodhi et al. (1996, 1997, and 1999) recommends the D_{50} of the riprap should be 2-3 times the maximum ice thickness. This was not used in the design of the bypass channel for reasons as explained in the previous ice analysis (USACE, 2011a) . These included cost, difficulty of finding and placing rock that large and the fact that the Yellowstone situation is different from the ice ride-up tests upon which the guidance was based. The approach taken was to design the riprap structures based on hydraulic conditions of the 100-year open water flood and add 1.0 ft the layer thickness *T*, scaling up the rock size distribution proportionally.

4. Ice Processes Related to Chutes and Bypass Channels

The literature review of ice processes related to chutes and bypass channels is not yet complete. Based on experience with large ice-affected rivers, ice processes play a major role in terms of overbank flooding and the flow to and from the floodplain. A major difference between fluvial and ice-affected processes is that ice jams may cause flow in overbank areas at much lower discharges than in open water conditions. The HEC-RAS analysis done in this study proved this out. On the lower Yellowstone River, breakup typically progresses downstream from warmer to colder climate in a series ice jams and releases. Jams in the main channel often push flow and ice into side channels and chutes, leaving behind high shear walls and ice pieces in the overbank areas when the jam releases as shown in Figure. 3. As the hydrograph increases to the breakup level, one would expect flow in overbank chutes to increase, floating up the freezeup ice cover and possibly forming small jams. The main breakup ice action would be expected to occur in the main channel however due to the higher velocities and depths and much greater ice supply. When these jams form as they have historically at many locations between Glendive and Sidney, the wide floodplains and side channels serve as relief mechanism accepting and storing flow and ice. Under these conditions, the flow area is large and overbank water velocities relatively low (≤ ~2 ft/s by HEC-RAS calculations at 40 Kcfs)

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which turns out to be a mitigation factor in terms of the design of bypass channel structures.

5. Ice-Hydraulic Processes Related to ice Loads on the Project

The ice analyses for previous design efforts, diversion found the lower Yellowstone River to be subject to heavy ice formation, dynamic ice breakups and ice jams. Because the Yellowstone flows northeastward from warmer to colder climate, the ice breakup progresses downstream in a series of jams and releases, and ice jam severity tends to increase in the downstream direction as the breaking front encounters stronger thicker ice. These events force flow and ice out of bank, either in side channels and chutes or over the entire floodplain width. Numerous ice jams and ice jam floods have occurred upstream of Intake at Glendive and downstream at Sidney (Haehnel and Tuthill, 2006). Jams have also been reported at Intake in the vicinity of the Richland County Line, Elk Island and Savage. All this suggests that the project reach is subject to the dynamic formation and release of ice jams. The most recent severe ice jam event on the Yellowstone occurred in February 7-13, 1996. Figure 2 shows the Yellowstone River discharge and AFDD for that winter at Miles City and Sidney.

On faster flowing rivers such as the Yellowstone, the predominant ice type is frazil which forms as small particles in super-cooled open water reaches. The frazil crystals stick together (flocculate) to form floes that tend to increase in size with distance traveled. The floes may accumulate along the channel sides to form border ice or stall in slack areas or channel obstructions to build an ice cover in the upstream direction. Only where water currents are slow (≤ 1 ft/s) can in situ thermal ice growth be expected. In the 1 to 1-1/4 ft/s velocity range, the frazil floes will accumulate edge-to-edge in a process known as juxtaposition. At higher water velocities, the floes will stack or "shove" into a thicker ice accumulation. The HEC-RAS model contains an ice routine that calculates ice accumulation thickness by these processes for both the freezeup and breakup cases.

Average December-January discharge at Sidney gage is 5800 cfs with a standard deviation of 1680 cfs for the 1910-2009 period (6900 cfs for post-Yellowtail Dam time frame). A higher freezeup discharge will cause a thicker freezeup ice accumulation, since the water velocities and shear forces on the ice underside will be greater. For the purposes of this study, an extreme case freezeup discharge is defined as the long term December-January average flow plus two standard deviations or 9160 cfs. Figures 4 and 5 show HEC-RAS simulated freezeup ice covers in the main river and bypass channel respectively for this flow level. Upstream of the bypass inlet, the shoved frazil ice accumulation in the main river is a much as 8 ft thick while in the bypass channel the simulated freezeup ice cover is hydraulically thickened to about 3 ft thick.

From review of past ice jam events, is estimated that a late-season ice cover such will release in the project reach at a discharge of about 20 Kcfs[2]. Figures 6 and 7 show this pre-release condition. Also, it is assumed that a breakup ice jam in the project area will

[2] Review of the early project reports indicates that the ice could release once depth at the dam crest exceeded 3 ft at river flows as low as 9,000 cfs.

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release at a discharge of about 40 Kcfs[3]. This is based on the Sidney Gage data that give the annual peak on 3/14/1996 of 19.48 ft (instantaneous peak Q = 30 Kcfs) as ice-affected while the 3/6/1994 peak of 24.03 ft (peak Q=75 Kcfs) is listed as open water. In 1994 ice jams were reported at many locations on the lower Yellowstone, but the river may have been clear of ice by the time of the instantaneous peak on 3/6. HEC-RAS was used to calculate breakup ice jam profiles in the vicinity of the project at discharges of 40 Kcfs (Figures 8 and 9 respectively).

Once the ice cover releases, it is assumed that the floes and thicker frazil ice masses travel downstream and impact the project at approximately open water surface elevations (WSE). Open water surface and velocity profiles were calculated for discharges of 20, 40, 60, 80 and 100 Kcfs (Figures 10 and 11). These elevations are used to estimate the height range that the ice floes and ice masses could impact bypass channel structural elements, as discussed in the next section.

6. Ice Forces and Design of Riprap

Most of the structural components affected by ice consist of riprap. The two concrete structures are the sill at the inlet to the bypass channel and the flow augmentation weir near the downstream end. The structures and their ice design issues are discussed below. Hydraulic and riprap design information is summarized in Table 1.

The 100-year event riprap size was calculated by the Isbash Equation which relies on water velocity, rock density and a stability coefficient (0.86 used in this case). The riprap was also sized by methods from USACE (2011a) which uses water velocity, flow depth and a number of empirical coefficients. This EM is one of the few design documents that considers ice, stating that in cases of heavy ice or debris loadings, the layer thickness should be increased 0.5 to 1.0 ft. Since conditions of heavy ice are expected in the bypass channel area, the open water design layer thickness *T* was increased by 1.0 ft and the rock size scaled up proportionally. Finally the riprap designs by these two methods, factored for ice were compared to preliminary riprap designs provided by the Omaha District (USACE, 2011b and Table 1).

a. Upstream Control Structure at Bypass Inlet

The plans for the upstream control structure call for a 15-ft long by 60-ft wide concrete sill surrounded by riprap. This is probably the most critical structure in terms of vulnerability to ice as its upstream approach lies on the outside of a bend and will be exposed to the full impact of ice runs on the main river. The Omaha District (NWO) design calls for Type C riprap (D_{50} = 12 in) and a layer thickness *T* of 3.5 ft for the 3.5:1 upstream and downstream slopes and 5:1 side slopes. The ice-factored Isbash and Corps EM methods give rock sizes and bed thicknesses quite similar to the NWO design. In terms of ice action, for the 20 Kcfs and greater flow range where breakup ice movement would be expected, the water depth and ice clearance over the 1990.3 ft elevation sill and

[3] These ice cover and breakup ice jam release discharges are very approximate and will vary greatly depending on ice thickness and ice strength.

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riprap blankets would be sufficient to avoid major ice impacts (Figures. 7a and 9a). Possible areas of vulnerability in terms of ice are 1.) The left hand side slope where the Yellowstone River transitions into the bypass, and 2.) The upstream interface between the concrete sill and the riprap bed. For the first case, one might consider increasing the average rock size to 16-24 in and the bed thickness to at least 4 ft. For the second case, should some of the riprap get scoured away exposing the front edge of the sill, the sill should be designed to withstand a horizontal ice loading of 10 kips/lineal ft.

b. Channel Plug

The channel plug being located off the alignment of the diversion channel will likely not experience many breakup ice impacts. By the time the assumed breakup flow of 20 Kcfs is reached, the bottom of the bypass ice cover would still be below the 2000 ft elevation of the plug crest so ice would not be expected to pass the structure (Figure 7a). At the 40 Kcfs assumed ice jam release discharge, the bottom of the bypass ice cover would be well above the crest of the adjacent channel plug (Figure 9a). Also, with overbank flow velocities on the order of 1 ft/s (Figure 9b), one would not expect rapid downstream movement of ice from the bypass channel to the location of the channel plug.

The preliminary riprap design proposed by the Omaha District is more than adequate to withstand conditions of severe ice based on the ice-factored Ishbash and EM 1110-2-1601 approaches (Table 1).

c. Riprap at Bends for Lateral Stability

The preliminary Omaha District plan calls for armoring the bypass channel bends with riprap with a D_{100} of 16 inches and a layer thickness of 24 inches. This is based on a velocity of 8.75 ft/s. Assuming a rock unit weight of 165 pcf and an Ishbash coefficient of 0.86, the calculated D_{50} would be about 12 in. In this case, the ice-factored Ishbash and EM 1110-2-1601 rock sizes and thicknesses are slightly greater than those calculated by NWO (Table 1).

The bend riprap protection is planned to extend up to the 10-year open water elevation. In the case of the assumed 20 Kcfs breakup discharge the top of the riprap would be at the mid-jam elevation (Figure 7a). For the assumed release discharge of 40 Kcfs, the bottom of an ice jam on the bypass channel, if it were still in place would be about 5 ft above the top of the riprap. Depending on how the ice jam release occurs, this process could result in ice impacts to the riprap.

d. Vertical Control Structures in Bypass Channel and at Outlet

The preliminary riprap design by the District gives comparable results to the ice-factored Ishbash and EM 1110-2-1601 approaches (Table 1). The tops of these vertical control structures will be

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1-2 ft below the channel invert as indicated in the HEC-RAS water surface and ice jam profiles. In the 20-40K breakup ice jam flow range, the channel invert and these structures will be well submerged with under ice clearances in the 12-20 ft range (Figures 7a and 9a). It is not expected that the bypass channel bed or vertical control structures will experience significant ice impacts.

f. Downstream Lateral Stability Structure

In the event of a large ice run or an ice jam and release sequence, this embankment will experience severe ice action comparable to existing conditions below the intake dam. The preliminary riprap design by the District is comparable to the results of the ice-factored Ishbash and EM 1110-2-1601 approaches (Table 1).

g. New Dam Crest

It is assumed that the new dam crest will be a horizontal weir with a crest elevation of about 1990.2 ft. In the ramp fish passage alternative, the dam crest was mildly trapezoidal with the invert at 1987 ft and the edges at 1991 ft. It is expected that ice will impact the level-crested dam in a similar way to the trapezoidal crest. In the previous 30 % design of the dam crest, it was anticipated that large ice floes could impact the dam crest over an elevation range of 1985 to 2000 ft. In terms of direct ice impacts to the upstream face of the dam, the design called for an ice loading of 15 kips/ lineal ft. For a thick frazil ice mass sliding horizontally over the top surface of the crest, the ice shear force was estimated to be 2 kips /ft^2. These ice loadings would apply to the revised level-crested dam design. . The 15 kips/ lineal ft loading on the dam face is conservative representing the high end found in the design literature. Although this design loading is applied to vertical concrete structures in rivers subject to heavy ice loadings, a sloped upstream face would be preferable since the ice would tend to ride up over the crest reducing the potential for damage to the concrete. Because the 15 kips/ft ice loading on the dam face is conservative, it would not need to be added to the 2 kips/ft^2 estimate for foe frazil ice masses ice shearing horizontally along the top surface of the dam.

h. Flow Augmentation Weir

A flow augmentation weir parallel to the Yellowstone River right bank immediately upstream of the diversion dam will add flow to the bypass channel fish entrance downstream of the dam during high flow events. The weir will be constructed of roller compacted concrete with compacted backfill along its upstream side.

The crest of the weir will be at the 7000 cfs water surface elevation of about 1991.0 ft based on HEC-RAS. This is only 0.8 ft higher than the dam crest 1990.2 ft shown in the current HEC-RAS model. Figure 6a shows a worst case ice cover profile at 20,000 cfs, the breakup discharge.

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These results indicate that the upstream ice will be sufficiently thick to impact the weir when it passes over. With increasing discharge under ice clearance increases and major ice impacts to the weir would be less likely (Figure 8a). Like the dam, the top surface of the flow augmentation weir will need to withstand horizontal forces due to ice sliding along its crest of 2 kips/ft^2. The upstream face of the weir will be vulnerable to severe ice action from ice runs in the main river. It is questionable whether the compacted backfill along the weir face shown in the preliminary plans will be adequate to withstand this type of ice action. A possibility is to eliminate the backfill and extend the concrete to the upstream face of the weir. This flow augmentation weir is a critical component of the main dam serving as the dam's right embankment.

The concrete wall on the upstream side of weir will experience heavy ice impacts and should be designed for an ice loading of 10 kips/lineal ft. This ice loading is conservative and need not be added to the estimated ice shear force of 2 kips/ft^2 on the top surface of the weir. The riprap on the where the concrete wall ties into the bank will also experience heavy ice action. Here, an average stone in the 1.5 -2.0 ft range and a layer thickness of about 4 ft is suggested.

7. Summary and Conclusions

1. This study analyzed ice-related design aspects of a proposed fish bypass channel at the Intake Diversion Dam on the Yellowstone River in Montana. Past ice related design efforts were reviewed and a HEC-RAS model used to develop a worst case ice formation, breakup and release scenario. HEC-RAS calculated results of depth, water velocity and ice thickness were used gage how ice will interact with the various structures making up the proposed bypass channel and size riprap which is the primary component of the these structures. Exceptions include two concrete weirs, one at the inlet and the other at the outlet of the bypass channel. The design ice forces for the concrete structures were estimated by conventional means as outlined in AASHTO (1998) the Ice Engineering Manual US Army (2008).

2. For the upstream concrete sill under a worst case scenario, an ice force of 5 kips/ft could act horizontally along the front edge. For the surface of the upstream sill and the downstream flow augmentation weir crests, a maximum horizontal ice force of 2 kips/ft^2 due to sliding ice is estimated. The concrete wall along the upstream edge of the flow augmentation weir is expected to experience high ice impacts. Here, an ice design load of 10 kips/ft is recommended.

3. Design of riprap to resist ice damage followed the approach taken in the earlier ice analysis of the riprap ramp (USACE 2011a). First an average riprap D^{50} and D^{30} were calculated by the Isbash and EM 1110-2-1601 methods respectively with velocity and depth inputs from a HEC-RAS simulated 100-year open water event. Following the guidance of the EM 1110-2-1601, the layer thickness was increased by 1.0 ft for heavy ice conditions and the rock size fractions scaled up proportionally. This approach

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produced riprap designs very similar to those provided in the Omaha District preliminary designs (USACE 2012 and Table 1).

4. Several areas where the preliminary riprap designs by the District could be scaled up are the left hand side of the transition from the Yellowstone River into the upstream control structure, and the right bank of the Yellowstone River immediately upstream of the flow augmentation weir. Here the rock size could be increased to 1.5-2.0 ft and the layer thickness to 4.0 ft.

3. References

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USACE (March 2011b) Intake Diversion Dam Modification, Decision Document, Attachment 7, Bypass Channel Concept

USACE January 2012, Lower Yellowstone Irrigation Project-Intake Bypass Channel 30% Design Features.

US Army (1994) EM 1110-2-1601 Chapter 3: “Riprap Protection” U S Army Corps of Engineers, Washington, DC. http://140.194.76.129/publications/eng-manuals/em1110-2-1601/toc.htm

US Army (2008) “Ice Engineering” EM 1110-2-1612, U S Army Corps of Engineers, Washington, DC. http://140.194.76.129/publications/eng-manuals/em1110-1-4014/toc.pdf

Table 1. Hydraulic Conditions for 100-year Open Water Event and Riprap Design for Sturctrual Components

									D_{50} (inches)			D_{100} (inches)			Layer Thickness (inches)		
Structure	HEC-RAS River Station (ft)	Hydraulic Depth (ft)	Average Velocity (ft/s)	Water Surface Slope	Channel Width (ft)	Bed Shear (lb/ft^2)	Bend Radius (ft)	Side Slope (H:V)	Omaha District	Factored Isbash	Factored EM	Omaha District	Factored Isbash	Factored EM	Omaha District	Factored Isbash	Factored EM
Bypass Inlet Weir	15,530	13.0	5.0	0.00026	185	0.21	500	5:1	Type C 12	10	12		20	24	27-40	30	36
Channel Plug	9,586	11.6	6.2	0.00053	230	0.38			20	12	8	30	24	16	45	36	24
Bypass Bends	6300 & 2900	13.0	7.0	0.00032	230	0.26	400-1400	7:1		12	10	16	24	18	24-36	36	28
Vertical Grade Control	9300 & 4800	13.0	6.0	0.0003	230	0.24		5:1	Type C 12	10	12		20	24	27-40	30	36
Bypass Outlet Weir	136	13.0	5.2	0.00075	240	0.61		5:1	Type C 12	10	12		20	24	27-40	30	36
Downstream Lateral Stability Structure	27,575	17.8	6.5	0.00032	800	0.36		3:1	12	10	7	24	20	12	36	30	34
Flow Augmentation Weir	28,203	16.8	6.8	0.0006	150	0.63			Recommend 1.5-2.0-ft riprap where concrete wall ties into bank.								

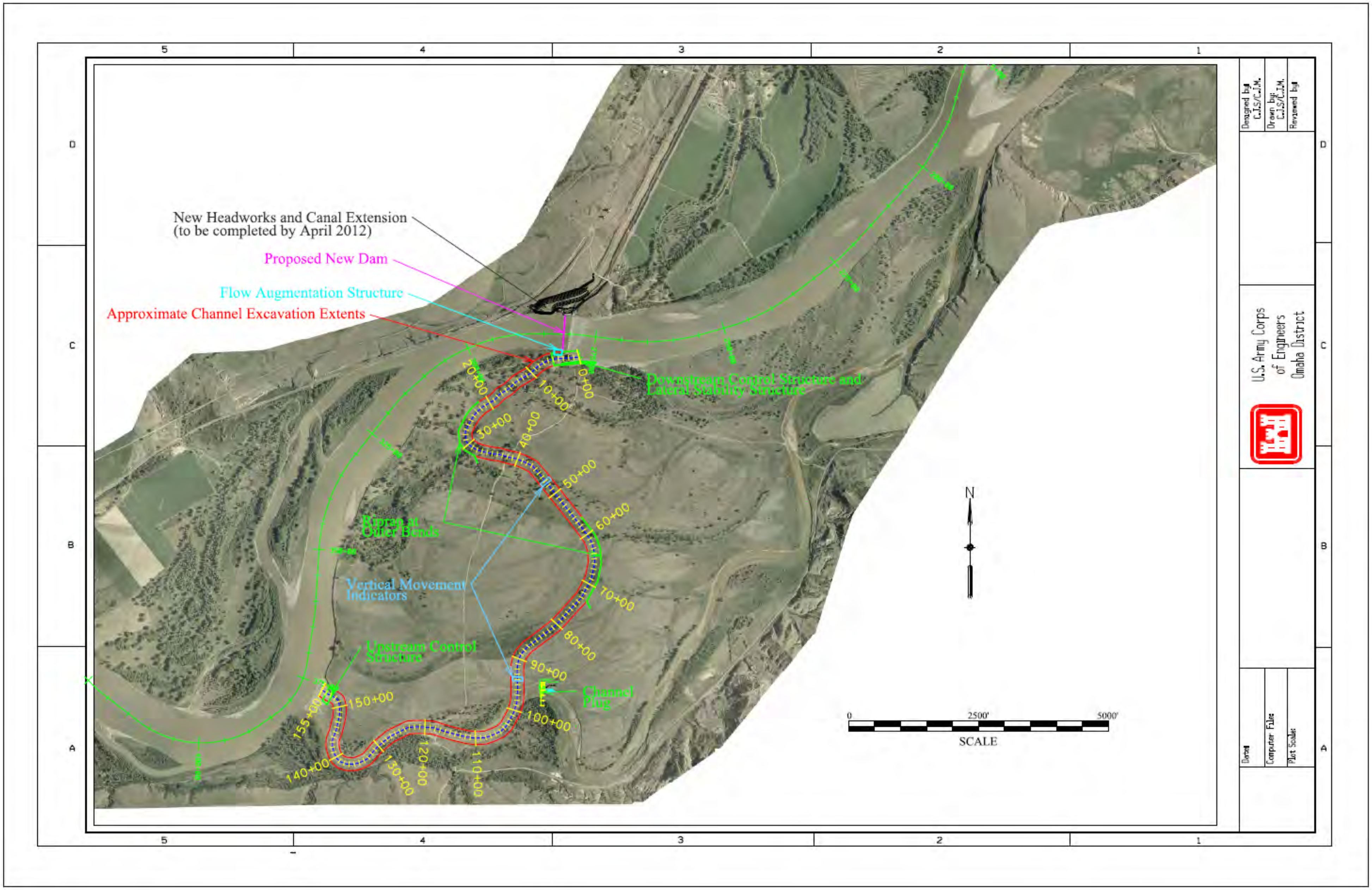

Figure 1. Map of preferred Intake Dam bypass plan as of Jan. 5, 2012 showing structural components.

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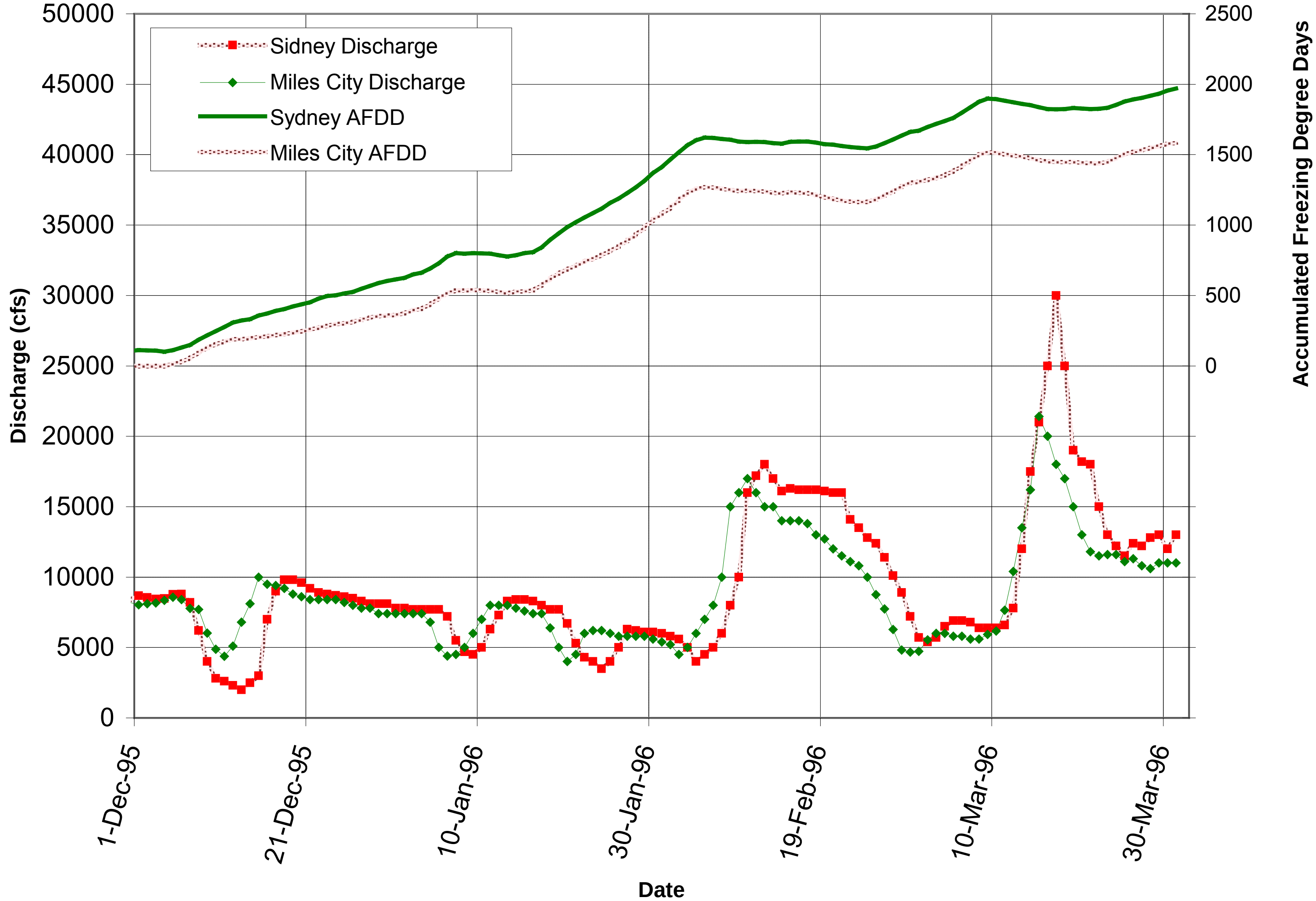

Fig. 2. Yellowstone River discharge and AFDD for the winter of 1996

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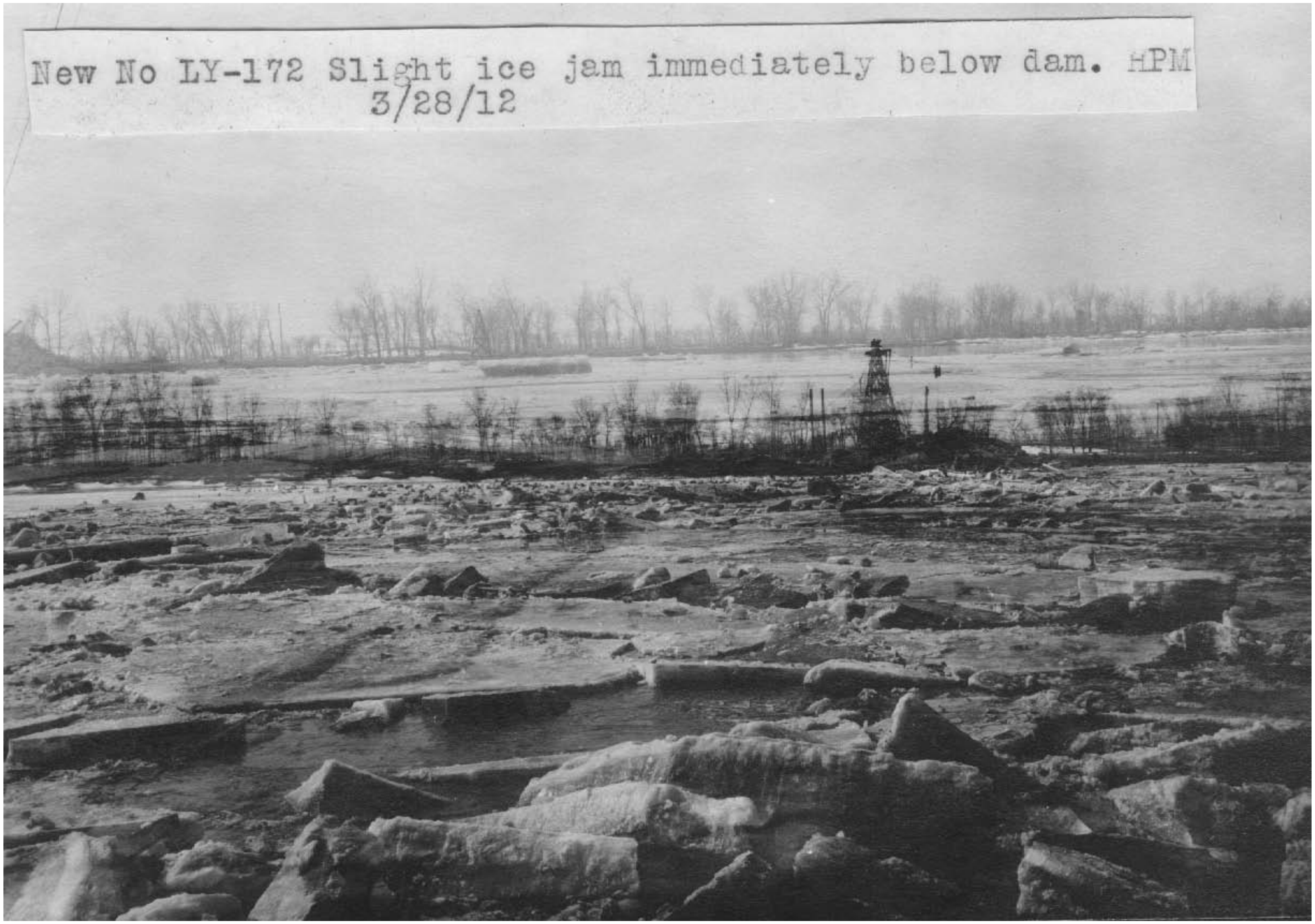

Fig. 3. Ice jam on the Yellowstone River at the Intake in 1912 forcing flow and ice into the right overbank area.

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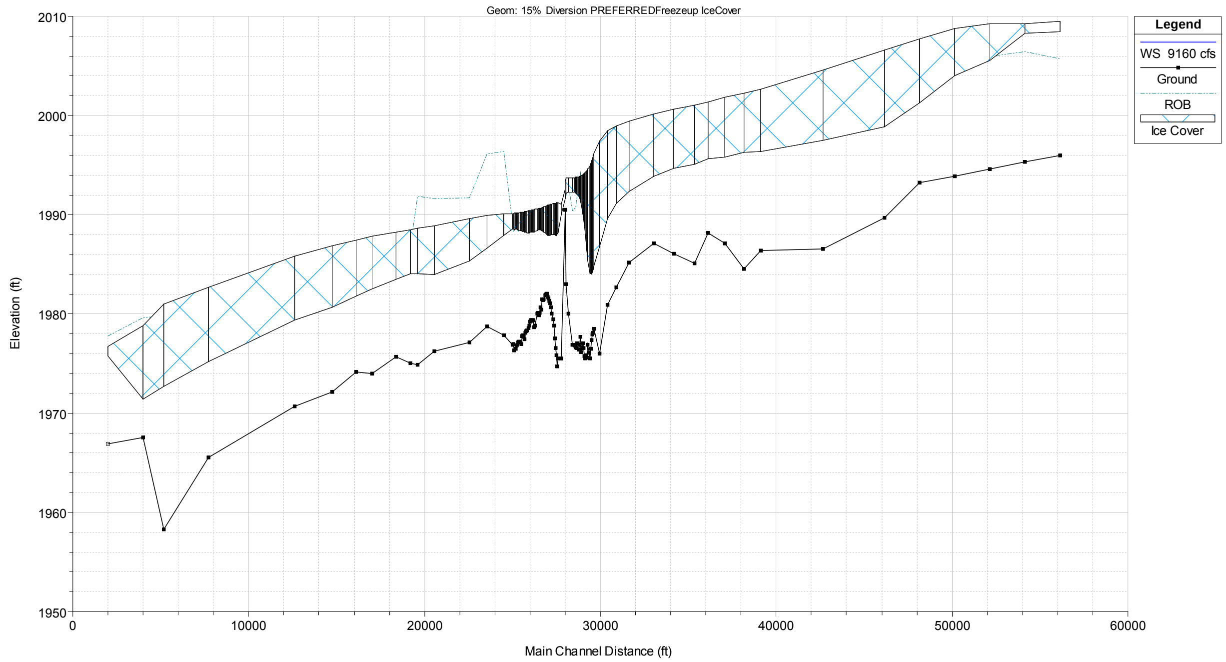

Fig. 4a. Freezeup ice accumulation on main river. Q_{river} = 9160 cfs with 15% passing the bypass channel. n_{ice} = 0.04, porosity = 0.4, V_{eros} = 5 ft/s.

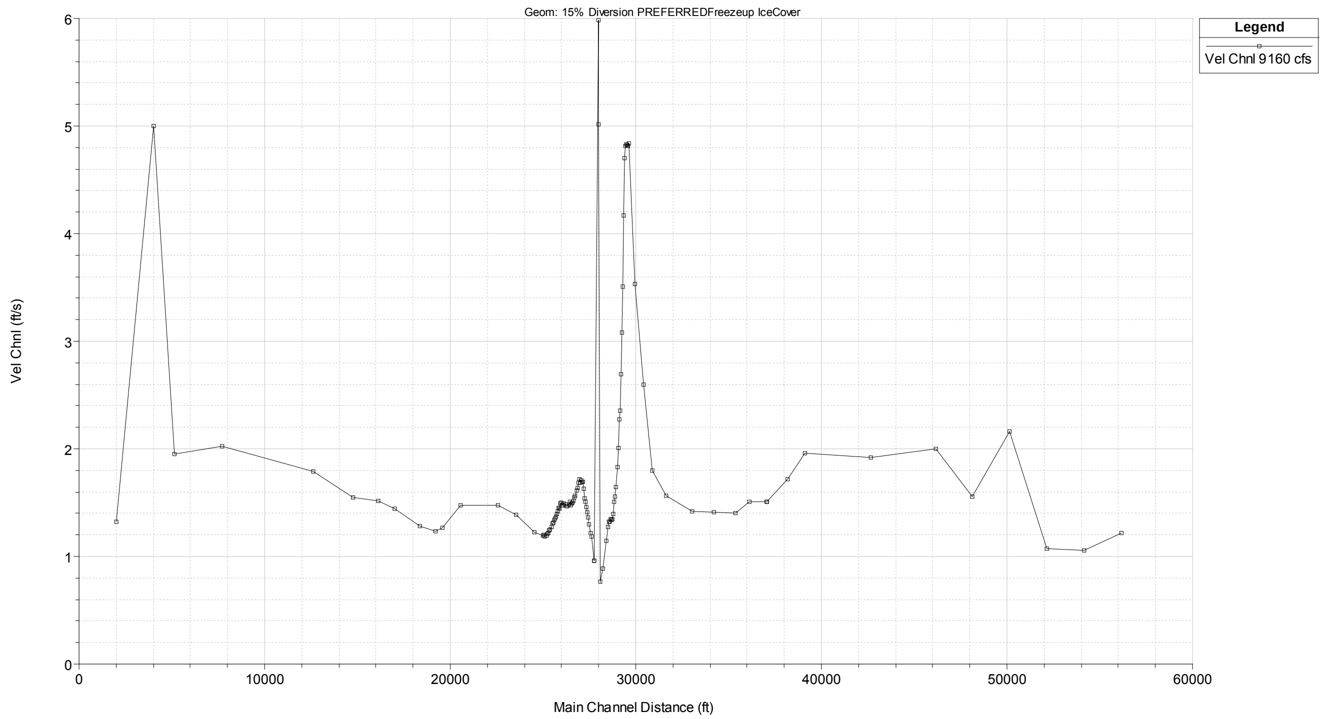

Fig. 4b. Average channel velocity in main river with freezeup ice accumulation. Q_{river} = 9160 cfs with 15% in the bypass channel

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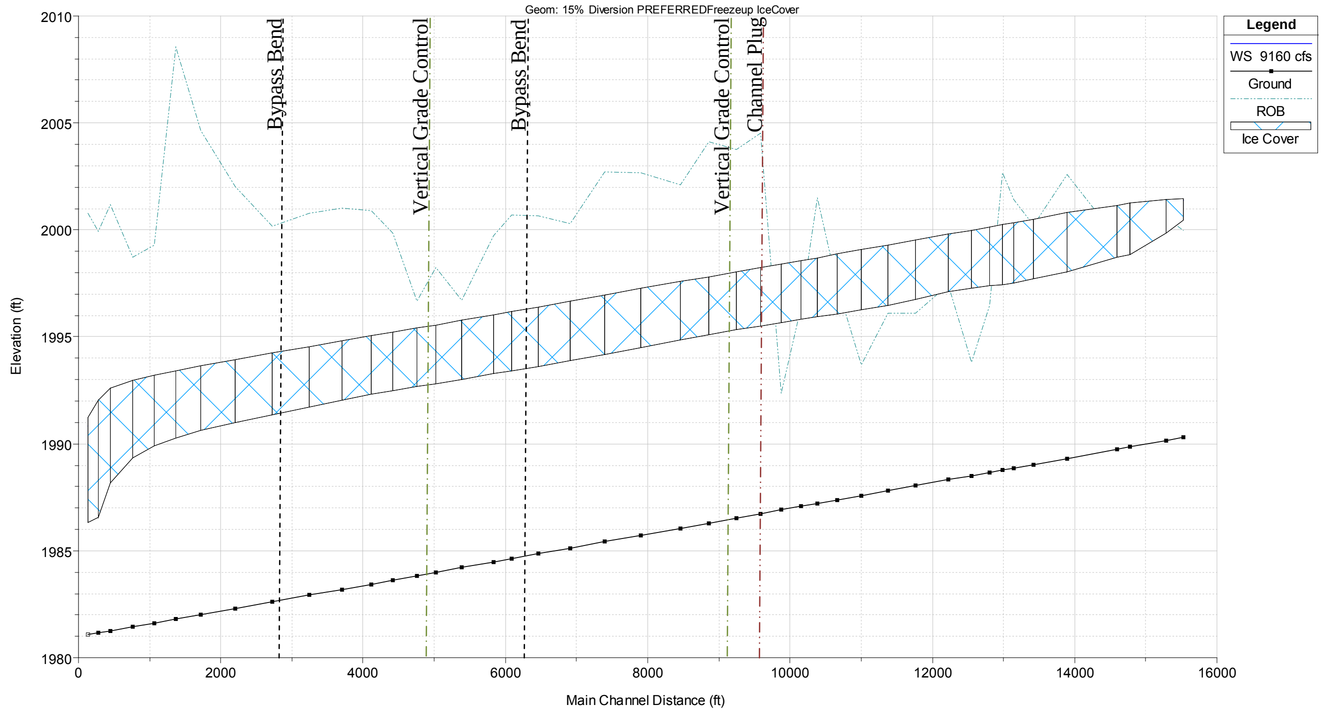

Fig. 5a. Freezeup ice accumulation on bypass channel with 15% diversion. Q_{river} = 9160 cfs with 15% passing the bypass channel.

n_{ice} = 0.04, porosity = 0.4, V_{eros} = 5 ft/s

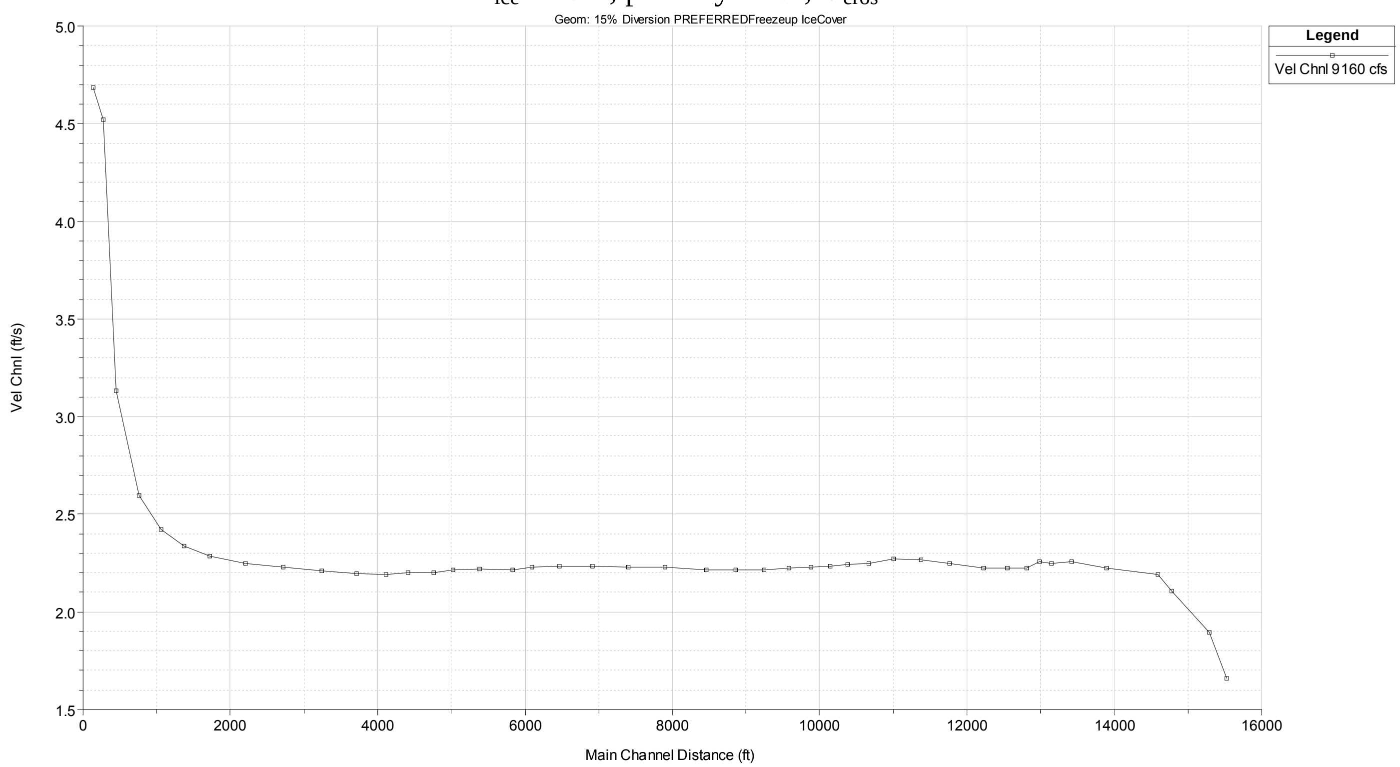

Fig. 5b. Average channel velocity in bypass channel freezeup with ice accumulation. Q_{river} = 9160 cfs with 15% passing the bypass channel.

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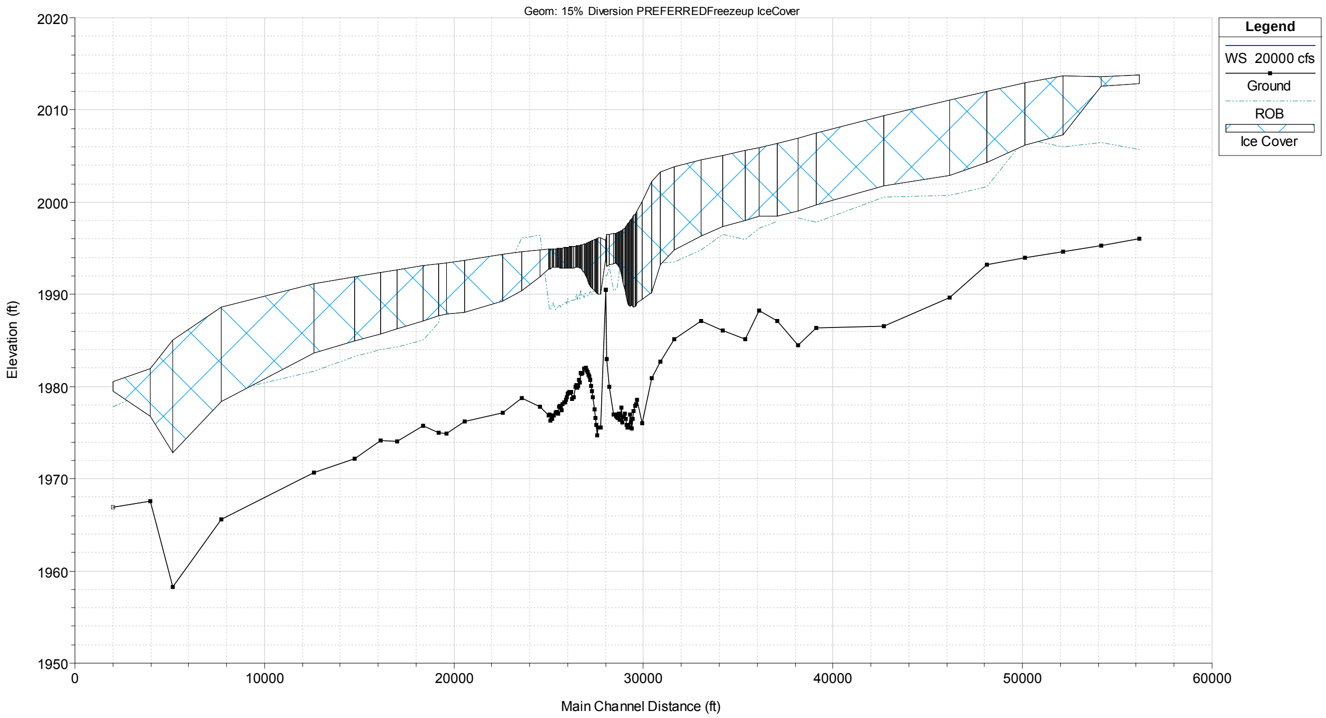

Fig. 6a.. Pre-breakup ice accumulation on main river. Q_{river} = 20,000 cfs with 15% diversion (Q_{bypass}=2600 cfs). n_{ice} = 0.04, porosity = 0.4, V_{eros} = 5 ft/s.

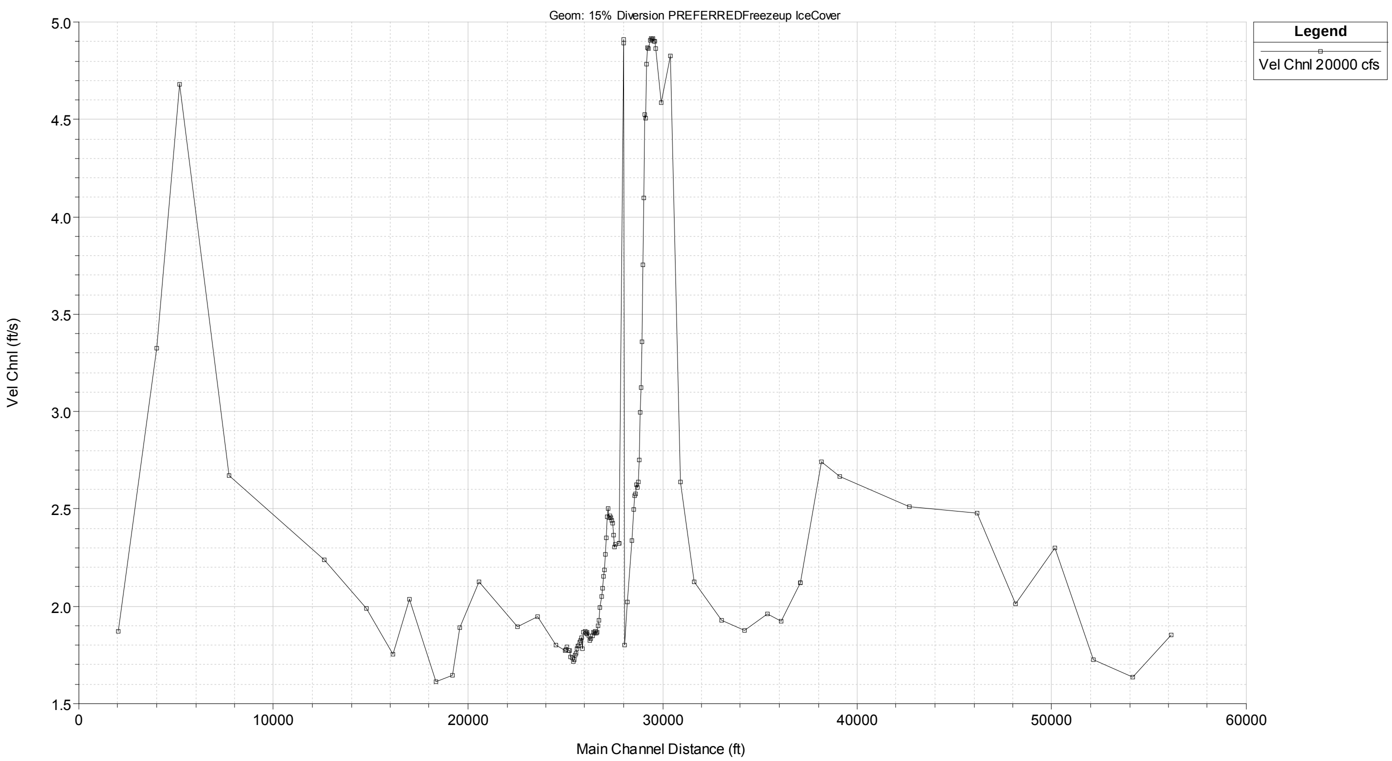

Fig. 6b. Average channel velocity in main river with pre-breakup ice accumulation. Q_{river} = 20,000 cfs with 15% diversion (Q_{bypass}=2600 cfs).

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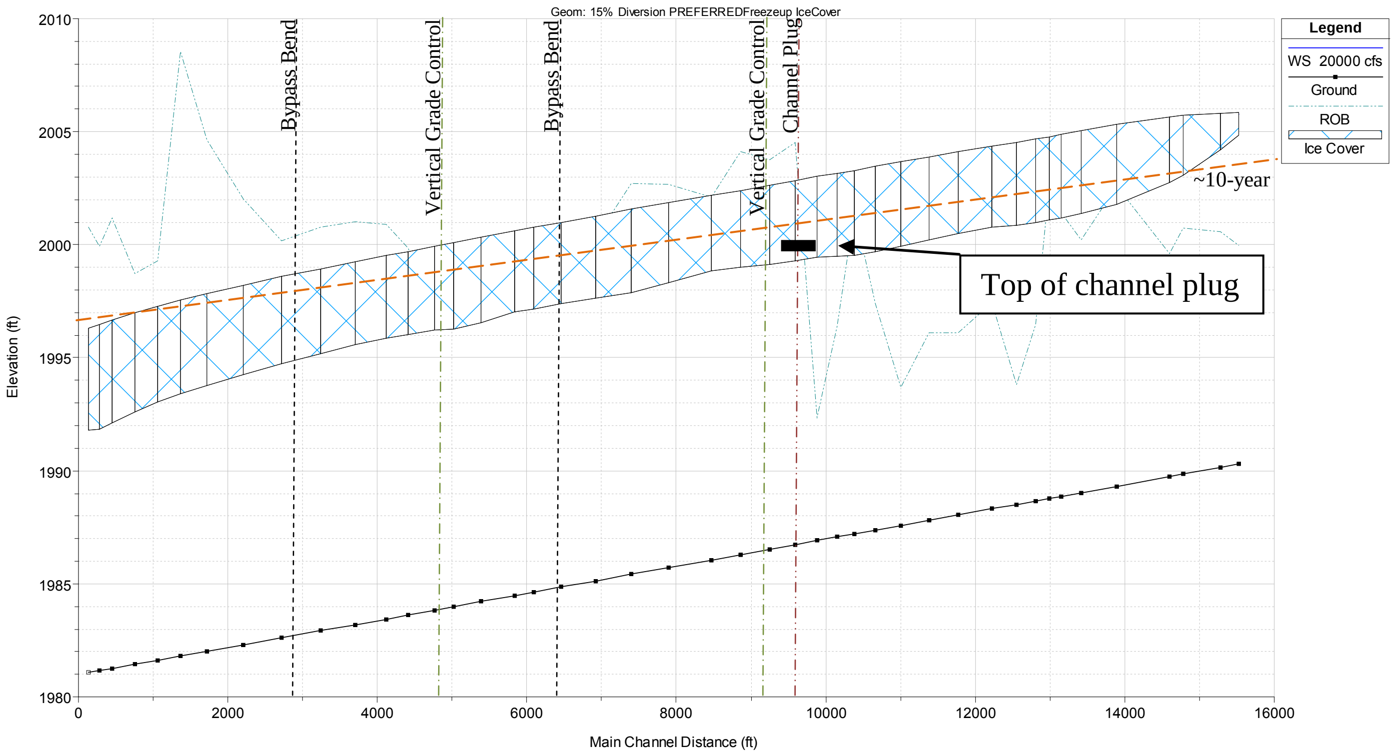

Fig. 7a. Pre-breakup ice accumulation on bypass channel. Q_{river} = 20,000 cfs with 15% diversion (Q_{bypass}=2600 cfs) n_{ice} = 0.04, porosity = 0.4, V_{eros} = 5 ft/s. 10-year elevation indicated by orange dashed line.

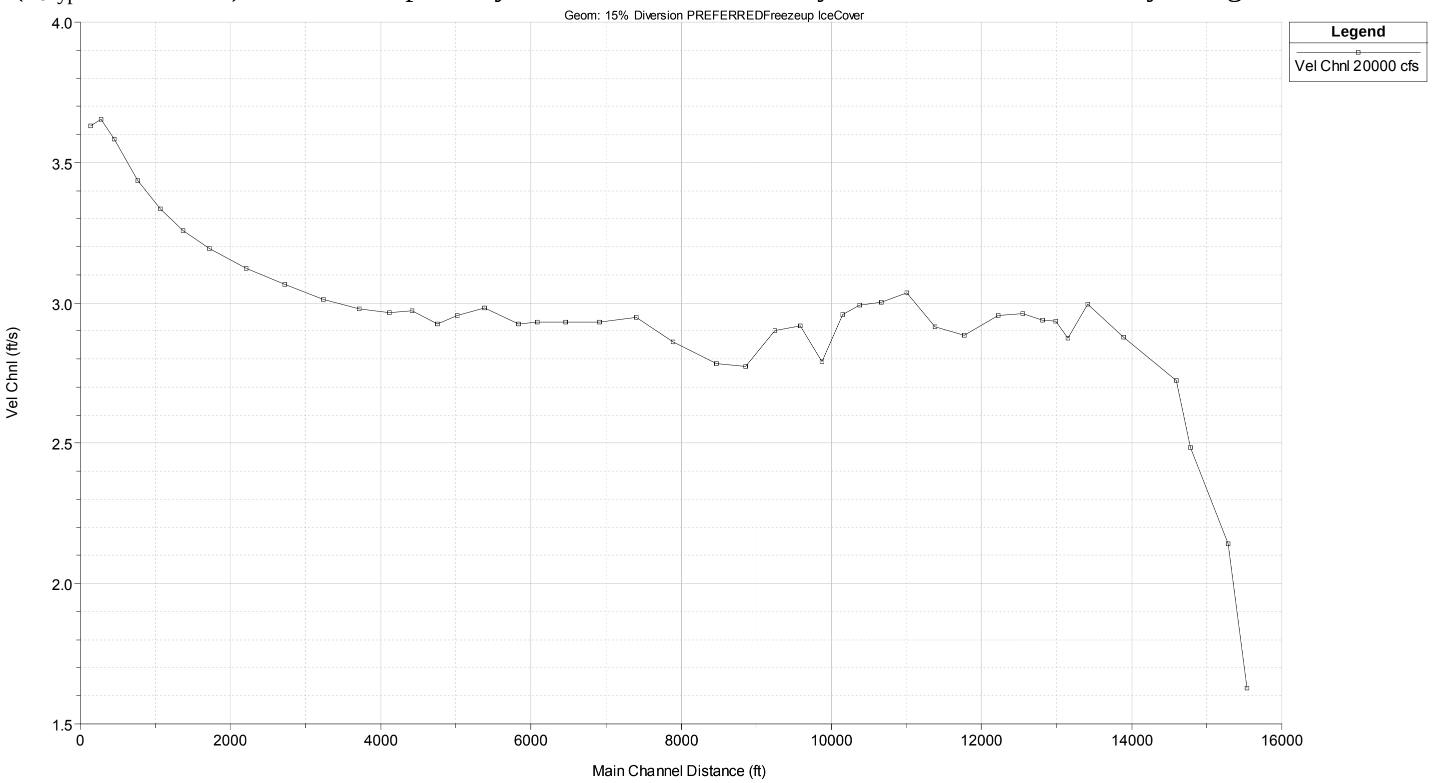

Fig. 7b. Average channel velocity in bypass channel with pre-breakup ice accumulation. Q_{river} = 20,000 cfs with 15% diversion (Q_{bypass}=2600 cfs).

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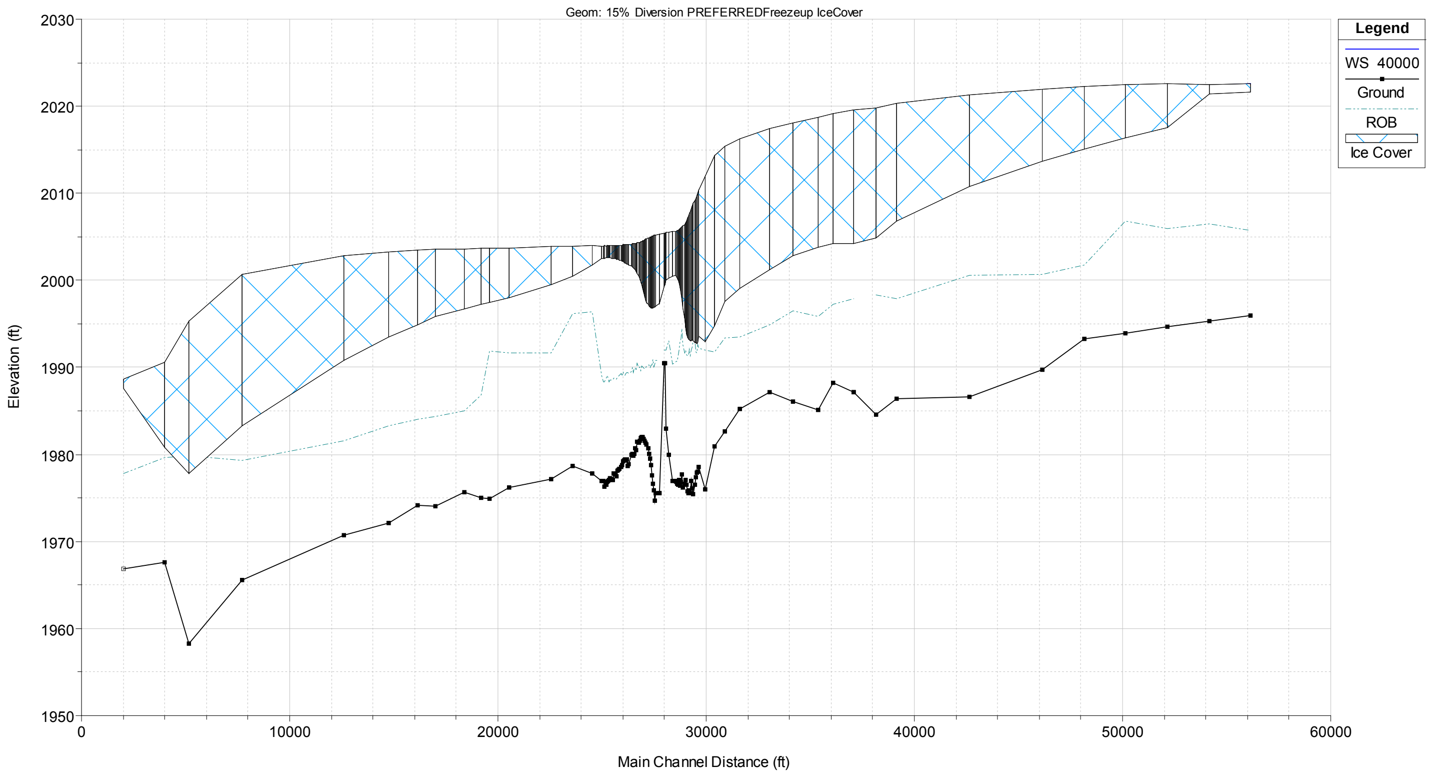

Fig. 8a.. Breakup ice jam on main river. Q_{river} = 40,000 cfs with 15% diversion (Q_{bypass}=5200 cfs). n_{ice} = 0.08, porosity = 0.4, V_{eros} = 5 ft/s.

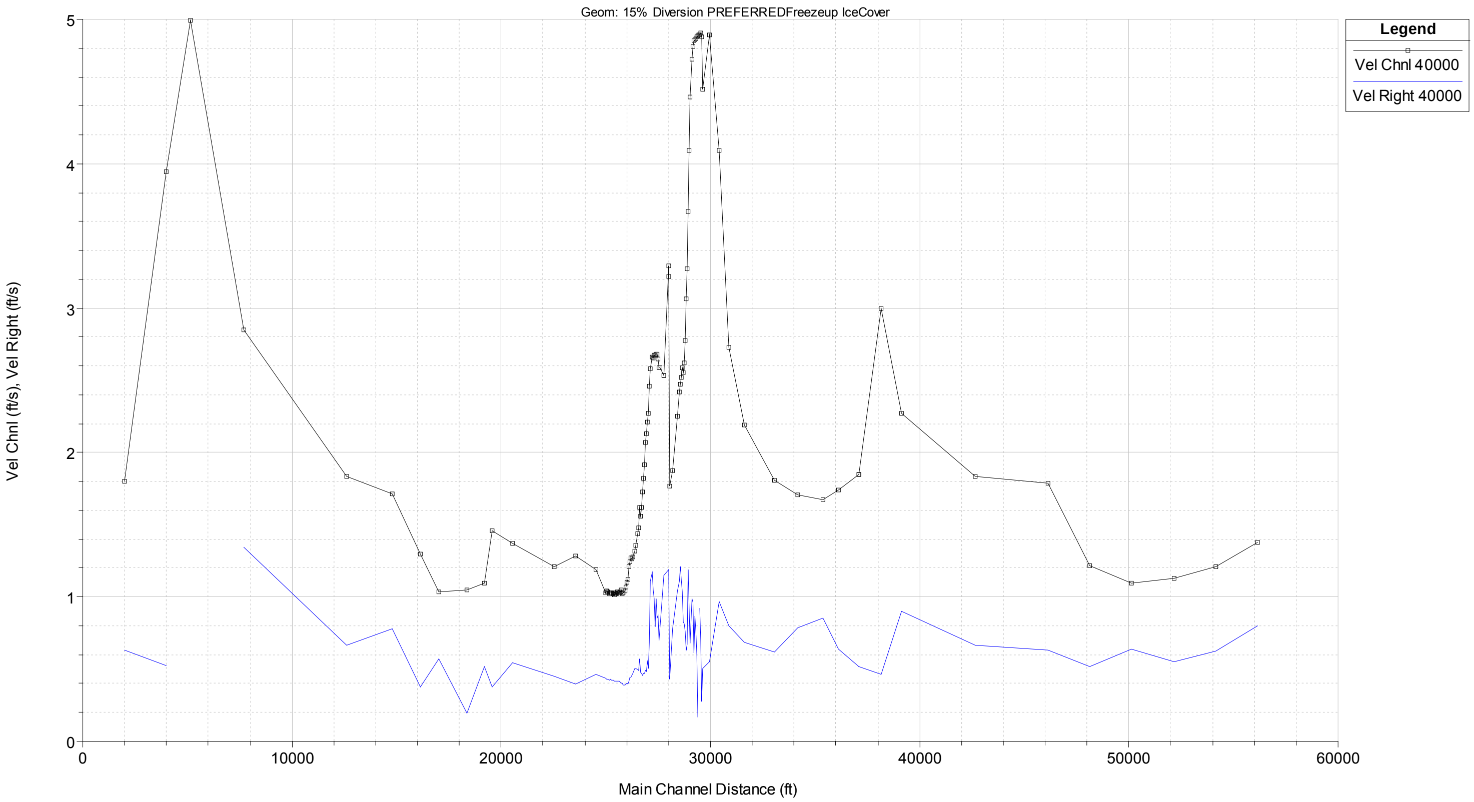

Fig. 8b. Average channel velocity in main river and right overbank with breakup ice jam. Q_{river} = 40,000 cfs with 15% diversion (Q_{bypass}=5200 cfs).

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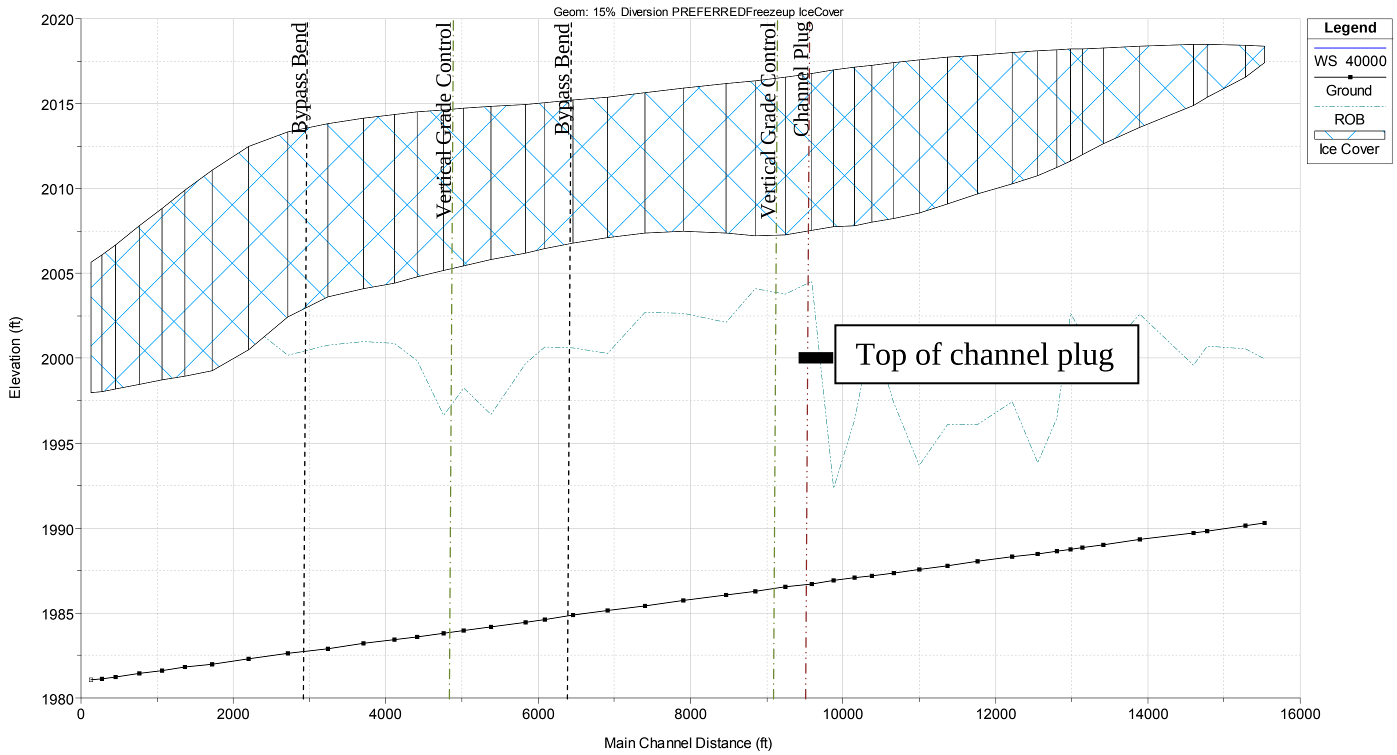

Fig. 9a. Breakup ice jam on bypass channel. Q_{river} = 40,000 cfs with 15% diversion (Q_{bypass}=5200 cfs). n_{ice} = 0.08, porosity = 0.4, V_{eros} = 5 ft/s.

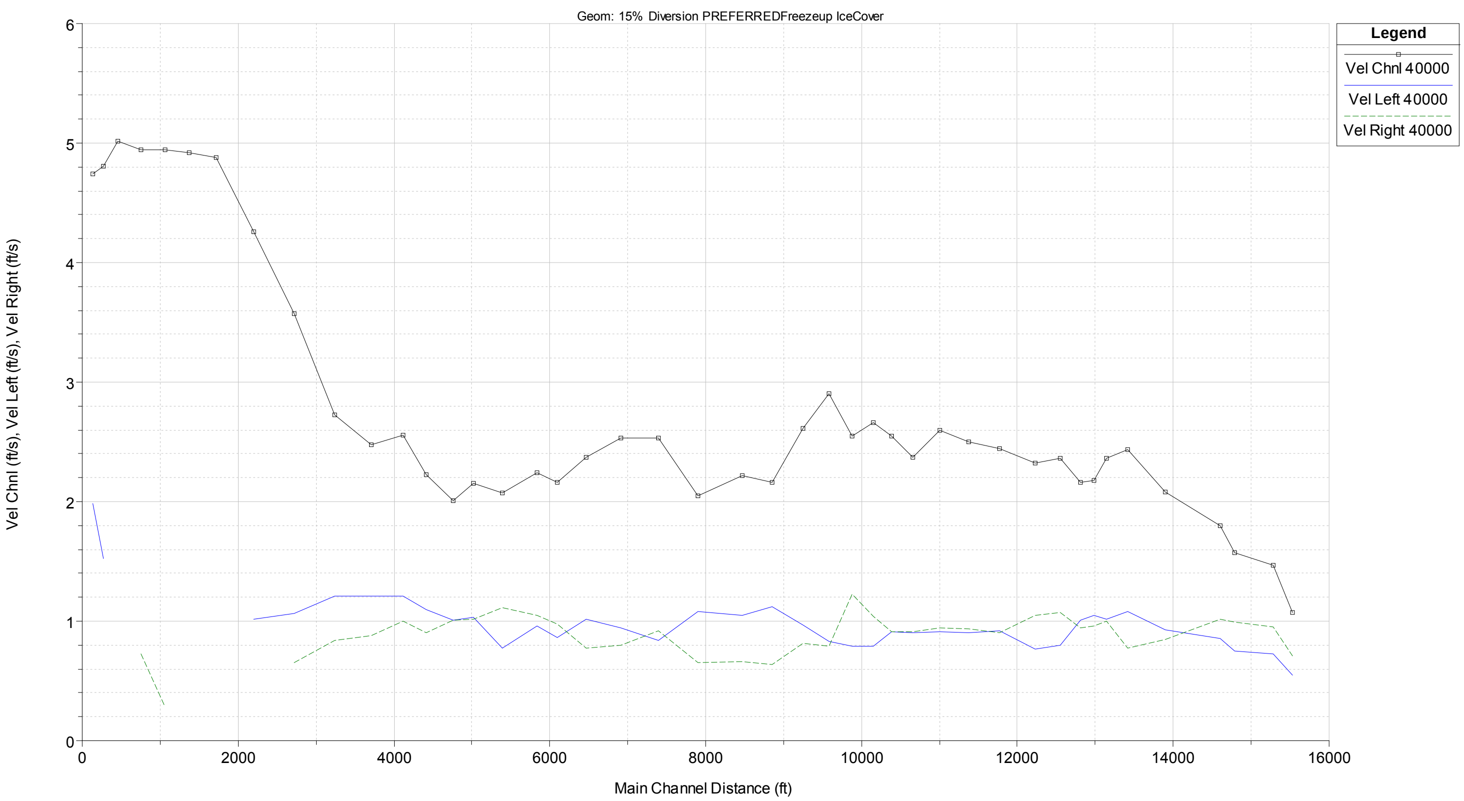

Fig. 9b. Average channel velocity bypass channel and overbanks with breakup ice jam. Q_{river} = 40,000 cfs with 15% diversion (Q_{bypass}=5200 cfs).

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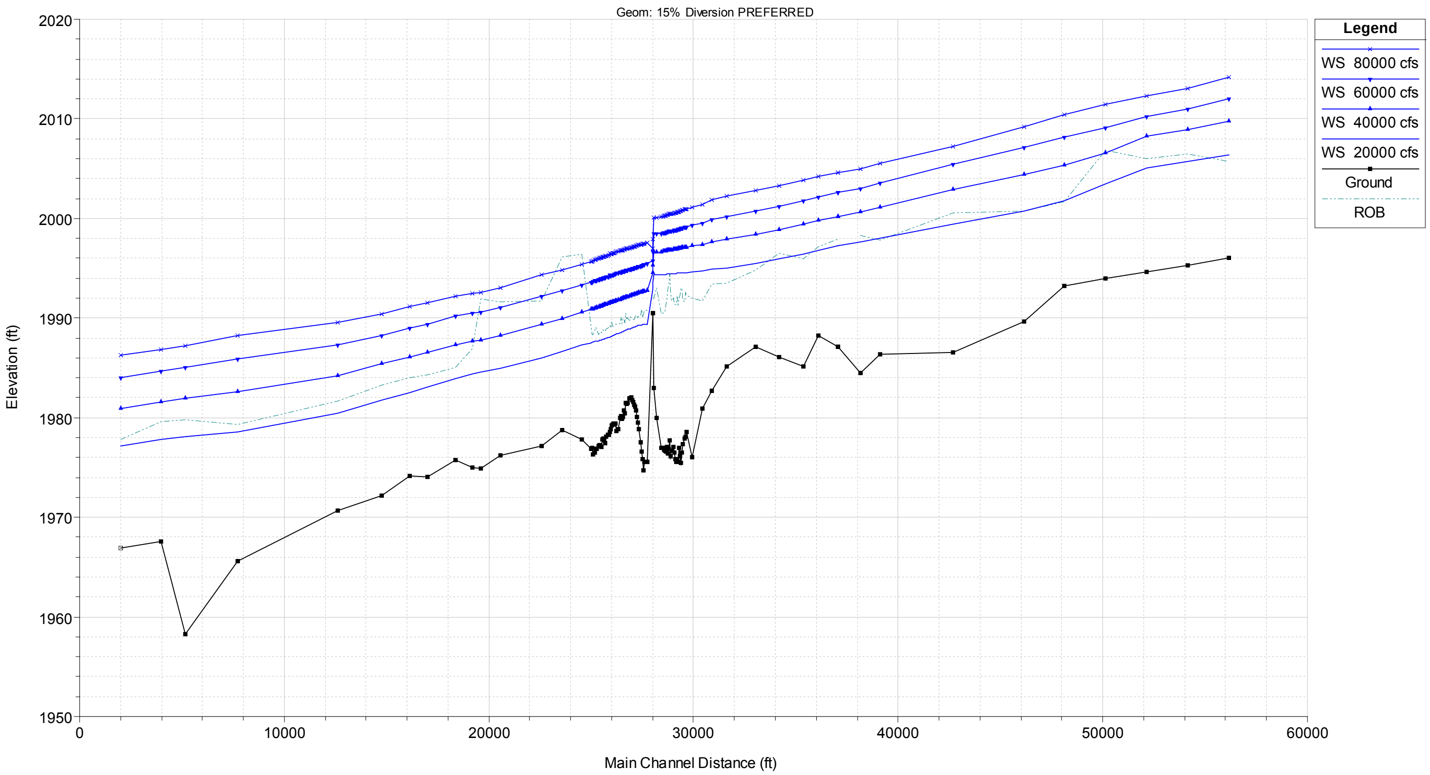

Fig. 10a. Open water surface profiles for river flows of 20, 40, 60 and 80 Kcfs with 15 % diversions.

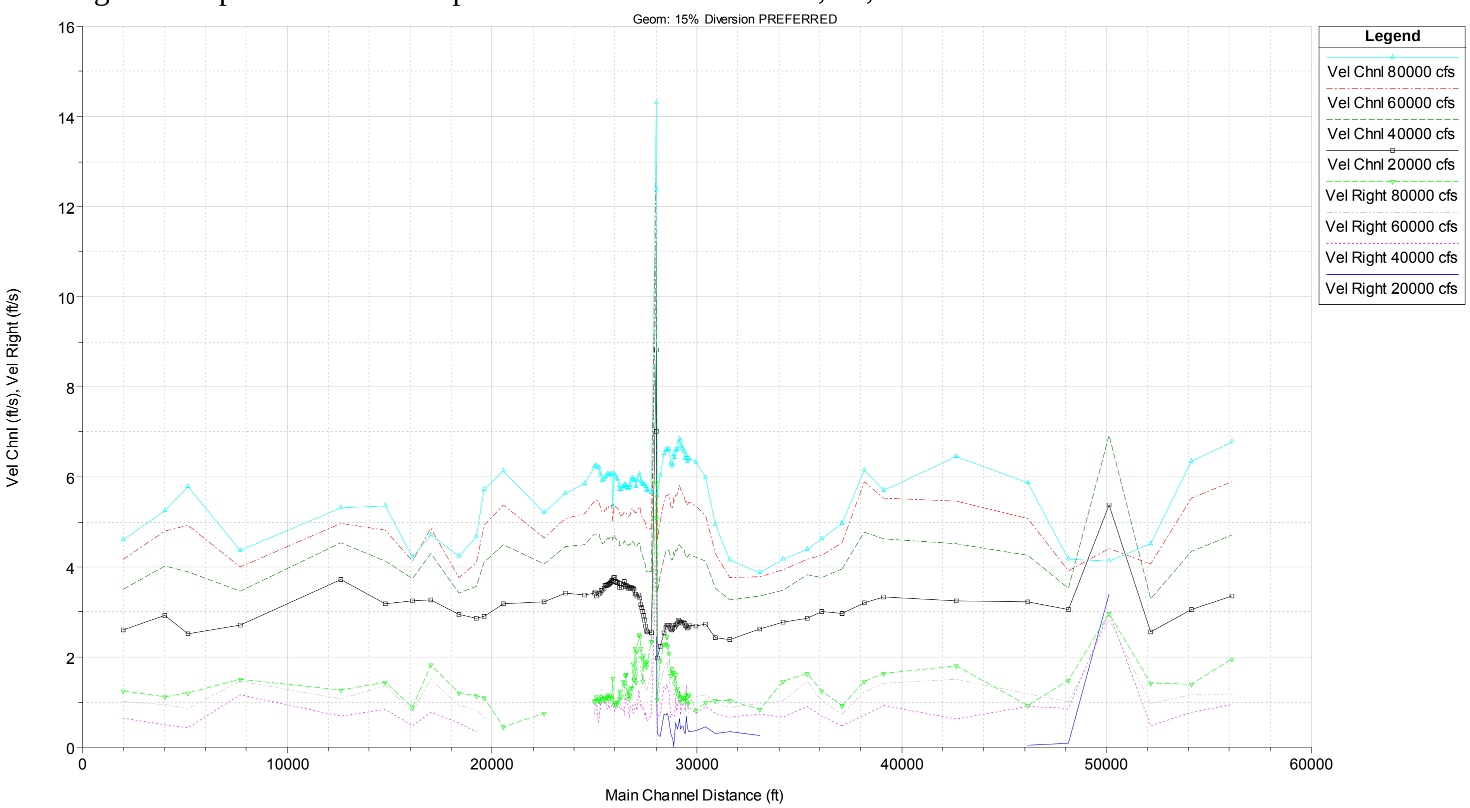

Fig. 10b. Average water velocity in the river channel and right overbank for river flows of 20, 40, 60 and 80 Kcfs with 15 % diversions.

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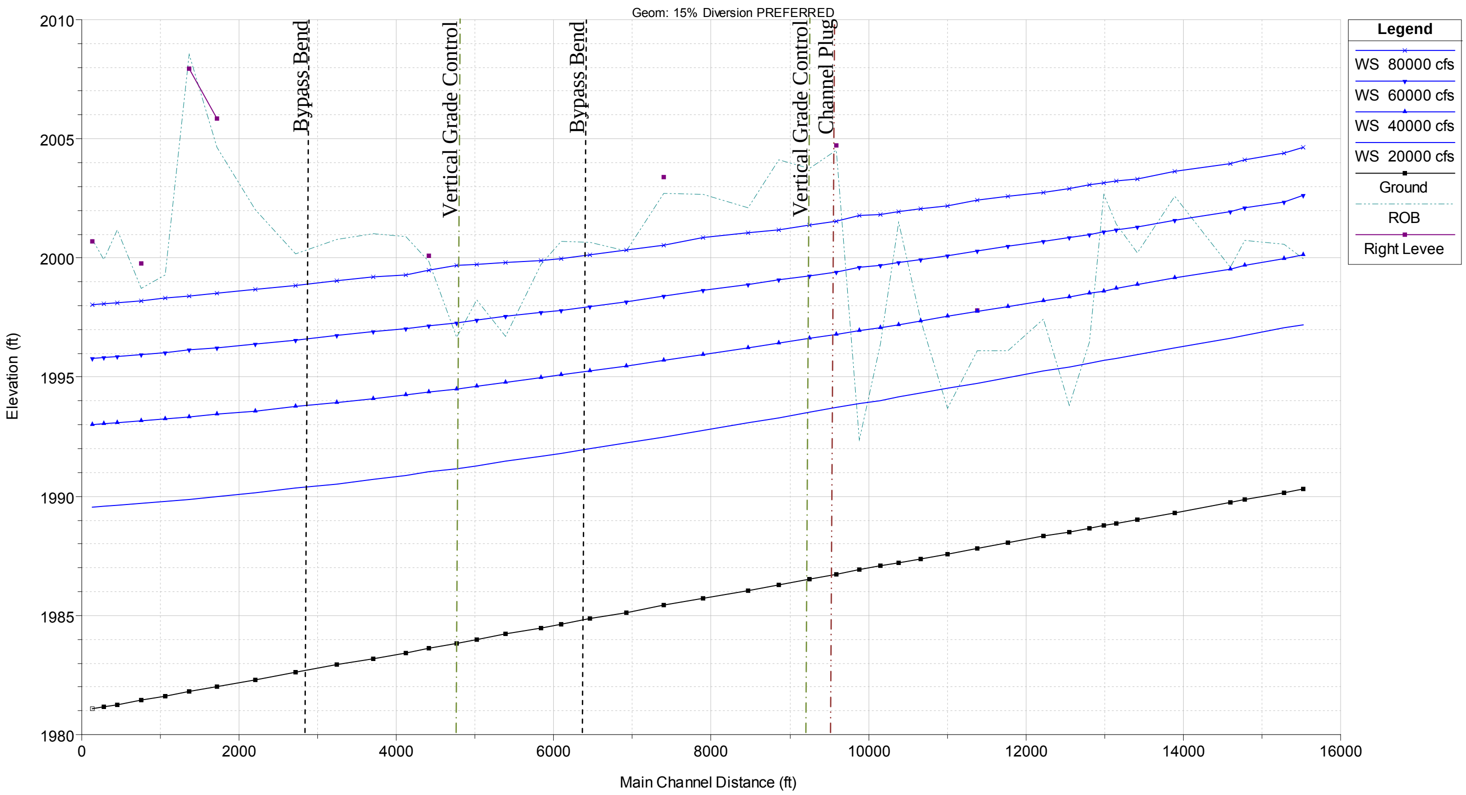

Fig. 111a. Bypass water surface profiles for river flows of 20, 40, 60 and 80 Kcfs with 15 % diversions.

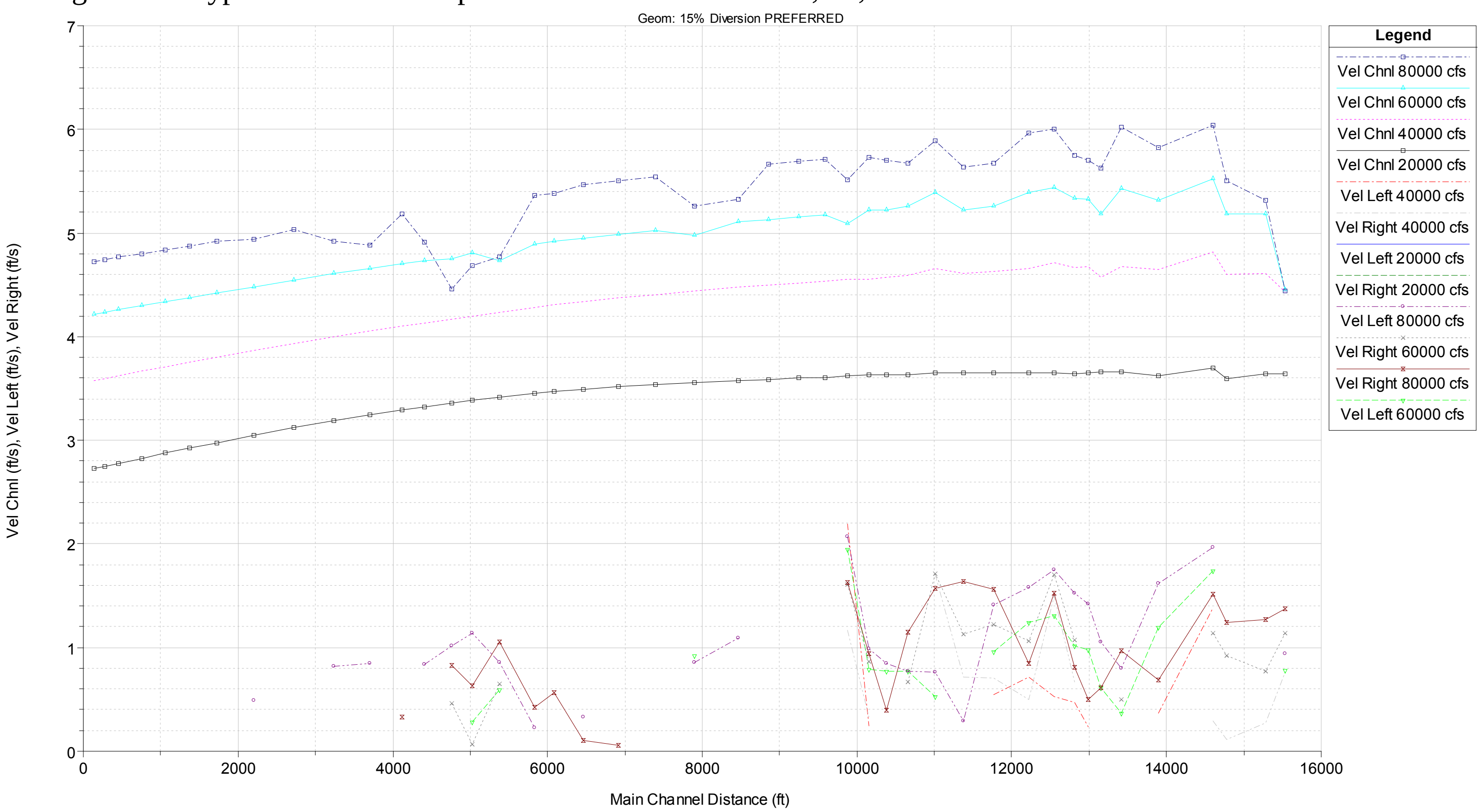

Fig. 11b. Average water velocity in the bypass and overbanks for river flows of 20, 40, 60 and 80 Kcfs with 15 % diversions.

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Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Intake Fish Passage Option Evaluation Summary
May 2012

Attachment 2

Fish Transport Overview

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Fish Transport Overview
May 2012

1. Introduction.

This document provides an overview of the potential application of fish transport at the Intake Diversion Dam Modification project. Prior to the Missouri River Basin Interagency Roundtable (MRBIR) meeting in Denver on Jan 10, 2011 COE leadership expressed interest in the potential for physically relocating fish above the dam as a short term method of preventing near term extirpation of the Yellowstone wild pallid sturgeon. Leadership of both USFWS and Reclamation responded positively to the idea at MRBIR.

In order to provide agency leadership with the information necessary to make an informed decision on whether to initiate trap and haul at Yellowstone project team members were solicited for information. Informal discussions held between the COE PM and some members of the Yellowstone Intake Project Delivery Team and members of the Biological Review Team suggest that there may be some usefulness for the idea as an efficacy test but as a management tool, especially long term, fish transport is of limited to no value. Development of written responses by staff level personnel pertaining to trap and haul was limited as many non-COE team members felt that the response to the question needed to be generated at an agency policy level. To date Montana Fish Wildlife and Parks (MtFWP) is the only agency providing official comments with an official response from USFWS currently under development.

In summary:

- Fish transport options were evaluated as a possible cost effective solution
- Project Delivery Team, Biologic Review Team, and stakeholder agency input was solicited
- Consensus is that fish transport is not recommended as either a short term or long term option
- There may be some value in fish transport as short term ramp effectiveness evaluation, however, any test would need a detailed implementation plan to examine all population issues of both hatchery and wild pallid sturgeon

The specific questions posed by John Hartley, the COE PM, related to catch and haul can be found below with summaries of responses, official and unofficial, received to date.

2. General Response

The Yellowstone Intake project was formulated to achieve multiple goals including providing volitional passage for pallid sturgeon and other aquatic species. Providing this passage would address both Reclamation's responsibilities at Yellowstone as well as COE BiOp requirements associated with the Missouri River, could provide for curtailing the yearly action of dumping rock on the dam and could provide for downstream recreational boat passage and possibly upstream jet boat passage (boat passage was a big issue during the Lewis and Clark bicentennial but has not been a design consideration to date {Mefford Reclamation personal comm.}). Trap and haul fails to meet any of these goals as a short term or long term proposal.

Among fish passage experts, trap and haul carries a stigma worldwide largely because it interrupts and can alter volitional movement of the species. Other notable concerns include the handling process imparts stress to the fish that can result in direct or delayed mortality and every step of the process requires considerable skilled labor. Well designed trap and haul facilities can be a very effective, but always carry a significant risk of problems arising that can result in fish mortality. This is of particular concern for long lived species where spawning adults are few and highly valued.

A more politically oriented concern addressed the issue that if management groups studying either wild populations or hatchery fish have to staff up to conduct a temporary trap and haul program the newly created jobs would then be on the line should the temporary program be discontinued. That could result in temporary programs becoming quasi-permanent.

3. Specific Questions.

Specific questions discussed and are summary of agency response are provided in the following sections.

1) Is transporting pallid sturgeon over Intake Dam a viable long/short term management tool for the species at this location. Why or why not?

General responses to the question indicated that pallids can be caught and physically transported, potentially without disrupting reproductive behavior. Limited numbers of wild population pallids in the lower Yellowstone and the sexual immaturity of the hatchery fish in the river limit the effectiveness of the project.

Specific reasons documenting that transport of pallid over the dam as a management action for this project is not considered viable are as follows:

A) The passage objective requiring that pallid sturgeon can move unimpeded upstream of Intake Dam will not be achieved using Catch and Haul.

B) The population recovery objective showing that the population is on track to becoming self-sustaining will not be achieved using Catch and Haul.

C) Evaluation of the conservation stocking program utilizing a captive broodstock indicates that age-specific survival rates, population abundances and stocking rates are adequate to prevent near term extirpation of the species in the reaches of the Missouri and Yellowstone rivers impacted by this project. Additional management strategies involving Trap and Haul are not considered necessary.

D) Biologically catch and haul would be introducing stress and potentially causing direct and indirect impacts to the pallid life history that may reduce the fitness of the population overall. It was noted that fish behavior in response to stressors can vary widely by individual and may differ by sex, age class, spawning condition and probably many other factors. Experience with physically handling pallids described by scientists at the COE Gavins Point Project is that pallids can be captured and tagged and spawning is still seen within a couple of weeks. In addition pallid sturgeon were relocated hundreds of miles to hatcheries where they have a fairly high success rate of spawning after transport. The COE Gavins Point scientists do not think transporting fish short distances would adversely affect their ability to spawn in the river, but they also admit there are uncertainties associated with the proposed action that will not be understood until this approach is further evaluated. Other studies show where in tagging studies of white and green sturgeon caught on the Sacramento as part of the GCID rock ramp evaluation study, over 50 percent of the sturgeon caught during their migratory period, tagged and then released some distance downstream of the rock ramp failed to move upstream. How much their movement behavior following restocking was due to handling was uncertain (Mefford, Reclamation, personal communication).

E) The logistics of trap and haul may be complicated and costly considering numbers of the native population are low and pallid behavior is not well understood and is far from predictable. Conducting a viable trap and haul operation for wild adult pallid would be challenging and involve considerable risk to the small population of wild adults remaining. For trap and haul operations conducted on other rivers a facility similar to the entrance to a fishway that provides adequate attraction conditions to guide fish into the trap area is built close to the barrier. Attraction requirements for a trap are likely to be very similar to those required for a bypass channel. A trap facility could require infrastructure including a deep structural channel with numerous gates that allow flow to pass through or around the trap, a hoist, trunk loading area, pumps for water supply, lighting, and good access, all protected for floods and ice. Trap facilities are typically manned seven days a week during the spawning season. Therefore, onsite provisions for personnel are probably necessary. Following trapping, fish must be removed from the trap and loaded into a transport truck for transport to restocking sites which must allow good access under all river and weather conditions, lighting and possibly a pumped non-potable water supply. The minimum infrastructure alternative is to hunt for individuals in the river and capture them which works for tagging experiments but probably will not be efficient enough of a process to be used as a spawning relocation management tool.

F) Ecologically, transporting pallid over the dam regularly (without allowing passage of the rest of the biological community) may affect the balance of the aquatic environment above Intake Dam.

2) **Is transporting reproductively ready pallid sturgeon over Intake Dam a worthwhile efficacy test of larval drift and the need to provide fish access above the existing Yellowstone structure? Why or why not?**

Results from efficacy tests utilizing trap and haul need to be considered in light of potential effects the relocation process had on the fish. Information about how the pallids will respond when naturally negotiating an implemented passage option will not be derived from a catch and haul program.
Gavins Point Project scientists have responded that a well designed short-term research effort could provide the information to alleviate many of the uncertainties that exist with passage at Intake including:

- Will pallid stay above the structure
- Will pallid sturgeon spawn successfully above the Intake structure?
- Will eggs be fertilized and develop?
- Will eggs hatch?
- Will larvae begin to drift?
- Is drift distance sufficient to support recruitment?

A summary of several studies evaluating the impact of increased larval drift and provision of pallid passage above Yellowstone intake was provided by MtFWP. The summary conclusion of the studies was that provision of passage and access to the spawning habitat above the structure is likely to be the best and most viable alternative for restoring a self-sustaining pallid population in the Great Plains Management unit. An Independent Review of the Science panel contracted for by Reclamation (Nov 2009, Jan 2010) has already addressed a number of the questions raised by the Gavins scientists and concluded that “additional analysis or research may marginally reduce

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uncertainties regarding the probability of success but is not likely to lead to fundamentally different conclusions, the true test and quantification of project benefits can only be made by project implementation and subsequent monitoring of the response."
Additional considerations related to the difficulty in getting useful data from an efficacy test using trap and haul include:

a) it will likely take significant numbers of spawning surgeon above the dam to produce enough larvae in a large river to be able to identify impacts on juvenile sturgeon recruitment.
b) what is the impact of the existing dam on larval drift and larval mortality due to turbulence, direct impact of larvae on rocks and predation?
c) How is the impact of the backroller at the toe of the dam on drift accessed?

3) Pallid sturgeon has been stocked above intake in the past. What is the status of that population and do/will the activities of that population provide the spawning and larval drift information that would be obtained by a new fish relocation plan thus negating the need for an additional phase of relocation.

The pallid sturgeon were stocked in the Yellowstone River between 2004-2010. These fish would not be able to provide information in regards to spawning and larval drift until they have reached sexual maturity (2016-2020)

The following information on pallid stocking comes from an email from George Jordan of the USFWS:

50 yearling pallid sturgeon equipped with radio transmitters were released near Forsyth in 2004. Some remained in this reach while others dropped down to below Intake.

The most relevant fish in relation to the discussion about possible reproductive condition fish above Intake are: the 2006 and 2007 year class fish.

-10,800 fingerlings were stocked in 2007 at two locations; Fallon and Forsyth.

-983 yearlings (2006 year class fish) were stocked in 2007 at Fallon and Forsyth.

-16,282 fingerlings were stocked in 2008 at three locations; Fallon, Miles City and Forsyth.

-2797 yearlings (2007 year class fish) were stocked in 2008 at Fallon and Forsyth.

Monitoring has established that many of the pallid sturgeon stocked in the Yellowstone move downstream into the Missouri River below the Yellowstone River confluence (where they are recaptured as juveniles). Stocked pallid sturgeons have been found to utilize both the lower Yellowstone and Missouri Rivers to some degree, regardless of their stocking location in.

Notes from Feb. 10, 2011 conversation between Tim Welker (Fish Biologist COE Gavins Point Project) and Ryan Wilson, (field biologist lead for sampling Segment 4 of the Corps' Pallid Sturgeon Population Assessment Program, USFWS-Bismarck) regarding Yellowstone River stockings and pallid captures in (Yellowstone River confluence to headwaters of Lake Sakakawea)

77% of all recaptures below the confluence(Segment 4) are from YR river stocking sites. The recapture rates in Segment 4 for each stocking site are as follows: Intake=45%; Sidney=19.4%; Fallon=15%; Forsythe=13%; Fairview=6%; Big Sky=<1%; Cartersville=<1% (a few telemetered fish without PIT tags were captured, so these were assumed to be fish stocked at Cartersville).
In 2008, a number (100) of telemetered fish were stocked in the Bighorn River near Billings. These fish have been recaptured in Segment 4.

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The last MTFWP survey report above Intake that Ryan had in his files was for 2008. They sampled above Intake at RM 97 and RM 92. Only 7 pallids were collected and they were fish stocked in 2007. Most of MTFWP sampling is conducted downstream of Intake. The thinking is that most of these juvenile fish eventually move downstream to the lower YR or the Missouri River below the confluence. It is not known if these fish try to move back above Intake, but, of course, there is no way for these fish to access this area once they pass downstream of the dam. The general pattern for adult pallid sturgeon is that they move out of the YR sometime in late June or early July into the MR below the confluence (and headwaters of Lake Sak.; Segment 4) with some fish moving into the MR above the confluence (Dave Fuller MTFWP- Ft. Peck has 6+ years of telemetry data on adult PDSG). MTFWP sampling in the summer focuses on the Sidney and Fairview areas. MTFWP doesn't catch many adults during this time, mainly recently stocked fish. It is not entirely clear if the younger juvenile fish exhibit this same pattern, or they stay in the lower YR or in the MR below the confluence (which is where many of these fish are captured).

4) Is physical relocation a potential Adaptive Management (AM) tool that could be used to relocate fish congregating at whatever passage structure we come up with but which are refusing to pass?

No. If we are uncertain about the success of our action, then we should move forward using AM as our management strategy to re-address the passage issue at Intake (assess, design, implement, monitor, evaluate and adjust) to ensure we achieve our project objectives. If we are confident in our action's ability to achieve the desired outcomes, then we should implement our actions within an AM framework until objectives are achieved. The accurate evaluation of passage success criteria (see below) would be adversely impacted by utilization of trap and haul as an AM tool since the long term criteria require documentation for naturally produced juvenile pallids in the lower Yellowstone and trap and haul would introduce a factor of uncertainty in that assessment.

Success Criteria USFWS October 23, 2009

Within 4 years after completion of the fish passage and entrainment projects at Intake Dam:

- Document that adult and stocked juvenile pallid sturgeon can move unimpeded upstream of Intake Dam.
- Document that adult and stocked juvenile pallid sturgeon can pass downstream of Intake Dam without being entrained into the irrigation canal.

Within 8 years after completion of the fish passage and entrainment projects at Intake Dam:

- Document the presence of naturally produced juvenile pallid sturgeon in the lower Yellowstone and Missouri rivers between Fort Peck Dam and Lake Sakakawea.
- Document that pallid sturgeon ($\geq$ 40 mm total length) can pass downstream of Intake Dam without being entrained.
- Indicate that naturally produced juvenile pallid sturgeon survival rates can be estimated and modeled to show that the population is on track to becoming self-sustaining.

5) If we get yes answers to either of the first 2 questions we need an implementation plan.

Currently, the Integrated Science Program has funded research for 2011 to track reproductively-ready pallid sturgeon on the Yellowstone River. It is anticipated that this research will provide insight and rigor to our lower Missouri River research activities

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regarding pallid spawning, egg success, and larval drift. This effort could easily be modified to incorporate transport of reproductively-ready pallid sturgeon above Intake. We could expand our current research scope (with input from the upper basin entities) to directly address specific uncertainties that are important to passage success at Intake Dam.

Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Intake Fish Passage Option Evaluation Summary
May 2012

Attachment 3

Performance Evaluation of Ramp Alternatives

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1 Introduction

This document is a collection of figures detailing hydraulic modeling results from evaluation of various ramp options investigated for the fish passage efforts at Intake Dam.

1.1 Passage Criteria

A set of criteria was established through consultation with the Biological Review Team in order to identify target velocities and depths suitable for Pallid Sturgeon passage. A list of key targets was established.

- **Velocity:** Allow some degree of passage under variable flow conditions to be experienced during spring and summer
  - **Fishery Flow Criteria**: Design target flow 6 ft/s max, 3-4 ft/sec max for longer ramp lengths. Pallid burst speed 7.5-8 ft/s max, best swimmers. Sustained swim speed 6 ft/sec for short distance
  - **BRT** Optimum adult velocity less than 4 ft/sec, juvenile 1-2 ft/sec
- **Depth:** Maintain minimum depth throughout passage season. Optimum depth >1 m, 0.5 m minimum
- **Attractive Flow:** Pallid sturgeon orient into the dominant current so acceptable options must provide sufficient attractive flow for locating and navigating the structure
- **Turbulence:** Minimize high turbulence \ hydraulic shear zones, and avoid excessive steep vertical drops which are not negotiable by pallid sturgeon
- **Connectivity**: Maintain a passage corridor from upstream to downstream

For sake of simplicity, in addition to depth and velocity one foot contours, hydraulic model results are presented using passage corridor figures. These figures display model results where depth and velocity results meet criteria as prescribed by the BRT.

Ideal conditions for passage are represented by velocities two to four feet per second with depths greater than one meter; however passage may be feasible at depths of half a meter. Passage likelihood is considered to be marginal at velocities greater than 4 feet per second with an upper threshold of 6 feet per second for sustained swimming speeds of the target species.

1.2 Project Features.

Primary project features are replacing the existing rock and timber fill Intake Diversion Dam with a concrete weir and rock ramp. This would maintain the existing surface elevation of the river upstream of the weir for diversion into the main canal, while improving fish passage and contributing to ecosystem restoration. A new main canal headworks structure with screens to minimize canal entrainment was awarded for construction in fall 2010. The rock ramp would be constructed downstream of the new weir by placing large angular rock sized for stability. The large rock would be backfilled with a granular choke stone to provide suitable substrate for passage.

1.3 Ramp Modeling.

Results presented in this report are derived from a 2D numerical model of the project. Computations were performed using the ADaptive Hydraulics (ADH) modeling utility. A rock ramp physical model was also constructed at the

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Reclamation Technical Services Center in Denver, Colorado. Results from the physical model generally confirmed ADH model results with the exception of near crest flow conditions.

The ADH model is a finite element code capable of modeling two-dimensional, depth-averaged, shallow water equations. This tool is developed and maintained at the U.S. Army Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory (CHL) and the work is funded primarily through the System Wide Water Resources Program (SWWRP).

The shallow water equations are applicable for situations in which the water depth is much less than the wavelength and therefore useful for estuarine and riverine modeling as well as other applications in which vertical velocity effects are not important aspects of the problem being solved.

Flow rates selected for analysis were 7,000, 15,000, 30,000, and 40,000 cfs. These flows cover the range of flows that would likely be encountered during pallid sturgeon migration. Design and analysis focused on the 15,000 and 30,000 cfs flows as the most important during the pallid migration period.

50% Exceedance by Month
April-8,470
May-14,800
June-30,700
July-17,100
August-7,080
September-6,660 cfs

Figure 1 - 50% Exceedance Flows by Month, April thru September

2 Ramp Rock Stability.

Evaluation of suitable rock size and layer thickness to provide ramp stability was performed for both hydraulic and ice forces. Analysis of ice forces on both the headworks and ramp structure was performed by the USACE Cold Regions Research and Experimentation Lab (CRREL) and is summarized in a separate document.

2.1 Hydraulic Stability.

For hydraulic stability, current USACE criteria was consulted with respect to determining the minimum rock size and layer thickness using computed flow depths and velocities. The concept design efforts utilized the Ishbash equation to determine minimum rock size throughout the ramp. Rock sizes were based on a 20-year peak flow rate and a layer thickness based on 1.5 times D_{100}. Using ADH model results, the maximum rock size varied from a D_{100} of 30 to 48 inches across the ramp.

Current USACE criteria were consulted with respect to determining the minimum rock size and layer thickness using computed flow depths and velocities. EM 1110-2-1601 provides guidance for a layer thickness of 1 times D_{100} or 1.5 times D_{50}. For greater flow turbulent areas, the HDC Sheet 712-1 provides rock sizing guidance based on flow velocity with a minimum layer thickness of 1.5 times D_{100} or 2 times D_{50}. The ETL 1110-2-120 also recommends a layer thickness of 1.5 times D_{100} for turbulent flow areas. Final determination of rock size and layer thickness for hydraulic stability will be required using computation results.

Final determination of rock size and layer thickness for hydraulic stability will be required using computation results.

- Rock size based on ADH model computed velocities and the Ishbash equation, 20-year event peak Yellowstone River flow
- Ramp D_{100} varies from 30 to 48 inches
- Rock layer thickness of 1.5 times D_{100}

2.2 Ice Forces

CRREL has provided evaluations of ice impacts on the structure at the initial concept design stage for the ramp in 2008 and 2009 as well as additional analysis provided early 2011.

While initial efforts suggested that the maximum rock size and the layer thickness could be adjusted across the ramp to address differences in hydraulic stress in different parts of the design, which would result in reduced rock cost, the CRREL ice analysis (see attachment 1) showed that much of the ice impact would be on the lower flow sides of the ramps. While there may still be possibility for some optimization near the toe of the ramp a more uniform rock gradation and layer thickness appears likely.

Previous ice analysis studies conducted in 2008 and 2009 indicated that an average diameter of 6 feet or more is necessary using guidance based on the maximum ice thickness. In the most recent analysis, the ice report indicates that the required rock size is smaller than that dictated by hydraulic analysis.

The CRREL evaluation included a review of the ice impact history at the site. Several times over the life of the structure ice has significantly damaged to nearly destroyed the crest structure. While crest hydraulic conditions are significantly different than the ramp, severe ice events should be expected to move ramp rock. Given the low construction tolerances for the uppermost ramp slope and the potential for ice induced rock movement, it is likely that ramp performance could be systematically degraded over time, especially near the crest. To mitigate this possibility to some extent it has been suggested that the uppermost portion of the crest be grouted. Costs for construction and maintenance of a grouted section have not been developed.

The ice evaluation results indicated that analysis of ice impacts on a full width, aerially extensive rock ramp has not been done before and that there is a significant amount of uncertainty in the results. Detailed analysis during final design more result in the recommendation for larger rock to provide ice stability. In addition, there is significant risk that the ramp could suffer repetitive damage during annual ice out events with a higher risk for greater damage during extreme ice events.

- Ice impact likely higher to lower ramp portion and side slopes
- Preliminary sizing of 6 foot D_{100} for ice forces, reduced to similar size to hydraulic force computation results
- Significant amount of uncertainty in analysis
- Ramp likely to experience damage during severe ice events that will require rock repositioning to meet passage objectives
- May need to grout a portion of the ramp below the crest for stability
- Ramp may suffer repetitive damage during annual ice out with higher risk for extreme events

3 Ramp Alternatives Considered

Several alternatives were formulated with the goal of balancing hydraulic performance necessary to achieve fish passage and total project costs. Results were presented to the BRT for review and a minimum threshold for ramp extents was established. Ramp alternatives evaluated are summarized in the table below.

Alternative Name	Description
Base Ramp	Full river width ramp
	0.4% average slope, includes 70 ft wide thalweg, average length of 1600 ft, river area of 32 acres
	Evaluated with both physical and numerical modeling
	Most suitable for BRT fish passage criteria
Tripled Slope Ramp	Tripled slope for entire ramp to reduce ramp extents
	Average length of 900 ft with 17 acre footprint
	Evaluated with numerical modeling only
	Eliminated from further consideration due to failure to meet BRT fish passage criteria
Full River Width TPC Ramp	Steepened ramp to a point where cost estimate approximately meets allowable TPC
	Average length of 550 ft with 9 acre footprint
	Evaluated with numerical modeling only
	Eliminated from further consideration due to failure to meet BRT fish passage criteria
Partial River Width TPC Ramp	Provided 300 ft wide passage corridor on right bank and set length to approximately meet TPC
	Evaluated with numerical modeling only
	Eliminated from further consideration due to failure to meet BRT fish passage criteria
Steepened Toe Ramp	Doubled slope on left bank for downstream portion of ramp
	Incorporated steepened toe to reduce ramp extents
	Average length of 1300 ft with 25 acre footprint
	Evaluated with both physical and numerical modeling
	Marginally meets BRT fish passage criteria
Double Slope Ramp	Doubled Sslope for entire ramp to reduce ramp extents
	Average length of 1000 ft with 19 acre footprint
	Evaluated with numerical modeling only
	Minimally meets BRT fish passage criteria
High Flow Bench Ramp	Modification of Double Slope Ramp
	Incorporates 100 ft top width widening on right bank
	Evaluated with numerical modeling only
	Improves passage at all flows
	Further refinement required if carried forward

Figure 2 - Ramp Alternatives Summary Table

3.1 Considered But Eliminated

3.1.1 Original Ramp Design

Following guidance provided by the BRT for hydraulic targets for depth and velocities necessary for Pallid Sturgeon migration through the project area, a base ramp geometry was developed for the project. The proposed ramp configuration includes a variable elevation crest combined with a variable slope rock ramp with features that mimic the natural thalweg of the river in plan form.

A 70’ channel inverted at elevation 1988.1 concentrates flows through the ramp. The invert elevation was selected to maintain water levels upstream of the dam sufficient for diversion of irrigation water through the headworks at 3000 cfs. The 70’ wide channel was selected to maintain depths of 3’ through the ramp at that 3000 cfs flow rate. The invert of the ramp traverses from the left bank to right as the ramp proceeds downstream, mimicking natural thalweg conditions in the area. The channel invert is at a 0% slope in the lateral direction. The remainder of the crest is sloped from 3.0% to 0.5% to eliminate areas where passing fish may be stranded. Lateral slopes are maintained throughout the ramp.

In order to lessen spikes in velocities experienced at the crest the ramp is variably sloped from the crest to the downstream toe. Upstream of the ramp, a 3:1 concrete crest slopes up to the proposed invert of the ramp. The concrete crest is completed with a 10’ flat section. The 3:1 crest serves to divert ice flows over the crest. Following the 10’ flat section, the remainder of the ramp would be constructed of rip-rap stone material. The first 500’ slopes at a rate of 0.002 ft/ft, followed by 400’ at 0.004 ft/ft, 400’ at 0.006 ft/ft, to ground at 0.008 ft/ft. The averaged slope of the ramp is approximately 0.0045 ft/ft.

The current configuration results in an overall ramp length of approximately 1600’. The layout occupies 32 acres of the river.

A physical model of this ramp geometry was constructed at the Reclamation Lab in Denver, Colorado. While in the majority of the model the results correlated with and confirmed the results of numerical modeling of the design, the physical model revealed higher velocities near the crest that were not apparent in numerical models. In order to maximize potential pallid passage modelers installed a rock boulder field downstream of the crest on the south side (right bank side) of the channel. The boulder field served to divert additional flow to the thalweg and towards the left bank of the channel which reduced crest velocities on the right side of the channel. One outcome of CRREL ice impact analysis was to show that such a boulder field could be subject to regular destruction by ice effects. Furthermore the movement of the boulders by the ice could result in a bulldozer effect which could cause damage to other parts of the ramp structure. Further consideration of a boulder field was retained as a potential adaptive management tool rather than as a design component.

This ramp geometry option was the most desirable to the BRT based the degree to which it met fish passage criteria, but it is currently considered infeasible due to excessive project costs.

3.1.2 Tripled Slope Ramp

In an effort to find a balance between an acceptable level of fish passage and total project costs, modifications were made to the original ramp geometry. To determine the minimal acceptable passage limits the ramp gradient was steepened to three times the original slope. Crest geometry remained the same, as did the thalweg configuration of the ramp.

The Tripled Slope geometry shortens the ramp length by 675’, reducing the total length from the crest to nearly 900’. The total area of this grading plan is 17 acres which represents a 47% reduction from the original design.

This option failed to provide suitable passage corridors for the range of flows studied and was eliminated on the basis of failing to provide adequate passage for the target species based on consultation with the BRT.

3.1.3 Total Project Cost Ramp (Full River Width)

In an effort to evaluate the flow conditions and possibility for fish passage resulting from a geometry that met the initial total project costs for a full river width ramp, a geometry was modeled that limited total riprap costs to approximately

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ten million dollars. The result was a total ramp length of 550', a reduction of over 1000' from the original proposal. The grading plan occupied 9 acres of river, a 72% reduction from the original plan.

This option failed to provide suitable passage corridors for the range of flows studied and was eliminated on the basis of failing to provide adequate passage for the target species based on consultation with the BRT.

3.1.4 Total Project Cost (Partial River Width)

In an effort to reduce ramp extent and evaluate hydraulic performance for a geometry that met total project costs, a partial river width ramp geometry was modeled that limited total riprap costs to approximately ten million dollars.

The result was a ramp that provided a 300' passage corridor on the right bank of the river with a 1.5% gradient. The remainder of the ramp was modeled as a 10% gradient.

This option was eliminated due to undesirable hydraulic performance and due to failure to meet BRT fish passage criteria. There was potential for scour along the steeper left side of the structure which would require substantial amounts of bank and bed protection to prevent erosive forces from damaging adjacent lands and the structure itself. The cost of this added protection would substantially offset the cost savings gained through construction of a partial channel width ramp.

3.2 Considered and Carried Forward

3.2.1 Steepened Toe Ramp

As an initial step in reducing riprap two facets of the original ramp geometry were modified. Analysis of the original rock ramp geometry indicated that most passage corridors were aligned along the right bank side of the proposed structure. In order to reduce the extents of the ramp, slopes along the left bank were approximately doubled. In addition, the toe of the ramp was adjusted to a steeper gradient of 2%. These changes were made without significant impacts to the hydraulic performance of the ramp as it relates to fish passage criteria.

The result was a ramp shortened in length relative to the original ramp design by 300' with a reduction in aerial extent of 22%.

Upon review by the BRT, this option was deemed a suitable alternative for Pallid Sturgeon passage, though less preferable than the originally proposed geometry.

3.2.2 Doubled Slope Ramp

To identify the threshold where increases in ramp slope resulted in insufficient passage, the original ramp was gradually steepened. The steepest configuration deemed biologically suitable comprised a geometry with a nominal slope approximately double that of the original design (0.9%).

The Doubled Slope geometry reduces ramp length relative to the original design by 600' with a reduction in aerial extent of 40%.

BRT review considers this option minimally acceptable as it relates to Pallid Sturgeon passage. All steeper gradient ramp options are considered unsuitable.

3.2.3 High Flow Bench Ramp

Following a late January meeting with the BRT to discuss passage criteria and progress status, a recommendation was made to include a "high flow bench" as a possible ramp feature. The ramp would serve to provide lower velocities in times of high flow on the fringes of the ramp as well as to provide additional passage potential around the crest.

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This feature was initially incorporated into the “Doubled Slope Ramp” by adding a 100’ wide bench along the right bank of the structure. Inclusion of this feature requires removal of the existing dam right bank abutment. While widening reduces depths across the crest, it also serves to alleviate some velocity concerns. The bench area is designed to become suitable for passage at flows upward of 30,000 cfs and is approximately 2’ higher than the adjacent portions of the ramp. The high flow bench would serve to provide an area of lower velocities during periods of flows exceeding 15,000 – 20,000 cfs

Initial USACE analysis indicates that inclusion of the bench could require a similar level and size of riprap as the rest of the ramp compared to other geometries evaluated. Since the bench is located at the margins of the channel near the crest it would be subject to significant yearly ice impact and could required significant maintenance. The BRT envisions such a bench to be constructed of mostly native granular material and to resemble point bars found near natural riffles on the Yellowstone. It may be possible that most of this section of the ramp could be filled with granular material similar to natural substrate present in the area, however, to maintain stability, larger sized riprap would need to be placed at the crest and as sills of larger riprap every 150-200’ as the bench proceeds downstream along the ramp. Since it the bench is located at the margins of the channel near the crest it would be subject to significant yearly ice impact and could required significant maintenance. Additional design is required to determine what level of protection will be required.

Initial ADH modeling of this option indicates improvements to the passage corridors at all flows. At lower flows the bench serves to maintain sufficient depths along the fringes of the ramp. Once inundated, the increased top width augments the area of suitable velocities for passage when compared with the currently proposed geometry. Presentations of the results to the BRT were met with mixed reactions. The utility of the high flow bench was acknowledged, however it was requested that the design be modified so the bench becomes functional at lower flow thresholds than currently configured. If a ramp alternative is ultimately selected for pallid pasage at Yellowstone intake modeling of the highflow bench alternative will take these recommendations into account. If BRT recommendations to lower the bench elevation are followed, it should be noted that material removal volume would be increased. In addition, bench erosion and stability risks would be higher. As a result, an increase in iniital project cost and O&M would be expected.

The cost estimate for the ramp only portion of this option will be similar to the cost of the double slope ramp plus added excavation costs for the bench. Rock protection costs are dependent on the final design for the feature, and could be significant.

4 Original Ramp Hydraulic Summary

The depth and velocity contours in this section display ADH results for the initial version ramp proposed at Intake. The figure below displays one foot contours of the original ramp (white). The ramp as proposed extends over 1600’ feet downstream of the crest.

A 70’ channel inverted at elevation 1988.1 concentrates flows through the ramp. The invert elevation was selected to maintain water levels upstream of the dam sufficient for diversion at 3000 cfs. The 70’ wide channel was selected to maintain depths of 3’ through the ramp at that 3000 cfs flow rate. The invert of the ramp traverses from the left bank to right as the ramp proceeds downstream, mimicking natural thalweg conditions in the area. The channel invert is at a 0% slope in the lateral direction. The remainder of the crest is sloped from 3.0% to 0.5% to eliminate areas where passing fish may be stranded. Lateral slopes are maintained throughout the ramp.

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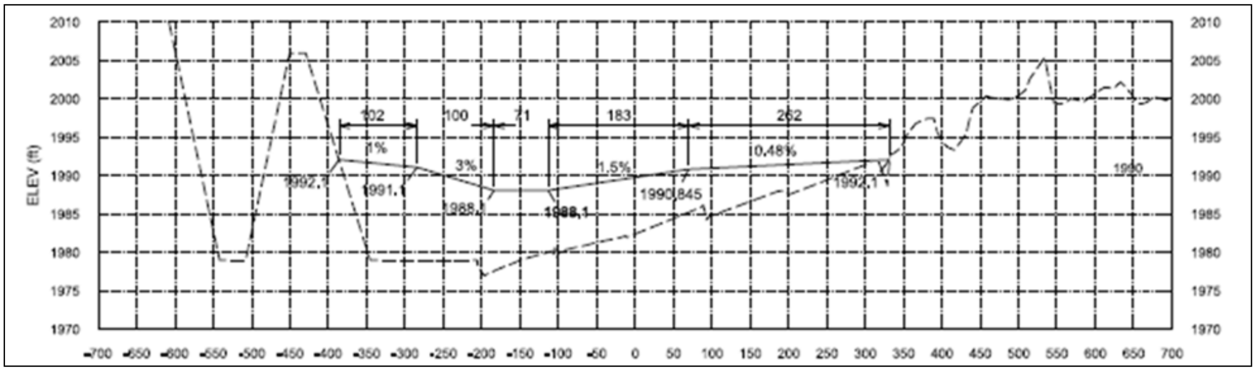

Figure 3 - Original Ramp Weir Crest Section

In order to lessen spikes in velocities experienced at the crest the ramp is variably sloped from the crest to the downstream toe. Upstream of the ramp, a 3:1 concrete crest slopes up to the proposed invert of the ramp. The concrete crest is completed with a 10’ flat section. The 3:1 crest serves to divert ice flows over the crest. Following the 10’ flat section, the remainder of the ramp will be constructed of rip-rap stone material. The first 500’ slopes at a rate of 0.002 ft/ft, followed by 400’ at 0.004 ft/ft, 400’ at 0.006 ft/ft, to ground at 0.008 ft/ft. The averaged slope of the ramp is approximately .0045 ft/ft. The current configuration results in an overall ramp length of approximately 1600’. The layout occupies 32 acres of the river.

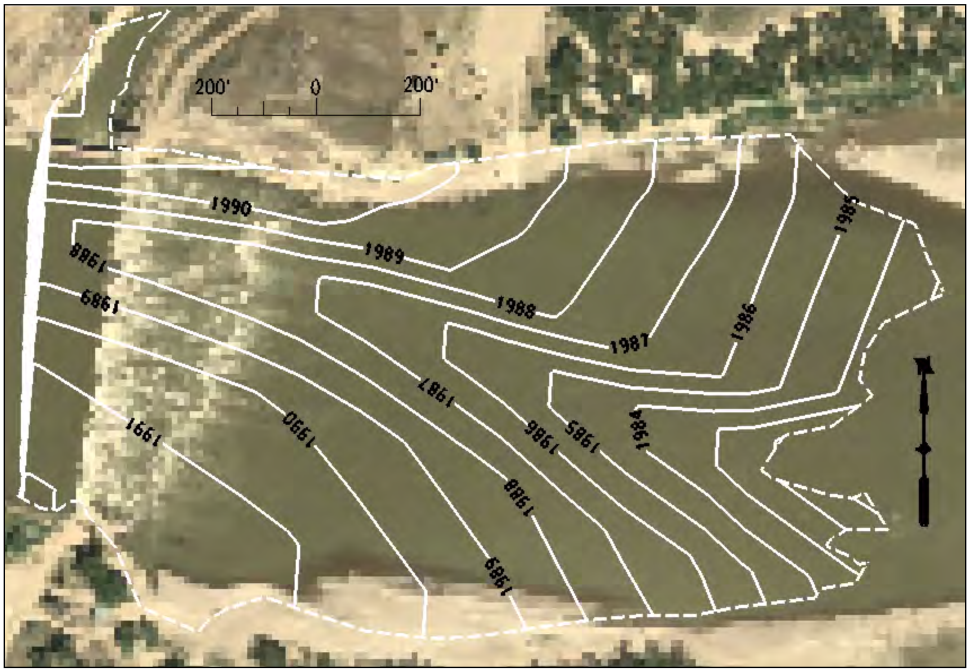

Figure 4 - Original Ramp 1' Contours

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Figure 5 - Original Ramp Depth Contours (presented in feet).

Depths throughout the ramp are greater than 1m at both 30 and 40 Kcfs. At 7 and 15 Kcfs, sufficient depths are only found through the low point of the ramp and diminish as the left and right edges of the ramp are approached. For all flows, a majority of the ramp is above depths of 0.5m.

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Figure 6 - Original Ramp Velocity Contours (presented in feet/second).

Velocities throughout the ramp are greater than 6 fps at 40,000 cfs, excluding the fringe areas. At 30,000 cfs, pathways are available through the crest at the 5-6 fps range. At 15,000 and 7,000 cfs, velocities meet the 4 fps criteria throughout, but are not necessarily accompanied with sufficient depths.

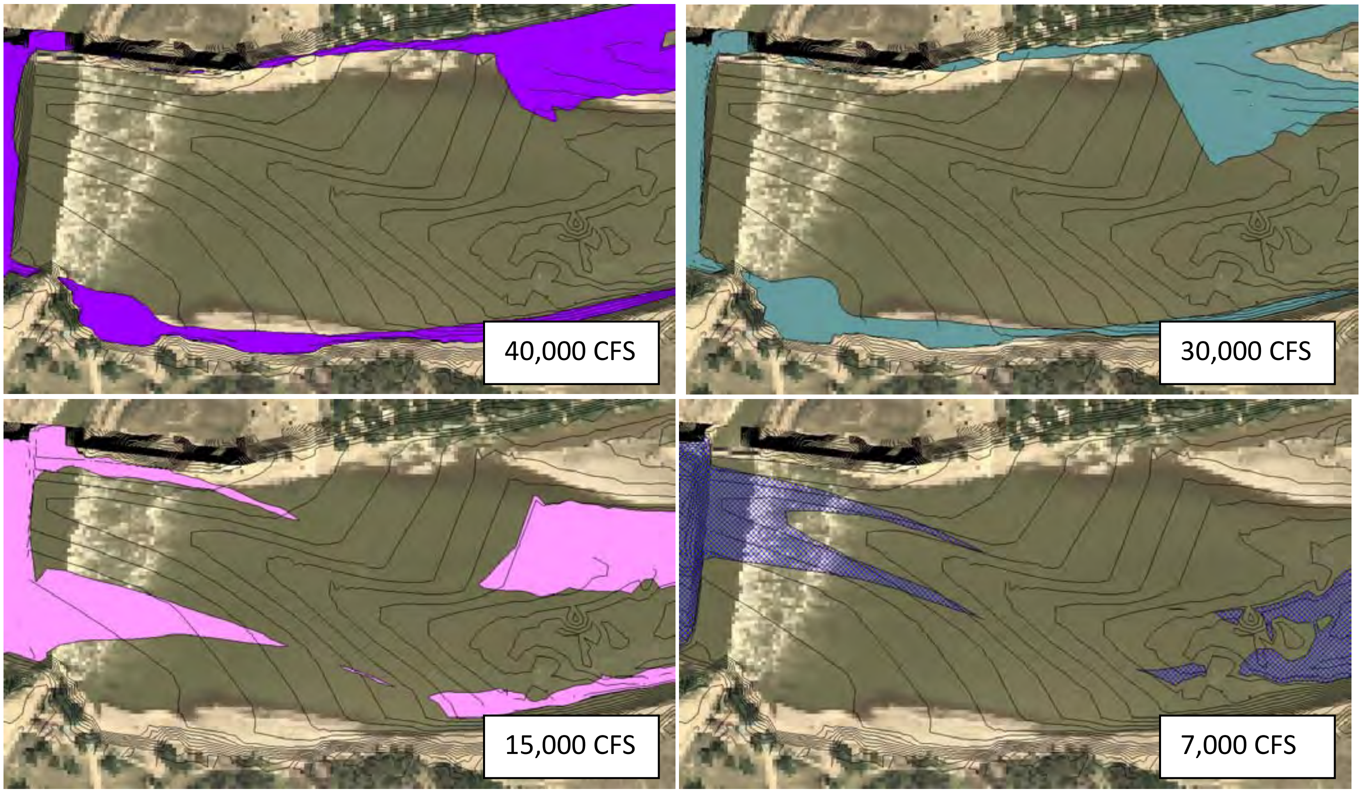

Figure 7 - Original Ramp passage delineations.

The colored portions of the figures above indicate areas of the ramp where hydraulic models show results meeting depth and velocity criteria of at least 1 m in depth and less than 4 feet per second velocities. The black lines indicate 1' contours of the proposed grading. At the two lower flows, insufficient depths prevent a passage corridor from extending the entire length of the ramp. At higher flows, the 4 fps velocity criteria is only met on the fringe areas of the ramp. Note the amount of area downstream of the ramp not meeting the specified criteria for all simulations.

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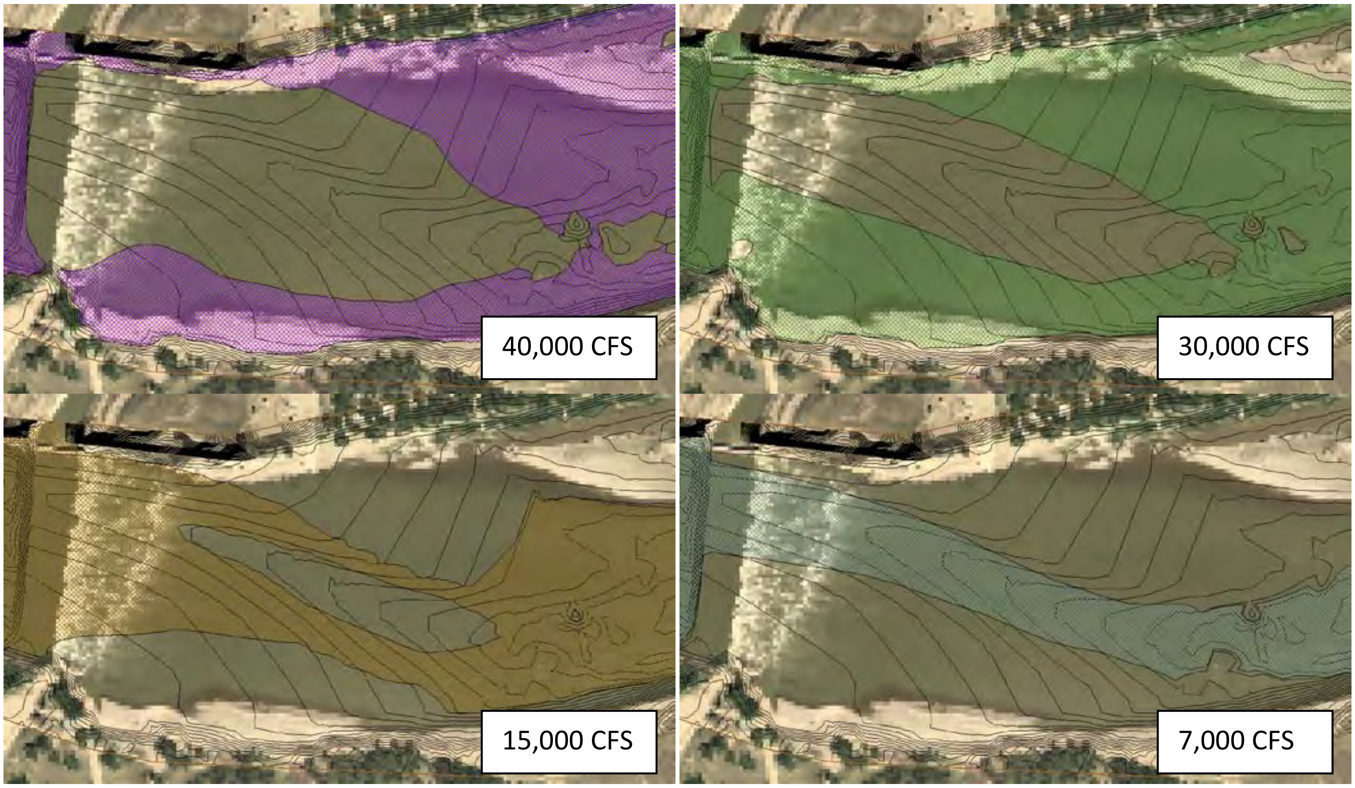

Figure 8 - Original Ramp passage delineations.

The colored portions of the figures above indicate areas of the ramp where hydraulic models show results meeting depth and velocity criteria of at least 1 m in depth and less than 6 feet per second velocities. The black lines indicate 1' contours of the proposed grading. Passageways are available at all flows under the 6 fps criteria, excluding the 40 Kcfs simulation at the crest. This problem could be mitigated through modification of the existing dam crest abutment.

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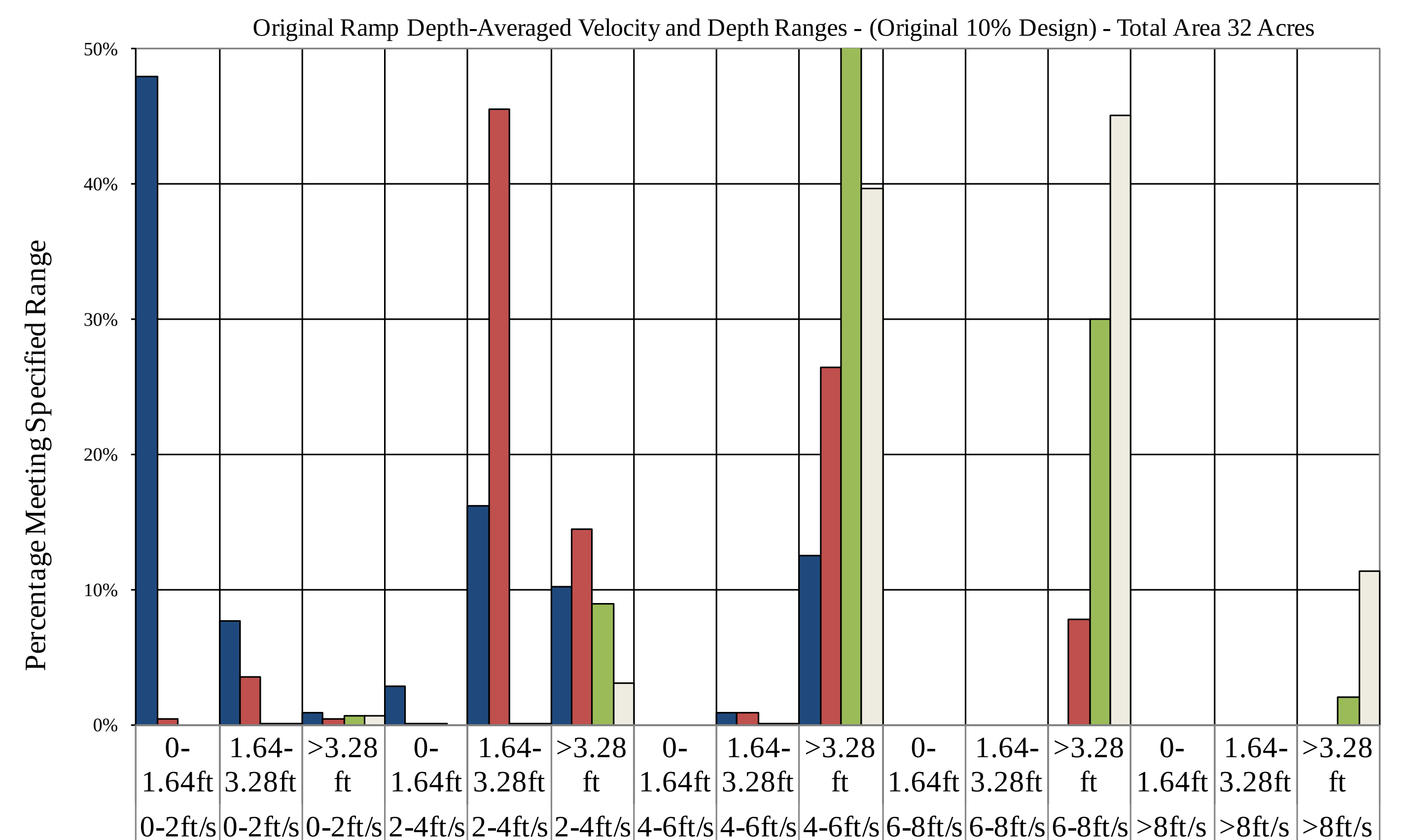

	0-1.64ft 0-2ft/s	1.64-3.28ft 0-2ft/s	>3.28 ft 0-2ft/s	0-1.64ft 2-4ft/s	1.64-3.28ft 2-4ft/s	>3.28 ft 2-4ft/s	0-1.64ft 4-6ft/s	1.64-3.28ft 4-6ft/s	>3.28 ft 4-6ft/s	0-1.64ft 6-8ft/s	1.64-3.28ft 6-8ft/s	>3.28 ft 6-8ft/s	0-1.64ft >8ft/s	1.64-3.28ft >8ft/s	>3.28 ft >8ft/s
7k	48%	8%	1%	3%	16%	10%	0%	1%	13%	0%	0%	0%	0%	0%	0%
15k	1%	4%	0%	0%	46%	15%	0%	1%	26%	0%	0%	8%	0%	0%	0%
30k	0%	0%	1%	0%	0%	9%	0%	0%	58%	0%	0%	30%	0%	0%	2%
40k	0%	0%	1%	0%	0%	3%	0%	0%	40%	0%	0%	45%	0%	0%	11%

Figure 9- Original Ramp Depth/Velocity Classifications.

This chart displays percent by area classifications of several depth and velocity combinations for the ramp for the set of flows modeled. The predominant flow classification for all flows is depths greater than a meter and velocities in the 4 to 6 fps range. However, at the 30,000 and 40,000 flow simulations significant portions of the ramp exceed 6 feet per second. Analysis of the velocity contours show that this occurs primarily in the thalweg of the ramp and areas adjacent.

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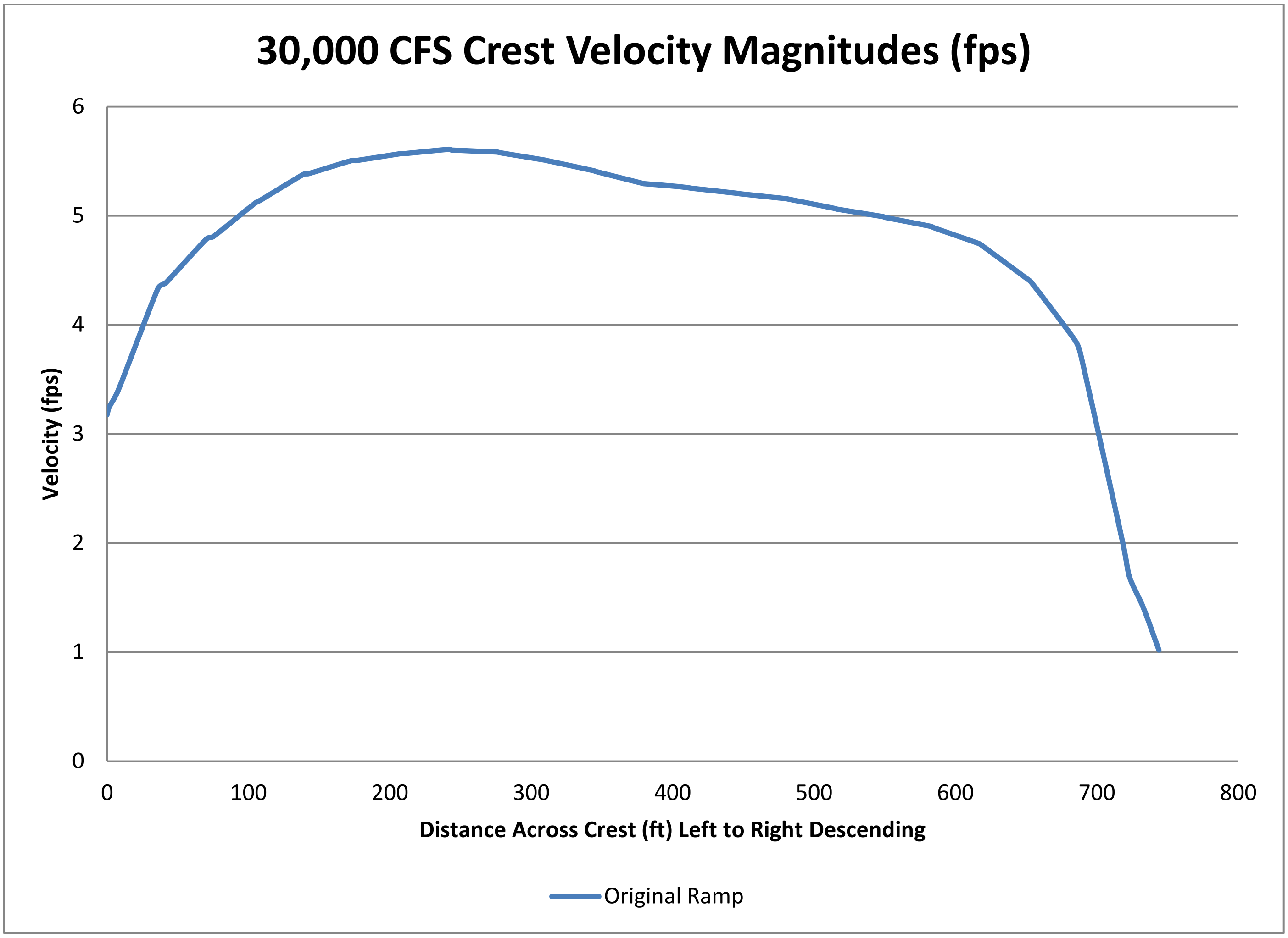

Figure 10 - Original Ramp crest velocity section.

This figure is a cross-section taken from near the proposed crest of the damn displaying velocities in fps for a 30,000 cfs flow rate. Numerical modeling indicates that velocities at the crest are below 6 fps across the crest for this geometry. 30,000 cfs represents a 50% exceedance by duration June flow rate.

5 Tripled Slope Ramp Hydraulic Summary

The depth and velocity contours in this section display ADH results for a shortened version of the ramp proposed at Intake. The figure below displays one foot contours of the original ramp (white) and the revised geometry (green). The revised geometry represents a tripling in slope from the original ramp. The crest and thalweg are of the same configuration as the original ramp proposal. The slope is increased to .006 ft/ft for the first 500' downstream, .012 for the next 400 feet, and .02 ft/ft for the tie into ground. The purpose of these revisions is to reduce material costs for construction of the ramp. It is important that appropriate passage corridors be maintained with any recommended geometry.

The original ramp extends approximately 1600' from the crest. The Doubled Slope geometry shortens the ramp length by 675', reducing the total length from the crest to nearly 1100'. The total area of this grading plan is 17 acres. This is a 47% reduction from the original plan.

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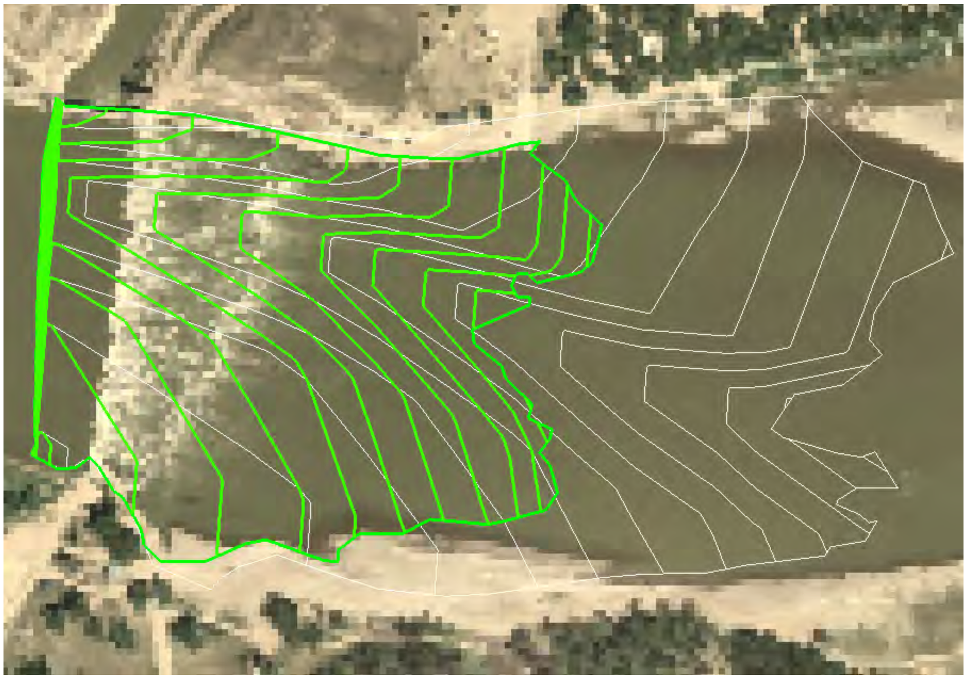

Figure 11 - 1' contours of original and Tripled Slope ramp.

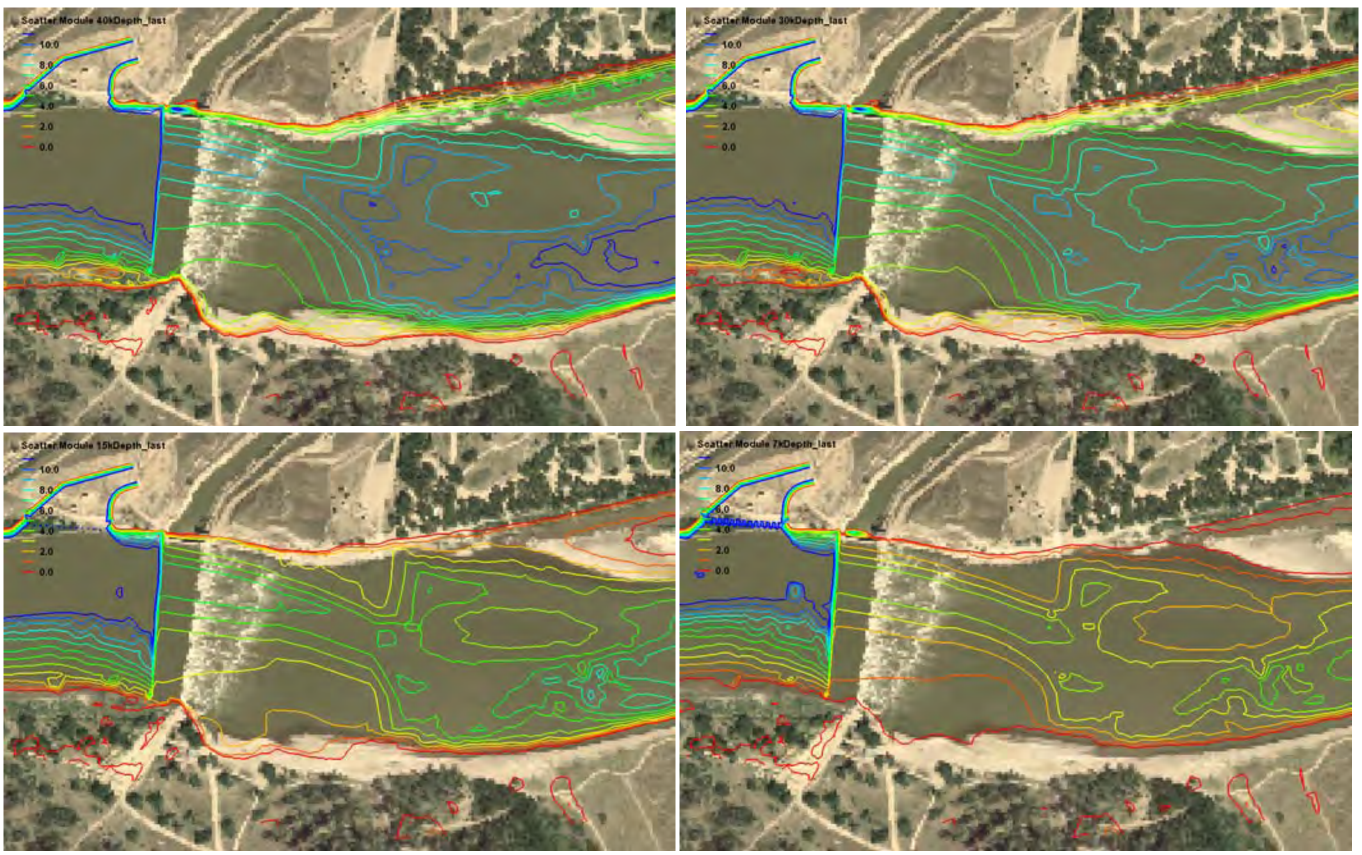

Figure 12 - Tripled Slope Ramp Depth Contours (presented in feet).

Depths throughout the ramp are greater than 1m at both 30 and 40 Kcfs. At 7 and 15 Kcfs, sufficient depths are only found through the low point of the ramp and diminish as the left and right edges of the ramp are approached. For all flows, a majority of the ramp is above depths of 0.5m.

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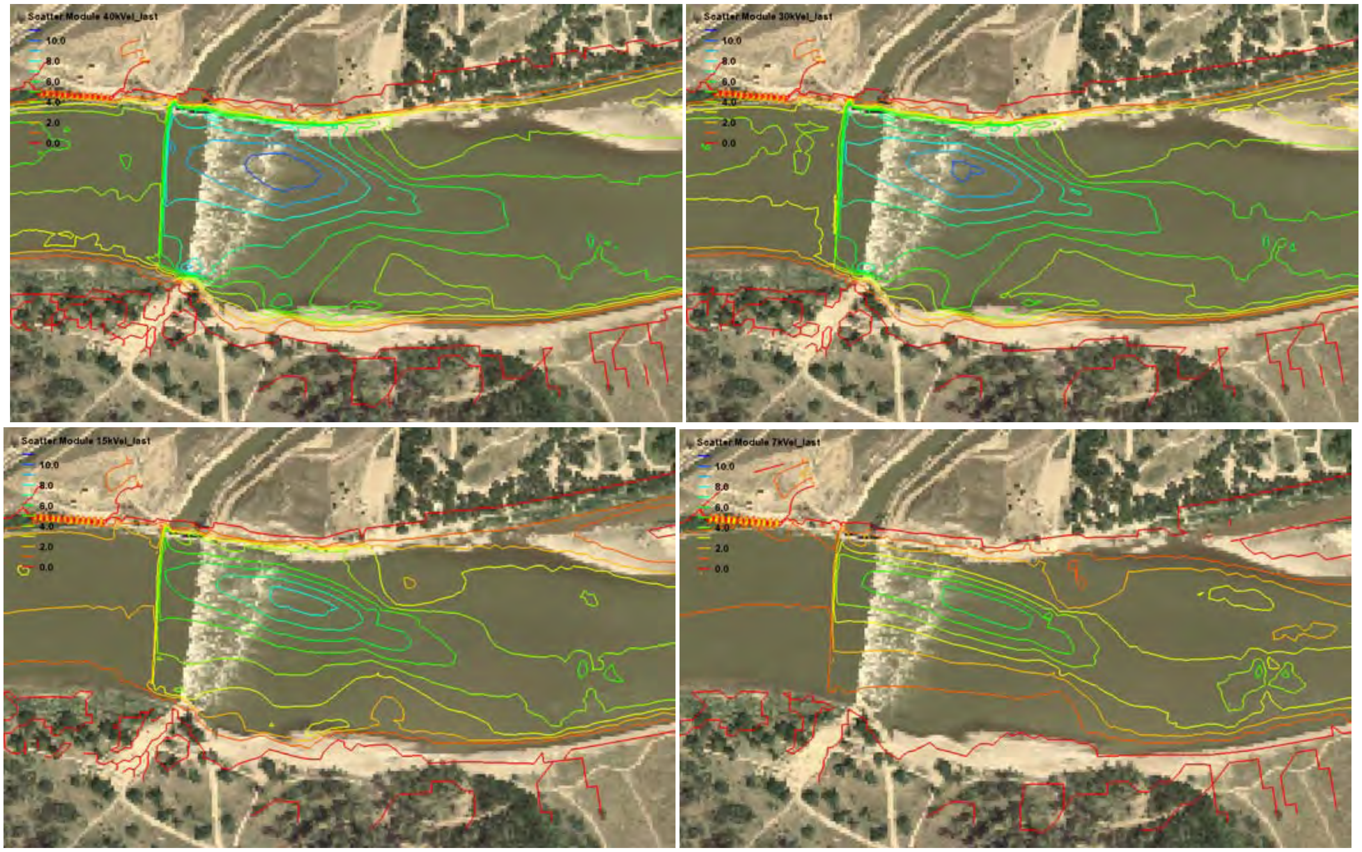

Figure 13 - Tripled Slope Ramp Velocity Contours (presented in feet/second).

Velocities throughout the ramp are greater than 6 fps at 40,000 cfs, excluding the fringe areas. At 30,000 cfs, pathways are available through the crest on the left side with velocities peaking at about 6.6 fps. This occurs not at the crest, but adjacent to the old weir crest abutment. At the crest, velocities on the left side are approximately 5.5 fps. At 15,000 and 7,000 cfs, velocities meet the 4 fps criteria throughout, but are not necessarily accompanied with sufficient depths.

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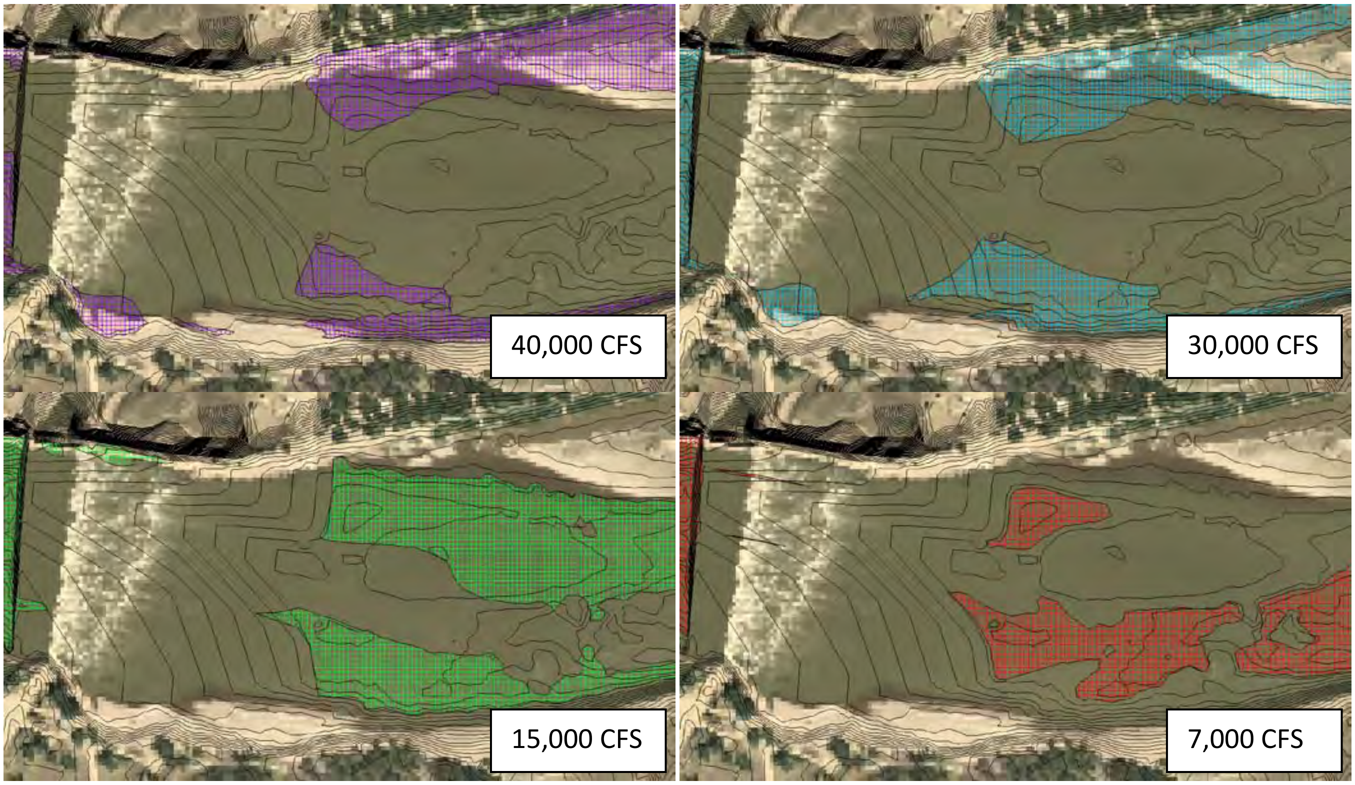

Figure 14 - Tripled Slope Ramp passage delineations.

The colored portions of the figures above indicate areas of the ramp where hydraulic models show results meeting depth and velocity criteria of at least 1 m in depth and less than 4 feet per second velocities. The black lines indicate 1' contours of the proposed grading. Corridors are not available at any of the flows simulated at this threshold.

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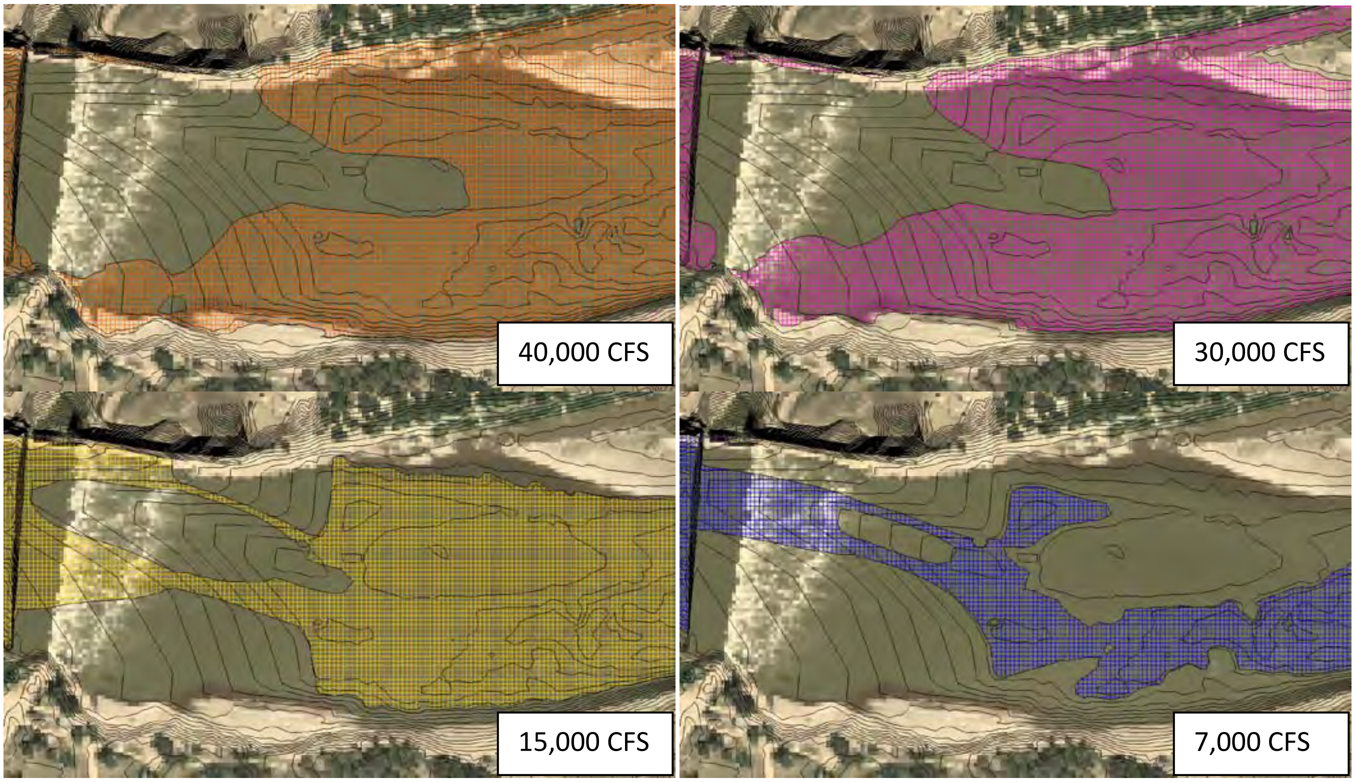
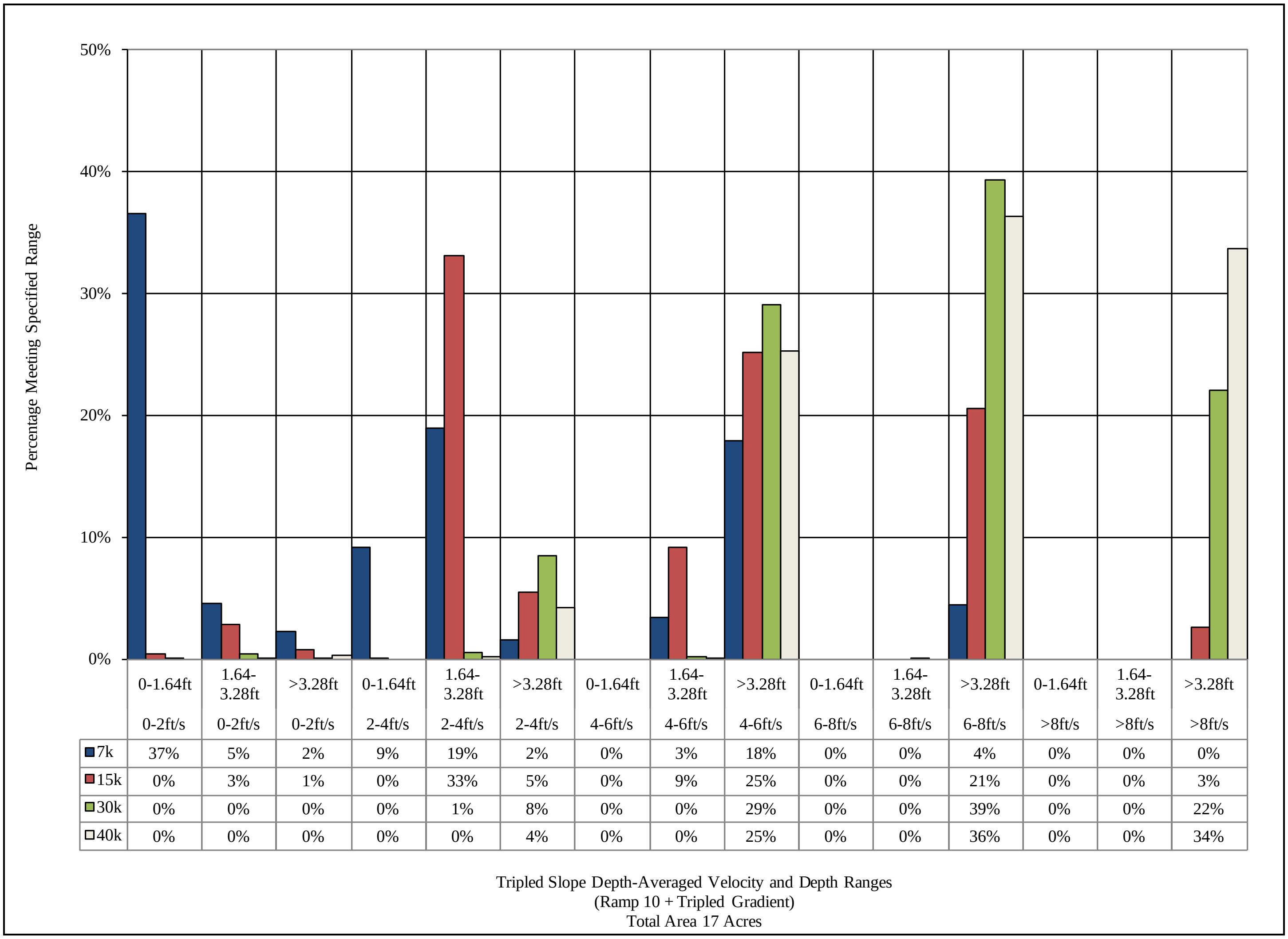


Figure 15 - Tripled Slope Ramp passage delineations.

The colored portions of the figures above indicate areas of the ramp where hydraulic models show results meeting depth and velocity criteria of at least 1 m in depth and less than 6 feet per second velocities. The black lines indicate 1' contours of the proposed grading. Passageways are available at all flows under the 6 fps criteria, excluding the 40 Kcfs and 30Kcfs simulations at the crest. This problem could be mitigated through modification of the existing dam crest abutment.



	0-1.64ft 0-2ft/s	1.64-3.28ft 0-2ft/s	>3.28ft 0-2ft/s	0-1.64ft 2-4ft/s	1.64-3.28ft 2-4ft/s	>3.28ft 2-4ft/s	0-1.64ft 4-6ft/s	1.64-3.28ft 4-6ft/s	>3.28ft 4-6ft/s	0-1.64ft 6-8ft/s	1.64-3.28ft 6-8ft/s	>3.28ft 6-8ft/s	0-1.64ft >8ft/s	1.64-3.28ft >8ft/s	>3.28ft >8ft/s
7k	37%	5%	2%	9%	19%	2%	0%	3%	18%	0%	0%	4%	0%	0%	0%
15k	0%	3%	1%	0%	33%	5%	0%	9%	25%	0%	0%	21%	0%	0%	3%
30k	0%	0%	0%	0%	1%	8%	0%	0%	29%	0%	0%	39%	0%	0%	22%
40k	0%	0%	0%	0%	0%	4%	0%	0%	25%	0%	0%	36%	0%	0%	34%

Figure 16 - Tripled Slope Ramp Depth/Velocity Classifications.

This chart displays percent by area classifications of several depth and velocity combinations for the ramp for the set of flows modeled. The predominant flow classification for all flows is depths greater than a meter and velocities in the 4 to 6 fps range for most flows. However, at the 30,000 and 40,000 flow simulations significant portions of the ramp exceed 6 feet per second.

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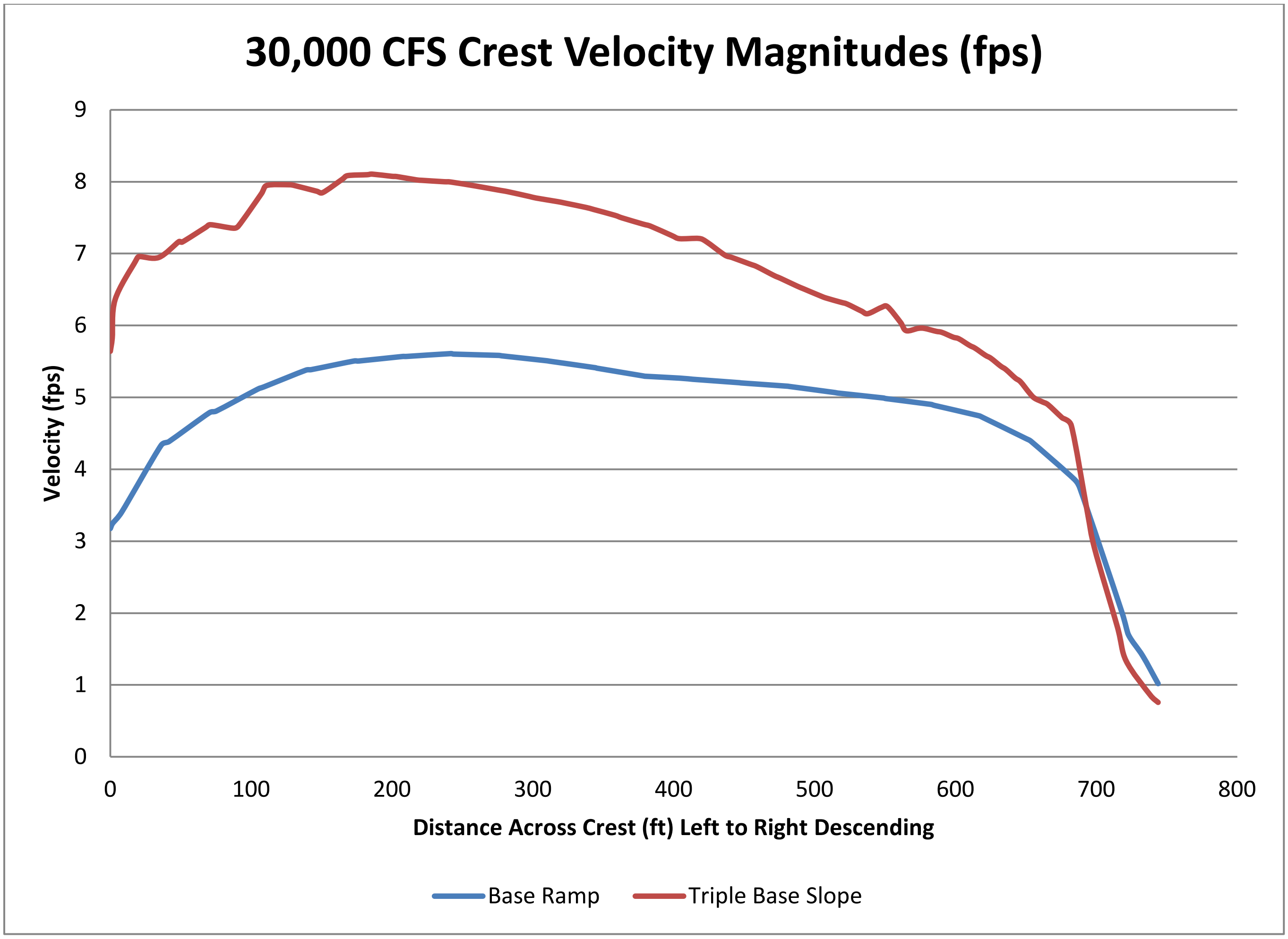

Figure 17 - Tripled Slope Ramp vs Original Ramp crest velocity comparison.

This figure is a cross-section taken from near the proposed crest of the damn displaying velocities in fps for a 30,000 cfs flow rate. Numerical modeling indicates that increases of 1-2.25 fps are resultant from the shortened geometry compared to the original proposal.

6 Total Rock Cost ~ $10 mil Ramp Hydraulic Summary

The depth and velocity contours in this section display ADH results for a shortened version of the ramp proposed at Intake. The figure below displays one foot contours of the original ramp (white) and the revised geometry (green). The slope is set to meet TPC projections based upon an assumed $70/ton rock cost and 6' layer thickness. The result is a total ramp length of ~550' and total ramp area of ~10 acres. Ramp slopes in the downstream direction vary from 1.5% to 2.2%. It is important that appropriate passage corridors be maintained with any recommended geometry.

The original ramp extends approximately 1600’ from the crest. The Total Rock Cost geometry shortens the ramp length by 1050’, reducing the total length from the crest to nearly 550’. The total area of this grading plan is 9 acres. This is a 72% reduction from the original plan.

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Figure 18 - 1' contours of original and Total Rock Cost Ramp.

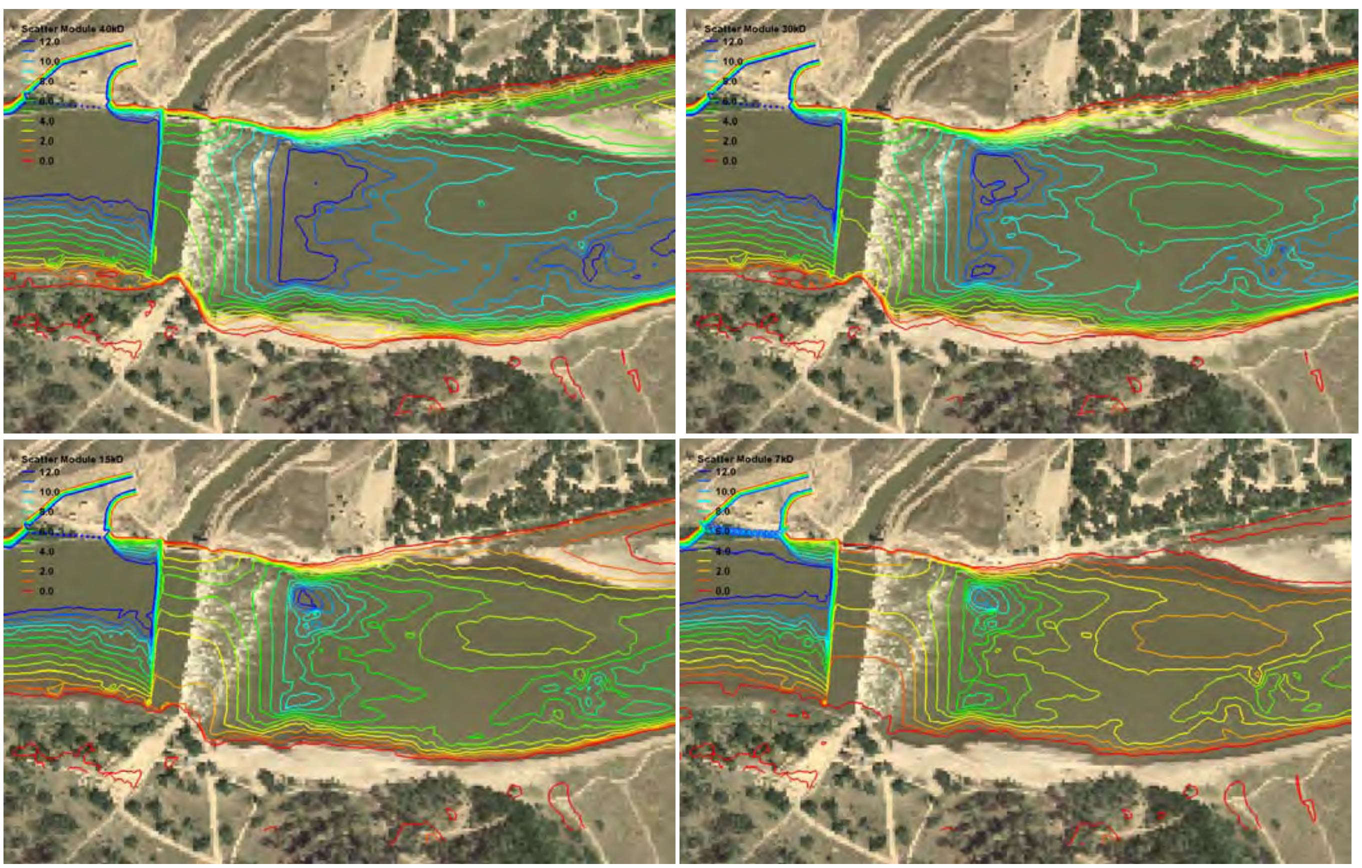

Figure 19 - Total Rock Cost Ramp Depth Contours (presented in feet).

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Depths throughout the ramp are greater than 1m at both 30 and 40 Kcfs. At 7 and 15 Kcfs, sufficient depths are only found through the low point of the ramp and diminish as the left and right edges of the ramp are approached. For all flows, a majority of the ramp is above depths of 0.5m.

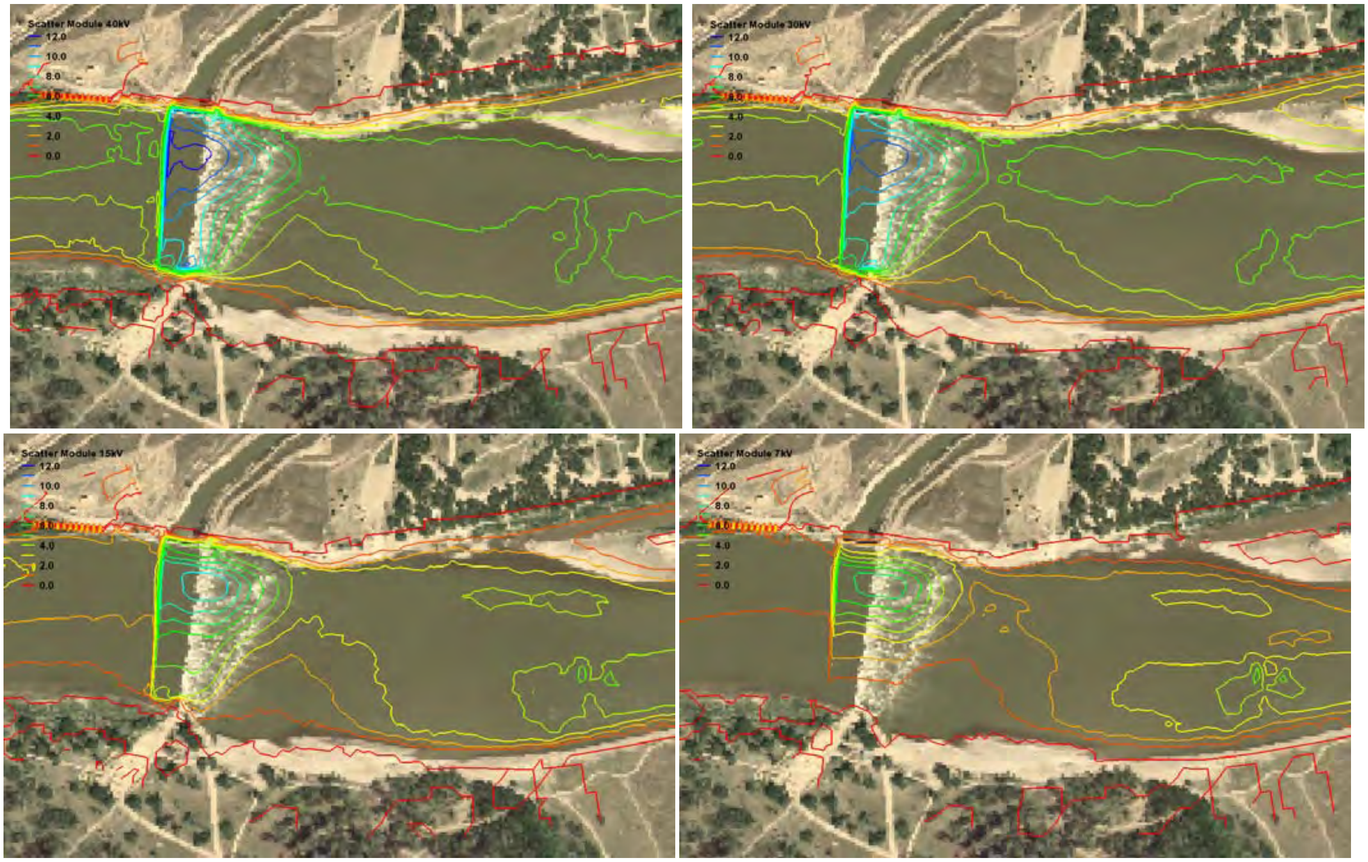

Figure 20 - Total Rock Cost Ramp Velocities (presented in feet per second).

Velocities throughout the ramp are greater than 6 fps at 40,000 cfs and 30,000 cfs. Velocities less than 6 feet per second are available at the 15,000 and 7,000 cfs flow rates, but are accompanied with depths less than a meter (though often greater than ½ meter). The result is that no passage corridors are found at any of the simulated flows, thus the figures are not presented.

7 Total Project Costs – Half River Width Ramp

The depth and velocity contours in this section display ADH results for a shortened version of the ramp designed to provide partial river width passage proposed at Intake. The figure below displays one foot contours of the original ramp (white) and the revised geometry (green). A 300' wide section on the left bank is intended to provide a passage corridor. The remainder of the ramp is sloped at a 10:1 to existing ground. The slope is set to meet TPC projections based upon an assumed $70/ton rock cost and 6' layer thickness. The result is a total ramp length of ~800' and total ramp area of ~10 acres. Passage corridor slopes in the downstream direction are approximately 1.5%. It is important that appropriate passage corridors be maintained with any recommended geometry.

The half river width ramp failed to provide a passage corridor at any of the flows model. This was primarily due to excessive velocities at the crest. Stability concerns also became apparent on the steeper section of the ramp due to

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velocities exceeding 15 fps in the 30,000 cfs and higher simulations. Supercritical flow is not a function of the ADH model used, but it is highly likely that this configuration would produce hydraulic jumps at the crest under many of the flow scenarios that could be encountered at the site.

Figure 21 - 1' contours of original and Half River Width Ramp.

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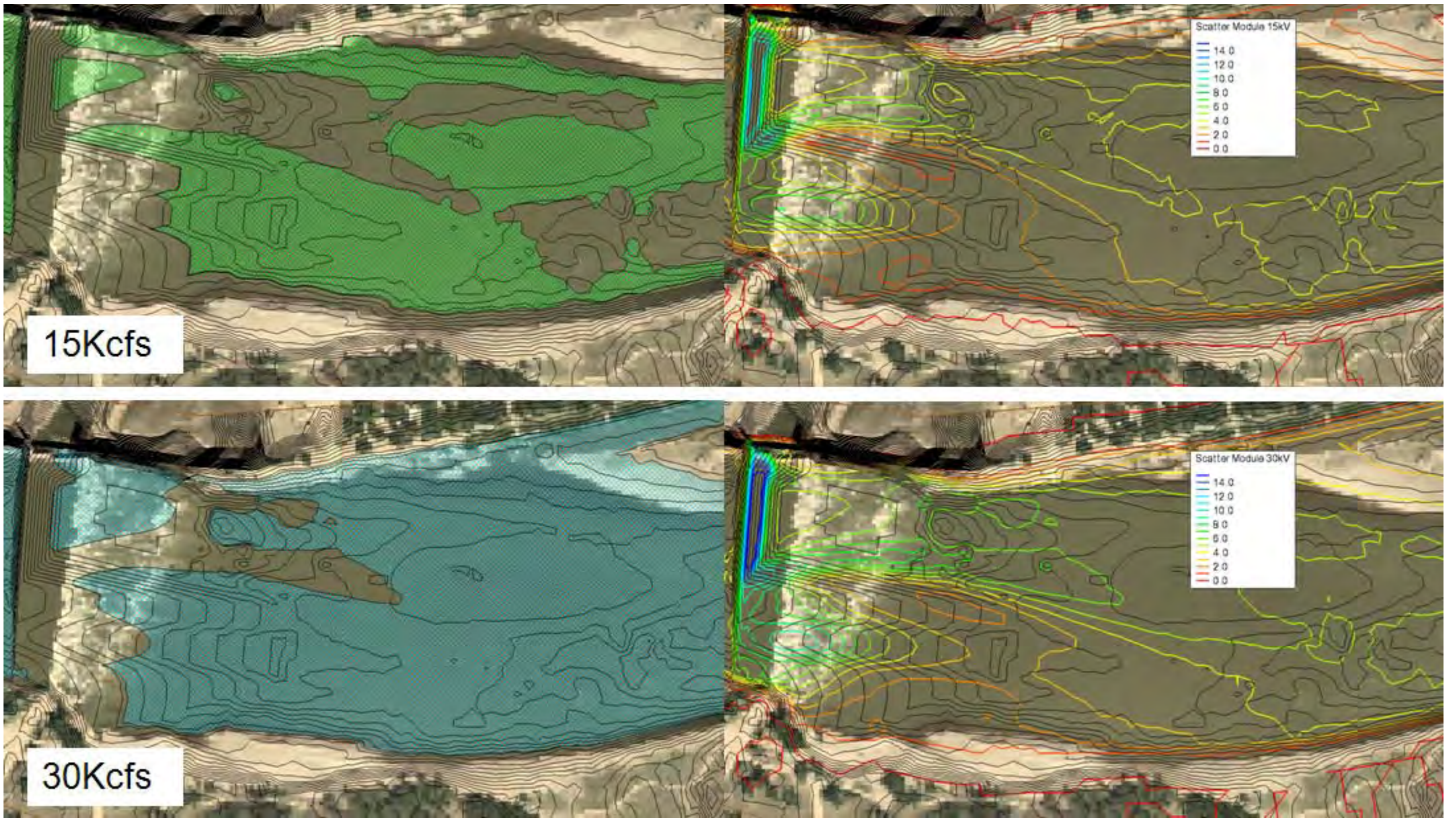

Figure 22 - Half River Width Ramp Velocities and Passage Classifications - 4 fps

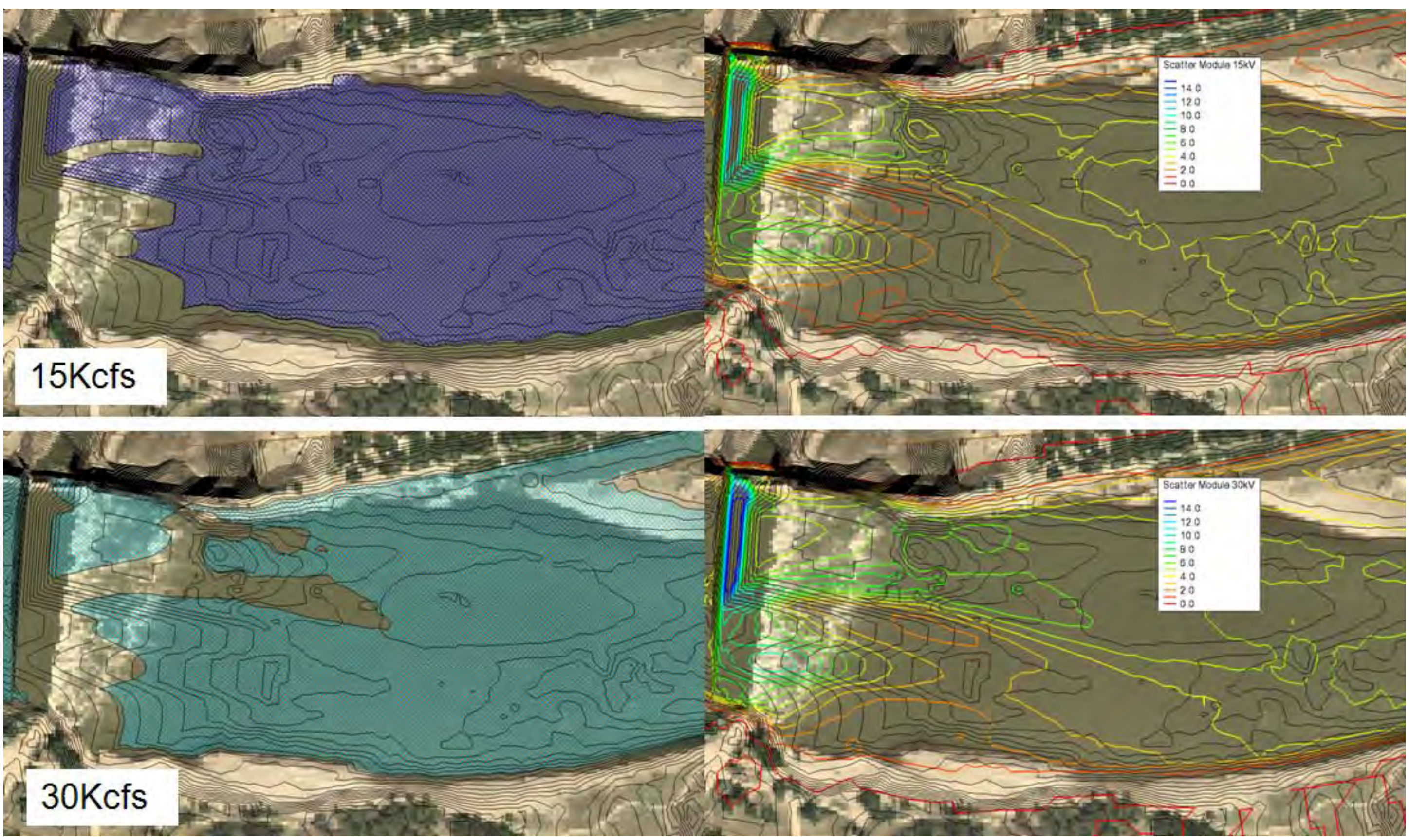

Figure 23 - Half River Width Ramp Velocities and Passage Classifications - 6 fps

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8 Steepened Toe Ramp Hydraulic Summary

The depth and velocity contours in this section display ADH results for a shortened version of the ramp proposed at Intake. The figure below displays one foot contours of the original ramp (white) and the revised geometry (green). The revised geometry represents a doubling in slope on the left bank compared to original and a 2% slope at the right bank toe. The purpose of these revisions is to eliminate areas where cut would be required to place stone on existing grades while maintaining the ability of the ramp to facilitate passage and provide diversion head. An additional purpose of these revisions is to reduce material costs for construction of the ramp. It is important that appropriate passage corridors be maintained with any recommended geometry.

Both the numerical hydraulic model and the physical model of the ramp show the revised geometry shows velocity and depth paths through the ramp similar to the original geometry. The original ramp extends approximately 1600' from the crest. The Steepened Toe geometry shortens the ramp length by 300', reducing the total length from the crest to nearly 1300'. The total area of this grading plan is 25 acres. This is a 22% reduction from the original plan.

Figure 24 - 1' contours of original and Steepened Toe ramp.

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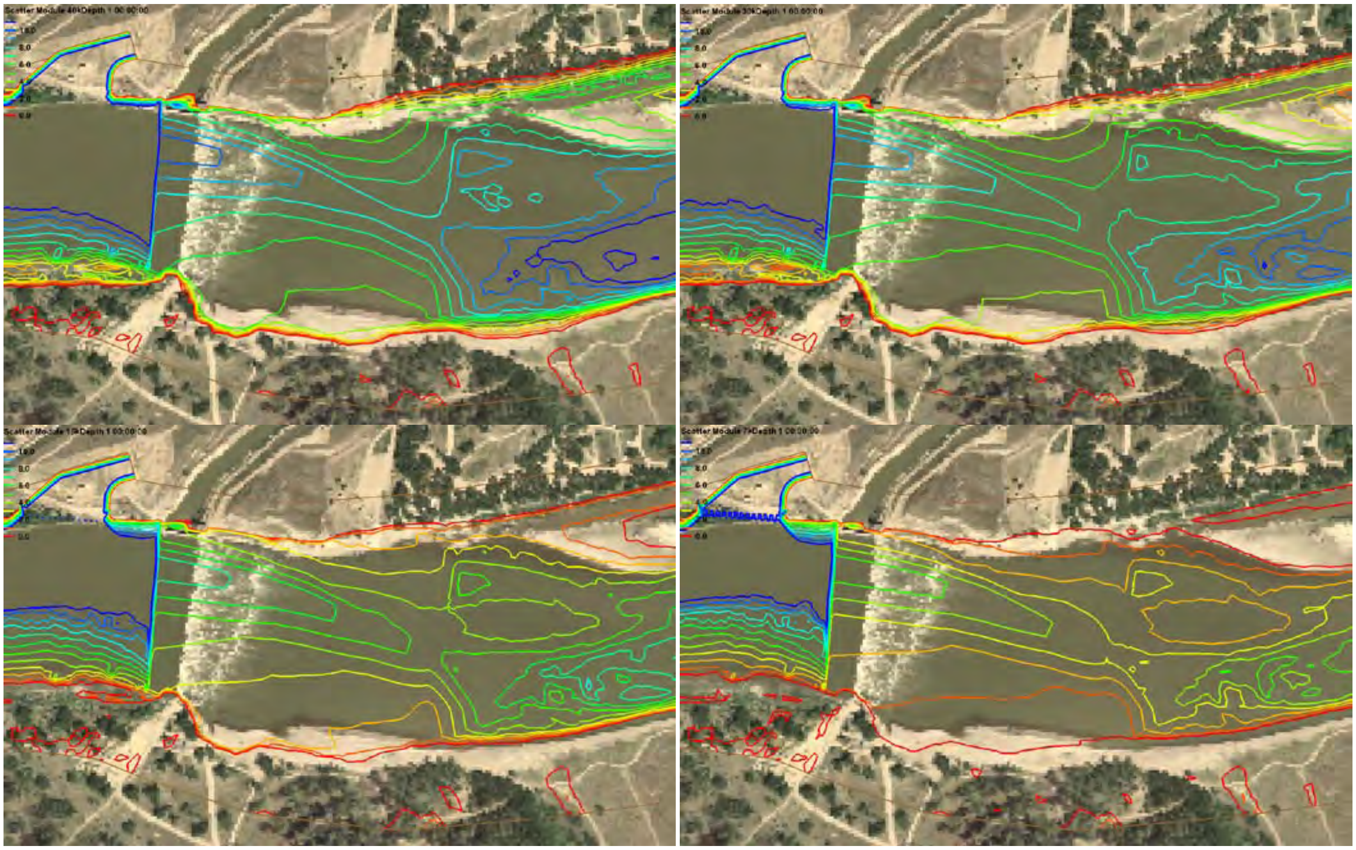

Figure 25 - Steepened Toe Ramp Depth Contours (presented in feet).

Depths throughout the ramp are greater than 1m at both 30 and 40 Kcfs. At 7 and 15 Kcfs, sufficient depths are only found through the low point of the ramp and diminish as the left and right edges of the ramp are approached. For all flows, a majority of the ramp is above depths of 0.5m.

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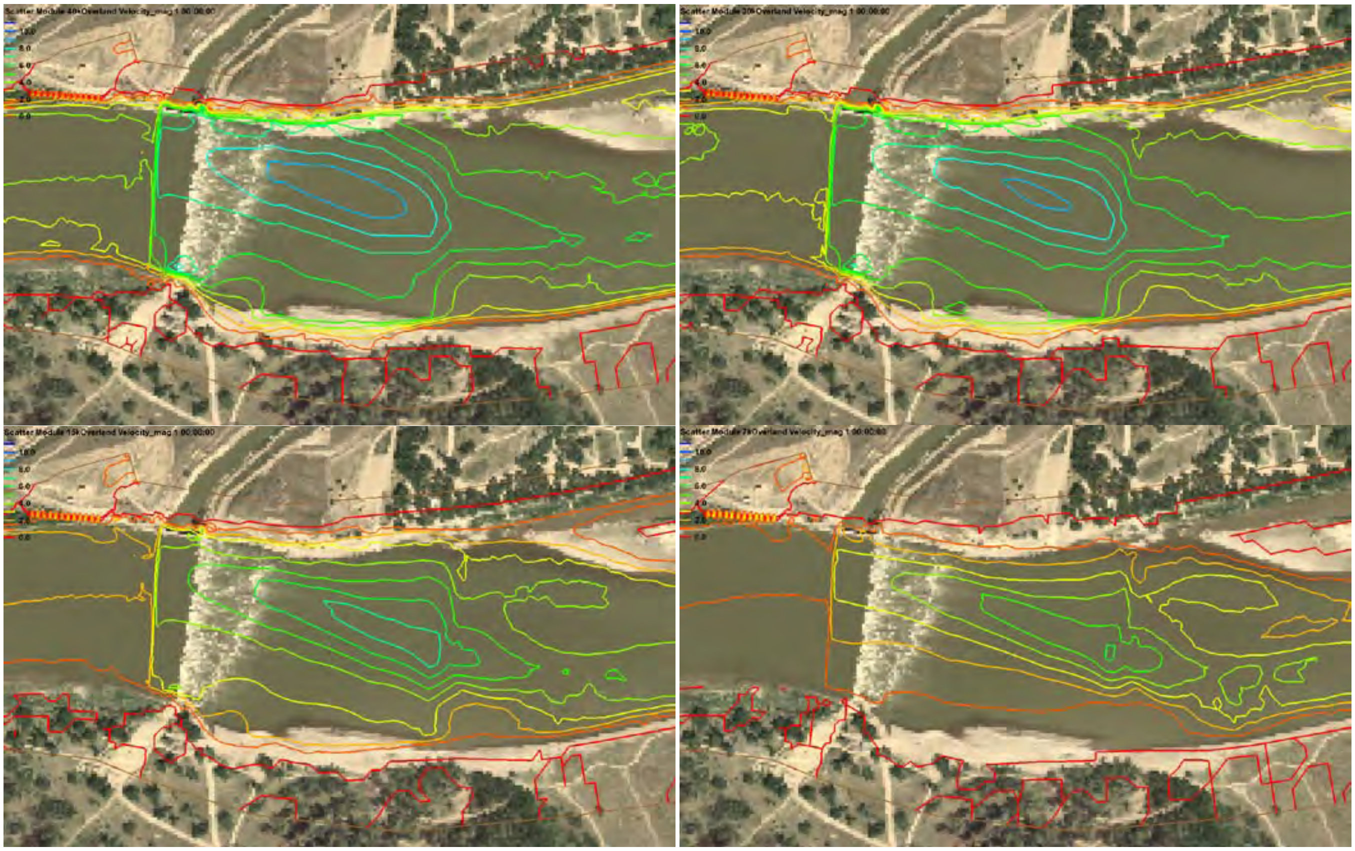

Figure 26 - Steepened Toe Ramp Velocity Contours (presented in feet/second).

Velocities throughout the ramp are greater than 6 fps at 40,000 cfs, excluding the fringe areas. At 30,000 cfs, pathways are available through the crest at the 5-6 fps range. At 15,000 and 7,000 cfs, velocities meet the 4 fps criteria throughout, but are not necessarily accompanied with sufficient depths.

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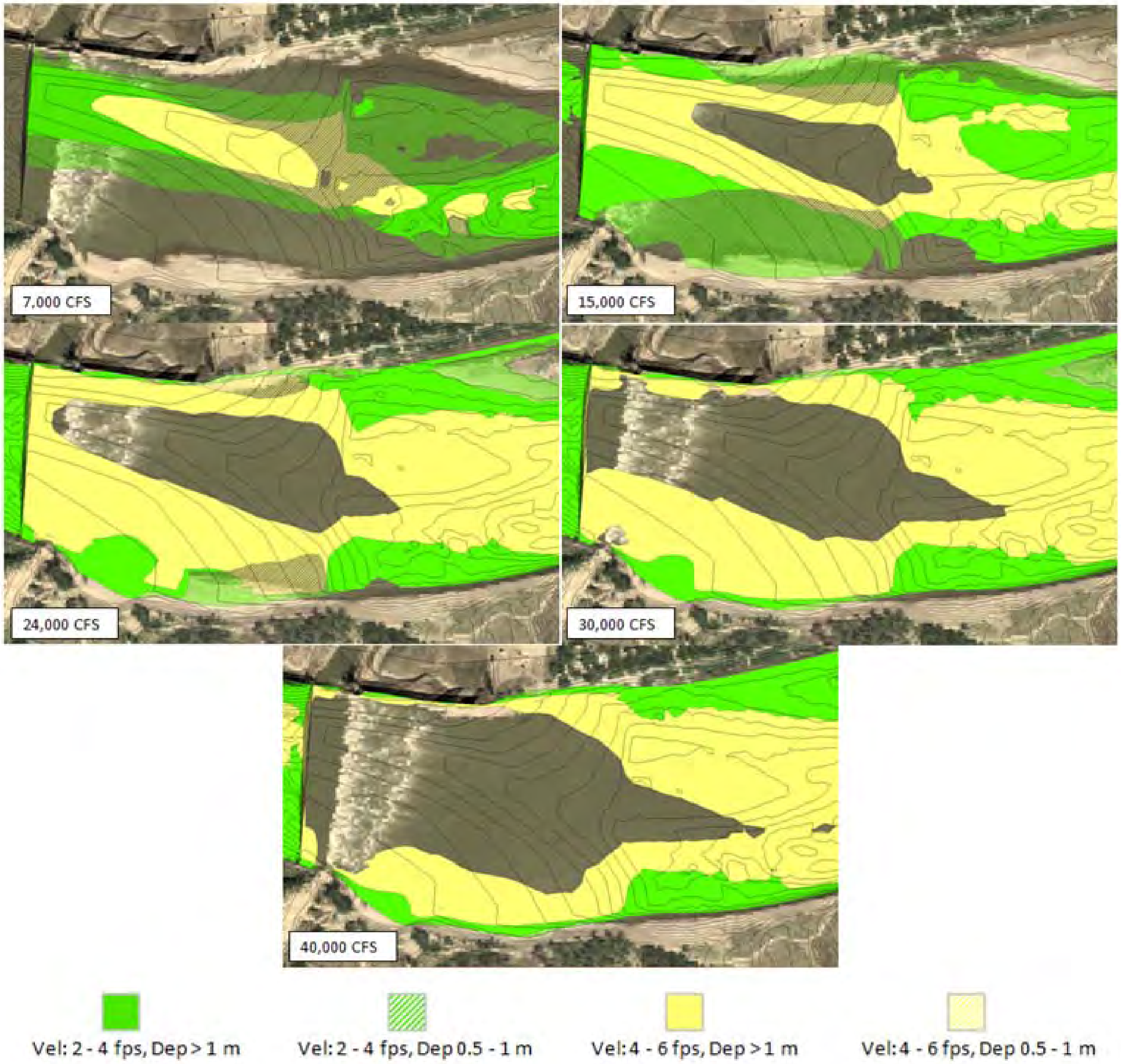
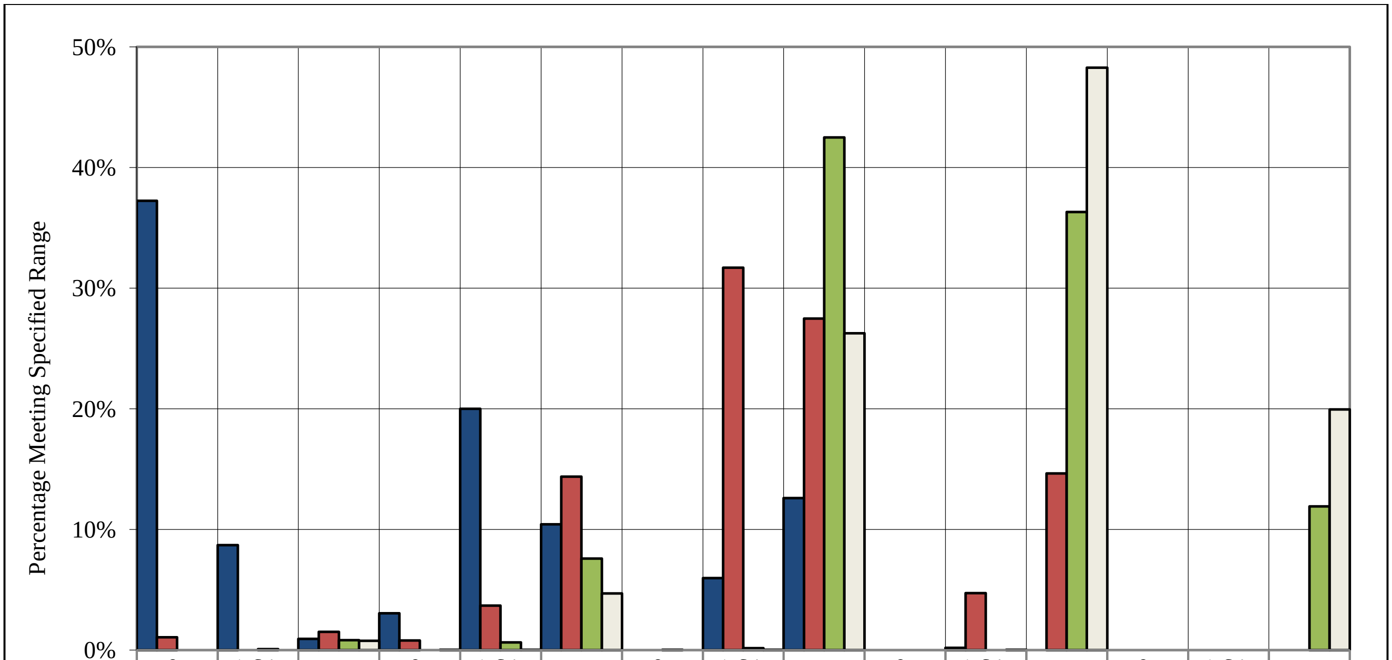


Figure 27 - Steepened Toe Ramp Passage Delineations.

The colored portions of the figures above indicate areas of the ramp where hydraulic models show results meeting depth and velocity criteria specified by the BRT. The black lines indicate 1' contours of the proposed grading. At the two lower flows, insufficient depths prevent a passage corridor from extending the entire length of the ramp at 4 fps. At higher flows, the 4 fps velocity criteria is only met on the fringe areas of the ramp. Note the amount of area downstream of the ramp not meeting the 4 fps criteria for all simulations.

Passageways are available at all flows under the 6 fps criteria, excluding the 40 Kcfs simulation at the crest. This problem could be mitigated through modification of the existing dam crest abutment.



	0-1.64ft 0-2ft/s	1.64-3.28ft 0-2ft/s	>3.28ft 0-2ft/s	0-1.64ft 2-4ft/s	1.64-3.28ft 2-4ft/s	>3.28ft 2-4ft/s	0-1.64ft 4-6ft/s	1.64-3.28ft 4-6ft/s	>3.28ft 4-6ft/s	0-1.64ft 6-8ft/s	1.64-3.28ft 6-8ft/s	>3.28ft 6-8ft/s	0-1.64ft >8ft/s	1.64-3.28ft >8ft/s	>3.28ft >8ft/s
7k	37%	9%	1%	3%	20%	10%	0%	6%	13%	0%	0%	0%	0%	0%	0%
15k	1%	0%	2%	1%	4%	14%	0%	32%	27%	0%	5%	15%	0%	0%	0%
30k	0%	0%	1%	0%	1%	8%	0%	0%	43%	0%	0%	36%	0%	0%	12%
40k	0%	0%	1%	0%	0%	5%	0%	0%	26%	0%	0%	48%	0%	0%	20%

Figure 28 - Steepened Toe Ramp Depth/Velocity Classifications.

This chart displays percent by area classifications of several depth and velocity combinations for the ramp for the set of flows modeled. The predominant flow classification for all flows is depths greater than a meter and velocities in the 4 to 6 fps range. However, at the 30,000 and 40,000 flow simulations significant portions of the ramp exceed 6 feet per second. Analysis of the velocity contours show that this occurs primarily in the thalweg of the ramp and areas adjacent.

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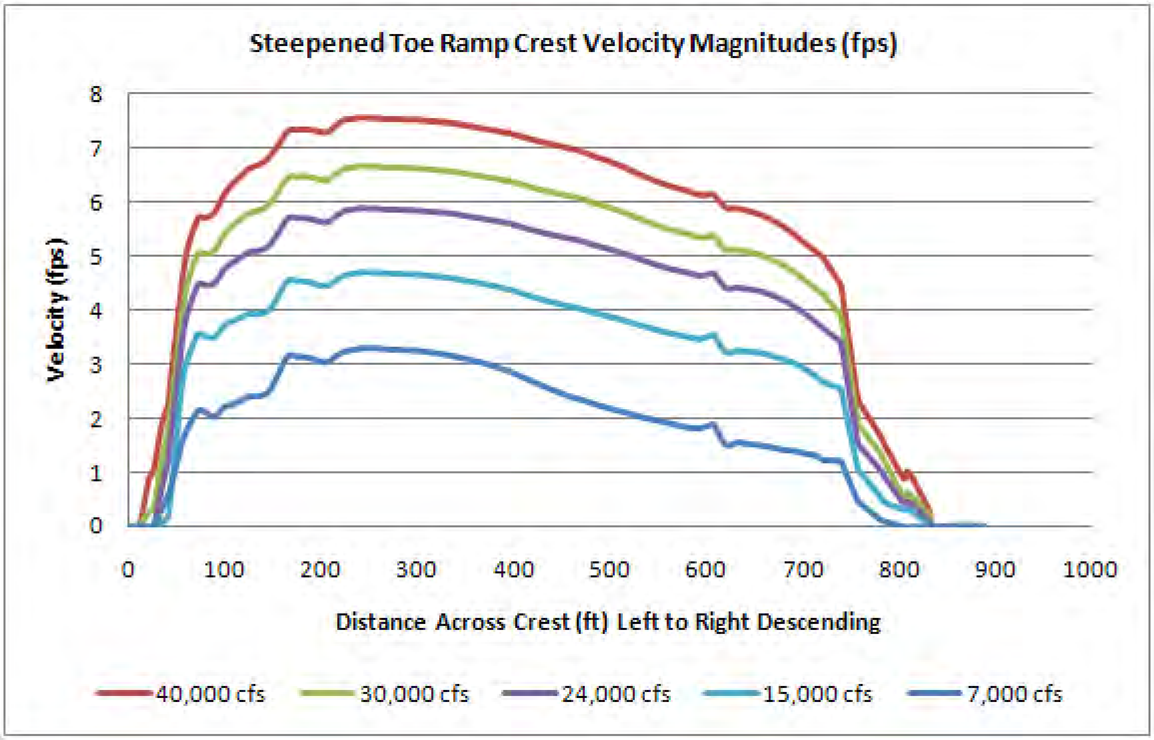

Figure 29 - Steepened Toe Ramp Velocities at the Crest.

This figure is a cross-section taken from near the proposed crest of the damn displaying velocities in fps for a several modeled flow rates. Numerical modeling indicates that increases of 0.5-1 fps are resultant from the shortened geometry compared to the original proposal.

9 Doubled Slope Ramp Hydraulic Summary

The depth and velocity contours in this document display ADH results for a shortened version of the ramp proposed at Intake. The figure below displays one foot contours of the original ramp (white) and the revised geometry (green). The revised geometry represents a doubling in slope from the original ramp. The crest and thalweg are of the same configuration as the original ramp proposal. The slope is increased to .004 ft/ft for the first 500' downstream, .008 for the next 400 feet, and .02 ft/ft for the tie into ground. The purpose of these revisions is to reduce material costs for construction of the ramp. It is important that appropriate passage corridors be maintained with any recommended geometry.

The original ramp extends approximately 1600’ from the crest. The Doubled Slope geometry shortens the ramp length by 600’, reducing the total length from the crest to nearly 1000’. The total area of this grading plan is 19 acres. This is a 40% reduction from the original plan.

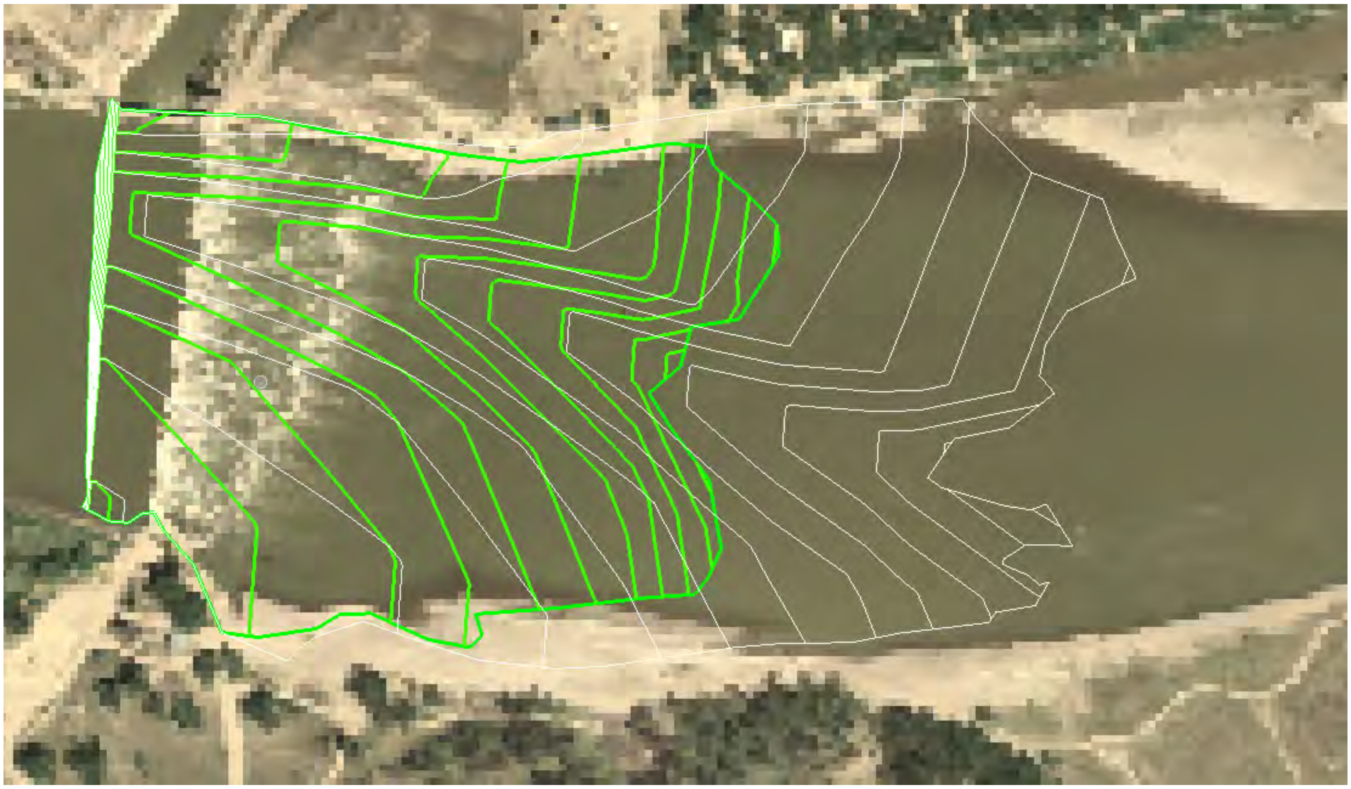

Figure 30 - 1' contours of original and Doubled Slope ramp.

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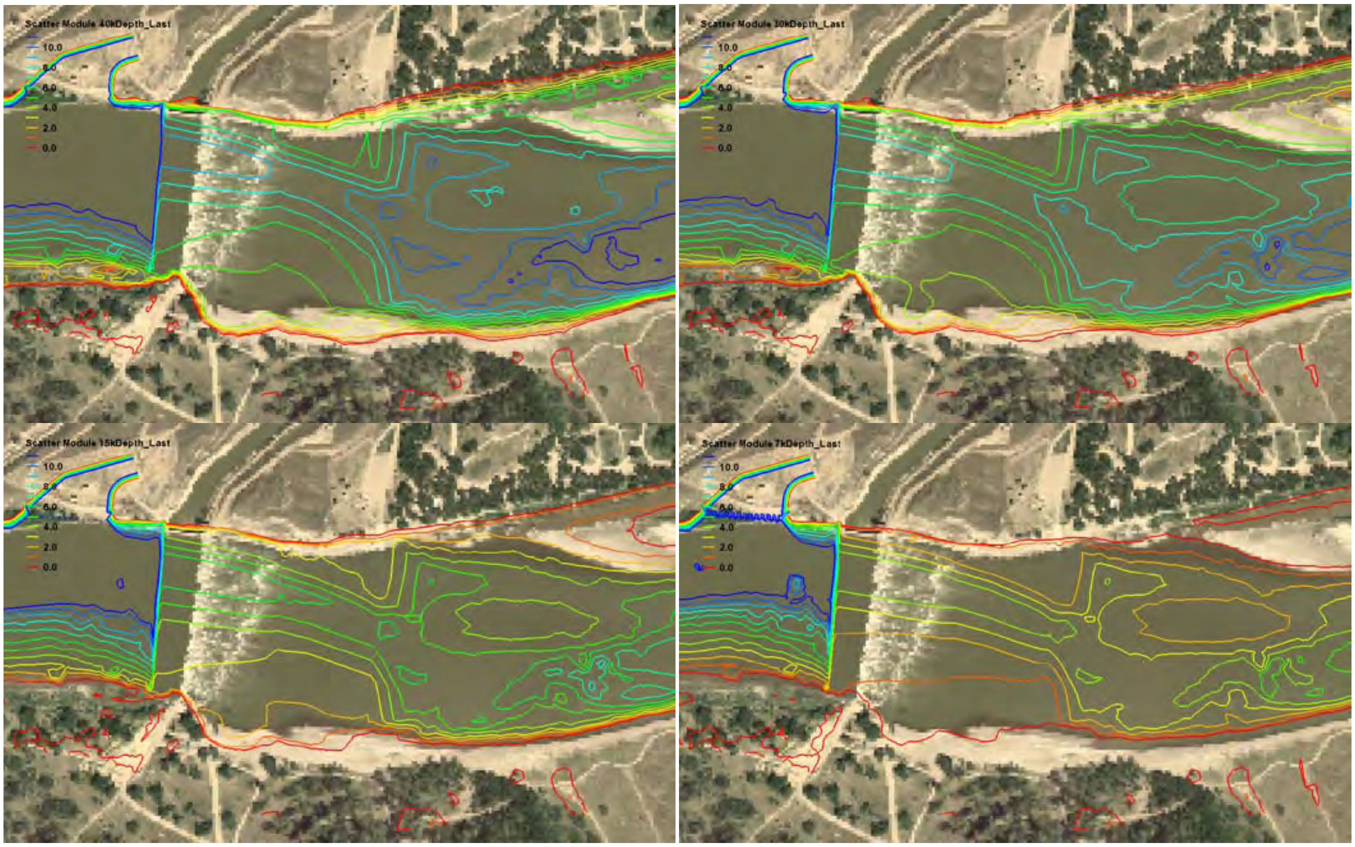

Figure 31 - Doubled Slope Ramp Depth Contours (presented in feet).

Depths throughout the ramp are greater than 1m at both 30 and 40 Kcfs. At 7 and 15 Kcfs, sufficient depths are only found through the low point of the ramp and diminish as the left and right edges of the ramp are approached. For all flows, a majority of the ramp is above depths of 0.5m.

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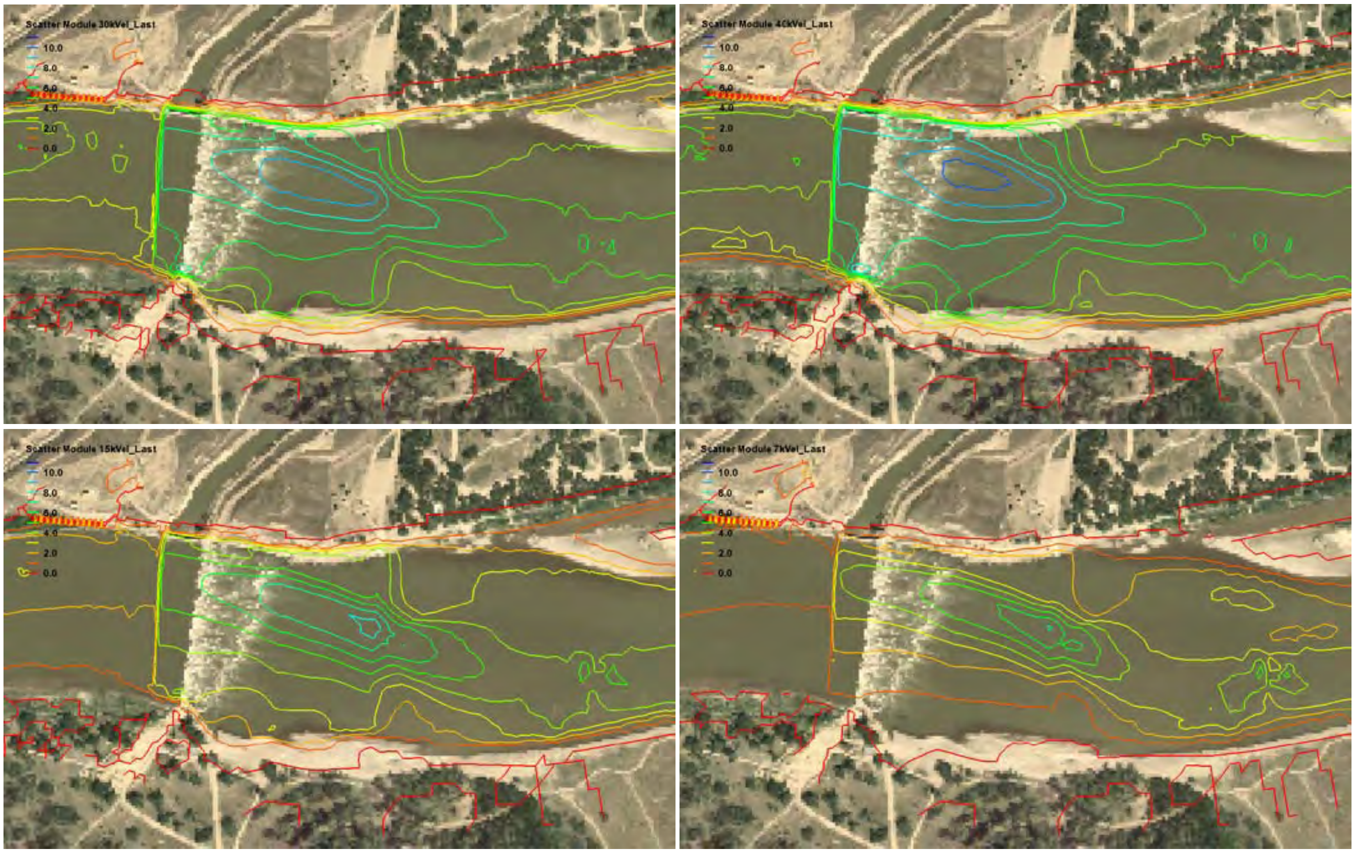

Figure 32 - Doubled Slope Ramp Velocity Contours (presented in feet/second).

Velocities throughout the ramp are greater than 6 fps at 40,000 cfs, excluding the fringe areas. At 30,000 cfs, pathways are available through the crest on the left side with velocities peaking at about 6.2 fps. This occurs not at the crest, but adjacent to the old weir crest abutment. At the crest, velocities on the left side are approximately 5.5 fps. At 15,000 and 7,000 cfs, velocities meet the 4 fps criteria throughout, but are not necessarily accompanied with sufficient depths.

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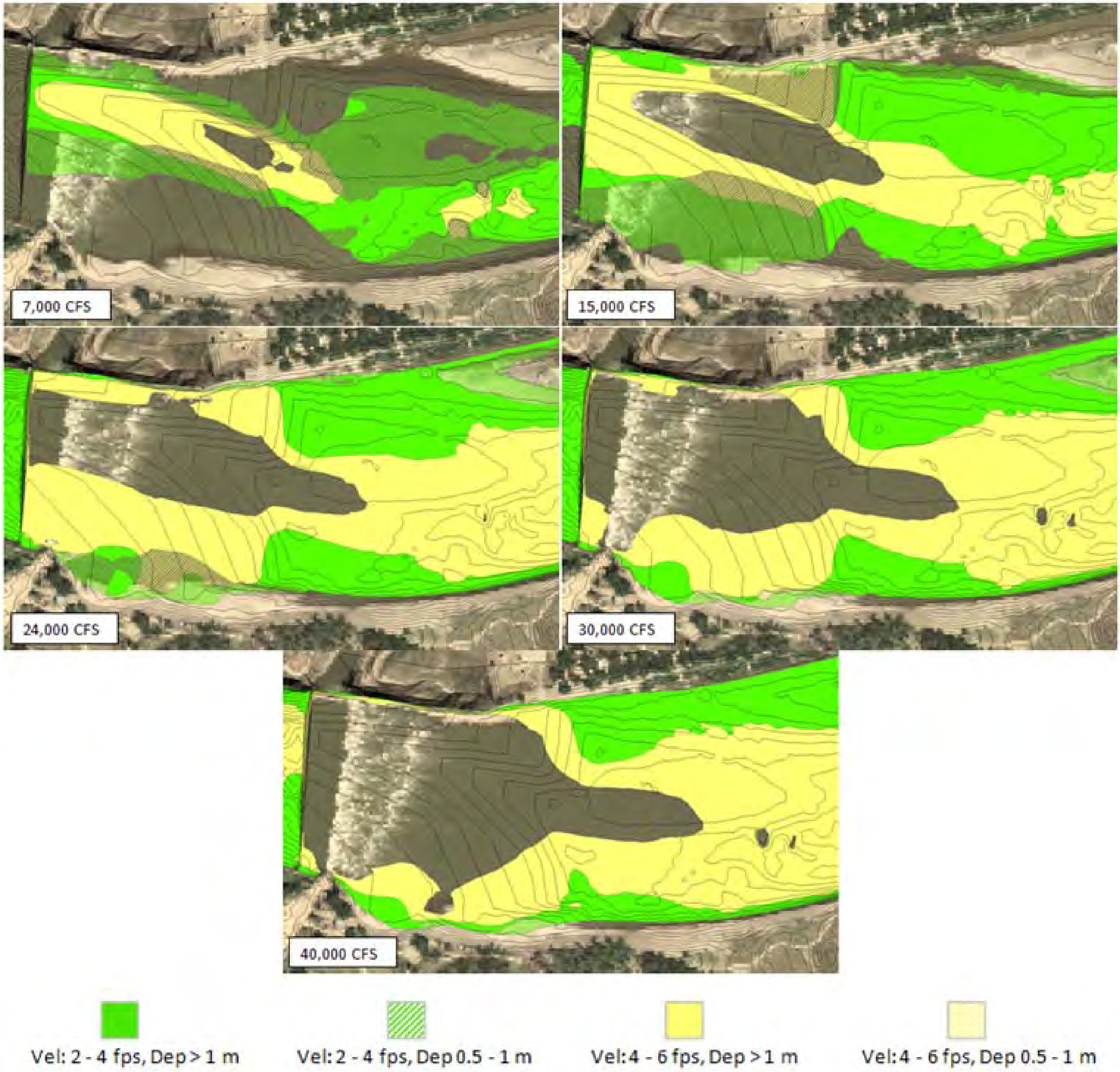

Figure 33 - Doubled Slope Ramp passage delineations.

The colored portions of the figures above indicate areas of the ramp where hydraulic models show results meeting depth and velocity criteria specified by the BRT. The black lines indicate 1' contours of the proposed grading. At the two lower flows, insufficient depths prevent a passage corridor from extending the entire length of the ramp. At higher flows, the 4 fps velocity criteria is only met on the fringe areas of the ramp.

Passageways are available at all flows under the 6 fps criteria, excluding the 40 Kcfs simulation at the crest. This problem could be mitigated through modification of the existing dam crest abutment.

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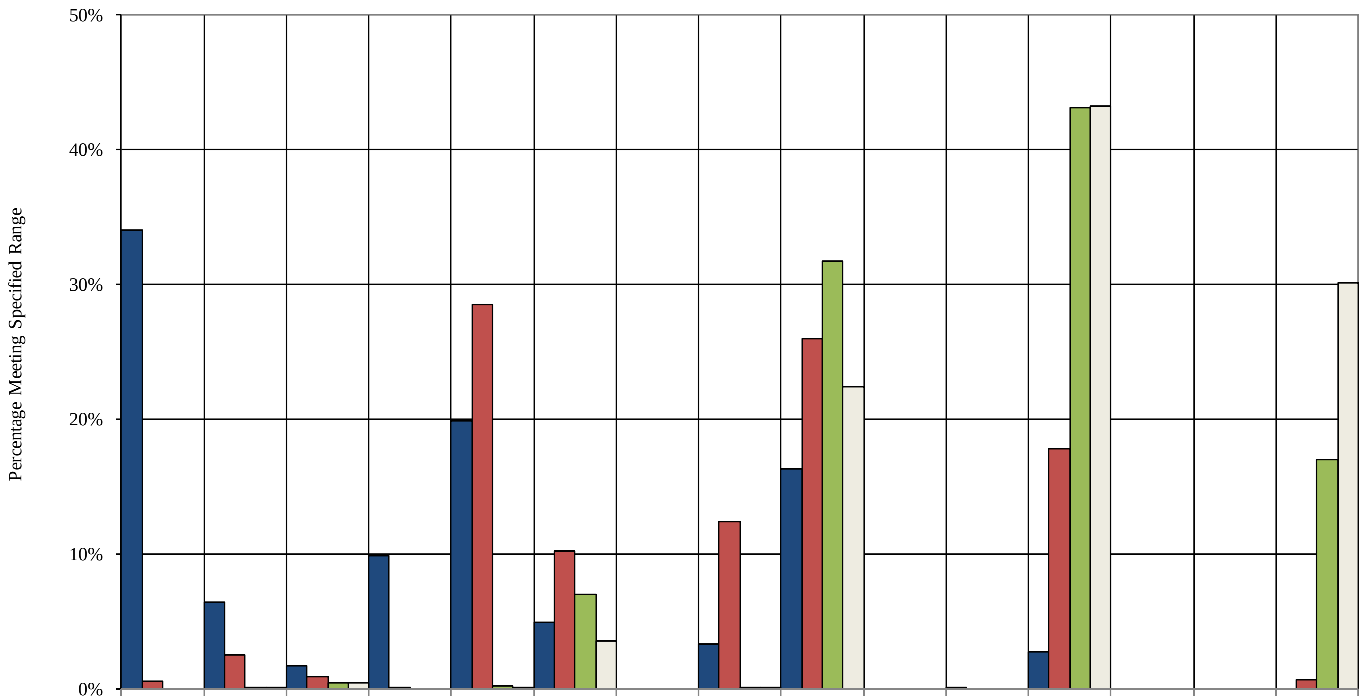

	0-1.64ft	1.64-3.28ft	>3.28ft	0-1.64ft	1.64-3.28ft	>3.28ft	0-1.64ft	1.64-3.28ft	>3.28ft	0-1.64ft	1.64-3.28ft	>3.28ft	0-1.64ft	1.64-3.28ft	>3.28ft
	0-2ft/s	0-2ft/s	0-2ft/s	2-4ft/s	2-4ft/s	2-4ft/s	4-6ft/s	4-6ft/s	4-6ft/s	6-8ft/s	6-8ft/s	6-8ft/s	>8ft/s	>8ft/s	>8ft/s
7k	34%	6%	2%	10%	20%	5%	0%	3%	16%	0%	0%	3%	0%	0%	0%
15k	1%	2%	1%	0%	29%	10%	0%	12%	26%	0%	0%	18%	0%	0%	1%
30k	0%	0%	0%	0%	0%	7%	0%	0%	32%	0%	0%	43%	0%	0%	17%
40k	0%	0%	0%	0%	0%	4%	0%	0%	22%	0%	0%	43%	0%	0%	30%

Doubled Slope Ramp Depth-Averaged Velocity and Depth Ranges
(Ramp 10 + Doubled Gradient)
Total Area 19 Acres

Figure 34 - Doubled Slope Ramp Depth/Velocity Classifications.

This chart displays percent by area classifications of several depth and velocity combinations for the ramp for the set of flows modeled. The predominant flow classification for all flows is depths greater than a meter and velocities in the 4 to 6 fps range for most flows. However, at the 30,000 and 40,000 flow simulations significant portions of the ramp exceed 6 feet per second.

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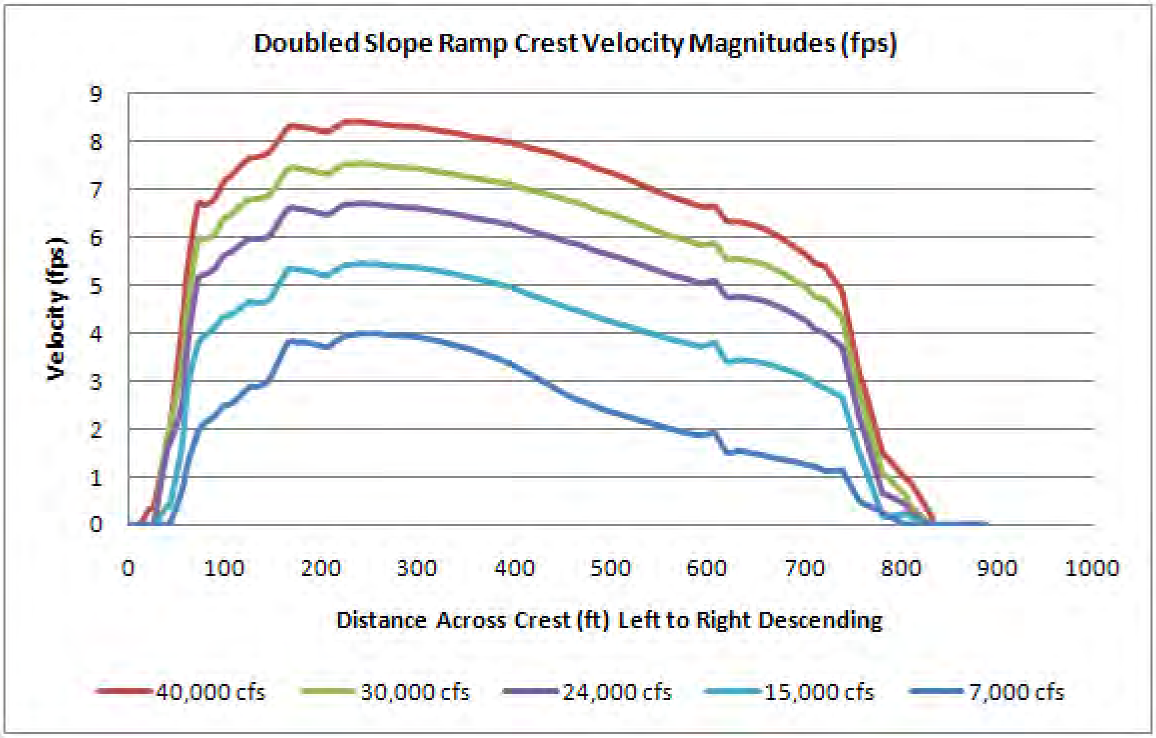

Figure 35 - Doubled Slope Ramp Velocities at the Crest.

This figure is a cross-section taken from near the proposed crest of the damn displaying velocities in fps for a 30,000 cfs flow rate. Numerical modeling indicates that increases of 1-1.75 fps are resultant from the shortened geometry compared to the original proposal.

10 High Flow Bench Ramp Hydraulic Summary

The depth and velocity contours in this section display ADH results for a shortened version of the ramp proposed at Intake. The figure below displays one foot contours of the original ramp (white) and the revised geometry (green). The revised geometry represents the addition of a high flow bench to the doubled slope ramp geometry presented earlier in this document. The crest and thalweg are of the same configuration as the original ramp proposal with the addition of a 100’ wide bench along the left bank, requiring partial removal of the existing dam abutment. The high flow bench is raised two feet from the ramp adjacent to it. The purpose of this revision is to display the effect top width widening has on increasing passage corridors.

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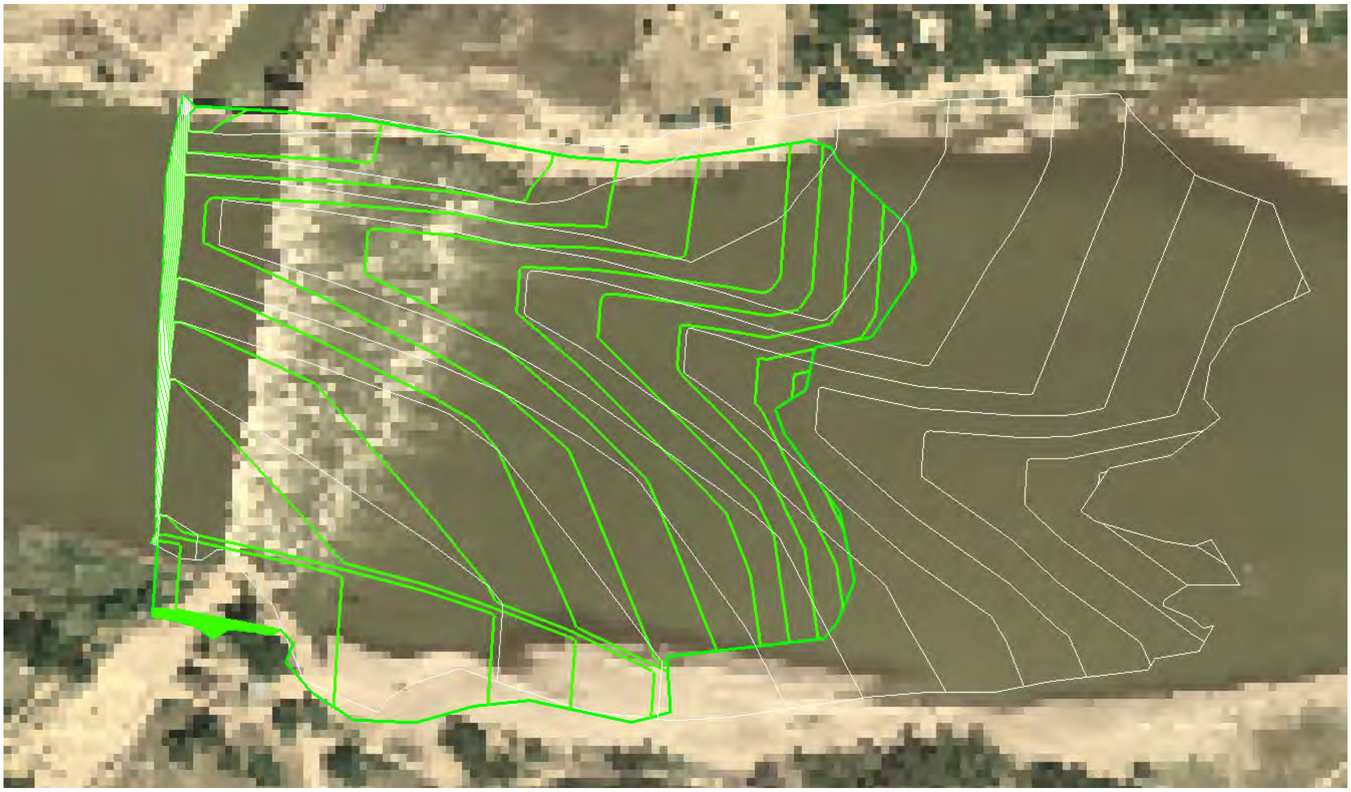

Figure 36 - 1' contours of original and High Flow Bench ramp.

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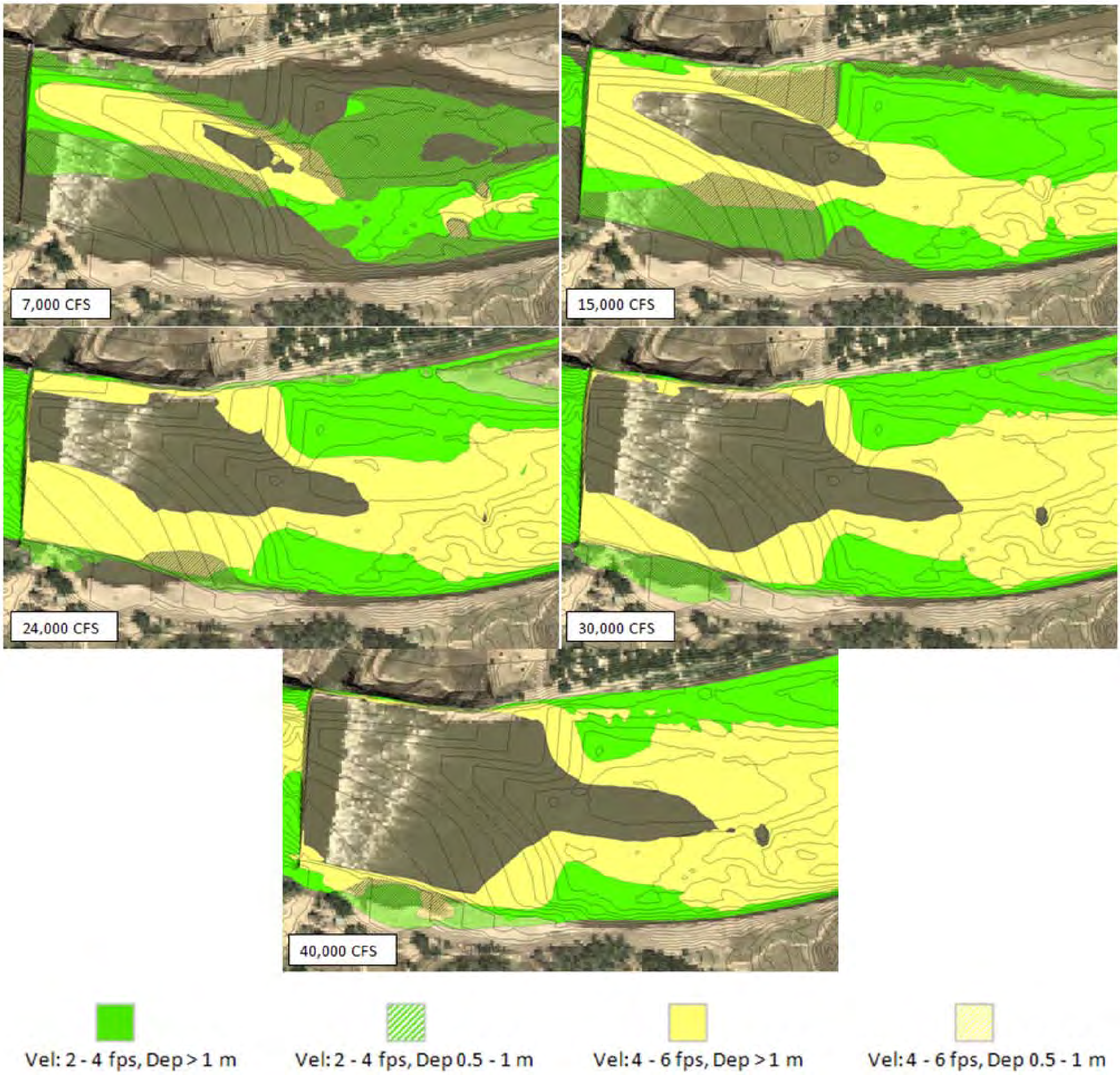

Figure 37 - High Flow Bench Passage Corridors

The colored portions of the figures above indicate areas of the ramp where hydraulic models show results meeting depth and velocity criteria specified by the BRT. The black lines indicate 1' contours of the proposed grading. At the two lower flows, insufficient depths prevent a passage corridor from extending the entire length of the ramp at 4 fps. At higher flows, the 4 fps velocity criteria is only met on the fringe areas of the ramp. Note the amount of area downstream of the ramp not meeting the 4 fps criteria for all simulations.

Passageways are available at all flows under the 6 fps criteria, excluding the 40 Kcfs simulation at the crest. This problem could be mitigated through modification of the existing dam crest abutment.

Inclusion of the high flow depths serves to increase suitable depths on the lower flow simulations and provide areas of suitable velocities during the higher flow simulations. If this option were to be included in a selected geometry, further adjustment to its configuration would be undergone to maximize the feature's utility.

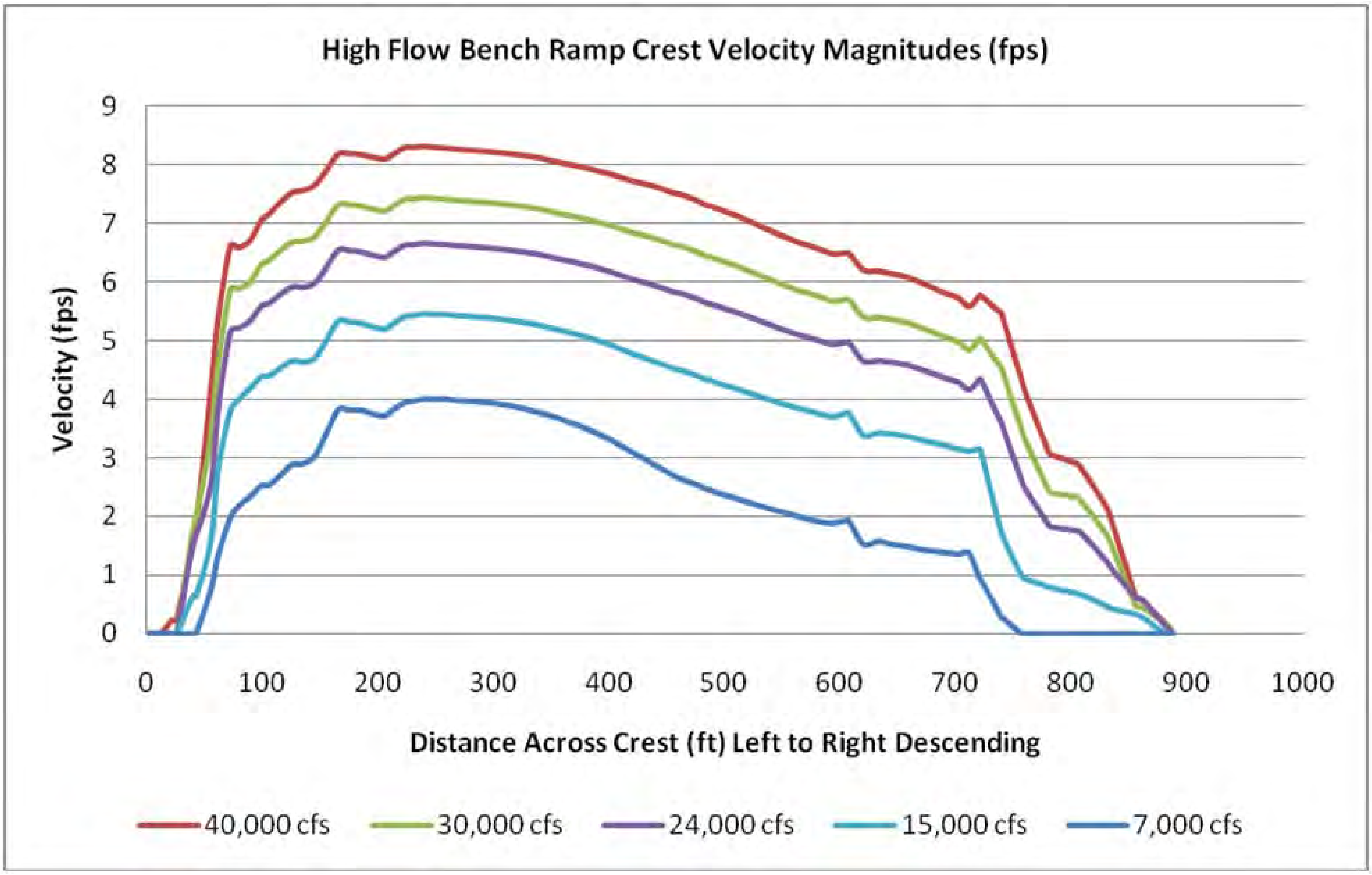

Figure 38 - High Flow Bench Crest Velocities

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Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Intake Fish Passage Option Evaluation Summary
May 2012

Attachment 4

Constructability Overview

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Constructability Overview
May 2012

1. Introduction.

Constructability is an extremely important component of the project which did not receive adequate consideration prior to certification of TPC for the ramp project. The original concept plan for construction of the 1% uniform ramp was for placement of material in the wet. The 2009 Value Engineering study included, as part of the suggestion for reduction of rock layer thickness and use of granular material for subgrade, use of a geotextile filter layer between the rock layer and the subgrade. This approach of filter layer construction is not feasible. As the geometry of the rock ramp becomes more complex, and is tied to such small changes in grade, the quality control component of the project becomes even more important. Quality control is most difficult during construction in the wet.

Three main options for ramp construction have been considered which include:

- The originally proposed construction in the wet
- Diversion of half of the stream using a center channel sheet pile wall with additional cut off sheet pile at each end of the work area to facilitate construction in the dry by halves (cofferdam alternative cost 3)
- Full stream diversion.

Construction in the wet is not considered feasible due to logistics and quality control concerns however, discussion of the approach is included for completeness as that approach formed a significant component of the original conceptual design.

Two options for full stream diversion have been proposed. In one option the Yellowstone flow is diverted via the existing high flow bypass channel which requires construction of an extensive system of levees to constrain flow and to protect the work area (cofferdam alternative cost 1). This option would have to have the diversion removed during the irrigation season to allow for function of the headworks.

The second option involves construction of a shorter bypass channel around the immediate work area which requires extensive excavation (cofferdam alternative cost 2). Both options would require extensive channel protection and grade control.

2. Ramp Construction

2.1 Construction in the Wet

The original concept for construction of the ramp called for placement of rock without dewatering of the site. A portion of the channel was to be isolated by installation of a dividing berm corresponding with the lateral limits of the cofferdam required for the construction of the weir. Stream flow downstream of the cofferdam would be stilled and redirected but water levels would be altered only by changing of flow direction. This method would require blocking-off half of the stream at a time or having to cross flowing water as the weir sections are completed and the cofferdam is removed if smaller cofferdam segments were used.

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An example of a full channel bypass ramp constructed in the wet is the Glen Colusa Gradient Control Structure on the Sacramento River in California. This structure comprises a 0.3% uniform grade ramp that was constructed in the wet by direct placement of rip rap on the existing river substrate using excavators and other heavy equipment working from barges. This structure has had significant maintenance and rework over it's operational life though it should be noted that ramp stability issues have not been documented to have had any adverse impact on fish passage. It should also be noted that the fish passing the Glen Colusa structure have different depth and velocity requirements than the pallid sturgeon.

A primary issue with construction in the wet is grade control and subgrade preparation. All of the ramp designs meeting BRT passage criteria required slopes at least 200 ft of 0.002 ft/ft with additive increases in slope by the same amount in successive segments of the ramp. Attaining a uniform grade at that level of detail for subgrade preparation and effective filter layer placement using an excavator is very unlikely. The existing rock debris field would have to manipulated to allow for proper placement and compaction of fill material, compaction of the material would be extremely difficult. The problem with wet construction becomes more significant as adjacent segments of channel are filled with rip rap which would raise the overall water level or move flows in the channel. At some point it may not be possible for construction equipment to enter the channel due to excessive water depths. It may be necessary to divert some component of the flow out of the river to attain workable water depths.

Another significant issue with construction in the wet is limited access to the site for work. Working benches will need to be constructed and expanded to facilitate access as the ramp is constructed. The number of crews working in the channel and the provision of rock to those crews will be limited by the number of haul roads constructed which in turn will impact the overall construction duration.

Work in the wet is not possible during ice out or periods of high spring flows. Access may also be complicated by pallid presence in the work area. The result of the access issues is a shortened work season beginning late in the summer and proceeding into the winter with slowed productivity during the winter period when work is possible.

There are additional safety concerns working in the wet, especially during the winter months winter when Hyperthermia becomes a significant risk. In addition, when working in turbid water adjacent to deeper portions of the channel there is added risk for substrate failure or operator error resulting in losing the machine in deep water or overturning of the machine and potential drowning of the operator.

2.2 Half stream Diversion

In this construction scenario a half stream cofferdam is constructed upstream of the weir construction similar to construction in the wet. The cofferdam parallel with the new crest structure would consist of granular fill with smaller material near the center and larger material near the outer slopes, and sheet pile driven at the centerline to a depth to cut-off seepage. A center line sheetpile wall would be constructed, probably parallel to the alignment of the natural thalweg. Construction of this structure would involve removing rock from the existing rock debris field, driving sheet pile, replacing the rock and adding additional rock to protect the sheet pile. A granular/ sheet pile cofferdam section would be continued to the river bank to complete the coffered area. As with construction in the wet, isolation of the work area would need to be completed if pallid sturgeon are present in the work area. Once the sheet pile is installed the site would be dewatered and seepage water would need to be continually pumped.

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Construction quality control would be significantly better under dry conditions. Based on currently available knowledge about the substrate material it is likely to be granular and capable of supporting tracked equipment which would facilitate rapid grading of large areas. Use of a geotextile filter layer would be feasible if cost effective. Rapid placement of the subgrade would provide more area for placement excavators to work during rock placement and more avenues for haul trucks to utilize to provide rock to those excavators. Overall productivity would be significantly improved as long as rock supply to the site was maintained.

Negative considerations for this approach include the added capital cost of the sheet pile, sheet pile installation and removal costs, and care of water costs. It is possible that high flows may also overtop the cofferdam at times which could periodically impact work within the dewatered area. The upstream and downstream portions of the cofferdam would have to be removed prior to ice out to allow for conveyance of ice out flows and high spring flows. It is very likely that a portion of the centerline sheet pile could be damaged and require replacement due to ice impacts. Quality control of portions of the ramp constructed adjacent to the sheetpile prior to dewatering and as the sheet pile is removed would be limited since that construction would be completed in the wet. It is likely that the buttress material next to the sheet pile may provide the only avenue for wall removal access which would impede removal efficiency and my pose safety risks working in deeper portions of a flooded stream channel with a narrow travel corridor. It is possible that a diversion of some portion of the flow could be required during work on the second half of the ramp due to channel capacity being taken up by the ramp material.

2.3 **Full Stream Diversion**

In this scenario a full channel width cofferdam is constructed in a location which facilitates utilization of the headworks structure for irrigation purposes for the duration of construction. As in the other two construction options it requires isolation of the work area prior to the arrival of pallid sturgeon. Once site controls are installed, however, they do not need to be removed until all ramp construction is complete and no breaks in the construction season would be required.

Under this option the entire ramp footprint would be accessible and with sufficient pre-staging of rock and other construction material on both sides of the channel, equipment utilization and construction management can be optimized. With the large open area grade control can more easily be maintained over the entire ramp than with the other two options. Other than placement and removal of the upstream cofferdam risks of working in or around water, especially during winter months, is significantly reduced or eliminated.

The significant negative aspect to this construction approach is the cost and technical implications of diverting the entire flow of the Yellowstone river around the work site which combine to make this option infeasible

3. Bypass Channel Construction

Construction of a bypass channel is much more straightforward than construction of a rock ramp. A significant portion of the work effort is the soil excavation to create the main channel. The alignment of the fish bypass channel exit near the existing diversion dam will require the tramway tower and support facility to be removed and relocated for preservation to meet SHPO requirements. Existing rock piles near the existing diversion dam abutment area will be utilized in the project.

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The lothologic profile in the excavation alignment comprises cohesive to non-cohesive fine grained soils(silt and fine sand with some clay) in the upper part of the excavation and coarse grained (cobble with gravel to fine sand matrix, often bimodal) non-cohesive soils in the lower part of the profile. Depositionally the deposts appear to represent a point bar deposit which has been overlain by finer grained overbank or similar deposits. Ground water is encountered at shallow depths in most areas of the bypass channel excavation as determined by test pits excavated during the summer of 2012. Given the coarse nature of the deeper sediments and the close proximity to the Yellowstone River influx of ground water was rapid in most of the test pits.

Removal of the upper fine grained unit may be facilitated by scrapers though it is likely that pumping of the groundwater from the underlying cobble and gravel layer would impede traction and the effective use of scrapers. The coarse grained layers have no cohesion and when excavated the side walls of the excavation suffer slope failure under the influence of groundwater seepage pressures until an angle of reposed has been achieved.

Initially, a small pilot ditch could be excavated along the entire length of the new channel alignment, or along part of it, to facilitate excavation drainage as long as it didn't adversely impact truck traffic and production. Complete excavation of the northern bank profile as part of that ditch and then working the excavation to the south would allow unimpeded traffic flow. Culverted haul roads could also be constructed to facilitate access across the ditch.   A haul road would be left alongside the north bank to facilitate rip rap placement, which would need to be complete before final access across the new channel was cut off.

The excavation for the rock structures would be performed prior to excavation of the main channel. Sheetpile cofferedams would be used to facilitate dewatering with water being pumped to the pilot ditch for conveyance away from the work areas. Limiting the excavated opening will limit the water infiltration. After the rock structure is completed the dewatering for this area can cease.   The remainder of the channel would be excavated by backhoes and off-road articulated haul trucks. To avoid instability and heaving of the channel bottom during the excavation process, the pilot channel would be deepened and groundwater allowed to equalize and the excavation process continue.

4 **Cofferdams.** The work within the new channel will be protected by a cofferdam at the upstream entrance and the downstream exit. These two cofferdams will be constructed early in the construction. The upstream cofferdam will consist of sheet pile driven below grade into the large alluvial material to prevent underseepage. The zone of the cofferdam will be large riprap on both the upstream and downstream with a 20' wide crest and 1V on 2H side slopes (help resist ice forces).   The cofferdam at the downstream exit will be lower in height because it will be below the existing diversion dam, it will be a similar cross section but most of the cross section will be cohesive material. Some of the rock placement on the new channel side slopes will be placed after the cofferdams are removed. The rock for the entrance, exit, vertical grade control, and horizontal control

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structures would come from either commercial sources in Wyoming or South Dakota, or development of a quarry near the site, or a combination of both.

5 New Crest Dam. To maintain the proper head for the operation of the new headworks the existing dam will either need to be modified or replaced. If replaced, the new diversion dam would consist of two lines of z-section sheet pile. The new dam will be located upstream of the existing dam by a distance of approximately 40 feet. The sheet pile will be driven in the river bottom which a fairly uniform strata of shales, claystones, siltstones, and the top elevation several feet above the river water level and will act as it's own cofferdam. This will allow the river to continue to flow during the construction process. Sheet pile rectangular cells will be created by t-sheets perpendicular to the main sheets. Rock will be placed on both sides of the structure to add stability to the sheetpile walls. There should be little infiltration or seepage of water into the sheet piles cells after dewatering pumping due to the impermeability of the river substrata. The cells will be pumped full of concrete. The sheet pile will be pulled if possible or cut off several feet the design crest elevation to avoid impact of the metal with electroreceptivity of some of the native fish. It is anticipated the cells would be completed in minimum lengths of 40 feet. The work can be performed from both banks of the river, and by using anchored barges.

An access road from the north bank is anticipated to be constructed along the existing rock ramp which would be required to be removed after construction is complete. The area between the new and existing diversion dams will be filled with granular sands, and gravels excavated from the bypass channel alignment, which would be capped with a riprap layer. This placement can be performed by hauling and dumping on the completed surface and worked from the south bank. The upstream face of the new diversion dam will be protected with the excavated granular material and capped with riprap on a 1V on 3H slope. That material would most likely be placed by barge. A barge inlet is planned to be excavated on the south bank upstream of the diversion dam. The inlet will be used to launch the barge(s) and to dock and load the barges during the construction duration.

Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Intake Fish Passage Option Evaluation Summary
May 2012

Attachment 5

Ramp Coffer Dam Concept Evaluation

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US Army Corps of Engineers
Omaha District

Project:	Intake Dam Modifications					Sheet No.	2/19
Subject:	Ramp Coffer Dam Concept Evaluation						
Computed by:	CJM	Date:	DEC2010	Checked by:		Date:	

1. PURPOSE

The purpose of this document is to summarize the results of a concept level analysis conducted to evaluate major features of various ramp coffer dam scenarios along with associated hydraulic impacts.

2. INTRODUCTION

A number of construction techniques and phasing methods are possible for construction of the dam crest and rock ramp. This document summarizes the major features required for several proposed methods.

NOTE: Minimal analysis has been completed on the various alternatives. The information contained herein is intended to allow for a comparison between the various construction methods, but values given are subject to change based on a more detailed evaluation.

Floodplain impacts and increased velocities are a concern for many of the possible situations. This document summarizes a concept level analysis conducted to evaluate the hydraulic conditions resulting from several of the methods.

Each of the methods described below allow for construction of the rock ramp in the dry. It was determined that adequate quality control would not be feasible otherwise.

3. ANALYSIS

This analysis was conducted using the hydraulic model HEC-RAS version 4.1.0. The HEC-RAS model is a one-dimensional hydraulic model that was developed to calculate water surface profiles for a uniform, steady state flow by the standard step method. The standard step method computational procedure is based on the solution of the one-dimensional energy equation and friction loss evaluated with Manning's equation.

Because HEC-RAS is a one-dimensional model, the velocities and velocity increases/reductions given in this document are based on cross sectional averages. Localized velocities cannot be evaluated with this model. Additionally, the water surface elevations presented herein are assumed to be at the same elevation across the river. Despite these limitations, the relative increase/decrease in water surface elevation and velocity are useful in providing a general idea of the change in hydraulic conditions for the various scenarios. The HEC-RAS model can be used to rapidly evaluate a large number of alternatives. Note that certain simplifying assumptions were made in the HEC-RAS model in order to evaluate all the various alternatives.

The construction conditions evaluated include the following:

1. Original Conditions (prior to headworks coffer dam (HCD) construction)
2. With HCD
3. With HCD and south half of ramp coffer dam (RCD)

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4. With south half of RCD, HCD removed
5. With HCD and south third of RCD
6. With south third of RCD, HCD removed
7. With HCD and south fourth of RCD
8. With south fourth of RCD, HCD removed
9. Coffer dam blocking entire river near location of new headworks with excavated bypass channel
10. Coffer dam blocking entire river near upstream end of Joe's island (near upstream end of right bank high flow channel)
11. South half of ramp complete, north half of RCD in place
12. Coffer dam blocking entire river near location of new headworks with only a pilot channel excavated, remaining flow allowed to spread out over the island

Concept evaluation of conditions 10-12 are presented in this report.

Conditions 2-8 and 11 were compared against original conditions to evaluate increases in energy grade elevation and increases in channel velocities. The comparison of energy grade is reflective of the change in water surface elevation, but provides a better idea of the actual impact without accenting limitations of the one-dimensional model. The energy grades and channel velocities for the 2-yr, 10-yr, 50-yr, and 100-yr events were computed using the HEC-RAS model.

In addition to comparing the various conditions head-on, a second comparison was made assuming a construction season limited to August through February. Assuming construction is limited to Aug-Feb avoids the spring flood as well as ice-out conditions. The USACE Omaha Hydrology Section developed seasonal discharge-frequency curves for Aug-Feb and Aug-Mar. The Aug-Feb discharge frequency values are compared against the annual values in Table 1.

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Table 1 Flow Frequency

Percent Chance Exceedance	Return Period (years)	Seasonal: Aug-Feb (cfs)	Annual-Post Yellowtail Dam (cfs)
0.2	500	128,507	114,000
0.5	200	96,637	105,000
1	100	77,223	97,200
2	50	61,117	89,400
5	20	43,967	78,700
10	10	33,515	70,100
20	5	24,764	60,600
50	2	14,982	45,300
80	1.25	9,961	33,300
90	1.11	8,334	28,200
95	1.05	7,314	24,500
99	1.01	5,949	18,600

Flow exceedance by duration values are given in Table 2.

Table 2 Flow-Duration Values

	Discharge (cfs)												
Percent Time Flow Equaled or Exceeded	Annual	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
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
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
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
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
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
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
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

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Full River Width Coffer Dams

Cursory analyses on full river coffer dams (conditions 10 and 12) have been completed. Available information is presented below.

Condition 10 Upstream Full River Coffer Dam (Coffer Dam 1)

Based on the available topographic data, it appears that the highest reasonable tie-off elevation on both banks without extensive levees is about elevation 2006 ft NAVD88. A coffer dam at this elevation could reasonably be tied off on both banks near the upstream end of Joe's Island near the right bank chute split (≈ RS 37500). The entire Yellowstone River would then be diverted in to the right bank chute. Figure 1 shows the elevation 2006 contours on both banks and potential coffer dam alignment. Figure 2 shows the Yellowstone River cross sectional geometry just downstream from the potential coffer dam alignment. Note that Figure 2 only extends to the left bank. The tie-off "levee" from the point where the coffer dam alignment turns to the southwest would only be approximately 2-4 feet high except where it crosses the left bank high flow channel where it would be approximately 6 ft high.

Based on the stage-discharge rating curve at the upper end of the right bank chute, a discharge of approximately 8000cfs can be conveyed by the chute before a coffer dam at elevation 2006 would be overtopped.. Therefore, during a 2-year event (based on the Aug-Feb seasonal discharges) a little more than half of the flow would be diverted into the right bank chute before overtopping or flanking the coffer dam. A flow of 8000cfs has greater than 90% annual chance of exceedance. As shown in Table 2, the 50% exceedance discharge by duration during October and November is 8710 and 8080 cfs, respectively. The high risk of coffer dam overtopping associated with the 2006 top elevation is assumed to be unacceptable. Figures 1 and 2 are shown for informational purposes only.

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Figure 1 Potential Upstream Coffer Dam Alignment

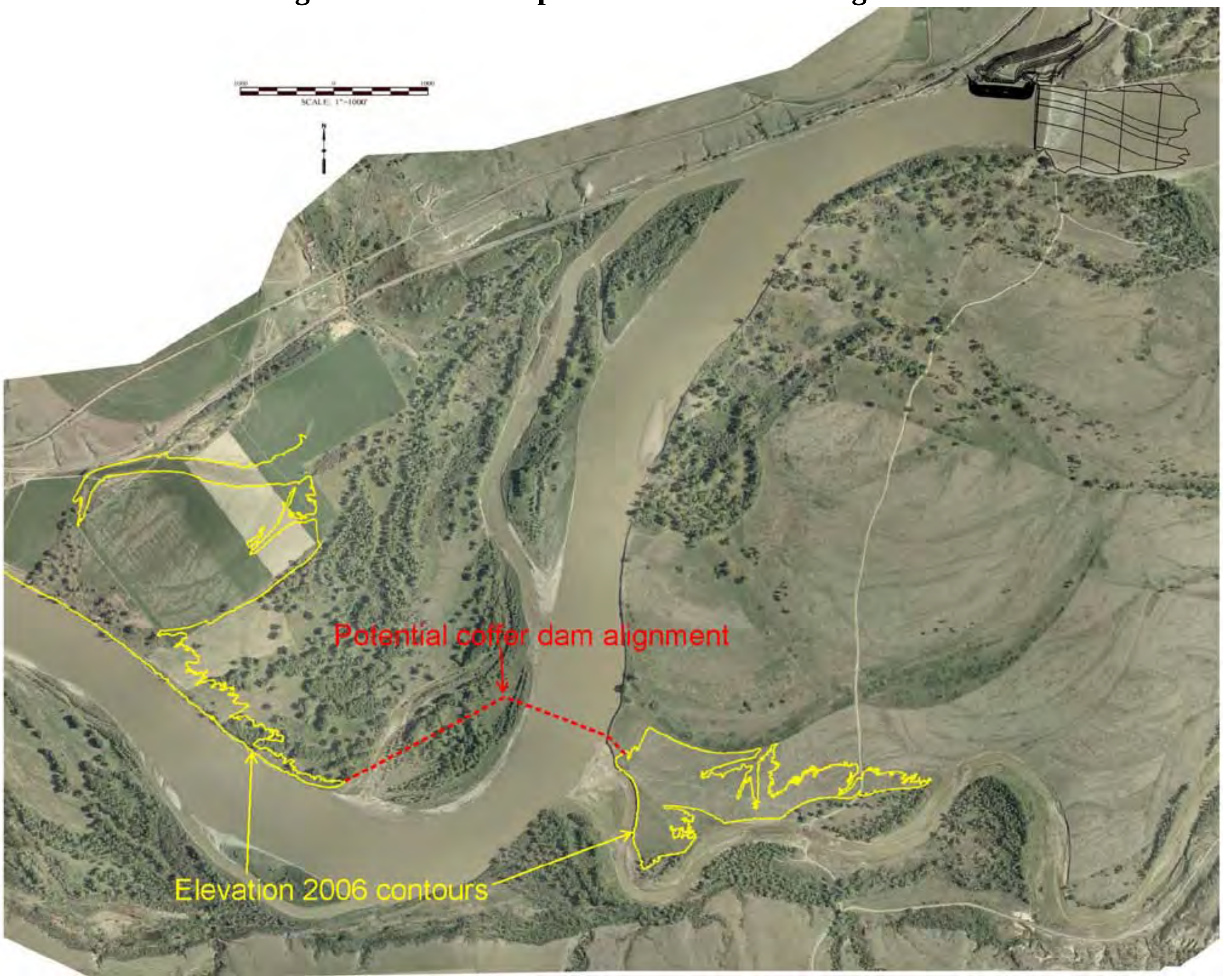

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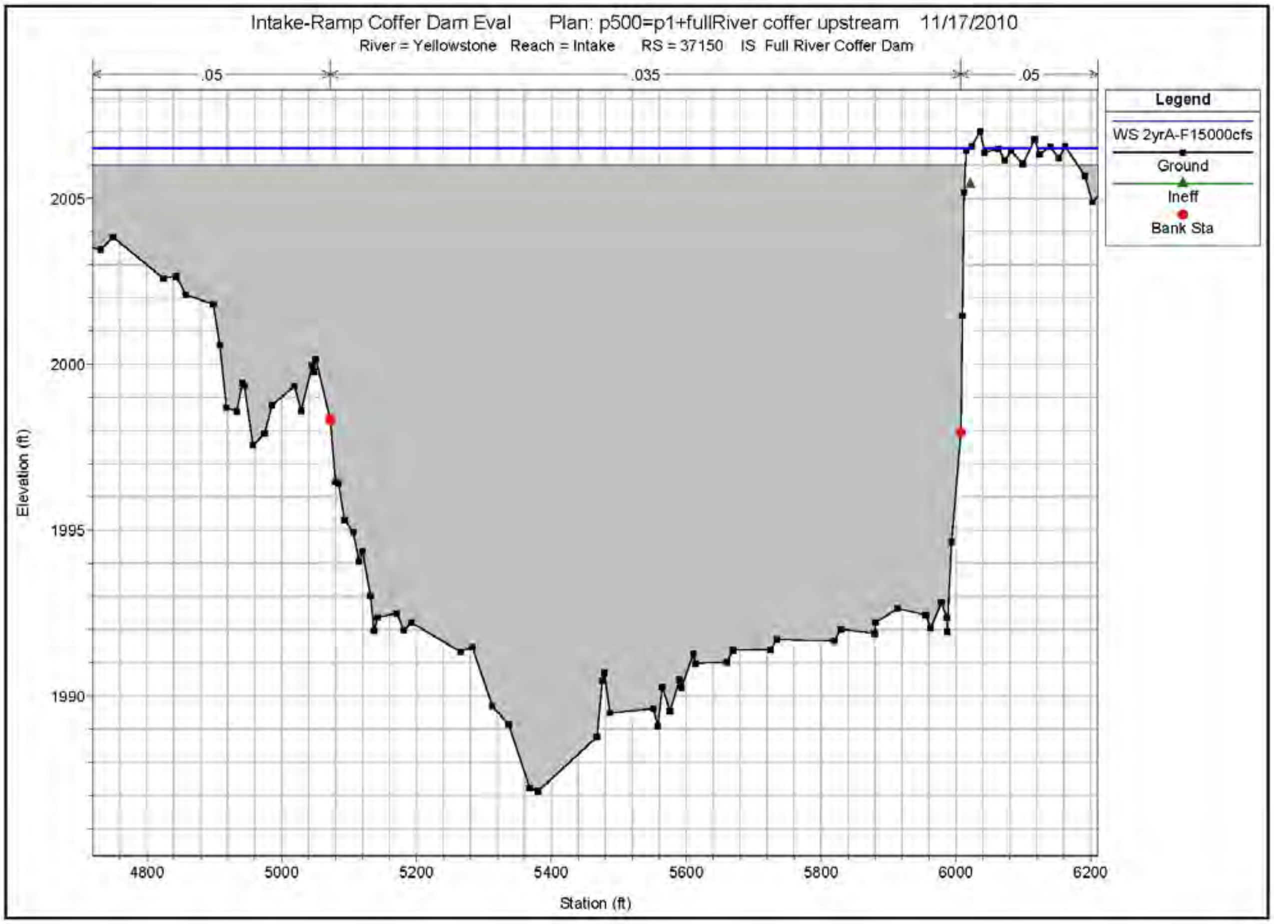
Figure 2 Upstream Coffer Dam Cross Section



Another option for the upstream coffer dam is to raise the top elevation and construct tie-off levees upstream and downstream in order to prevent flanking. Two options for the downstream tie-off levee were evaluated: one that follows the chute and one that extends from the coffer dam directly to the downstream side of the proposed ramp footprint (see Figure 3). The advantage of the longer levee option is that it requires stabilization at only one location to bring flow back into the main channel. However, the shorter levee option results in lower levee/coffer dam elevations and is the preferred alternative.

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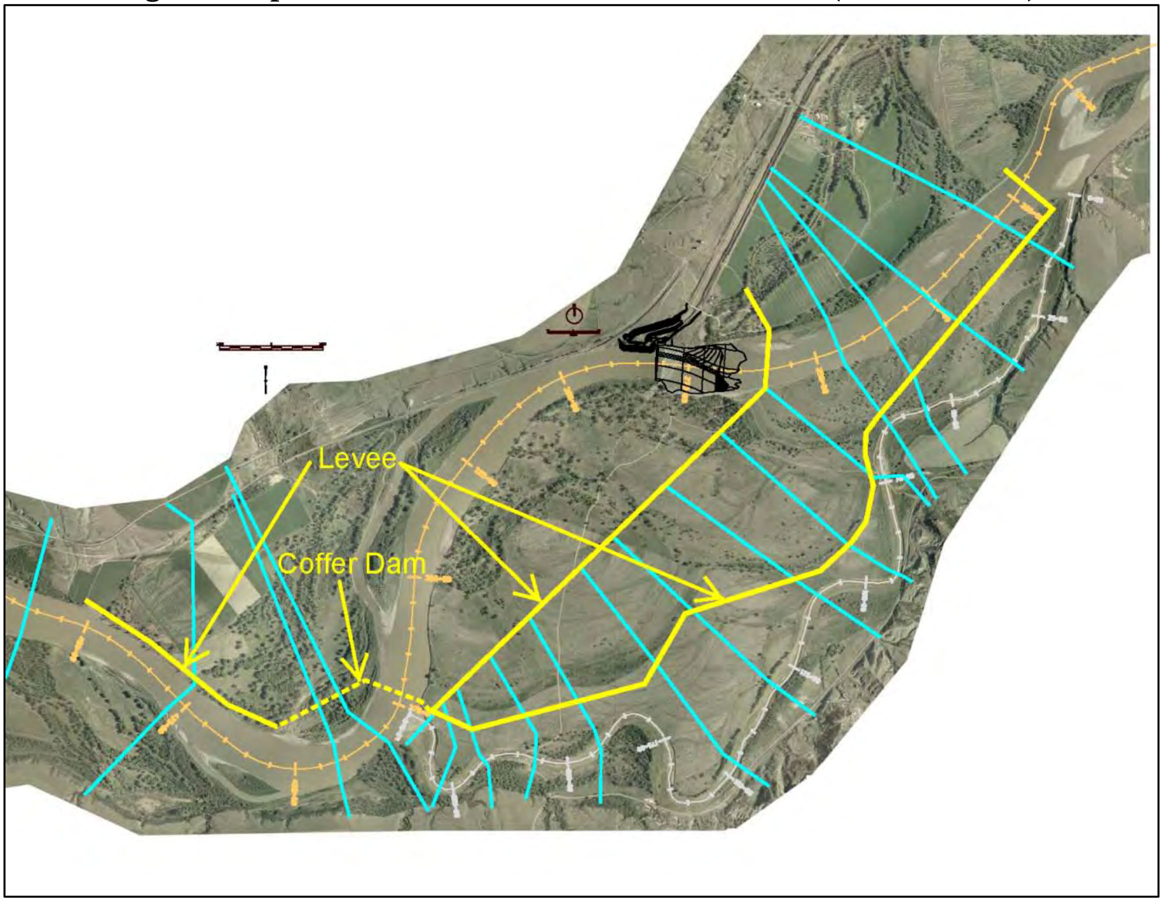
Figure 3 Upstream Coffer Dam with Tie-off Levees (Coffer Dam 1)



Using a discharge of 33,500cfs (10-year Aug-Feb flow), the top of coffer dam would need to be at or above approximately 2011.3 ft NAVD88 based on the computed water surface elevation (i.e. no freeboard). The 10-year profile is approximately 3.4 feet above the 2-year (Aug-Feb) profile. Thus, a zero freeboard 10-year levee is equivalent to a 3.4 feet freeboard 2-year levee height. The 50-year profile is approximately 2.7 feet above the 10-year profile.

The 33,500 cfs profile results in a coffer dam with a maximum height of approximately 24 ft in the Yellowstone River thalweg. The levee would extend upstream approximately 6500 ft to tie off with natural ground at elevation 2012 (not accounting for freeboard). Downstream, the levee would extend from the coffer dam directly to the downstream end of the proposed ramp footprint, then across the main channel to prevent backwater from entering the project area. The total levee length would be approximately 18,000 ft in addition to the approximately 950 ft of sheetpile coffer dam. A cellular coffer dam or other method may be required due to the height (i.e. a single row of sheet pile may not be adequate). Maximum upstream levee height would be approximately 14 ft (on the left bank upstream, crossing the left high flow chute). The average levee height would be in

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the 5-8 ft range. At the downstream tie-off, the levee would cross the main channel and would reach a maximum height of approximately 18 ft. Water surface elevations and approximate ground elevations at each cross section are shown on Plate 1.

While velocities against most of the levee are expected to be relatively low, some sort of erosion protection would likely be required. At a minimum, the lower portion of the levee should be wrapped with a geomembrane or similar erosion protection material.

For either coffer dam top elevation, erosion protection would be required at the downstream end of the high flow channel to bring the diverted flow back into the main channel. Minimal analysis has been completed on the required protection, but at a minimum, a rock riprap grade control structure would be required. The following grade control structure dimensions can be used for estimating purposes.

- Width ≈ 500 ft (approximately twice the 2-yr top width)
- Length ≈ 75 ft (based on ≈ 5 ft elevation difference, assume transition at 1V:15H)

For the coffer dam with tie-off levee at the downstream end of the proposed ramp location, additional erosion protection would be required between the tie-off levee and the downstream end of the chute where flow cascades over the bank and enters the Yellowstone River main channel. There are two locations that are depressed relative to surrounding ground. It is assumed flow would concentrate in these two locations and bank armoring would be required. For estimating purposes, the following configuration can be used for each bank armoring site:

- Width ≈ 600 ft (based on approximate width of depressions)
- Elevation difference ≈ 16 ft
- Slope length ≈ 100ft (based on 6:1)

To account for uncertainties in flow concentration locations and because this is a concept level evaluation, it is recommended that the volume of rock computed for the bank armoring be doubled. The additional rock could be stockpiled on site and placed as needed depending on flow conditions.

Additional rock will be required on the levee face at the location of the upstream bank armoring site as it is anticipated that flow will concentrate in the depression near the levee as it flows into the main channel. Rock should also be placed on the levee face as it crosses the main channel as this is one location where flow will be returning.

Rock size for the scour hole and for the bank armoring can be assumed to be similar to that used in the rock ramp. While this rock would likely not be able to be used in the ramp, it may be possible to reclaim the rock following construction and stockpile it for use in future ramp repairs or adjustments. Rock erosion protection quantity is summarized as follows:

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Bank Armoring for Yellowstone River

2 sites, each 600’x100’, doubled for concept estimate = 26,800 sq yds

U/S Levee Face

200’x20’=225 sq yds

Main Channel Levee

100’x20’=450 sq yds

Grade Control Structure, D/S end of high flow channel

500’x75’=4200 sq yds

Total Rock Area ≈ 32,000 sq yds

All rock – assume 30 inch D100. For short duration reliability during construction period, 1D100 or 30 inch layer thickness is reasonable.

Figure 4 shows the location of the proposed grade control structure and bank armoring.

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Figure 4 Grade Control and Bank Armoring Locations

Condition 12 Full River Coffer Dam at new Headworks (Coffer Dam 2)

The concept of condition 12 is to build a full river width coffer dam between the ramp crest and new headworks. A small bypass channel would be excavated to carry low flows around the ramp footprint, and higher flows would be allowed to spread out over the island. A levee that ties into the coffer dam on the upstream side and wraps around the proposed ramp footprint, preventing backwater from entering the work area, would be required. Similar to the full coffer dam upstream, erosion protection features would be required to convey water back into the main channel. A general overview of the concept is presented in Figure 5.

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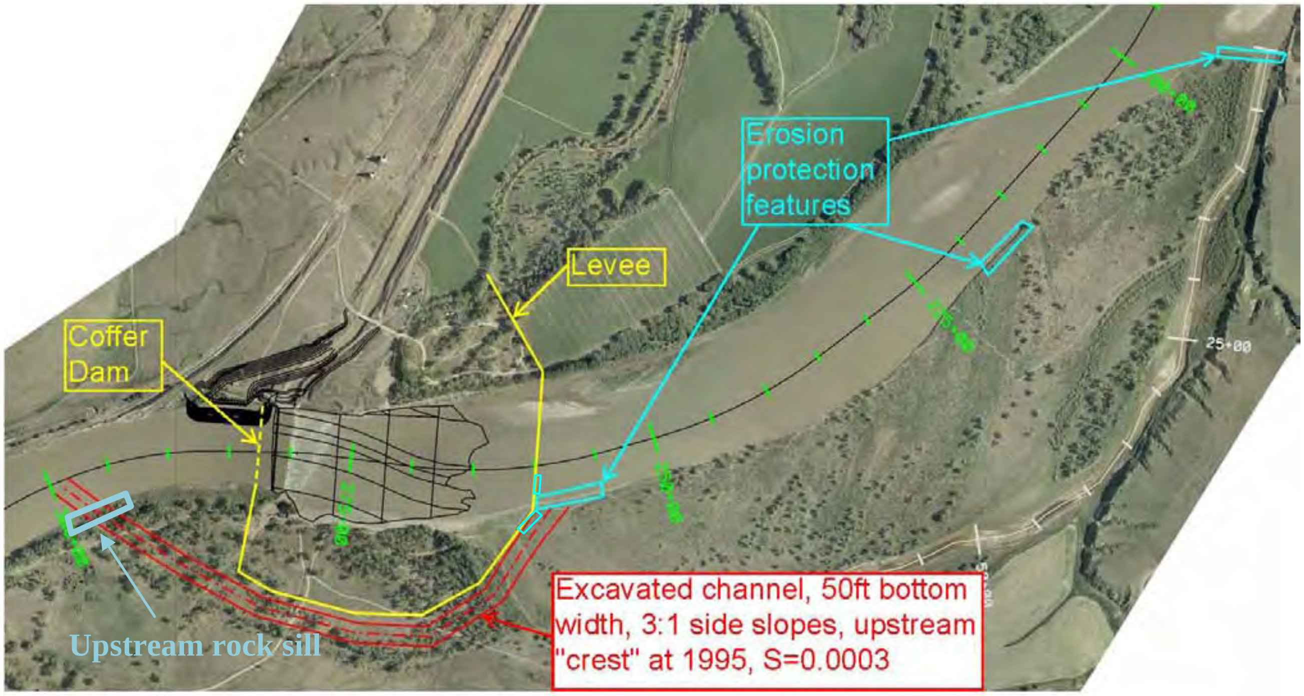
Figure 5 Full River Coffer Dam at New Headworks-Overview (Coffer Dam 2)



Major features included in this concept are summarized below:

- Coffer dam
  - Length ≈ 700ft
  - Water surface for 2-yr Aug-Feb discharge = 2002.1
  - Water surface for 10-yr Aug-Feb discharge = 2003.9
  - Water surface for 100-yr Aug-Feb discharge = 2006.6
  - Maximum height ≈ 26ft (based on 10yr water surface, no freeboard)
- Earthen levee
  - Length ≈ 5600 ft
  - Maximum height ≈ 20 ft for 10-yr water surface (in main channel thalweg downstream)
  - Average height range for 10-yr, not including freeboard = 4-6ft
  - Profile range – 10-yr exceeds 2-yr by about 1.8 feet along the levee
- Excavated channel
  - Length ≈ 4500ft
  - Bottom width = 50ft
  - Side slopes = 3H:1V
  - Channel slope = 0.0003 ft/ft
  - Upstream "crest" invert elevation = 1995
  - Average depths of excavation ≈ 3-7 ft
- Erosion protection
  - Upstream rock sill crest

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- Width ≈ 500ft
- Length (in direction of flow) ≈ 150 ft
- Rock area ≈ 8400 sq yds
- Rock size similar to that used in ramp, 30 in D100

- Grade control structure at downstream end of high flow chute
  - For concept level estimating purposes, assume similar to that described in upstream full river coffer dam alternative, total rock of 4200 sq yds
- Bank armoring
  - For concept level estimating purposes, assume similar to that described in upstream full river coffer dam alternative, total rock of 26,800 sq yds
  - Note that the downstream end of the excavated channel will require additional rock due to the significant drop from excavated invert to main channel invert. A large drop in water surface and high velocities are expected. Rock protection area estimated as 225'x500' = 13,000 sq yds (based on 15ft drop at 1V:15H and top width of 10-yr)
- All erosion protection rock = 52,400 sq yds. Assume 30 inch D100. Layer thickness of 1xD100 suitable for short duration construction period

Water surface elevations for the 10-year Aug-Feb discharge are shown in Figure 6.

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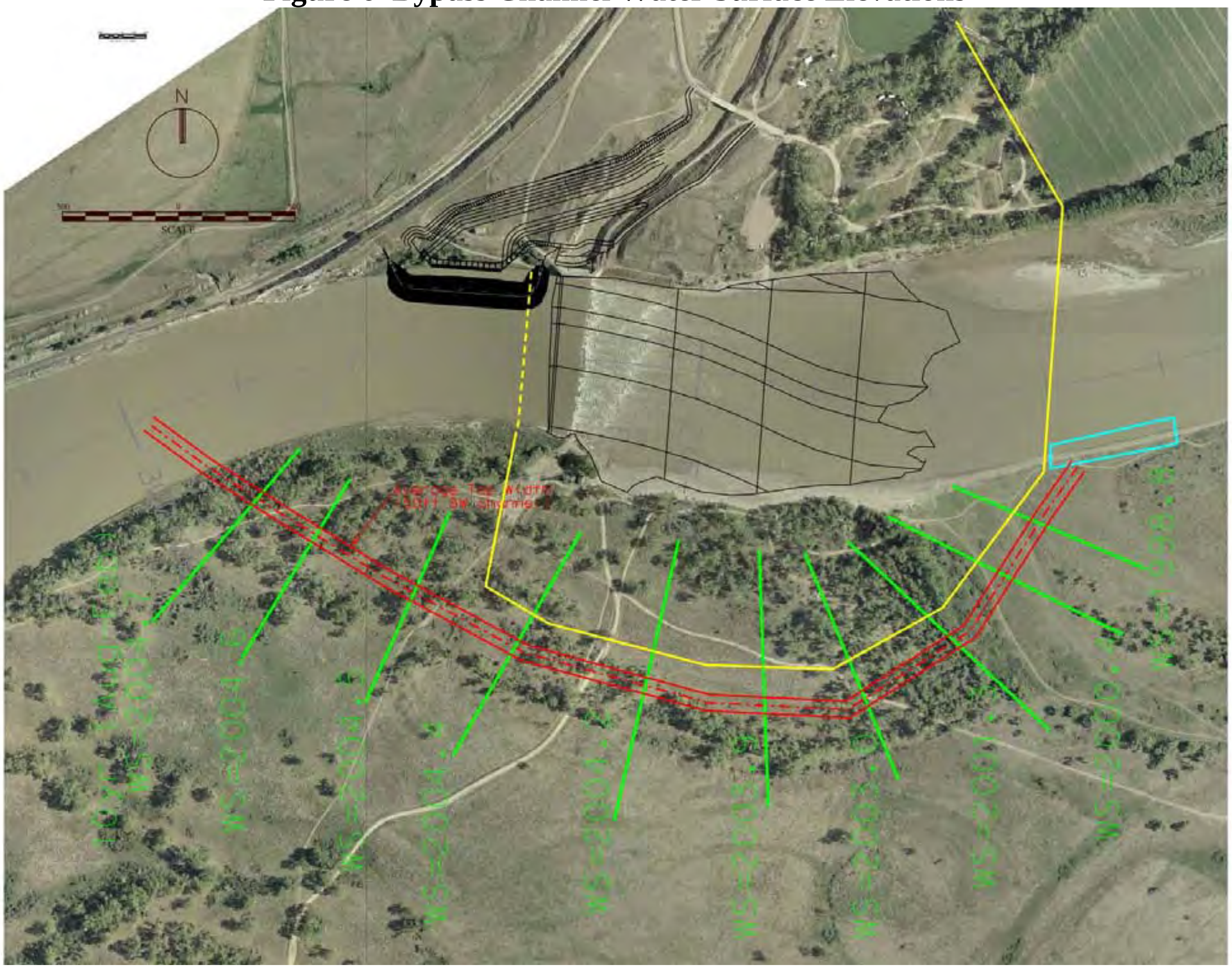
Figure 6 Bypass Channel Water Surface Elevations



Partial River Coffer Dams (Coffer Dam 3)
Cursory evaluations were conducted on a number of partial river coffer dam alternatives. It was then determined that construction issues would prevent constructing the ramp in thirds or fourths. This section evaluates construction of the ramp in halves.

For this evaluation, construction sequencing was assumed to take place in two phases. Phase 1 would include a coffer dam blocking off the north half of the ramp work area. This phase would include construction of the ramp thalweg. Phase 2 would coffer off the south half of the ramp with all flow going over the completed north half of the ramp. The portion of the coffer dam extending longitudinally down the middle of the river was assumed to start in the middle on the upstream end, extend downstream to where it intersects the thalweg rock layer thickness boundary, then continue to the downstream end of the ramp at the edge of the thalweg rock layer thickness boundary. The assumed coffer dam location is shown in Figure 7.

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Figure 7 Half Ramp Coffer Dam-General Plan (Coffer Dam 3)

The assumed coffer dam location follows the edge of the thalweg rock layer thickness boundary so as to minimize quality control difficulties associated with crossing the thalweg. While this alignment constricts flow to a relatively narrow conveyance area during the first half of construction, it is nearly offset during the second half of construction when all flow is directed over the north half of the completed ramp (rather than existing conditions river bed).

Both phases were evaluated in the HEC-RAS model. Figure 8 shows the computed water surfaces for both phases as well as the proposed coffer dam elevations. Proposed coffer dam elevations are based on the higher computed water surface elevation between the two phases plus two feet of freeboard. Computations were performed for both the all season and Aug-Feb 10-year profiles.

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Figure 8 Half Coffer Dam Profiles

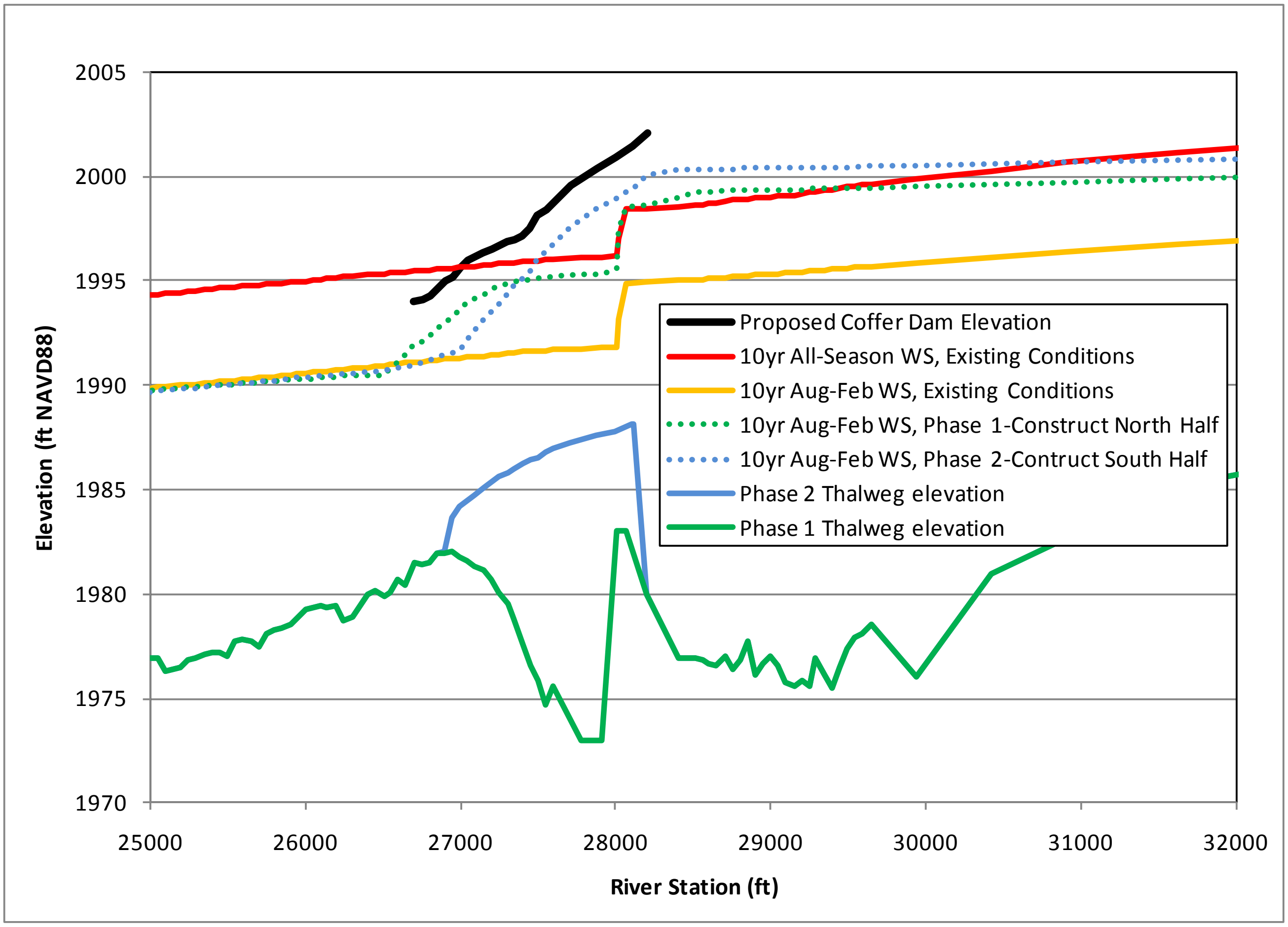

Table 3 compares energy grade elevations and velocities between existing conditions and the two phases.

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Table 3 Half Coffer Dam Energy and Velocity Comparision

	Max increase in energy grade elevation over original conditions (ft)					Average increase in velocity over original conditions (ft/s), RS 26696-28406 (existing dam is RS 28000, proposed crest is RS 28120, d/s end of ramp is RS 26696)			
	2-yr	10-yr	50-yr	100-yr		2-yr	10-yr	50-yr	100-yr
Phase 1-Construct North Half	4.4	5.2	5.3	4.7		3.6	3.3	1.1	1.3
Phase 2-Construct South Half	8.2	8.8	9.1	9.3		5.0	5.7	6.1	6.1
Phase 1-Construct North Half *	-1.1	0.1	2.1	3.5		0.4	1.6	2.7	1.9
Phase 2-Construct South Half *	2.3	3.3	5.7	7.1		1.4	3.1	4.6	5.2

* Comparison in these rows uses all-season discharges for original conditions and August-February discharges for construction

To prevent erosion and potential undermining of the longitudinal coffer dam, rock placement is recommended. Rock with a similar gradation as the ramp rock could be used and placed along the wet side of the coffer dam. It is assumed that the rock used to stabilize the coffer dam could be reclaimed and used in the rock ramp.

Figure 8 shows the cross sections used in the HEC-RAS model and Table 4 gives water surface elevations for several discharges for both Phases.

Figure 8 HEC-RAS Cross Sections

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Stationing of Critical Features:

River Station	Feature
37,300	Upstream Coffer Dam
28,115	New Weir Upstream Crest (U/S of 10 ft section)
28,105	Ramp Rock Start (D/S side of concrete weir)
26,900	Ramp centerline toe along thalweg
26,700	Downstream coffer dam alignment-half coffer
26,050	Downstream coffer dam alignment-full coffer upstream and downstream (further downstream to take advantage of topography for flow returning to the river)

Table 4 Half River Coffer Dam Water Surface Elevations

	Water Surface Elevations (ft NAVD88)						
	Phase 1			Phase 2			
	2-yr (Aug-Feb)	10-yr (Aug-Feb)	100-yr (Aug-Feb)	2-yr (Aug-Feb)	10-yr (Aug-Feb)	100-yr (Aug-Feb)	Concept top of coffer dam elevation
River Station	15,000 cfs	33,500 cfs	77,200 cfs	15,000 cfs	33,500 cfs	77,200 cfs	(ft NAVD88)
28407	1995.4	1999.0	2002.6	1996.3	2000.3	2006.8	
28203	1995.2	1998.6	2002.1	1996.2	2000.1	2006.8	2002.1
28115	1995.2	1998.5	2002.1	1995.9	1999.5	2005.2	2001.5
28105	1991.0	1997.5	2002.1	1995.8	1999.5	2005.2	2001.5
27998	1990.9	1995.6	2002.1	1995.5	1998.9	2004.1	2000.9
27880	1990.9	1995.3	2002.1	1995.2	1998.4	2003.4	2000.4
27706	1990.9	1995.3	2002.0	1994.6	1997.6	2002.3	1999.6
27597	1990.9	1995.2	2001.9	1994.0	1996.8	2001.1	1998.8
27550	1990.8	1995.2	2001.9	1993.8	1996.4	2000.4	1998.4
27498	1990.8	1995.1	2001.9	1993.6	1996.1	2000.1	1998.1
27447	1990.7	1995.0	2001.9	1993.2	1995.5	1998.6	1997.5
27399	1990.7	1995.0	2001.8	1992.9	1995.1	1998.0	1997.1
27348	1990.7	1995.0	2001.8	1992.6	1994.8	1997.7	1997.0
27301	1990.6	1994.9	2001.7	1992.3	1994.4	1997.3	1996.9
27249	1990.5	1994.7	2001.7	1991.9	1993.9	1996.8	1996.7
27199	1990.4	1994.6	2001.5	1991.6	1993.5	1996.5	1996.6
27147	1990.2	1994.3	2001.4	1991.2	1993.1	1996.5	1996.3
27093	1990.1	1994.2	2001.3	1990.8	1992.6	1996.2	1996.2
27045	1989.9	1994.0	2001.2	1990.4	1992.2	1996.2	1996.0
26998	1989.8	1993.7	2001.0	1990.0	1991.7	1996.1	1995.7
26946	1989.5	1993.2	2000.8	1988.8	1991.6	1996.1	1995.2
26900	1989.3	1993.0	2000.6	1988.5	1991.5	1996.1	1995.0
26850	1989.1	1992.7	2000.5	1988.4	1991.3	1996.0	1994.7
26799	1988.8	1992.3	1997.6	1988.3	1991.2	1995.9	1994.3
26751	1988.5	1992.1	1997.5	1988.2	1991.1	1995.8	1994.1
26697	1987.7	1992.0	1995.2	1988.0	1991.0	1995.7	1994.0
26646	1987.7	1991.5	1994.2	1987.9	1990.9	1995.7	

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4. SUMMARY

Three methods of construction phasing were considered and a concept level analysis was completed to lay out major features. The three concepts considered were a full river coffer dam upstream using the existing high flow chute, a full river coffer dam near the proposed ramp crest using a bypass channel, and a two phase half river coffer dam. Results of this cursory analysis are expected to be used to develop a concept level cost estimate for the various methods.

NOTE: Minimal analysis has been completed on the various alternatives. The information contained herein is intended to allow for a comparison between the various construction methods, but values given are subject to change based on a more detailed evaluation.

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Intake Diversion Dam Modification

Lower Yellowstone Project, Montana

Intake Fish Bypass Option Evaluation Summary

May 2012

Updated 6JULY2012

Attachment 6

Bypass Channel Hydraulics and Sediment

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1. INTRODUCTION

Analysis was completed to evaluate a bypass channel for fish passage around the Intake Dam. The concept level analysis completed in April 2011 was used as a starting point. Three concept bypass channels were proposed in April 2011, one each for diversion percentages of 10%, 15%, and 30% of total Yellowstone River flow. Coordination with the U.S. Department of Interior, Bureau of Reclamation (Reclamation) and the Biological Review Team (BRT) led to refinements in the cross sectional shape and alignment of the proposed channel. This document discusses the 30% design of the current proposal consisting of a bypass channel that diverts approximately 15% of total Yellowstone River flow.

Hydraulic modeling was completed with HEC-RAS to evaluate the proposed alignment and channel cross section configuration. In addition to the hydraulic analysis, HEC-RAS was used to evaluate general sediment transport tendencies with various bypass designs. Two main objectives of the sediment modeling were to evaluate bypass channel stability and transport capacity within the main channel of the Yellowstone River.

Sustainability and passage issues with the flow and sediment relationship between the Yellowstone River and the bypass channel may be summarized as:

- Divert too much flow into the bypass channel and the main Yellowstone River channel has sediment deposition with impacts to the irrigation diversion
- Divert too much sediment into the bypass channel and the bypass channel has deposition
- Divert only clear water into the bypass channel causes stability issues with bank/bed erosion
- Fish passage may be compromised with insufficient bypass channel flow depth, attractive flow, or turbulent flow conditions
- Sediment erosion and deposition model results contain a high degree of uncertainty

This Attachment is meant to serve as a general overview of the bypass channel analysis. Additional information is presented in the six appendices to this attachment. Details pertaining to the sediment analyses are presented in Appendices A and B. Appendix C lists and describes the components of the 30% design. Appendix D compares 11 natural side channels on the Yellowstone River with the proposed bypass. Appendix E consists of an evaluation of the characteristics of an armor layer in the bypass channel. Appendix F is from the USGS and describes their sediment sampling efforts on the Yellowstone River near Intake in 2011. Appendix G is an analysis completed by Reclamation pertaining to the bypass channel configuration (channel cross sectional shape, upstream and downstream ends, and flow augmentation structure concept).

2. BYPASS FEATURES

The following proposed features summarize the bypass channel alternative.

2.1 Bypass channel excavation

A bypass channel would be excavated from the inlet of the existing high flow chute to just downstream of the existing dam and rubble field. The proposed alignment is

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approximately 15,500 ft long at a slope of 0.0006 ft/ft (natural Yellowstone River slope in the project area is approximately 0.0004 ft/ft to 0.0007 ft/ft) and excavation is currently estimated at approximately 1.2 million cubic yards. The channel cross section has a 40ft bottom width and side slopes varying from 1V:12H to 1V:3H. The bypass channel would divert approximately 15% of total Yellowstone River flows.

2.2 Upstream control structure

A structure designed to control discharge into the bypass channel would be situated on the upstream end of the channel. The structure would likely be composed of riprap with a concrete sill. The control structure would be backfilled with natural river size rock to give the appearance of a seamless channel invert. The purpose of the structure is to provide stability during extreme events to prevent excessive flow through the bypass.

2.3 Channel plug

A channel "plug" would be constructed approximately 1 mile downstream from the upstream end of the bypass in the existing high flow chute to keep normal flows in the proposed bypass. The channel plug would have a low-level discharge pipe and would be designed for overtopping during larger events to maintain the existing chute's current functionality.

2.4 Riprap at bends for lateral stability

Bank riprap is proposed at two outside bends to minimize the risk of losing the bypass channel planform.

2.5 Vertical control structures

Two vertical control structures (riprap sills) are proposed for maintaining channel slope and allowing for early identification of channel movement. Similar to the upstream control structure, these would be overexcavated and backfilled with natural river size rock to give the appearance of a seamless channel invert while providing stability during extreme events.

2.6 Downstream vertical control structure

A riprap sill is proposed at the downstream end of the channel to maintain channel elevations (similar to vertical control structures).

2.7 Downstream lateral stability structure

A riprap bank stabilization feature would be constructed on the descending right bank of the bypass channel to prevent downstream migration (relative to the Yellowstone River) of the downstream end of the bypass channel.

2.8 New dam

In order to maintain irrigation diversion capabilities without impacting the bypass channel, a new dam is proposed. The new dam would preclude the necessity of adding large rock to the crest of the existing dam to maintain diversion capabilities (as is currently done).

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2.9 Flow augmentation structure

A weir constructed using roller compacted concrete would be constructed near the tie-in between the downstream end of the bypass channel and Yellowstone River. The weir would increase attractive flow in the bypass channel when Yellowstone River discharges are above approximately 7,000cfs. The flow augmentation structure is proposed as a potential future adaptive management technique to increase flow at the downstream end of the bypass channel if monitoring determines additional flow is required for successful passage.

2.10 Armor Layer

Current modeling efforts indicate a degradational trend within the bypass channel. Modeling shows that an increase in size of the bypass bed material minimizes expected degradation; therefore, construction of an armor layer is proposed. The proposed armor layer would be similar to naturally formed armor layers found in the Yellowstone River on bars. The intent would be to minimize bypass channel degradation while providing substrate similar to reaches upstream and downstream from the project.

3. BACKGROUND

Passage of the pallid sturgeon around Intake Dam by means of a bypass channel has been discussed and evaluated for over a decade. This analysis uses best available information along with suggestions from Reclamation and the BRT.

Criteria used to develop and evaluate the alternatives are based on suggestions from the Biological Review Team (BRT). The main criteria used to develop the 30% design alternative pertain to depth and velocity. Similar to previous evaluations of the rock ramp and bypass channel, flow and depth ranges as shown in Table 1 were used based on BRT passage criteria. The target range is velocity less than 4 ft/s and depth greater than 1 meter with scaled passage ability for ranges of 4-6 ft/s and/or 0.5-1.0m.

Table 1 Depth and Velocity Ranges used for Evaluation

Depth Ranges	
(m)	(ft)
0-0.5	0-1.64
0.5-1.0	1.64-3.28
>1.0	>3.28

Velocity Ranges
(ft/sec)
0-2
2-4
4-6
6-8
>8

4. ANALYSIS

The analysis used HEC-RAS version 4.1.0 dated January 2010. A previously created existing conditions model was used as the base model. Using various bypass channel alignments, new cross sections were extracted from a LiDAR based digital terrain model (DTM) using Bentley's Microstation/InRoads software package.

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4.1 General Modeling Information

Three separate HEC-RAS models were used in the overall analysis. First, an inclusive model was created to evaluate flow splits between the main channel of the Yellowstone River and the proposed bypass channel. Second, because HEC-RAS does not have the capability to evaluate sediment transport through flow splits, separate models were created to evaluate sediment in the main channel and the bypass channel.

Sediment transport modeling is notoriously difficult. The data utilized to predict bed change is fundamentally uncertain and the theory employed is empirical and highly sensitive to a wide array of physical variables. Sediment transport measurements often show variations over more than one order of magnitude. This inherent uncertainty in sediment transport is compounded when numerical models are used to simplify natural processes. While HEC-RAS is a useful tool for evaluating sediment transport, the results of the model should not be used as quantitative estimates of scour/deposition or degradation/aggradation. The model can provide useful information pertaining to general trends, but many parameters used in the model have wide ranges of uncertainty and the computed results should be used with caution. Review of model results should consider that this is still preliminary design with detail suitable at this design phase.

The inclusive model evaluated only the hydraulics (i.e. no sediment) and was used to develop channel configurations (length/slope, bottom widths, depths, etc.) for various flow split percentages. A large range of discharges were modeled from extreme low flows (3000cfs) to the 0.2% annual chance of exceedance flood (500-year). The low flow of 3000cfs represents the 90-95% exceeded by duration discharge during the low flow month of August based on gaging records at Sidney and Glendive. Table 2 gives flow-frequency values and Table 3 gives flow-duration values.

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Table 2 Flow Frequency

Percent Chance Exceedance	Return Period (yrs)	Discharges (cfs) for various scenarios. Recommended values areAnnual Post Yellowtail Dam; seasonal values used in evaluation of various construction timelines to lower risk.					
		Seasonal: Aug-Feb	Seasonal: Aug-Mar	Annual (period of record)	**Annual-Post Yellowtail Dam**	Winter (1Jan-15Apr) Post Yellowtail Bulletin 17b	Winter (1Jan-15Apr) Post Yellowtail Top Half
0.2	500	128,507	192,400*	192,400	**114000**	249000	213000
0.5	200	96,637	172,300*	172,300	**105000**		
1	100	77,223	148,907	156,900	**97200**	128000	123000
2	50	61,117	114,710	141,400	**89400**	94600	94100
5	20	43,967	78,968	120,600	**78700**	61500	62800
10	10	33,515	57,696	104,200	**70100**	43100	43800
20	5	24,764	40,334	86,900	**60600**		
50	2	14,982	21,709	60,400	**45300**	14900	12300
80	1.25	9,961	12,688	41,200	**33300**		
90	1.11	8,334	9,886	33,400	**28200**		
95	1.05	7,314	8,171	28,000	**24500**		
99	1.01	5,949	5,925	19,800	**18600**		

* Discharges reduced to not exceed annual discharges

Table 3 Flow Duration

	Discharge (cfs)												
Percent Time Flow Equaled or Exceeded	Annual	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
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
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
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
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
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

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The focus of the main channel sediment model was to evaluate general tendencies in bed change for a range of flow split percentages. Of particular concern is deposition of sediment in front of the irrigation diversion headworks. Excessive deposition would require frequent maintenance and is not desirable.

The third model included only the bypass channel and focused on general tendencies in bed change. Many assumptions were used in creating the model and no calibration/verification was possible. Therefore, the model results contain a high degree of uncertainty.

4.2 Bypass Channel Configuration

The various bypass alignments were developed based on length required to obtain the desired channel slope as well as to minimize excavation quantities. Four alignments are shown in Figure 1.

Alignments 1 and 3 have similar lengths (≈15,500ft) and alignment 2 is slightly shorter (≈13,500ft). Alignment 3 was developed to maximize the use of historic channel scars and swales following a site visit in August 2011 and supersedes Alignment 1. Alignment 4 is 1.5 times longer than Alignment 3, representing a slope of 0.0004 ft/ft vs. the 0.0006 ft/ft slope of Alignment 3. Alignments 1 and 2 are shown only because they were discussed in the original concept evaluation (April 2011). The longest, Alignment 4, was only recently considered based on comments from the BRT pertaining to the pallid's preferred substrate and the natural armor layer that would be expected to develop for the flatter slope.

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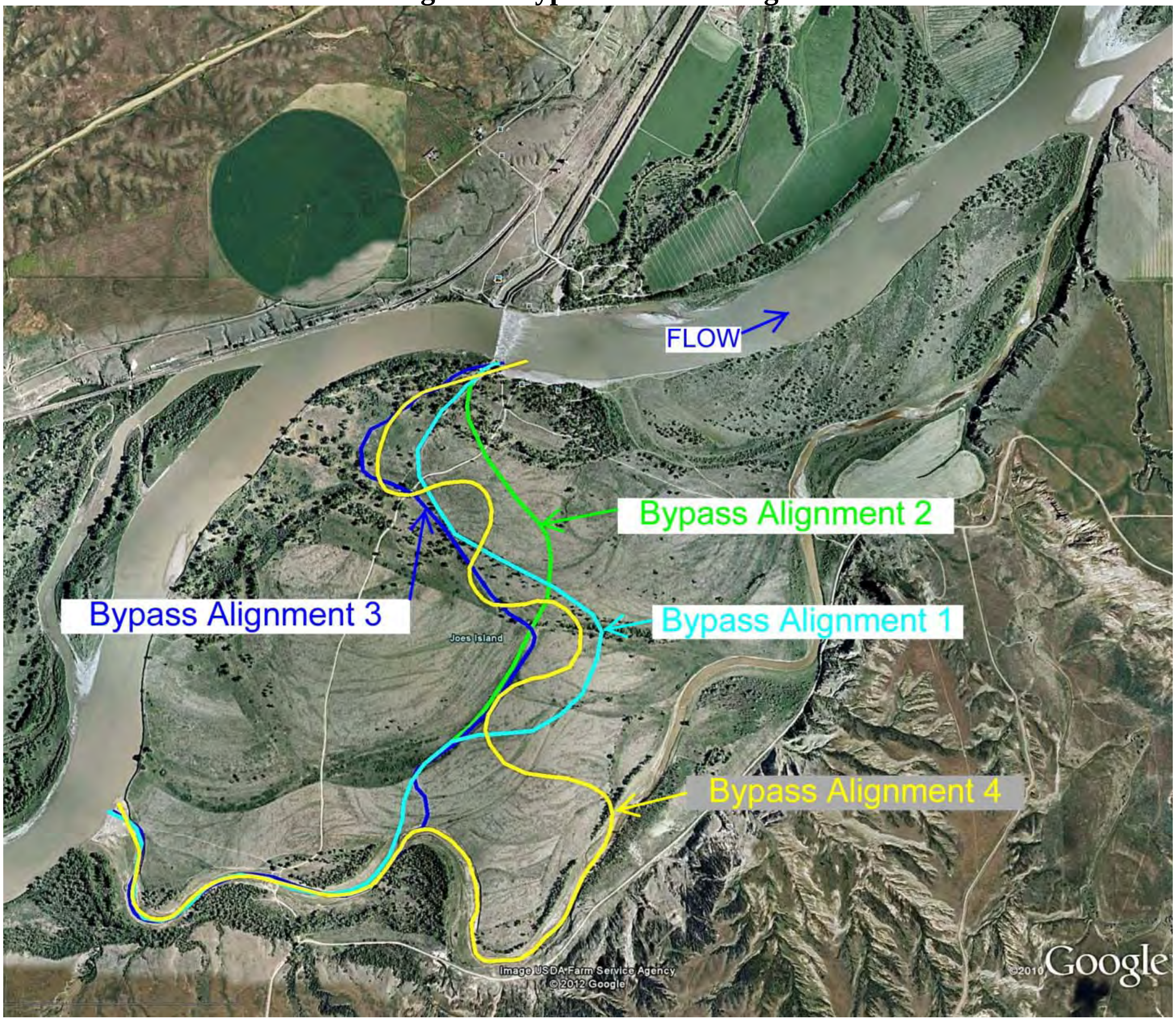
Figure 1 Bypass Channel Alignments



Following the concept analysis presented in April 2011, further coordination with Reclamation and the BRT led to a channel with a 40 ft bottom width, side slopes varying between 1V:12H and 1V:3H, and a longitudinal slope of 0.0006 ft/ft. The channel section is shown in Figure 2.

Details on the channel configuration, in addition to other project components, can be found in Appendix C, 30% Design Features.

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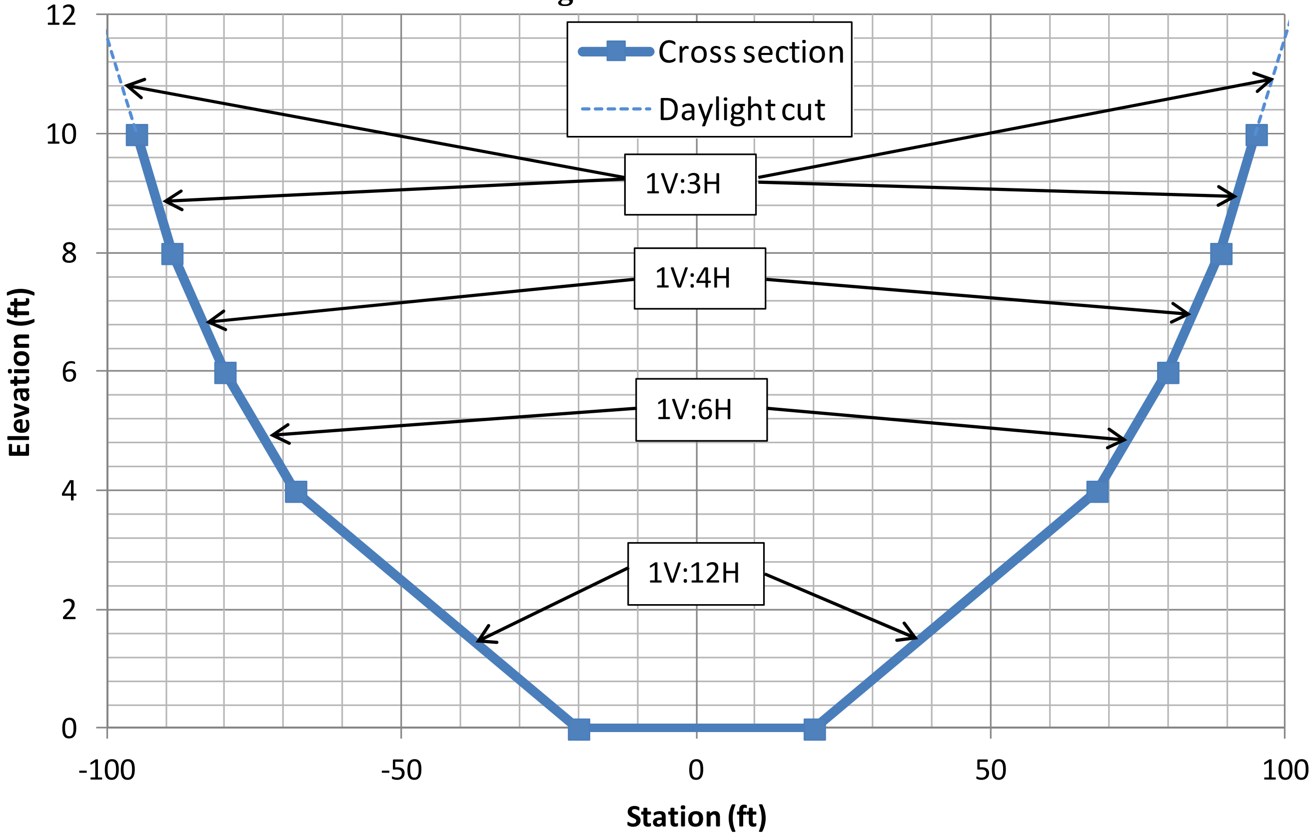
Figure 2 Channel Section



Many alternatives were developed using the inclusive model to roughly determine flow splits between the Yellowstone River and the bypass channel. The selected alternative (15% Diversion) for the 30% design is summarized in Table 4 along with three others for comparison.

The channel configuration using Alignment 3 and the section shown in Figure 2 will hereinafter be referred to as the **15% base bypass alternative**. It diverts 10%-17% of Yellowstone River flows and is considered the 15% diversion alternative. Also evaluated were 10% and 30% diversion alternatives.

The 10% diversion alternative utilizes a cross section similar to that shown in Figure 2, but half the width (i.e. bottom width is 20ft, each side slope section only half as wide). The 30% diversion utilizes the same side slopes as those shown in Figure 2 with a 200ft bottom width.

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Table 4 Bypass Channel Flow Splits and Configurations

Recurrence interval (annual, post-Yellowtail flows)	Total Yellowstone River discharge	BASE (existing right bank chute assuming new headworks with existing dam)		10% Diversion		15% Diversion		30% Diversion		Long Alignment	
		Flow Splits for Base and Alternatives									
	(cfs)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)
<2-yr	3000	0	0	220	7	310	10	890	30	273	9
<2-yr	7000	0	0	650	9	860	12	2220	32	755	11
<2-yr	15000	0	0	1550	10	2140	14	4770	32	1897	13
<2-yr	30000	790	3	3220	11	4510	15	9290	31	4019	13
2-yr	45300	2280	5	5180	11	7170	16	13720	30	6417	14
5-yr	60600	4050	7	7340	12	9900	16	18130	30	8937	15
10-yr	70100	5220	7	8770	13	11690	17	20780	30	10558	15
20-yr	78700	6090	8	9990	13	13210	17	23240	30	11919	15
50-yr	89400	7280	8	11540	13	14940	17	26260	29	13534	15
100-yr	97200	8090	8	12650	13	16280	17	28170	29	14815	15
500-yr	114000	9920	9	15570	14	19290	17	32490	29	17760	16

Pertinent Bypass Channel Parameters

	10% Diversion	15% Diversion	30% Diversion	Long Alignment
Alignment	3	3	3	4
Bypass Channel Length (ft)	15500	15500	15500	23250
Bypass Channel Longitudinal Slope	0.00060	0.00060	0.00060	0.00040
Bypass Channel Bottom Width	20	40	200	40
Bypass Channel Side Slopes	Vary from 1V:12H to 1V:3H			
Approximate Excavation Quantity (cubic yards)	800,000	1,200,000	2,600,000	1,700,000

The upstream and downstream inverts were the same for all four alternatives. The very upstream invert (fishway exit) was set at 1990.3 ft NAVD88 and the downstream invert (fishway entrance) is 1981.0 ft NAVD88 for a total drop of 9.3 ft.

Typical channel cuts from the 15% alternative are compared to existing ground in Figure 3. Note that the “RS=####” in the upper portion of the figure refers to the river station in feet from the downstream end of the bypass channel.

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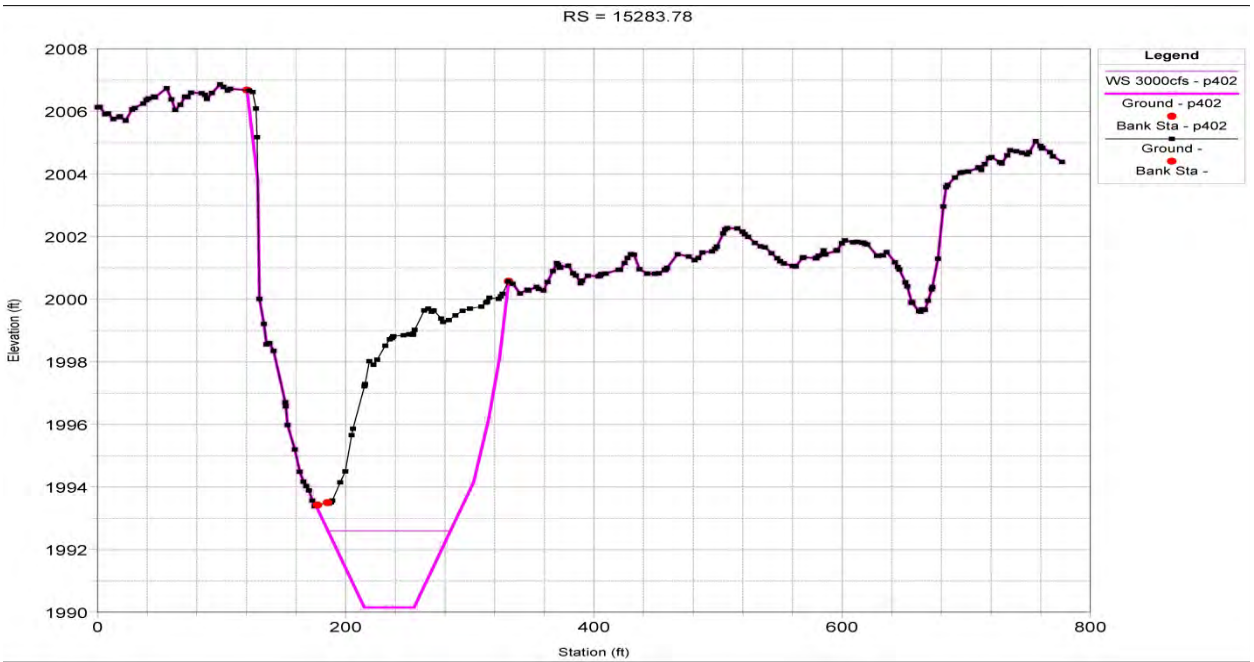
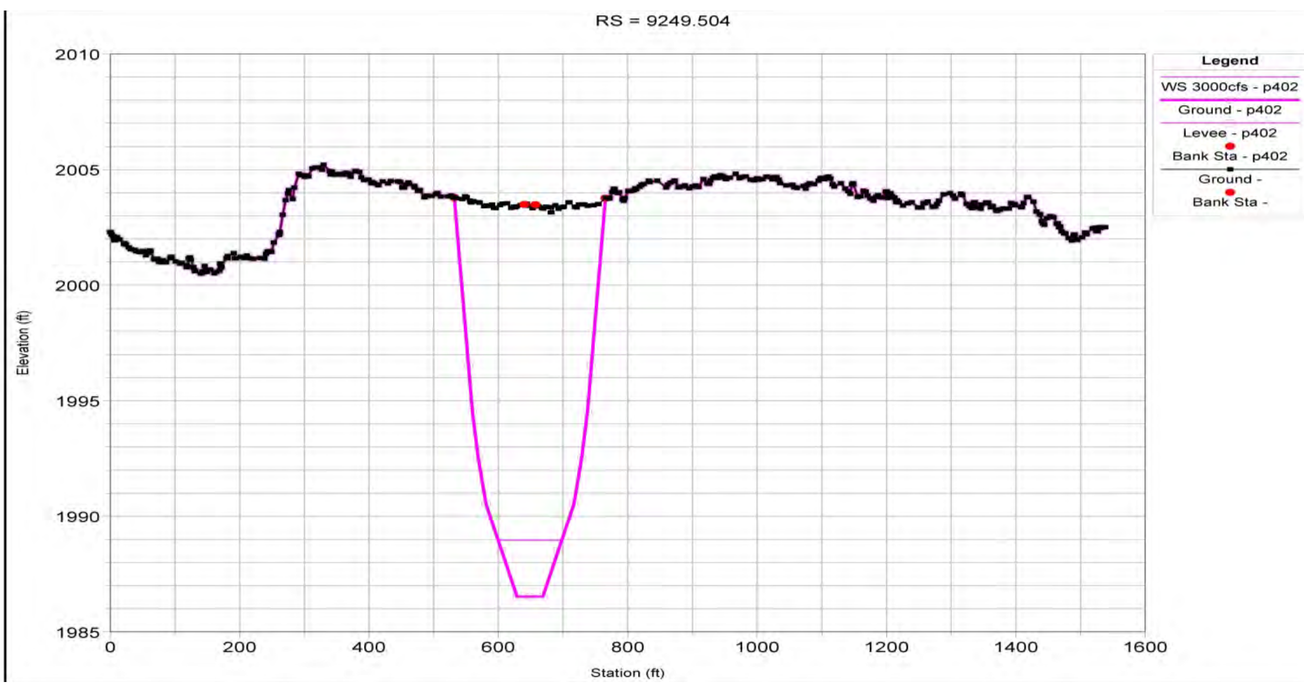
Figure 3 Typical Channel Cuts

(Note: first cut shown is upstream in existing high flow chute, second is downstream across Joe's Island)





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5. HEC-RAS MODEL LIMITATIONS

Limitations of the 1-dimensional HEC-RAS model preclude the evaluation of certain detailed project features and functions. Detailed evaluation of the following items (in addition to others) is beyond or limited by the capabilities of HEC-RAS:

- Downstream (fishway entrance) configuration (i.e. precise orientation and guide wall configuration).
- Connection of main channel thalweg to bypass channel.
- Bank/toe protection. Because the HEC-RAS model is 1-dimensional, it does not directly account for secondary velocities in bends.
- Sediment modeling was performed with limited detail for this design phase. Sediment transport modeling is notoriously difficult. The data utilized to predict bed change is fundamentally uncertain and the theory employed is empirical and highly sensitive to a wide array of physical variables. Sediment transport measurements often show variations over more than one order of magnitude. This inherent uncertainty in sediment transport is compounded when numerical models are used to simplify natural processes.

6. RESULTS AND DISCUSSION

6.1 Configuration

For discussion purposes, the “configuration” of the bypass channel consists of the following elements (in addition to others):

- Horizontal alignment
- Length
- Longitudinal slope
- Bottom width
- Side slopes
- Inlet and outlet configuration
- Scour protection

Various combinations of the bypass channel elements can be used to produce a range of flow diversion percentages, velocity/depth characteristics, and excavation/riprap quantities. It is apparent that trade-offs between various elements may have desirable or undesirable effects (e.g. a larger bottom width generally allows a higher percentage of diverted flow but results in larger excavation quantities; with other elements kept constant, shorter length results in smaller excavation quantities but gives a steeper slope and increased velocities, etc.)

The selected configuration for the 15% Bypass Alternative was developed in conjunction with Reclamation and the BRT. Comments received from a BRT review of the draft 30% design in March 2012 indicate that a coarse sand bed material might be more attractive to pallid sturgeon than gravels or cobbles. Further evaluation of a longer, flatter bypass channel may be warranted in the next design phase. The evaluation would compare the costs associated with the increased excavation quantities for a longer channel with the costs associated with adding the channel armor layer. At this time (March 2012) the longer channel concept is not well developed. As the channel slope is decreased, flow velocities and sediment transport also decrease and

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sustainability may be in question. However, quantities for purposes of a cost estimate were estimated to allow for a comparison between excavation costs and the channel armor layer.

Ten Yellowstone River side channels, in addition to the existing high flow chute at Intake, were evaluated and compared based on available GIS information and aerial photography. Based on visual observations of the side channels and measurements using GIS, chute lengths, sinuosity, and top widths were estimated. Results are summarized below in Table 5. See Appendix D, Reference Reach Comparison, for details.

Table 5 Reference Reach Summary

Reach Identifier	Orientation and distance from Intake Dam	Approximate chute length	Estimated energy grade slope in reference reach chute
		(ft)	(ft/ft)
1	54 miles d/s	9900	0.00008
2	38 miles d/s	9400	0.0004
3	34 miles d/s	11400	0.0002
4	31 miles d/s	22100	0.0004
5	19 miles d/s	10600	0.0006
6	9 miles d/s	8700	0.0006
7	Existing chute at Intake	24700	0.0005
8	17 miles u/s	5000	0.0006
9	23 miles u/s (at Glendive)	13600	0.0003
10	28 miles u/s	10400	0.0005
11	33 miles u/s	7500	0.0006
PROPOSED BYPASS	Proposed bypass at Intake	15500-23250	0.0004-0.0006

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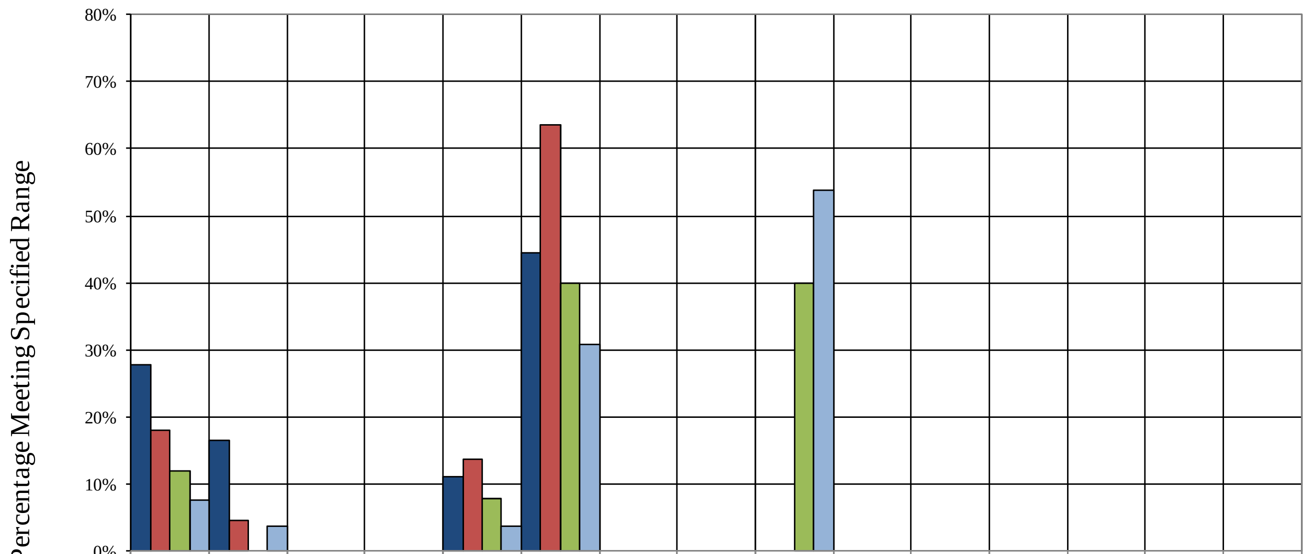
6.2 Depths and Velocities

Figure 4 shows the percentage of the base 15% alternative meeting various depth/velocity ranges. Note that because of the simplified channel geometry within the HEC-RAS model, the depth and velocities throughout the bypass channel are relatively uniform. Additionally, the classification chart was only created for the 15% diversion alternative. Classification charts for the other alternatives are expected to look similar to the 15% chart.

Figure 4 Percentages of Bypass Meeting Depth/Velocity Ranges



	0-1.64ft 0-2ft/s	1.64-3.28ft 0-2ft/s	>3.28ft 0-2ft/s	0-1.64ft 2-4ft/s	1.64-3.28ft 2-4ft/s	>3.28ft 2-4ft/s	0-1.64ft 4-6ft/s	1.64-3.28ft 4-6ft/s	>3.28ft 4-6ft/s	0-1.64ft 6-8ft/s	1.64-3.28ft 6-8ft/s	>3.28ft 6-8ft/s	0-1.64ft >8ft/s	1.64-3.28ft >8ft/s	>3.28ft >8ft/s
7k	28%	17%	0%	0%	11%	44%	0%	0%	0%	0%	0%	0%	0%	0%	0%
15k	18%	5%	0%	0%	14%	64%	0%	0%	0%	0%	0%	0%	0%	0%	0%
30k	12%	0%	0%	0%	8%	40%	0%	0%	40%	0%	0%	0%	0%	0%	0%
40k	8%	4%	0%	0%	4%	31%	0%	0%	54%	0%	0%	0%	0%	0%	0%

Bypass Channel-Alignment 3-15% Diversion Alternative

40ft Bottom width, varying side slopes, Channel slope=0.0006

The depth/velocity ranges computed for Figure 3 were determined using the flow/velocity distribution feature within HEC-RAS. This feature allows for the estimation of a velocity distribution across the channel rather than a simple average velocity for the whole section. A screen shot of the velocity distribution from HEC-RAS is shown in Figure 5.

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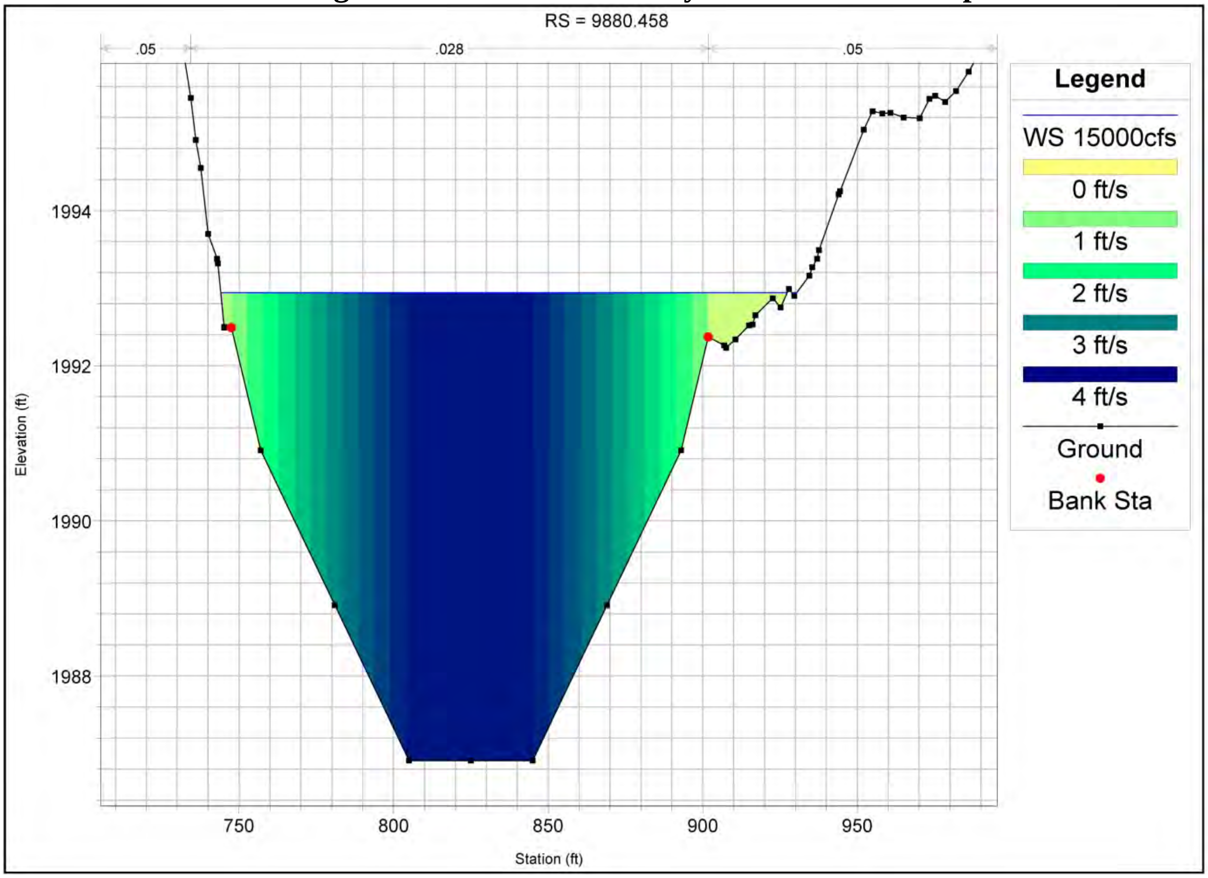
Figure 5 HEC-RAS Velocity Distribution Example



6.3 Channel Stability

Channel stability analysis performed for this study has a high degree of uncertainty due to the limited available data and detail of modeling. The relative risk for stability measures pertains to the frequency and magnitude of required maintenance. While some bypass channel dynamics is acceptable, continued bank failure or erosion would likely impact fish passage performance of the channel and also alter the desired flow split between the main Yellowstone River and the bypass channel.

Proposed stability measures for the 15% alternative include the following:

- Upstream and downstream grade control and lateral control
- Two grade control structures, spaced approximately evenly between the upstream and downstream ends
- Riprap revetments at 2 outside bends
- Channel armor layer to prevent excessive degradation

As discussed above in section 5.1, there may be some tradeoff with the channel armor layer by excavating a longer, flatter channel. However, this evaluation has not been completed as of March 2012.

An analysis was completed to evaluate characteristics of the armor layer that would likely form naturally in the bypass channel. Details on the analysis can be found in Appendix E, Bypass Channel-Stable Channel Materials Analysis.

6.4 Sediment Continuity

Sediment continuity refers to the requirement to maintain sediment transport for both the Yellowstone River and the bypass channel. In the existing condition, the Yellowstone River is able to transport both suspended material and bed material over the diversion dam. Available data indicates that the system is in an equilibrium condition and does not exhibit any long term aggradation or degradation trends. The potential for the bypass channel to disrupt sediment continuity limits the maximum bypass channel flow rate.

Sediment sampling was conducted in 2011 by the U.S. Geological Survey for the U.S. Army Corps of Engineers. Details pertaining to the sampling efforts and results of the sampling can be found in Appendix F, DRAFT USGS Sediment Sampling Report.

There is a risk of sediment deposition within the bypass channel due to the desire for low velocities for fish passage. Depending on the size and type of materials entering the bypass channel over the range of flows, large quantities of sediment could deposit within the channel. Additionally, during extreme events that inundate the entire island with depths greater than a few feet (50-100 year range or greater), a large portion of the bypass channel could be filled with sediment. The need to maintain sediment transport through the bypass channel may impact the ability to meet fish passage design criteria.

Another concern relative to sediment is current versus with-project transport capacity in the vicinity of the dam and headworks structure. Depending on the configuration of the diversion inlet and nearby flow patterns, it is likely that the bypass channel will take very little, if any, bedload sediment from the main channel. This larger sediment will then continue downstream and could potentially be deposited in front of the dam and headworks since there would be less flow available to transport similar volumes of larger sediment. The current system apparently transports much of this bedload up and over the existing dam. With 10-15% of total flows diverted around the dam, sediment buildup in front of the headworks is a concern. Extensive analysis and data collected over a period of many years is required to evaluate the sediment balance within the system (from upstream to downstream of the bypass channel as well as downstream in the irrigation canal).

As a result of the requirement to maintain sediment transport in the Yellowstone River, a minimum river flow is required. Consequently, the maximum bypass channel flow will also be limited. Detailed analysis is required to define the upper limit for bypass channel flow. Based on observations in other rivers, sustainable chutes with flows in the range of 5 – 10% of the main channel flow are often observed. On the lower Missouri River, stability issues and main channel deposition have occurred when chute flows exceed 10%. The requirement to maintain sediment transport may limit bypass channel flows to a rate less than desired for fish passage.

Based on available gage data, the Yellowstone River at Sidney is estimated to have moved approximately 900,000 tons of sediment from May 1 through August 30 in 2007 as suspended

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load. Assuming an additional 15% moving as bed load gives a total load of over 1,000,000 tons. Using a density of 95 lb/ft^3 , computations estimate results in over 800,000 yd^3 of material being transported by the Yellowstone River during a single irrigation season. The potential for deposition that is outside the scope of an O&M issue is a concern.

One possible solution to sedimentation in the vicinity of the headworks would be to construct a sluiceway through the dam crest and existing rock field. The sluice would likely be a gated set of large culverts with a training wall to create high velocities in front of the headworks to flush sediment deposits.

Three sediment sluice options were described in the Final Environmental Assessment (2010). However, since construction of the new headworks and backfilling behind the old headworks, two of the proposed sluice options that used the old headworks structure as a gate structure are no longer feasible. Therefore, sediment sluicing would likely use an in-channel sluiceway consisting of a gate structure located just upstream from the dam crest, four 8-ft wide by 6-ft tall vertical lift gates, and two covered conduits 20.5-ft wide by 10-ft high extending downstream through the dam crest to the toe of the existing rock field.

6.5 Ice Impacts

The upstream end of the bypass channel would likely be subject to significant ice forces. Since the April 2011 concept analysis, further evaluation has been conducted. The U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) conducted a study and provided recommendations. Based on CRREL's analysis, the upstream control structure has been changed from a riprap structure to a concrete structure.   The draft report provided by CRREL is included as Attachment 1.

The CRREL evaluation estimated required riprap design configurations (rock size and layer thickness) and compared their estimates with the preliminary design provided by Omaha District. The comparison indicated that Omaha District designs were very similar to those estimated by CRREL. See Table 6 for the riprap comparison taken from the CRREL report (Attachment 1).

Table 6 Riprap Design Comparison (source: Attachment 1, CRREL Draft report)

	Average Velocity	D_{50} (inches)			D_{100} (inches)			Layer Thickness (inches)		
Structure	(ft/s)	Omaha District	Factored Isbash	Factored EM	Omaha District	Factored Isbash	Factored EM	Omaha District	Factored Isbash	Factored EM
Bypass Inlet Weir	5.0	12	10	12		20	24	27-40	30	36
Channel Plug	6.2	20	12	8	30	24	16	45	36	24
Bypass Bends	7.0		12	10	16	24	18	24-36	36	28
Vertical Grade Control	6.0	12	10	12		20	24	27-40	30	36
Bypass Outlet Weir	5.2	12	10	12		20	24	27-40	30	36
Downstream Lateral Stability Structure	6.5	12	10	7	24	20	12	36	30	34
Flow Augmentation Weir	6.8	CRREL recommendation: use 1.5-2.0-ft riprap where concrete wall ties into bank.								

In addition to evaluating riprap design, the CRREL evaluation computed ice forces for use in structural design of concrete features. For the upstream concrete sill under a worst case scenario, an ice force of 5 kips/ft could act horizontally along the front edge. For the surface of the upstream sill and the downstream flow augmentation weir crests, a maximum horizontal ice force of 2 kips/ft^2 due to sliding ice is estimated. The concrete wall along the upstream edge of the flow augmentation weir is expected to experience high ice impacts. Here, an ice design load of 10 kips/ft is recommended.

The CRREL report suggested increasing rock size and layer thickness in two areas: the left hand side of the transition from the Yellowstone River into the upstream control structure and the right bank of the Yellowstone River immediately upstream of the flow augmentation structure.

6.6 Access to Dam (Right Abutment) and Left Bank of Bypass Channel

The current concept for access to the left bank of the bypass channel is to construct temporary crossings on an as-needed basis.

6.7 Impacts to Depth/Velocity at Proposed Dam Crest

A new, raised concrete dam is proposed just upstream from the existing dam. Two reasons exist for the proposed new dam:

- The new headworks structure requires additional head for diversion due to head losses through the new screens and
- Continued placement and loss of large rock on the dam may adversely impact the bypass channel entrance.

The current dam has required frequent maintenance (addition of large rock to crest) in order to provide the necessary head for diversion using the *old* headworks structure. Construction of the new headworks, substantially completed by March 2012, included the installation of fish screens to prevent entrainment of fish in the irrigation canal. Flow through the screens includes head losses, thus requiring additional head in order to divert the irrigation district's full water right during low flow periods on the Yellowstone River. The additional head (estimated to be approximately 0.5-0.7 ft by the screen manufacturer) will be gained by increasing the top elevation of the dam crest. Additionally, to prevent the need for annual placement of riprap on the crest as is currently done, a concrete crest is proposed. The proposed crest location is approximately 50 ft upstream from the existing crest.

With the new headworks requiring an estimated 0.7 ft of additional head, the addition of rock to the existing structure would likely increase not only initially but over time due to the higher potential for loss of protruding rocks on the crest to ice.

The amount of dam raise is not a set amount. The elevation of the current top of dam/top of rock is unknown; even if the current top was known, it has been established that the top changes annually depending on ice conditions, high flow events, and the irrigation district's placement of rock.

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The proposed top of dam elevation is 1990.5 ft NAVD88. This is likely similar to or slightly above the top of rock elevation just after the irrigation district places rock and perhaps 2 ft above top of rock just before rock placement.

Concerns have been raised over the impact to passability over a new crest structure for fish species that are currently able to pass the existing rock field and dam. A hydraulic model (HEC-RAS) was used to evaluate and compare existing and proposed conditions. Comparison of depths and velocities over the crest between existing and proposed conditions is difficult due to changing conditions and lack of data on the existing crest. For purposes of this analysis, the existing crest was assumed to be at 1989.5 ft NAVD88. **This is an assumed elevation and the crest is known to vary by at least 2 ft.**

Existing conditions vs. proposed conditions water surface profiles and average channel velocities are shown in Figures 6 and 7, respectively. Note that in Figures 6 and 7, proposed conditions assume the bypass channel is diverting approximately 15% of total Yellowstone River flows. Therefore, proposed depths and velocities are slightly lower on the rock field due to the lower discharges.

Figures 8 and 9 show depths and velocities over the existing vs. proposed crests *not counting the bypass channel flow diversion*. Results indicate slightly longer lengths of higher velocities, but do not show higher overall velocities for the proposed crest structure.

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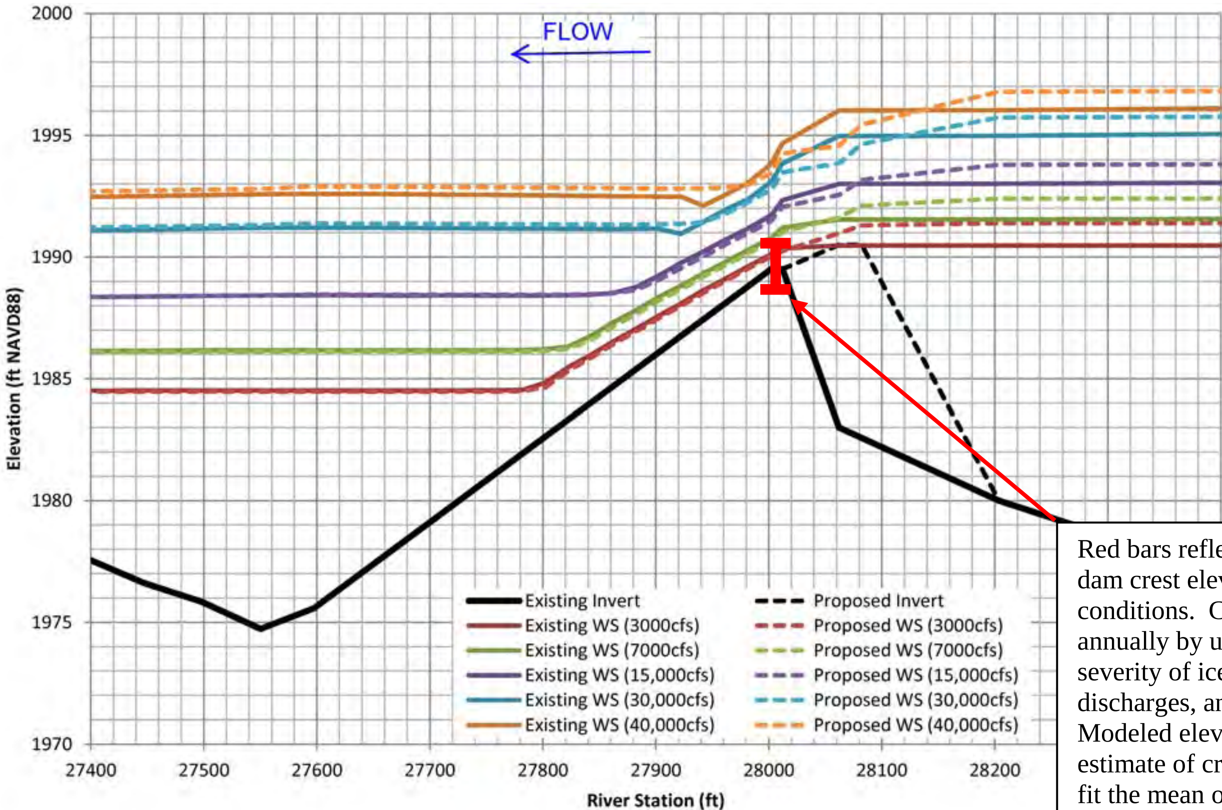
Figure 6 Dam Crest Water Surface Profiles, Existing vs. Proposed, with Bypass Channel



Red bars reflect variable and uncertain dam crest elevation for existing conditions. Crest elevation varies annually by up to 2 ft depending on severity of ice runs, extreme discharges, and rock placement. Modeled elevation was based on best estimate of crest elevation required to fit the mean of the measured stage-discharge rating curve.

Figure 7 Dam Crest Average Velocities, Existing vs. Proposed, with Bypass Channel

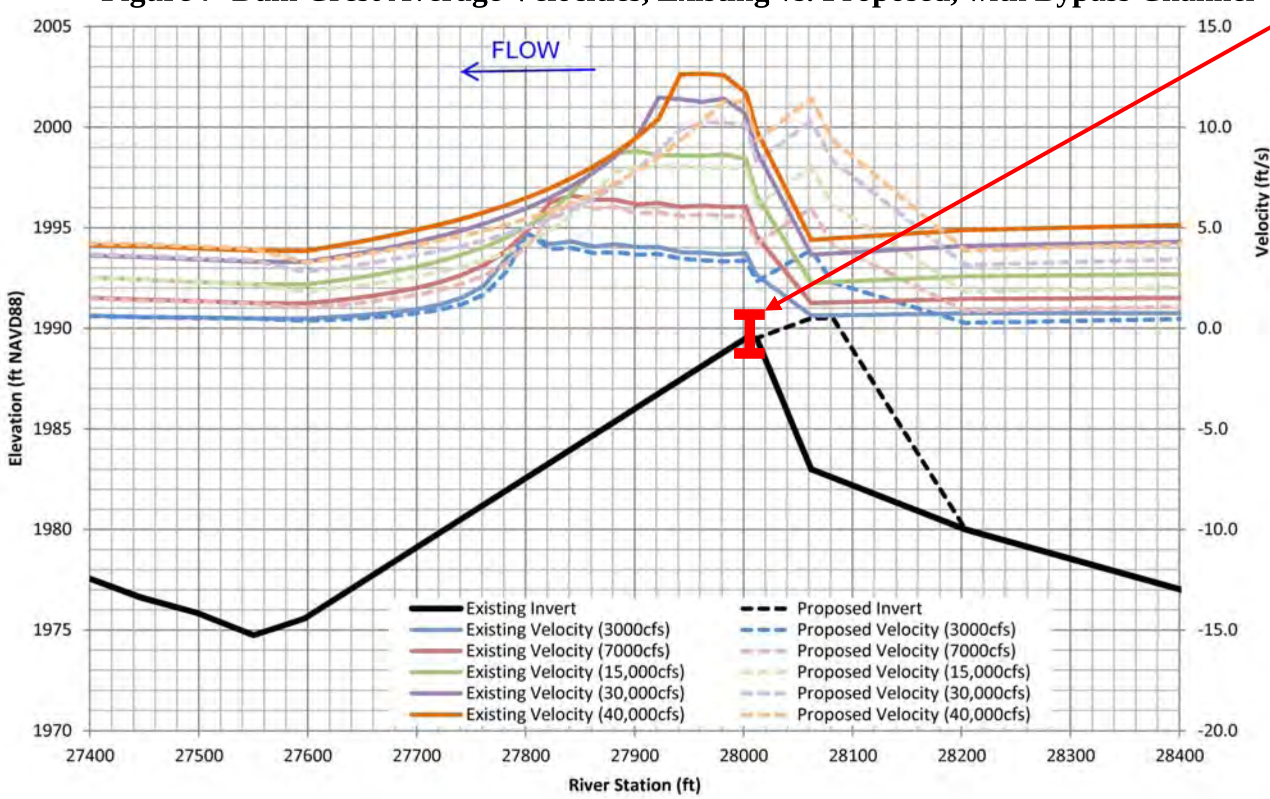


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Figure 8 Dam Crest Water Surface Profiles, Existing vs. Proposed, without Bypass Channel

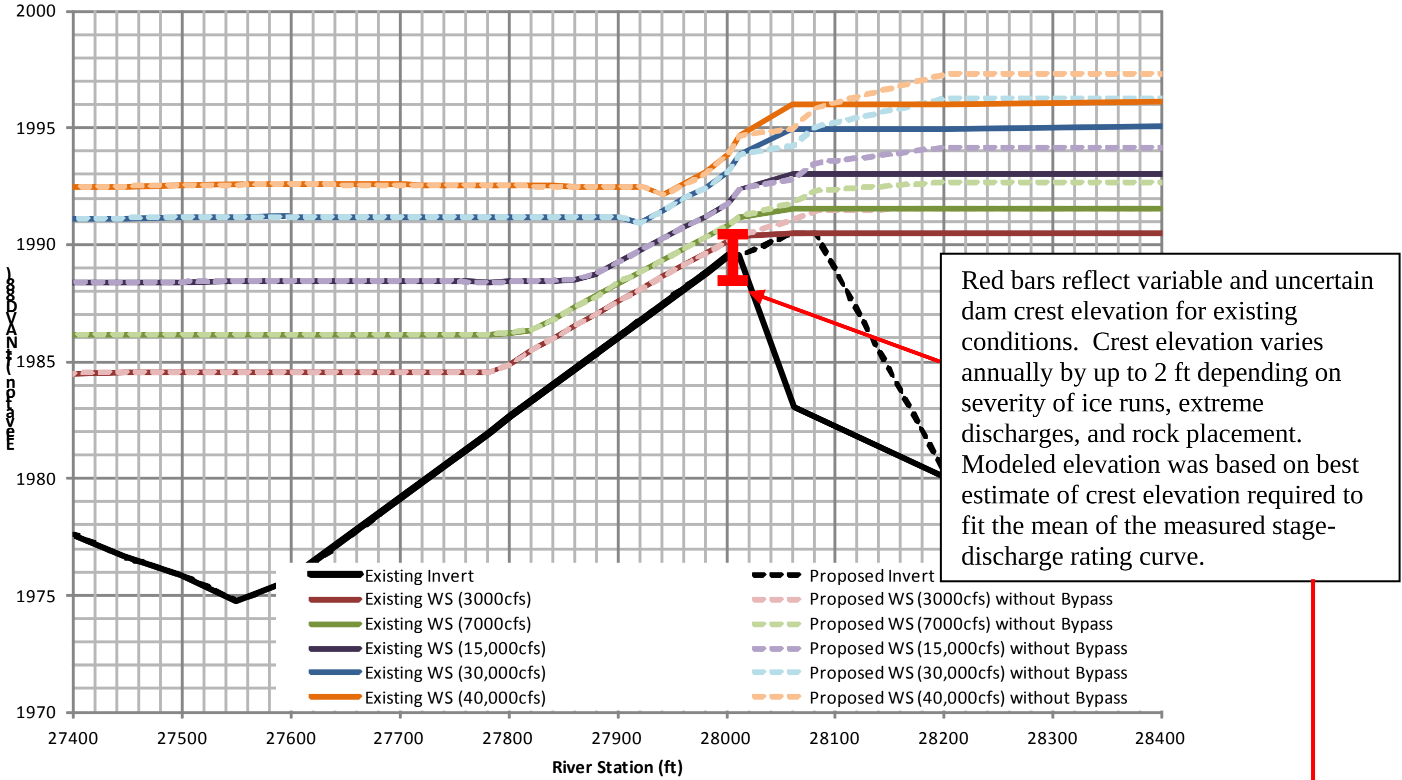
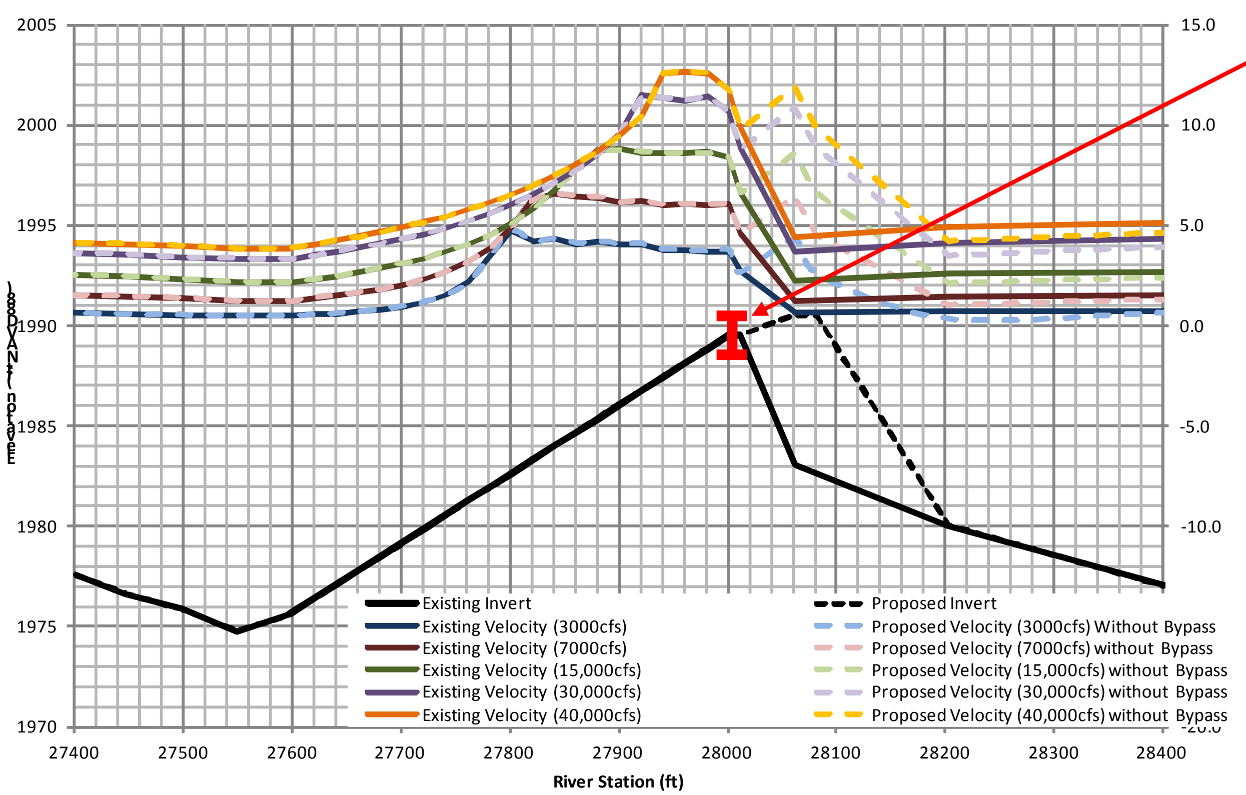


Figure 9 Dam Crest Average Velocities, Existing vs. Proposed, without Bypass Channel



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6.8 Future Design Work

Additional data needs and design work to carry this concept to final design would include:

- Collection of additional bathymetry data in the Yellowstone River in the vicinity of the fishway exit (upstream end)
- Collection of additional soil borings on the final alignment to assist in design of scour protection
- Further evaluation of optimal diversion discharge percentage
- Further evaluation of a longer channel
- Potential 2-dimensional evaluation of bypass channel
  - Compare depth/velocity to BRT criteria
  - Evaluate depth / velocity changes in detail at the diversion dam, comparing with and without bypass channel conditions
  - Determine scour protection requirements
  - Adjust inlet/outlet configuration
  - Adjust planform
  - Evaluate sediment transport within the chute and main Yellowstone River
  - Collect suspended and bedload sediment data in the Yellowstone River in the vicinity of the proposed fishway exit (upstream end of bypass channel)
- Detailed evaluation of island access crossing needs and requirements
- Geomorphologic assessment of existing right bank chute
- Sediment transport study of existing right bank chute

7. SUMMARY AND CONCLUSIONS

Design Analysis. The bypass channel design analysis consisted of the following evaluations:

- Yellowstone River-Hydraulic and sediment modeling was conducted to evaluate the maximum feasible flow split and associated sediment transport characteristics. Results indicate that the maximum feasible diversion percentage is in the range of 15%, with a 10% split preferred. Modeling indicates that diversion of more than 15% of total Yellowstone River flow would likely result in sediment deposition in the Yellowstone River, which is undesirable due to operation and maintenance challenges posed by working in the river. Note that no calibration data for the sediment modeling is available, limited sediment data and bed material data is available, and discharges for the simulation were based on flow records at Sidney and Glendive for the past 20 years. Additional sediment modeling and evaluation in the future design phase is required.
- Bypass Channel-Hydraulic and sediment modeling was conducted to evaluate flow/depth characteristics as well as sediment transport within the bypass channel. Results of the hydraulic modeling indicate that flow and depth criteria set forth by the BRT are met at all flows. Results of sediment modeling in the bypass channel indicate a slightly degradational trend, but the results are highly sensitive to several inherently uncertain input parameters to the model. Note that no calibration data for the sediment modeling is available, limited sediment data and bed material data is available, and discharges for the simulation were based on flow records at Sidney and Glendive for the past 20 years. Additional sediment modeling and evaluation in the future design phase is required.

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- Channel Stability Computations-In addition to the HEC-RAS sediment modeling completed on the bypass channel, stability computations were performed using various methods to evaluate channel stability (Shields stability analysis, critical shear stress, Copeland method). The evaluation indicates general agreement among the various methods for substrate material size required for channel stability, approximately 1-2 inches in diameter. As the design is refined in future evaluation, the stability analysis will also be updated. The requirement for an armor layer will be reviewed based on results from the stability analysis.
- Rock Sizing-Rock sizing for the bypass channel features was completed using HDC 712-1 (Isbash method) and guidance in EM 1110-2-1601. An evaluation by CRREL verified rock sizes were adequate considering ice effects.
- Geomorphologic Comparison of Similar Side Channels-A cursory evaluation was conducted to compare existing natural side channels on the Yellowstone River in the vicinity of Intake to the proposed bypass channel. The analysis indicates that general planform characteristics of the proposed bypass fall into the range of similar side channels.

Bypass Passage Evaluation

- Channel Section-The bypass chute design was performed with the goal of meeting passage objectives given the uncertainties in pallid sturgeon behavior. The channel section has compound bottom slopes to provide a range of depth and velocity diversity for a range of flows. The variation will optimize the potential for suitable habitat availability and also result in substrate size variability. The channel section still has normal side slopes with minimal impact to total quantity. The selected side slopes are compatible with long term sustainability to avoid bank failures.
- Flow depth-Flow depths were evaluated based on BRT criteria indicating preferred depths of greater than 1 meter with scaled passage ability with depths between 0.5 and 1 meter. During extreme low flows on the Yellowstone River (3000 cfs) most of the bypass channel has a depth of 0.5-1 meter, with the downstream end greater than a meter. When flows reach 7000 cfs (representative of April and August 50% exceeded by duration flows), bypass channel depths are greater than a meter (around 4ft or greater). At 15,000 cfs, representative of May and July 50% exceeded by duration flows, depths are in the 6-7ft range. At 30,000 cfs, representative of the June 50% exceeded by duration flow, depths are in the 8-10 ft range. Note that depths given are maximum depths in the thalweg; the actual depth would range from 0 to the maximum along the relatively flat sloped channels (1V:12H to 1V:3H).
- Flow velocity-Target flow velocities are lower than 4 ft/s for adult pallids and lower than 2 ft/s for juveniles based on BRT criteria. Because modeling to date is one-dimensional and considers a constant slope, uniform channel, average velocities throughout the bypass are fairly consistent. During extreme low flow periods on the Yellowstone River (3000cfs), bypass channel velocities are just under 2 ft/s. During 7,000-30,000cfs flows, bypass velocities are generally in the 2-4 ft/s range with areas in the 4-6 ft/s range (areas of 4-6 ft/s are located in the thalweg during 30,000 cfs flows; computed velocities outside of the thalweg are in the 2-4 ft/s range). Note that for all flows between 0-40,000cfs, models indicate areas with velocities less than 2 ft/s exist on the fringes (i.e. outside of the thalweg along the flat slope areas).

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- Turbulence-Minimal turbulence is expected in the majority of the bypass channel due to the relative uniformity of the cross section. Areas of concern are the exit and entrance where bypass and Yellowstone River flows converge and diverge. Of particular interest is the bypass channel entrance (downstream end) due to the potential for excessive turbulence and shear zones to disrupt upstream pallid migration. Additional numerical modeling and potentially physical modeling would be utilized to further evaluate turbulence in the critical areas to maximize passage ability based on input from the biological community. Additionally, monitoring of the project would be expected to identify potential areas of high turbulence preventing passage. Adaptive management techniques, including movement/manipulation of riprap or addition of training structures (especially at the bypass entrance) have been identified as having potential to minimize turbulence concerns.

Bypass Stability Features

A number of stability features are included in the bypass channel to maximize the potential for a long term, sustainable fish passage project.

- Stable Channel Design-The design follows standard stable channel design principles that use a variable bottom slope and meander pattern. Past projects have illustrated that a straight alignment or flat bottom channel is not stable which would create a passage risk as the chute develops a stable planform. Stability features in the chute are not optional if we want to provide a long term sustainable project that meets the objective of providing both passage and irrigation diversion.
- Upstream Control-Riprap with a concrete sill, designed to provide stability during extreme events. The structure would be backfilled with natural river size rock to give the appearance of a seamless channel invert.
- Channel Plug-Rock-lined earthen embankment at point where proposed bypass channel diverges from existing high flow chute, designed to keep flows in bypass during low flows. A low-level discharge pipe allows for normal flows to pass into the existing chute while the rock lining allows for overtopping during extreme events in order for the existing chute to maintain its current functionality.
- Riprap at Bends-Standard bank stabilization techniques at critical locations to prevent major loss of channel planform during extreme events. Some channel movement is expected and desired to attain the appearance of a natural channel.
- Vertical Control-Two riprap sills are proposed for maintaining channel slope during extreme events and for early identification of channel movement. The sills would be overexcavated and backfilled with natural river size rock similar to the upstream control structure.
- Downstream Control-Both vertical and horizontal riprap control structures would be constructed on the downstream end of the bypass channel. The horizontal control is intended to prevent downstream migration of the bypass while the vertical control is intended to maintain channel elevation.
- Armor layer-Bypass channel sediment modeling and stability computations indicate that substrate material in the 1-2 inch size range is required for channel stability (i.e. armor layer). Based on available field data, a natural armor layer with this approximate material size would be expected to form over time. However, while this size of material is apparently available along the alignment (based on limited field data), it is outside the

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ability of current sediment modeling practices to predict with the required precision the amount of degradation that would occur prior to formation of a stable armor layer. Additional field data collection along the final chute alignment and evaluation in future design is required to further evaluate the armor layer.

New Dam

A new, raised concrete dam is proposed just upstream from the existing dam. Two reasons exist for the proposed new dam:

- The new headworks structure requires additional head for diversion due to head losses through the new screens.
- Continued placement and loss of large rock on the dam may adversely impact the bypass channel entrance which is located immediately downstream of the dam.
- The raised dam alters depth and flow velocities in the dam crest vicinity. Since the bypass chute is taking more flow than the current condition, the depth and velocity change comparison at the dam crest is complex.

The current dam has required frequent maintenance (addition of large rock to crest) in order to provide the necessary head for diversion using the *old* headworks structure. With the new headworks requiring an estimated 0.7 ft of additional head, the addition of rock to the existing structure would likely increase not only initially but over time due to the higher potential for loss of protruding rocks on the crest to ice.

The amount of dam raise is not a set amount. The elevation of the current top of dam/top of rock is unknown; even if the current top was known, it has been established that the top of rock elevation changes annually depending on ice conditions, high flow events, and the irrigation district's placement of rock. The proposed top of dam elevation is 1990.5 ft NAVD88. This is likely similar to or slightly above the top of rock elevation just after the irrigation district places rock and perhaps 2 feet above top of rock just before rock placement during normal maintenance activities.

Comparison of depths and velocities over the crest between existing and proposed conditions is difficult due to changing conditions and lack of data on the existing crest. For comparison purposes, the top of existing dam was assumed to be 1989.5 ft NAVD88. Results of the comparison are presented in section 6.7 above. With the bypass channel, the diversion of 15% of total Yellowstone River flow results in lower velocities and depths over the crest for the same recurrence interval event. Without the bypass channel diversion (i.e. assuming no bypass channel), results generally indicate slightly longer lengths of higher velocities, but do not show higher overall velocities for the proposed crest structure.

Ice Impacts

The Yellowstone River is subject to heavy ice formation with dynamic ice breakups and ice jams. The upstream control structure is likely the most critical structure in terms of vulnerability to ice as its upstream approach lies on the outside bend and will be exposed to the full impact of ice runs on the main river. For this reason, the invert portion of this structure includes a concrete sill. Riprap ties the sill into the side slopes. Due to the exposure of this structure to large ice

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forces, O&M for the riprap portion has been estimated at a 5% replacement per year, essentially giving the structure a 20-year design life.

The remaining structures in the bypass channel consist of riprap and will likely be subject to far fewer ice breakup impacts than the upstream control structure.

The new dam crest will be subject to large ice forces. The CRREL analysis indicates a preference for a sloped upstream face, but does not allow for a reduction in ice forces for structural computations. The proposed crest uses a large riprap wedge in front of the concrete weir to minimize damage to the upstream concrete face.

Operation and Maintenance

Operation and Maintenance (O&M) quantities and costs were developed based on experience with past projects and assumptions involving frequency of riprap replacement and costs. Additional O&M costs were estimated for removal of sediment in front of the headworks. Note that the current design and modeling efforts do not indicate that sediment will deposit in front of the headworks; however, the limited available data, modeling uncertainties, and natural variability limit the accuracy of computations. Total annual O&M costs for the bypass channel (not including O&M of the newly constructed headworks) and new dam crest are estimated at approximately $140,000 per year.

Adaptive Management

The following adaptive management features were considered for post-construction enhancement of the bypass channel on an as-needed basis:

- Flow augmentation structure. The flow augmentation structure would be located on the right bank of the Yellowstone River near the proposed dam. The purpose of the structure would be to increase attractive flow at the bypass entrance (downstream end) by 5-7%.
- Localized repairs of high turbulence areas if found to be affecting fish passage.
- Modifications to the configuration of the bypass entrance (downstream end) to increase attraction to the bypass channel. These modifications would consist of movement and manipulation of riprap and the existing rock field material in the main channel of the Yellowstone River and a short distance upstream into the bypass channel.
- Intake diversion weir revisions (new dam crest). Modifications to the hydraulic characteristics of the proposed new dam crest may be required if passage of other native species (besides pallids) is found to be negatively impacted by the new crest. Modifications to the proposed concrete crest would likely not be feasible, so the AM proposal would be to manipulate the riprap between the proposed new crest and the existing dam crest in order to improve depth/velocity diversity at the proposed crest.

Future Work

Additional data collection and analyses are required to finalize design of the bypass channel, including:

- Collection of additional sediment data in the vicinity of the bypass exit (upstream end). The measured sediment load at the site is nearly an order of magnitude lower than that reported at the Sidney gage. While sediment data naturally varies, the measured data at

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sediment modeling will require additional sediment data collection in order to decrease the range of input parameters used for sensitivity analysis.

- Collection of additional soil borings along the final bypass channel alignment. Additional soil borings will be used as input to the armor layer analysis. Also, more soil borings will help alleviate concerns with potential excavation constructability issues (e.g. shallow bedrock).
- Detailed modeling of the bypass entrance (downstream end). The bypass channel entrance (downstream end) is critical to passage success. Detailed modeling (likely 2-dimensional) of the entrance will be required to assess flow conditions and various configurations to increase the likelihood of fish finding and using the bypass channel.
- Detailed modeling of the bypass exit (upstream end). The bypass exit is critical to sediment continuity and stability of the entire bypass channel. Detailed modeling of the flow and sediment split will be required to analyze stability of the bypass channel.
- Detailed modeling of the system. The entire bypass channel system, including the bypass entrance, bypass exit, bypass channel, remaining existing high flow channel and Yellowstone River from upstream of the exit to downstream of the existing high flow channel will be required to assess overall stability of the system.
- Collection of water surface elevations related to irrigation diversion. Design of the headworks included assumptions on head loss through the new fish screens. Measured water surfaces in the Yellowstone River along with diverted discharge into the canal will allow for fine tuning of the proposed dam crest elevation.
- Modeling of the proposed dam and irrigation diversion headworks. Data measured during the first season of headworks operation will be used to assess the proposed dam crest elevation and configuration needed to meet irrigation diversion requirements as well as to evaluate flow conditions over the proposed and existing dam for passage of species that currently pass the rock field.

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Attachment 6 Bypass Channel

Appendix A

Sediment Analysis-Main Channel Yellowstone River

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US Army Corps
of Engineers ®
Omaha District

Intake Dam
Lower Yellowstone Irrigation Project Modifications
Bypass Main Channel Sediment Impacts Analysis
CENWO-ED-HF
December 22, 2011

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Intake Dam – Main Channel Sediment Analysis

1. INTRODUCTION

A technical analysis was performed to evaluate potential impacts of the creation of a bypass channel around the dam at Intake, MT as it relates to the sediment processes in the main channel. The goal of the analysis was to determine if removing a percentage of flow from the main channel would cause deposition to occur beyond background levels behind the weir currently present at the site. This summary report discusses the evaluation and presents results of the analysis.

2. PROJECT DESCRIPTION

2.1 EXISTING CONDITION

The project at Intake consists of a low head diversion dam and headworks located on the Yellowstone River. Under current conditions, sediment deposition in the main channel does not impact operations of the structure. It is assumed that while some deposition may occur behind the structure under low flows, flood events serve to pass any entrained sediment over the diversion dam and down the Yellowstone River.

2.2 PROPOSED CONDITION

In order to facilitate fish passage around the diversion weir at Intake, it has been proposed that a bypass be constructed. The bypass would outlet just downstream of the existing diversion dam and have an upstream entrance at the location of an existing high flow chute several miles upstream from the diversion structure. Attractive flow is an issue with the bypass proposal, so flow diversions ranging from approximately 5 to 30% of total Yellowstone River flow have been evaluated. It is the goal of this evaluation to determine if the reduction in stream power of the Yellowstone River resultant from the bypass alternative will alter the existing sediment processes at the diversion site.

3. SEDIMENT DATA

Sediment data for the purpose of this evaluation was procured from two separate sources. The first being on-site samples collected by the USGS in 2008 and 2011. The second being gage data obtained from the USGS gaging station at Sidney, MT. Additional details on sediment data used can be found in the reference “Lower Yellowstone Project Fish Passage and Screening, Preliminary Design Report, Appendix A-2, Hydraulics.”

3.1SUSPENDED SEDIMENT DATA

Two sources of suspended data were run through the analysis to establish an upper and lower bound for the sediment loading at the project site. On-site samples collected in 2011by the USGS provided the lower bound for the simulation, while data collected from the USGS gaging station provided the upper bound. The USGS also collected sediment samples in 2008; however, the focus of the 2008 sampling effort was to evaluate sediment entering the canal through the headworks. Further details pertaining to the 2008 sampling effort can be found in the Hydraulic Appendix to the original EA.

There is a significant difference in magnitude between the two curves despite the lack of any major tributaries between the project site and the gaging station at Sidney. However, loading curves established from the on-site sediment collection efforts fall in the lower ranges of data taken from the gaging station at Sidney. Potential causes of the discrepancy include timing of collection of the on-site data (data was most often collected on the falling limb of event hydrographs) and differences in sampling methodologies. Also, as with any set of sediment data, there is great variability in the concentrations measured given a certain flow. The large selection of data points available from the Sidney gage provides greater certainty in the estimation of average sediment loadings in this portion of the Yellowstone.

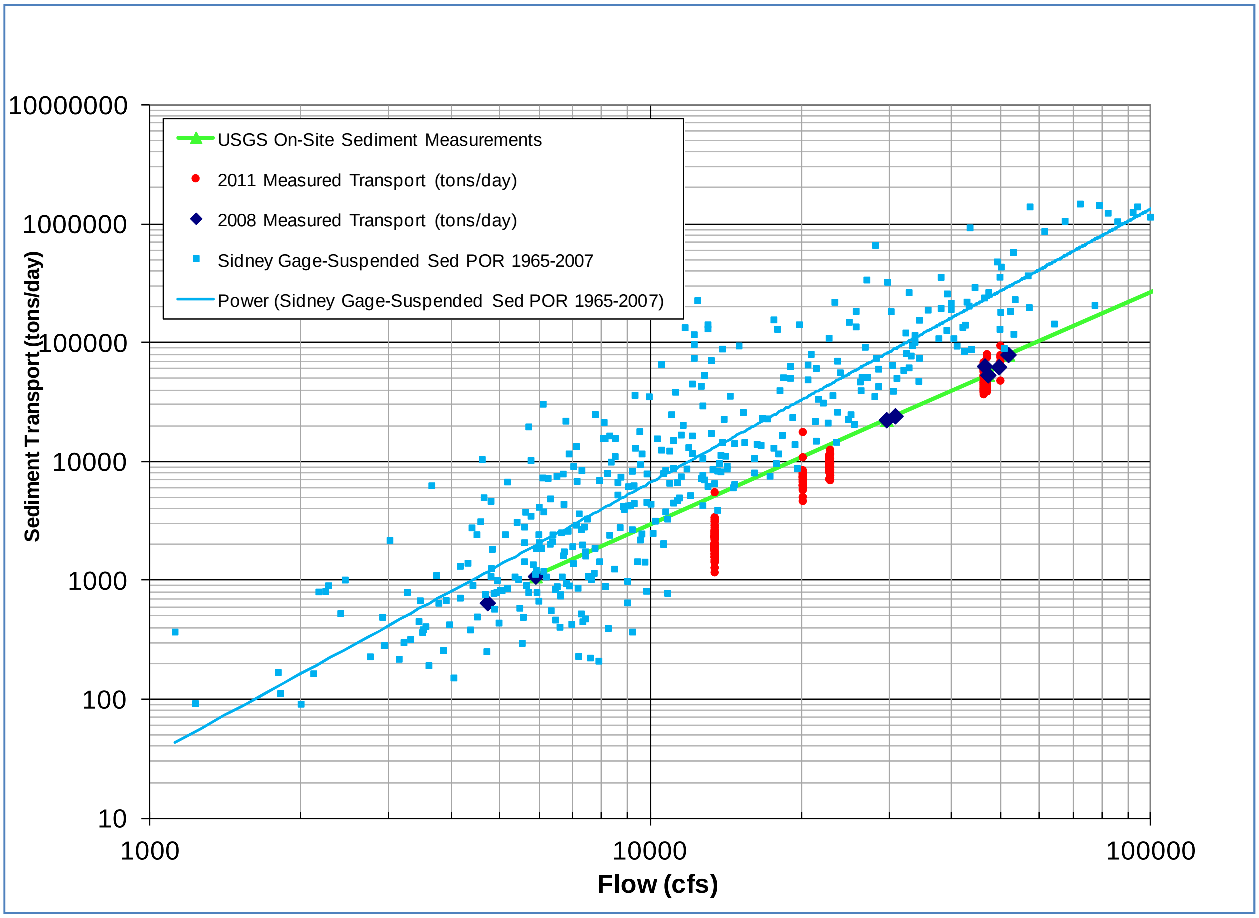

Figure 1 - Suspended Sediment Rating Curve

3.2 BED DATA

Makeup of the bed was determined from on-site samples taken in 2008 from multiple locations throughout the project site. For the purposes of the model, results from Wohlman counts were used to represent the bed in the HEC-RAS model. This bed makeup would be the most likely to resist degradation and represents the most conservative configuration.

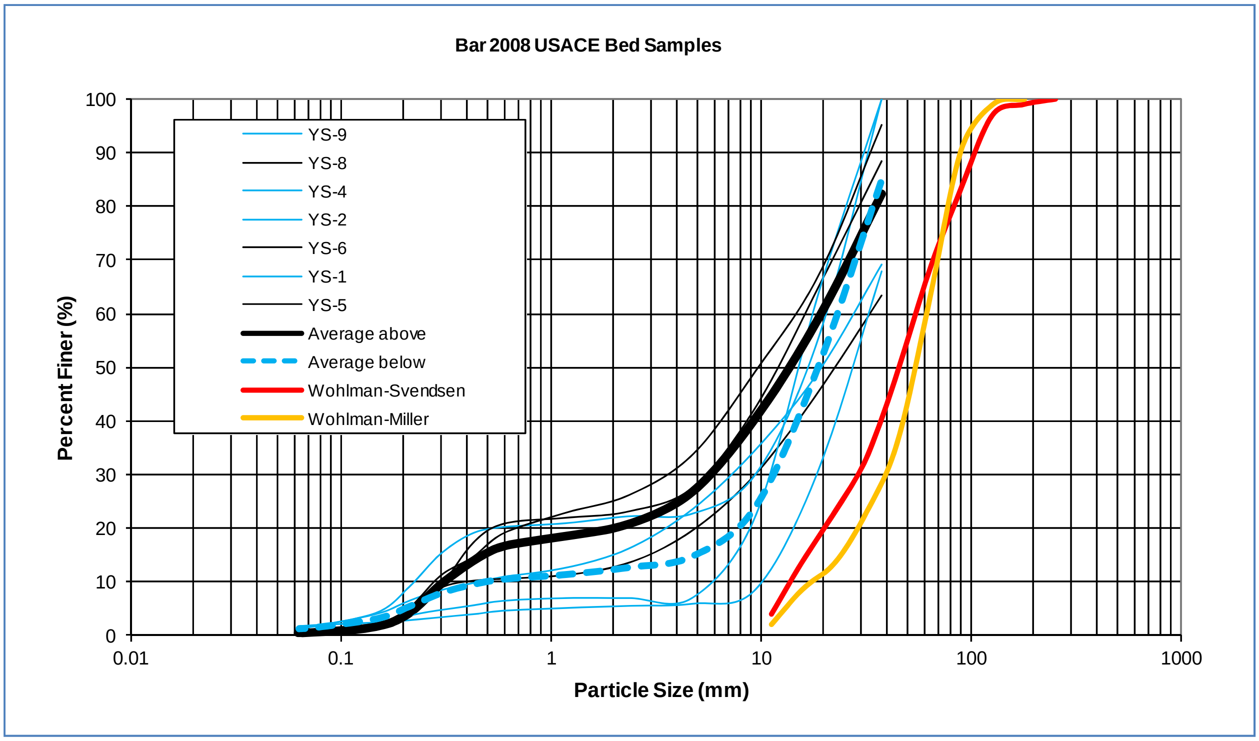

Figure 2 - Bed Gradation Curves

4. HEC-RAS MODELING

Analysis was performed to evaluate potential impacts of the creation of a bypass channel around the dam at Intake, MT as it relates to the sediment processes in the main channel. The analysis used HEC-RAS version 4.1.0 dated January 2010. A previously created existing conditions model was used as the base model. In order to facilitate use of the sediment transport function in HEC-RAS, the existing model was modified by removing all split flow junctions and removing sections to improve stability of transport simulations.

5. SEDIMENT TRANSPORT MODELING

Calibration data was not available for this modeling simulation. The approach was to simulate conditions that would most likely promote aggradation behind the dam. The bed layer was selected on the coarser side of the data available and two sources of suspended data were utilized to form upper and lower bounds of the expected loading at the project site.

5.1 QUASI-UNSTEADY FLOW

Current sediment capabilities in HEC-RAS are based on quasi-unsteady hydraulics. The quasi-unsteady approach approximates a flow hydrograph by a series of steady flow profiles associated with corresponding flow durations.

A 20-year simulation was used to evaluate long term trends in the project reach. Daily flow data from the USGS Sidney gage were downloaded, covering the time period from 27Sep1991 to 27Sep2011. An absence of major tributaries makes the Sidney gage a fair approximation of flow conditions at the project site.

Computation increments (CIs) between 0.1 hours and 24 hours were evaluated and varied based on the flow encountered at a given time step. Using a computation interval of 0.1 hours provided the best

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indication of model stability, but resulted in unacceptable run times when utilized over the entire flow range.

In order to simulate the installation of the bypass, flows from the Sidney gage were reduced by a flat percentage ranging from 5 to 30% for the various alternatives.

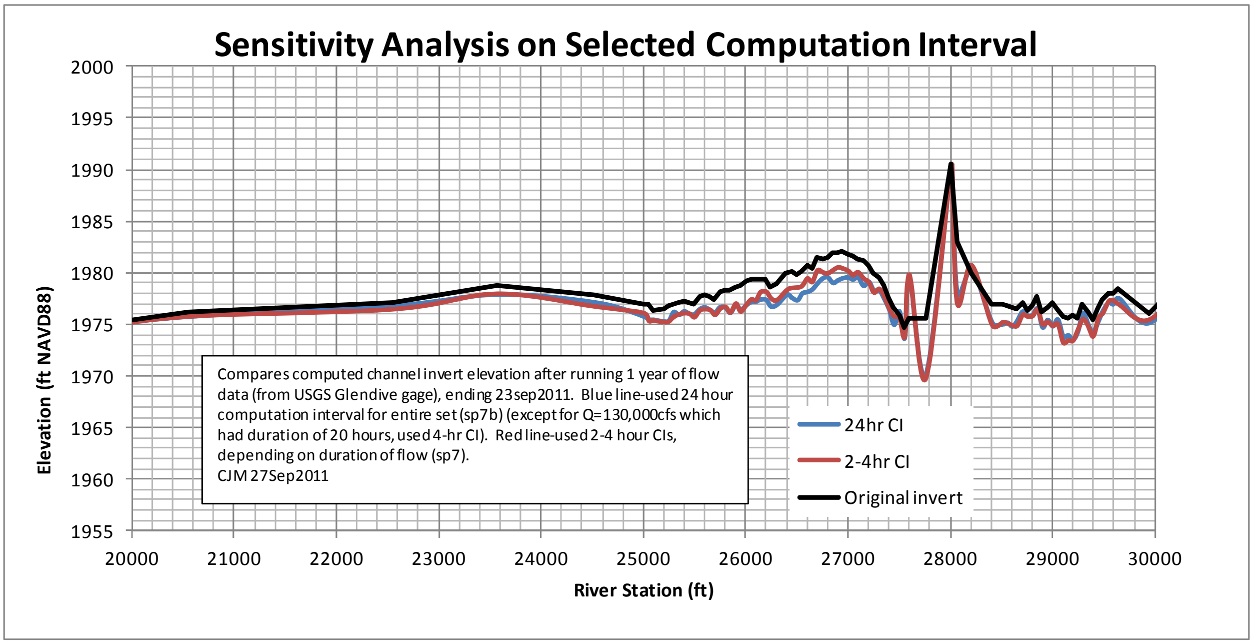

Figure 3- Sensitivity Analysis on Selected Computation Interval

5.2 Bed Gradation

Several sources of data were employed to determine the expected range of bed material gradations including bed and bank samples collected in 2008. During calibration runs, all bed materials resulted in a general degradational trend for the existing conditions. Wohlman counts from the 2008 samples were selected as the preferred gradation as they represented the most conservative condition with the goal in mind to identify aggradational potential.

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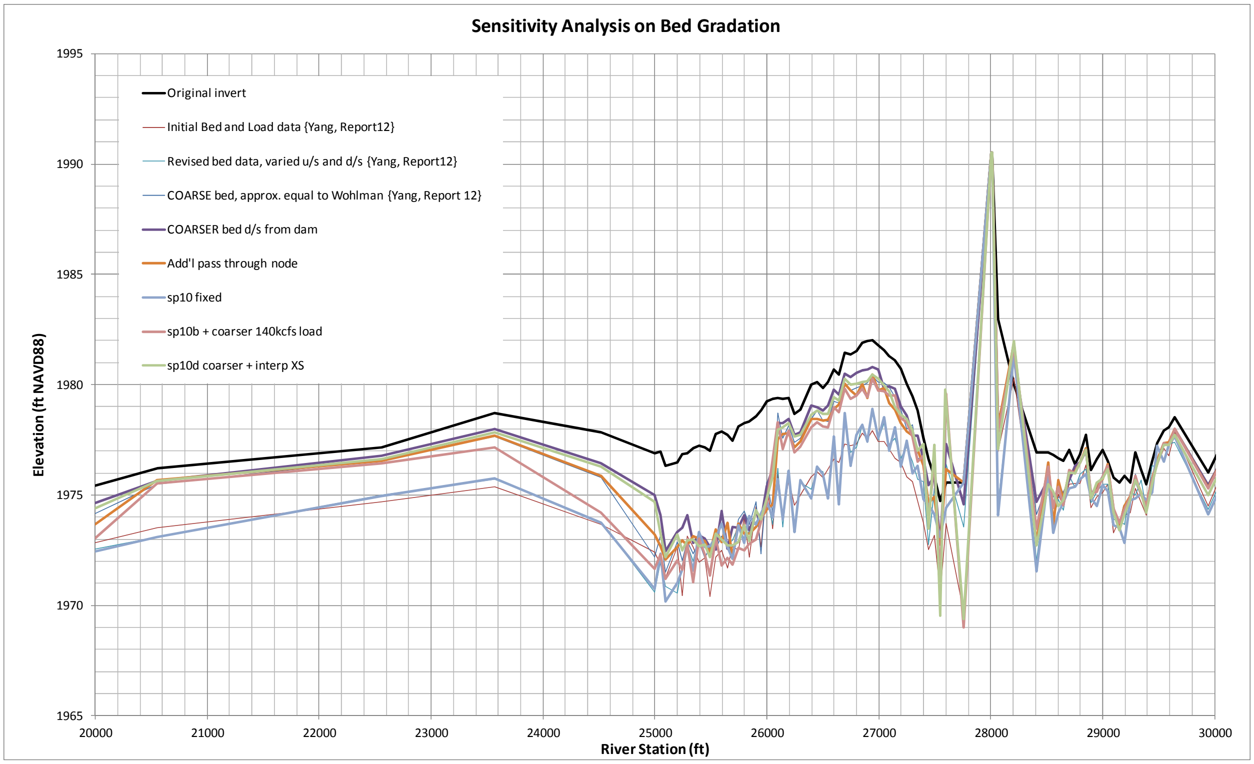

Figure 4 - Sensitivity Analysis on Bed Gradation

5.3 SEDIMENT LOADING

The suspended sediment load was input into the model utilizing a rating curve correlating total load to flow encountered. Two rating curves were used to define the upper and lower bound of expected load. Gradations for the two curves were assumed to be similar.

In order to simulate the installation of the bypass, the rating curve flows were shifted by the percentage of reduction in flow while total tons per day remained static. The assumption that no sediment load was diverted to the bypass was made to provide a conservative estimate of the effects of the bypass on sediment processes in the main channel.

Table 1 - USGS On-Site Sediment Rating Curve

Flow	5890	29600	47200	51800	140000
tons/day	1090	22470	53680	79650	500000
Clay					
VFM					
FM					
MM					
CM	0.82	0.78	0.76	0.67	0.55
VFS	0.06	0.07	0.08	0.12	0.15
FS	0.1	0.11	0.13	0.15	0.17
MS	0.02	0.03	0.03	0.06	0.08
CS					0.05

Table 2 - Sidney Gage Sediment Rating Curve

Flow	5000	20000	40000	50000	140000

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tons/day	1344.945	32756.46	161656.5	270261.9	2894847
Clay					
VFM					
FM					
MM					
CM	0.82	0.78	0.76	0.67	0.55
VFS	0.06	0.07	0.08	0.12	0.15
FS	0.1	0.11	0.13	0.15	0.17
MS	0.02	0.03	0.03	0.06	0.08
CS					0.05

5.4 Transport Function

The Laursen-Copeland transport function was selected because of its applicability to sediments in the silt range. Both the 2011 measured data and the Sidney data show over 60-90% of the material in suspension is finer than sand. Yang, Toffaleti, England-Hansen, and Achers-White also give reasonable results. Meyer-Peter Muller computes fairly significant aggradation, which is expected due to its tendency to underpredict the transport potential of finer materials.

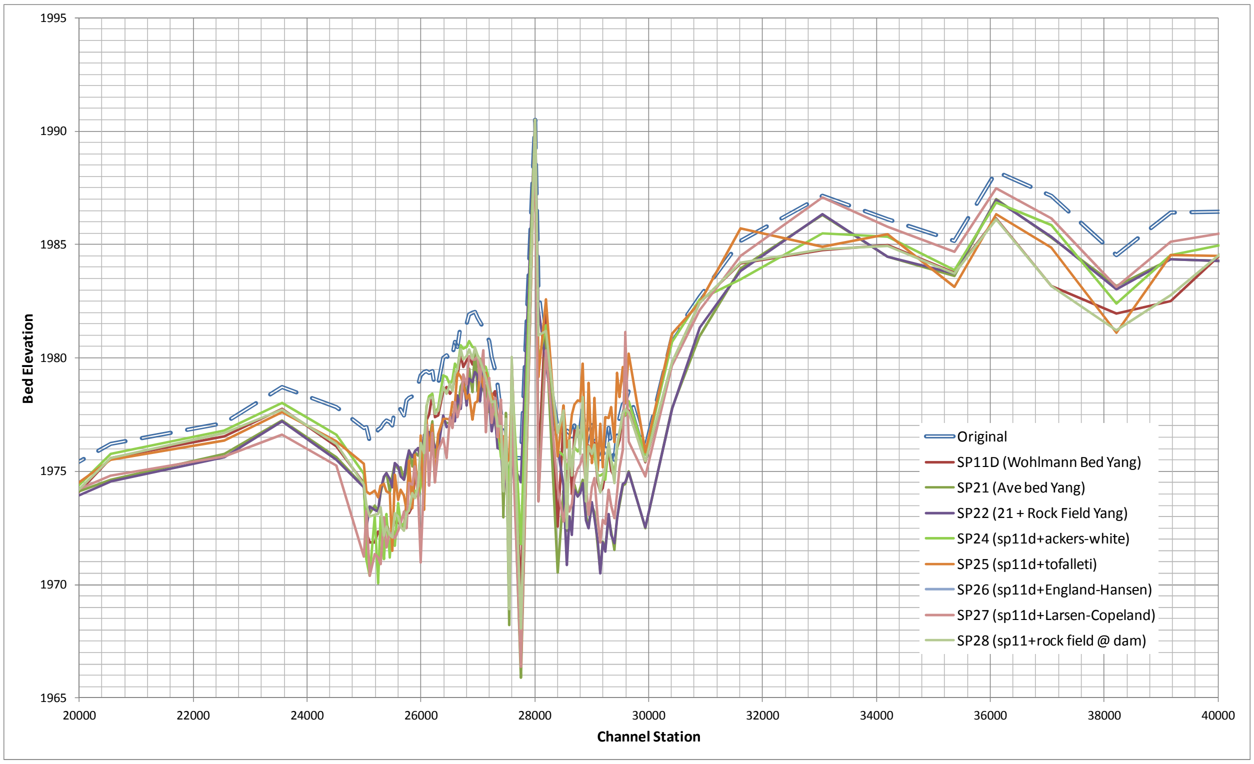

Figure 5 - Sensitivity to Transport Function

5.5 Sorting Method and Fall Velocity

The default methods for sorting and fall velocity were selected for this analysis. HEC-RAS does provide an alternative sorting method to the default; however it is intended only with use of the Wilcock transport method which was not utilized in this simulation.

Several methods are available for computing fall velocity as well. The default method in HEC-6 (Report 12) was selected for this simulation. A sensitivity analysis was not performed.

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6. RESULTS

Results varied greatly between use of the two sediment loading curves.

Using on-site sediment data, the channel showed little change in its transport potential up to a 30% reduction in total flow in the main channel. Reductions above the 30% threshold showed significant aggradation behind the diversion structure, though the channel remained stable as little as one-mile upstream from the diversion site. The diversion is located at approximate station 28000 on the plots.

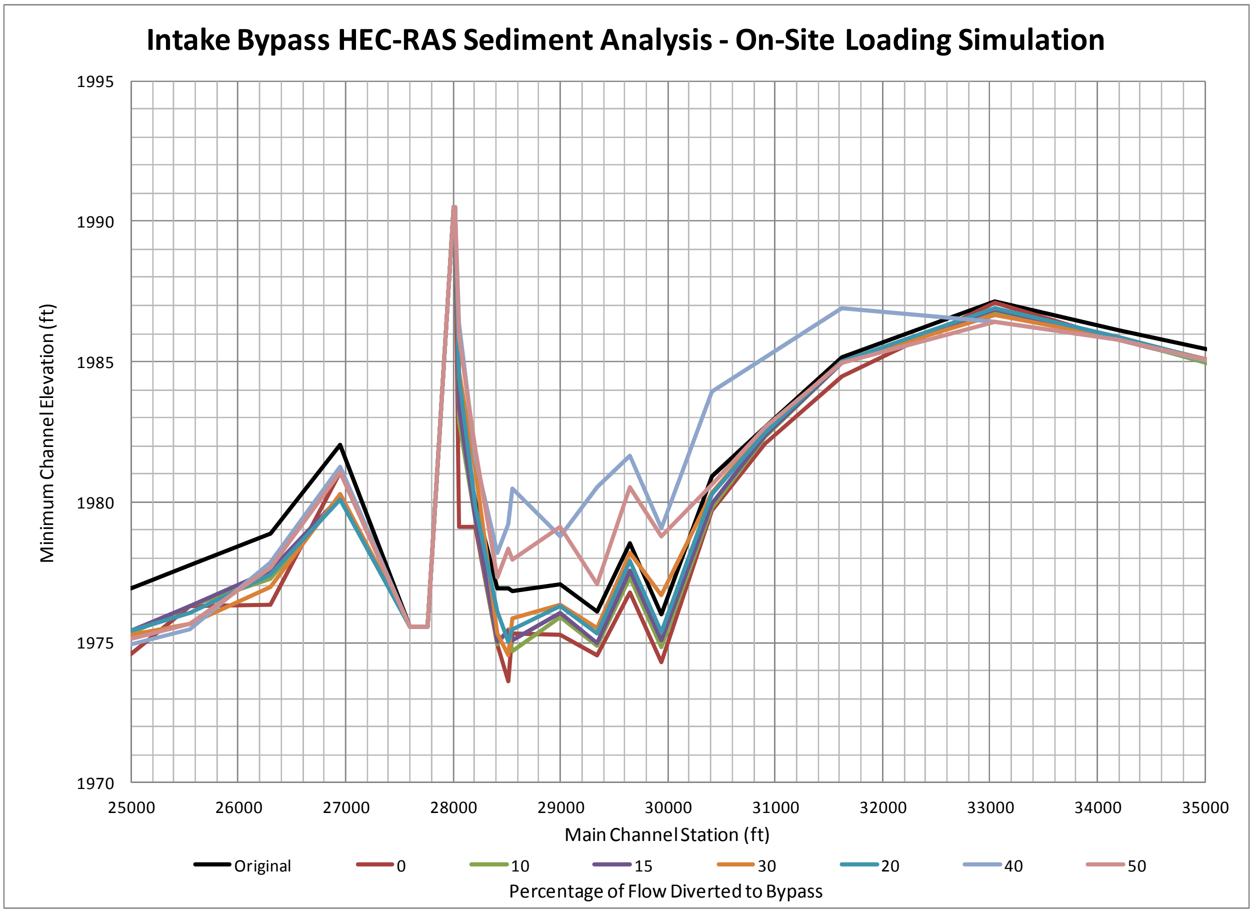

Figure 6 - Minimum Channel Elevation (On-Site Data Simulations)

When applying the Sidney gage sediment loading to the simulation, results showed slight aggradational potential even under existing conditions. However, this rate accelerated greatly once flows were reduced by 20%, leading to the conclusion that the 15% reduction should be the upper limit for targeted diversions.

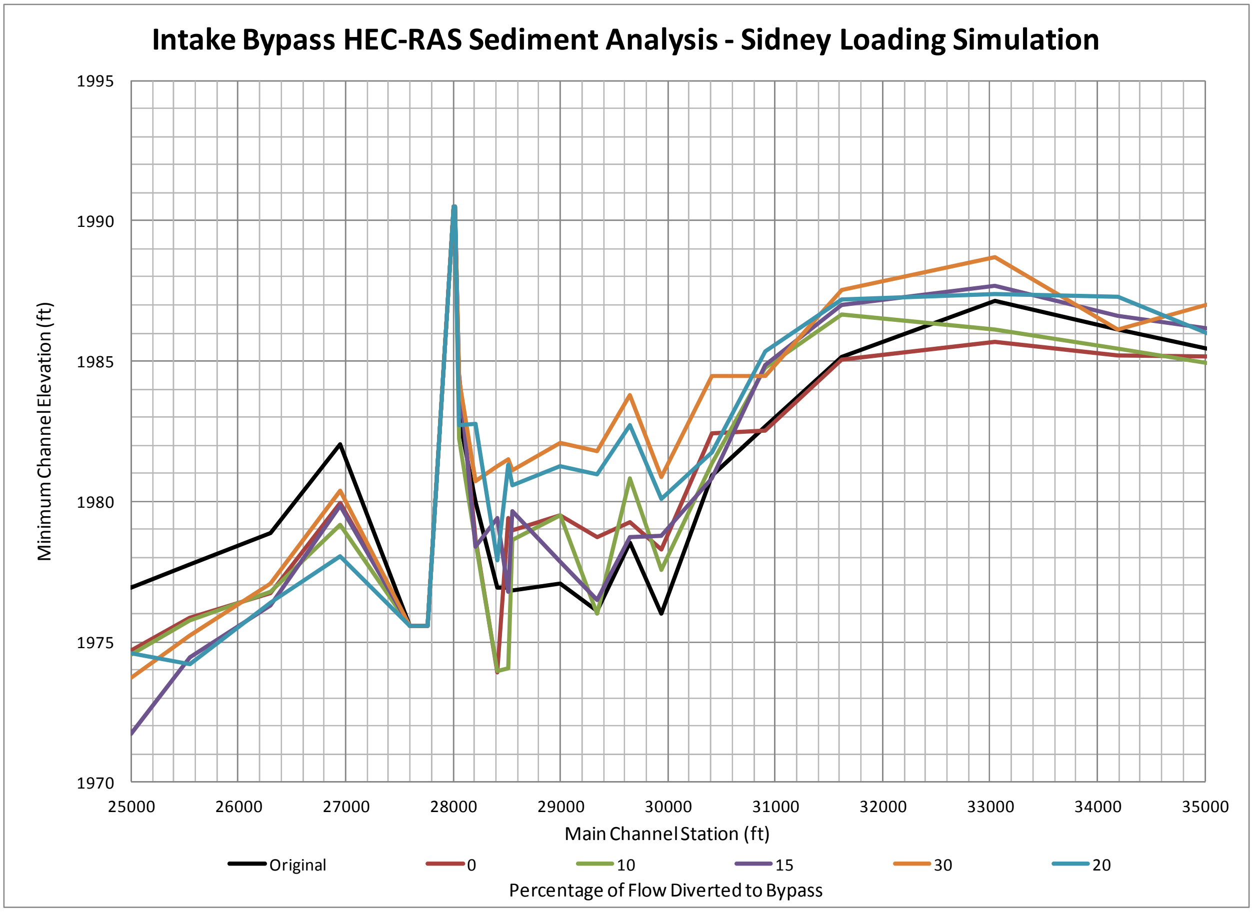

Figure 7 - Minimum Channel Elevation (Sidney Data Simulations)

7. RECOMMENDATIONS

It is paramount that the sediment dynamics of the diversion at Intake be unaltered from existing conditions following construction of the proposed bypass. Currently, the dam requires no in channel management of sediments.

If stream power were reduced to a point where sediments began to regularly accumulate in the vicinity of the headworks operations of the fish screens could be impeded, resulting in continued entrainment of fish species within the canal. Efforts to remove any sediments that accumulate would result in significant increases in annual O&M costs that the project is subject to.

Given the results of the two sediment loading analysis and the potential implications of any major alterations to the sediment processes at the site, it is recommended that a conservative approach be taken. Therefore, based upon the analysis utilizing Sidney gage suspended sediment loadings, a diversion of greater than 15% would present a risk to alter the sediment transport dynamics currently present in the Yellowstone River at the Intake Diversion project and should be the maximum amount of diversion for the proposed bypass.

8. REFERENCES

U.S. Army Corps of Engineer. CPD-68 “HEC-RAS, River Analysis System, User’s Manual, Version 4.1.” 2010.

U.S. Army Corps of Engineers. EM 1110-2-1601 “Hydraulic Design of Flood Control Channels.” 1994.

U.S. Army Corps of Engineers, Omaha District. “Lower Yellowstone Project Fish Passage and Screening, Preliminary Design Report, Appendix A-2, Hydraulics.” October 2009.

U.S. Army Corps of Engineers, Omaha District. “Trip Report 26-28August2008.”

Attachment 6 Bypass Channel

Appendix B

Sediment Analysis-Bypass Channel

19March2012

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Intake Dam

Lower Yellowstone Irrigation Project Modifications

Bypass Channel-Sediment Analysis
30% Design-Hydraulics

U.S. Army Corps of Engineers
Omaha District
DRAFT FEB2012

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1. INTRODUCTION

This report describes sediment evaluation and HEC-RAS modeling used in support of the design of a bypass channel at the Bureau of Reclamation's Lower Yellowstone Irrigation Project (commonly referred to as Intake). This report focuses on sediment modeling of the bypass channel itself; concurrent modeling of the main Yellowstone River is being conducted by USACE Omaha District Sediment and Channel Stabilization Section. The purpose of the bypass channel is to allow Pallid sturgeon (as well as other native species) to pass from downstream of the low head (≈8-10ft) structure to upstream.

The intent of this analysis is to reach approximately a 30% design level for the bypass channel. The bypass channel is one of two remaining alternatives currently being considered; the other is a flat slope (≈0.5%) rock ramp. The 30% design of the bypass channel is intended to allow for a fair comparison of cost estimates between the bypass channel and rock ramp. This report does not discuss the rock ramp alternative further.

2. BACKGROUND

Passage of the Pallid sturgeon around Intake Dam by means of a bypass channel has been discussed and evaluated for over a decade. This analysis uses best available information along with suggestions from U.S. Department of Interior, Bureau of Reclamation (Reclamation) to evaluate several bypass alternatives.

Criteria used to develop and evaluate the alternatives are based on suggestions from the Biological Review Team (BRT). The criteria used to develop alternatives include:

- A range of percentage of flow diverted from 10% to 35%
- Similar to previous evaluations of both the rock ramp and bypass channel, flow and depth ranges as shown in Table 1 were used based on BRT passage criteria. The preferred range for Pallid passage is depths greater than a meter with velocities lower than 4 ft/s.

Table 1 Depth and Velocity Ranges used for Evaluation

Depth range		Velocity range (ft/sec)				
(m)	(ft)	*0-2*	*2-4*	*4-6*	*6-8*	*>8*
0-0.5	*0-1.64*					
0.5-1.0	*1.64-3.28*					
>1.0	*>3.28*					

The above criteria were used to evaluate numerous alternatives based solely on hydraulics (i.e. no sediment modeling included). Three alternatives were selected representing 10%, 15%, and 30% diversion. Table 2 summarizes these three alternatives. Additional details on the initial evaluation were presented in a concept analysis in April 2011 (see Reference 5.)

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Table 2 ORIGINAL Bypass Channel Flow Splits and Configurations

Recurrence interval (annual, post-Yellowtail flows)	Total Yellowstone River discharge	BASE (existing right bank chute assuming new headworks with existing dam)		10% Diversion		15% Diversion		30% Diversion	
		Flow Splits for Base and Alternatives							
	(cfs)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)
<2-yr	3000	0	0	210	7	570	19	830	28
<2-yr	7000	0	0	750	11	1260	18	2540	36
<2-yr	15000	0	0	1600	11	2280	15	5280	35
<2-yr	20000	0	0	2120	11	2850	14	6930	35
<2-yr	25000	190	1	2640	11	3420	14	8410	34
<2-yr	30000	790	3	3170	11	3990	13	9840	33
2-yr	45300	2280	5	4970	11	5910	13	14210	31
5-yr	60600	4050	7	7190	12	7920	13	18540	31
10-yr	70100	5220	7	8670	12	8740	12	21110	30
20-yr	78700	6090	8	9830	12	10460	13	23520	30
50-yr	89400	7280	8	11410	13	11830	13	26480	30
100-yr	97200	8090	8	12600	13	12950	13	28480	29
500-yr	114000	9920	9	15620	14	15870	14	32710	29

Pertinent Bypass Channel Parameters	10% Diversion	15% Diversion	30% Diversion
Alignment	2	1	2
Bypass Channel Length (ft)	13550	15650	13550
Bypass Channel Longitudinal Slope	0.00059	0.00045	0.00059
Low Flow Channel Depth (ft)	2	N/A	2
Low Flow Channel Bottom Width (ft)	10	N/A	10
Low Flow Channel Side Slopes	1V:3H	N/A	1V:3H
Main Bypass Channel Bottom Width	50	61	300
Main Bypass Channel Side Slopes	1V:5H	1V:4H	1V:5H
Approximate Excavation Quantity (cubic yards)	650,000	950,000	2,460,000

3. HYDRAULIC MODELING

The analysis used HEC-RAS version 4.1.0 dated January 2010. A previously created existing conditions model was used as the base model. Using three different alignments, new cross sections were extracted from a LiDAR based digital terrain model (DTM) using Bentley's Microstation/InRoads software package.

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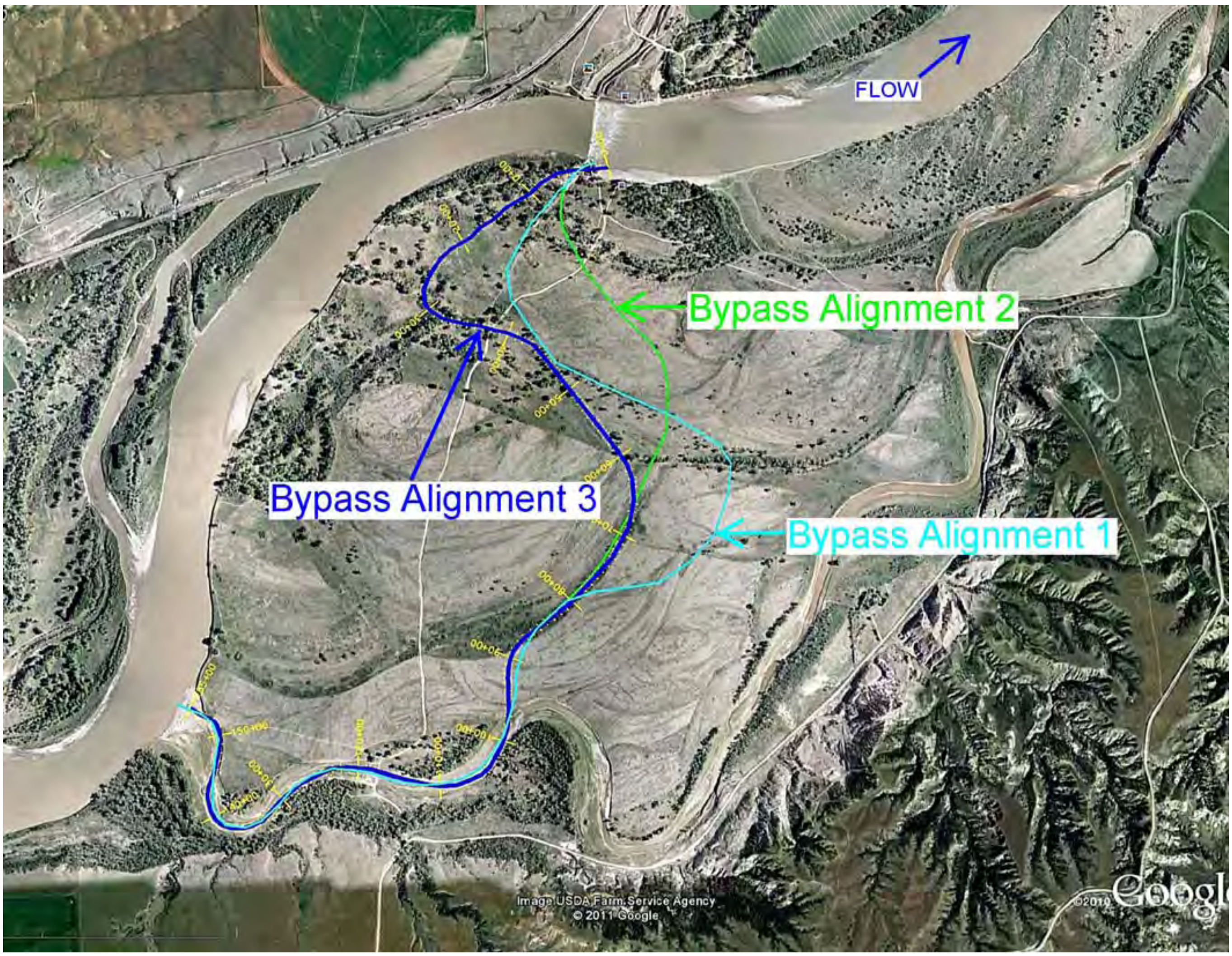
The three alignments were developed based on length required to obtain the desired channel slope as well as to minimize excavation quantities. The alignments are shown in Figure 1 and are hereafter referred to as Alignments 1, 2, and 3. Alignments 1 and 3 have similar lengths (≈15,500ft) and alignment 2 is slightly shorter (≈13,500ft). Alignment 3 was developed to maximize the use of historic channel scars and swales following a site visit in August 2011. Figure 1 shows Alignments 1 and 2 discussed in the original concept evaluation as well as the currently selected Alignment 3.

Figure 1 Potential Alignments



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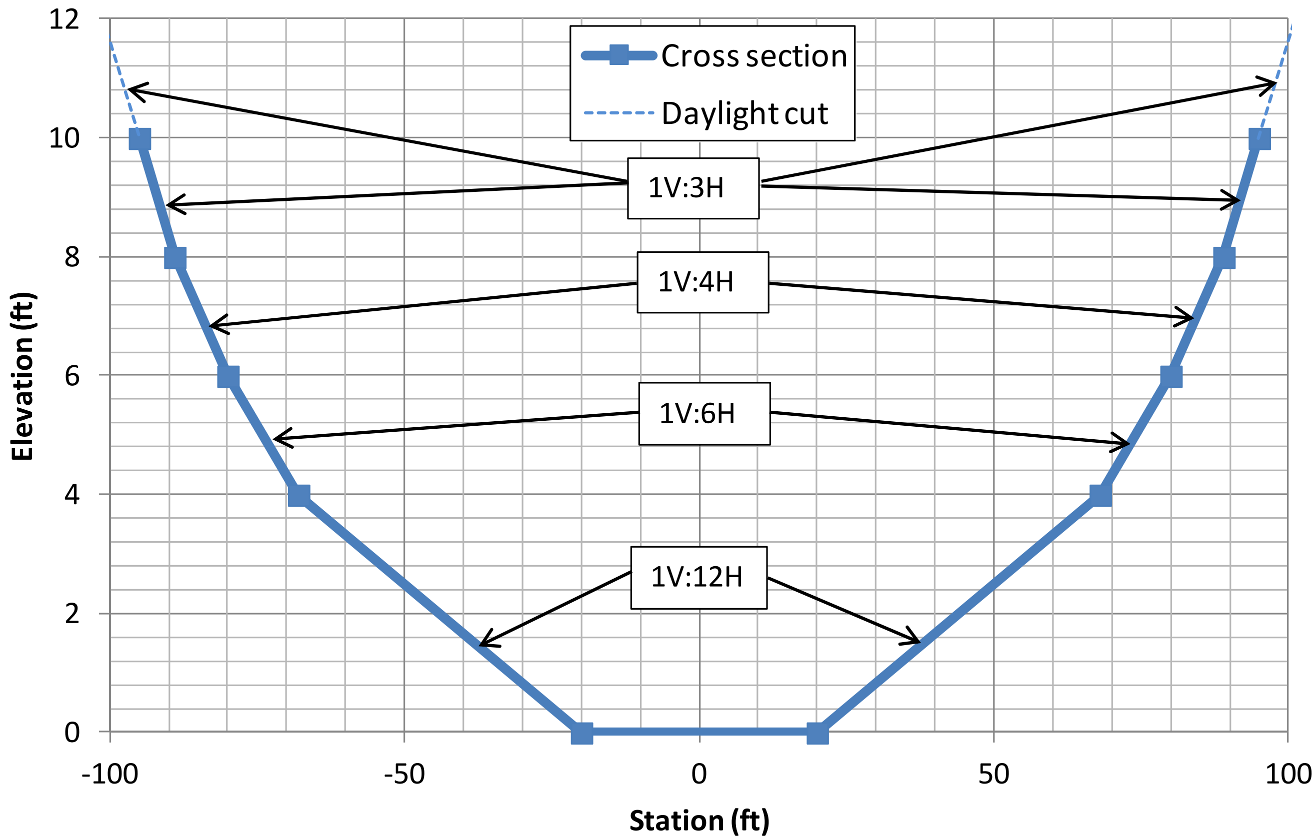
A large number of alternatives were developed to roughly determine flow splits between the Yellowstone River and the bypass channel. Three that were originally selected for further evaluation in April 2011 are summarized and compared in Table 2.

Following the concept analysis in April 2011, further coordination with Reclamation and the BRT led to a channel section with a 40 ft bottom width, side slopes varying between 1V:12H and 1V:3H with a longitudinal slope of 0.0006 ft/ft. This channel section is shown in Figure 2.

Figure 2 Channel Section



The channel configuration using Alignment 3 and the section shown in Figure 2 will hereinafter be referred to as the **15% base bypass alternative**. It diverts 10%-17% of Yellowstone River flows and is considered the 15% diversion alternative. Also evaluated were 10% and 30% diversion alternatives.

The 10% diversion alternative utilizes a cross section similar to that shown in Figure 2, but half the width (i.e. bottom width is 20ft, each side slope section only half as wide). The 30% diversion utilizes the same side slopes as those shown in Figure 2 with a 200ft bottom width.

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Table 3 summarizes the current bypass alternatives in the same format as Table 2 presented the original alternatives.

Table 3 Current Bypass Channel Flow Splits and Configurations

Recurrence interval (annual, post-Yellowtail flows)	Total Yellowstone River discharge	Flow Splits for Base and Alternatives: BASE (existing right bank chute assuming new headworks with existing dam)		10% Diversion		15% Diversion		30% Diversion	
	(cfs)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)	(cfs)	(%)
<2-yr	3000	0	0	220	7	310	10	890	30
<2-yr	7000	0	0	650	9	860	12	2220	32
<2-yr	15000	0	0	1550	10	2140	14	4770	32
<2-yr	30000	790	3	3220	11	4510	15	9290	31
2-yr	45300	2280	5	5180	11	7170	16	13720	30
5-yr	60600	4050	7	7340	12	9900	16	18130	30
10-yr	70100	5220	7	8770	13	11690	17	20780	30
20-yr	78700	6090	8	9990	13	13210	17	23240	30
50-yr	89400	7280	8	11540	13	14940	17	26260	29
100-yr	97200	8090	8	12650	13	16280	17	28170	29
500-yr	114000	9920	9	15570	14	19290	17	32490	29

Pertinent Bypass Channel Parameters	10% Diversion	15% Diversion	30% Diversion
Alignment	3	3	3
Bypass Channel Length (ft)	15500	15500	15500
Bypass Channel Longitudinal Slope	0.00060	0.00060	0.00060
Bypass Channel Bottom Width	20	40	200
Bypass Channel Side Slopes	Vary from 1V:12H to 1V:3H		
Approximate Excavation Quantity (cubic yards)	800,000	1,200,000	2,600,000

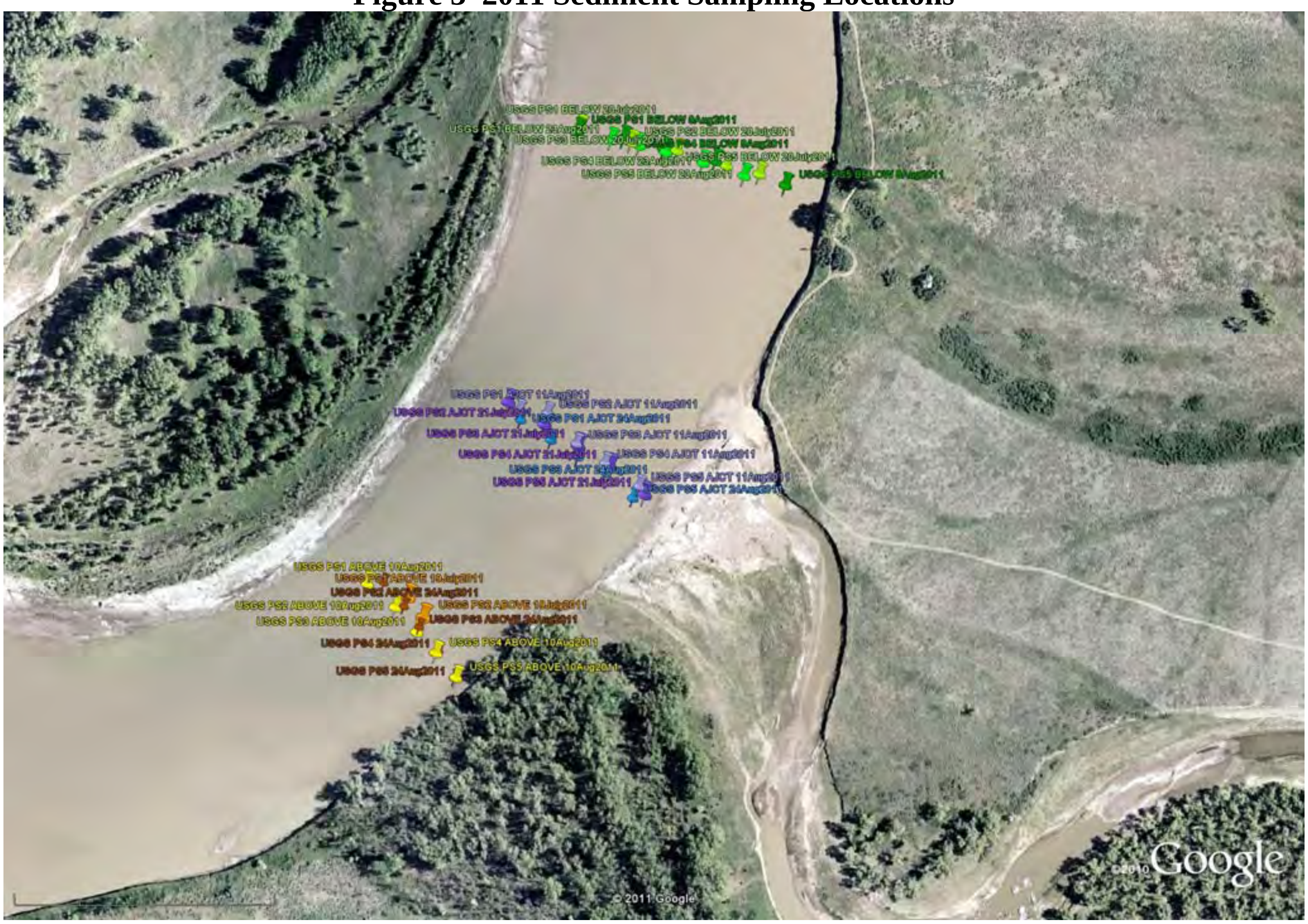
4. SEDIMENT DATA

Two sediment data collection efforts have been completed by the USGS as requested by USACE. The first effort occurred in 2008 and consisted of four sampling runs between 24June2008 and 28August2008. Each of these runs gathered suspended and bedload data at three locations: just upstream of Intake Dam, just downstream of Intake Dam, and in the irrigation canal just downstream from the old headworks. The intent of the 2008 effort was to provide increased knowledge of sedimentation processes in the immediate vicinity of the dam. More details concerning the 2008 sediment data can be found in section 2.8 of Reference 6. Reference 6 also provides some information on the sediment data available from the USGS gage (06329500) on the Yellowstone River at Sidney, MT (42 miles downstream). In addition to the

USGS data gathered in 2008, USACE gathered several grab samples from both the banks and in-channel bars. These grab samples were sent to the USGS lab for analysis. Details on these samples can be found in reference 7.

The second sediment data collection effort occurred in 2011, with three sampling runs occurring in July and August (19-21July, 9-11August, and 23-24August). Three locations were sampled during each run: adjacent to the upstream end of the existing high flow chute as well as just above and below (see Figure 3 for sample locations). The intent of the 2011 effort was to provide increased knowledge of the size and concentration of sediment, especially as it relates to vertical distribution. As such, point samples were taken at each cross section. Six point samples were taken in each of the five equal-discharge-increment verticals for a total of 30 point samples at each cross section during each sampling run.

Figure 3 2011 Sediment Sampling Locations



During the July sampling effort, flow was entering the existing right bank chute. However, because of equipment malfunction during collection of the upstream data and the limited number

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of samples, it is difficult to make conclusions on the impact of the existing chute on sediment transport.

Figures 4-6 show the suspended sediment concentration and D_{90} for the three cross sections based on the USGS data.
Figure 7 compares suspended transport for the Sidney gage with the measured data.

Figures 8 and 9 show the suspended load particle size distributions for the 2008 and 2011 data as well as the Sidney data.

Figure 4 Relative Concentration and D_{90} Upstream from High Flow Channel

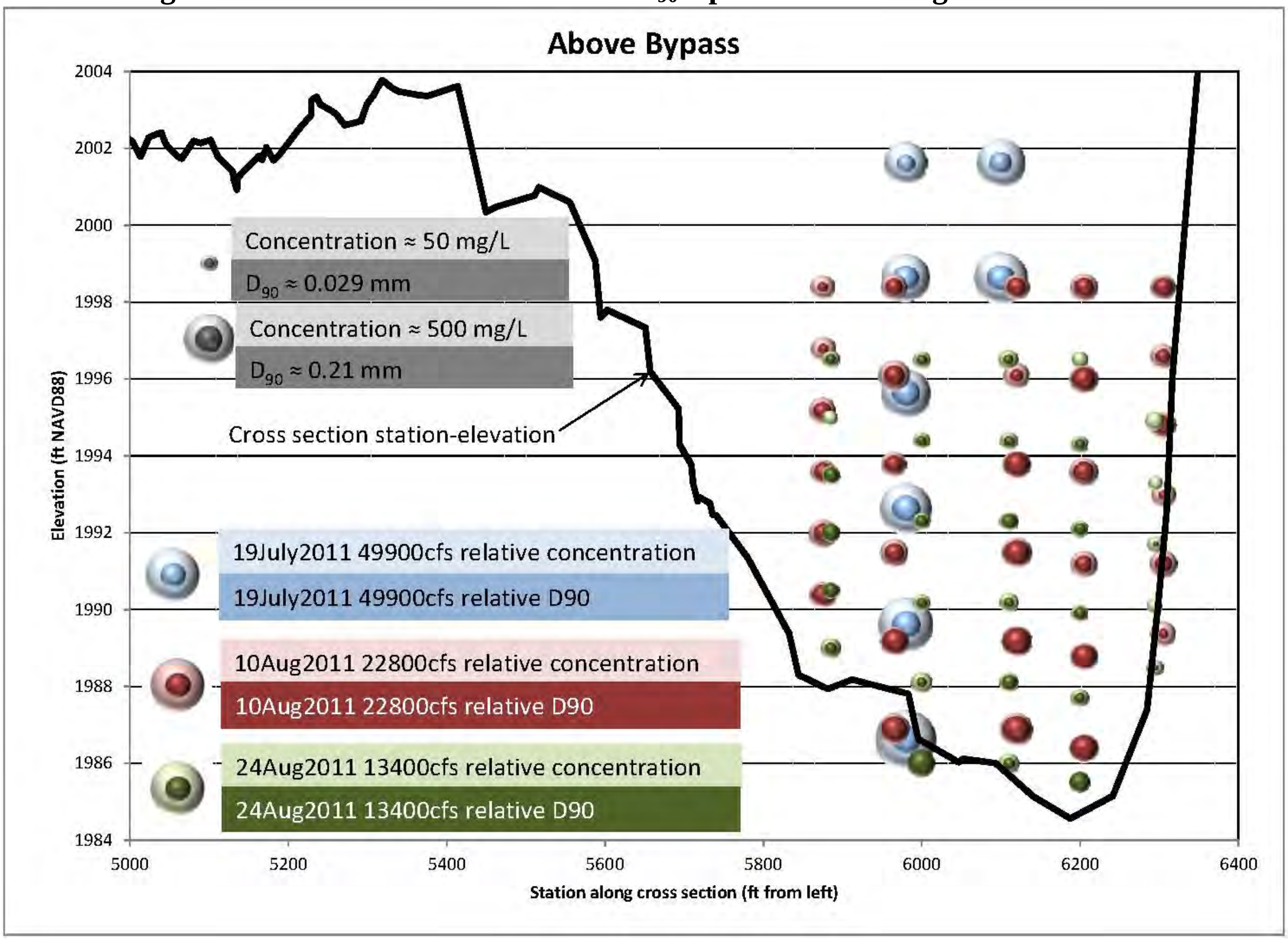

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Figure 5 Relative Concentration and D_{90} Adjacent to High Flow Channel

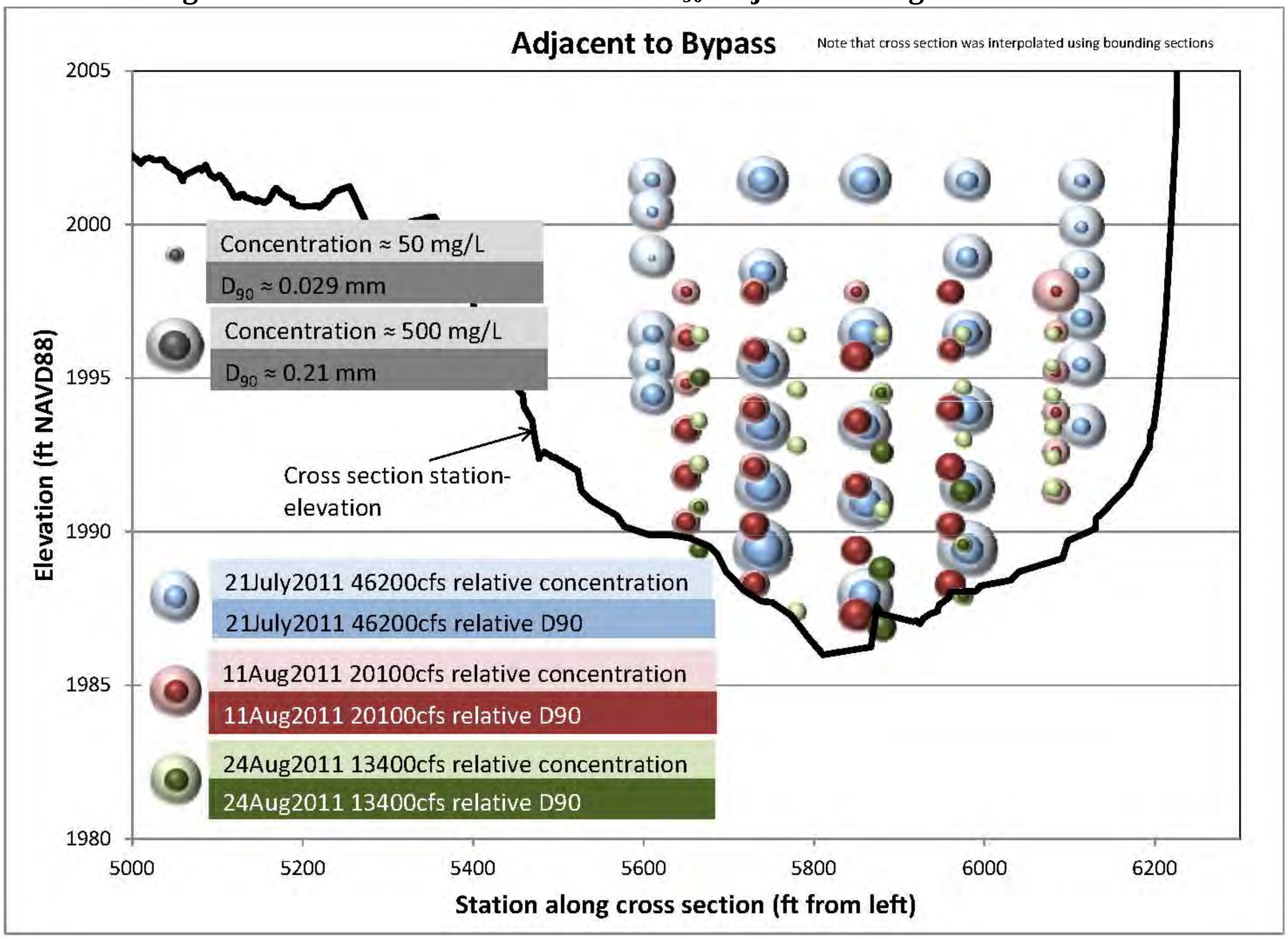

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Figure 6 Relative Concentration and D_{90} Downstream from High Flow Channel

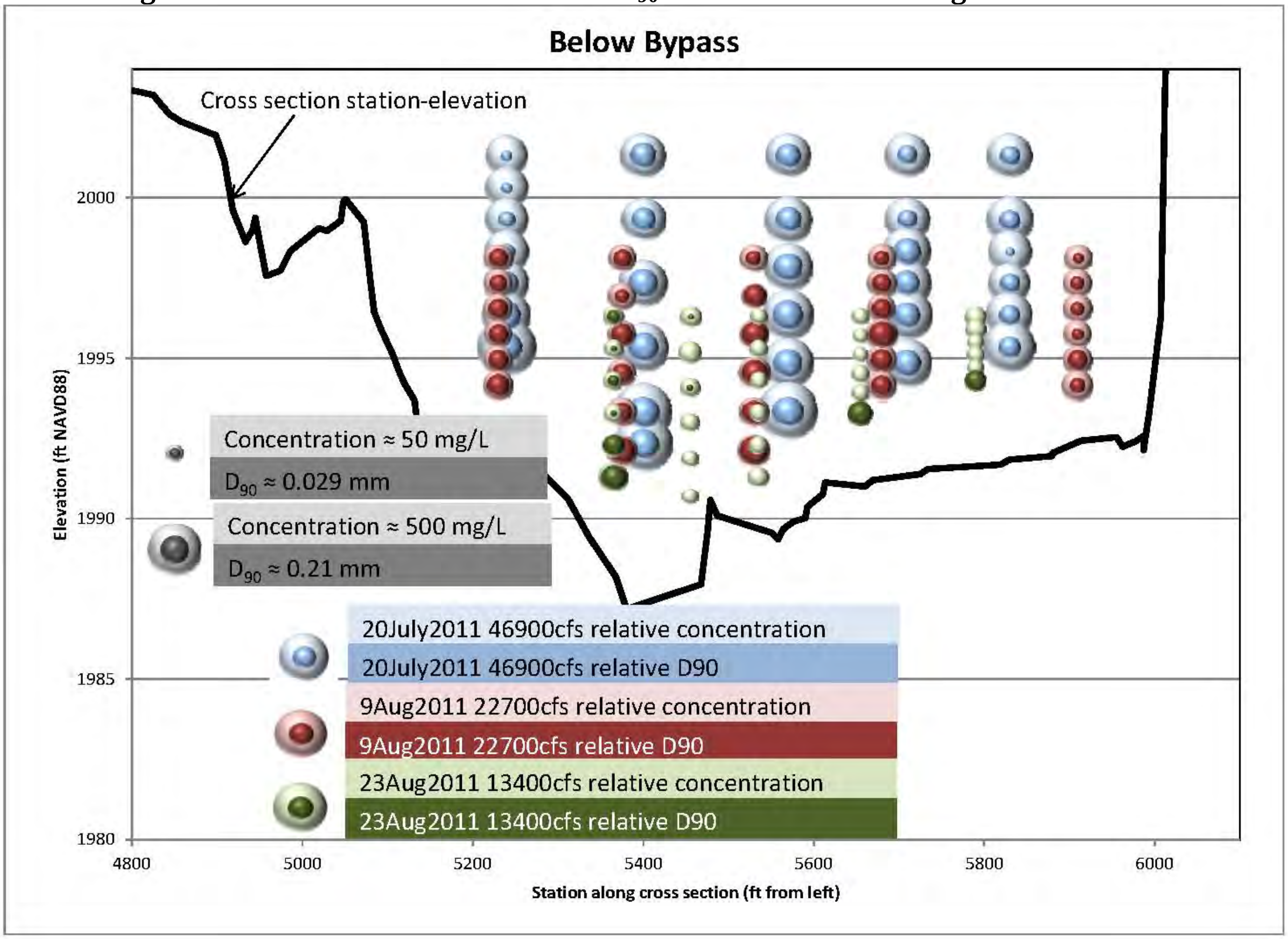

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Figure 7 Suspended Sediment Transport

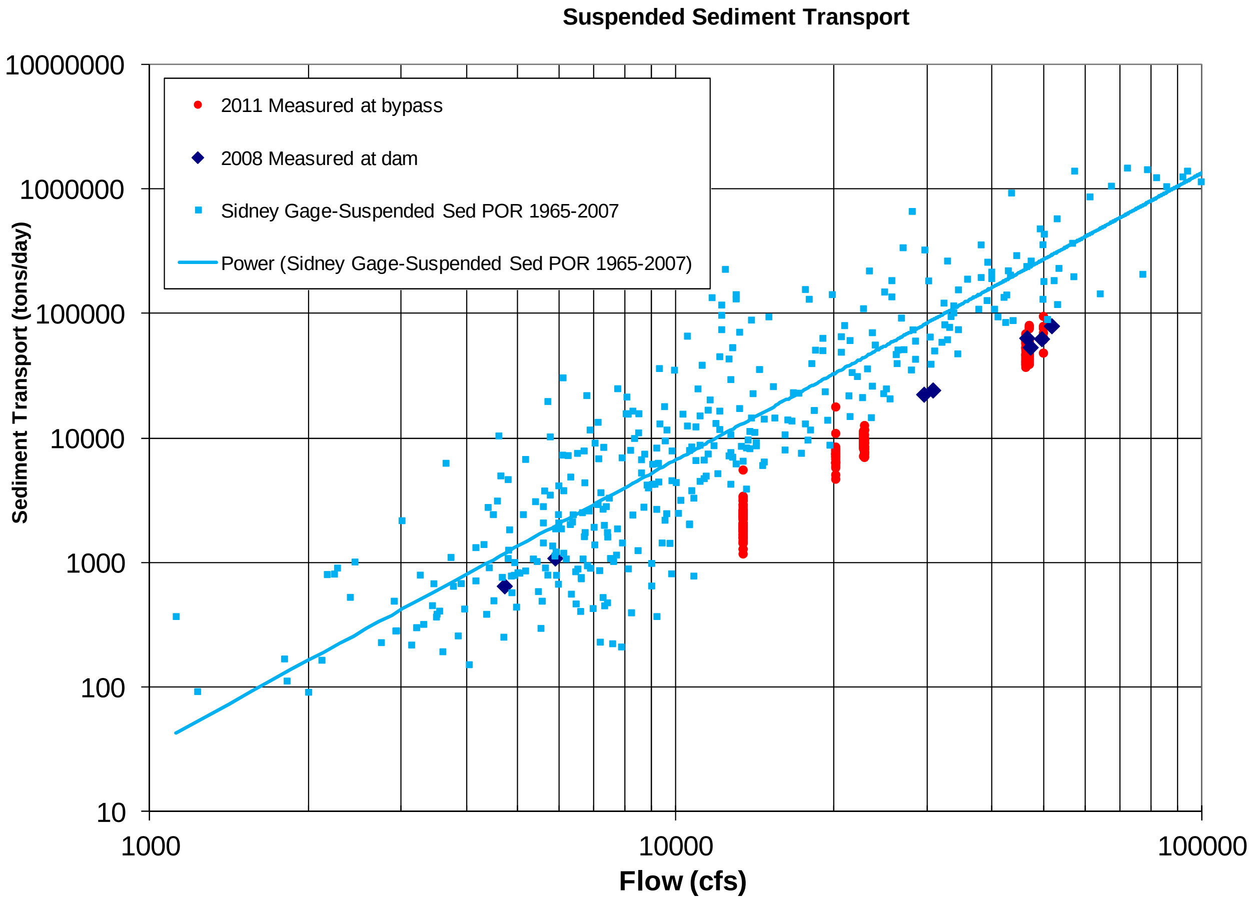

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Figure 8 Suspended Load Particle Size Distribution, Measured Data

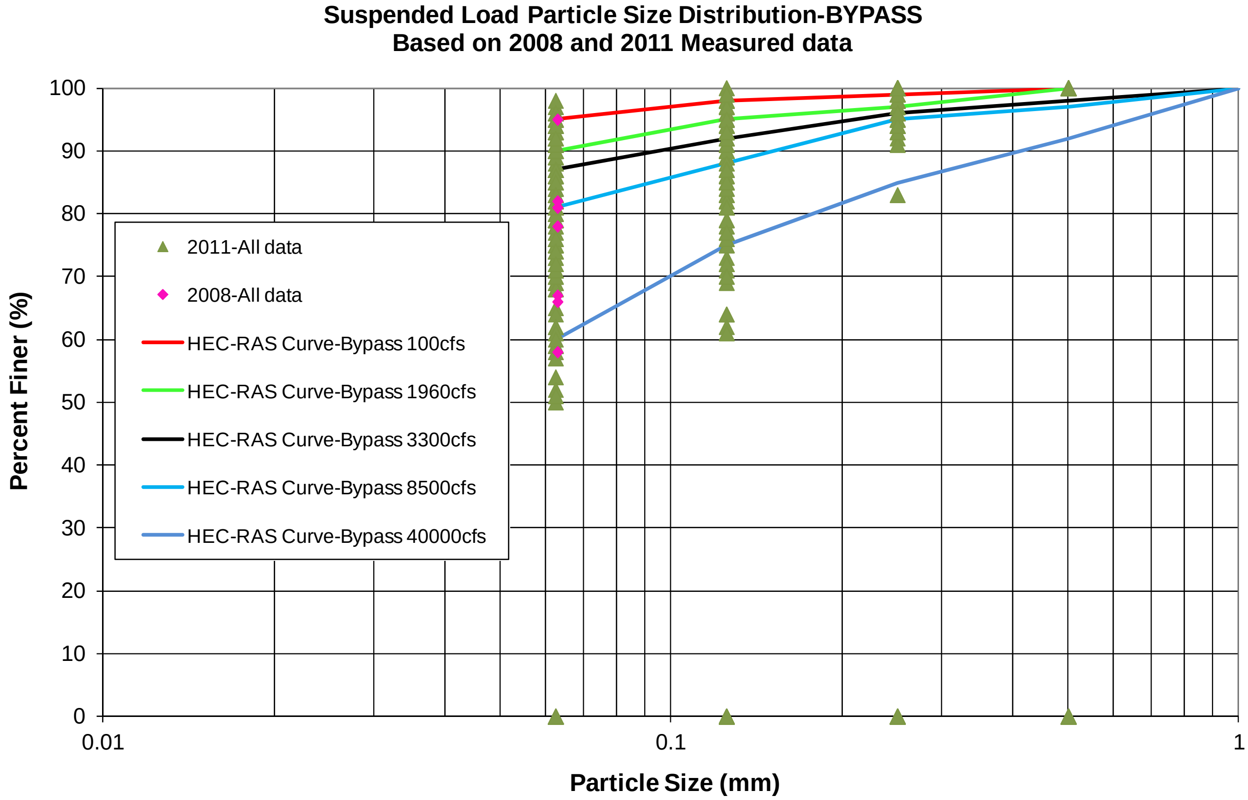

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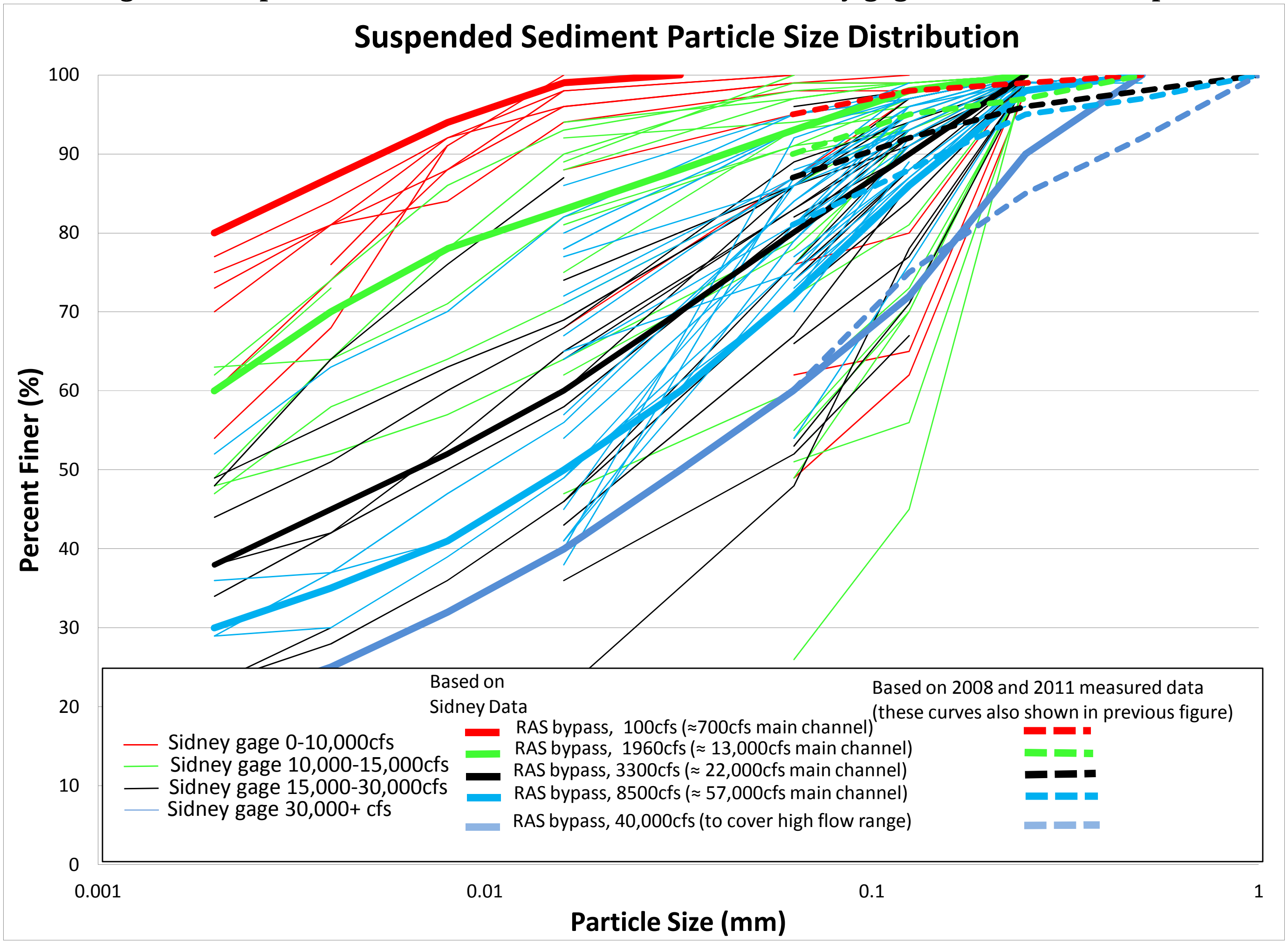
Figure 9 Suspended Load Particle Size Distribution, Sidney gage, for HEC-RAS Input



In addition to the point samples, the USGS gathered bedload data during each of the sampling runs. Total bedload as reported by the USGS is given in Table 4. Measured bedload gradation from 2008, 2011, and the Sidney gage are compared in Figure 10.

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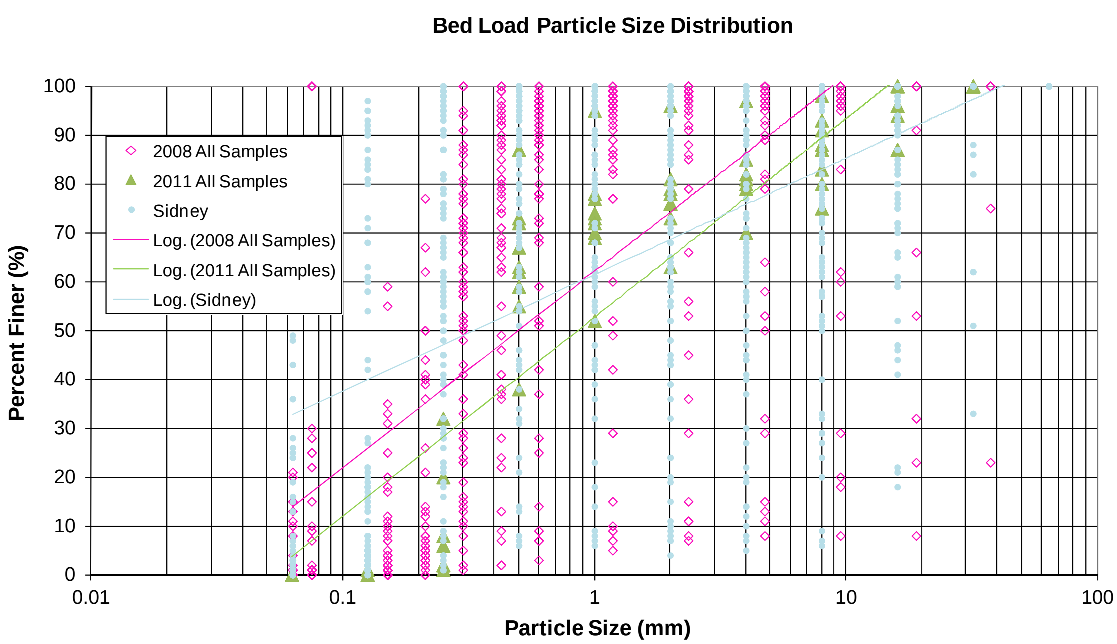
Table 4 Bedload Transport

Location	DATE	Discharge (cfs)	Bedload (tons/day)
Above upstream end of high flow channel	7/19/2011	49900	254
	8/10/2011	22800	36
	8/24/2011	13400	5
Adjacent to upstream end of high flow channel	7/21/2011	46200	255
	8/11/2011	20100	53
	8/24/2011	13400	20
Below upstream end of high flow channel	7/20/2011	46900	301
	8/9/2011	22700	96
	8/23/2011	13400	55
Just upstream from dam	6/18/2008	29600	829
	6/25/2008	51800	836
	7/9/2008	47200	738
	8/27/2008	5890	3
Just downstream from dam	6/17/2008	30800	571
	6/24/2008	49600	0
	7/8/2008	46500	1524
	8/26/2008	4720	36
Irrigation canal just below headworks	6/19/2008	1130	1
	6/26/2008	1310	0
	7/10/2008	1350	1
	8/28/2008	1050	0

Figure 10 Bed Load Particle Size Distribution



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Test pits and borings were also gathered from the island area in the vicinity of the proposed alignments in order to provide increased knowledge about the potential bed material to be encountered upon excavation.

Figure 11 shows test pit (TP) locations.

A narrative describing the test pit digging follows (per email from hole logger, John Hartley):

> Lithology
>
> An upper zone 3-8+ ft thick comprising silt to silt with very fine sand to very fine sand. This layer was not present in TP-1. Occasionally stringers or thin beds of coarser sands would be observed in the side wall but 1.5cy bucket sampling just doesn't capture nuance. Also some clay both in the matrix and accessional as blobs in the bucket..lens or thin layer? Walls stand up until undermined at which time they fall down fairly rapidly. Essentially no cohesion. Overbank flood deposits
>
> Underlying the silty layer was a unit of very rounded river gravel and cobbles (1-5 inches diameter with 2-3 inches being predominant). Usually the matrix was silt to very fine sand, usually mostly silt. Bimodal distribution of the very coarse and very fine. Other zones had a well graded matrix with silt to very coarse sand and the gravels. Gravel was anywhere from about 40% est of the unit to > probably 80%. All could be generalized as channel gravel with a fine grained non-cohesive matrix. It may take drag lines to effectively excavate I'm afraid.
>
> Tim was 100% correct in predicting that test pits would rapidly turn into sink holes once they got past the water table. TP 5 for some strange reason was dry and we got close to 25 ft out of it. In the units with higher percentages of gravel the material was usually saturated and basically flowed when dumped from the bucket resulting in pure gravel and useless samples. Got some pictures. In most places the water poured in, in a few...I believe tp7 it came in slower but it still came. In TP1 head was sufficient to cause boils during excavation and the backfilled excavation was quick. We added trees to the surface because until that settles a person walking into would not get out without help. When the water poured in the matrix washed out, the gravel collapsed, and the sink hole gr. Usually after 2-3 ft below the water table additional excavation was just an exercise in keeping up with caving so most holes terminated around 12-15 ft
>
> The entrance to the channel adjacent to the Yellowstone is armored with imbricate cobbles in the 3-5 inch range with smaller clasts infilling the voids. The same material was found throughout the TP-1 section but with matrix material included. Probably a case of the river bedload during flood being the very coarse material (the reason why we didn't get anything in the sediment sampling bedload samples, they didn't fit??) with the finer material being contributed over the years during lower flow or lesser flood stages.
>
> Bottom line. Lower coarse grained highly permeable saturated channel deposits overlain by overbank and flood deposits. Need to check the elevations once the "logs" get plotted to see how thinks line out but gut check says the top of the gravel layer was around the same place over the whole island suggesting the Yellowstone is happy with it's channel bottom elevation there. On the downside all measurements were pure eyeball due to excavation safety. On the upside the operator and I usually saw things the same or within a foot so we should be somewhat close.

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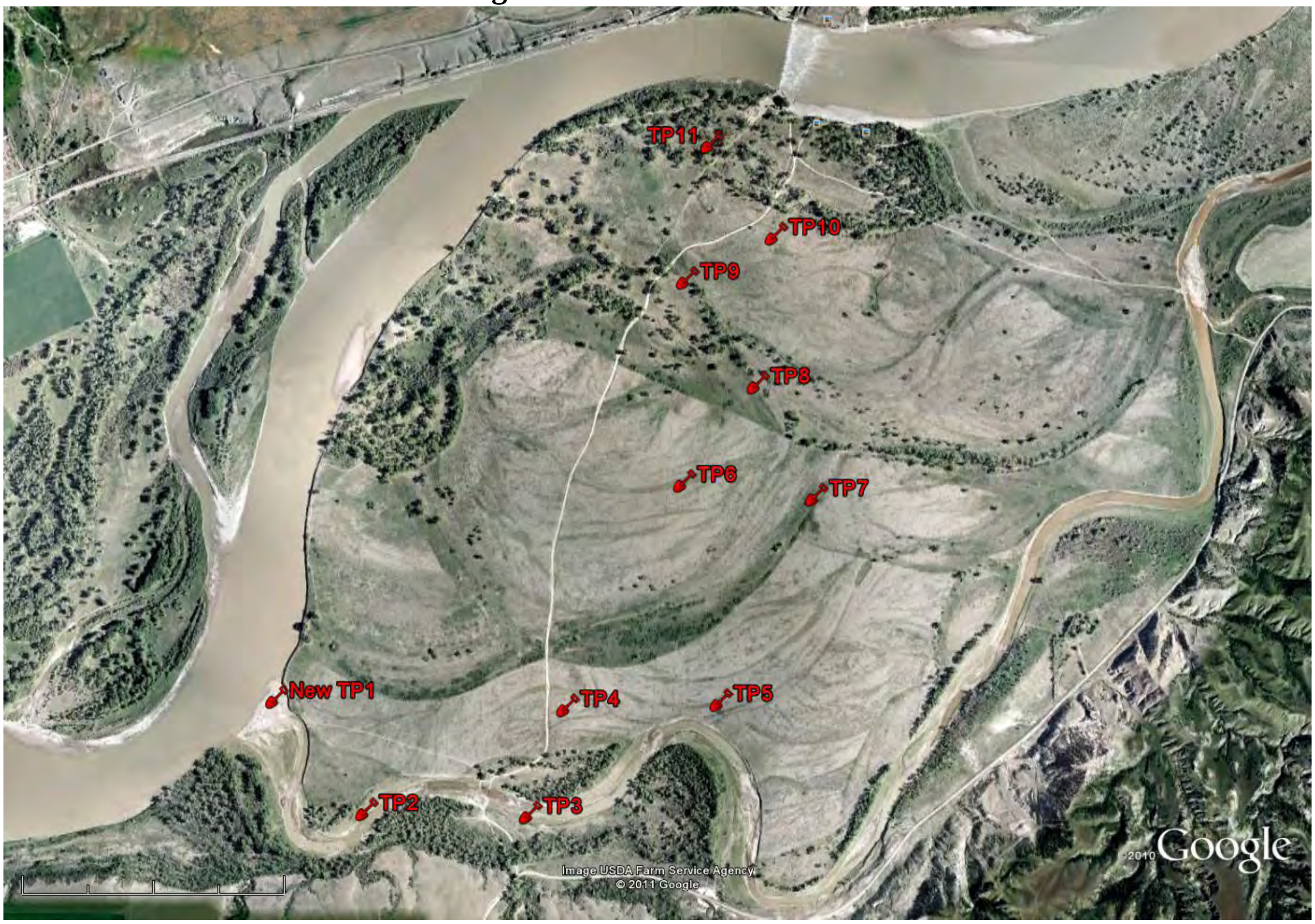
Figure 11 Test Pit Locations



Test pit particle size distributions are plotted in Figure 12 along with bar and bank particle size distributions from samples gathered in 2008. Details pertaining to the 2008 sampling effort are included in Reference 7.

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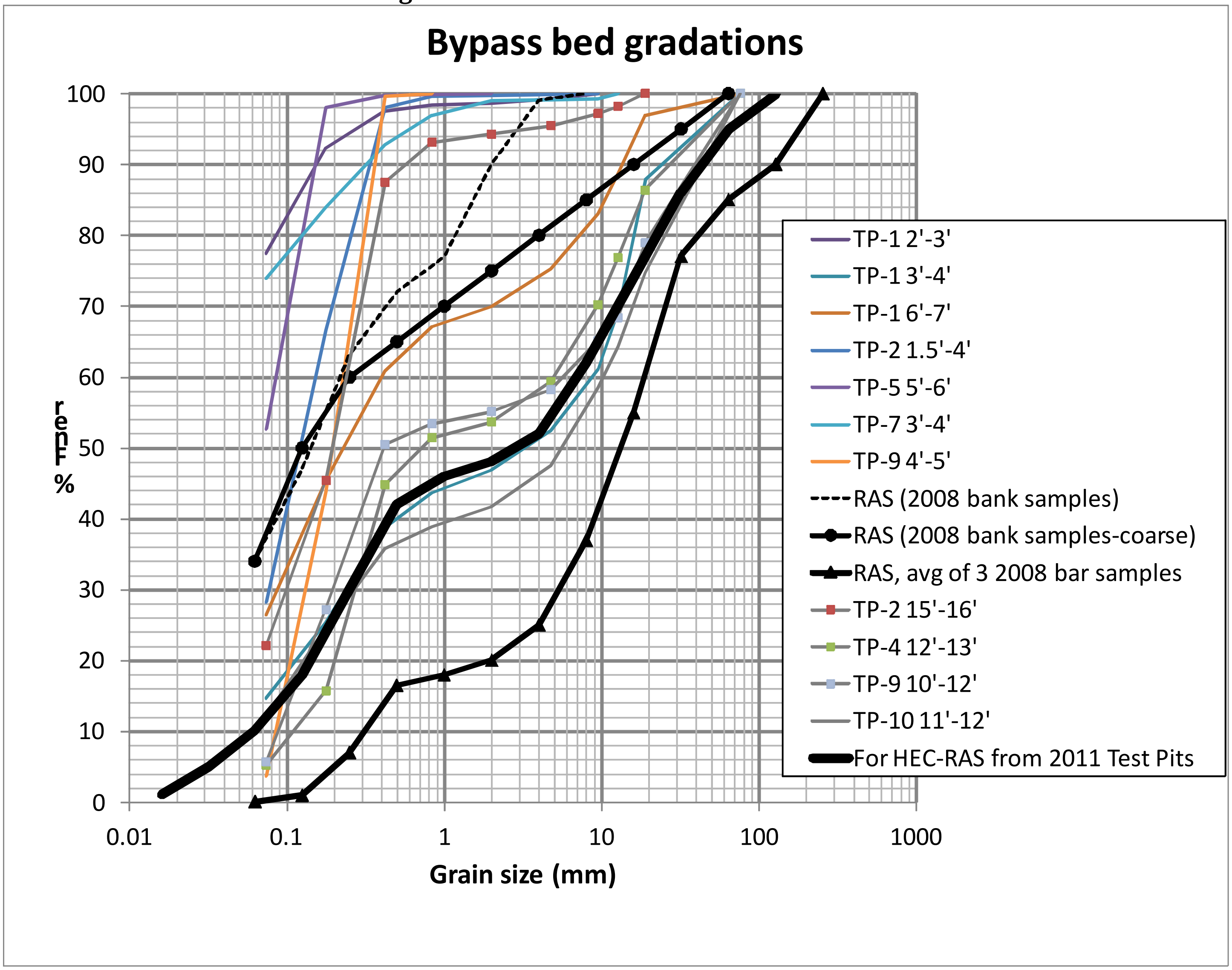
Figure 12 Bed Particle Size Distributions



5. SEDIMENT MODELING OF SELECTED BYPASS CHANNEL

5.1 Quasi-Unsteady Flow

A 20-year simulation was used to evaluate long term trends in the bypass channel. Daily flow data from the USGS Sidney gage were downloaded, covering the time period from 27Sep1991 to 27Sep2011. The flow data from the Sidney gage was then reduced according to the flow splits given in Table 3. These reduced flows were then entered into the quasi-unsteady flow file using a flow duration of 24 hours.

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The downstream boundary condition was set to a rating curve based on a separate split flow HEC-RAS model that contains the Yellowstone River and bypass channel.

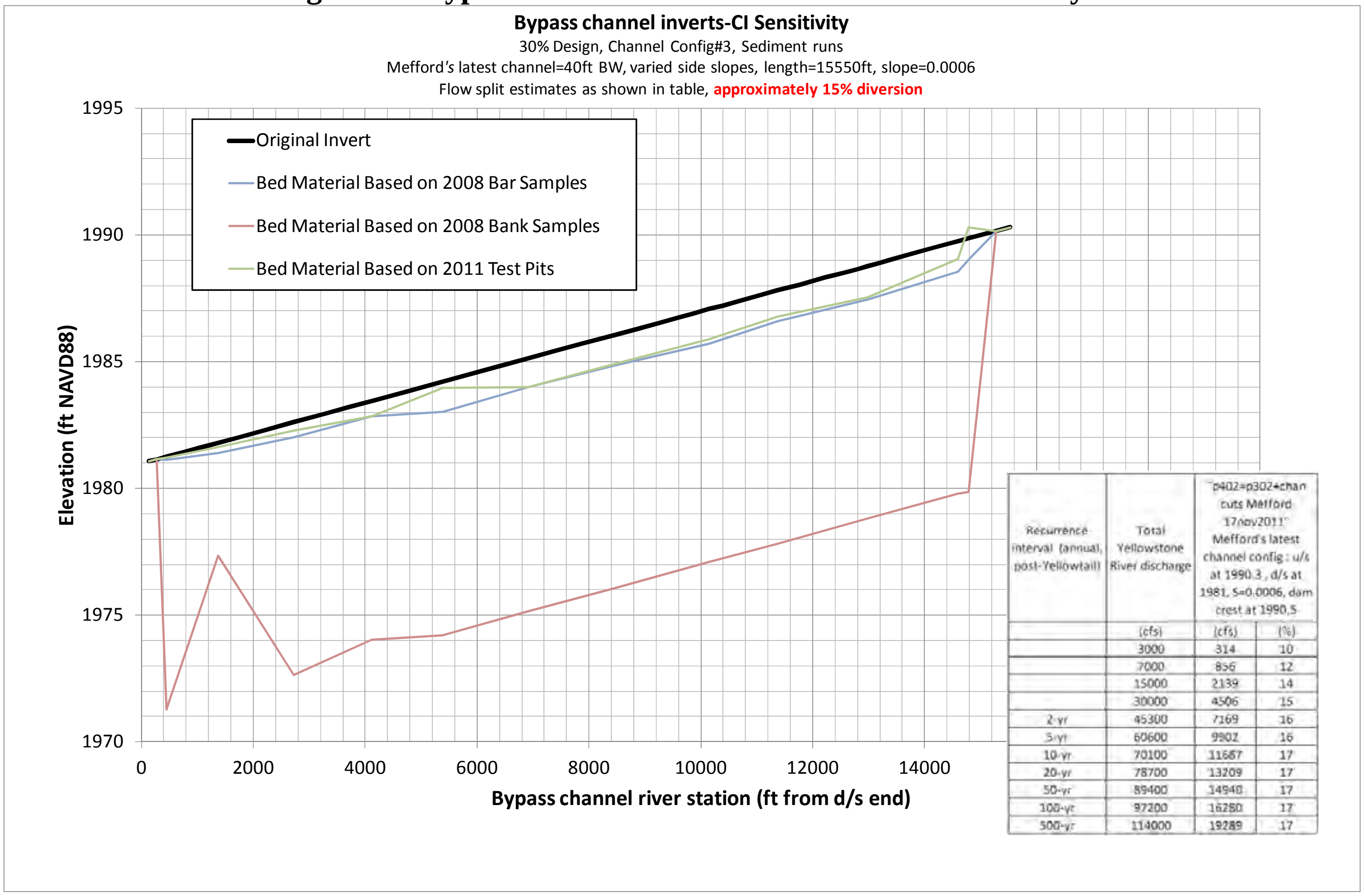
5.2 Bed Gradation

Because the future bed material of the proposed bypass channel is largely unknown, a range of bed material gradations was analyzed.
Several sources of data were employed to determine the expected range of bed material gradations including bed and bank samples collected in 2008 and test pit data collected in 2011. Section 4 and Figure 12 give details on sample data. Figure 12 shows average values from the 2008 bank and bar data as well as a user-generated curve with the lower end loosely based on the bank samples (labeled “RAS (2008 bank samples-coarse)”.

The selected bed material gradation is based on the 2011 test pit data since the samples were collected in the vicinity (both horizontal and vertical) of the proposed bypass channel bed. The selected curve is labeled “For HEC-RAS from 2011 Test Pits” in Figure 12.

Figure 14 summarizes a sensitivity analysis on bed material gradation by showing bypass channel invert profiles at the end of the 20 year simulation. Note that the maximum depth of degradation is set to 10 ft for all cross sections. The sensitivity indicates low sensitivity at coarser gradations (between the test pits and bar samples) but high sensitivity if the specified bed material is finer than the test pit data. The threshold where the bed turns significantly degradational is highly uncertain.

Figure 13 Bypass Channel Inverts-Bed Material Sensitivity



Recurrence interval (annual, post-Yellowtail)	Total Yellowstone River discharge	"p402=p302+chan cuts Mefford 17nov2011" Mefford's latest channel config : u/s at 1990.3 , d/s at 1981, S=0.0006, dam crest at 1990.5	
	(cfs)	(cfs)	(%)
	3000	314	10
	7000	856	12
	15000	2139	14
	30000	4506	15
2-yr	45300	7169	16
5-yr	60600	9902	16
10-yr	70100	11687	17
20-yr	78700	13209	17
50-yr	89400	14940	17
100-yr	97200	16280	17
500-yr	114000	19289	17

5.3 Incoming Sediment

Available suspended sediment data is discussed in Section 4 and Reference 6. The selected incoming sediment is based on engineering judgment considering both the 2011 sample data and the Sidney USGS gage data and is shown in Figure 9.

5.3.1 Total Load

Total load for a range of bypass discharges was computed based on estimated concentration of sediment entering the bypass (see Table 5).

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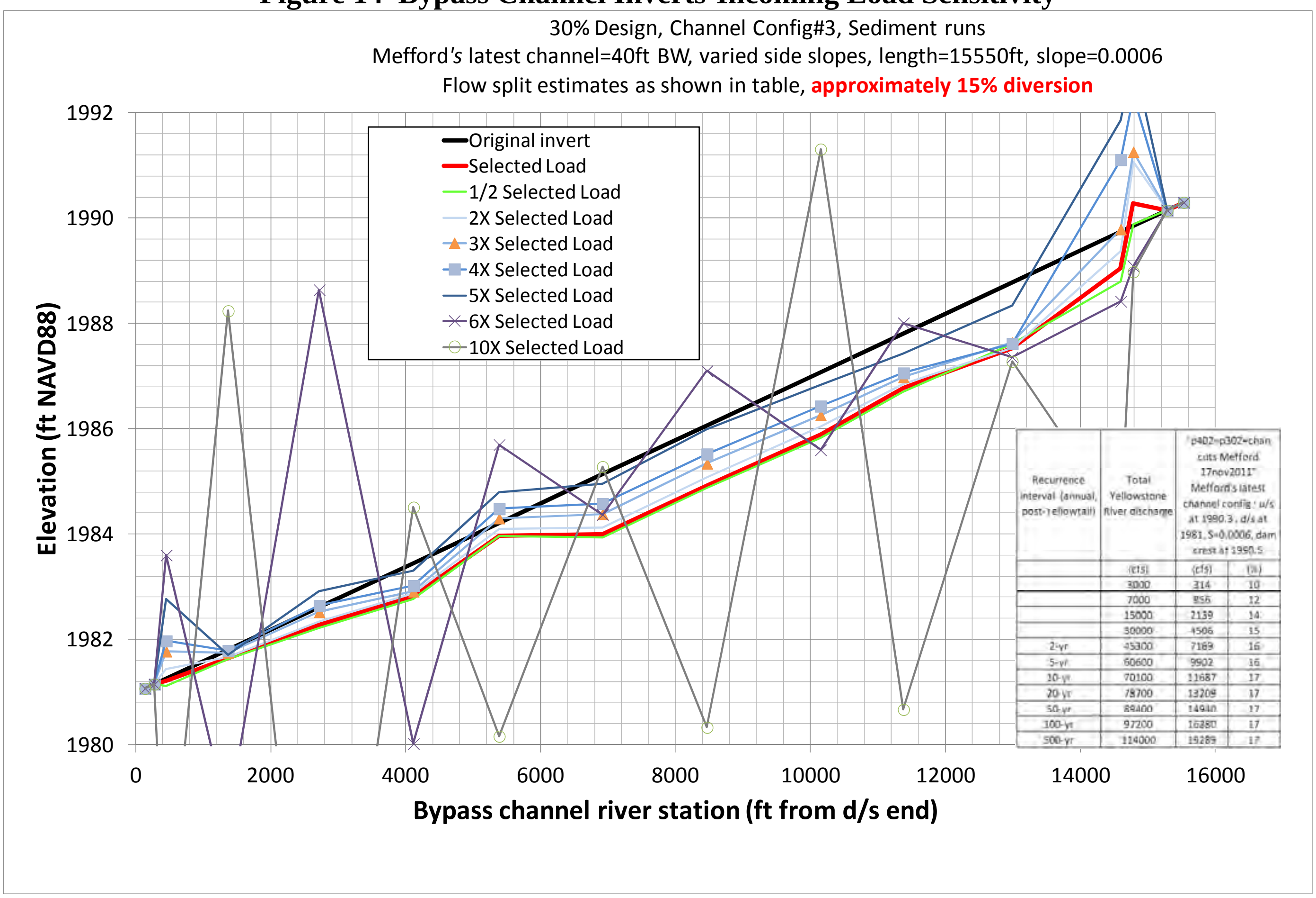
Table 5 Total Load

Discharge (bypass)	Conc	Total load
(cfs)	(mg/l)	(tons/day)
100	320	86
1960	65	344
3300	150	1337
8500	320	7344
40000	320	34560

Because of the high level of uncertainty associated with the incoming total sediment load, a sensitivity analysis was conducted. Figure 15 summarizes the sensitivity analysis by showing bypass channel inverts at the end of the 20 year simulation. Model instability occurs when the selected load is increased by a factor of approximately 6.

Figure 14 Bypass Channel Inverts-Incoming Load Sensitivity



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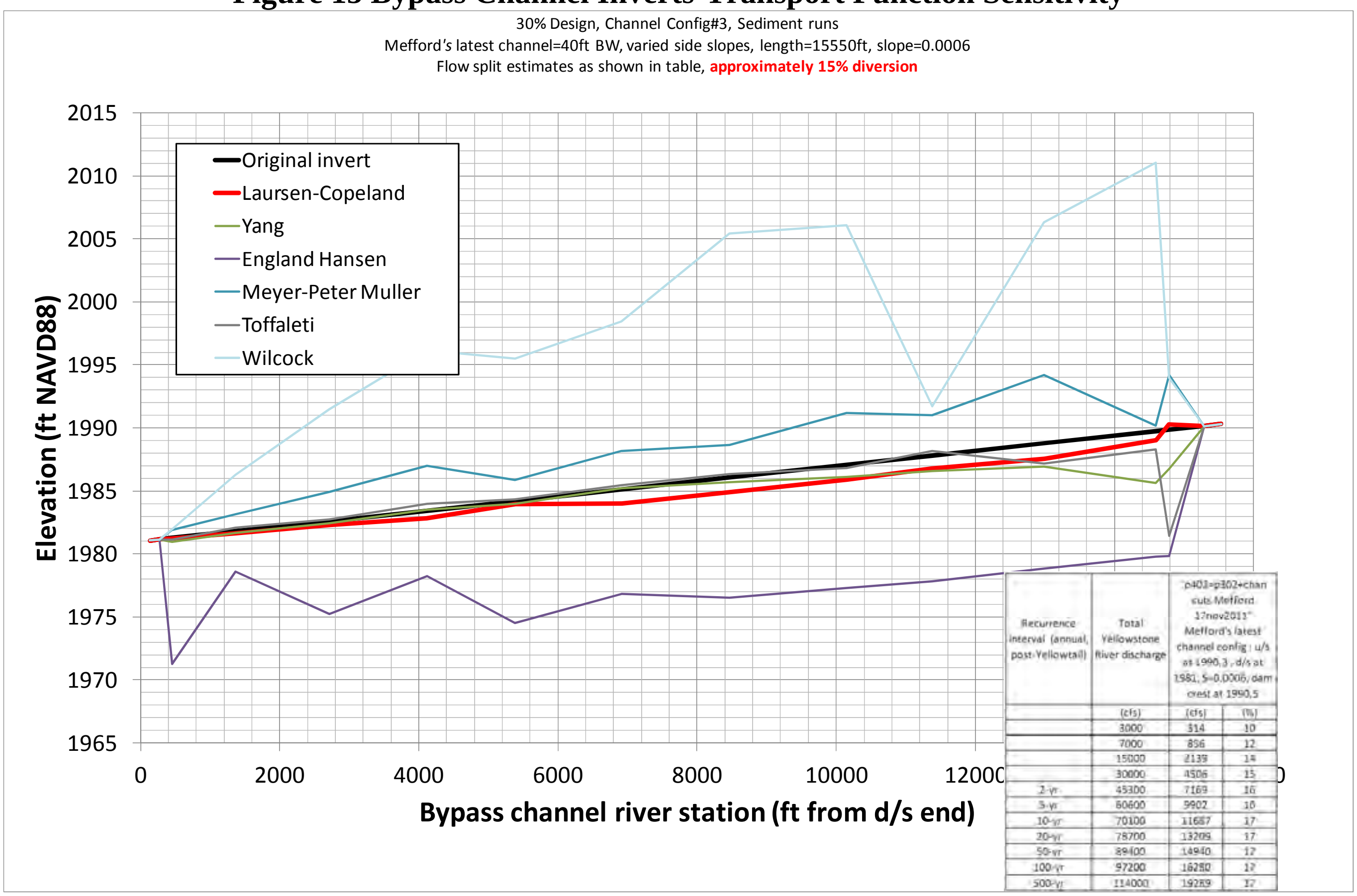
5.3.2 Particle Size Distribution

The incoming sediment particle size distribution is based on the 2011 sample data and Sidney gage data as shown in Figure 9.

5.4 Transport Function

The Laursen-Copeland transport function was selected because of its applicability to sediments in the silt range. Both the 2011 measured data and the Sidney data show over 60-90% of the material in suspension is finer than sand. Yang and Toffaleti also give reasonable results. Meter-Peter Muller computes fairly significant aggradation, which is expected due to its tendency to underpredict the transport potential of finer materials. Figure 16 shows results of the sensitivity analysis conducted on the selected transport function.

Figure 15 Bypass Channel Inverts-Transport Function Sensitivity



Recurrence interval (annual, post-Yellowtail)	Total Yellowstone River discharge	"p401=p302+chan cuts Mefford 17nov2011" Mefford's latest channel config : u/s at 1990.3 , d/s at 1981, S=0.0006, dam crest at 1990.5	
	(cfs)	(cfs)	(%)
	3000	314	10
	7000	856	12
	15000	2139	14
	30000	4506	15
2-yr	45300	7169	16
5-yr	60600	9902	16
10-yr	70100	11687	17
20-yr	78700	13209	17
50-yr	89400	14940	17
100-yr	97200	16250	17
500-yr	114000	19289	17

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6. CONCLUSIONS AND RECOMMENDATIONS

The HEC-RAS sediment routine was used to evaluate the bypass channel. Because of the nature of the proposed channel, calibration is not possible. Therefore, the results of the model are highly uncertain and should not be construed as providing quantitative estimates of aggradation/degradation.

Simulations conducted using the best estimates of all parameters and most applicable transport function indicate a slightly degradational trend. However, as shown in section 5, varying the transport functions or estimated parameters can result in a range of anywhere from 10+ ft of degradation to 10+ ft of aggradation over the 20-year period of simulation.

It is recommended that for the currently proposed channel configuration, a channel armor layer be constructed to prevent excessive vertical movement of the channel. The channel armor layer should be constructed of material with a D_{50} in the range of 37 to 45 mm (1.4” to 1.8”) based on an analysis conducted to determine the armor layer characteristics (see Reference 8).

REFERENCES

1. **U.S. Army Corps of Engineers.** EM 1110-2-1601 “Hydraulic Design of Flood Control Channels.” 1994.

2. **U.S. Army Corps of Engineers.** EM 1110-2-1602 “Hydraulic Design of Reservoir Outlet Works.” 1980.

3. **U.S. Army Corps of Engineers.** EM 1110-2-1603 “Hydraulic Design of Spillways.” 1990.

4. Chow, Ven T. “Open Channel Hydraulics.” McGraw-Hill Inc. 1959.

5. **U.S. Army Corps of Engineer, Omaha District.** “Bypass Channel Concept Evaluation.” April2011

6. **U.S. Army Corps of Engineers, Omaha District.** “Lower Yellowstone Project Fish Passage and Screening, Preliminary Design Report, Appendix A-2, Hydraulics.” October 2009.

7. **U.S. Army Corps of Engineers, Omaha District.** “Trip Report 26-28August2008.”

8. **U.S. Army Corps of Engineers, Omaha District.** “Intake Dam Fish Bypass Channel Stable Channel Material Analysis.” March 2012.

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Attachment 6 Bypass Channel

Appendix C

30% Design Features

19March2012

Updated 6JULY2012

NOTE:
FOR CONSISTENCY, FIGURES AND DESCRIPTIONS WILL BE UPDATED UPON COMPLETION OF PLANS AND SPECS.

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EXECUTIVE SUMMARY

A bypass channel for fish passage is being evaluated and compared to the previously analyzed rock ramp. This document summarizes the current configuration of the bypass channel.

The design effort for the proposed bypass channel is scheduled to be at approximately 30% by May 2012. Therefore, the data contained herein is between concept level and 30%, and should not be construed as a final design. Features are subject to change and input from the BRT is critical to understanding areas of concern as well as locations where adaptive management techniques are most likely to be successfully applied in the future.

The channel and associated features were designed using criteria developed by the BRT. The focus of the design effort was percentage of flow diverted and depths/velocities in the bypass channel. Considerable time was spent evaluating the appropriate percentage of flow to be diverted through the bypass channel. Sediment and flow data were collected around the upstream end of the bypass, hydraulic and sediment models were created, and empirical equations were consulted to determine the response not only to the bypass, but to the main channel of the Yellowstone River. A range of diversion flow between 10% and 35% was considered.

Sediment modeling indicates that diversion of greater than approximately 15% of the total flow leads to depositional tendencies in the main channel of the Yellowstone River. This is undesirable due to the impact to irrigation diversion in addition to maintenance issues in the vicinity of the fish screens on the newly constructed headworks.

Past BRT discussions indicated that the chances of pallid sturgeon using the bypass increase with percentage of flow diverted. To address attractive flow at the mouth of the bypass channel, the alternative being evaluated is a flow augmentation structure (FAS). The FAS would consist of a weir (essentially an extension of the proposed dam crest) that would discharge into the bypass channel near the downstream end (see Plates 1 and 15-16). The FAS would increase flows in the bypass channel entrance (downstream end) by around 4-6% in the May-June timeframe. While an FAS configuration is presented herein, final design would include physical modeling and additional numerical modeling to attain the most desirable flow patterns in and around the bypass channel entrance.

A number of channel configurations were modeled by Reclamation and USACE evaluating varying side slopes, channel widths, and channel slopes. The selected alternative meets design objectives (with respect to depth and velocity) with the exception of depth at the exit (upstream end) for a river flow of 10,000 ft^3/s. This was considered acceptable as the exit depth is similar to downstream river thalweg depths and exceeds the target depth prior to river flows reaching 20,000 ft^3/s. Reclamation's "Lower

Yellowstone Intake Diversion Dam, Fish Bypass Channel Entrance and Exit Pre-appraisal Study, Progress Report," dated January 2012, contains additional details and is attached.

The following proposed features are described in this document and are shown in Plate 1. Note: Several rock spanning structures are proposed. These structures are at grade and do not protrude into the flowline. The rock structures consist of larger material for stability that will be backfilled with natural river size rock during construction. The intent would be to provide a more natural substrate for fish passage with the underlying large rock to provide stability when needed.

1. **Bypass channel excavation-**A bypass channel would be excavated from the inlet of the existing high flow chute to just downstream of the existing dam and rubble field. The proposed alignment is approximately 15,500 ft long at a slope of 0.0006ft/ft (natural Yellowstone River slope in the project area is approximately .0004ft/ft to .0007ft/ft) and excavation is currently estimated at approximately 1.2 million cubic yards. The channel cross section has a 40ft bottom width and side slopes varying from 1V:12H to 1V:3H. The bypass channel would divert approximately 15% of total Yellowstone River flows.
2. **Upstream control structure**-A structure designed to control discharge into the bypass channel would be situated on the upstream end of the channel. The structure will be composed of either all riprap or riprap with a concrete sill. With either construction material, the control structure would be backfilled with natural river size rock to give the appearance of a seamless channel invert. The purpose of the structure is to provide stability during extreme events to prevent excessive flow through the bypass.
3. **Channel plug**-A channel "plug" would be constructed approximately 1 mile downstream from the upstream end of the bypass in the existing high flow chute to keep normal flows in the proposed bypass. The channel plug would be an earthen embankment with rock riprap armor. The channel plug would have a low-level discharge pipe and would be designed for overtopping during larger events to maintain the existing chute's current functionality.
4. **Riprap at bends for lateral stability**-Bank riprap is proposed at two outside bends to minimize the risk of losing the bypass channel planform.
5. **Vertical control structures**-Two vertical control structures (riprap sills) are proposed for maintaining channel slope and allowing for early identification of channel movement. Similar to the upstream control structure, these would be overexcavated and backfilled with natural river size rock to give the appearance of a seamless channel invert while providing stability during extreme events.
6. **Downstream vertical control structure**-A riprap sill is proposed at the downstream end of the channel to maintain channel elevations (similar to vertical control structures).

7. **Downstream lateral stability structure**-A riprap bank stabilization feature would be constructed on the descending right bank of the bypass channel to prevent downstream migration (relative to the Yellowstone River) of the downstream end of the bypass channel.
8. **New dam**-In order to maintain irrigation diversion capabilities without impacting the bypass channel, a new dam is proposed. The new dam would preclude the necessity of adding large rock to the crest of the existing dam to maintain diversion capabilities (as is currently done). The proposed dam configuration is a concrete crest placed underwater in sheet pile "cells" with approximate dimensions of 24 ft (in the direction of flow) by 40 ft. See Attachment 7 for additional details on the proposed crest.
9. **Flow augmentation structure-POTENTIAL ADAPTIVE MANAGEMENT FEATURE-**A weir constructed using roller compacted concrete would be constructed near the tie-in between the downstream end of the bypass channel and Yellowstone River. The weir would increase attractive flow in the bypass channel when Yellowstone River discharges are above approximately 7,000cfs. The flow augmentation structure would only be constructed as a response to lack of passage and is not included in the proposed bypass channel configuration.
10. **Armor Layer**-Evaluation is currently underway to determine the necessity of artificially constructing an armor layer. The proposed armor layer would be similar to naturally formed armor layers found in the Yellowstone River on bars. The intent would be to minimize bypass channel degradation while providing substrate similar to reaches upstream and downstream from the project.

PURPOSE

This document describes the features required for the proposed bypass channel at Reclamation's Lower Yellowstone Irrigation Project (Intake). The intent of this document is to provide enough detail to allow for a 30% level of design cost estimate. Geotechnical and structural personnel will use this information to provide quantities to cost estimating.

FEATURES

The following features are described in this document:

1. Bypass channel excavation
2. Upstream control structure (referred to as "exit" from fish perspective)
3. Channel plug
4. Riprap at bends for lateral stability
5. Vertical control structures
6. Downstream vertical control structure (referred to as "entrance" from fish perspective)
7. Downstream lateral stability structure
8. New dam
9. Flow augmentation structure
10. Armor Layer

Each of these features is shown on Plate 1 and described in the following sections. All elevations are referenced to NAVD88 vertical datum.

1. BYPASS CHANNEL EXCAVATION

The main element of the bypass alternative is channel excavation. As shown in Plate 1, the upstream 1/3 of the proposed channel uses the existing high flow chute. Approximately 5000ft from the upstream end, the proposed channel diverges from the existing chute and continues across Joe's Island for the remaining 10,000ft where it flows back into the river just below the existing dam and rock field.

The upstream invert elevation is 1990.3 ft NAVD88 and the downstream invert elevation is 1981.0 ft NAVD88 for a total drop of 9.3 ft over a length of 15,500 ft (slope=0.0006 ft/ft).

The proposed channel section has a 40ft bottom width with side slopes varying from 1V:12H to 1V:3H. The section shape was developed with input from the BRT and is shown in Plate 2. Plates 3-8 compare existing ground to the proposed channel cuts at select locations. The river station (RS) shown on Plates 3-8 can be correlated to location using the stationing shown in Plate 1.

2. UPSTREAM CONTROL STRUCTURE

The upstream control structure is located at approximate station 15+250 on the proposed alignment, just downstream of where the existing high flow channel inlets. The purpose of the upstream control structure is to maintain the designed rate of diversion into the proposed bypass. The structure is centered around a 60' wide concrete sill inverted at elevation 1990.3. The concrete sill will be 15' long in the direction of flow. Loose rock riprap will protect the sideslopes of the structure. In the downstream direction, the riprap will slope at 10H:1V for a distance of 35'. In the upstream direction, the riprap will slope at 3.5H:1V for a distance of 12.25' followed by a 25' long horizontal blanket. A width of 60' will be maintained throughout the structure. The structure will be backfilled to the proposed grade of the channel. The embankments of the structure will be sloped at a 5:1 rate and rock lined up to the projected 10 year water surface elevation. Plates 9-10 show the plan, profile, and cross section of the proposed control structure.

3. CHANNEL PLUG

The channel plug is located in the existing high flow chute just downstream from where the proposed channel diverges (see Plate 1). The purpose of the channel plug is to prevent water from leaving the proposed bypass channel during low to normal flows. The channel plug would be constructed as an earthen embankment with rock riprap armor.

The top elevation of the plug is 2000ft NAVD88, just above the 5-year water surface elevation of 1999.8 ft NAVD88 and nearly a foot below the 10-year water surface elevation of 2000.9 ft NAVD88. Plate 11 shows the plan, profile, and cross section of the proposed channel plug. Water surface elevations were taken adjacent to the channel plug structure in the proposed bypass channel.

The plug is designed as an overtopping section to allow flow into the existing high flow chute during higher Yellowstone River flows so that the existing chute retains its functionality. The high flow chute currently begins carrying water during a Yellowstone River discharge of approximately 25,000-30,000 cfs. With-project conditions would allow flow into the remaining existing high flow chute at a discharge of approximately 60,000cfs.

To accommodate overtopping flows, the crest is 15 ft wide and the downstream face of the plug is on a 1V:6H slope with riprap as described in the bullets below:

- D_{100}=30 inches
- D_{50}=20 inches
- Layer thickness=45 inches (=1.5*D_{100})

The downstream toe should transition to a horizontal blanket approximately 50 ft long, then should extend on a 1V:3H slope into native ground two layer thicknesses or approximately 7.5 ft.

The upstream face is on a 1V:3H slope and should include riprap toed into existing ground two layer thicknesses (7.5ft).
See Plate 11 for plan, profile, and cross section views of the proposed channel plug.
A low level output pipe is proposed to allow small flows through the plug at a Yellowstone River flow of approximately 7000 cfs and above. The proposed low level outlet pipe would be 18" in diameter and would have an upstream invert elevation of approximately 1991 ft NAVD88, a length of approximately 130 ft, and a downstream invert elevation of 1990 ft NAVD88. A rock-lined preformed scour hole would dissipate energy on the downstream end of the pipe (see Plate 12).

4. RIPRAP AT BENDS FOR LATERAL STABILITY

Riprap is proposed at two outside bends to prevent significant lateral movement. The upstream bend riprap section is approximately 2000 ft long and the downstream section is approximately 1500ft long. Some lateral movement of the bypass channel is expected and may inadvertently enhance depth/velocity diversity in the bypass channel. The proposed riprap locations were selected due to the potential for significant adverse consequences of lateral channel movement. The upstream bend riprap location is in the vicinity of an existing swale created by an old channel scar. Loss of the bend in this area could result in bypass channel avulsion. The downstream bend riprap location is located where the bypass channel comes within 700 ft of the Yellowstone River. Significant lateral channel movement in this area would put the bypass channel and Yellowstone River at risk.

The riprap section would consist of rock with a D_{100} of 16 inches and a layer thickness of 24 inches (based on Isbash using velocity of $1.25*V_{avg}=1.25*7=8.75$ft/s). The section would be placed on a 1V:3H slope, such that the bottom portion would be buried by the channel's flatter slopes. The section would extend from the channel invert to approximately 15ft above the invert (approximate 10yr depth). The section includes a weighted toe along the invert. The area of the weighted toe is 1.5 times the area required to extend the 24 inch layer on a 1V:3H slope down two layer thicknesses (4 ft). (Note- the area was multiplied by 1.5 to account for self-launching of the weighted toe).

See Plate 13 for additional details on the riprap section.

Rough quantity computations indicate a volume of approximately 25,000 tons:

$$3500\text{ft} * 60\text{ft} * 2\text{ft} / 27\text{ft}^3/\text{yd}^3 * 1.55\text{ton}/\text{yd}^3 = 24{,}111\text{tons}$$

5. VERTICAL CONTROL STRUCTURES

Two buried riprap sections are proposed at approximately stations 4800 and 9400 with the intention of monitoring vertical movement within the channel.

The sections would be over excavated, then a bedding and riprap section would be constructed with a top elevation approximately 1-2 ft below the final invert. Material similar to that composing the rest of the excavated channel would be used to bring the section up to final grade.
Following high flow events or long durations of low flow, the indicator sections could be evaluated to determine channel performance and stability. Adaptive management measures could be taken if necessary.

Plate 14 shows a typical plan view and section for the indicator sections.

6. DOWNSTREAM VERTICAL CONTROL STRUCTURE

The downstream vertical control structure is configured similar to the vertical control structures described in section 5 and Plate 14. It is shown on Plate 15 in relation to the other downstream features (lateral stability structure and flow augmentation structure).

7. DOWNSTREAM LATERAL STABILITY STRUCTURE

The downstream lateral stability structure consists of a riprap revetment designed to allow for a smooth transition from the main channel of the Yellowstone River to the bypass channel. Additionally, the structure is intended to prevent the downstream end of the bypass channel from migrating eastward (downstream).

The upstream end of the structure conforms to the typical channel section side slopes (see Plate 2). Downstream from the vertical control structure, the side slopes transition to a 3H:1V slope (see Plate 15).

The riprap section should consist of 24 inch D_{100} material with a layer thickness of 36 inches.

The upstream end of the structure is keyed into the bank using a riprap filled trench that is 20ft wide and 10ft deep extending 100ft into the bank at a 45 degree angle (see Plate 15). The trench size was determined using a post-launch section of 45ft by 3ft, factored by 1.5 to account for stone lost during launching.

8. NEW DAM

A new dam is proposed just upstream from the existing dam for the following reasons:

- Installation of fish screens increased required head for diversion
- Continued placement of rock on the existing dam crest is not desirable from a fish passage perspective or from a bypass channel maintenance perspective

The proposed dam configuration is a concrete crest placed underwater in sheet pile "cells" with approximate dimensions of 24 ft (in the direction of flow) by 40 ft. See Attachment 7 for additional details on the proposed crest.The new dam will be located approximately 50 ft upstream from the existing dam. The area between the old and new dams will be partially filled with riprap similar to the material that composes the existing rubble field.

9. FLOW AUGMENTATION STRUCTURE

The flow augmentation structure concept, developed by Reclamation, is intended as a potential adaptive management feature to increase attractive flow in the bypass channel. The structure would function as a weir once Yellowstone River discharges reached approximately 7000 cfs. The structure location is shown in Plate 15. This structure would ONLY be constructed as a response to lack of passage and is not included in the preferred alternative. It is include here to provide information on one potential AM feature.

The structure would consist of a roller compacted concrete weir crest followed by a 50ft horizontal riprap blanket. The riprap blanket terminates on the relatively flat side slopes of the proposed bypass channel. Details are shown in Plates 16-18. Table 1 shows the increase in bypass channel flow due to the flow augmentation structure (weir).

Table 1 Bypass Channel and Weir Flows

River Flow	Bypass Flow	Weir Flow	Canal Flow	Bypass Flow as % of River Flow	Weir Flow as % of River Flow	Bypass and Weir Flow as % or River Flow
ft^3/s	ft^3/s	ft^3/s	ft^3/s			
5000	560	0	1400	15.6%	0.0%	15.6%
10000	1300	150	1400	15.1%	1.7%	16.9%
20000	2880	840	1400	15.5%	4.5%	20.0%
30000	4430	1610	1400	15.5%	5.6%	21.1%
40000	6040	2400	1400	15.6%	6.2%	21.9%
70000	11050	4760	1400	16.1%	6.9%	23.0%

10. ARMOR LAYER

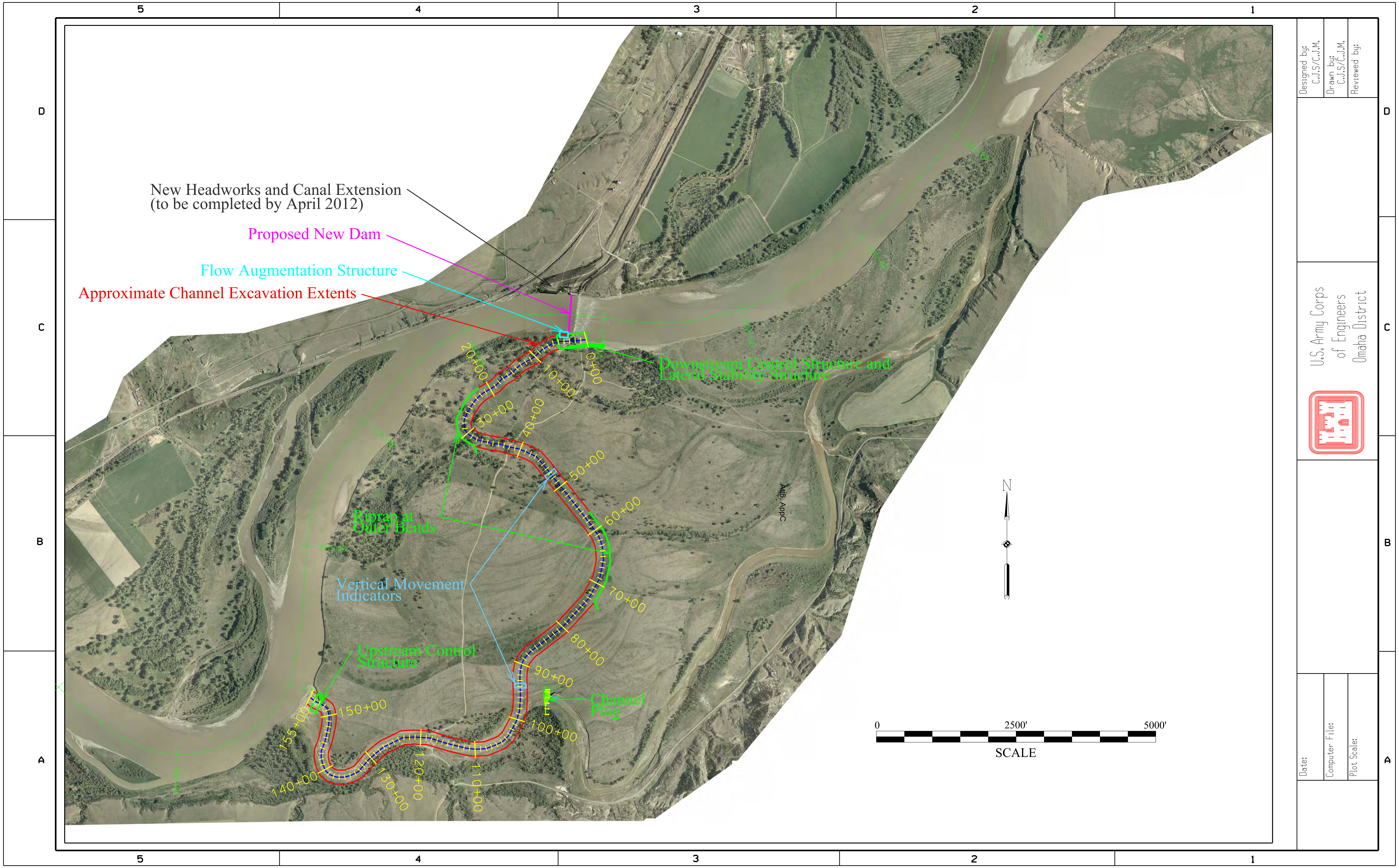
Evaluation is currently underway to determine the necessity of artificially constructing an armor layer. Preliminary sediment transport modeling of the proposed bypass channel indicates a slightly degradational tendency, highly dependent on the bed material inputs to the model. The proposed armor layer would be similar to naturally formed armor layers found in the Yellowstone River on bars and would represent what would be expected were the newly excavated channel be allowed to form the layer naturally.

The intent of the armor layer would be to prevent degradation of the channel leading to poor fish passage performance as well as diverting too much water into the bypass.

The alternative to constructing an artificial armor layer is to underexcavate the channel and allow the armor layer to develop over time. Risks associated with this method include the potential for too little or not enough degradation prior to attaining a stable armor layer.

The armor layer gradation would be similar to available measured data from 2008 Yellowstone River bar samples (see Photo 1) in the vicinity of Intake Dam (D_{50}≈16mm, D_{90}≈128mm). The armor layer would be continuous from upstream to downstream (i.e. the vertical control structures would be covered with the armor layer so as to minimize flow discontinuities).

Photograph 1 Mid-channel bar downstream from Intake Dam


New Headworks and Canal Extension
(to be completed by April 2012)
Proposed New Dam
Flow Augmentation Structure
Approximate Channel Excavation Extents
Downstream Control Structure and
Lateral Stability Structure
Riprap at
Outer Bends
Vertical Movement
Indicators
Upstream Control
Structure
Channel
Plug
0+00
10+00
20+00
30+00
40+00
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2500'
5000'
SCALE
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Designed by:
C.J.S/C.J.M.
Drawn by:
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Reviewed by:
U.S. Army Corps
of Engineers
Omaha District
Date:
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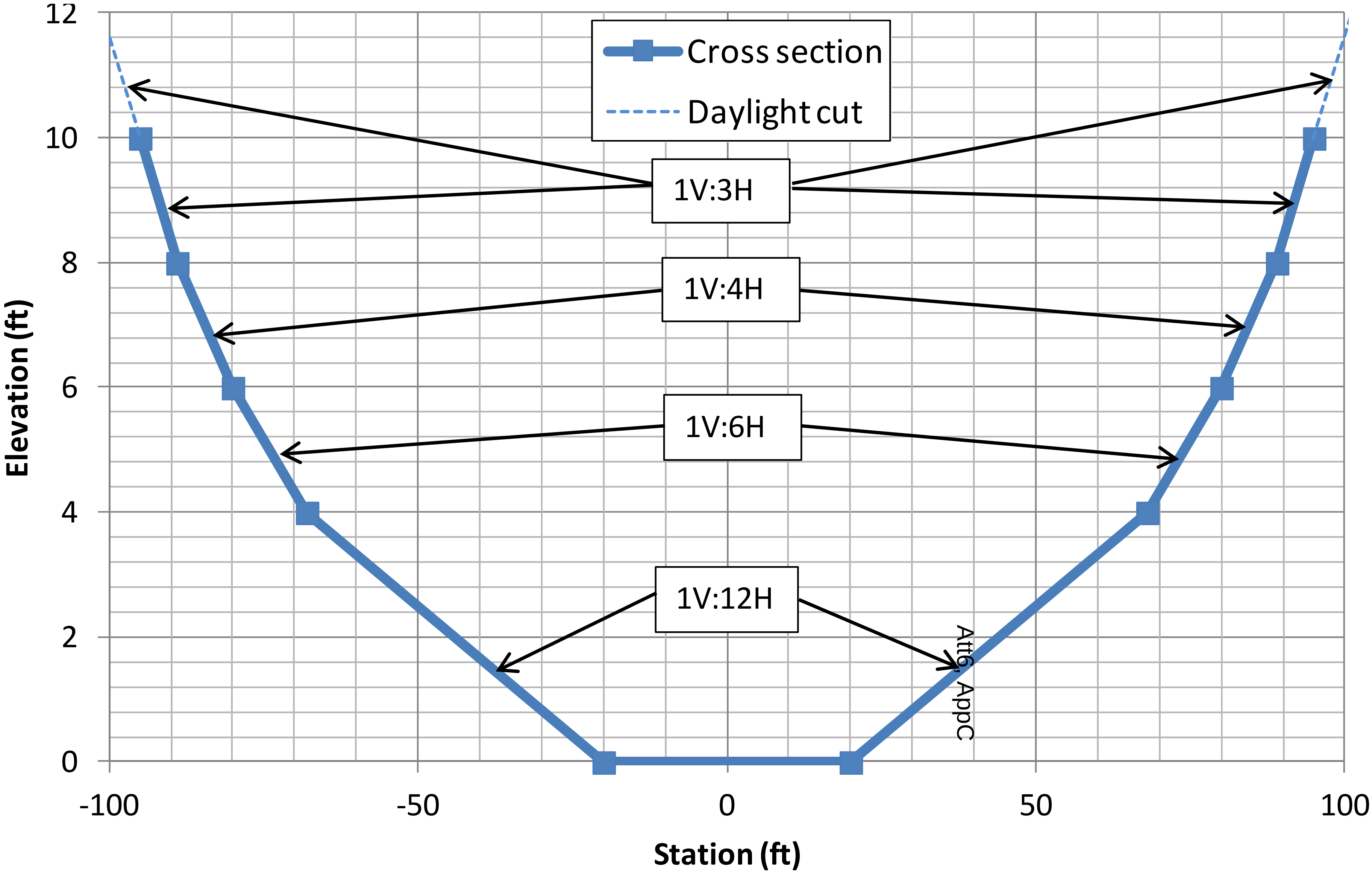

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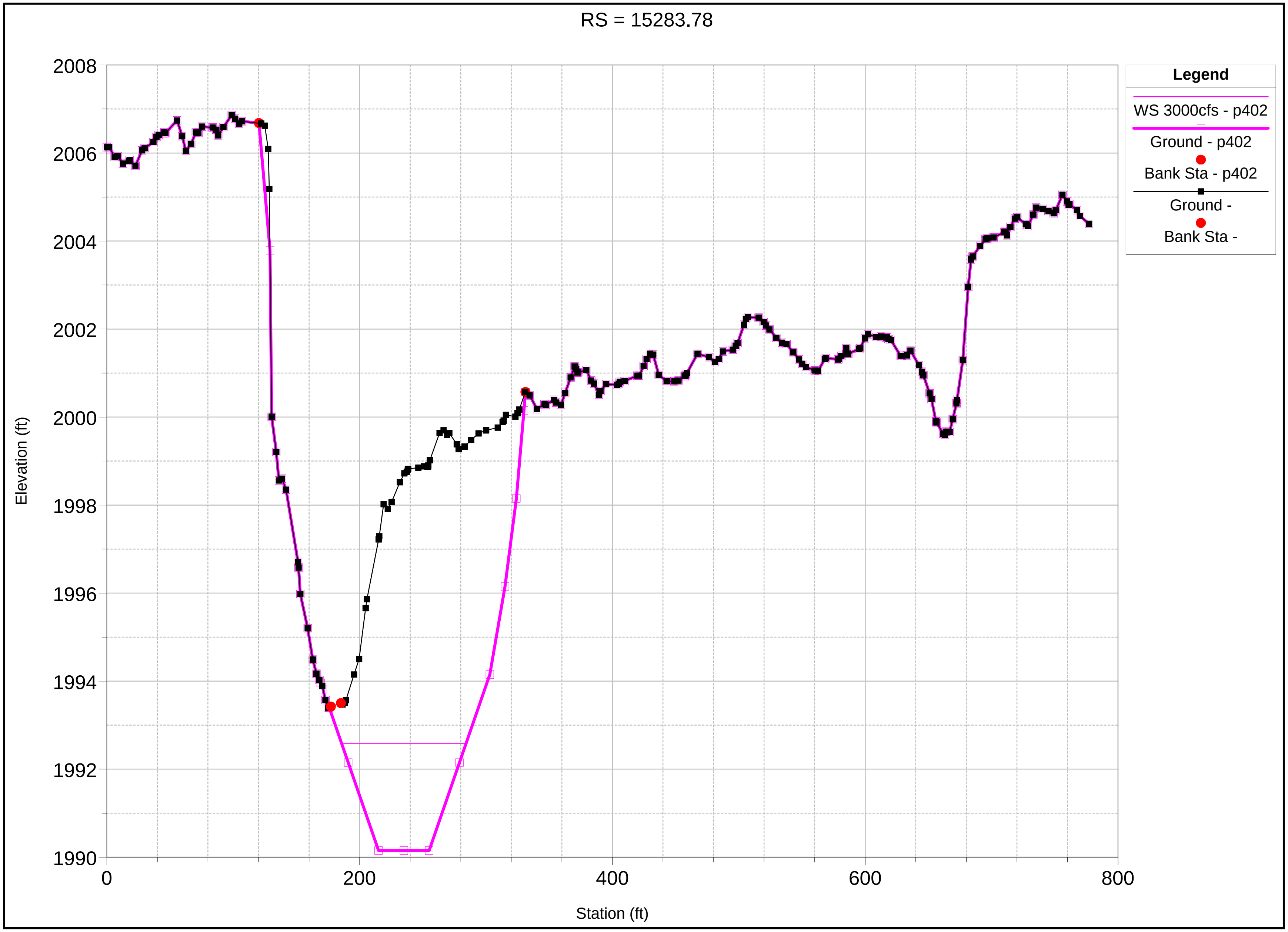

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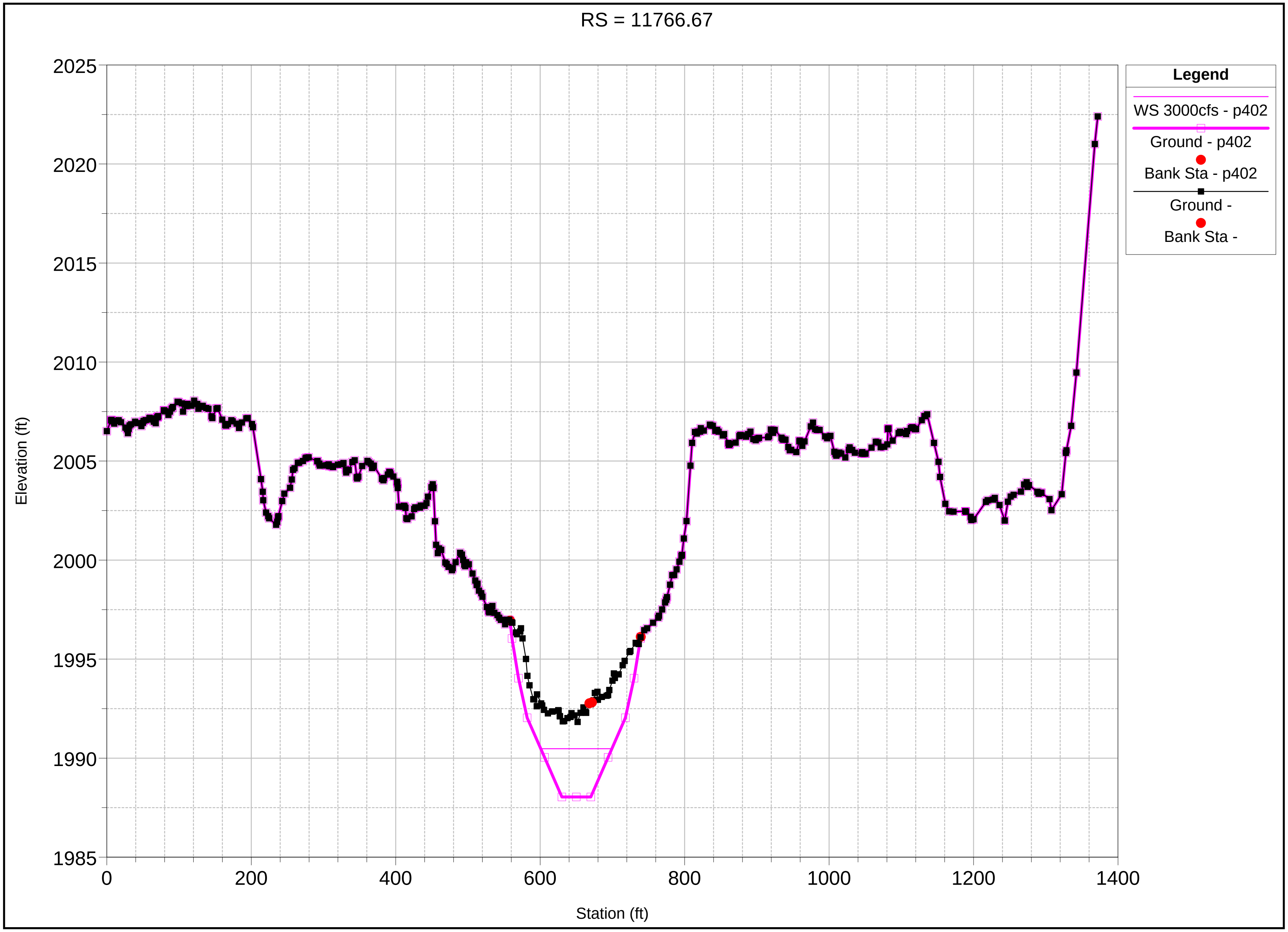

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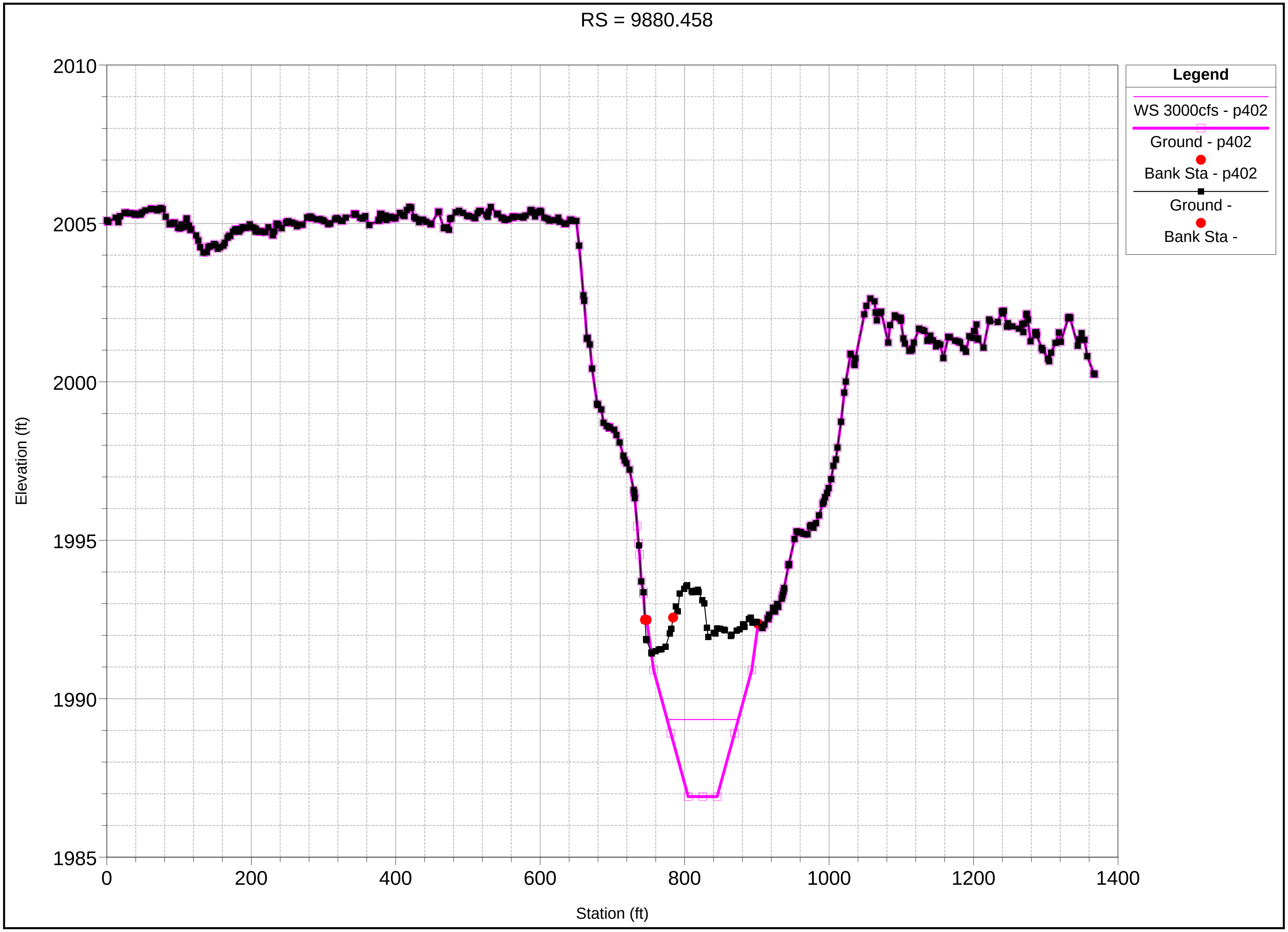
Plate 4



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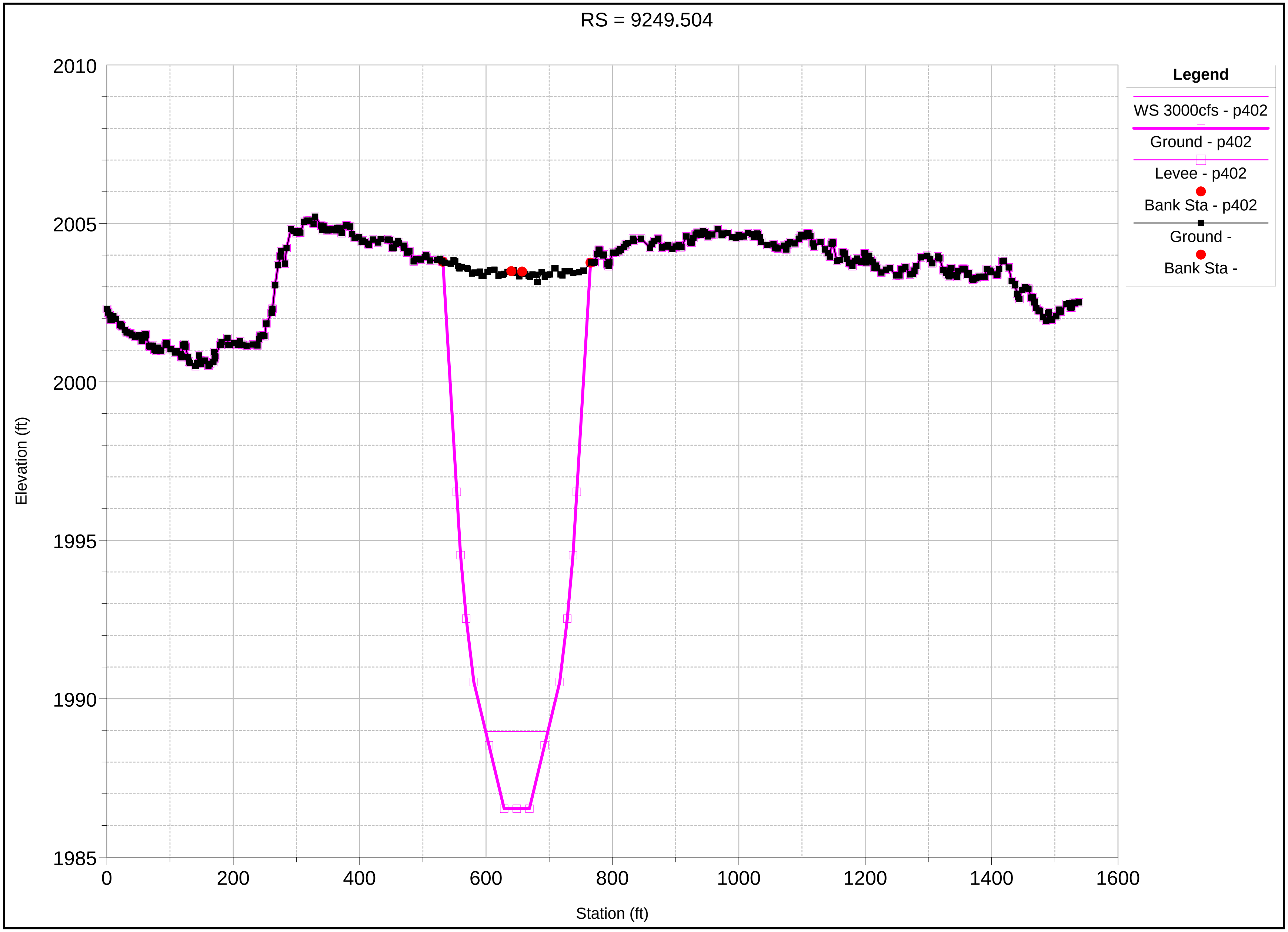

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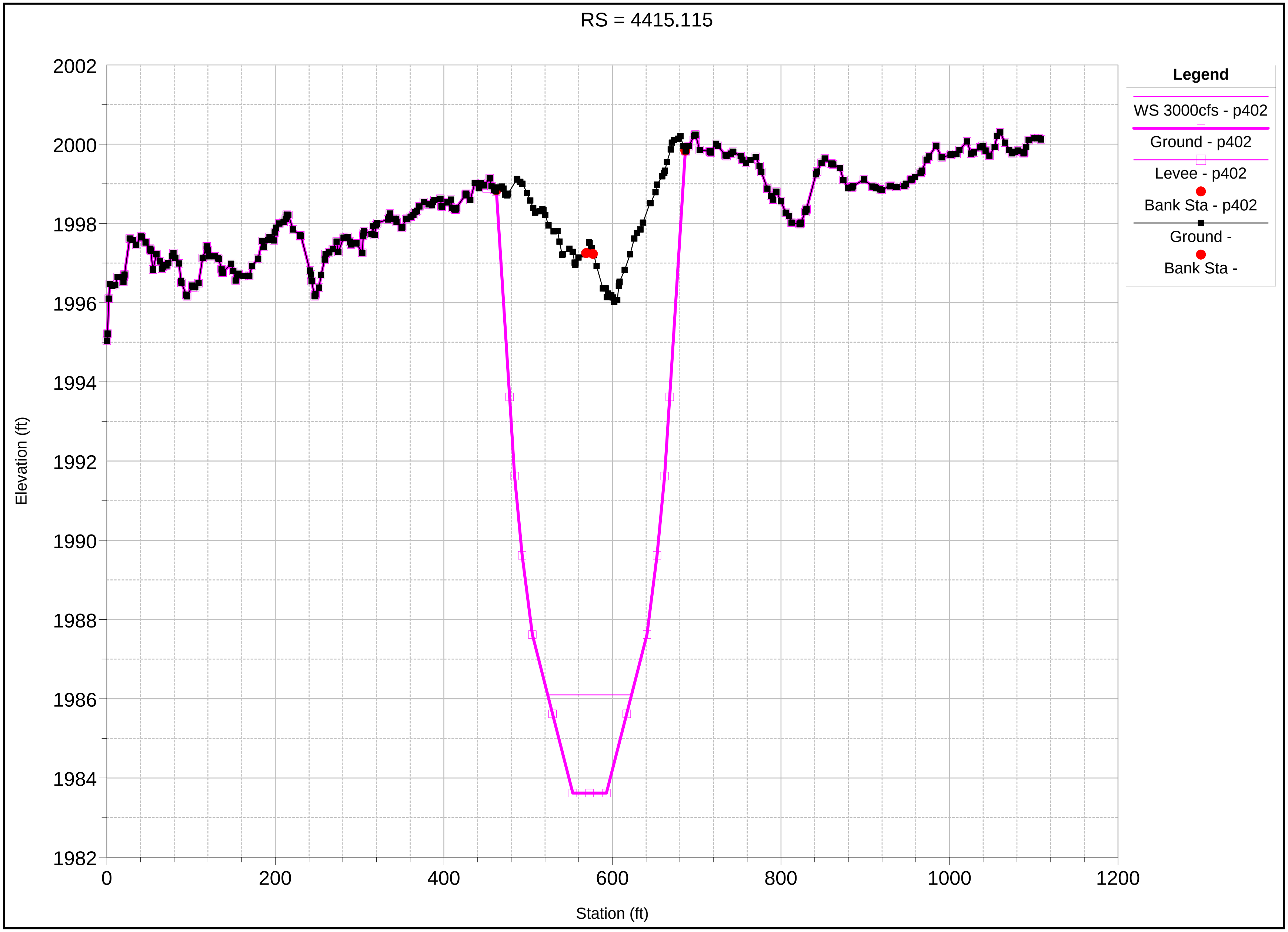
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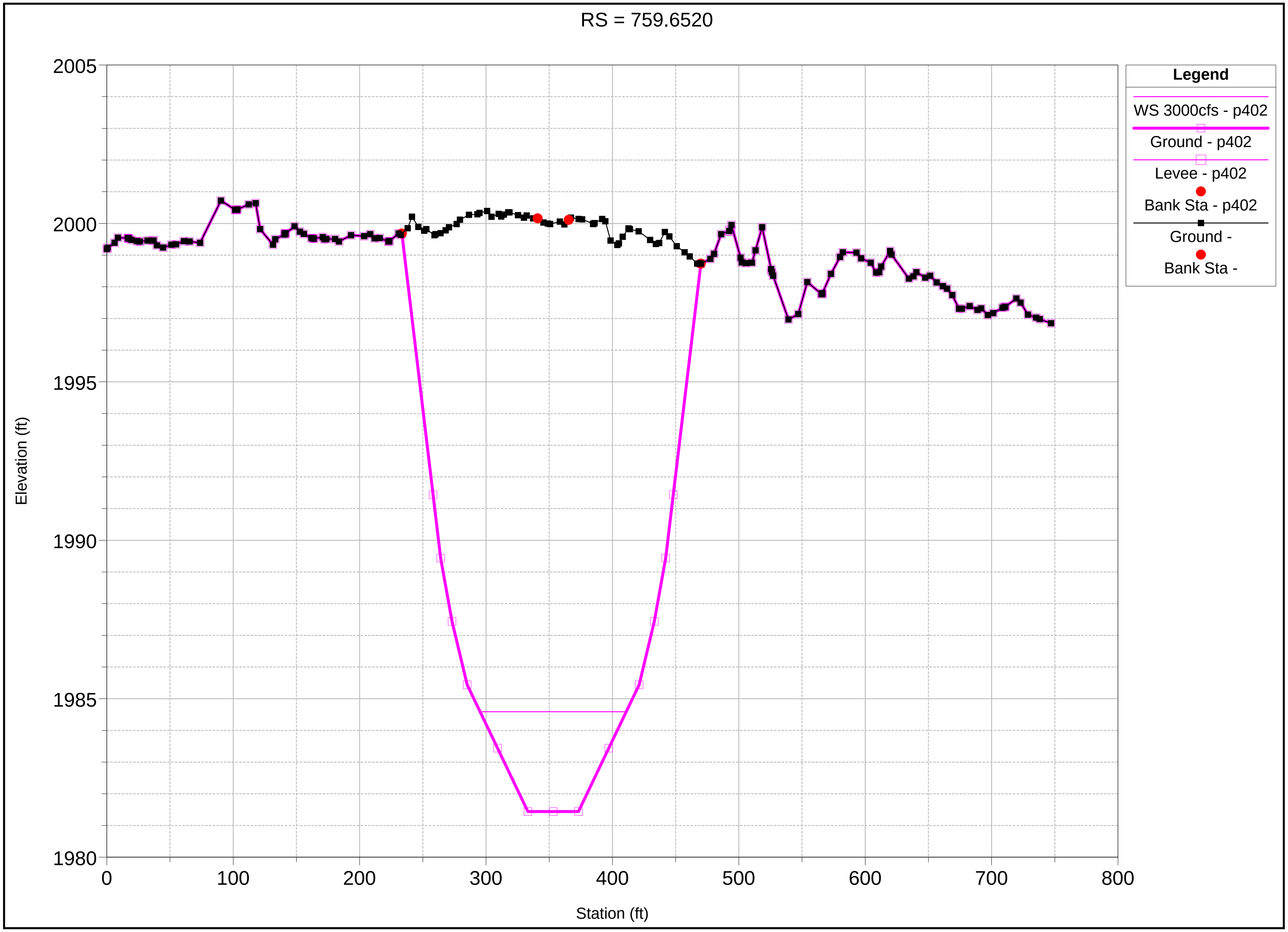
Elevation (ft)
Station (ft)
2002
2000
1998
1996
1994
1992
1990
1988
1986
1984
1982
0
200
400
600
800
1000
1200
Legend
WS 3000cfs - p402
Ground - p402
Levee - p402
Bank Sta - p402
Ground -
Bank Sta -



Att6, AppC

Return to TOC

US ARMY CORPS
OF ENGINEERS
OMAHA DISTRICT

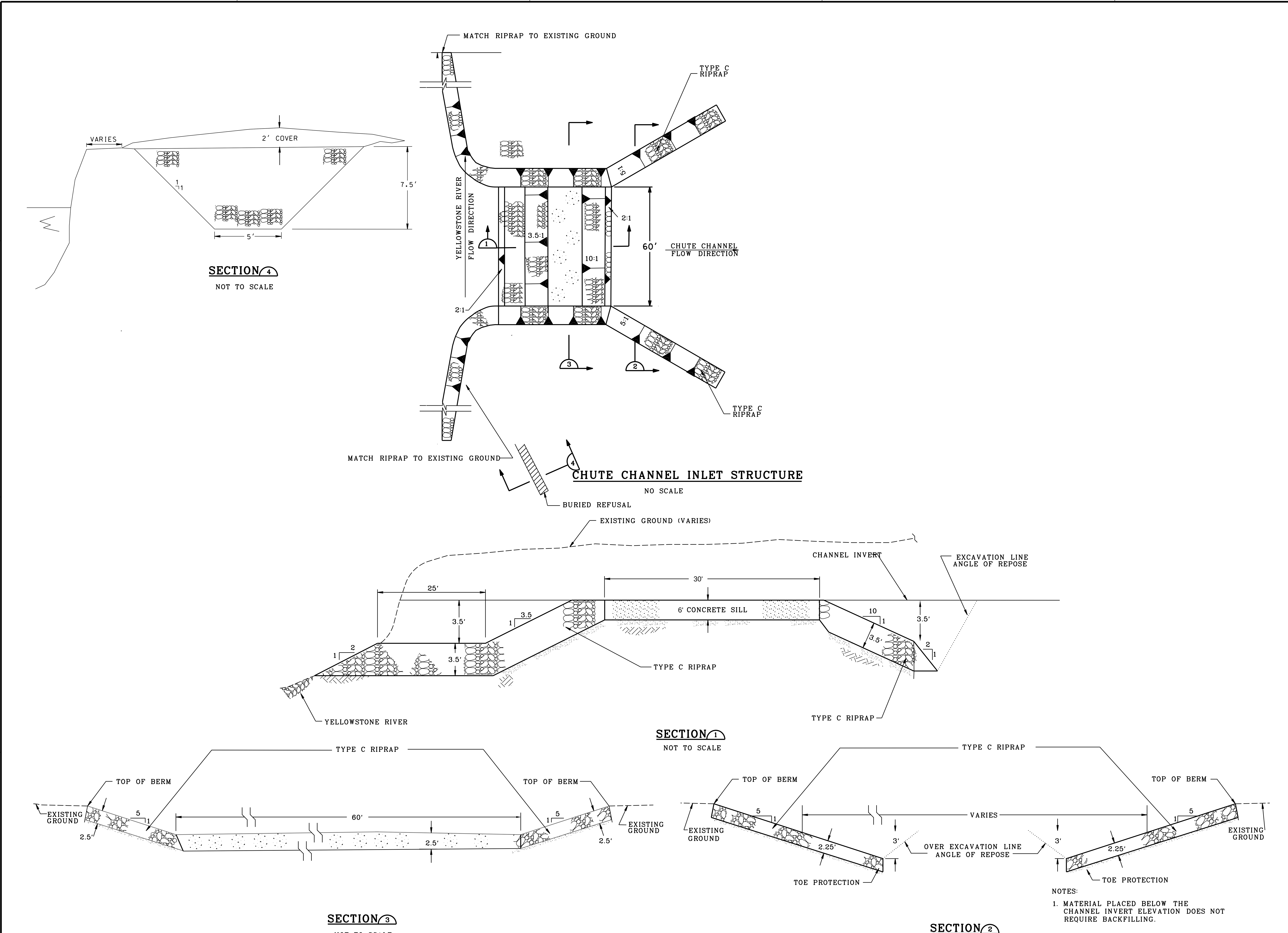
SYMBOL	DESCRIPTION	DATE	APPROVED

APPROVED BY:
ENGINEERING DIVISION, CHF.
DATE:

DESIGNED BY: C.J.S.
DWN BY: C.J.S.
CKD BY:
REVIEWED BY:
SUBMITTED BY: CHIEF: CHAN. STAB. & SED. SECTION
DATE: DEC 16, 2011
DESIGN FILE NO.
PLOT SCALE RATIO:
DRAWING CODE: X
REV.
SPEC. NO.
XXXXX-XX-X-XXXX
CONTRACT NO.
TASK ORDER #XX

INTAKE FISH PASSAGE BYPASS
YELLOWSTONE RIVER MONTANA
UPSTREAM GRADE CONTROL STRUCTURE
PLAN VIEW

SHEET
REFERENCE
NUMBER:



Return to TOC

US ARMY CORPS OF ENGINEERS OMAHA DISTRICT

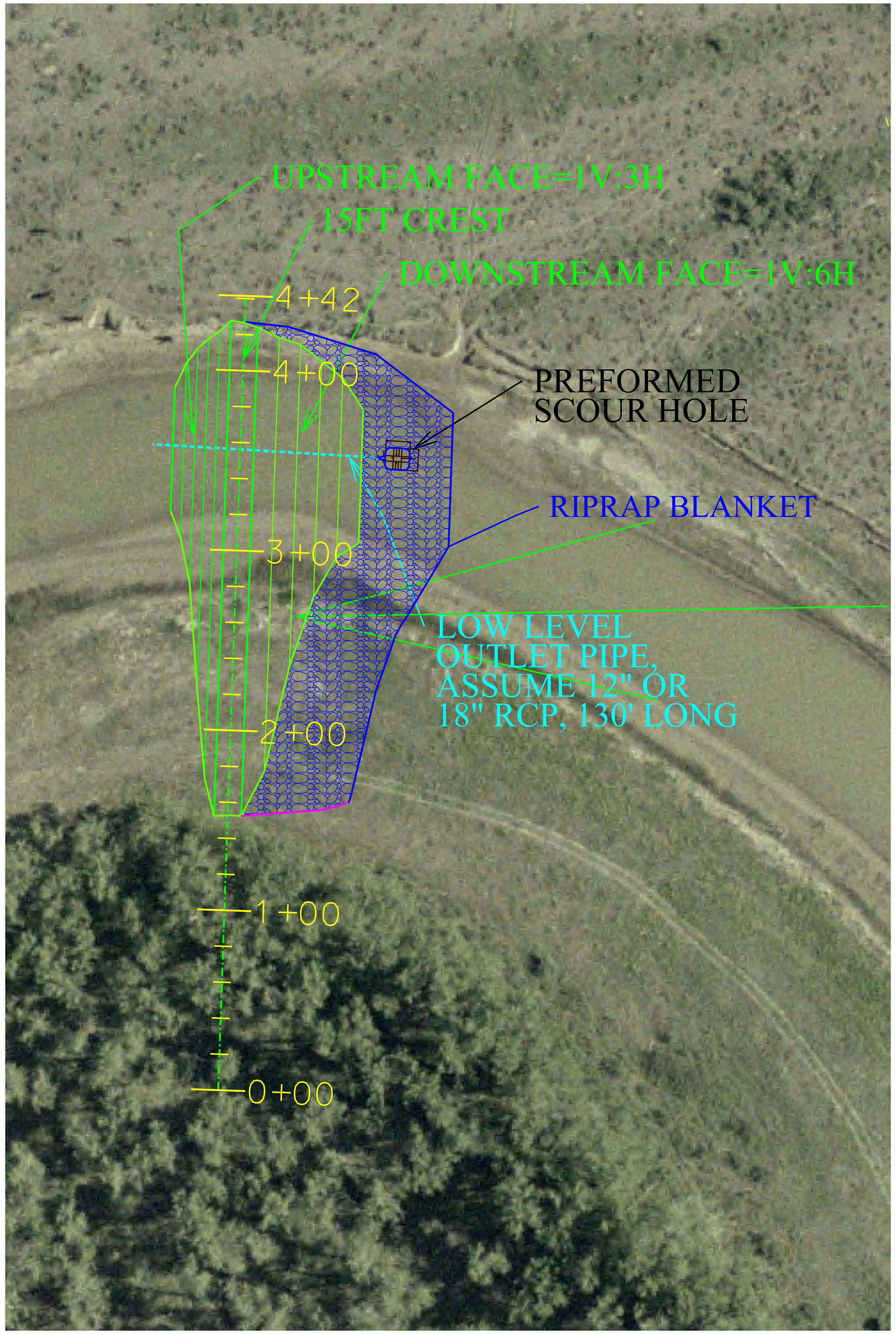
SYMBOL	DESCRIPTION	DATE	APPROVED

APPROVED BY: ENGINEERING DIVISION, CHF. DATE:

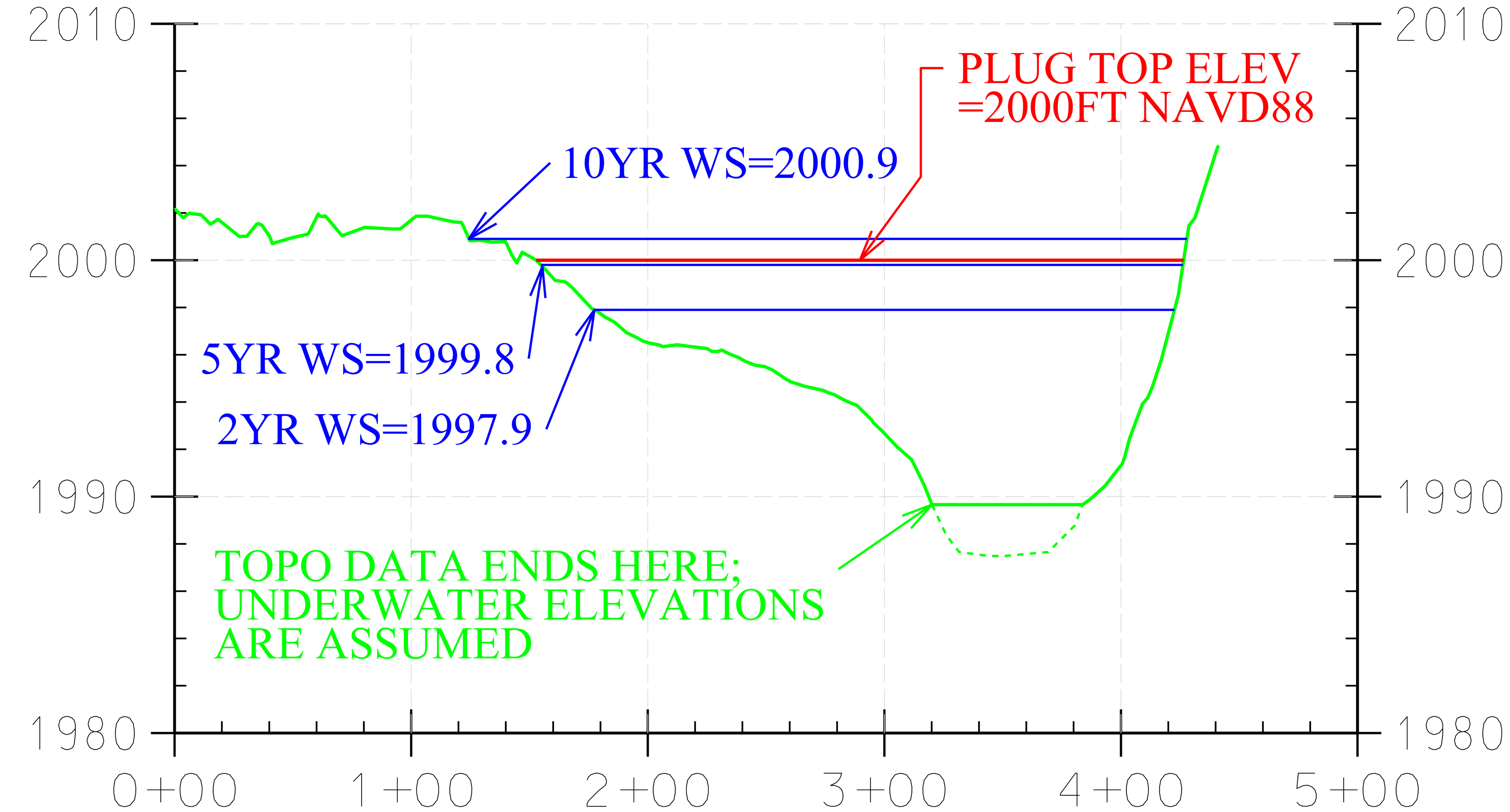
DESIGNED BY: C.J.S. DWN BY: C.J.S. CKD BY: REVIEWED BY: SUBMITTED BY: CHIEF: CHAN. STAB. & SED. SECTION
DATE: DEC 16, 2011 DESIGN FILE NO. PLOT SCALE RATIO: DRAWING CODE: X
REV. SPEC. NO. XXXXX-XX-X-XXXX CONTRACT NO. TASK ORDER #XX

INTAKE FISH PASSAGE BYPASS
YELLOWSTONE RIVER MONTANA
UPSTREAM GRADE CONTROL STRUCTURE
SECTIONS

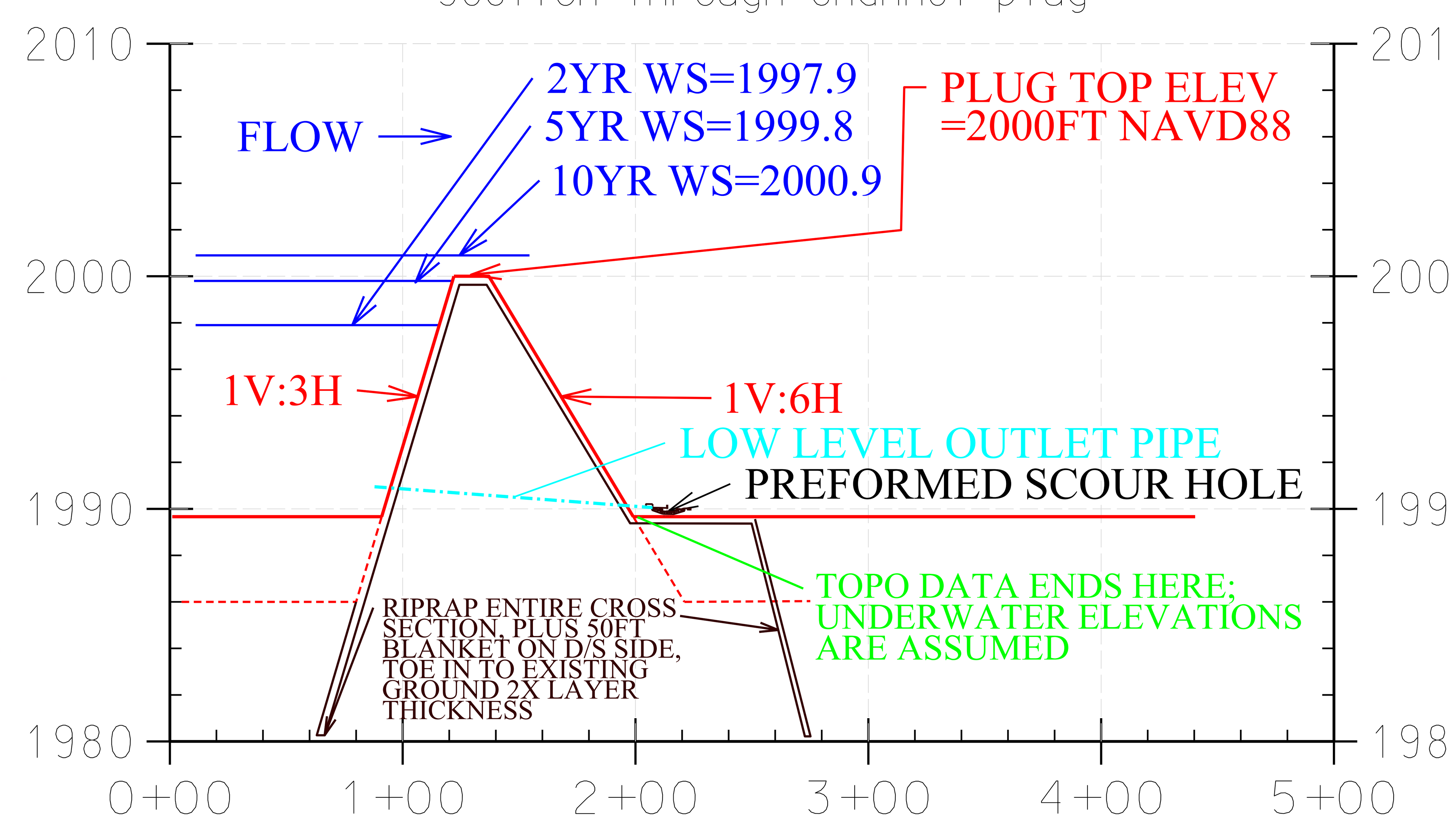
SHEET REFERENCE NUMBER:



Section along Channel Plug CL



Section through channel plug



Designed by: C.J.S/C.J.M.

Drawn by: C.J.S/C.J.M.

Reviewed by:

U.S. Army Corps of Engineers Omaha District

Date:

Computer File:

Plot Scale:

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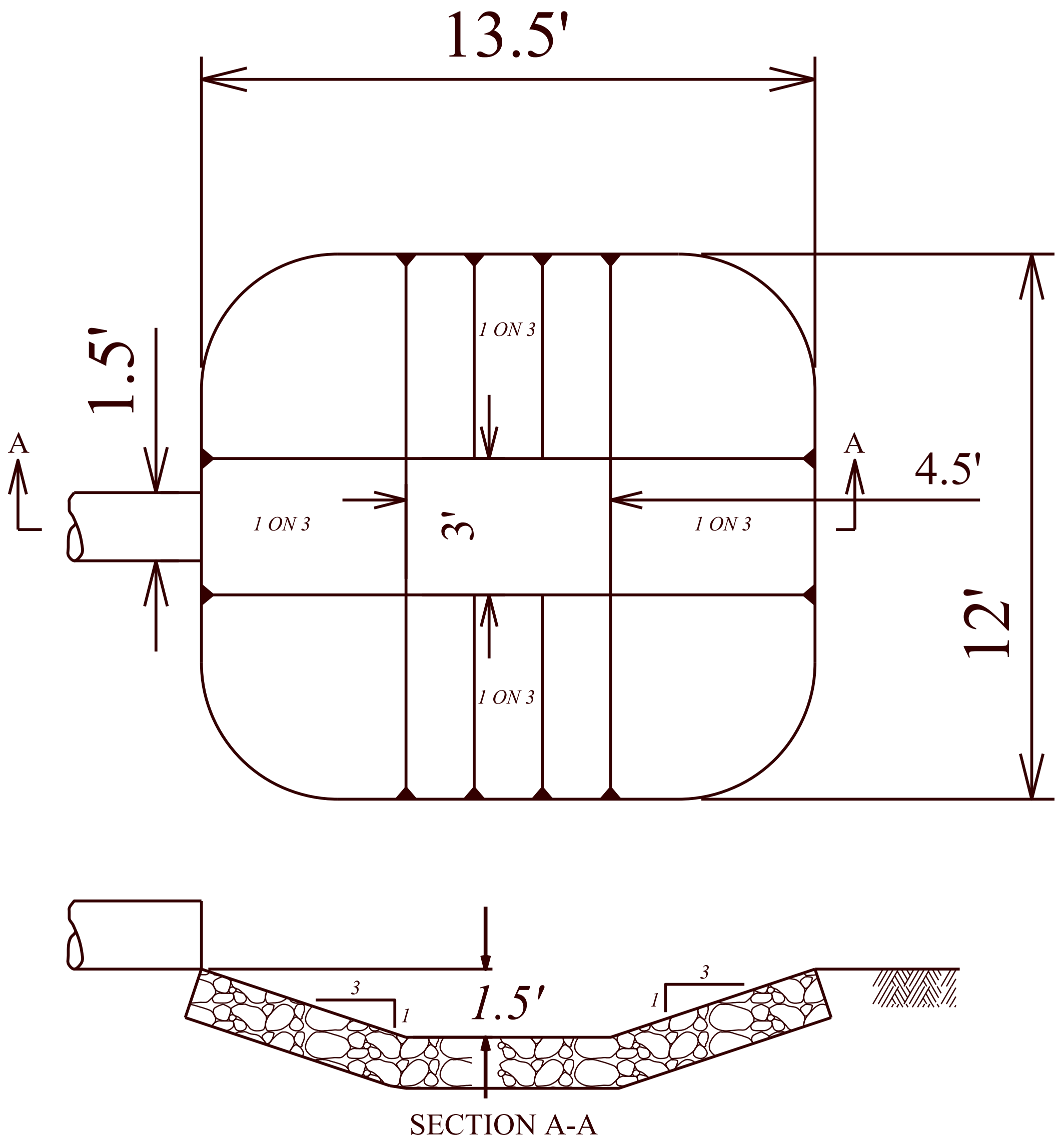

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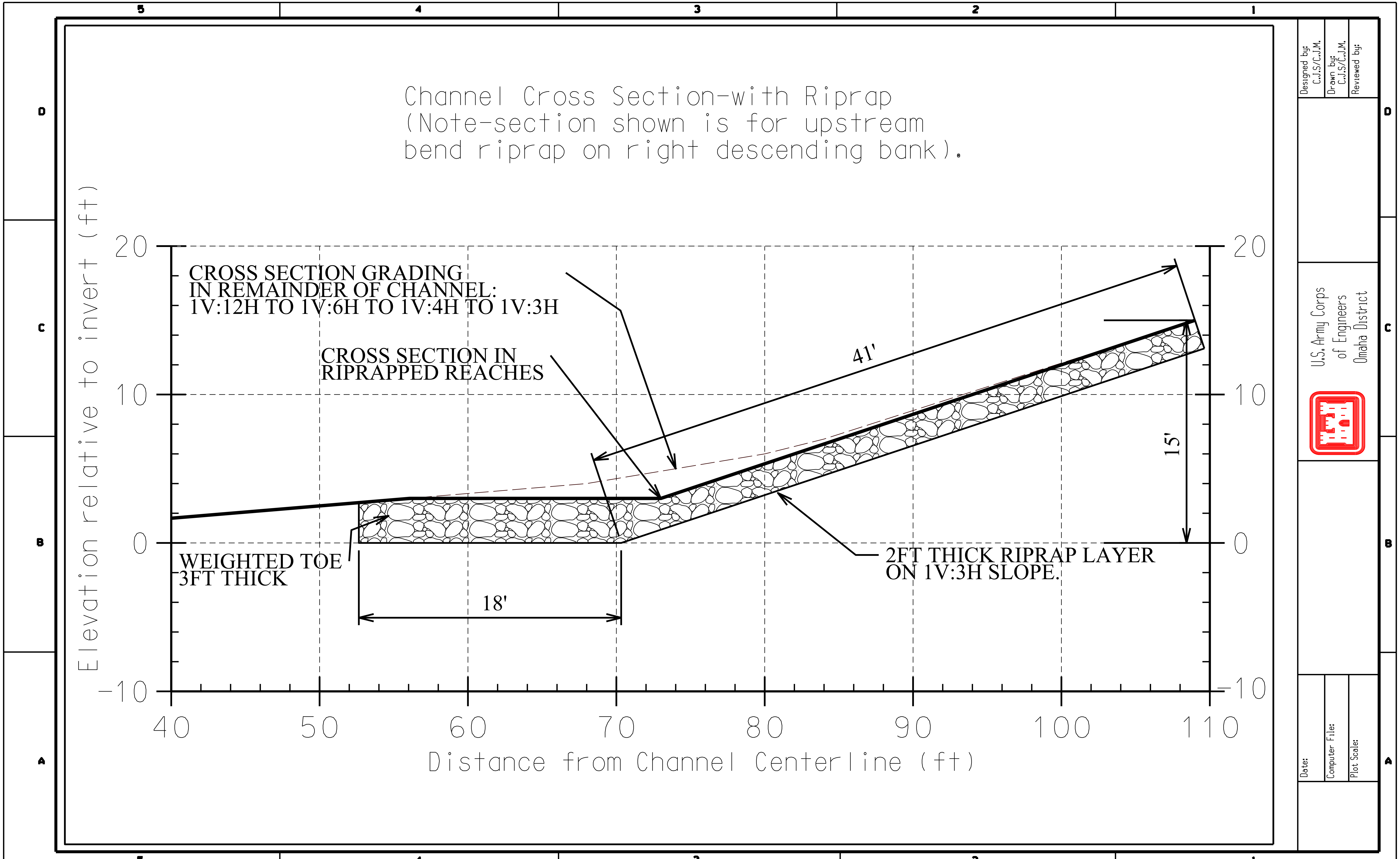
Date:		U.S. Army Corps of Engineers Omaha District	Designed by: C.J.S/C.J.M.
Computer File:			Drawn by: C.J.S/C.J.M.
Plot Scale:			Reviewed by:

Plate 12



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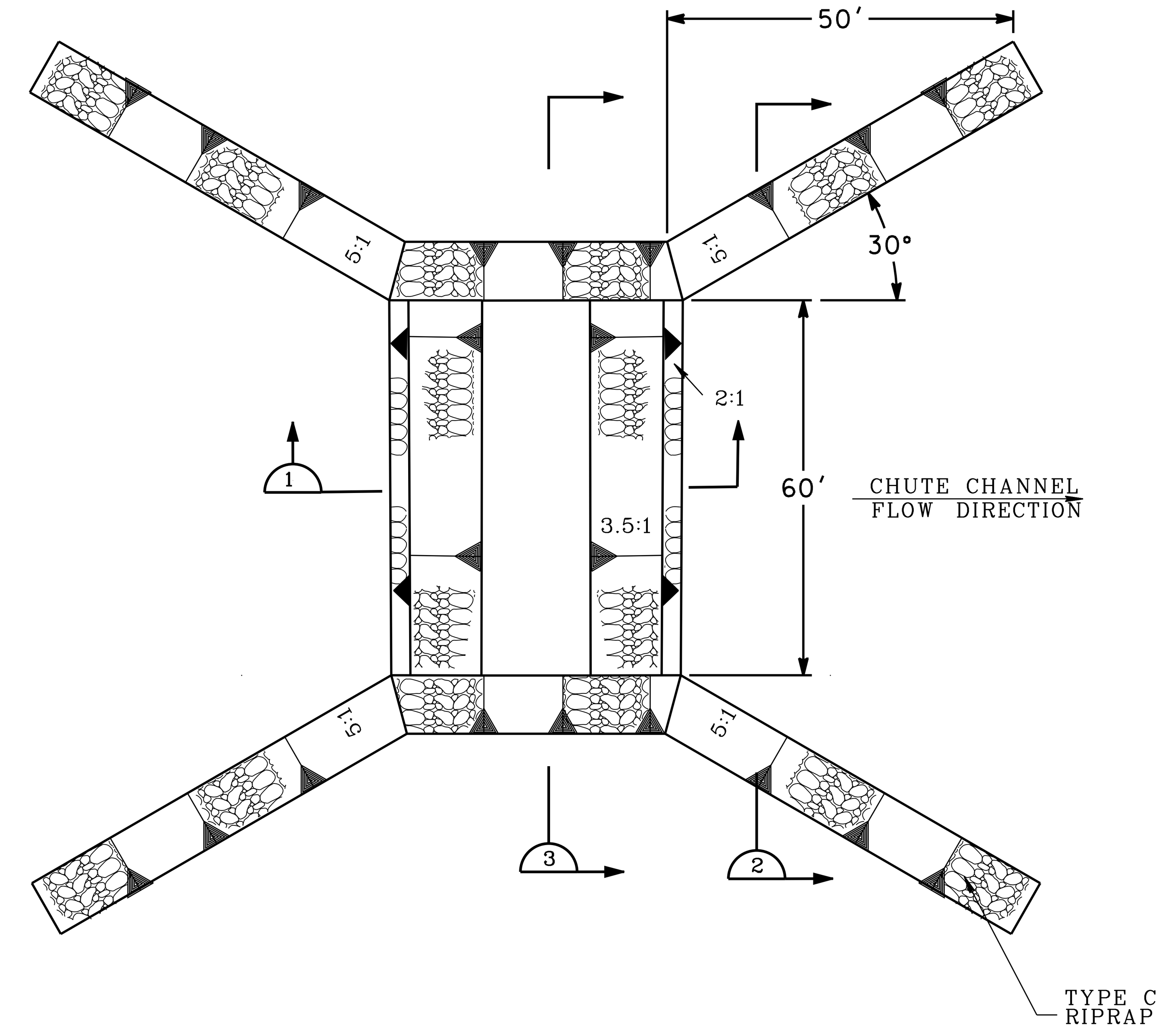
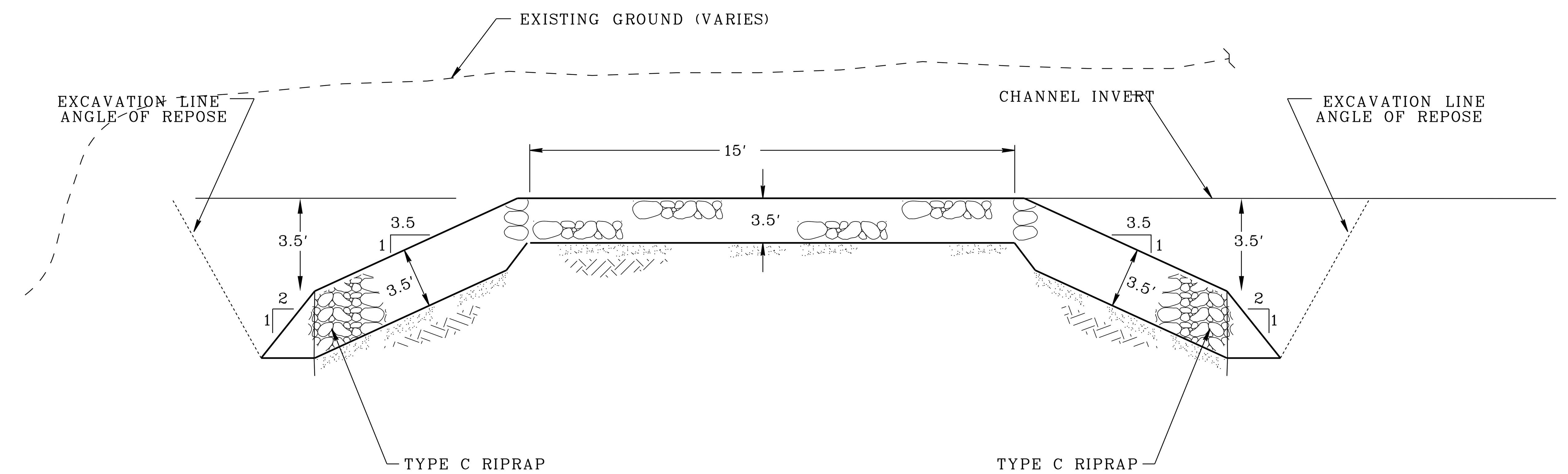


BYPASS VERTICAL CONTROL SILLS

NO SCALE



SECTION 1

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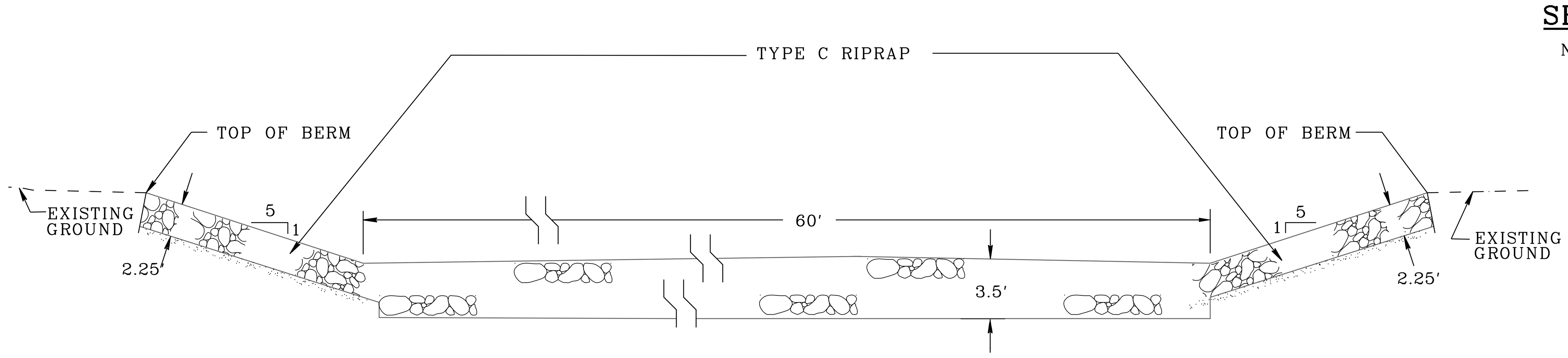

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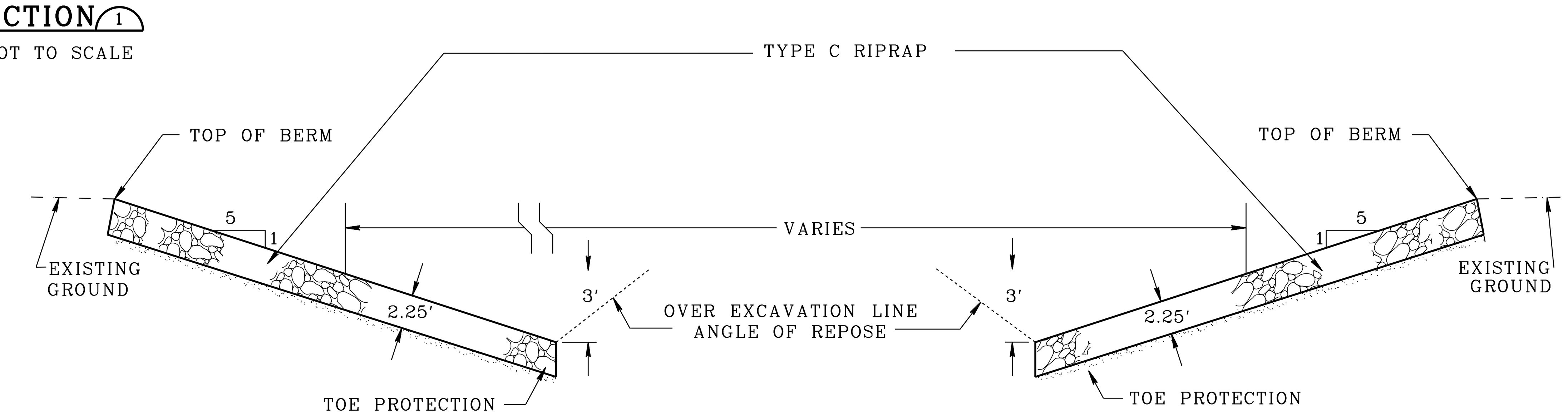

NOTES:

1. MATERIAL PLACED BELOW THE CHANNEL INVERT ELEVATION DOES NOT REQUIRE BACKFILLING.

SECTION 2

NOT TO SCALE

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

SYMBOL	DESCRIPTION	DATE	APPROVED

APPROVED BY:
ENGINEERING DIVISION, CHF.
DATE:

DESIGNED BY: C.J.S.
DWN BY: C.J.S.
CKD BY:
REVIEWED BY:
SUBMITTED BY:
CHIEF: CHAN. STAB. & SED. SECTION
DATE: DEC 16, 2011
DESIGN FILE NO.
PLOT SCALE RATIO:
DRAWING CODE: X
REV.
SPEC. NO.
XXXXX-XX-X-XXXX
CONTRACT NO.
TASK ORDER #XX

INTAKE FISH PASSAGE BYPASS
YELLOWSTONE RIVER MONTANA
BYPASS VERTICAL CONTROL SILLS

SECTIONS

SHEET REFERENCE NUMBER:

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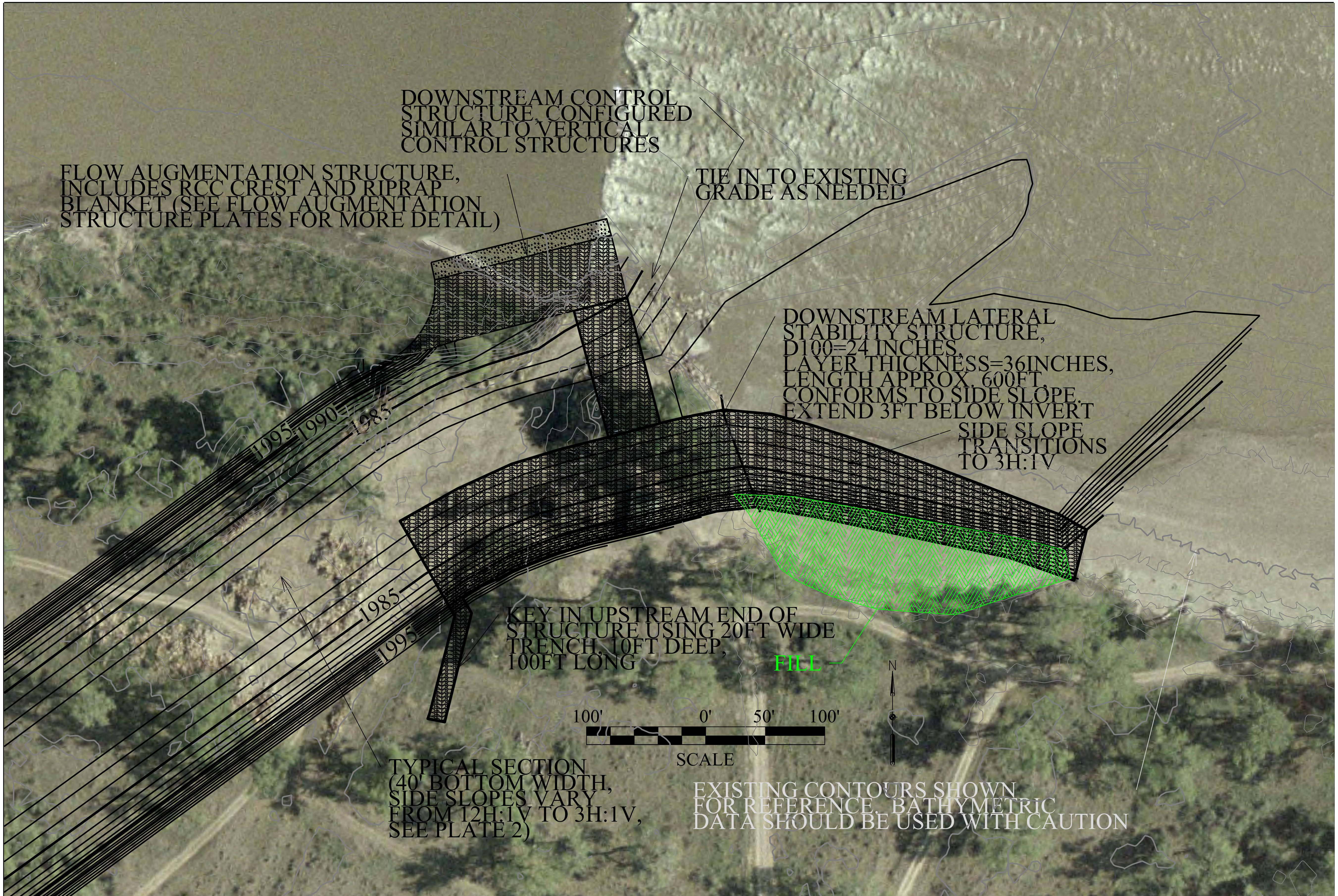

Designed by: C.J.S/C.J.M.

Drawn by: C.J.S/C.J.M.

Reviewed by:

U.S. Army Corps of Engineers Omaha District

Date:

Computer File:

Plot Scale:

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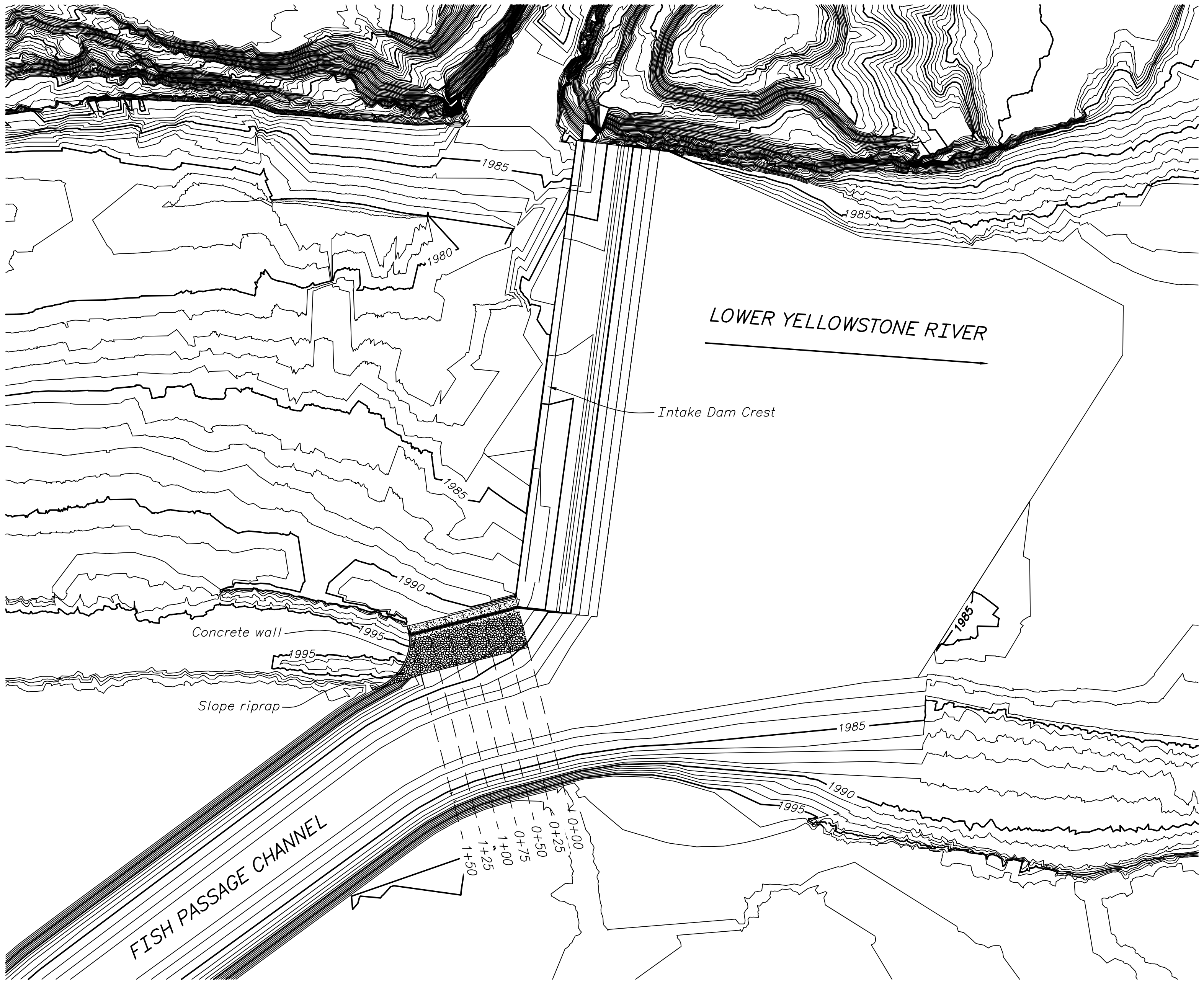
Att6, AppC



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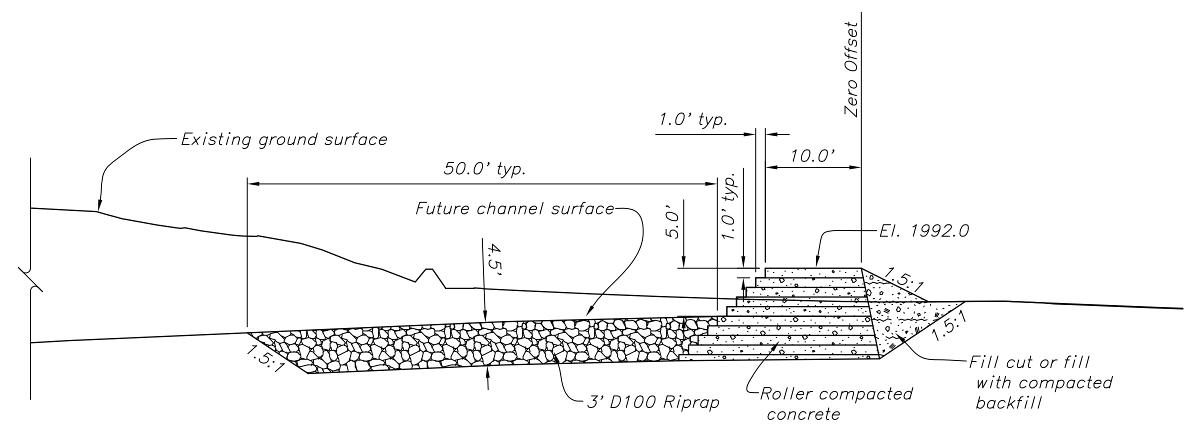

TYPICAL SECTION

(Not to scale)

NOTE

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RECLAMATION
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BUREAU OF RECLAMATION

LOWER YELLOWSTONE RIVER
INTAKE DAM
FISH PASSAGE – OPTION 1

DESIGNED
DRAWN
CHECKED
TECH. APPR. NAME – TITLE
APPROVED PEER REVIEWER – NAME – TITLE
DENVER, COLORADO Dec. 20, 2011

SHEET 1 OF 3

CADD SYSTEM
AutoCAD Rel. 18.1s (LMS Tech)
CADD FILENAME
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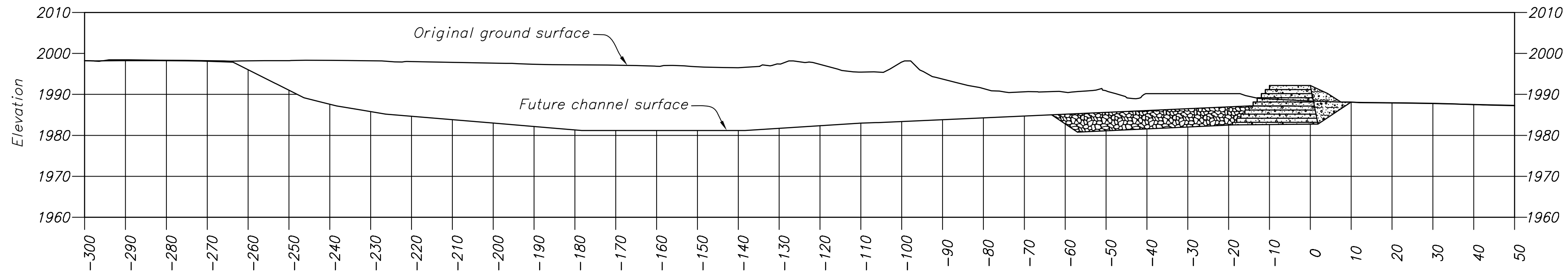
DATE AND TIME PLOTTED
December 23, 2011 12:07
PLOTTED BY
MCAMPBELL

RECLAMATION
Managing Water in the West



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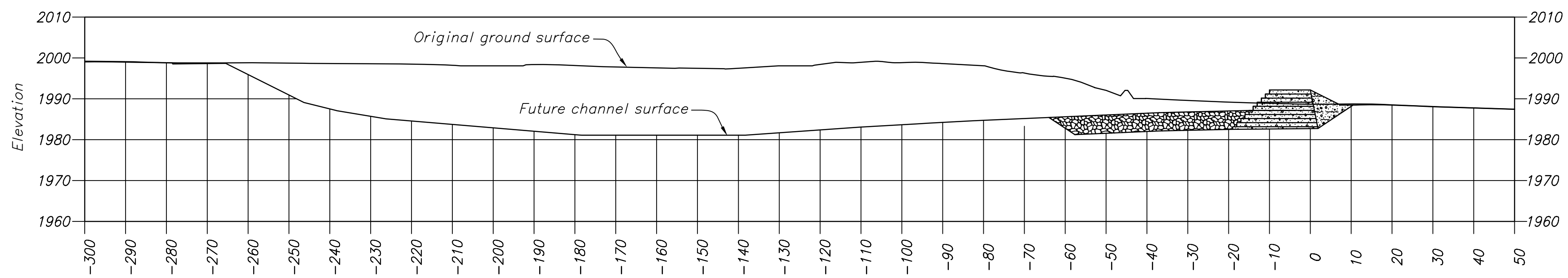

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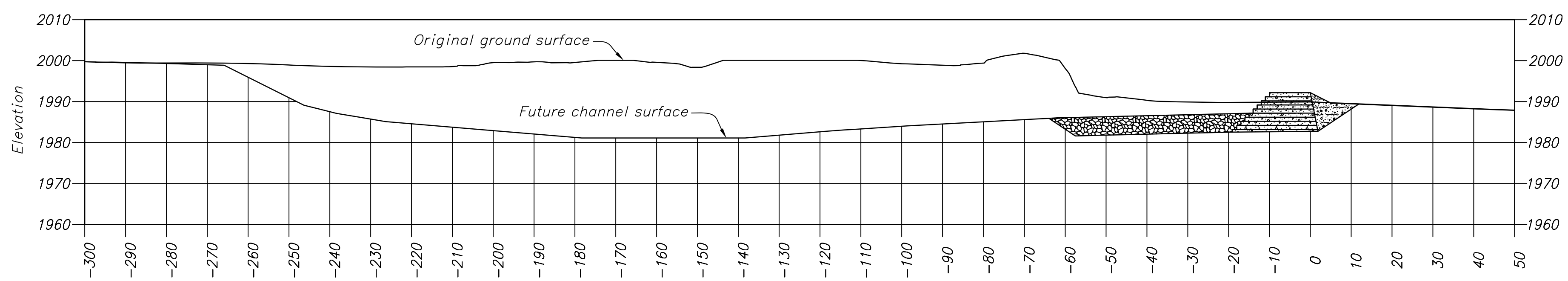

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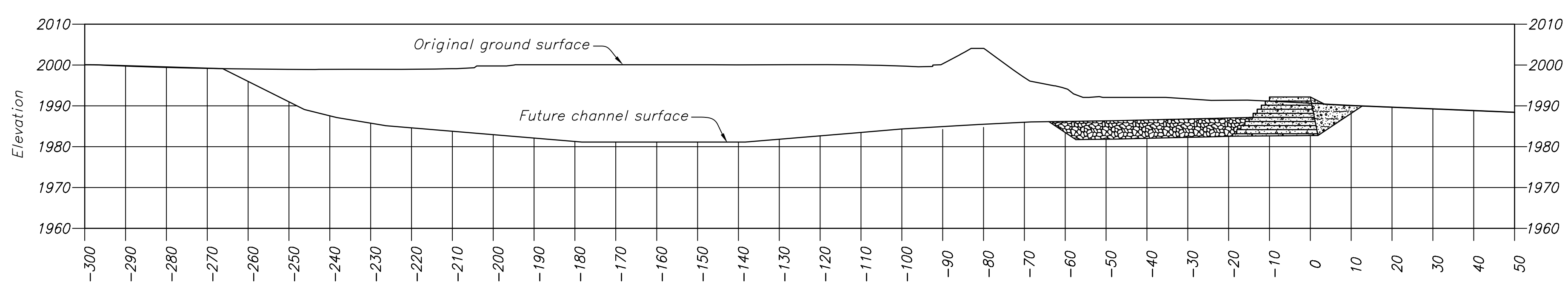

SECTION AT STA. 0+75

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LOWER YELLOWSTONE RIVER
INTAKE DAM
FISH PASSAGE - OPTION 1

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DENVER, COLORADO Dec. 20, 2011

SHEET 3 OF 3

DATE AND TIME PLOTTED
December 23, 2011 12:07
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MCAMPBELL

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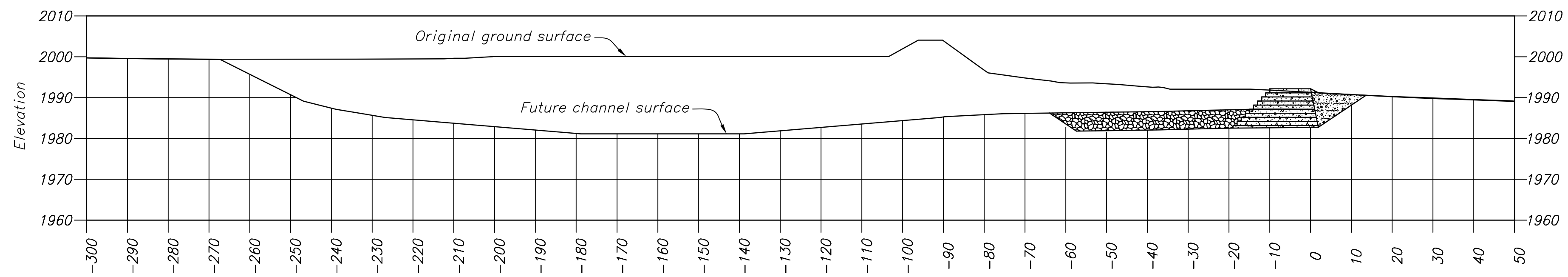

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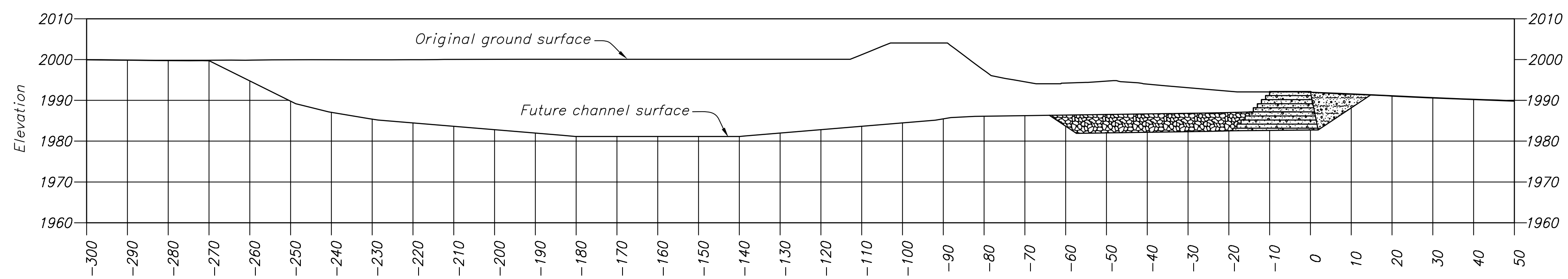

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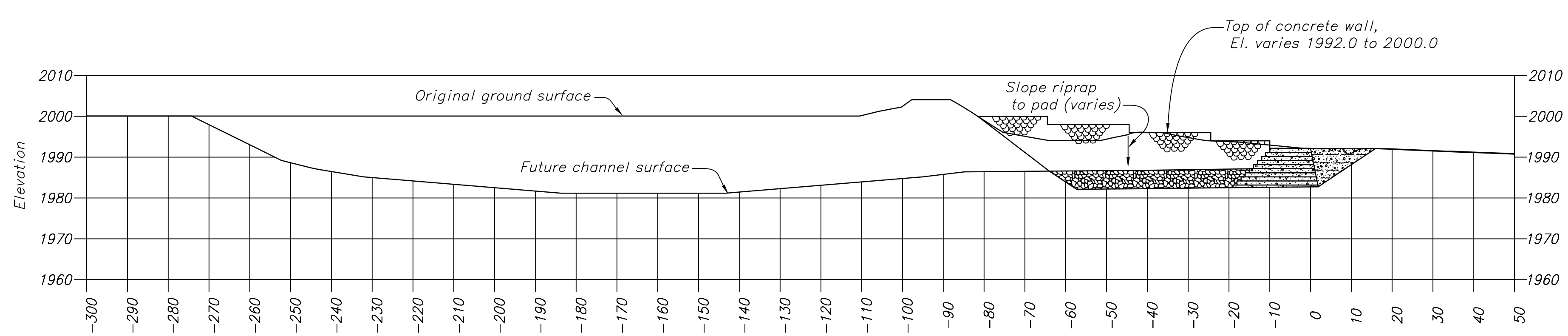

SECTION AT STA. 1+50

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BUREAU OF RECLAMATION

LOWER YELLOWSTONE RIVER
INTAKE DAM
FISH PASSAGE - OPTION 1

DESIGNED
DRAWN
CHECKED
TECH. APPR. NAME - TITLE
APPROVED PEER REVIEWER - NAME - TITLE
DENVER, COLORADO Dec. 20, 2011

SHEET 3 OF 3

DATE AND TIME PLOTTED December 23, 2011 12:08 PLOTTED BY MCAMPBELL

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Attachment 6 Bypass Channel

Appendix D

Reference Reach Comparison

19March2012

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US Army Corps of Engineers
Omaha District

Project:	Lower Yellowstone-Intake				Sheet No.	1/1
Subject:	Reference Reach Comparison					
Computed by:	CJM	Date:	MAR2012	Checked by:		Date:

INTRODUCTION

This document describes an evaluation of eleven side channels on the Yellowstone River, including the existing high flow chute at Lower Yellowstone, Intake. Six of the side channels evaluated are downstream from Intake, four are upstream.

The intent of the evaluation is to compare existing, natural side channels to the proposed bypass channel at intake. It should be stressed that the comparison is simply a GIS exercise and does not guarantee project performance. Additional data and a more in-depth analysis will be required to determine the long term stability of the project.

COMPARISON

Available GIS data, aerial photography, and HEC-RAS data were used to compare 11 natural side channels within 60 river miles of Intake Dam. The comparison consisted mainly of measuring side channel length and width and using HEC-RAS or available LiDAR data to estimate energy grades. Dates of aerial photography were used to estimate discharges at the sites based on the USGS gages at Glendive and Sidney.

Plate 1 consists of a table summarizing the evaluation along with assumptions used.

Plate 2 shows a general overview of the area.

Plates 3-13 show each of the individual sites.

CONCLUSIONS

Both the short and long bypass channels (15500 ft and 23250ft) fall in the relative range of the reference reaches compared.

The chute to main channel length ratio for the shorter bypass, while falling in the range, is on the high end of those compared with only one reference reach higher. The longer bypass is higher by a third than the highest reference reach.

Both channels fall in the range of estimated energy grade slope.

Chute sinuosity for the shorter bypass falls at the high end of the evaluated range while the longer channel is again nearly a third higher than the reference reach with the highest sinuosity.

The top width of both of the proposed channels falls in the range of the reference reaches.

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Reach Identifier	River Mile[1]	Bank	Orientation and distance from Intake Dam[2]	Approximate chute length	Approximate main channel length	Chute to Main Channel length ratio	Approximate energy grade slope in main channel[3]	Estimated energy grade slope in reference reach chute	Straight line distance, end to end of chute	Sinuosity (in this context, used ratio of chute length to straight line distance)	Approximate range of chute top width		Approximate Yellowstone River discharge at which chute flows (rough estimates broken into broad classes)
											Low flow[4]	Mid range flow[5]	
				(ft)	(ft)		(ft/ft)	(ft/ft)	(ft)		(ft)	(ft)	(cfs)
1	18.3-19.8	Right	54 miles d/s	9900	7900	1.3	0.0001	0.00008	6400	1.5	50-110	140-200	≤5000
2	34.0-35.7	Left	38 miles d/s	9400	8900	1.1	0.0004	0.0004	8100	1.2	15-30	30-50	≤5000
3	37.7-39.4	Right	34 miles d/s	11400	9000	1.3	0.0003	0.0002	7500	1.5	20-50[6]	50-120	20,000<x<40,000
4	41.0-43.3	Left	31 miles d/s	22100	12400	1.8	0.0006	0.0004	11300	2.0	20-90[7]	100-160	5000<x<20,000
5	52.7-54.6	Right	19 miles d/s	10600	10000	1.1	0.0006	0.0006	9200	1.2	60-200[7]	250-400	5000<x<20,000
6	62.7-64.6	Right	9 miles d/s	8700	9800	0.9	0.0005	0.0006	5700	1.5	120-280[8]	N/A	<5000
7	70.8-74.3	Right	Existing chute at Intake	24700	18400	1.3	0.0007	0.0005	16200	1.5	40-120[6]	N/A	25,000-30,000
8	90.0-90.8	Left	17 miles u/s	5000	4500	1.1	0.00065	0.0006	4200	1.2	40-120[7]	N/A	5000<x<20,000
9	94.5-96.5	Left	23 miles u/s (at Glendive)	13600	10800	1.3	0.0004	0.0003	10000	1.4	60-200	N/A	<5000
10	99.8-101.8	Right	28 miles u/s	10400	10500	1.0	0.0005	0.0005	9500	1.1	40-150[9]	N/A	>5000[10]
11	105.8-107.1	Right	33 miles u/s	7500	6800	1.1	0.0007	0.0006	6400	1.2	70-100[9]	N/A	>5000[10]
PROPOSED BYPASS	72.4-47.3	Right	Proposed bypass at Intake	15500-23250	9600	1.6-2.4	0.0007	0.0004-0.0006	8300	1.9-2.8	100	190	≤5000

Footnotes:

1	Approximate downstream and upstream extents of chute based on main channel river station in miles from mouth
2	Intake Dam is located at approximately RM 73
3	For reaches 1-6 (located in Richland County), used 2007 LiDAR survey data assuming data in river reflects approximate water surface elevation. For reaches 7-11 (located in Dawson County), used USACE created HEC-RAS model and averaged energy grade slope from range of profiles.
4	Based on measurements using aerial photography from 15Oct2007 to 2Nov2007 for reaches 1-6 (Richland County), discharges at Glendive (and Sidney)≈5000-7000cfs; aerials from 1-2May2004 for reaches 7-11 (Dawson County), discharges at Glendive≈4000-6000cfs, at Sidney≈3000cfs.
5	Based on measurements using Google Earth imagery from 22June2009, discharge at Glendive≈48,000cfs, Sidney≈46,000cfs where available.
6	Appears that chute is intermittent; i.e. may not be carrying water at low Yellowstone River flow. Using aerial photography from ArcGIS Map Service, ESRI_Imagery_World_2D, still shows intermittent flow in chute but with additional area inundated; date noted for imagery is 14July2005, discharge at Glendive≈17,000cfs, Sidney≈16,000cfs. July 2005 imagery was on receding limb of hydrograph that reached >40,000cfs near the end of June/beginning of July.
7	Appears that chute is intermittent; i.e. may not be carrying water at low Yellowstone River flow. However, aerial photography from ArcGIS Map Service, ESRI_Imagery_World_2D, shows continuous flow in chute; date noted for imagery is 14July2005, discharge at Glendive≈17,000cfs, Sidney≈16,000cfs
8	Contains mid channel bars
9	Appears that chute is intermittent; i.e. may not be carrying water at low Yellowstone River flow. Only other available aerial photography from ArcGIS Map Service, ESRI_Imagery_World_2D, still shows intermittent flow in chute; date noted for imagery is 31July2005, discharge at Glendive≈6300cfs. July 2005 imagery was on receding limb of hydrograph that reached >40,000cfs near the end of June/beginning of July.
10	May be much larger than 5000cfs; lack of available data prevents determination of range.

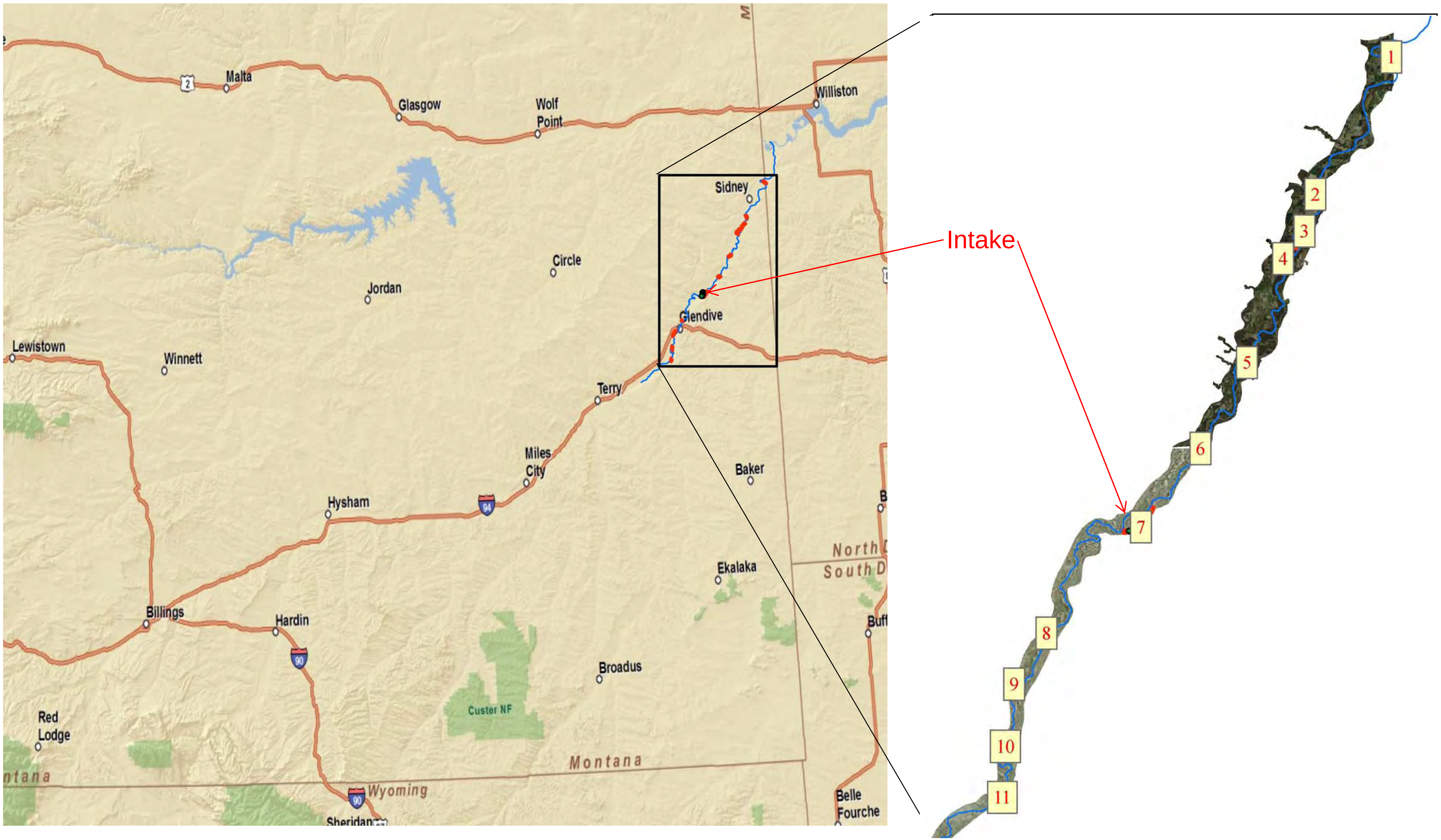

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Plate 2

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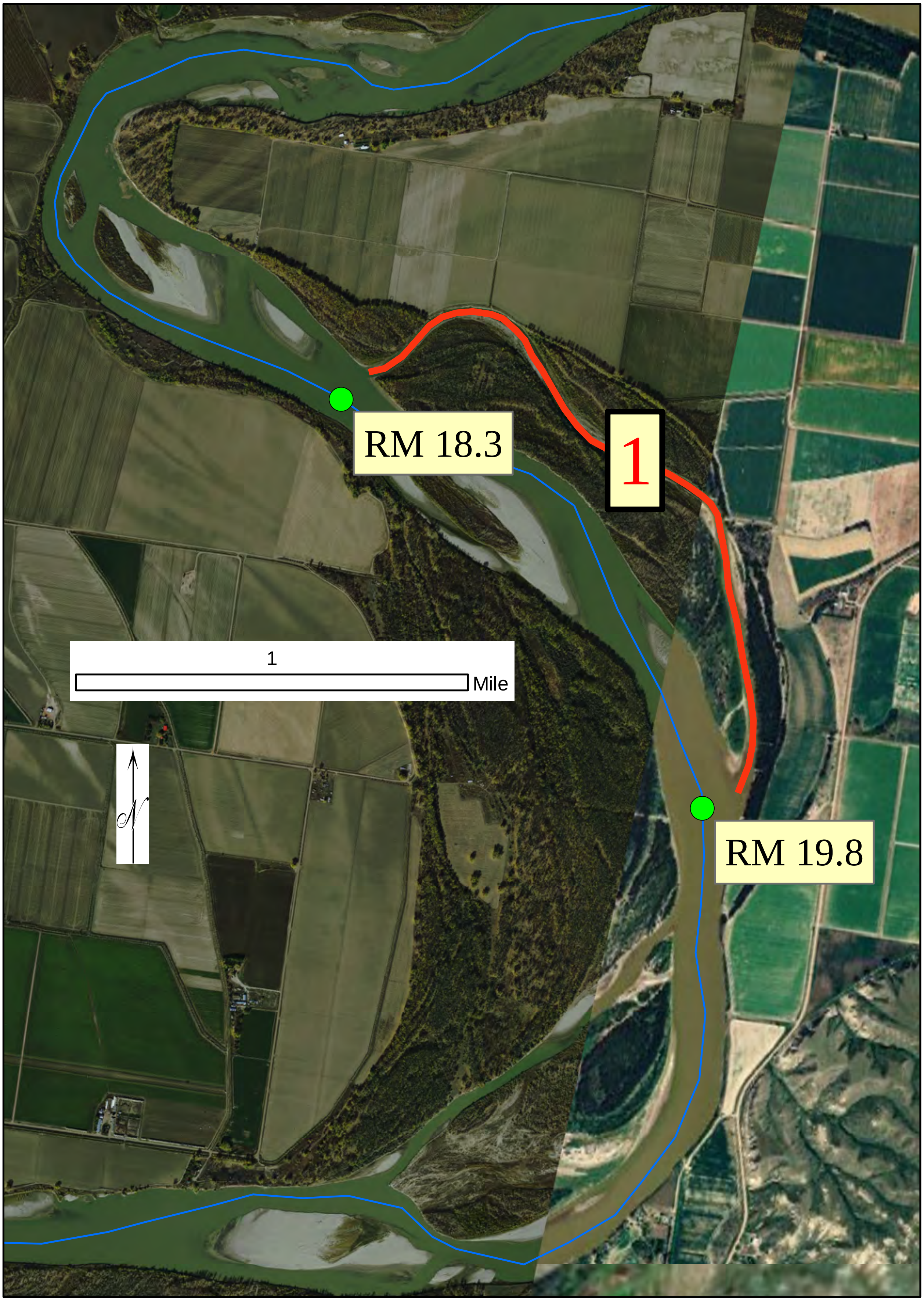

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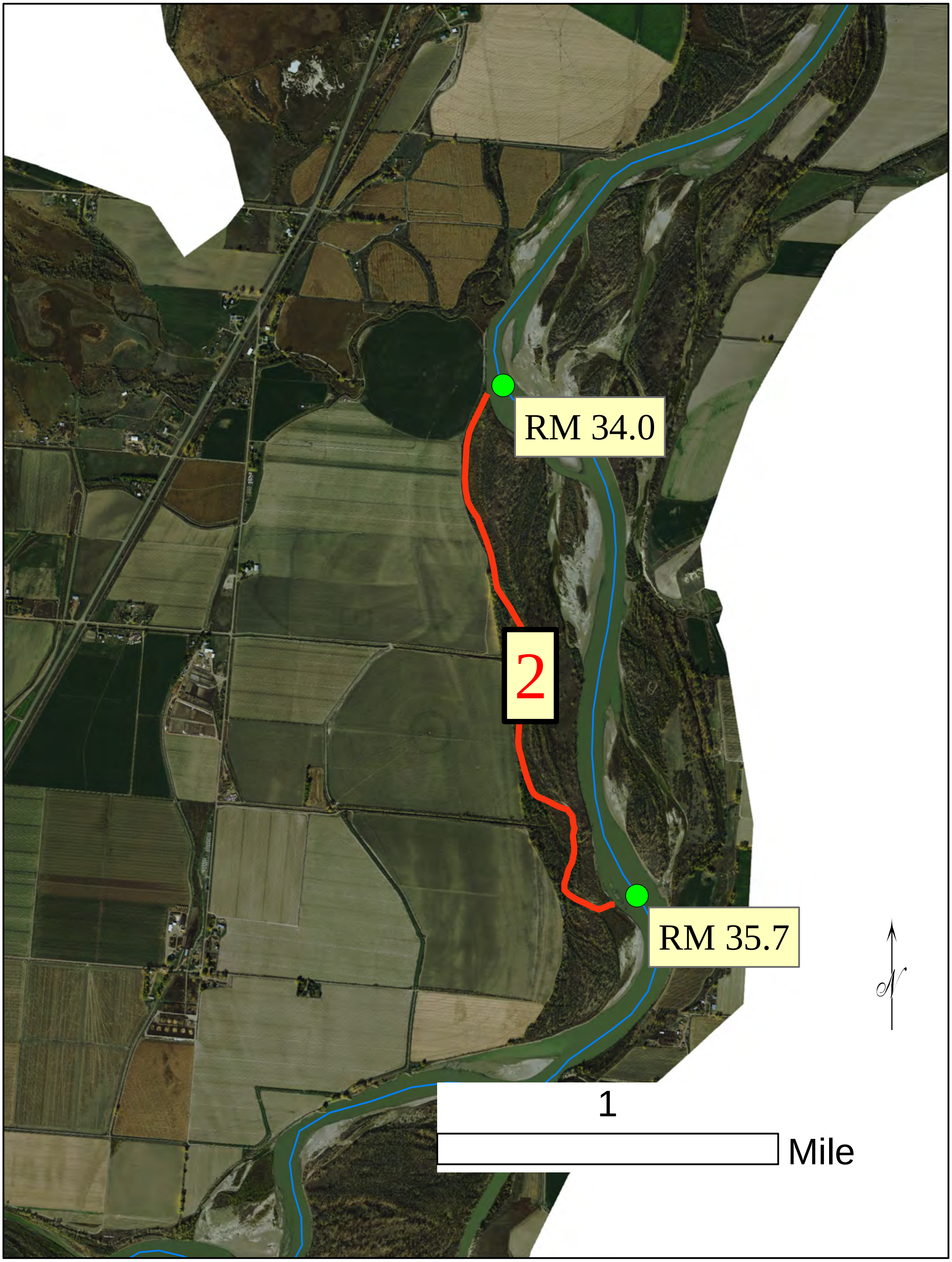

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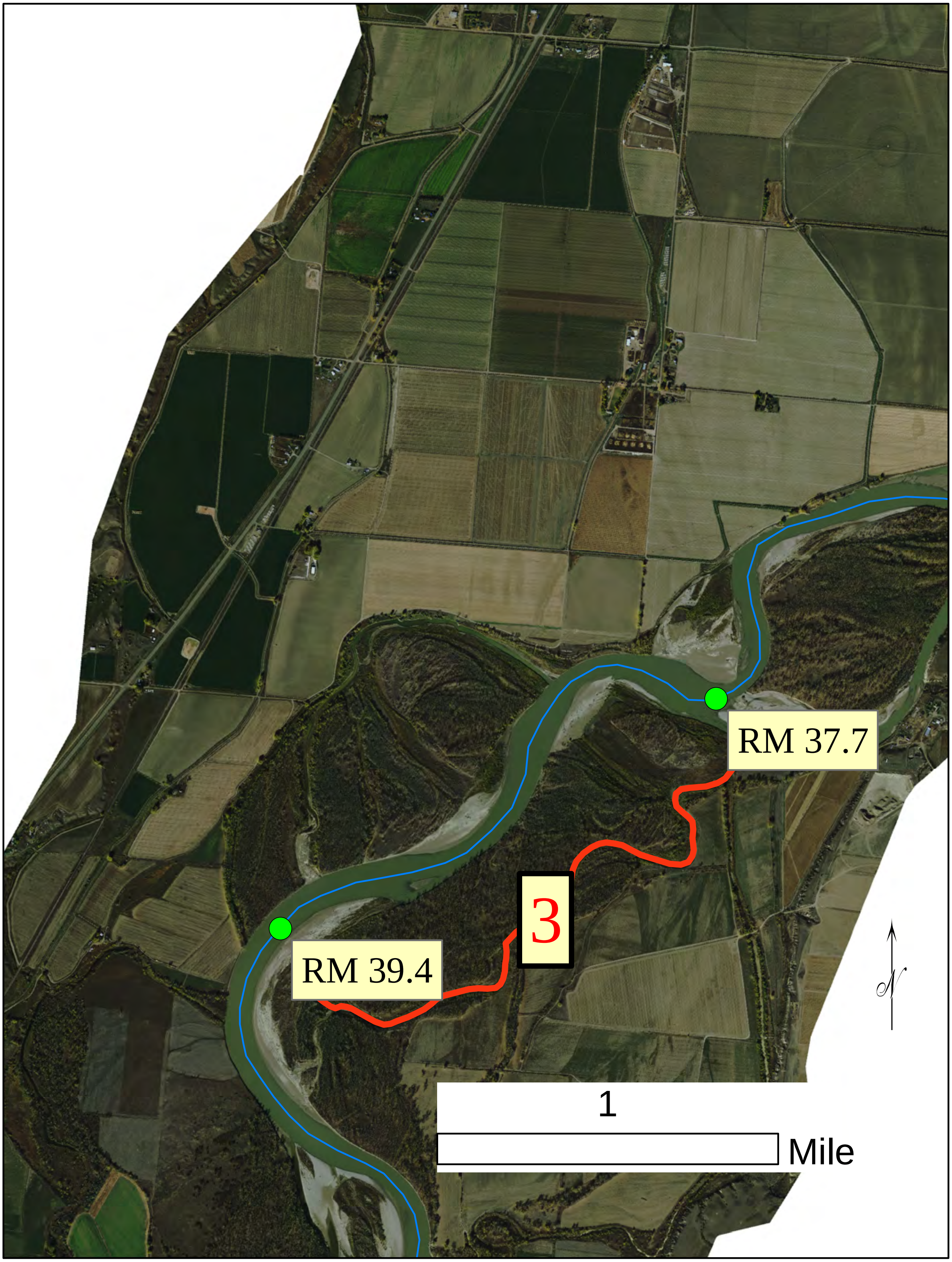

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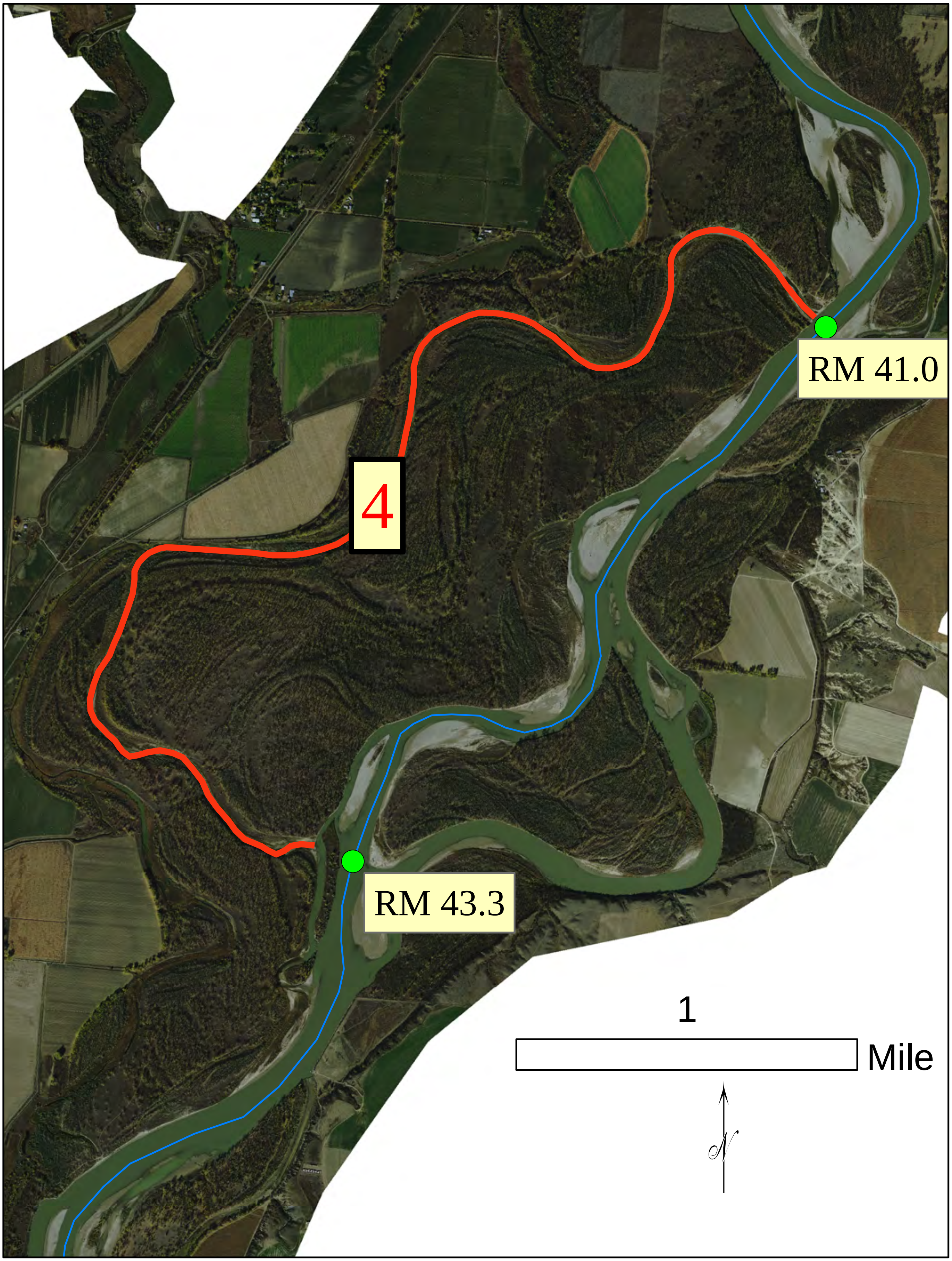

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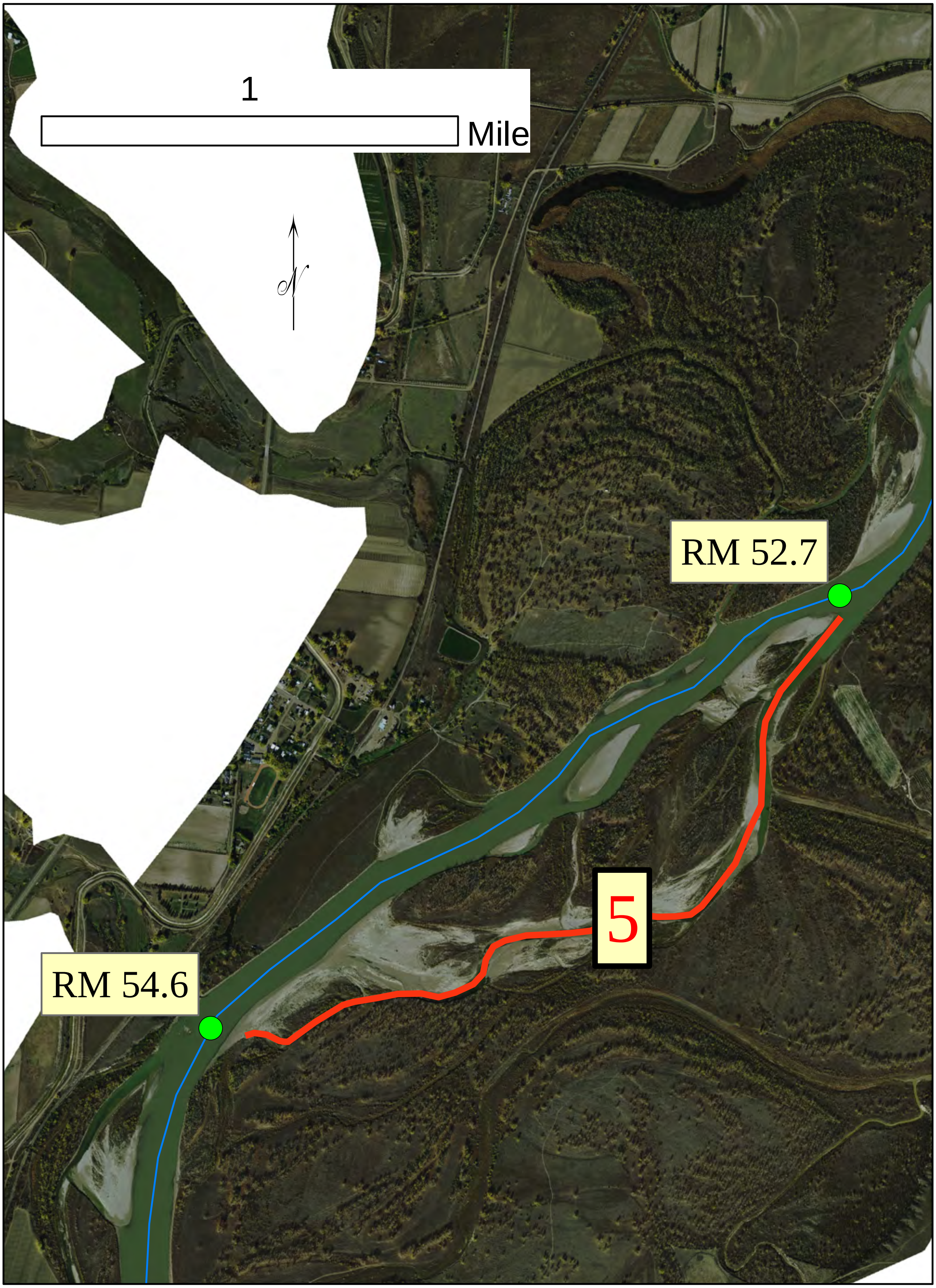

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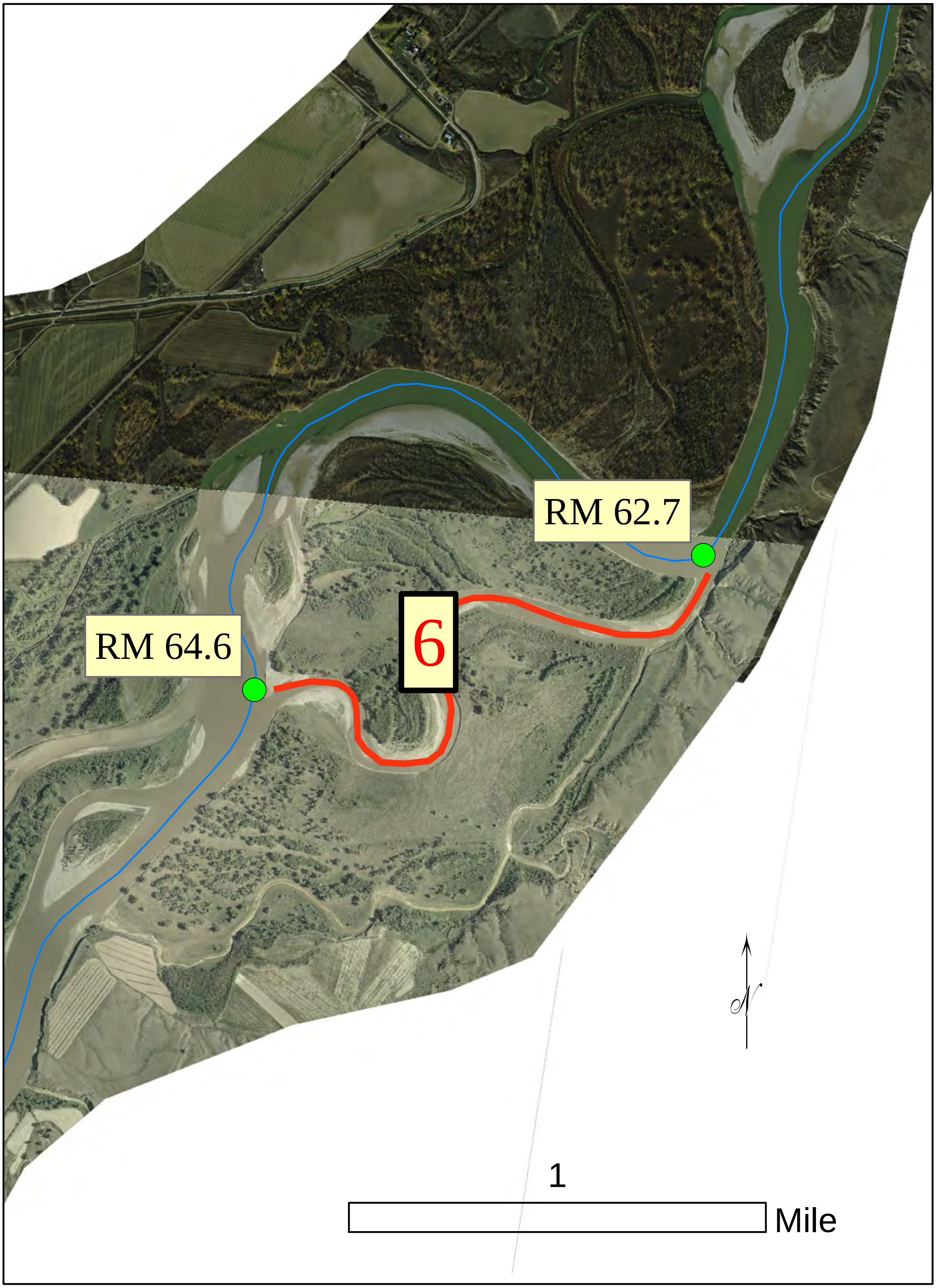

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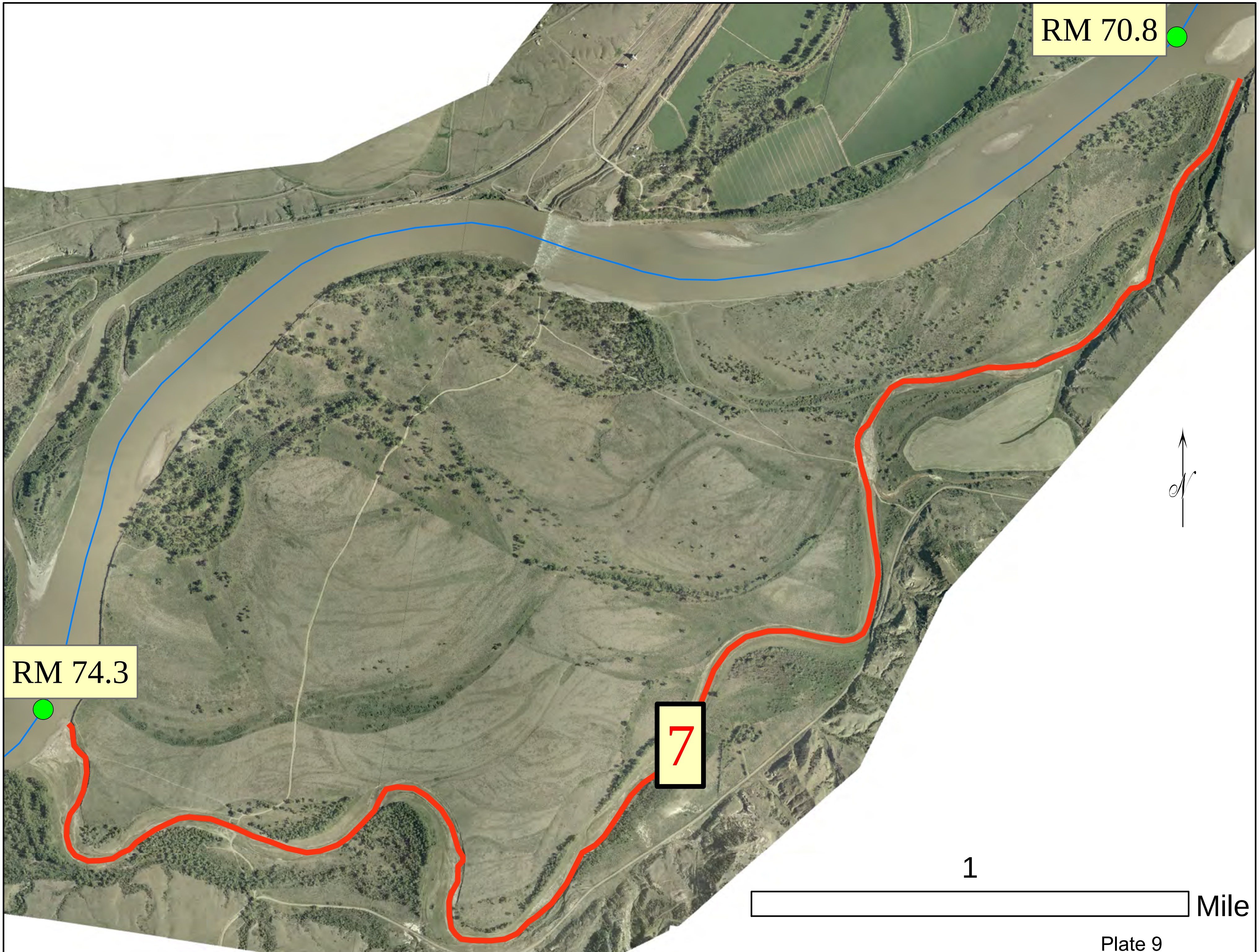

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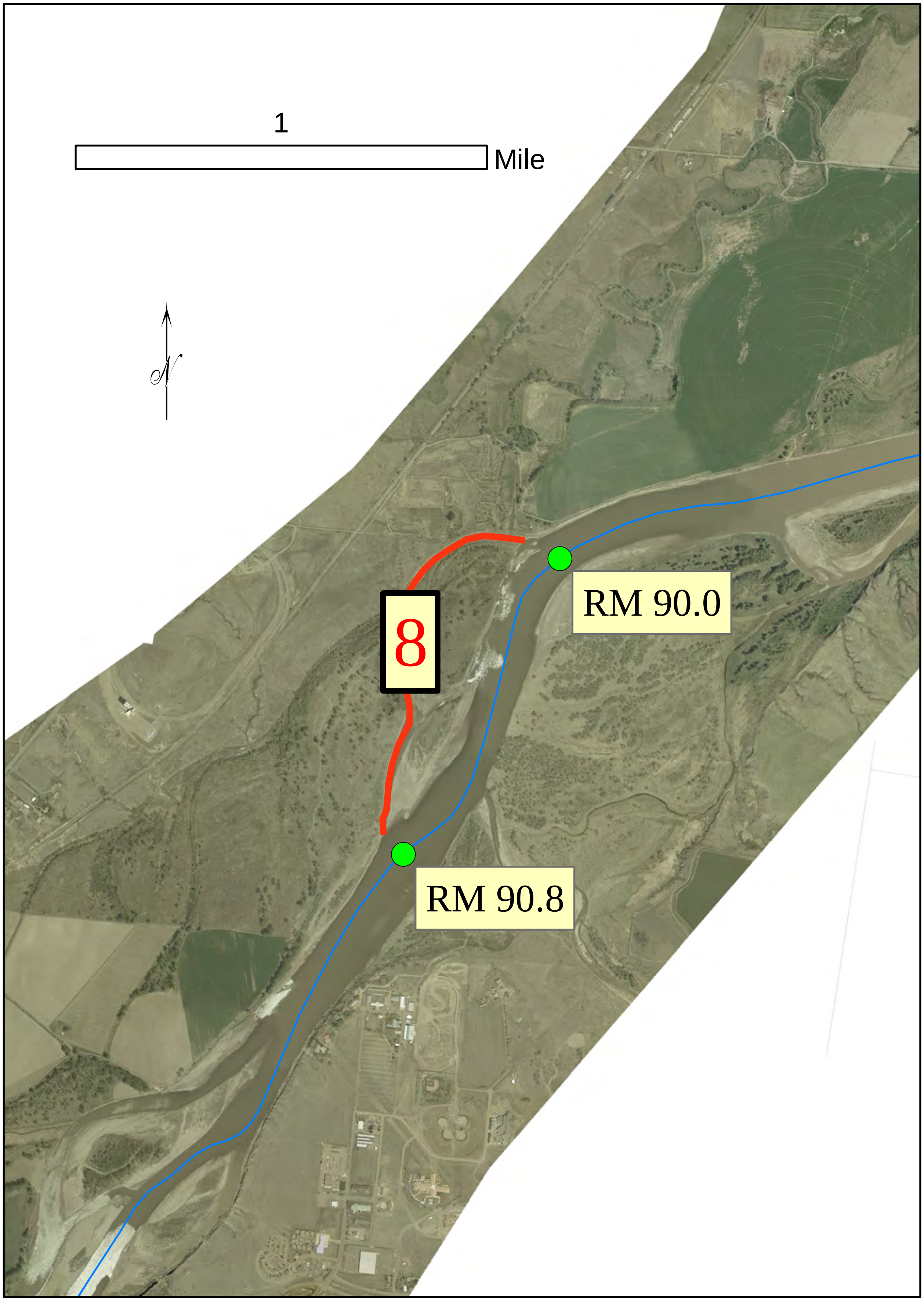

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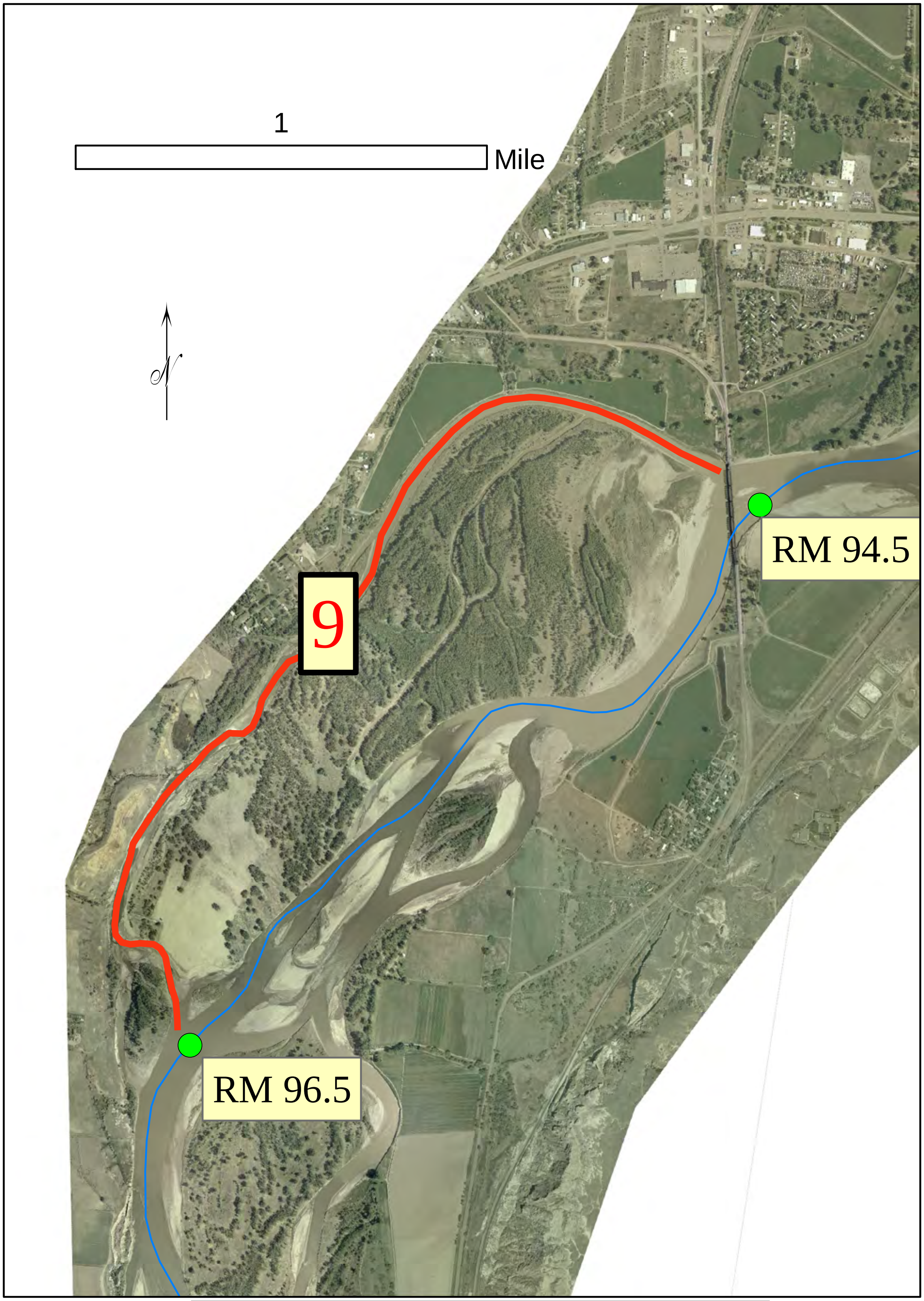

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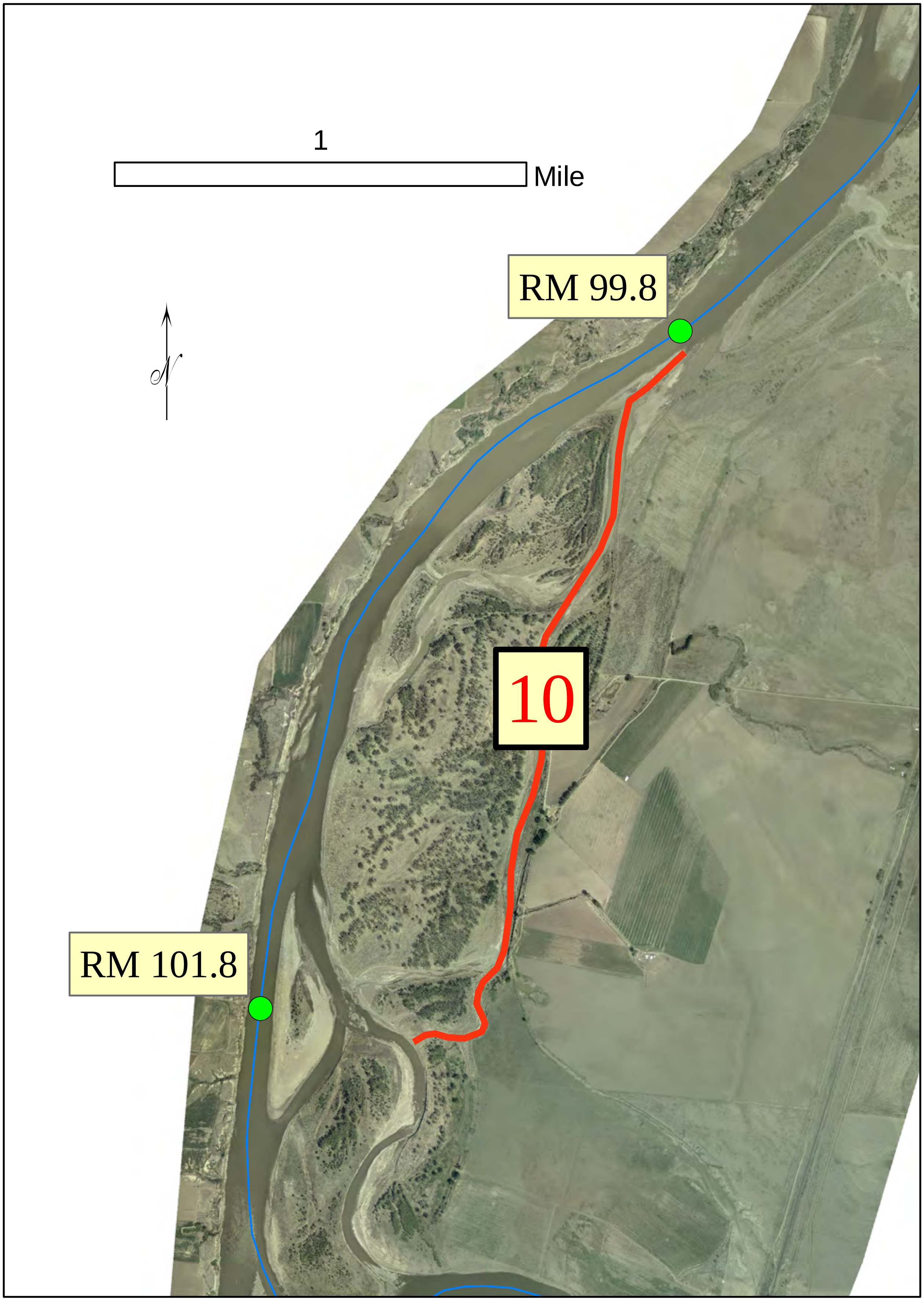

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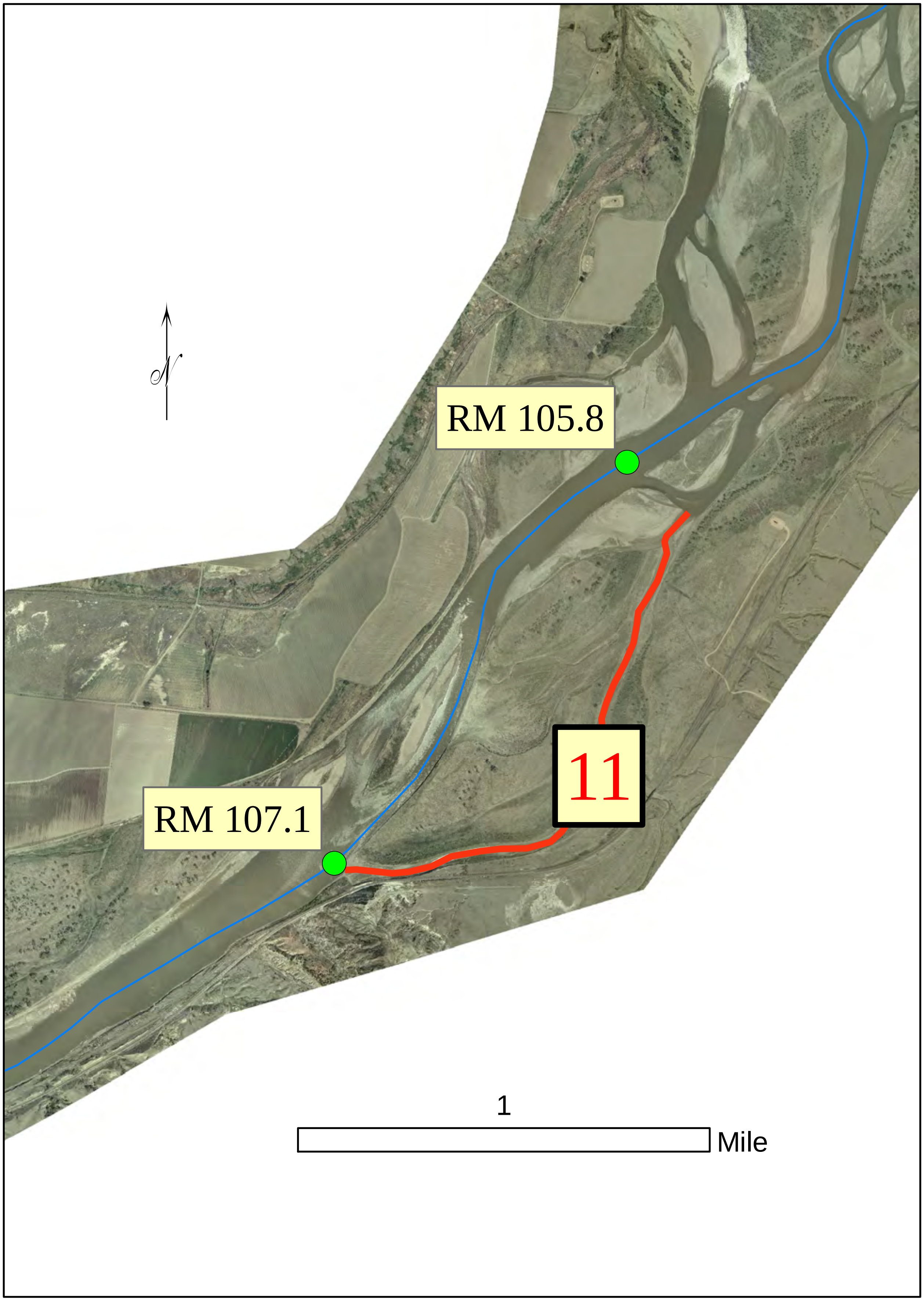

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Attachment 6 Bypass Channel

Appendix E

Bypass Channel-Stable Channel Materials Analysis

19March2012

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US Army Corps of Engineers ®
Omaha District

Intake Dam Fish Bypass Channel
Stable Channel Material Analysis
CENWO-ED-HF
March 2012

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1. INTRODUCTION

The purpose of this document is to analyze various methods to approximate the characteristics of the armor layer likely to form for the proposed bypass project at Intake. Various methods will be utilized ranging from HEC-RAS Stable Channel Analysis, estimates on stability thresholds, analysis of test pit results at the site, and Shields stability criteria.

2. HEC-RAS Stable Channel Analysis Tool

The channel design functions within HEC-RAS are based upon the methods available in the SAM Hydraulic Design Package. For this analysis, the Copeland method was utilized to define the potential for aggradation or degradation within the proposed channel. The tool only allows for analysis of trapezoidal sections, so the proposed channel was approximated using a 60' bottom width and 5:1 sideslopes. For this analysis, 5500 cfs was selected as the channel forming discharge.

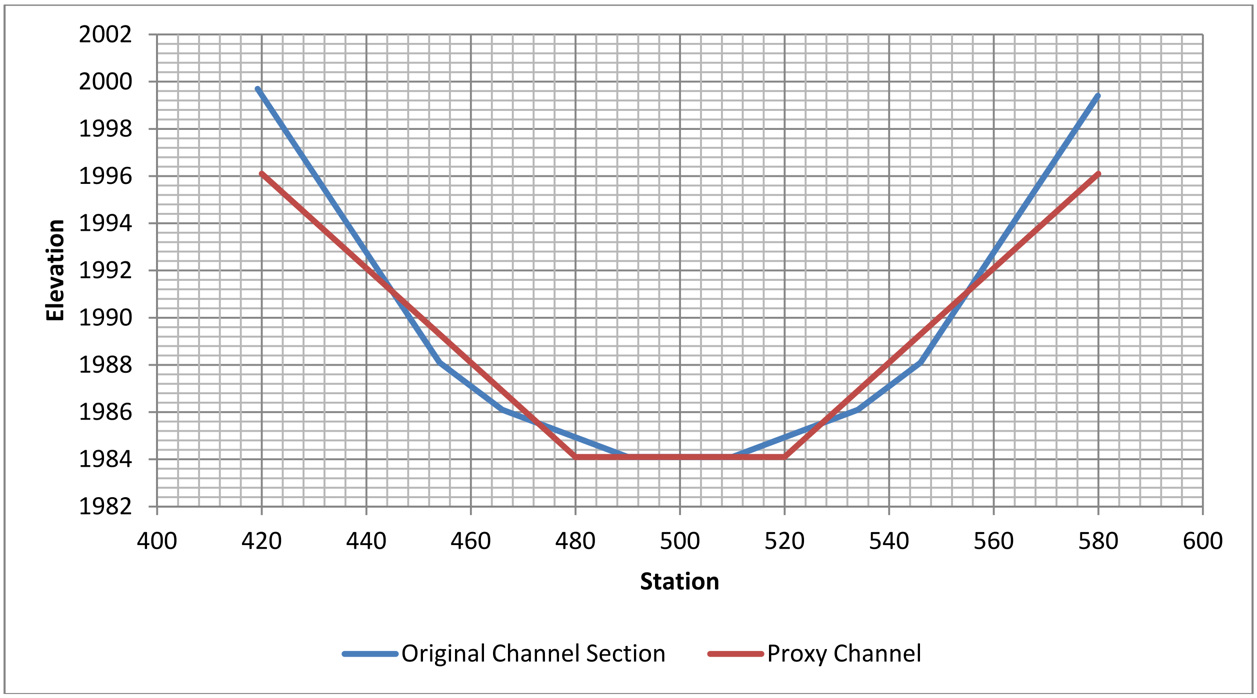

Figure 1 - Channel Approximation

Sediment samples collected on site by the USGS in the summer of 2011 were used to approximate sediment load and the upstream contributing section was assumed to have a 40' base width and a channel slope of .0006 ft/ft. HEC-RAS utilizes the upstream geometry to approximate sediment concentrations, though the concentration can also be input manually.

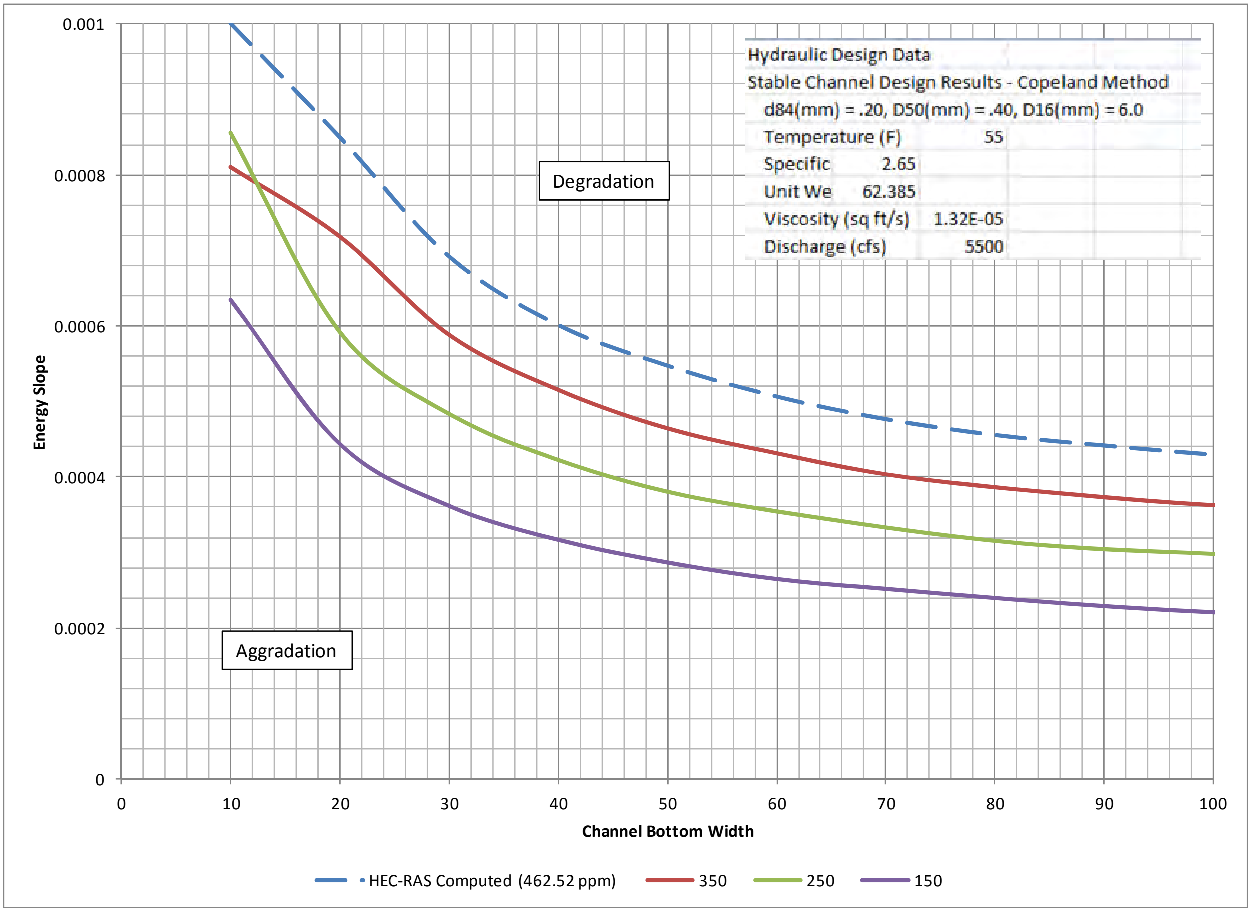

Figure 2 - HEC-RAS Copeland Method Stable Channel Curves for Proposed Bypass

For the proposed geometry, HEC-RAS computed a suspended sediment concentration of 462.52 mg/l. Samples from USGS collection efforts ranged from 397 to 537 mg/l.

3. Stability Thresholds for Stream Restoration Materials

Traditional approaches for characterizing erosion potential can be placed in one of two categories; maximum permissible velocity and tractive force (critical shear stress). In May of 2001, ERDC published a document summarizing these methods.

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Boundary Category	Boundary Type	Permissible Shear Stress (lb/sq ft)	Permissible Velocity (ft/sec)	Citation(s)
Soils	Fine colloidal sand	0.02 - 0.03	1.5	A
	Sandy loam (noncolloidal)	0.03 - 0.04	1.75	A
	Alluvial silt (noncolloidal)	0.045 - 0.05	2	A
	Silty loam (noncolloidal)	0.045 - 0.05	1.75 – 2.25	A
	Firm loam	0.075	2.5	A
	Fine gravels	0.075	2.5	A
	Stiff clay	0.26	3 – 4.5	A, F
	Alluvial silt (colloidal)	0.26	3.75	A
	Graded loam to cobbles	0.38	3.75	A
	Graded silts to cobbles	0.43	4	A
	Shales and hardpan	0.67	6	A
Gravel/Cobble	1-in.	0.33	2.5 – 5	A
	2-in.	0.67	3 – 6	A
	6-in.	2.0	4 – 7.5	A
	12-in.	4.0	5.5 – 12	A
Vegetation	Class A turf	3.7	6 – 8	E, N
	Class B turf	2.1	4 - 7	E, N
	Class C turf	1.0	3.5	E, N
	Long native grasses	1.2 – 1.7	4 – 6	G, H, L, N
	Short native and bunch grass	0.7 - 0.95	3 – 4	G, H, L, N
	Reed plantings	0.1-0.6	N/A	E, N
	Hardwood tree plantings	0.41-2.5	N/A	E, N
Temporary Degradable RECPs	Jute net	0.45	1 – 2.5	E, H, M
	Straw with net	1.5 – 1.65	1 – 3	E, H, M
	Coconut fiber with net	2.25	3 – 4	E, M
	Fiberglass roving	2.00	2.5 – 7	E, H, M
Non-Degradable RECPs	Unvegetated	3.00	5 – 7	E, G, M
	Partially established	4.0-6.0	7.5 – 15	E, G, M
	Fully vegetated	8.00	8 – 21	F, L, M
Riprap	6 – in. d_{50}	2.5	5 – 10	H
	9 – in. d_{50}	3.8	7 – 11	H
	12 – in. d_{50}	5.1	10 – 13	H
	18 – in. d_{50}	7.6	12 – 16	H
	24 – in. d_{50}	10.1	14 – 18	E
Soil Bioengineering	Wattles	0.2 – 1.0	3	C, I, J, N
	Reed fascine	0.6-1.25	5	E
	Coir roll	3 - 5	8	E, M, N
	Vegetated coir mat	4 - 8	9.5	E, M, N
	Live brush mattress (initial)	0.4 – 4.1	4	B, E, I
	Live brush mattress (grown)	3.90-8.2	12	B, C, E, I, N
	Brush layering (initial/grown)	0.4 – 6.25	12	E, I, N
	Live fascine	1.25-3.10	6 – 8	C, E, I, J
	Live willow stakes	2.10-3.10	3 – 10	E, N, O
Hard Surfacing	Gabions	10	14 – 19	D
	Concrete	12.5	>18	H

Table 1 - Permissible Shear and Velocity for Selected Materials (Source: ERDC TN-EMRRP-SR-29)

Figure 3 summarizes permissible velocities and shear stresses for various types of channel lining materials. Utilizing modeling results from a HEC-RAS simulation of the proposed bypass alternative, an ideal channel lining material can be identified.

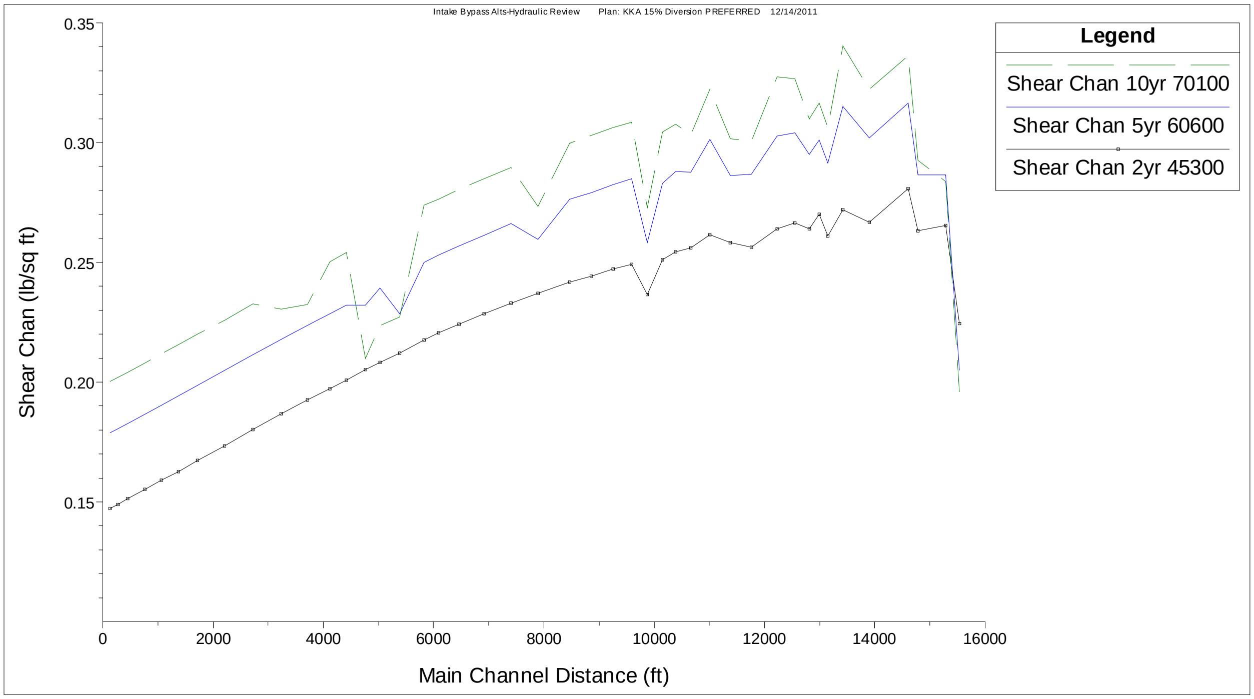

Figure 3 - Bypass Alternative Shear Stress

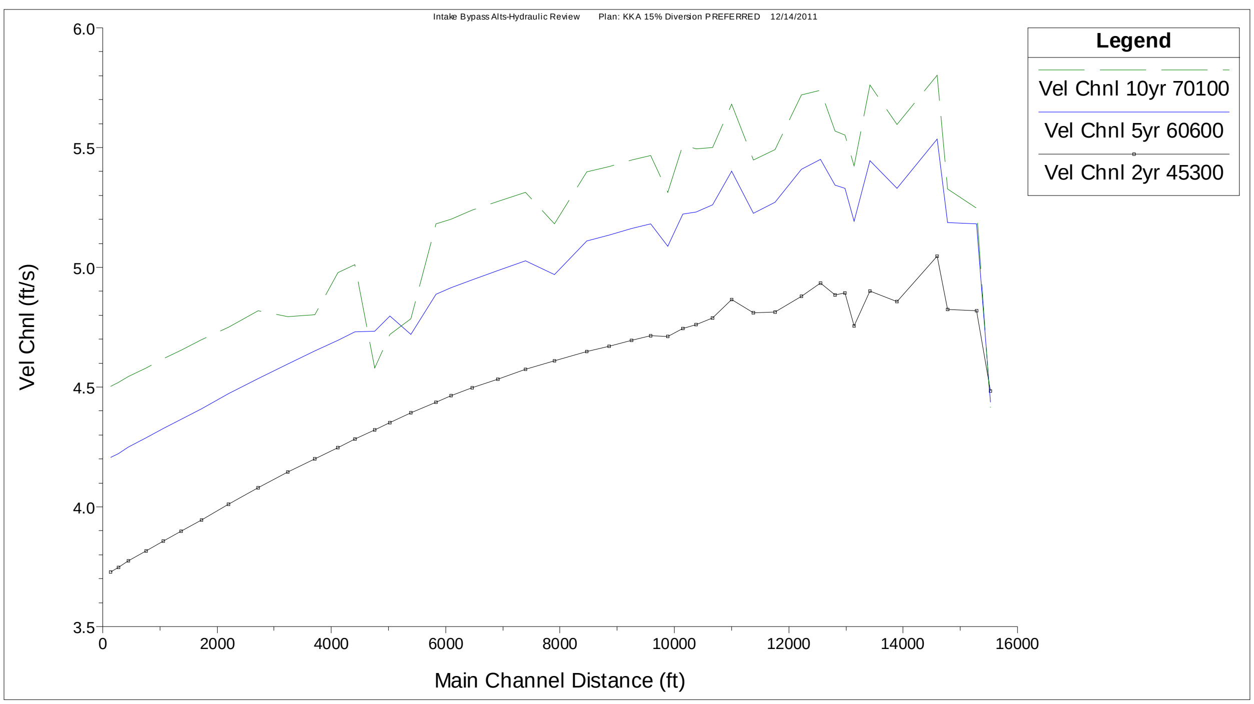

Figure 4 - Bypass Alternative Channel Velocity

Shear stresses in the proposed bypass range from 0.15 lb/sq ft to 0.33 lb/sq ft for the 2 to 10 year events on the Yellowstone River. Average channel velocities for similar events range from 3.7 to 5.7 feet per second. According to guidance summarized in Figure 3, a stable channel material would consist of 1 to 2 inch cobbles based on shear stress and velocity conditions present in the proposed bypass alternative.

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4. Field Investigations

4.1 Test Pit Results

In October of 2011, test pits were dug throughout the island in an attempt to define the types of materials to be encountered in the proposed excavation. Eleven sites were selected throughout the proposed alignment.

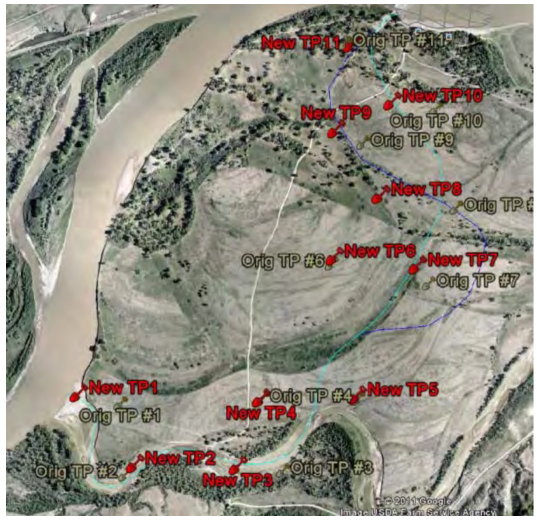

Figure 5 - Bypass Test Pit Locations

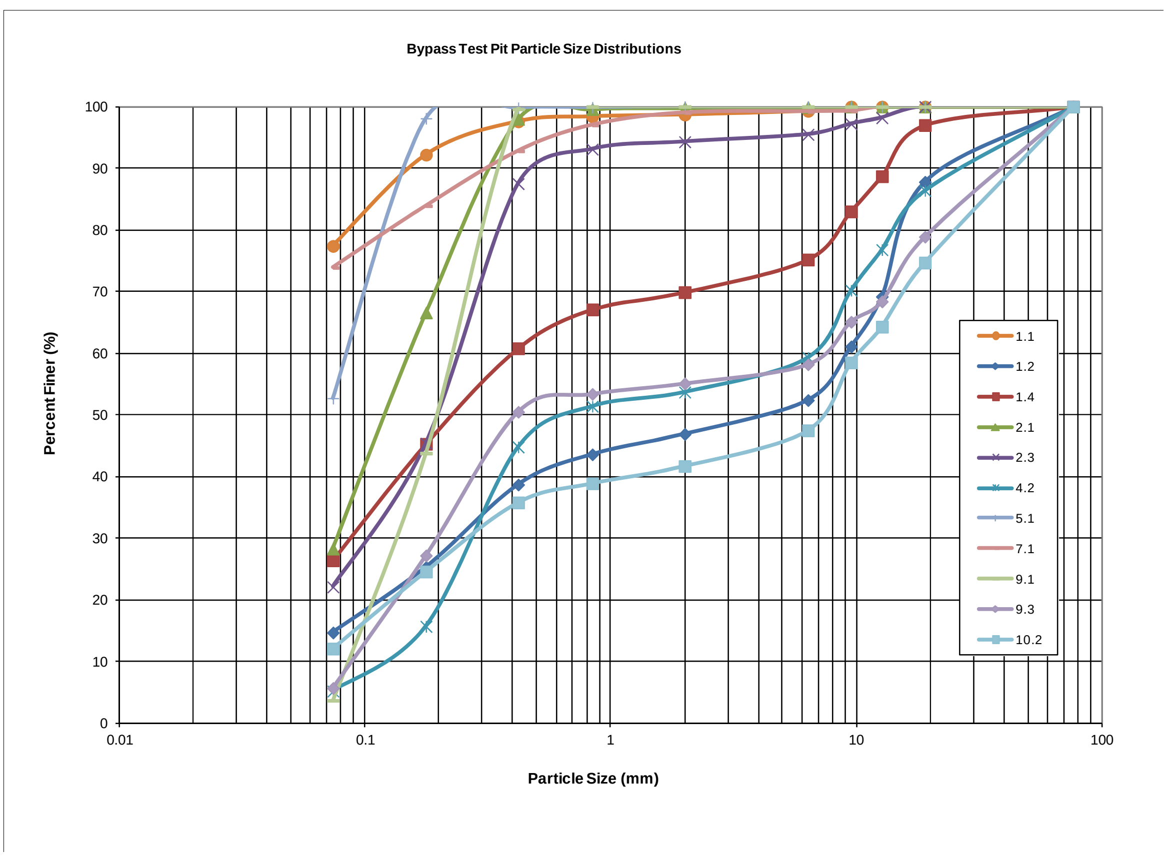

Figure 6 - Bypass Test Pit Results

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Group	Boring and Sample Nos.	Depth (ft)	USCS	3"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#80	#200
A	1.1	2-3	CL	100	100	100	100	99.3	98.7	98.4	97.6	92.2	77.4
B	1.2	3-4	GC	100	87.8	69.2	61.1	52.4	46.9	43.6	38.7	25.3	14.7
C	1.4	6-7	SC	100	97	88.7	83	75.2	69.9	67.1	60.8	45.3	26.4
D	2.1	1.5-4	SC-SM	100	100	100	100	99.9	99.8	99.6	98	66.6	28.3
B	2.2	5-6											
E	2.3	15-16	SC-SM	100	100	98.2	97.2	95.5	94.3	93.1	87.5	45.4	22.1
E	3.1	8-9											
D	4.1	5-6											
F	4.2	12-13	SP-SC	100	86.4	76.8	70.2	59.3	53.7	51.4	44.8	15.7	5.2
F	4.3	24-25											
G	5.1	5-6	ML	100	100	100	100	100	100	99.9	99.8	98.1	52.7
D	5.2	12-13											
E	5.3	17-18											
F	5.4	22-23											
F	6.1	6-7											
F	6.2	10-11											
H	7.1	3-4	CL-ML	100	100	100	99.3	99.2	99	97	92.8	83.9	73.9
B	7.2.1	8-9											
B	7.2.2	8-9											
D	8.1	5-6											
F	8.2	7-8											
I	9.1	4-5	SP	100	100	100	100	100	100	100	99.6	43.8	3.7
F	9.2.1	6-7											
F	9.2.2	6-7											
J	9.3	10-12	SP-SC	100	78.9	68.4	65.1	58.2	55.1	53.4	50.5	27.2	5.7
C	10.1	4.5-5											
K	10.2	11-12	GC	100	74.7	64.3	58.5	47.5	41.7	38.9	35.8	24.6	12.1
I	11.1	3-4											
C	11.2	11-12											
C	11.3	15-16											

Table 2 - Bypass test pit results

Analysis revealed much of the material of the island to be sandy in nature with spots of clays, silts, and gravels. Of the deeper samples collected (1.2, 1.4, 2.3, 4.2, 9.3, 10.2), D50’s ranged from 0.2 – 6.5 mm and D90’s ranged from 0.4 – 13 mm.

Most pit excavations were halted once the water table was reached, due to the material no longer being stable along the vertical walls. Photos collected at the site hint at the presence of material coarser than that documented in the analyzed samples at the bottom of many of the pits.

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Figure 7 - Bypass Test Pit Photos

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4.2 Boring Logs

In November of 2011, 22 boring logs were collected on Joe's Island where the planned bypass would be excavated. Standard penetration test data and disturbed samples were collected in accordance with ASTM D 1586.

The borings were relatively uniform in their findings. In general, the investigation found much of the island to be covered with 6 – 10 feet of Silts (ML), Clays (CL), and Sands (SM). Below this layer, often encountered was a layer of Silty Sandy Gravel (GW) composed of fine to coarse sands and gravel. Though not analyzed for gradation, soils found in this layer would likely contain material appropriate for the formation of an armor layer in the proposed channel and would likely intersect with the proposed excavation invert.

The boring log findings are consistent with the photos captured during the test pit efforts.

5. Shields Stability Analysis

An estimate of the minimum material size for stable bed material can be derived from the equation:

$$d_n \times \frac{S}{(S.G.-1)\sigma} = D_{50}$$

where d_n represents normal depth, S the design slope, $S.G.$ the specific gravity of the material, and σ as the shields factor ranging from .03 to .08 for maximum and minimum ranges. For an assumed flow rate of approximately 7200 cfs (approximating the 2 year flow rate in the bypass) the equation produces a range of 2 to 0.6 inches for the recommended D_{50} for bed stability.

6. Armor Layer Quantity Estimates

Based upon gradation results from the test pits collected, an effort was made to estimate how much material would need to be processed in order to provide for an artificial armor layer to be placed in the proposed bypass. Placement of this layer would increase the short-term stability of the channel following construction by preempting the suspension of materials too small to resist shear stresses to be encountered once the proposed bypass is completed.

The assumption of the analysis was that any material smaller than 20mm would be screened and removed from the source material. The result presented gradations with a D_{50} ranging from 37 to 45 mm (1.4 to 1.8")

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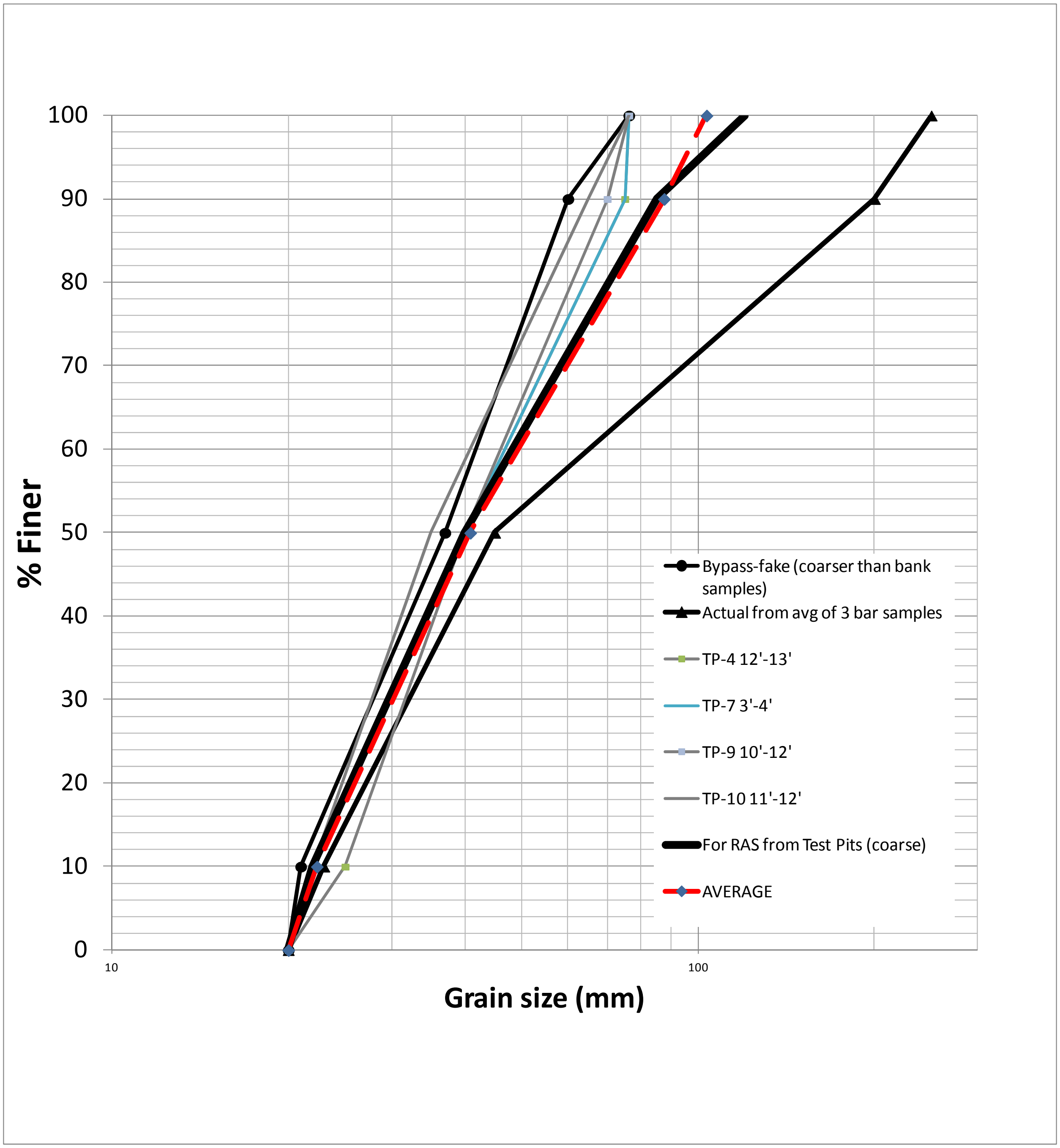

Figure 8 - Bypass bed gradations based upon screening material <20 mm in diameter

From this gradation a variety of placement configurations could be selected based upon level of design. Volume estimates vary greatly based upon the percentage of material assumed to be of the proper size as well. The table below summarizes possible configurations and volumes.

Width	Length	Layer Thickness	Volume	Weight	%Material >20mm	%Material >20mm	%Material >20mm	Volume of Processed Material (CY)	Volume of Processed Material (CY)	Volume of Processed Material (CY)
ft	ft	ft	CY	TONS	Min	Ave	Max	Min	Ave	Max
60	15500	0.5	17222	28417	0.03	0.18	0.38	574074	95679	45322
60	15500	0.75	25833	42625	0.03	0.18	0.38	861111	143519	67982
60	15500	1	34444	56833	0.03	0.18	0.38	1148148	191358	90643
90	15500	0.5	25833	42625	0.03	0.18	0.38	861111	143519	67982
90	15500	0.75	38750	63938	0.03	0.18	0.38	1291667	215278	101974
90	15500	1	51667	85250	0.03	0.18	0.38	1722222	287037	135965
120	15500	0.5	34444	56833	0.03	0.18	0.38	1148148	191358	90643
120	15500	0.75	51667	85250	0.03	0.18	0.38	1722222	287037	135965
120	15500	1	68889	113667	0.03	0.18	0.38	2296296	382716	181287

Table 3 - Screened armor layer volume estimates

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Attachment 6 Bypass Channel

Appendix F

DRAFT USGS Sediment Sampling Report

19March2012

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Prepared in cooperation with the U.S. Army Corps of Engineers, Omaha District

Sediment Characteristics for the Yellowstone River at a Proposed Bypass Chute near Glendive Montana, 2011

By Brent R. Hanson and Joel M. Galloway

Open-file Report 2012-XXXX

U.S. Department of the Interior
U.S. Geological Survey

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KEN SALAZAR, Secretary

U.S. Geological Survey
Marcia K. McNutt, Director

U.S. Geological Survey, Reston, Virginia: 2012
Revised and reprinted: 2012

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Conversion Factors

Multiply	By	To obtain
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
square mile (mi^2)	259.0	hectare (ha)
pint (pt)	0.4732	liter (L)
quart (qt)	0.9464	liter (L)
gallon (gal)	3.785	liter (L)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
ton per day (ton/d)	0.9072	metric ton per day

Temperature in degrees Celsius (°C) may be converted to degrees Fahrenheit (°F) as follows:

°F=(1.8×°C)+32

Temperature in degrees Fahrenheit (°F) may be converted to degrees Celsius (°C) as follows:

°C=(°F-32)/1.8

Vertical coordinate information is referenced to the insert datum name (and abbreviation) here for instance, "North American Vertical Datum of 1988 (NAVD 88)."

Horizontal coordinate information is referenced to the insert datum name (and abbreviation) here for instance, "North American Datum of 1983 (NAD 83)."

Altitude, as used in this report, refers to distance above the vertical datum.

Specific conductance is given in microsiemens per centimeter at 25 degrees Celsius (µS/cm at 25 °C).

Concentrations of chemical constituents in water are given either in milligrams per liter (mg/L) or micrograms per liter (µg/L).

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Sediment Characteristics for the Yellowstone River at a Proposed Bypass Channel near Glendive Montana, 2011

By Brent R. Hanson and Joel M. Galloway

Abstract

In 2011, sediment data was collected by the U.S. Geological Survey in cooperation with the U.S. Army Corps of Engineers on the Yellowstone River at the location of a proposed bypass chute. The sediment data was collected to provide an understanding of the sediment dynamics of the given reach of the Yellowstone River. Samples of suspended sediment (point and integrated) and bedload were collected at three sites during July 19-21, August 9-11, and August 23-24, 2011. Suspended sediment concentrations in the integrated samples collected at the three sites generally decreased with decreasing streamflow. Point samples collected at the three sites showed the variability of suspended sediment concentrations in the cross-section at each site. In general, the highest suspended concentrations were found near the channel bed and towards the center of the channel with lower suspended sediment concentrations near the channel banks and water surface. The particle sizes of suspended sediment from point samples showed similar distributions at each site for the three sampling periods. The majority of sediment in the bedload had a particle size smaller than 16 mm. Suspended sediment was the primary component of the total sediment load for all three sampling locations on the Yellowstone River during the late summer of 2011. Suspended sediment contributed at least 98 percent of the total sediment load at each of the three sites.

Introduction

A diversion dam located on the Yellowstone River near Glendive, Montana currently impedes the upstream migration of the endangered *Scaphirhynchus albus,* commonly known as the pallid sturgeon (Bureau of Reclamation, 2011) (fig.1). A proposed bypass chute would result in the excavation of a natural side channel and provide a bypass around the diversion dam. The bypass channel would improve passage for the pallid sturgeon and other fish in the Yellowstone River to proceed with their upstream migrations.

In 2011, sediment data was collected by the U.S. Geological Survey (USGS) in cooperation with the U.S. Army Corps of Engineers (USACOE) on the Yellowstone River in the vicinity of a proposed bypass chute near Glendive, Montana. The sediment data were collected to provide an understanding of the sediment dynamics of the Yellowstone River reach above and below the bypass chute. The USACOE will use the sediment data with hydraulic modeling to evaluate the potential degradation and aggradation effects the bypass chute may have within the Yellowstone River reach (Curtis J. Miller, U.S. Army Corps of Engineers, written commun., 2011). The models will be used to select a favorable channel configuration for the bypass that will minimize the negative impacts on sediment transport that the bypass chute may generate.

Figure 1. Location of study area

Methods of Data Collection

The following sections describe methods used by the USGS for the collection and analysis of sediment samples and measurement of streamflow. Data were collected by the USGS at three sites on the Yellowstone River in the vicinity of the proposed bypass chute near Glendive, Montana.

Samples of suspended sediment (point and integrated) and bedload were collected at the three sites; one site upstream of the bypass chute entrance (above bypass chute), one site at the entrance of the bypass chute (adjacent to bypass chute) and one site downstream of the bypass chute entrance (below bypass chute) (fig. 1). Streamflow was measured at each site prior to collection sample collection. Samples were collected during July 19-21, August 9-11, and August 23-24, 2011 (fig. 2).

Figure 2. Daily mean streamflow for the Yellowstone River near Glendive, Montana (U.S. Geological Survey station number 06327500) and sample collection dates near a proposed bypass chute, 2011.

Integrated suspended-sediment concentration (SSC) samples were collected three times in 2011 to estimate the amount of suspended material being transported past the three sites during different streamflow conditions. To collect samples that represent the vertical and horizontal variability of suspended sediment in the stream channels, samples were collected using depth-integrated samplers (D-96 and DH-2) (Davis, 2005) and the equal-discharge increment (EDI) method (Edwards and Glysson, 1999). The EDI method involved the collection of vertically integrated, isokinetic (velocity entering the sampler nozzle was the same as the velocity of the stream) samples at 5 intervals representing equal percentages of the total streamflow across the stream cross section (20 percent of the total streamflow in each section).

Suspended-sediment samples were also collected at discrete vertical points (point samples) at each site to estimate the vertical distribution of particle sizes and concentrations at the three sites. Point samples were collected using a US P-61-A1 suspended-sediment sampler that is designed to open and close at varying depths in the water column (Davis, 2005). Samples were collected at six different depths including near the water surface, one foot above the channel bottom, and at four evenly spaced points in the vertical between those points at each of the 5 EDI sample collection locations for each site.

Bedload samples were collected to estimate the sediment transport along or near the streambed at the three sites. Bedload samples were collected using a cable-suspended Helley-Smith Model 8035 sampler (Davis, 2005). For each sampling site, bedload samples were collected at 20 equal-width sections across the channel according to methods described by Edwards and Glysson (1999). The bedload samples were then composited in a 1-L plastic container.

All samples of suspended sediment (integrated and point) and bedload were analyzed for concentration and particle-size distribution at the USGS Iowa Water Science Center Sediment Laboratory in Iowa City, Iowa, using methods described by Guy (1969). Some suspended-sediment samples were not analyzed for the complete particle-size distribution because of insufficient sediment mass present in the sample. Results from the analysis were stored in the USGS National Water Information System (NWIS) database (*http://nwis.waterdata.usgs.gov/nd/nwis/qw*).

Streamflow data were collected for use with the sediment concentration data to calculate sediment loads. Streamflow was measured using an acoustic Doppler current profiler (ADCP) with the methods and procedures described in Mueller and Wagner (2009). Streamflow was measured for each sampling site prior to the collection of sediment samples.

Suspended-sediment loads were estimated for the three sites using the measured streamflow data and SSC data collected during the three sampling events. Loads were estimated using equation 1 (Porterfield, 1972):

$$Q_s = Q_w \times C_s \times K \qquad (1)$$

where

Q_s is the suspended-sediment load (sediment discharge), in tons (English short tons) per day (tons/day);

Q_w is the instantaneous streamflow (water discharge), in cubic feet per second (ft^3/s);

C_s is the SSC, in milligrams per liter (mg/L); and

K is a coefficient (0.0027) to convert the units of measurement of water discharge and SSC into tons/day and assumes a specific gravity of 2.65 for sediment.

The bedload was calculated from the measured data using equation 2 (Edwards and Glysson, 1999):

$$Q_b = K \times (W_T/t_T) \times M_T \qquad (2)$$

where

Q_b is the bedload discharge, in tons/day;

K is a conversion factor (0.381 for a 3-inch nozzle).

W_T is the total width of the stream from which samples were collected, in feet, and is equal to the increment width times the total number of vertical samples;

t_T is the total time the sampler was on the streambed, in seconds, computed by multiplying the individual sample time by the total number of vertical samples; and

M_T is the total mass of sample collected from all verticals sampled in the cross section, in grams

Sediment Characteristics

The three locations on the Yellowstone River were sampled for suspended sediment and bedload during three different hydrologic flow conditions (fig. 2; table 1) Streamflow ranged from 49,900 (above bypass chute) to 46,200 cubic feet per second (cfs) (adjacent to bypass chute) for the July 19-21 samples, and from 22,800 (above bypass chute) to 20,100(cfs) (adjacent to bypass chute) for the Aug 9-11 samples. During the Aug 23-24 sampling, all three sites had the same streamflow of 13,400 cfs. Due to safety and timing constraints, the suspended samples were collected during the falling limb of the above average high flows during the summer of 2011.

Suspended Sediment Concentration

SSC in the integrated samples collected at three sites on the Yellowstone River in 2011 decreased with decreasing streamflow (table 1). The SSC was 428 milligrams per liter (mg/l) above the bypass chute during the highest streamflow (July 19, 2011), and 72 mg/l at the lowest streamflow (August 24, 2011). The SSC for samples collected at the locations adjacent to bypass chute and below bypass chute had similar results ranging from 438 to 83 mg/l and 452 to 75 mg/l, respectively.

Table 1. Measured streamflow, suspended-sediment concentrations, and fall diameters for three sites near the proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011.

Point samples collected at the three sites showed the variability of SSC in the cross-section at each site (figs. 3-5 and table 2). In general, the highest SSC were found near the channel bed and towards the center of the channel with lower SSCs near the channel banks and water surface (figs. 3-5 and table 2). The maximum SSC for the point samples was found above the bypass chute, one foot above the channel bed near the center of channel at 694 mg/l on July 19, 2011(table 2) at a measured streamflow of 49,900 cfs (table 1). The minimum SSC was 32 mg/l below the bypass near the right edge of water on Aug 23, 2011 with a measured streamflow of 13,400 cfs.

Due to equipment malfunction, the full point sample set was not collected for the above bypass chute on July 19, 2011. Due to the incomplete sample set, a concentration contour graph was not prepared for the July 19, 2011 point sample data.

Table 2. Suspended-sediment concentration, fall diameter, and sieve diameter for samples collected at discrete vertical depths at three sites near the proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011.

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Figure 3. Distribution of suspended-sediment concentrations from point samples collected on the Yellowstone River above a proposed bypass chute near Glendive, Montana, August, 2011.

Figure 4. Distribution of suspended-sediment concentrations from point samples collected on the Yellowstone River adjacent to a proposed bypass chute near Glendive, Montana, July and August, 2011.

Figure 5. Distribution of suspended-sediment concentrations from point samples collected on the Yellowstone River below a proposed bypass chute near Glendive, Montana, July and August, 2011.

Particle Size Distribution of Suspended Sediment and Bedload

The particle size distribution of suspended sediment from point samples collected at three sites on the Yellowstone River in 2011 showed similar distributions at each site for the three sampling periods (figs, 6-8 and table 2). Most of the suspended sediment was smaller than 0.062 mm in size. At the highest measured flows of 49,900 to 46,200 cfs (July 19-21, 2011; table 1), 50 to 95 percent of the suspended sediment in the Yellowstone River was finer than 0.062 mm across the channel with an average of 71 percent finer than 0.062 mm. At the lowest measured flows of 13,400 cfs (August 23-24,2011), the amount of suspended sediment finer than 0.062 mm increased to an average of 82 percent ranging of 58 to 99 percent finer than 0.062 mm across the channel. In general, the coarsest material for each sample was found to be near the center of the channel and near the channel bed (figs. 6-8). For all three sampling visits, the suspended-sediment size generally tended to decrease near the channel banks and the surface of the water column.

Due to equipment malfunction, the full point sample set was not collected for the above bypass chute on July 19, 2011. Due to the incomplete sample set, a particle size distribution contour graph was not prepared for the July 19, 2011 point sample data.

Figure 6. Distribution of suspended-sediment particle sizes from point samples collected on the Yellowstone River above a proposed bypass chute near Glendive, Montana, August, 2011.

Figure 7. Distribution of suspended-sediment particle sizes from point samples collected on the Yellowstone River adjacent to a proposed bypass chute near Glendive, Montana, July and August, 2011.

Figure 8. Distribution of suspended-sediment particle sizes from point samples collected on the Yellowstone River below a proposed bypass chute near Glendive, Montana, July and August, 2011.

Overall, the bedload sediment had a particle size less than 16 mm (table 3). Most of the bedload particles sizes measured were between 0.25 to 0.50 mm. For the site above bypass chute, 41 to 85 percent of the bedload material was between 0.25 to 0.50 mm in the three samples. For the site adjacent to the bypass chute, 54 to 60 percent of the bedload material was between 0.25 to 0.50 mm in the three samples. The bedload at the site below the bypass chute had 37 to 58 percent of the material between 0.25 to 0.50 mm in the three samples.

Table 3. Sieve diameters and mass of bedload samples for three sites near a proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011.

Sediment loads

Suspended sediment was the primary component of the total sediment load for all three sampling locations on the Yellowstone River during July and August, 2011 (table 4). Suspended sediment contributed at least 98 percent of the total sediment load at each of the three sites.

Table 4. Sediment loads for three sites near a proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011.

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The sampling location above the bypass chute had the greatest suspended sediment load among the three sites at the highest streamflow and least suspended-sediment load among the three sites at the lowest measured streamflow. At the highest measured streamflow of 49,900 cfs (July 19, 2011; table 1), the site above the bypass chute had a suspended sediment load of 57,700 tons/day and at the lowest measured streamflow of 13,400 cfs (August 24, 2011) the same sampling location had a suspended load of 2,600 tons/day (table 4).

The amount of bedload measured at the three sites on the Yellowstone River in 2011 generally decreased as streamflow decreased. At the highest measured streamflow (July 19-21, 2011; table 1), the measured bedload amount ranged from 254 tons/day above bypass chute to 301 ton/day below bypass chute (table 4). At the lowest measured streamflow (August 23-24, 2011), the measured bedload ranged from 5 tons/day above bypass chute to 55 ton/day adjacent to bypass chute.

The total sediment load within the channel is comprised of the suspended sediment load and the bedload. The highest total sediment load was found during the highest measured streamflow of 49,900 cfs above the bypass on July 19, 2011 with a total load of 57,954 tons/day (table 4). The lowest total sediment load was also found above the bypass with 2,605 tons/day at a streamflow of 13,400 cfs on August 24, 2011.

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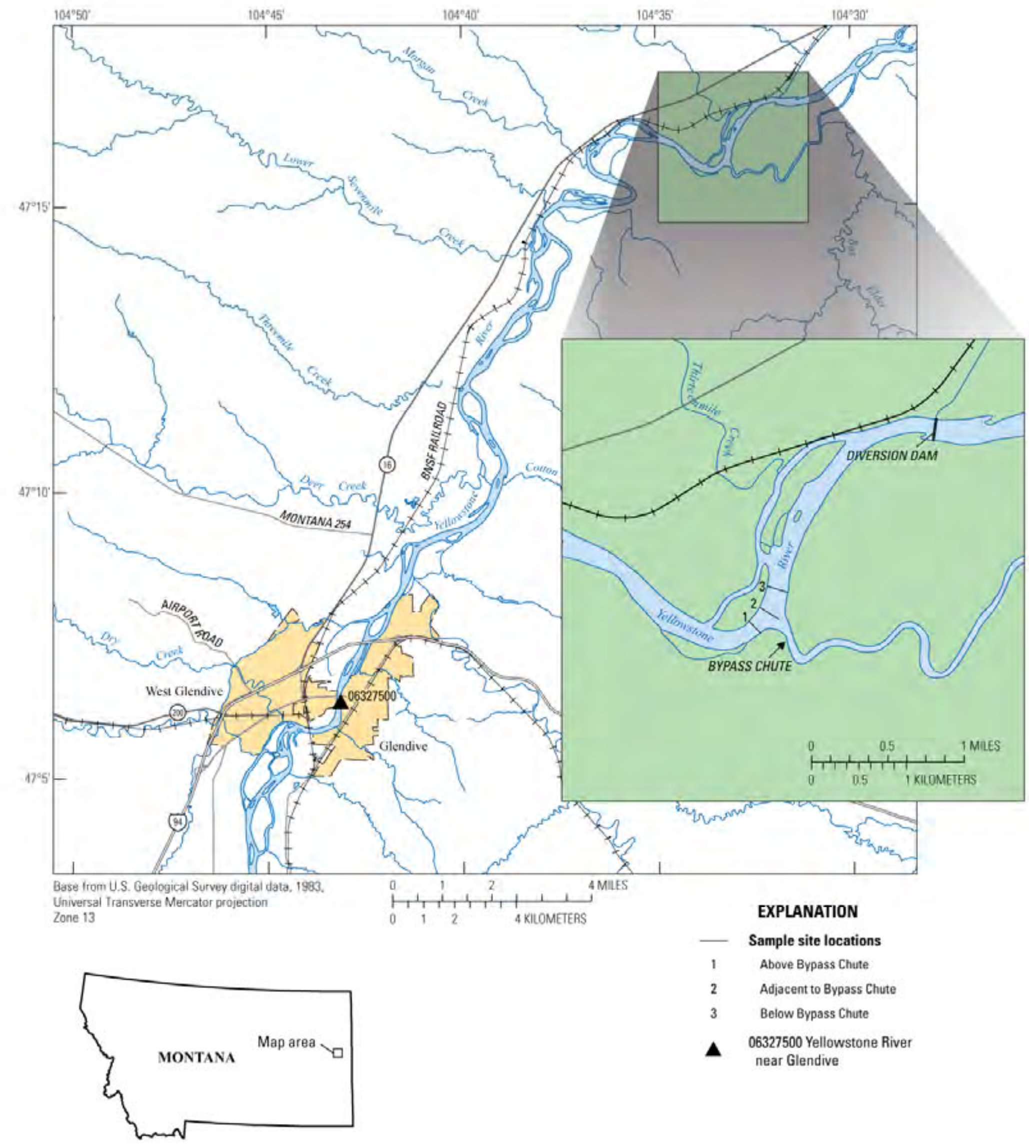

Figure 1. Location of study area

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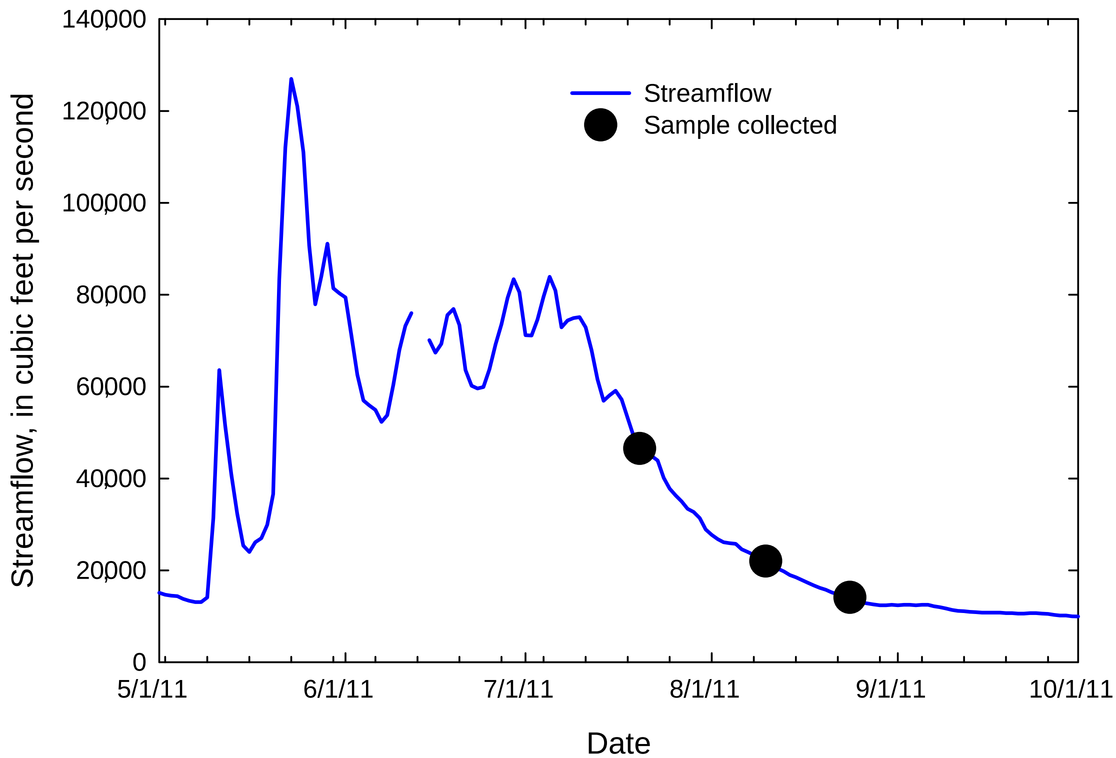

Figure 2. Daily mean streamflow for the Yellowstone River near Glendive, Montana (U.S. Geological Survey station number 06327500) and sample collection dates near a proposed bypass chute, 2011.

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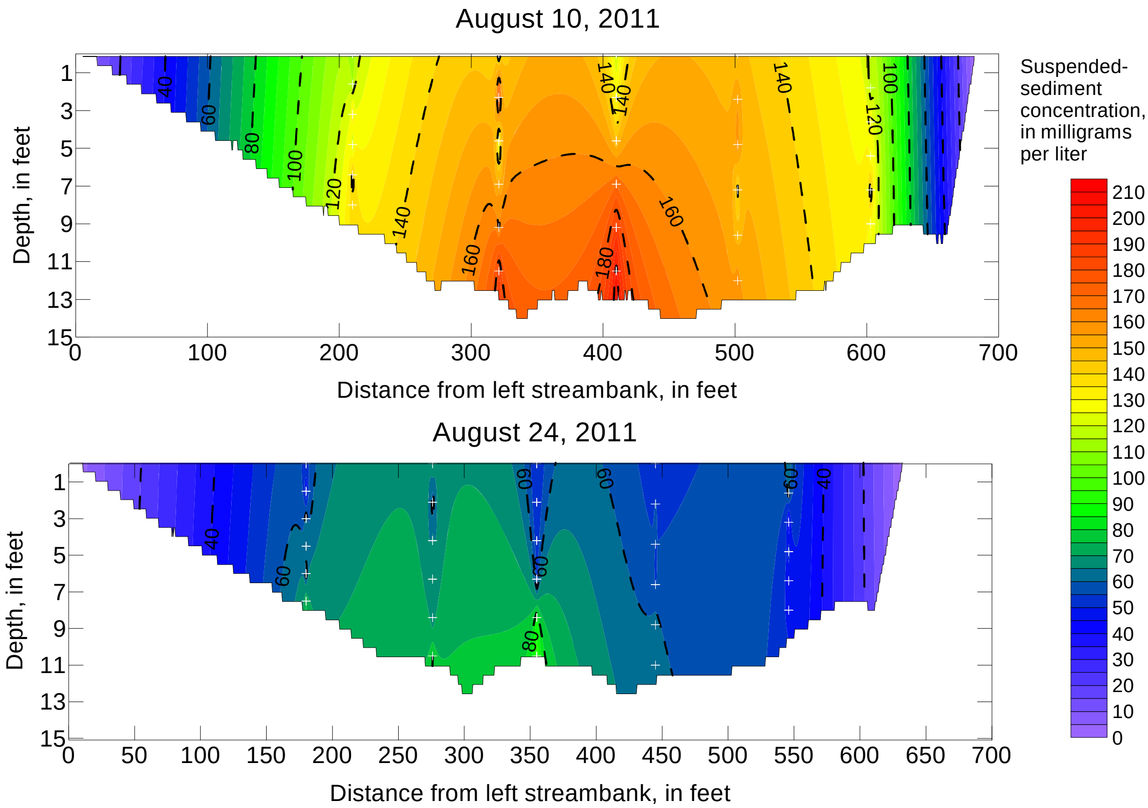

Figure 3. Distribution of suspended-sediment concentrations from point samples collected on the Yellowstone River above a proposed bypass chute near Glendive, Montana, August, 2011.

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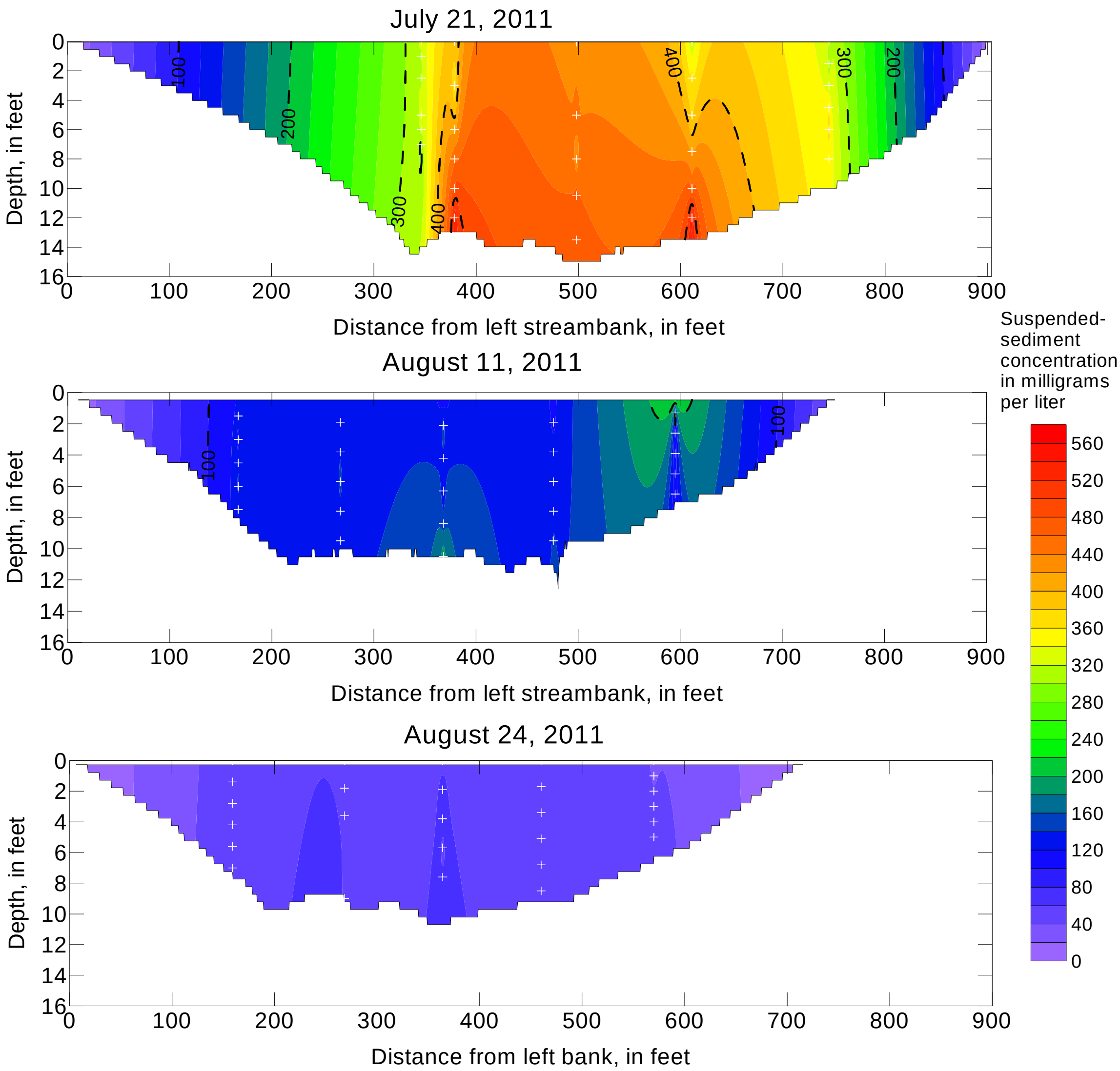

Figure 4. Distribution of suspended-sediment concentrations from point samples collected on the Yellowstone River adjacent to a proposed bypass chute near Glendive, Montana, July and August, 2011.

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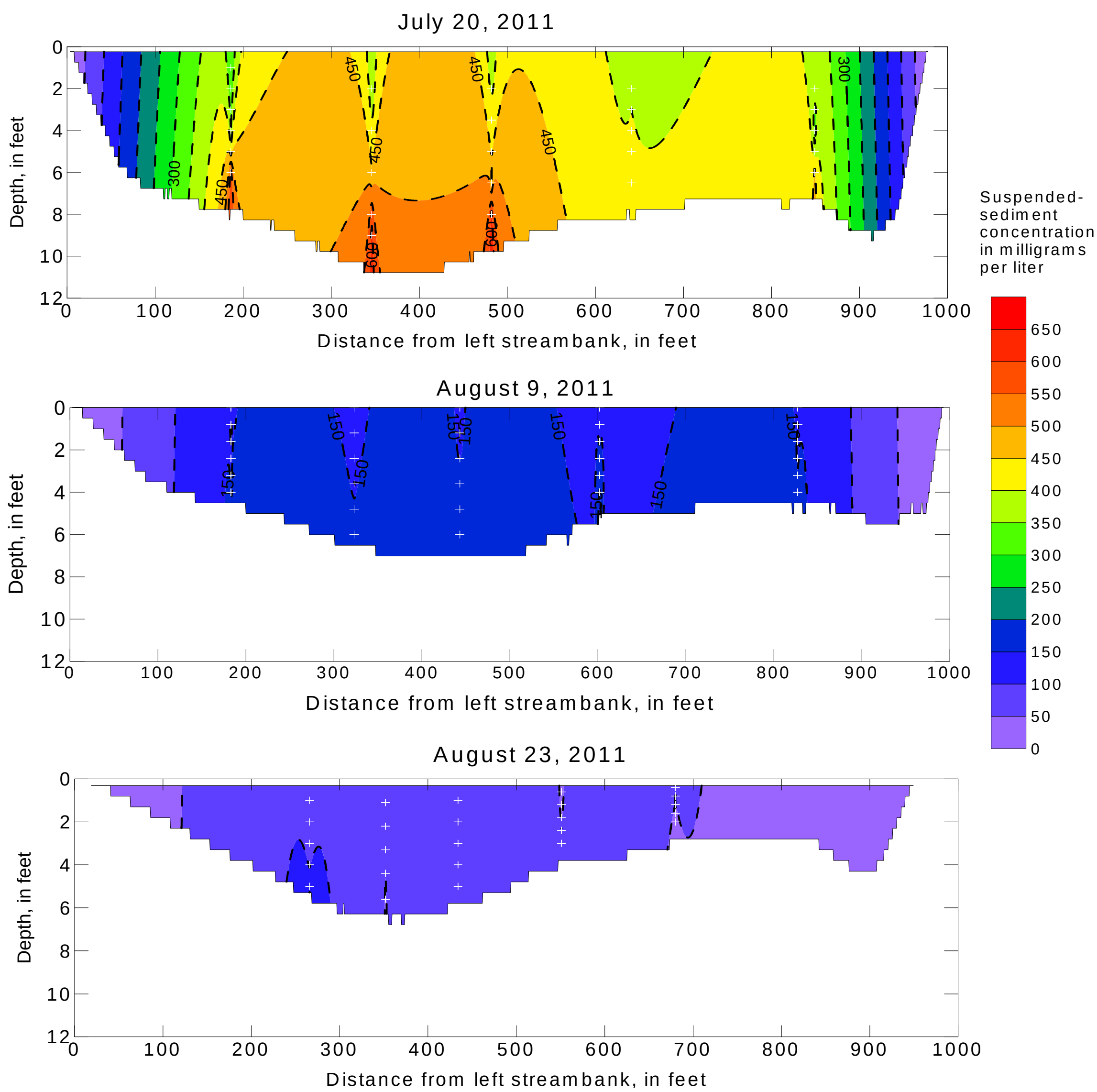

Figure 5. Distribution of suspended-sediment concentrations from point samples collected on the Yellowstone River below a proposed bypass chute near Glendive, Montana, July and August, 2011.

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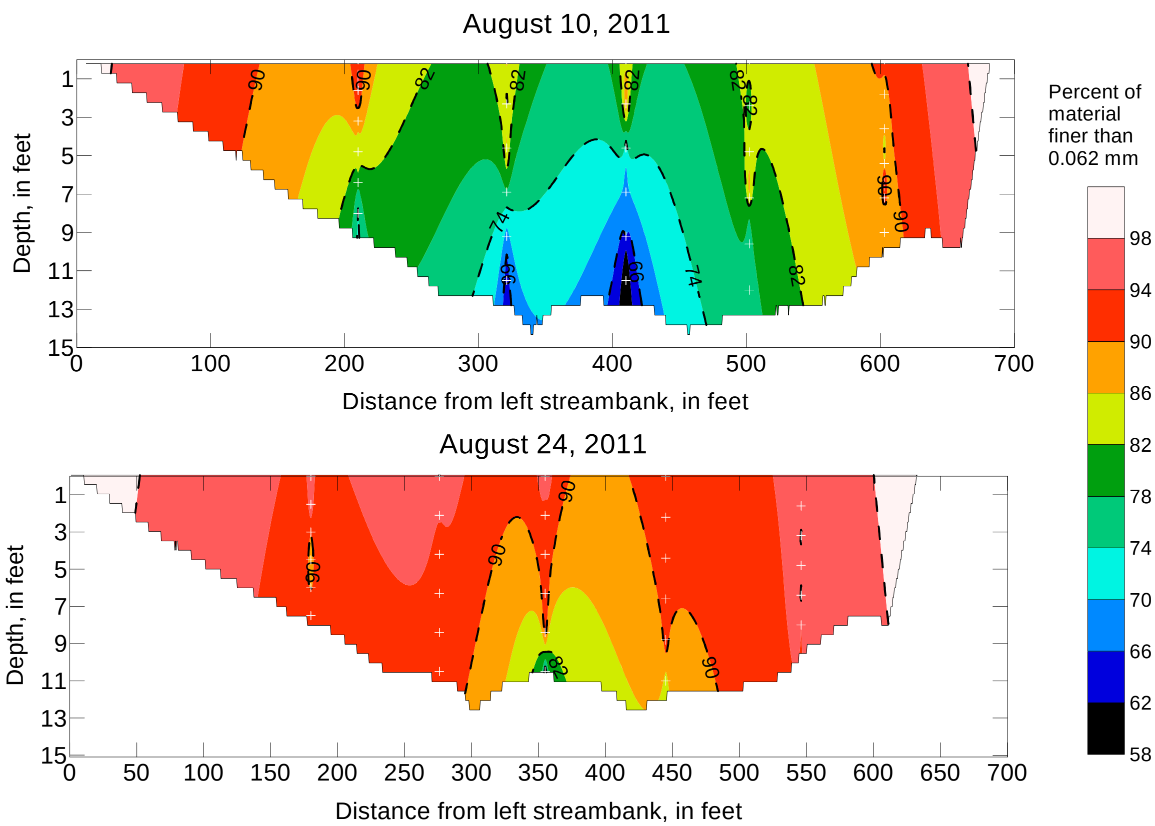

Figure 6. Distribution of suspended-sediment particle sizes from point samples collected on the Yellowstone River above a proposed bypass chute near Glendive, Montana, August, 2011.

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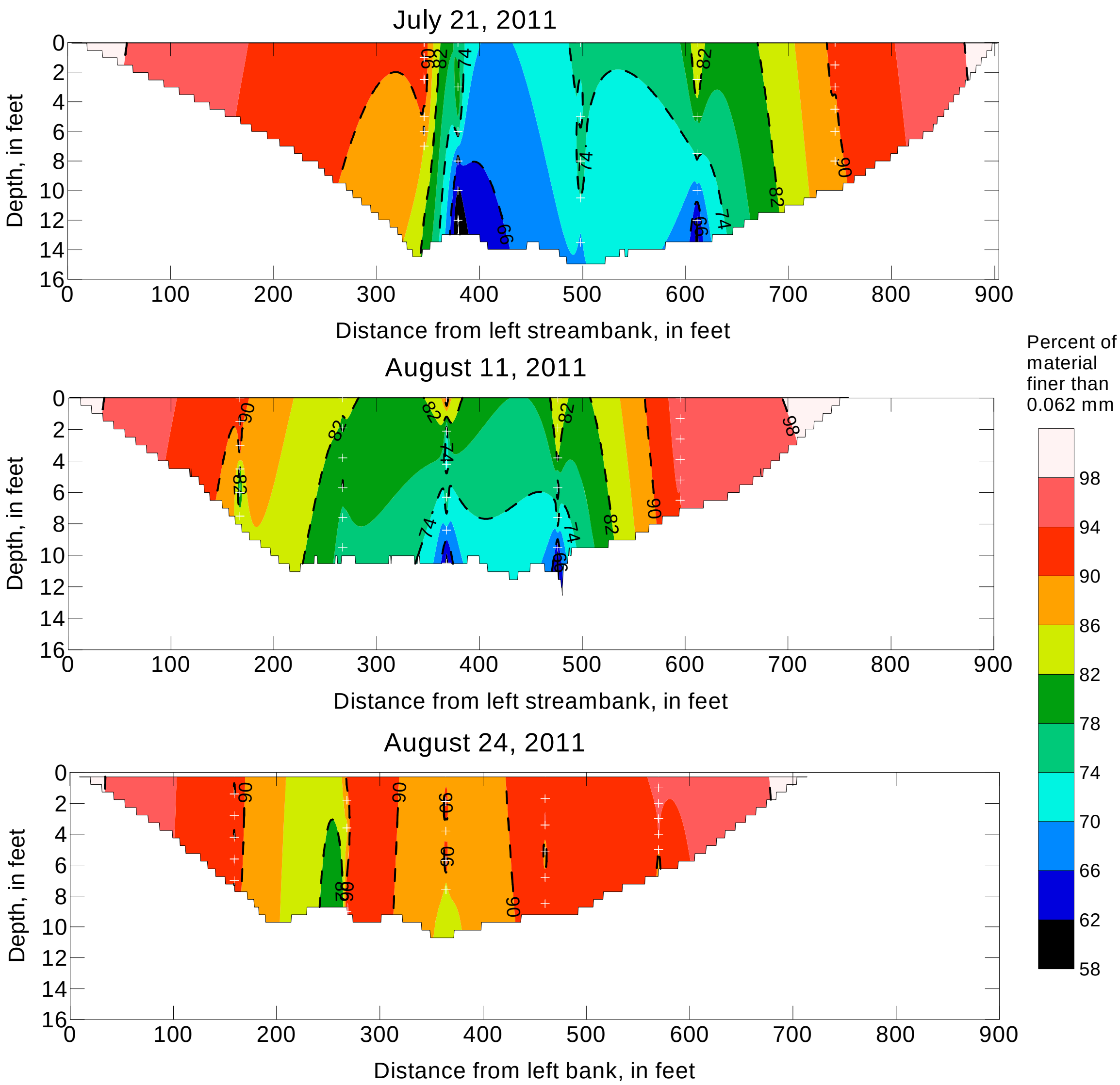

Figure 7. Distribution of suspended-sediment particle sizes from point samples collected on the Yellowstone River adjacent to a proposed bypass chute near Glendive, Montana, July and August, 2011.

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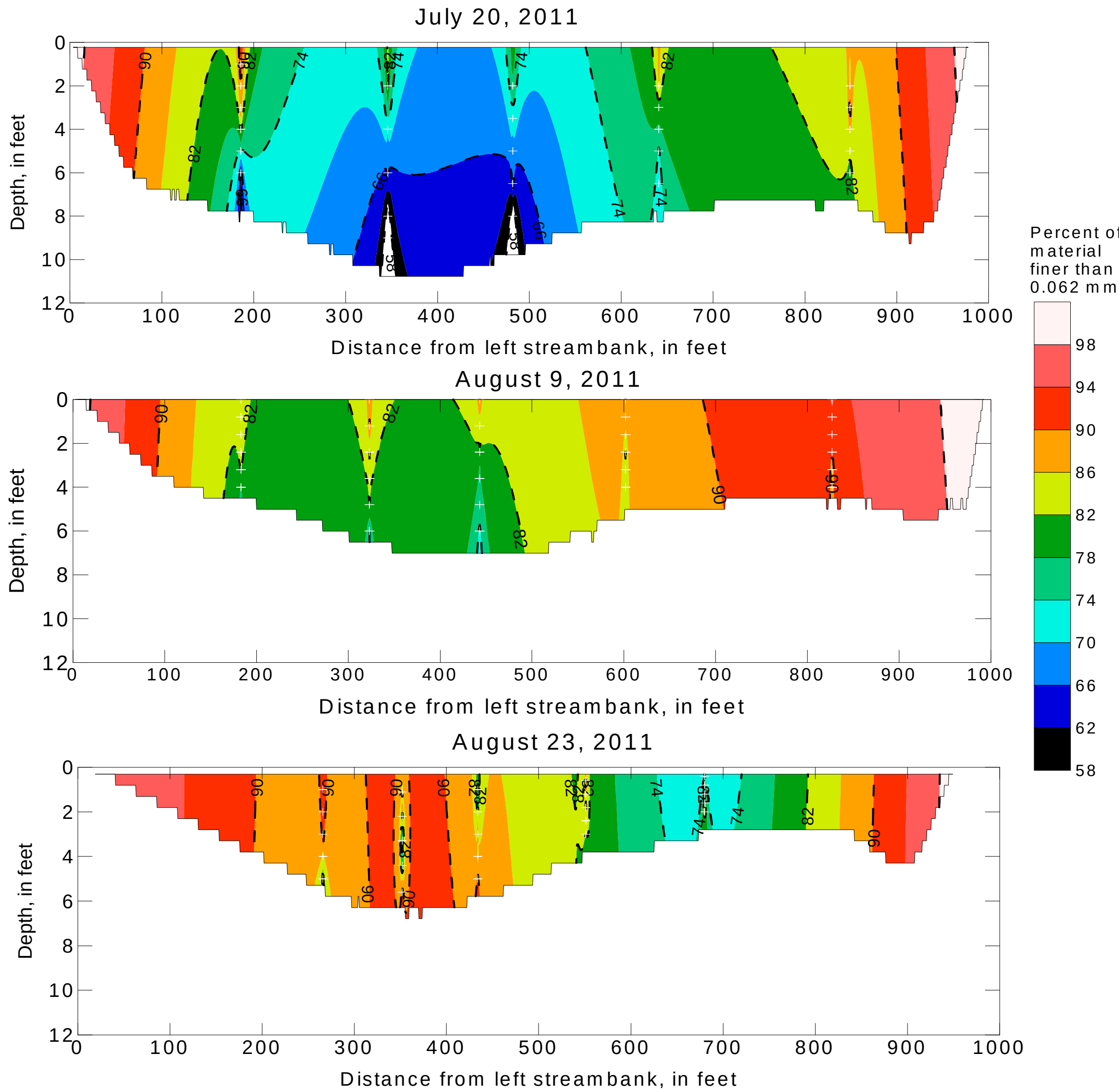

Figure 8. Distribution of suspended-sediment particle sizes from point samples collected on the Yellowstone River below a proposed bypass chute near Glendive, Montana, July and August, 2011.

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Table 1. Measured streamflow, suspended-sediment concentrations, and fall diameters for three sites near the proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011.

[ft, feet; ft^3/s, cubic feet per second; µS/cm, microsiemens per centimeter; deg. C, degrees Celsius; mg/L, milligram per liter; mm, millimeter]

Dates	Water Surface elevation (ft above NAVD88)	Measured streamflow (ft^3/s)	Specific conductance (µS/cm at 25 deg. C)	Water Temperature (deg. C)	Suspended-sediment concentration (mg/L)	Suspended-sediment fall diameter (values in percent finer than size)			
						0.062 mm	0.125 mm	0.250 mm	0.500 mm
Above bypass chute									
7/19/2011	1,995.12	49,900	337	23.9	428	73	81	95	100
8/10/2011	1,992.24	22,800	452	21.6	134	77	88	98	99
8/24/2011	1,990.20	13,400	505	22.0	72	82	93	100	100
Adjacent to bypass chute									
7/21/2011	1,994.70	46,200	345	22.0	438	71	79	95	100
8/11/2011	1,992.06	20,100	453	22.0	117	79	92	100	100
8/24/2011	1,990.92	13,400	505	22.5	83	84	98	100	100
Below bypass chute									
7/20/2011	1,994.84	46,900	354	23.4	452	69	80	96	100
8/9/2011	1,992.34	22,700	456	21.6	151	78	91	99	100
8/23/2011	1,990.99	13,400	500	23.8	75	81	93	100	100

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Table 2. Suspended-sediment concentration, fall diameter, and sieve diameter for samples collected at discrete vertical depths at three sites near the proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011

[DD, decimal degrees; ft, feet mm, millimeter; mg/L, milligram per liter]

Dates	Latitude (DD)	Longitude (DD)	Location in cross section, distance from left bank looking downstream (ft)	Sampling depth (ft)	Suspended-sediment concen-tration (mg/L)	Suspended-sediment fall diameter (values in percent finer than size)				Percent finer than suspended sediment sieve diameter of 0.062 mm
						0.062 mm	0.125 mm	0.250 mm	0.500 mm	
Above bypass chute										
7/19/2011	47.2611800	104.554758	344	0.0	352	87	94	100	100	--
	47.2611800	104.554758	344	3.0	454	72	79	100	100	--
	47.2611800	104.554758	344	6.0	467	68	75	97	100	--
	47.2611800	104.554758	344	9.0	511	59	72	97	100	--
	47.2611800	104.554758	344	12.0	577	58	73	96	100	--
	47.2611800	104.554758	344	15.0	694	50	61	95	100	--
	47.2610143	104.554569	426	0.0	450	79	89	100	100	--
	47.2610143	104.554569	426	3.0	555	62	76	97	100	--
8/10/2011	47.2613008	104.555254	210	0.0	112	93	94	100	100	--
	47.2613008	104.555254	210	1.6	113	96	99	100	100	--
	47.2613008	104.555254	210	3.2	136	87	94	100	100	--
	47.2613008	104.555254	210	4.8	118	85	94	100	100	--
	47.2613008	104.555254	210	6.4	145	82	95	100	100	--
	47.2613008	104.555254	210	8.0	136	74	91	100	100	--
	47.261108	104.55491	321	0.0	131	87	95	100	100	--
	47.261108	104.55491	321	2.3	184	81	91	100	100	--
	47.261108	104.55491	321	4.6	123	87	93	100	100	--
	47.261108	104.55491	321	6.9	155	80	94	100	100	--
	47.261108	104.55491	321	9.2	159	71	84	100	100	--
	47.261108	104.55491	321	11.5	187	65	81	99	100	--
	47.2608996	104.554644	410	0.0	122	84	93	100	100	--
	47.2608996	104.554644	410	2.3	129	91	96	100	100	--
	47.2608996	104.554644	410	4.6	146	74	85	100	100	--
	47.2608996	104.554644	410	6.9	170	68	83	100	100	--
	47.2608996	104.554644	410	9.2	187	70	82	94	100	--
	47.2608996	104.554644	410	11.5	202	59	69	100	100	--
	47.260728	104.55441	502	0.0	145	84	93	100	100	--
	47.260728	104.55441	502	2.4	154	80	86	100	100	--
	47.260728	104.55441	502	4.8	159	83	90	100	100	--
	47.260728	104.55441	502	7.2	138	88	93	100	100	--
	47.260728	104.55441	502	9.6	148	75	86	100	100	--
	47.260728	104.55441	502	12.0	158	76	84	95	100	--
	47.260523	104.554162	603	0.0	119	87	91	100	100	--
	47.260523	104.554162	603	1.8	114	90	92	100	100	--
	47.260523	104.554162	603	3.6	132	81	92	100	100	--
	47.260523	104.554162	603	5.4	123	97	99	100	100	--
	47.260523	104.554162	603	7.2	148	87	97	100	100	--
	47.260523	104.554162	603	9.0	125	93	97	100	100	--
8/24/2011	47.2613346	104.555041	180	0.0	56	94	98	100	100	--
	47.2613346	104.555041	180	1.5	43	IM	IM	IM	IM	98
	47.2613346	104.555041	180	3.0	65	90	97	100	100	--
	47.2613346	104.555041	180	4.5	63	87	98	100	100	--
	47.2613346	104.555041	180	6.0	57	90	99	100	100	--
	47.2613346	104.555041	180	7.5	76	92	97	100	100	--
	47.2611368	104.554795	276	0.0	70	98	98	100	100	--
	47.2611368	104.554795	276	2.1	56	95	95	100	100	--
	47.2611368	104.554795	276	4.2	67	90	92	100	100	--
	47.2611368	104.554795	276	6.3	65	93	95	100	100	--
	47.2611368	104.554795	276	8.4	66	90	93	100	100	--
	47.2611368	104.554795	276	10.5	80	92	95	100	100	--
	47.260968	104.554606	355	0.0	52	97	98	100	100	--
	47.260968	104.554606	355	2.1	50	92	95	100	100	--
	47.260968	104.554606	355	4.2	55	91	95	100	100	--
	47.260968	104.554606	355	6.3	51	93	94	100	100	--
	47.260968	104.554606	355	8.4	86	93	95	100	100	--
	47.260968	104.554606	355	10.5	93	71	77	100	100	--
	47.2607695	104.554365	445	0.0	49	IM	IM	IM	IM	94
	47.2607695	104.554365	445	2.2	56	93	94	100	100	--
	47.2607695	104.554365	445	4.4	55	92	99	100	100	--
	47.2607695	104.554365	445	6.6	54	91	92	100	100	--
	47.2607695	104.554365	445	8.8	63	93	96	100	100	--
	47.2607695	104.554365	445	11.0	64	84	91	100	100	--
	47.2605396	104.554175	546	1.6	64	IM	IM	IM	IM	94
	47.2605396	104.554175	546	3.2	39	IM	IM	IM	IM	99
	47.2605396	104.554175	546	4.8	54	96	96	100	100	--

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Table 2. Suspended-sediment concentration, fall diameter, and sieve diameter for samples collected at discrete vertical depths at three sites near the proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011

[DD, decimal degrees; ft, feet, mm, millimeter; mg/L, milligram per liter]

Dates	Latitude (DD)	Longitude (DD)	Location in cross section, distance from left bank looking downstream (ft)	Sampling depth (ft)	Suspended-sediment concen-tration (mg/L)	Suspended-sediment fall diameter (values in percent finer than size)				Percent finer than suspended sediment sieve diameter of 0.062 mm
						0.062 mm	0.125 mm	0.250 mm	0.500 mm	
	47.2605396	104.554175	546	6.4	44	IM	IM	IM	IM	99
	47.2605396	104.554175	546	8.0	50	94	94	100	100	--
Adjacent to bypass chute										
7/21/2011	47.2627976	104.553531	346	0.0	336	91	99	99	100	--
	47.2627976	104.553531	346	1.0	305	92	96	100	100	--
	47.2627976	104.553531	346	2.5	311	95	97	100	100	--
	47.2627976	104.553531	346	5.0	348	88	94	97	100	--
	47.2627976	104.553531	346	6.0	325	91	94	98	100	--
	47.2627976	104.553531	346	7.0	292	85	91	97	100	--
	47.2626110	104.553086	379	0.0	397	75	83	96	100	--
	47.2626110	104.553086	379	3.0	362	82	89	96	100	--
	47.2626110	104.553086	379	6.0	414	76	81	94	100	--
	47.2626110	104.553086	379	8.0	459	64	72	94	100	--
	47.2626110	104.553086	379	10.0	479	62	71	91	100	--
	47.2626110	104.553086	379	12.0	537	57	64	83	100	--
	47.2624346	104.552677	498	0.0	417	75	83	95	100	--
	47.2624346	104.552677	498	5.0	455	73	79	97	100	--
	47.2624346	104.552677	498	8.0	422	77	85	96	100	--
	47.2624346	104.552677	498	10.5	472	74	78	97	100	--
	47.2624346	104.552677	498	13.5	478	69	75	96	100	--
	47.2622623	104.552286	611	0.0	320	85	93	100	100	--
	47.2622623	104.552286	611	2.5	370	86	92	97	100	--
	47.2622623	104.552286	611	5.0	373	78	84	97	100	--
	47.2622623	104.552286	611	7.5	422	75	81	97	100	--
	47.2622623	104.552286	611	10.0	451	69	81	98	100	--
	47.2622623	104.552286	611	12.0	543	61	72	97	100	--
	47.2620336	104.55186	745	0.0	321	91	96	100	100	--
	47.2620336	104.55186	745	1.5	307	93	99	100	100	--
	47.2620336	104.55186	745	3.0	331	91	95	97	100	--
	47.2620336	104.55186	745	4.5	357	88	93	100	100	--
	47.2620336	104.55186	745	6.0	369	90	97	100	100	--
	47.2620336	104.55186	745	8.0	345	89	96	98	100	--
8/11/2011	47.2627555	104.553385	167	0.0	115	94	98	100	100	--
	47.2627555	104.553385	167	1.5	136	89	96	100	100	--
	47.2627555	104.553385	167	3.0	108	95	98	100	100	--
	47.2627555	104.553385	167	4.5	134	81	93	100	100	--
	47.2627555	104.553385	167	6.0	147	75	90	97	100	--
	47.2627555	104.553385	167	7.5	123	86	96	98	100	--
	47.2627316	104.553041	267	0.0	123	85	95	100	100	--
	47.2627316	104.553041	267	1.9	128	80	92	97	100	--
	47.2627316	104.553041	267	3.8	139	79	93	99	100	--
	47.2627316	104.553041	267	5.7	143	83	94	100	100	--
	47.2627316	104.553041	267	7.6	137	75	86	100	100	--
	47.2627316	104.553041	267	9.5	137	76	92	100	100	--
	47.2624758	104.552682	368	0.0	92	96	98	100	100	--
	47.2624758	104.552682	368	2.1	154	71	82	95	100	--
	47.2624758	104.552682	368	4.2	134	72	87	100	100	--
	47.2624758	104.552682	368	6.3	128	79	90	100	100	--
	47.2624758	104.552682	368	8.4	147	69	83	100	100	--
	47.2624758	104.552682	368	10.5	198	59	70	93	100	--
	47.2623151	104.552333	476	0.0	115	86	89	100	100	--
	47.2623151	104.552333	476	1.9	114	86	92	100	100	--
	47.2623151	104.552333	476	3.8	129	83	89	100	100	--
	47.2623151	104.552333	476	5.7	134	71	85	97	100	--
	47.2623151	104.552333	476	7.6	132	79	88	100	100	--
	47.2623151	104.552333	476	9.5	143	61	77	100	100	--
	47.2621248	104.551915	595	0.0	323	96	98	100	100	--
	47.2621248	104.551915	595	1.3	85	95	98	100	100	--
	47.2621248	104.551915	595	2.6	105	96	100	100	100	--
	47.2621248	104.551915	595	3.9	117	96	98	100	100	--
	47.2621248	104.551915	595	5.2	116	94	100	100	100	--
	47.2621248	104.551915	595	6.5	113	94	97	100	100	--
8/24/2011	47.2626747	104.553373	159	0.0	46	IM	IM	IM	IM	91
	47.2626747	104.553373	159	1.4	62	89	97	100	100	--
	47.2626747	104.553373	159	2.8	45	IM	IM	IM	IM	94
	47.2626747	104.553373	159	4.2	56	IM	IM	IM	IM	88

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Table 2. Suspended-sediment concentration, fall diameter, and sieve diameter for samples collected at discrete vertical depths at three sites near the proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011

[DD, decimal degrees; ft, feet mm, millimeter; mg/L, milligram per liter]

Dates	Latitude (DD)	Longitude (DD)	Location in cross section, distance from left bank looking downstream (ft)	Sampling depth (ft)	Suspended-sediment concen-tration (mg/L)	Suspended-sediment fall diameter (values in percent finer than size)				Percent finer than suspended sediment sieve diameter of 0.062 mm
						0.062 mm	0.125 mm	0.250 mm	0.500 mm	
	47.2626747	104.553373	159	5.6	55	96	99	100	100	--
	47.2626747	104.553373	159	7.0	55	90	96	100	100	--
	47.2625065	104.553001	268	0.0	43	IM	IM	IM	IM	91
	47.2625065	104.553001	268	1.8	53	IM	IM	IM	IM	89
	47.2625065	104.553001	268	3.6	55	IM	IM	IM	IM	85
	47.2625065	104.553001	268	9.0	51	IM	IM	IM	IM	95
	47.2623663	104.552669	364	0.0	51	IM	IM	IM	IM	86
	47.2623663	104.552669	364	1.9	69	94	95	100	100	--
	47.2623663	104.552669	364	3.8	73	87	91	100	100	--
	47.2623663	104.552669	364	5.7	52	IM	IM	IM	IM	95
	47.2623663	104.552669	364	7.6	65	83	89	100	100	--
	47.2623663	104.552669	364	9.5	71	79	84	100	100	--
	47.2622328	104.552327	460	0.0	51	IM	IM	IM	IM	94
	47.2622328	104.552327	460	1.7	49	IM	IM	IM	IM	93
	47.2622328	104.552327	460	3.4	40	IM	IM	IM	IM	95
	47.2622328	104.552327	460	5.1	52	86	89	100	100	--
	47.2622328	104.552327	460	6.8	53	92	94	100	100	--
	47.2622328	104.552327	460	8.5	54	92	92	100	100	--
	47.2620273	104.551995	570	0.0	35	IM	IM	IM	IM	97
	47.2620273	104.551995	570	1.0	35	IM	IM	IM	IM	92
	47.2620273	104.551995	570	2.0	43	IM	IM	IM	IM	97
	47.2620273	104.551995	570	3.0	48	IM	IM	IM	IM	93
	47.2620273	104.551995	570	4.0	43	IM	IM	IM	IM	96
	47.2620273	104.551995	570	5.0	55	IM	IM	IM	IM	89
Below bypass chute										
7/20/2011	47.2651015	104.552643	186	0.0	304	92	95	97	100	--
	47.2651015	104.552643	186	1.0	315	92	96	100	100	--
	47.2651015	104.552643	186	2.0	323	89	95	98	100	--
	47.2651015	104.552643	186	3.0	335	90	92	97	100	--
	47.2651015	104.552643	186	4.0	350	82	89	98	100	--
	47.2651015	104.552643	186	5.0	409	76	85	93	100	--
	47.2651015	104.552643	186	6.0	597	60	70	92	100	--
	47.2649765	104.552019	346	0.0	369	82	87	96	100	--
	47.2649765	104.552019	346	2.0	355	83	89	99	100	--
	47.2649765	104.552019	346	4.0	419	70	78	97	100	--
	47.2649765	104.552019	346	6.0	452	68	76	97	100	--
	47.2649765	104.552019	346	8.0	587	54	64	91	100	--
	47.2649765	104.552019	346	9.0	611	52	62	95	100	--
	47.2649045	104.551457	482	0.0	357	81	88	96	100	--
	47.2649045	104.551457	482	2.0	396	77	86	100	100	--
	47.2649045	104.551457	482	3.5	417	72	82	95	100	--
	47.2649045	104.551457	482	5.0	454	68	79	93	100	--
	47.2649045	104.551457	482	6.5	443	70	77	97	100	--
	47.2649045	104.551457	482	8.0	625	51	61	94	100	--
	47.2647773	104.550853	641	0.0	355	87	93	100	100	--
	47.2647773	104.550853	641	2.0	355	87	94	98	100	--
	47.2647773	104.550853	641	3.0	408	79	86	96	100	--
	47.2647773	104.550853	641	4.0	409	78	87	100	100	--
	47.2647773	104.550853	641	5.0	441	73	85	99	100	--
	47.2647773	104.550853	641	6.5	429	71	81	99	100	--
	47.2647086	104.550448	849	0.0	370	87	92	94	100	--
	47.2647086	104.550448	849	2.0	386	84	93	100	100	--
	47.2647086	104.550448	849	3.0	333	95	98	100	100	--
	47.2647086	104.550448	849	4.0	304	85	94	97	100	--
	47.2647086	104.550448	849	5.0	377	84	93	100	100	--
	47.2647086	104.550448	849	6.0	470	79	91	98	100	--
8/9/2011	47.2650712	104.55267	183	0.0	143	85	97	100	100	--
	47.2650712	104.55267	183	0.8	153	86	97	100	100	--
	47.2650712	104.55267	183	1.6	154	85	93	100	100	--
	47.2650712	104.55267	183	2.4	140	84	94	97	100	--
	47.2650712	104.55267	183	3.2	144	84	92	100	100	--
	47.2650712	104.55267	183	4.0	163	76	90	96	100	--
	47.265012	104.55211	323	0.0	138	87	95	100	100	--
	47.265012	104.55211	323	1.2	132	91	96	100	100	--
	47.265012	104.55211	323	2.4	136	81	91	100	100	--
	47.265012	104.55211	323	3.6	140	85	94	100	100	--

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Table 2. Suspended-sediment concentration, fall diameter, and sieve diameter for samples collected at discrete vertical depths at three sites near the proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011

[DD, decimal degrees; ft, feet mm, millimeter; mg/L, milligram per liter]

Dates	Latitude (DD)	Longitude (DD)	Location in cross section, distance from left bank looking downstream (ft)	Sampling depth (ft)	Suspended-sediment concen-tration (mg/L)	Suspended-sediment fall diameter (values in percent finer than size)				Percent finer than suspended sediment sieve diameter of 0.062 mm
						0.062 mm	0.125 mm	0.250 mm	0.500 mm	
	47.265012	104.55211	323	4.8	157	82	94	98	100	--
	47.265012	104.55211	323	6.0	170	74	84	100	100	--
	47.2649188	104.551618	443	0.0	142	90	93	100	100	--
	47.2649188	104.551618	443	1.2	115	83	90	100	100	--
	47.2649188	104.551618	443	2.4	151	82	87	96	100	--
	47.2649188	104.551618	443	3.6	180	74	85	100	100	--
	47.2649188	104.551618	443	4.8	159	80	88	100	100	--
	47.2649188	104.551618	443	6.0	183	72	81	96	100	--
	47.2648178	104.550986	602	0.0	143	90	93	100	100	--
	47.2648178	104.550986	602	0.8	152	87	93	100	100	--
	47.2648178	104.550986	602	1.6	138	87	92	100	100	--
	47.2648178	104.550986	602	2.4	173	81	85	100	100	--
	47.2648178	104.550986	602	3.2	161	83	91	100	100	--
	47.2648178	104.550986	602	4.0	148	86	92	100	100	--
	47.2646167	104.550129	827	0.0	142	96	99	100	100	--
	47.2646167	104.550129	827	0.8	146	91	97	100	100	--
	47.2646167	104.550129	827	1.6	157	91	96	100	100	--
	47.2646167	104.550129	827	2.4	135	93	97	100	100	--
	47.2646167	104.550129	827	3.2	156	85	93	100	100	--
	47.2646167	104.550129	827	4.0	173	90	93	100	100	--
8/23/2011	47.2649966	104.552262	266	0.0	68	91	93	100	100	--
	47.2649966	104.552262	266	1.0	60	95	95	100	100	--
	47.2649966	104.552262	266	2.0	64	92	95	100	100	--
	47.2649966	104.552262	266	3.0	64	95	97	99	100	--
	47.2649966	104.552262	266	4.0	89	83	92	100	100	--
	47.2649966	104.552262	266	5.0	151	81	87	100	100	--
	47.2649493	104.551916	352	0.0	85	94	97	100	100	--
	47.2649493	104.551916	352	1.1	92	IM	IM	IM	IM	79
	47.2649493	104.551916	352	2.2	68	94	95	100	100	--
	47.2649493	104.551916	352	3.3	63	IM	IM	IM	IM	68
	47.2649493	104.551916	352	4.4	50	IM	IM	IM	IM	77
	47.2649493	104.551916	352	5.6	48	IM	IM	IM	IM	94
	47.2648925	104.551593	434	0.0	49	IM	IM	IM	IM	85
	47.2648925	104.551593	434	1.0	53	IM	IM	IM	IM	70
	47.2648925	104.551593	434	2.0	51	IM	IM	IM	IM	85
	47.2648925	104.551593	434	3.0	52	IM	IM	IM	IM	88
	47.2648925	104.551593	434	4.0	54	IM	IM	IM	IM	83
	47.2648925	104.551593	434	5.0	71	92	99	100	100	--
	47.264804	104.551138	551	0.0	50	IM	IM	IM	IM	81
	47.264804	104.551138	551	0.6	41	IM	IM	IM	IM	94
	47.264804	104.551138	551	1.2	39	IM	IM	IM	IM	91
	47.264804	104.551138	551	1.8	55	IM	IM	IM	IM	74
	47.264804	104.551138	551	2.4	49	IM	IM	IM	IM	91
	47.264804	104.551138	551	3.0	61	80	87	100	100	--
	47.2646951	104.550634	680	0.0	79	IM	IM	IM	IM	58
	47.2646951	104.550634	680	0.4	72	IM	IM	IM	IM	63
	47.2646951	104.550634	680	0.8	46	IM	IM	IM	IM	96
	47.2646951	104.550634	680	1.2	53	IM	IM	IM	IM	93
	47.2646951	104.550634	680	1.6	46	IM	IM	IM	IM	88
	47.2646951	104.550634	680	2.0	32	81	93	100	100	88

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Table 3. Sieve diameters and mass of bedload samples for three sites near a proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011

[mm, millimeters]

Date	Bedload-sediment fall diameter (values in percent finer than size)										Mass (grams)
	0.062 mm	0.125 mm	0.25 mm	0.5 mm	1 mm	2 mm	4 mm	8 mm	16 mm	32 mm	
Above bypass chute											
7/19/2011	0	1	32	73	77	79	81	83	87	100	375.4
8/10/2011	0	0	2	87	95	96	97	98	100	100	167.8
8/24/2011	1	1	6	63	69	73	79	93	100	100	35.5
Adjacent to bypass chute											
7/21/2011	0	0	2	62	73	76	80	88	94	100	295.3
8/11/2011	0	0	1	55	70	76	81	87	100	100	221.3
8/24/2011	0	1	8	67	74	76	78	80	100	100	138.0
Below bypass chute											
7/20/2011	0	0	20	72	78	81	85	91	96	100	322.1
8/9/2011	0	0	1	59	72	78	82	87	96	100	308.8
8/23/2011	0	0	1	38	52	63	70	75	87	100	275.5

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Table 4. Sediment loads for three sites near a proposed bypass chute on the Yellowstone River near Glendive, Montana, July and August, 2011

Date	Load, in tons per day		
	Suspended sediment	Measured bedload	Measured total sediment
Above bypass chute			
7/19/2011	57,700	254	57,954
8/10/2011	8,250	36	8,286
8/24/2011	2,600	5	2,605
Adjacent to bypass chute			
7/21/2011	54,600	255	54,855
8/11/2011	6,350	53	6,403
8/24/2011	3,000	20	3,020
Below bypass chute			
7/20/2011	57,200	301	57,501
8/9/2011	9,260	96	9,356
8/23/2011	2,710	55	2,765

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Attachment 6 Bypass Channel

Appendix G

Fish Bypass Channel Entrance and Exit-Reclamation

10April2012

RECLAMATION
Managing Water in the West

PAP-1053

Lower Yellowstone Intake Diversion Dam

Fish Bypass Channel Entrance and Exit Pre-appraisal Study

Progress Report March 2012

By Bryan Heiner, Dale Lentz, and Josh Mortensen

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Hydraulic Investigations and Laboratory Services Group
Denver, Colorado

Lower Yellowstone Intake Diversion

Fish Bypass Channel Entrance and Exit Pre-appraisal Study

Progress Report

Prepared By:

Bryan Heiner
Hydraulic Engineer, Hydraulic Investigations and Laboratory Services Group, 86-68460

Dale Lentz
Hydraulic Engineer, Hydraulic Investigations and Laboratory Services Group, 86-68460

Josh Mortensen
Hydraulic Engineer, Hydraulic Investigations and Laboratory Services Group, 86-68460

Technical Approval: Robert F. Einhellig, P.E.
Manager, Hydraulic Investigations and Laboratory Services Group, 86-68460

7/27/2012

Peer Review: Brent Mefford, P.E. Date
Technical Specialist, Hydraulic Investigations and Laboratory Services Group, 86-68460

This study was funded by the U.S. Corps of Engineers, Omaha District. The Corps is the lead agency for the design and construction of fish passage at Intake Diversion Dam on the Yellowstone River near Glendive Montana.

Project Background

Intake dam is a Bureau of Reclamation irrigation diversion dam on the Yellowstone River approximately 70 miles upstream from its confluence with the Missouri River. It presents a barrier to fish migration on the Yellowstone. The project consists of two phases, first the construction of a fish screen structure to prevent fish entrainment (including the federally protected pallid sturgeon). Construction of a fish screen structure was initiated in 2010. The second phase includes design and construction of fish passage over or around the diversion dam. Two alternatives for fish passage were identified from project scoping studies for further design development. These are a rock ramp downstream of the diversion dam that would provide passage over the structure and a split-channel bypass that would provide passage around the diversion dam. Feasibility level design of the rock ramp alternative based on numerical and physical modeling has been completed. This report covers pre-appraisal level development of the split-channel bypass alternative focusing on design of the channel entrance and exit which are key to achieving fish passage and long term channel stability.

The Biological Review Team (BRT) provided guidance for the split-channel bypass fishway design in March, 2011. Their recommendations addressing bypass entrance and exit issues were as follows:

> (1) The BRT has concern that existing tracking data for pallid sturgeon indicates limited use of side channels during upstream migratory movement. The BRT recommends removing the 10% Diversion option and focusing on options capable of conveying 15%, 20%, 25%, and 30 % of the river flow.
>
> (2) We recognize the limitations of the 1-dimensional HEC-RAS model, but additional data related to the shear flow, mixing zone, and attraction flow at the fish entrance to the bypass channel are essential. Future analysis will be improved with additional data depicting the fish-way entrance and its orientation relative to the base of the dam and the main river thalweg.
>
> (3) A possible low weir to divert additional attraction water was discussed, and it would be good to review and evaluate some possible alignments as soon as possible.
>
> (4) Future analysis will be improved with the addition of model cross-sectional data at the water entrance and exit. Specifically, the BRT requests details on anticipated depths at the modeled discharges for these locations.
>
> (5) The bypass channel should be constructed such that 2 meters of water depth is possible at discharges exceeding 10,000 ft^3/s to better mimic those habitat parameters that coincide with adult pallid sturgeon locations (Bramblett 1996; Bramblett and White 2001; MFWP unpublished data).

The proposed bypass channel would extend from immediately downstream of the diversion weir to approximately 2 miles upstream of the diversion, Figure 1. Bypass channel entrance and exit used herein are referenced to upstream fish movement. The bypass entrance and exit refer to the downstream and upstream end of the channel respectively, opposite that of flow direction.

Split-Channel Bypass Alternative - Design Data Assumptions

Preliminary design data for the bypass were established based on applicable design data from the rock ramp fishway studies and recommendations of the BRT. Table 1 presents design data used for the basis of the designs presented in this report.

Table 1 - Considerations for the fish bypass design

Split-channel Bypass Flow	The bypass conveys a minimum of 15 percent of river flow at the dam for river flows larger than10,000 ft^3/s. Bypass flows of greater than the minimum are considered highly desirable for increased fish attraction and passage.
Average Fishway Velocity	Average flow velocity of about 1 m/s (~3 ft/s) at 10,000 ft^3/s river flow increasing to about 2 m/s (~6 ft/s) at 70,000 ft^3/s river flow. This range is similar to the mean river velocity measured about 1000 ft downstream of the dam and BRT recommendations.
Channel slope	Average bypass channel slope should be similar to that of the river below the dam which is about 0.00055.
Bypass Entrance Shape	Information from sturgeon tracking and habitat use studies were compiled with river cross section data below the dam. These data support a channel shape with a wide, nearly flat invert at the center of the channel transitioning to shallow sloping banks. The invert slope should gradually increase toward the bank lines.
Bypass Channel Entrance Depth	A thalweg depth of about 2 m (~6 ft) at 10,000 ft^3/s increasing to about 5 m (~16 ft) at 70,000 ft^3/s. Depth at the bypass entrance should be similar to the thalweg depth downstream of the dam.
Bypass Channel Exit Depth	A minimum thalweg depth of about 2 m (~6 ft) for river flows above 10,000 ft^3/s.

Bypass Entrance Orientation to the River	Flows from the bypass should merge with river flow in a downstream direction avoiding large eddies and strong shear zones.
Irrigation Diversion Criteria	Diverting water into the bypass must not impact the ability of the irrigation diversion to meet established diversion criteria. A minimum water surface of 1991.1 is required at Intake diversion headworks.
Channel Bed Roughness	The bypass channel entrance should be designed to support large areas of silt/sand and small gravel bed materials. Riprap required on the channel bed should be set below design grade and choked with fines.

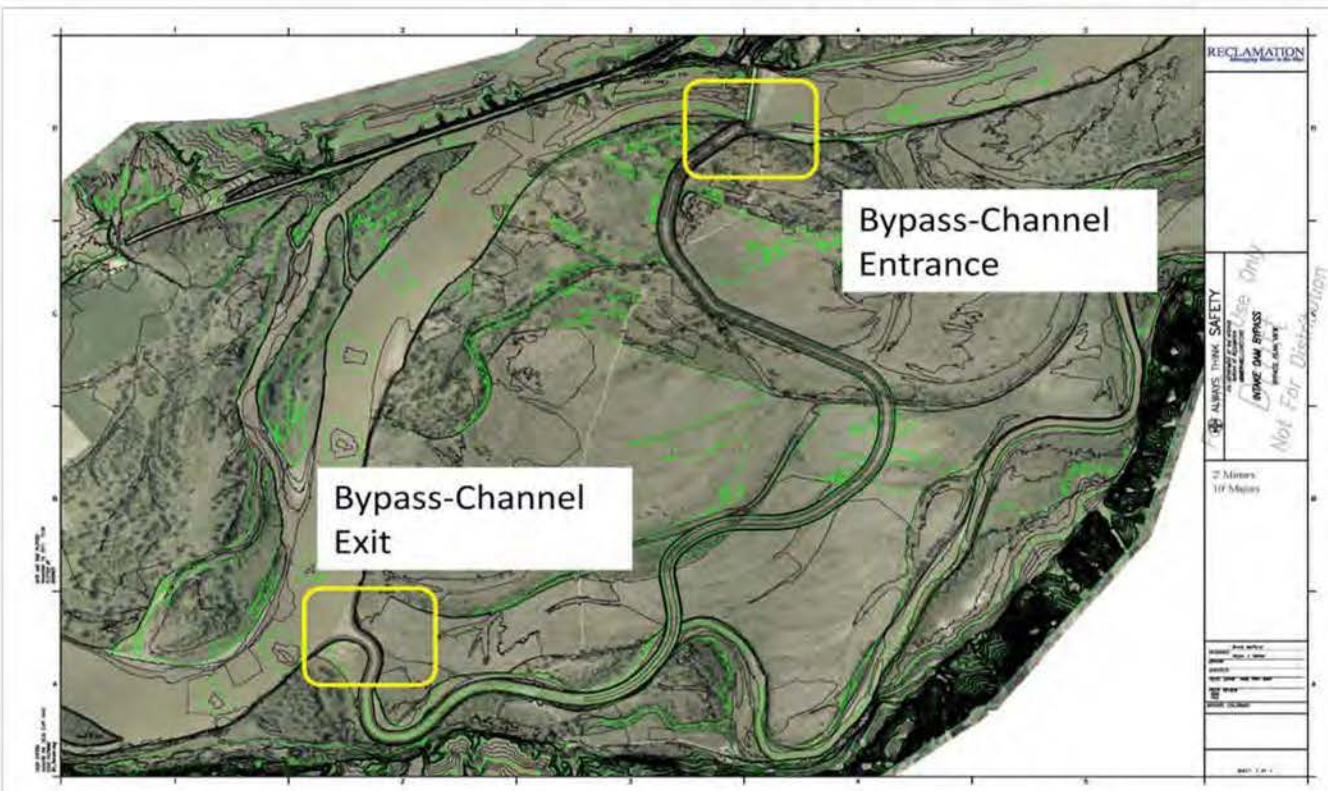

Figure 1 - Preliminary split-channel bypass design showing passage entrance and exit locations.

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Bypass Channel Shape

A characteristic shape for the bypass channel was developed assuming the channel should approximately emulate the main stem river downstream from the influence of the diversion dam. The bypass channel shape chosen has a flat invert with shallow side slopes that become steeper closer to the banks, Figure 2. The bypass channel side slopes are similar to the natural bank slopes found in river transitions between bends downstream of the diversion dam. The importance of providing shallow sloping banks off the channel invert were presented by the BRT and are supported by studies of channel habitat utilization by sturgeon in the Yellowstone River, (Bramblett, R. 1996, DNRC, 1977).

The bypass channel shape given in Figure 2 was used in this study to evaluate the channel entrance and exit transition shapes, alignments and hydraulic performance. As the bypass channel design advances, the shape of the bypass channel between the entrance and exit will likely include attributes of bends, transitions and straighter runs. The length of the outer bank slope (1V:2H) shown is approximate. For all drawings and flow simulations conducted for this study the outer bank slope was carried to daylight at the elevation of the natural topography.

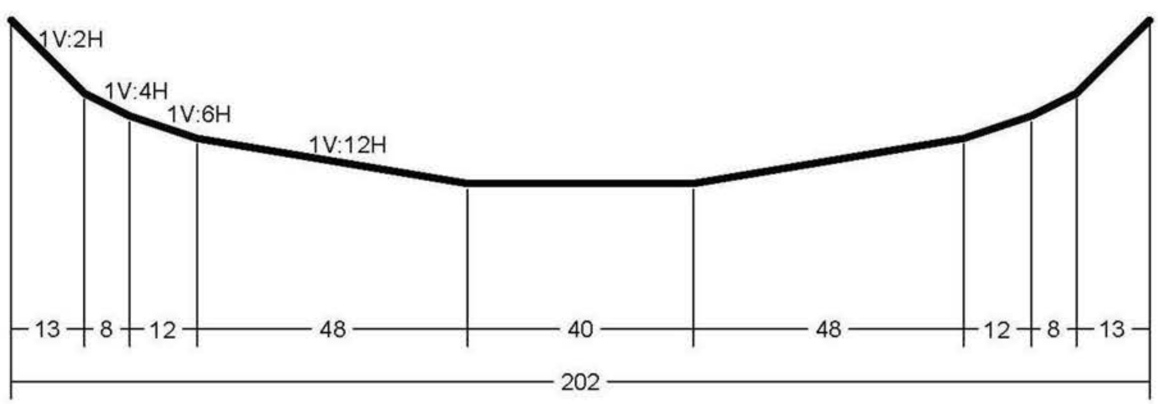

Figure 2 - Typical bypass channel section showing slopes and slope widths. Note: the plot's vertical scale is exaggerated by a factor of two.

Bypass Channel Entrance

The entrance to the bypass channel should be located approximately adjacent to the right abutment of the diversion dam. For this study, a new dam at the existing dam location with a crest geometry matching that developed for the design of the fish screens/headworks was assumed. Future selection of an upstream dam location should not significantly alter the findings of this study if the bypass entrance and dam are relocated together.

HEC-RAS Modeling

A HEC-RAS 1 dimensional numerical model of the Lower Yellowstone River around the Intake diversion dam was given to Reclamation from USACE. The model boundaries extended approximately 5 miles upstream and downstream of Intake dam. It also included approximately 4 miles of the irrigation canal and the proposed headworks structure. This existing model was modified to model various bypass alternatives, primarily focusing on the hydraulic characteristics of the bypass relating to fish passage and the percentage of flow in the bypass without affecting the ability to divert irrigations flows. Alternatives were not evaluated with respect to sediment transport capabilities, ice stability, or cut and fill. For all bypass alternatives the following parameters were assumed; channel roughness represented by a manning's n of .028, an invert elevation at the channel entrance of 1981.0 and a bypass channel length of 15500 ft. The sensitivity of the flow split between the river and bypass channel to higher roughness values was not analyzed. A Manning's n of .028 was considered a conservative low value for the purpose of guaranteeing diversion water.

Six bypass alternatives were modeled with varying slopes and channel widths. Major hydraulic parameters for each alternative are presented in Table 2. Alternatives 1 and 2 simulated 202 ft wide channels (assuming a reference depth of 14.5 ft) with slopes of 0.0005 and 0.0006, respectively. The steeper slope results in a higher invert elevation of the bypass channel at the upstream junction. In bypass alternatives 3 through 6, the horizontal invert of the bypass channel was increased by 50 ft, giving a reference channel width of 252 ft. These four alternatives then vary in channel slope from 0.0005 to 0.0007. Based on the HEC-RAS simulation results given in Table 2, Bypass 2 was carried forward for the development of bypass channel entrance and exit designs to a pre-appraisal level (no cost estimates). Bypass 2 was chosen as it represents the minimum excavation of the alternatives studied that meets the design data objectives. Bypass 2 meets all objectives with the exception of depth at the exit for a river flow of 10,000 ft^3/s. This was considered acceptable as the exit depth is similar to downstream river thalweg depths and exceeds the target depth prior to river flows reaching 20,000 ft^3/s.

A plot of average channel velocity in the bypass channel and downstream river is presented in Figure 3 for Bypass 2. Flow velocity for normal depth conditions in the upper reach of the bypass channel is similar to that of the downstream river. Tailwater inundation of the bypass entrance and lower channel reach results in a gradual reduction in bypass channel velocity from upstream toward the entrance. The entrance of the channel is designed to provide optimum flow depth, velocity and bed substrate for sturgeon habitat and movement as described by Bramblett, 1996. Plan and sections for the proposed bypass entrance are given as drawings 1 – 4 at the end of the report.

Table 2 - Bypass Alternatives with varying slopes and widths

Plan Name	Bypass 1	Bypass 2	Bypass 3	Bypass 4	Bypass 5	Bypass 6
Channel Length	15500	15500	15500	15500	15500	15500
Slope	0.0005	0.0006	0.0005	0.0006	0.00065	0.0007
Downstream Invert El.	1981	1981	1981	1981	1981	1981
Upstream Invert El.	1988.75	1990.3	1988.75	1990.3	1991.08	1991.85
Bypass width	202	202	252	252	252	252
Bypass Flow						
5 KCFS	1044	559	1544	894	613	366
10 KCFS	2024	1311	2867	1956	1524	1134
20 KCFS	3897	2905	5318	4099	3503	2902
30 KCFS	5618	4481	7535	6123	5555	4721
40 KCFS	7417	6151	9749	8245	7469	6684
70 KCFS	12814	11304	16304	14545	13658	12720
Main Channel Flow (Downstream of Bypass and Upstream of Irrigation Diversion)						
5 KCFS	3957	4442	3457	4107	4388	4635
10 KCFS	7977	8690	7134	8045	8477	8867
20 KCFS	16104	17096	14683	15902	16498	17099
30 KCFS	24383	25520	22466	23878	24446	25280
40 KCFS	32584	33850	30252	31756	32532	33317
70 KCFS	57187	58697	53697	55456	56343	57281
Main Channel Flow (Downstream of Irrigation Diversion)						
5 KCFS	2562	3041	2120 *	2708	2987	3234
10 KCFS	6579	7290	5733	6645	7077	7467
20 KCFS	14707	15694	13284	14502	15098	15698
30 KCFS	22983	24121	21067	22479	23047	23880
40 KCFS	31181	32451	28853	30357	31132	31915
70 KCFS	55787	57297	52298	54059	54944	55878
Bypass Flow/ Total Flow (Upstream of Bypass)						
5 KCFS	20.9%	11.2%	30.9%	17.9%	12.3%	7.3%
10 KCFS	20.2%	13.1%	28.7%	19.6%	15.2%	11.3%
20 KCFS	19.5%	14.5%	26.6%	20.5%	17.5%	14.5%
30 KCFS	18.7%	14.9%	25.1%	20.4%	18.5%	15.7%
40 KCFS	18.5%	15.4%	24.4%	20.6%	18.7%	16.7%
70 KCFS	18.3%	16.1%	23.3%	20.8%	19.5%	18.2%

Plan Name	Bypass 1	Bypass 2	Bypass 3	Bypass 4	Bypass 5	Bypass 6
Bypass Flow/ Flow Downstream of Dam						
5 KCFS	40.7%	18.4%	72.8%	33.0%	20.5%	11.3%
10 KCFS	30.8%	18.0%	50.0%	29.4%	21.5%	15.2%
20 KCFS	26.5%	18.5%	40.0%	28.3%	23.2%	18.5%
30 KCFS	24.4%	18.6%	35.8%	27.2%	24.1%	19.8%
40 KCFS	23.8%	19.0%	33.8%	27.2%	24.0%	20.9%
70 KCFS	23.0%	19.7%	31.2%	26.9%	24.9%	22.8%
Bypass Data						
10 KCFS						
Avg Velocity, upstream channel	3.0	2.8	3.2	3.0	2.8	2.6
Max Depth, upstream channel	6.1	4.8	5.8	4.6	4.0	3.3
% cross-section with 1.5-3 m depth	34%	0%	30%	0%	0%	0%
Avg Velocity, downstream channel	3.0	1.9	2.9	2.0	1.5	1.1
Max Depth, downstream channel	6.2	6.2	6.2	6.2	6.2	6.2
20 KCFS						
Avg Velocity, upstream channel	3.8	3.7	3.9	3.8	3.7	3.6
Max Depth, upstream channel	8.3	6.9	7.9	6.6	6.0	5.4
% cross-section with 1.5-3 m depth	59%	43%	55%	40%	33%	25%
Avg Velocity, downstream channel	3.6	2.6	3.5	2.7	2.3	1.9
Max Depth, downstream channel	8.6	8.7	8.6	8.7	8.7	8.7
30 KCFS						
Avg Velocity, upstream channel	4.3	4.2	4.4	4.3	4.3	4.2
Max Depth, upstream channel	9.8	8.5	9.4	8.1	7.6	6.9
% cross-section with 1.5-3 m depth	72%	62%	70%	58%	51%	43%
Avg Velocity, downstream channel	3.9	3.1	3.8	3.1	2.8	2.4
Max Depth, downstream channel	10.5	10.5	10.5	10.5	10.5	10.6
40 KCFS						
Avg Velocity, upstream channel	4.7	4.7	4.8	4.8	4.7	4.7

Plan Name	Bypass 1	Bypass 2	Bypass 3	Bypass 4	Bypass 5	Bypass 6
Max Depth, upstream channel	11.2	9.8	10.8	9.5	8.8	8.2
% cross-section with 1.5-3 m depth	45%	73%	48%	71%	66%	59%
Avg Velocity, downstream channel	4.3	3.5	4.2	3.5	3.2	2.8
Max Depth, downstream channel	12.0	12.1	12.0	12.1	12.1	12.1
70 KCFS						
Avg Velocity, upstream channel	5.7	5.7	5.7	5.8	5.8	5.8
Max Depth, upstream channel	14.6	13.2	14.1	12.7	12.1	11.4
% cross-section with 1.5-3 m depth	21%	29%	23%	33%	38%	43%
Avg Velocity, downstream channel	5.1	4.5	4.9	4.4	4.1	3.8
Max Depth, downstream channel	15.8	15.9	15.8	15.9	16.0	16.0
All simulations assume a canal diversion of 1400 cfs.						
* For the given bypass design and river flow the max irrigation diversion is 1338 cfs.						

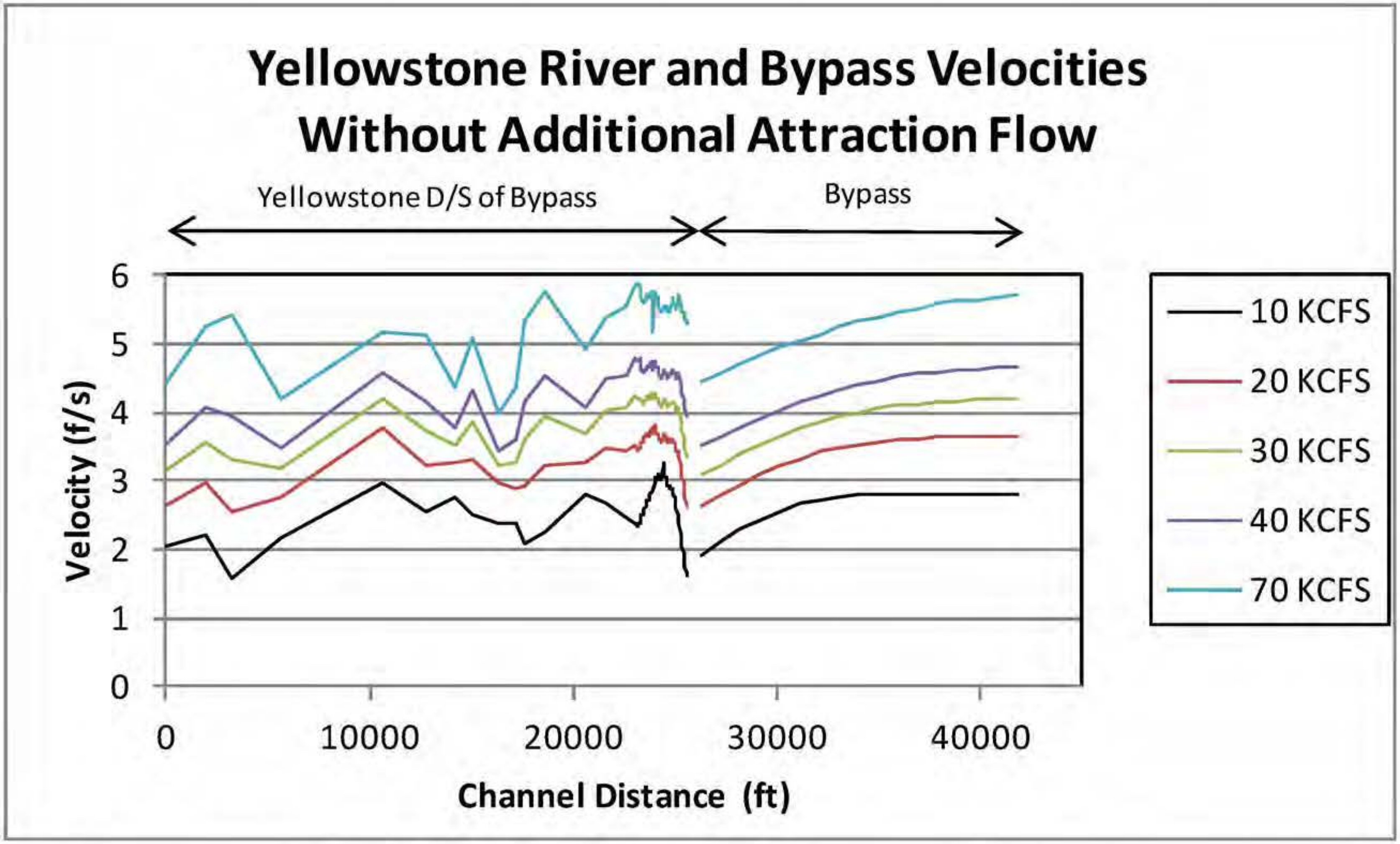

Figure 3 - Average channel velocity in the Yellowstone River downstream of the bypass entrance (left side) and in the bypass (right side).

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Split-Channel Bypass Exit Design

The bypass channel exit (fish passage exit) is shown at approximately the location of the existing natural high flow channel bifurcation, Figure 1. For pre-appraisal, this location was selected as it provides sufficient separation from the diversion dam to permit a large meandering bypass channel to be constructed at a slope similar to the river. The site is located on the outside of a shallow bend and supports a natural bifurcation to a high flow channel. These factors suggest the location is favorable for achieving a stable bypass channel design. However, further sediment and flow modeling of the bifurcation will be needed to confirm the location. The bypass exit (flow entrance) shown is shaped as a gradual contraction to minimize zones of sharp flow acceleration and shear lines within the flow that could cause upstream migrating fish to become disoriented at the flow split. Behavioral reactions of sturgeon negotiating flow transitions are poorly understood, therefore, the design of the bypass channel exit attempts to make the flow split as hydraulically smooth as possible. Plan and sections for the proposed bypass exit are given as Drawings 5-8.

Auxiliary Flow Lateral Weir Option

Two alternatives that include a lateral weir located in the entrance of the bypass channel were developed as options for increasing bypass channel flow for fish attraction. The lateral weir alternatives presented are designed as an extension of the right abutment of the diversion dam. Proper alignment of the lateral weir to the bypass channel and effective dissipation of flow energy passing over the weir are necessary to achieve improved attraction without creating shear flows that may misguide or significantly delay fish movement.

Several weir crest elevations, weir lengths and weir alignments were investigated for this study. A weir crest elevation of 1992.1 matching the elevation of the diversion dam at the right abutment was selected for the lateral weir. Flow over the weir crest commences at a river flow measured upstream of the bypass exit of about 10,000 ft^3/s. This crest elevation prevents the side-channel flow from impacting the ability of the diversion headworks to divert at full capacity for all river flows above 5,000 ft^3/s (measured upstream of the bypass exit). Plan and section views of the two lateral weir options developed are shown on Drawings 9-11 (Option 1) and 12-14 (Option 2). The options vary only in their alignment to the diversion dam. Option 1 is aligned at an angle to the diversion dam and approximately parallel with the bypass channel. This alignment attempts to reduce false attraction to the weir flow for fish by minimizing channel length between the bypass and weir. The option 2 weir is aligned at a right angle to the diversion dam providing greater length for dissipation of flow energy, but may increase false attraction for fish to the weir flow.

HEC-RAS Modeling of Lateral Weir for Auxiliary Attraction Flow

A lateral weir located on river right immediately upstream of Intake dam was added to the HEC-RAS Model for Bypass 2 (see Table 2). Various lengths and elevations of the weir were analyzed. A weir with a crest length of 150 ft at elevation 1992.1 was selected to achieve between 5 percent and 10 percent auxiliary flow augmentation to the bypass channel entrance without impacting irrigation diversion. The default coefficient for a broad crested lateral weir in

HEC-RAS is 2.0. Assuming an ogee crest, the lateral weir discharge coefficient can increase to around 3.3. For this study, simulations were conducted with discharge coefficients of both 2.0 and 3.0 to determine the possible range of auxiliary attraction flow. Results of model runs with lateral weir coefficients of 2.0 and 3.0 are presented in Table 3.

A plot of average channel velocity in the bypass channel, through the bypass channel entrance with a 150 ft lateral weir and continuing along the downstream river is presented in Figure 4. Flow over the lateral weir at river flows above about 20,000 ft^3/s yield a strong increase in bypass channel velocity in the bypass mouth. With the addition of weir flow, flow velocity at the mouth of the entrance rises to about the level of that of the downstream river. The 150 ft long weir provides from 2 percent (10,000 ft^3/s river) to 7 percent (70,000 ft^3/s river) additional river flow to the bypass channel entrance using a weir coefficient of 2.0.

Table 3 - Bypass channel and weir flows with a weir coefficient of 2 and 3

Weir Elevation=	1992.1
Weir Length=	150
Weir coefficient=	**2**

River Flow, ft^3/s	Bypass Flow, ft^3/s	Weir Flow, ft^3/s	Canal Flow, ft^3/s	Bypass Flow as % of River Flow	Weir Flow as % of River Flow	Bypass & Weir Flow as % of River Flow
5000	558.9	0.0	1399.3	15.5%	0.0%	15.5%
10000	1305.8	145.5	1400.0	15.2%	1.7%	16.9%
20000	2884.0	843.8	1400.8	15.5%	4.5%	20.0%
30000	4428.4	1608.6	1400.0	15.5%	5.6%	21.1%
40000	6037.6	2401.4	1400.3	15.6%	6.2%	21.9%
70000	11052.1	4755.7	1400.0	16.1%	6.9%	23.0%

Weir Elevation=	1992.1
Weir Length=	150
Weir coefficient=	**3**

River Flow, ft^3/s	Bypass Flow, ft^3/s	Weir Flow, ft^3/s	Canal Flow, ft^3/s	Bypass Flow as % of River Flow	Weir Flow as % of River Flow	Bypass & Weir Flow as % of River Flow
5000	552.2	0.0	1401.4	15.3%	0.0%	15.3%
10000	1304.8	210.1	1400.2	15.2%	2.4%	17.6%
20000	2883.8	1208.3	1402.2	15.5%	6.5%	22.0%
30000	4414.4	2301.1	1401.7	15.4%	8.0%	23.5%
40000	6018.9	3433.8	1400.0	15.6%	8.9%	24.5%
70000	10976.6	6774.9	1400.1	16.0%	9.9%	25.9%

Both options are shown with 1 ft high stepped aprons on the downstream side of the weir. The stepped aprons were included assuming roller compacted concrete construction would be used for the weir. The steps will help dissipate a portion of the flow energy on the apron and breakup the flow nappe before it merges with bypass channel flow. However, weir height to critical depth on the weir is less than 10 and therefore, the steps contribute relatively little to energy dissipation of flow on the apron. Table 4 presents the elevation drop between the river and bypass channel water surfaces, weir unit discharge, head on the weir and the estimated residual energy at the bypass water surface.

Table 4 - Lateral weir flow parameters

Weir Elevation= 1992.1
Weir Length= 150
Weir coefficient= 2

River Flow	River WSEL	Bypass WSEL	Difference	Unit Discharge	Head on Weir	$H_{residual}/H_{max}$ [1]
ft^3/s	ft	ft	ft	ft^3/s	ft	
10000	1992.8	1987.2	5.6	0.97	0.7	0.5
20000	1994.2	1989.7	4.5	5.62	2.1	0.83
30000	1995.3	1991.6	3.8	10.7	3.2	0.94
40000	1996.3	1993.1	3.2	16.0	4.3	0.95
70000	1998.8	1997.0	1.8	31.7	4.9	~1

Weir Elevation= 1992.1
Weir Length= 150
Weir coefficient= 3

River Flow	River WSEL	Bypass WSEL	Difference	Unit Discharge	Head on Weir	$H_{residual}/H_{max}$ [1]
ft^3/s	ft	ft	ft	ft^3/s	ft	
10000	1992.8	1987.2	5.5	1.4	0.7	0.52
20000	1994.2	1989.7	4.5	8.0	2.1	0.86
30000	1995.3	1991.6	3.7	15.3	3.2	0.95
40000	1996.2	1993.1	3.0	22.9	4.3	0.97
70000	1998.6	1997.0	1.6	45.2	4.9	~1

[1] Reference Boes and Hager (2003) "Design of stepped spillways", ASCE Journal of Hydraulics Engineering, Sept.
* River data at station 28062 (50 ft upstream of dam), bypass data at station 194.7 (station where LW flow enters).

This study does not provide detail on the merging of flows from the bypass channel, lateral weir and river. This will need to be investigated using physical and three dimensional numeric models. Limited 3-dimensional numerical flow modeling was conducted for this study to identify major flow patterns for developing the initial alignment of the bypass entrance and lateral weir option.

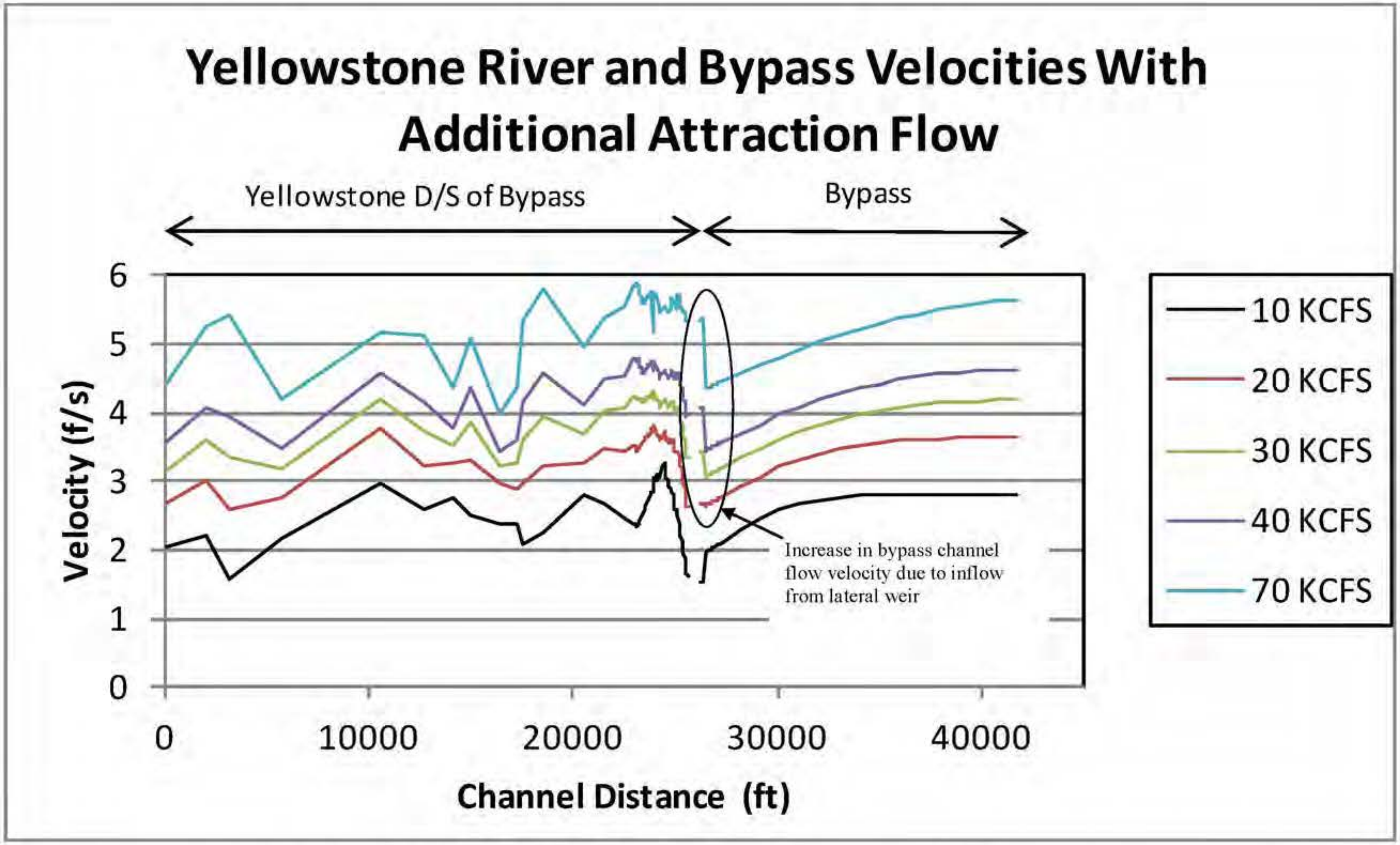


Figure 4 - Average channel velocity in the Yellowstone River downstream of the bypass entrance (left side) and in the bypass (right side) with additional attraction flow from lateral weir

CFD Modeling

Preliminary Flow3D[1] modeling of the bypass entrance with a lateral weir was conducted to determine major flow patterns associated with merging bypass channel flow, lateral weir flow and river flow. The model was not run to stabilization and results should not be used to make any quantitative conclusions. Figure 5 shows a plan view of the velocity magnitudes in ft/s that occur at a river flow of 40,000 ft^3/s (upstream of the bypass exit). Although a course-grid model was used containing many assumptions, the complex interaction that occurs when the bypass re-enters the river for lateral weir option 1 suggests that favorable approach conditions to the bypass can be achieved with further analysis. Due to the limitations of CFD modeling to quickly look at multiple weir alignments it is recommended that a physical model be used to further analyze the bypass entrance conditions.

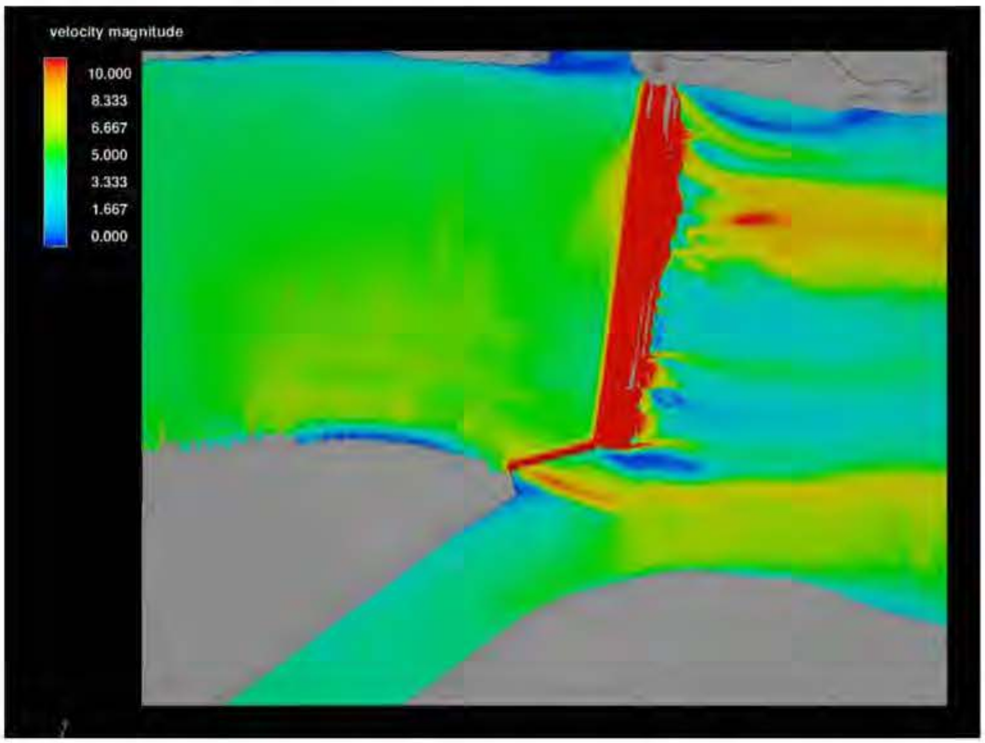


Figure 5 - Flow3D model of the fish bypass entrance and river confluence (flow is from left to right)

Drawings

Preliminary bypass channel alignments were provided by the Corps. Reclamation used the initial layouts and created a dynamic AutoCAD Civil3D model which allowed refinement of the entrance and exit geometries. Considering that the Corps uses different modeling programs and cannot open Civil3D files, the dynamic models were converted to standard AutoCAD files and

[1] a commercially available computation fluid dynamics (CFD) program

PDF's. Drawings include plan and section views for the bypass entrance without a lateral weir, bypass exit and two different configurations of the bypass entrance with a lateral weir. For the drawings, it was assumed that the new crest geometry will be placed in approximately the same location as the existing dam. All drawings can be found at the end of the report.

References

Bramblett, G.R., 1996, "Habitats and Movements of Pallid and Shovelnose Sturgeon in the Yellowstone and Missouri Rivers, Montana and North Dakota," PhD thesis, Biological Sciences, Montana State University-Bozeman

Elser, A. A., et al., July 1977, "The Effect of Altered Streamflow on Fish of the Yellowstone and Tongue Rivers, Montana," Montana Department of Natural Resources and Conservation Water Resources Division, Technical Report No. 8.

List of Drawings

Bypass Channel Entrance with no Lateral Weir

1. Fish Passage Inlet Sheet 1 – Plan
2. Sheet 2 – Sections 0+00 to 1+50
3. Sheet 3 – Sections 2+00 to 3+50
4. Sheet 4 – Sections 4+00 to 4+50

Bypass Channel Exit

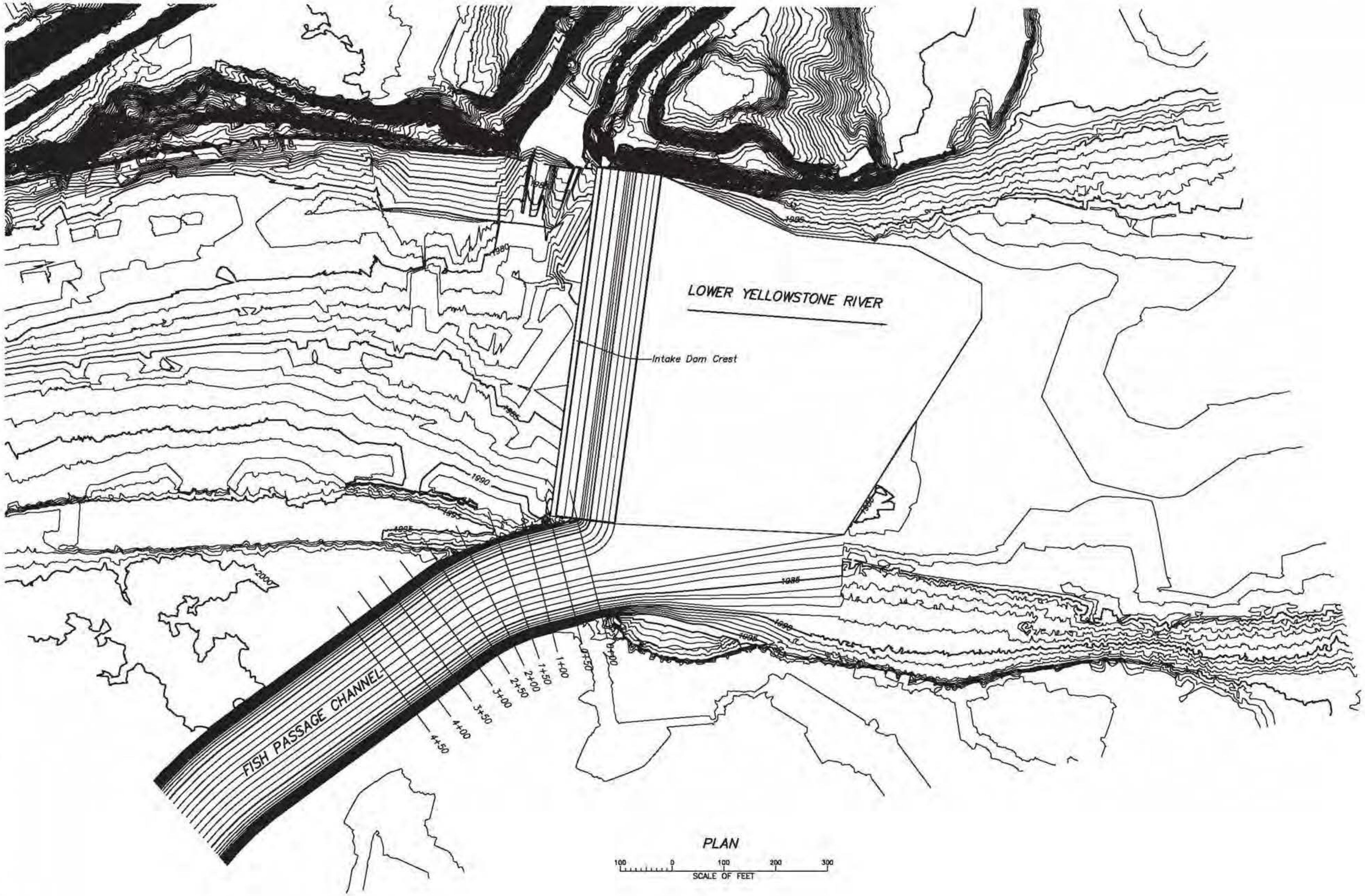
5. Fish Passage Outlet Sheet 1 – Plan
6. Sheet 2 – Sections 155+50 to 157+00
7. Sheet 3 – Sections 157+50 to 158+50
8. Sheet 4 – Sections 159+00 to 159+50

Bypass Channel Entrance with Lateral Weir Option 1

9. Fish Passage Option 1 Sheet 1 - Plan and Typical Section
10. Sheet 2 – Sections 0+00 to 0+75
11. Sheet 3 – Sections 1+00 to 1+50

Bypass Channel Entrance with Lateral Weir Option 2

12. Fish Passage Option 2 Sheet 1 - Plan and Typical Section
13. Sheet 2 – Sections 0+00 to 0+75
14. Sheet 3 – Sections 1+00 to 1+50



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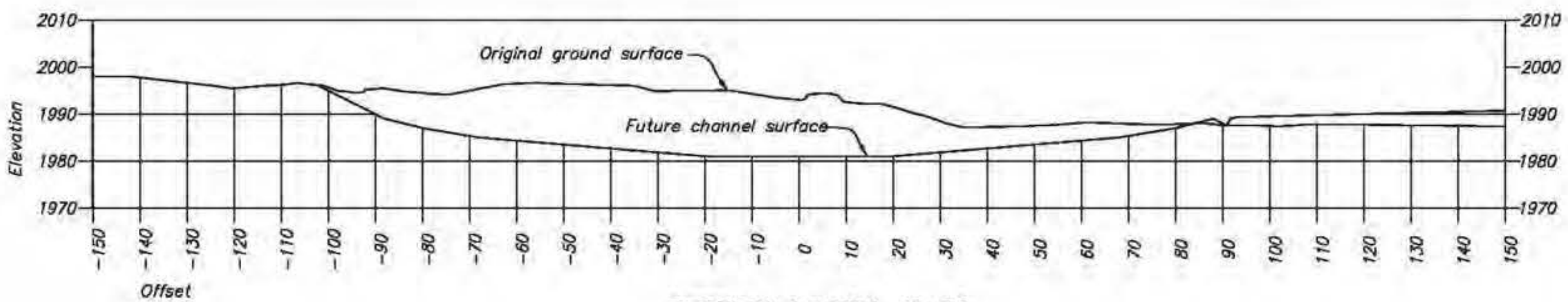

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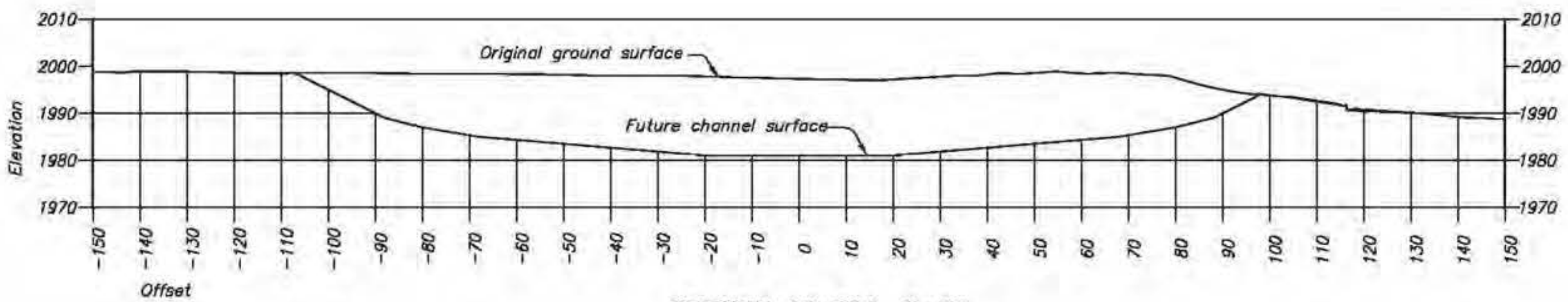

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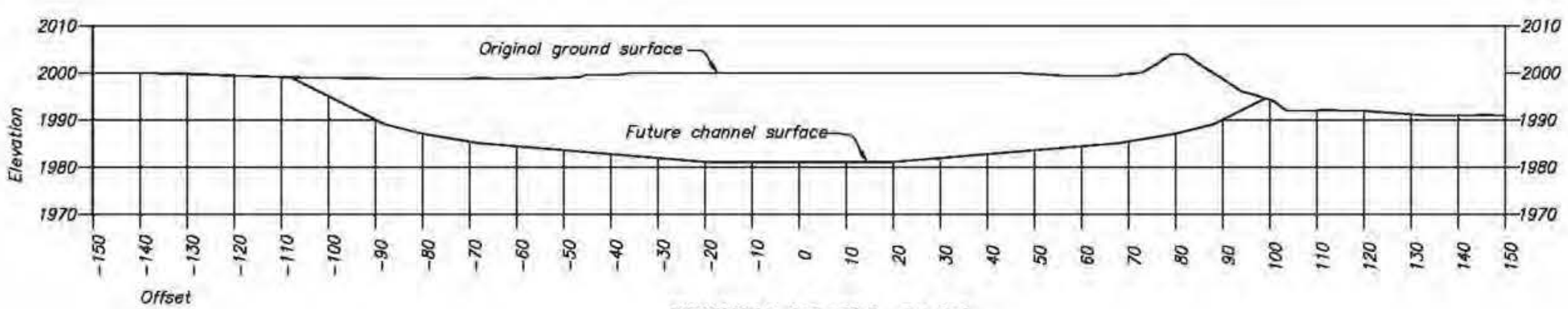

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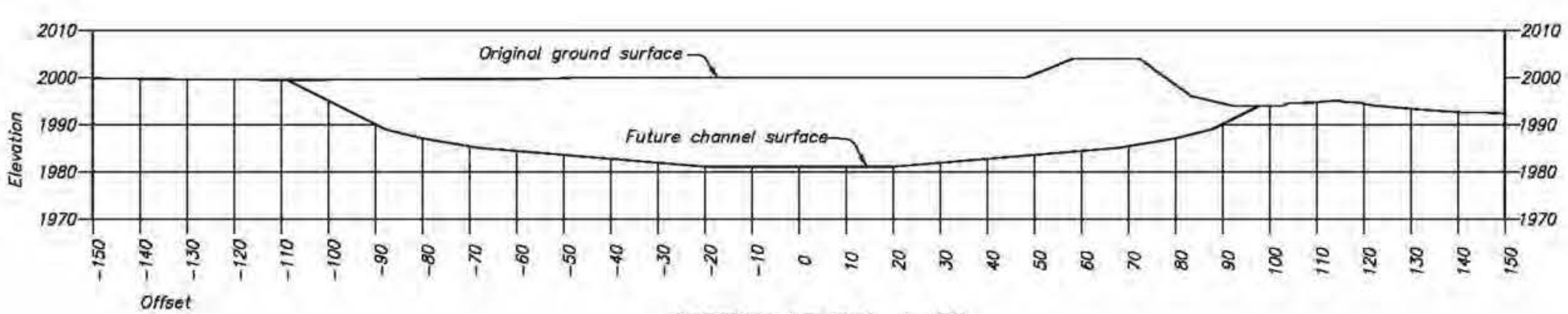

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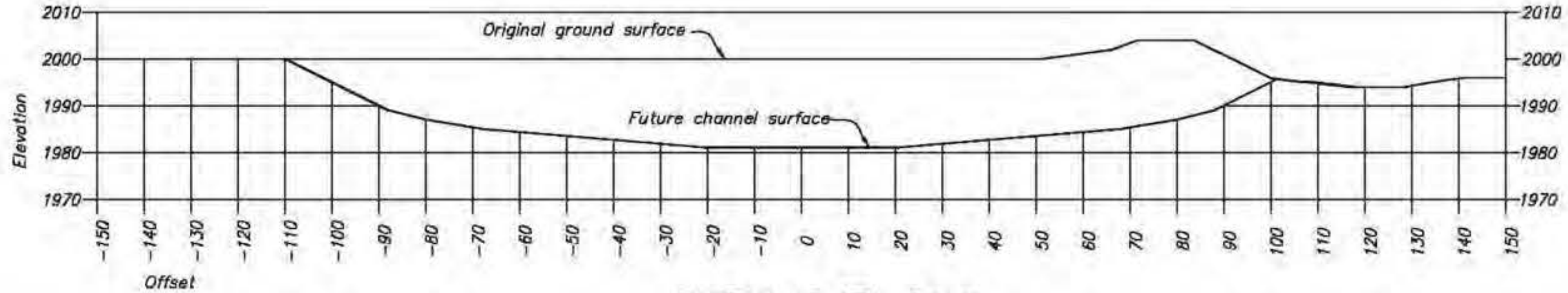

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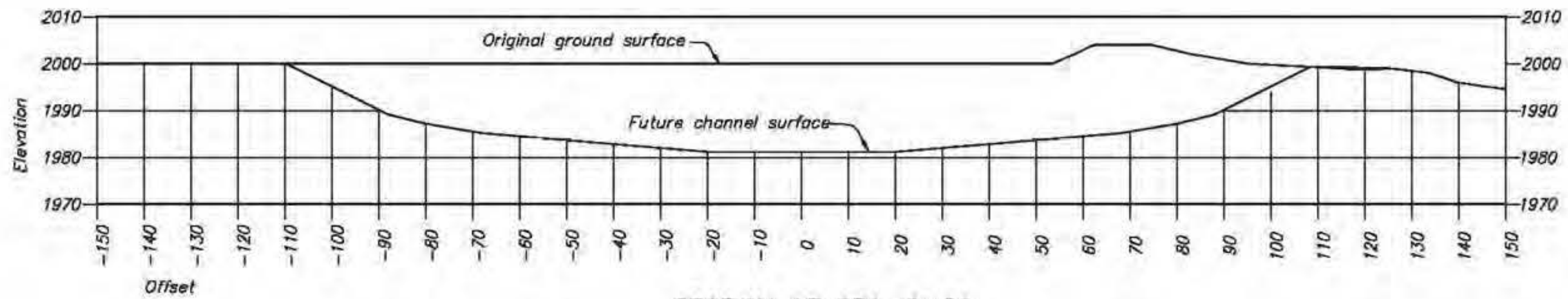

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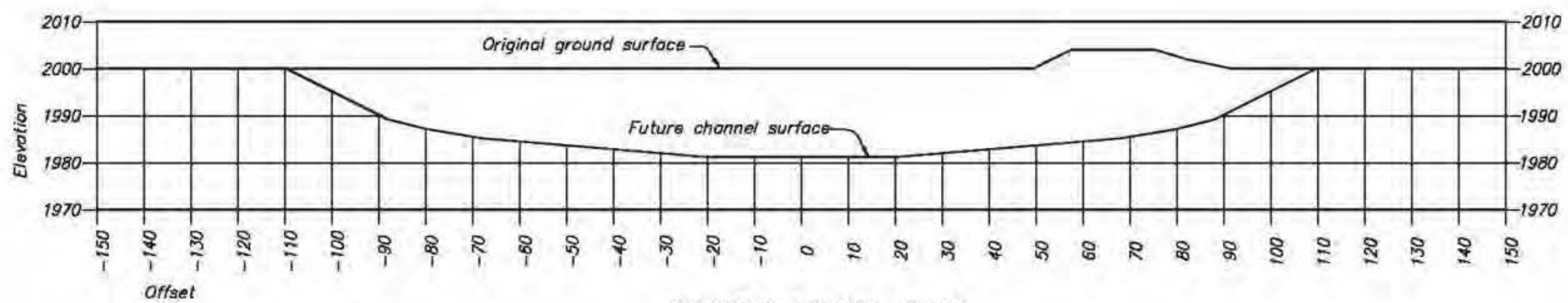

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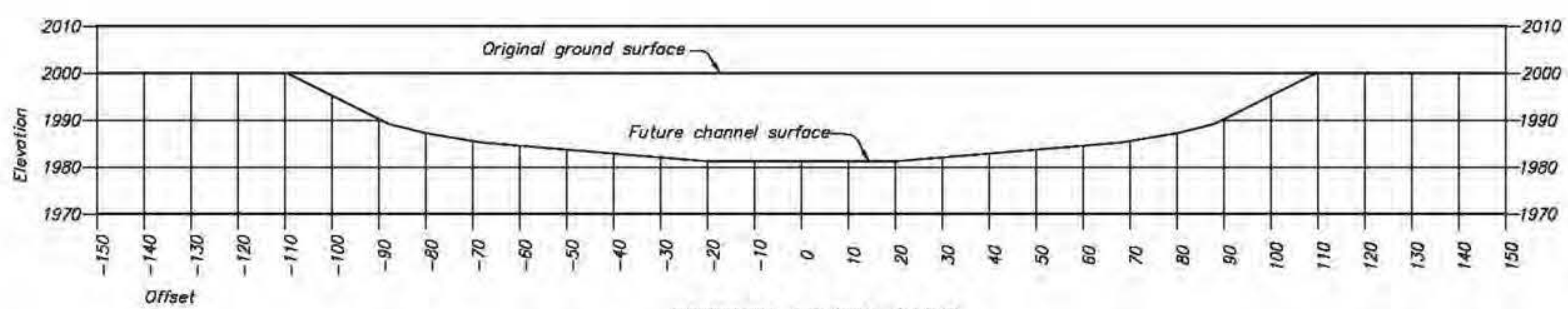

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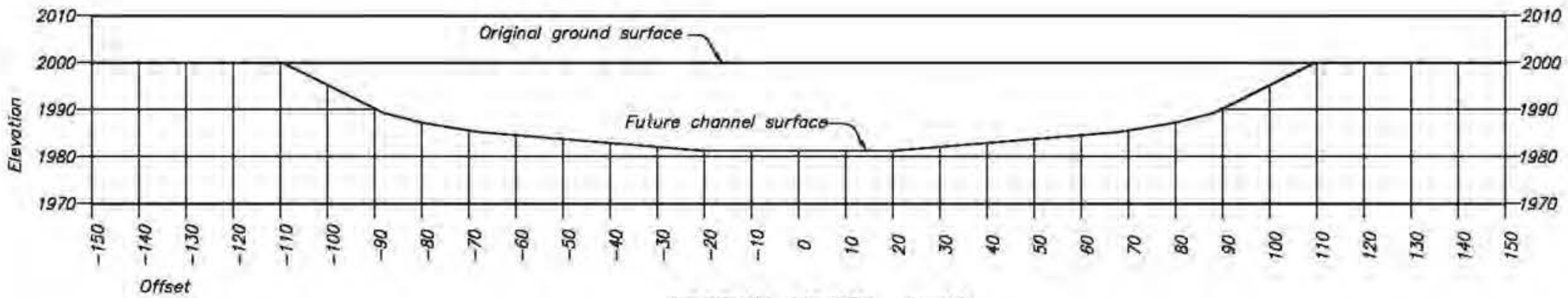

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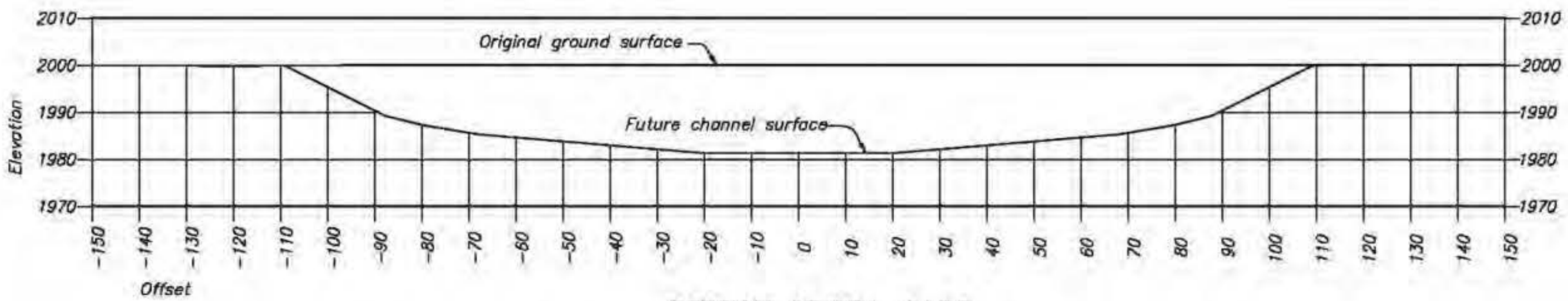

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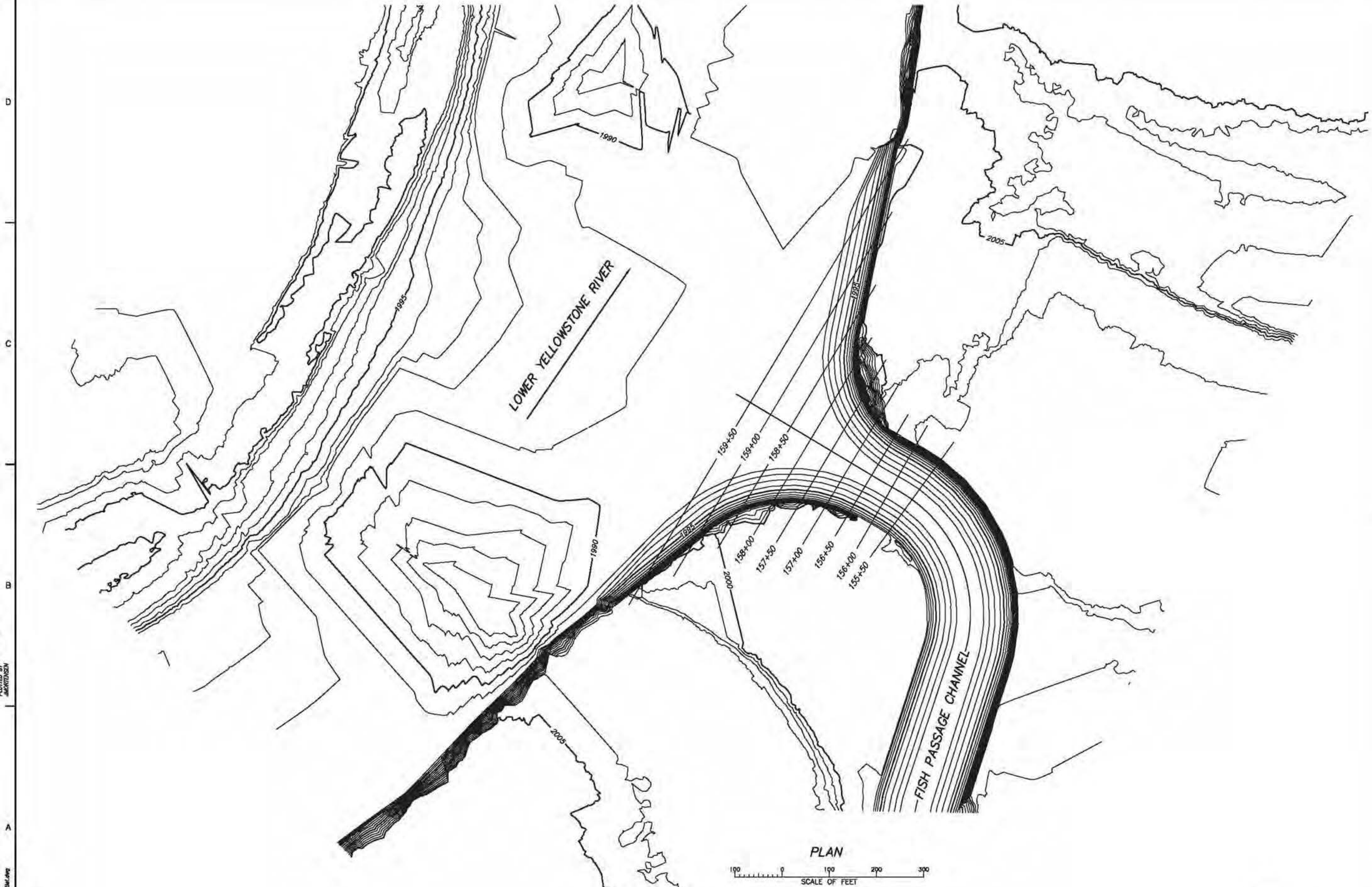

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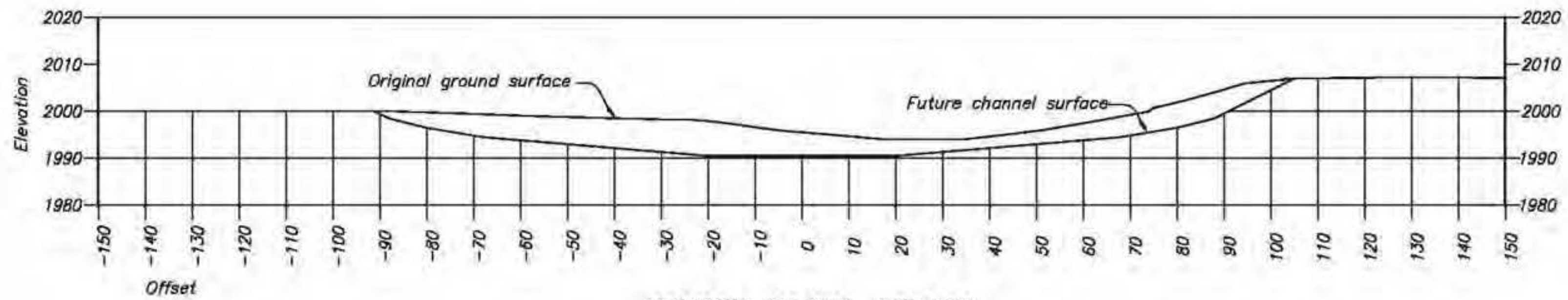

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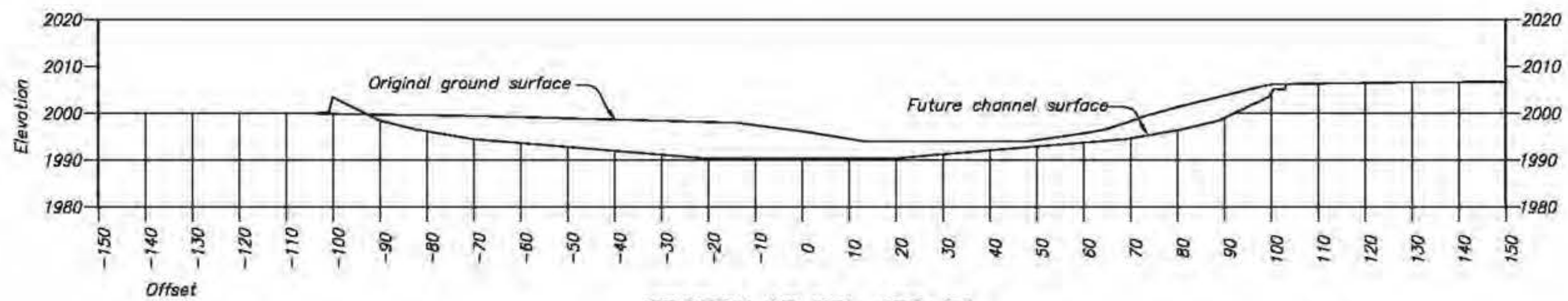

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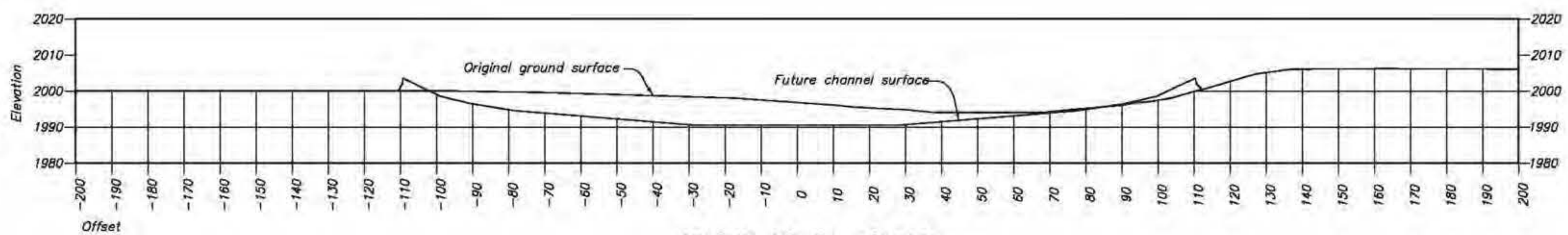

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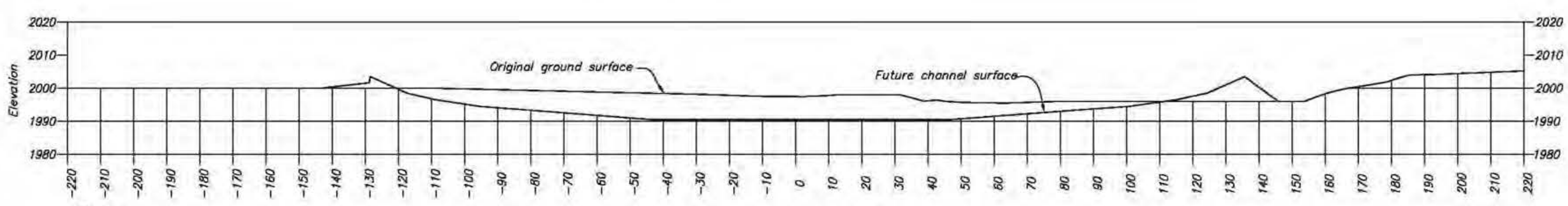

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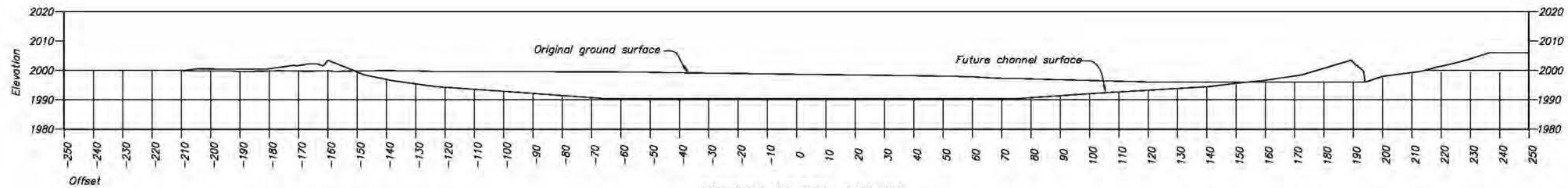

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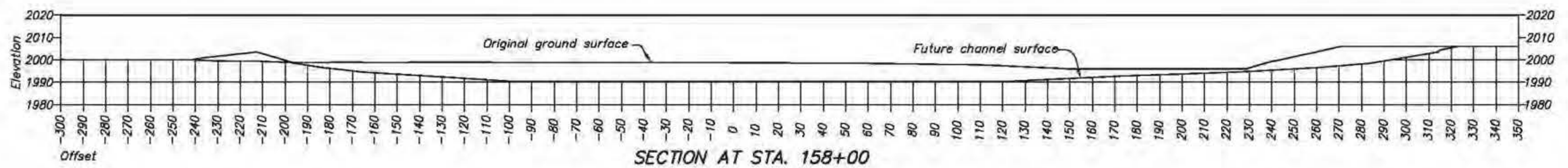

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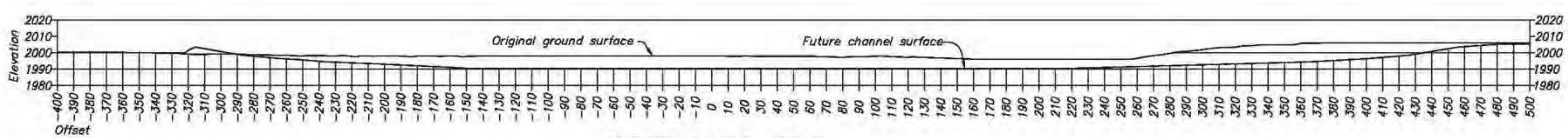

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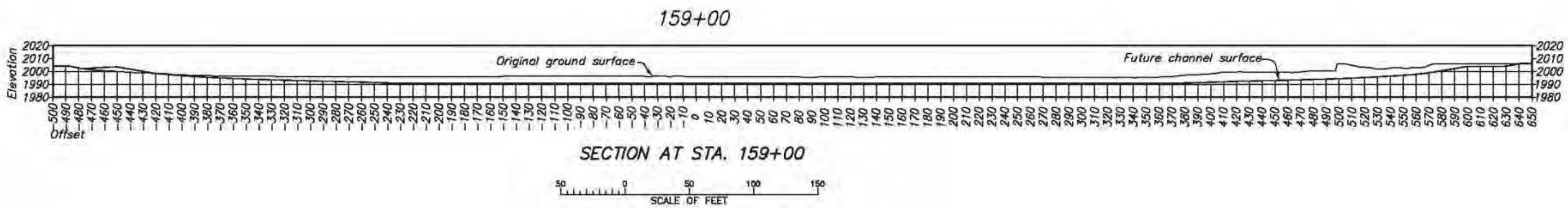

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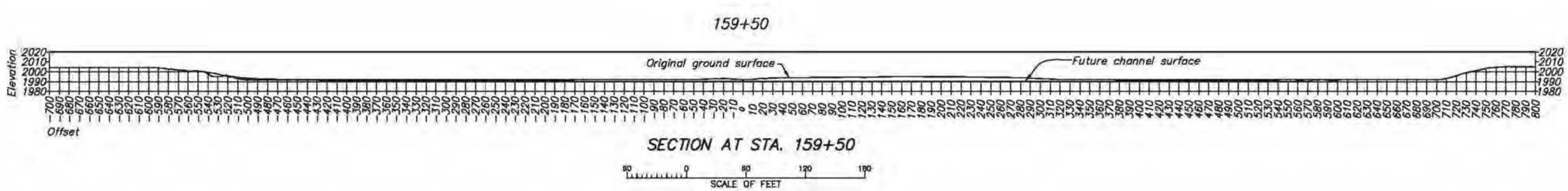

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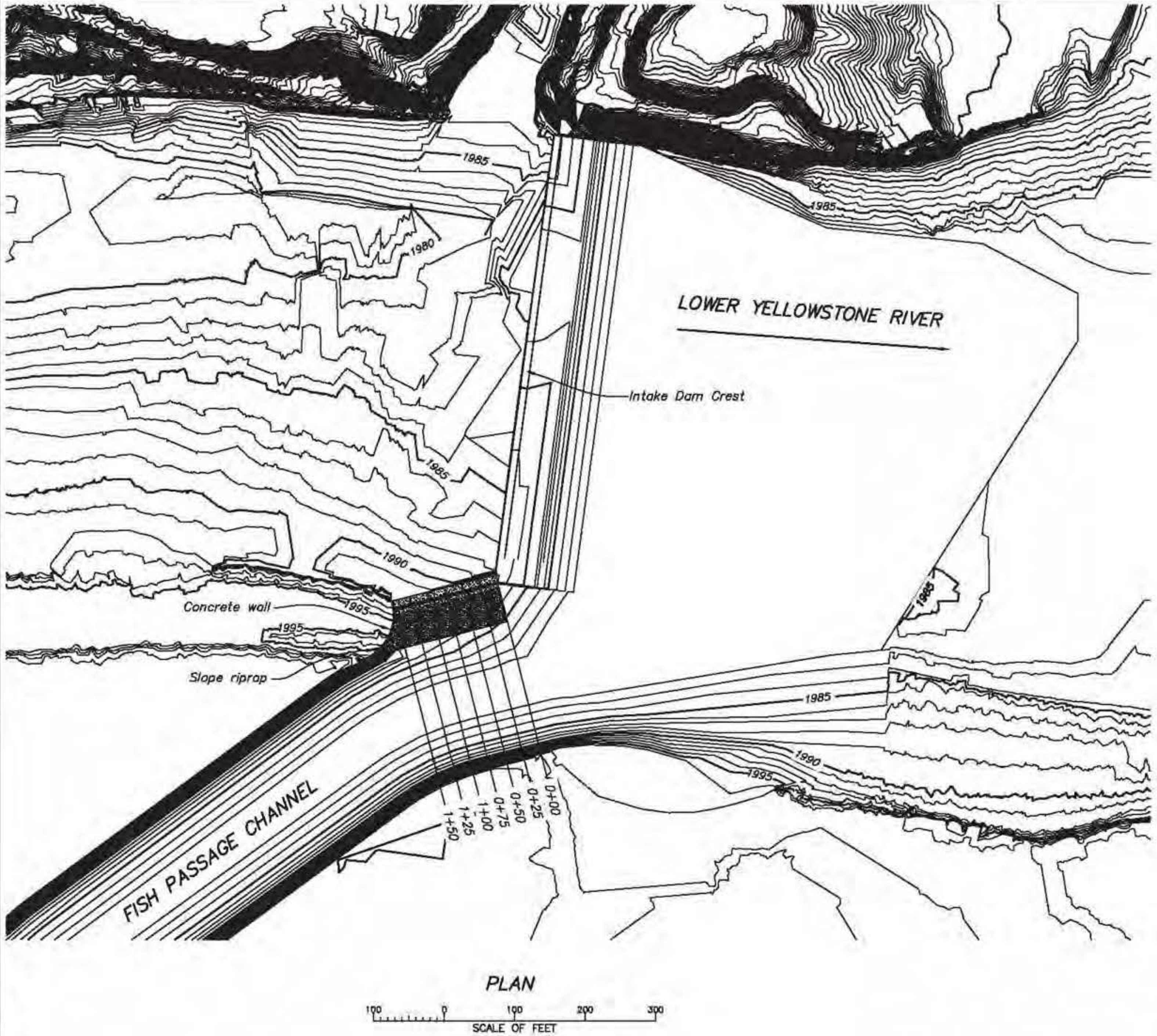

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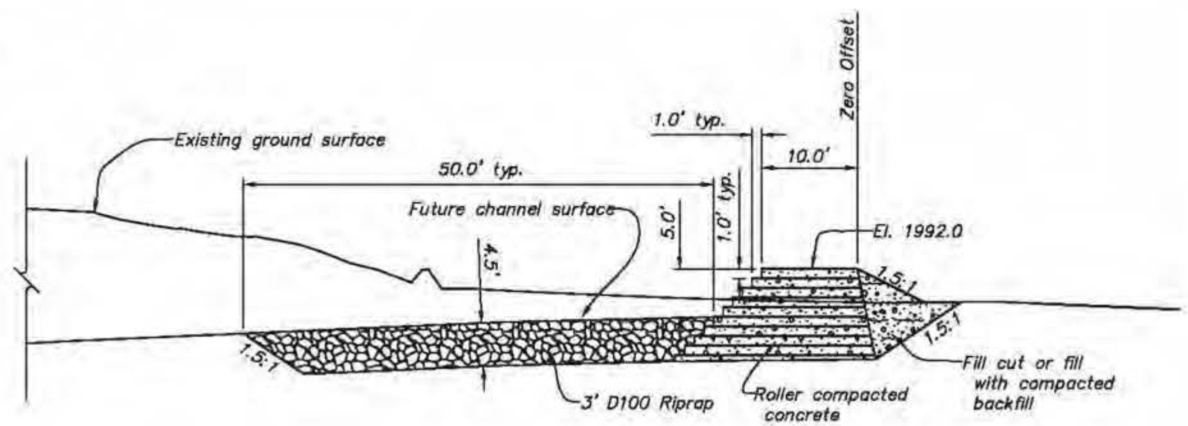

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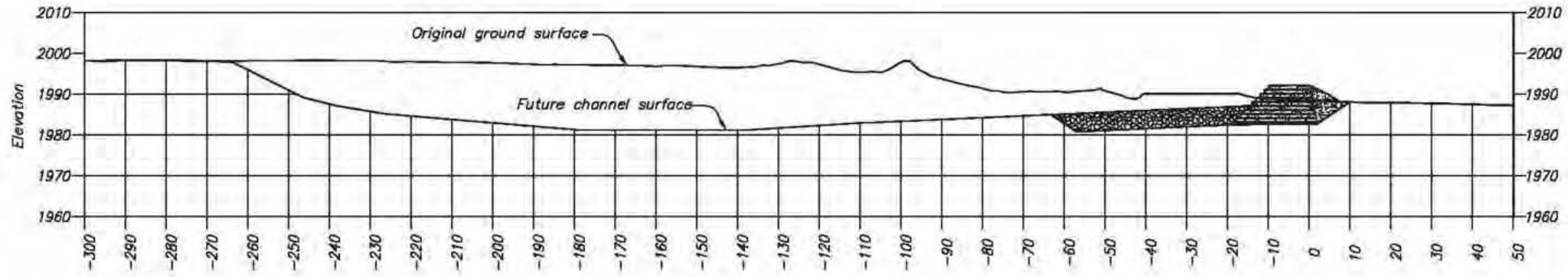

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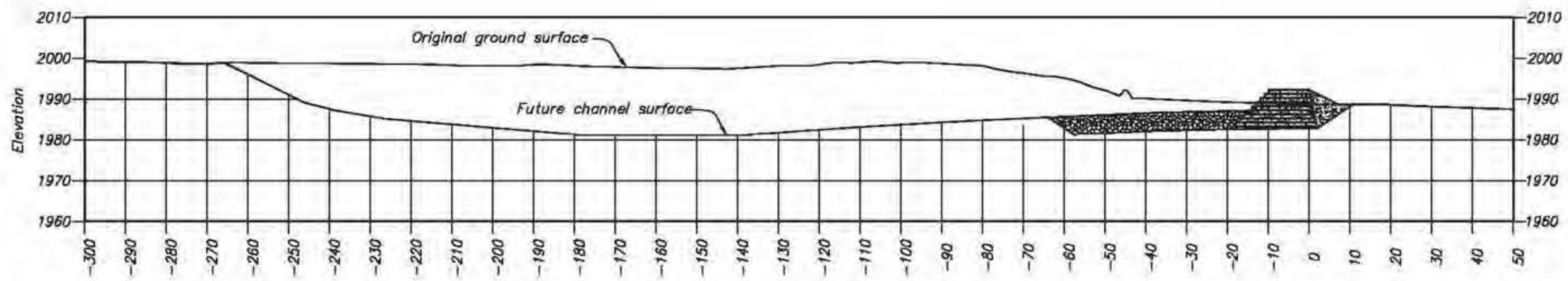

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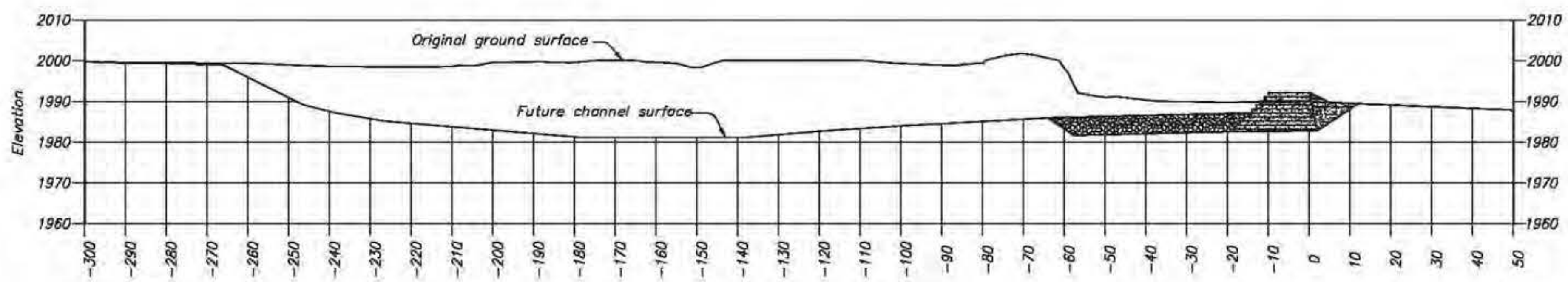

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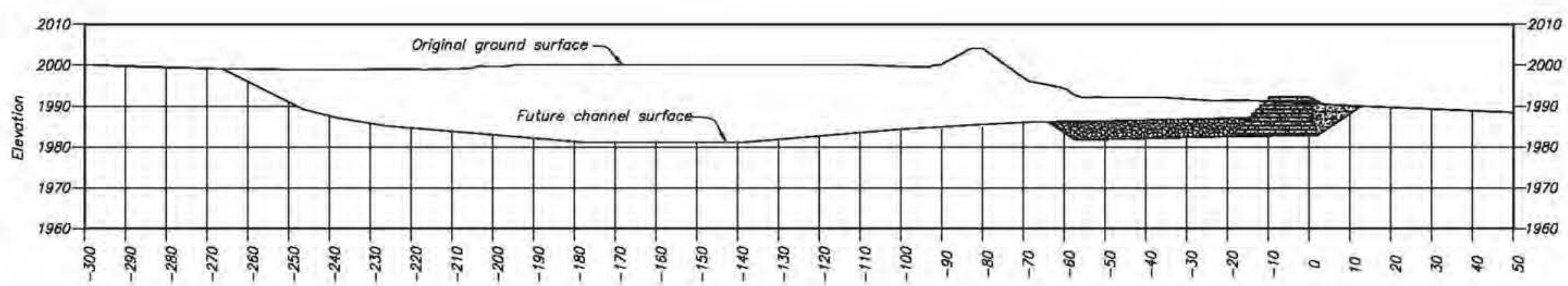

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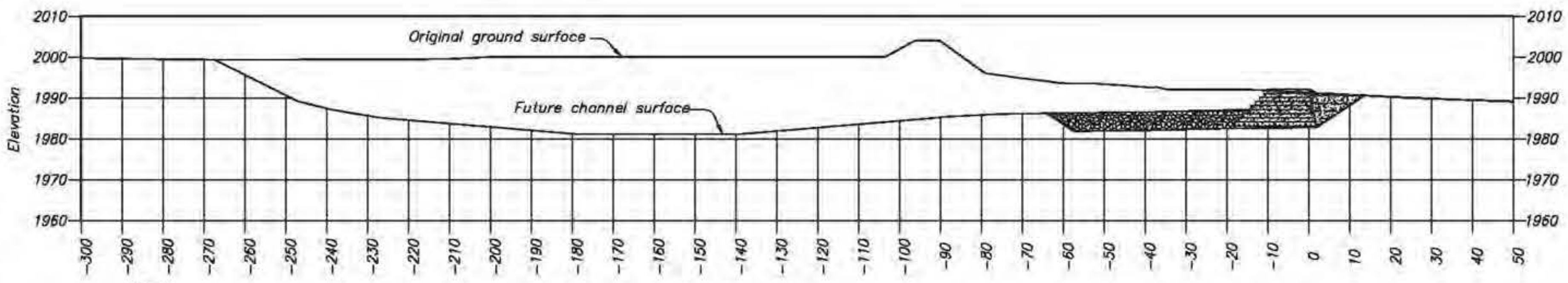

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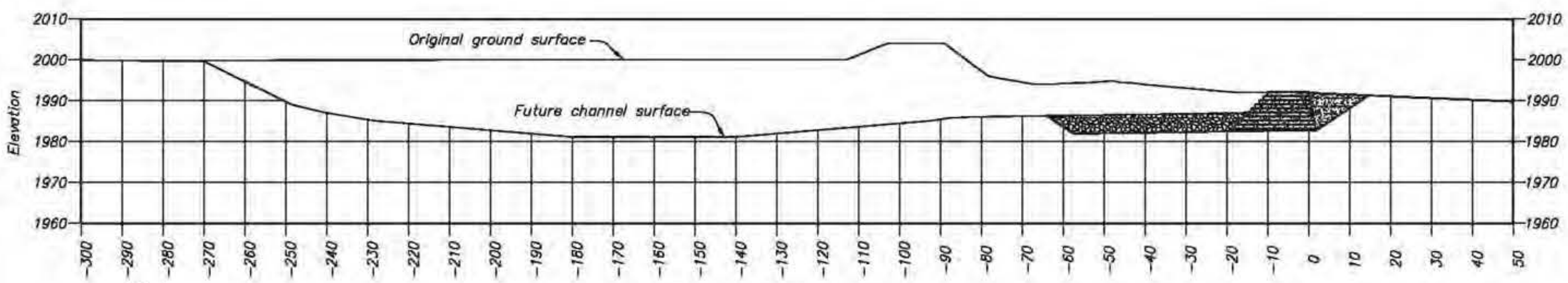

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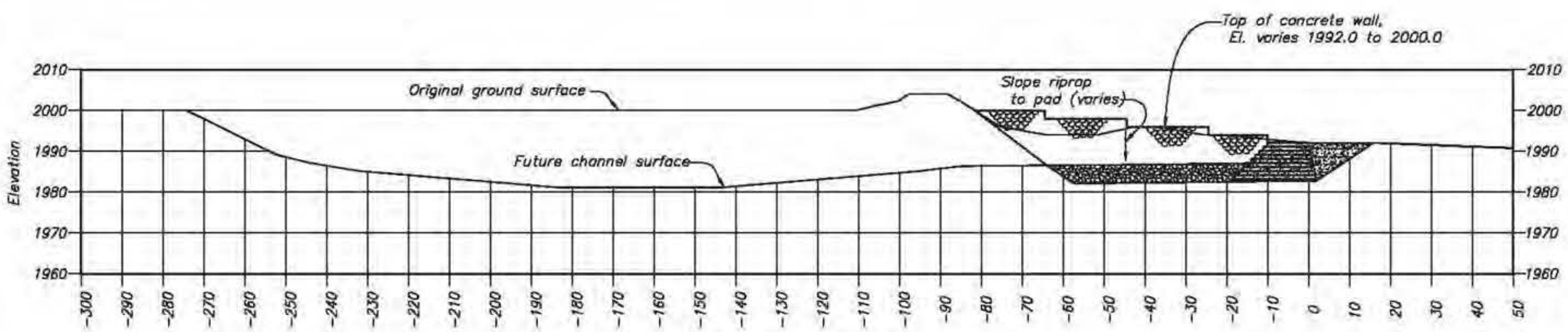

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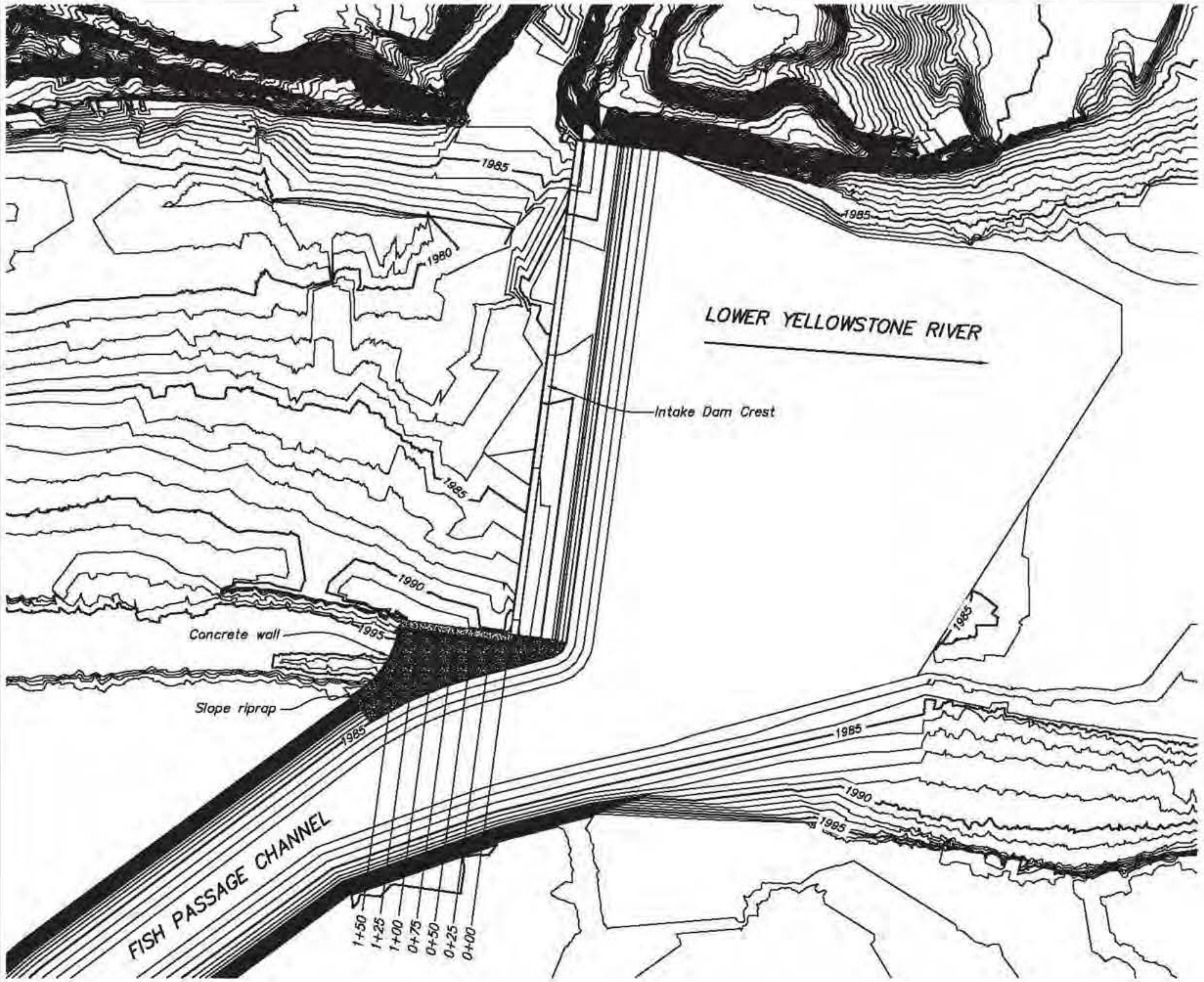

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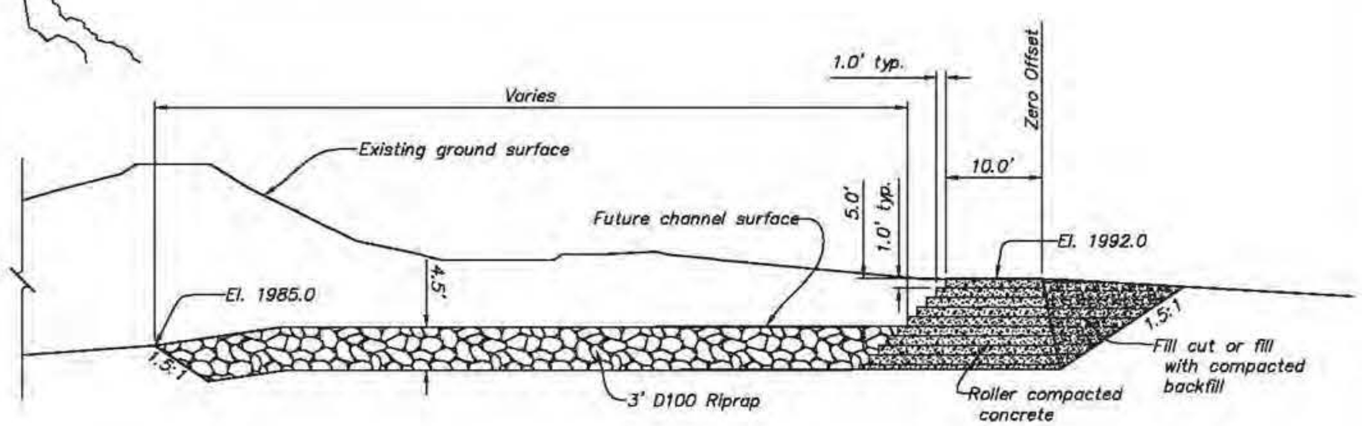

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LOWER YELLOWSTONE RIVER
INTAKE DAM
FISH PASSAGE - OPTION 2

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Drawing No. 12
SHEET 1 OF 3

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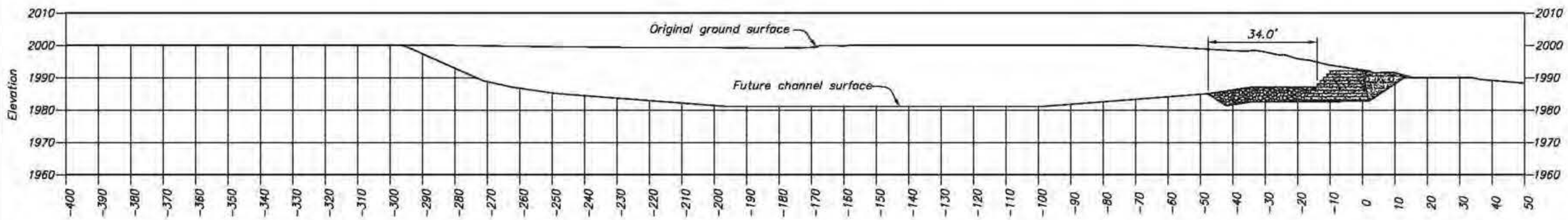

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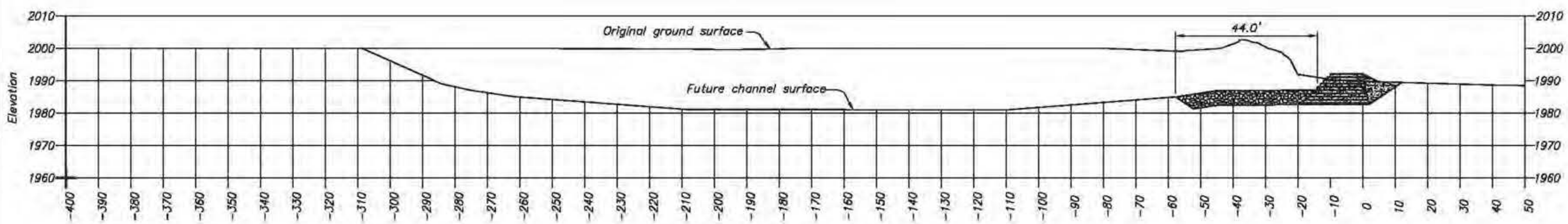

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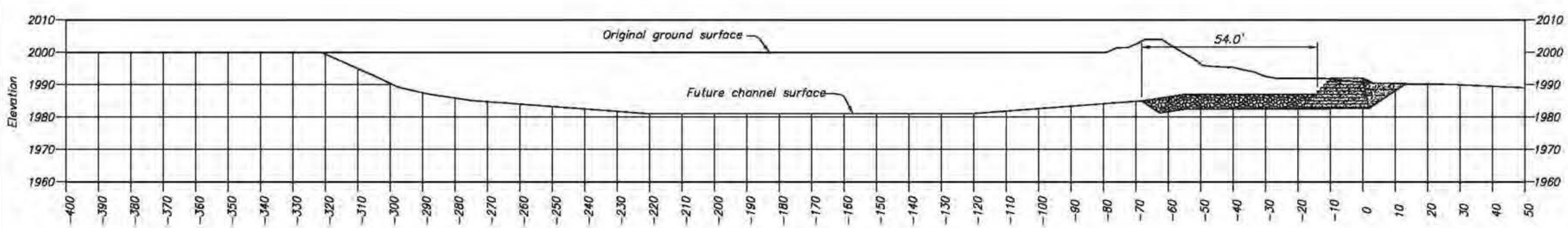

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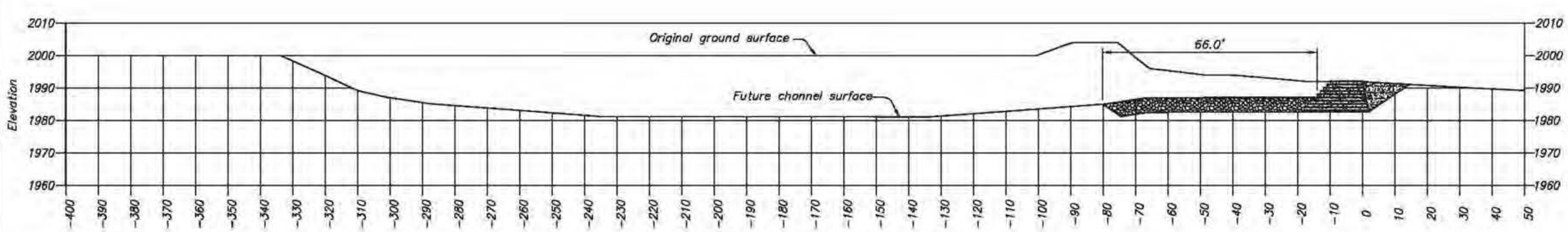

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SHEET 2 OF 3

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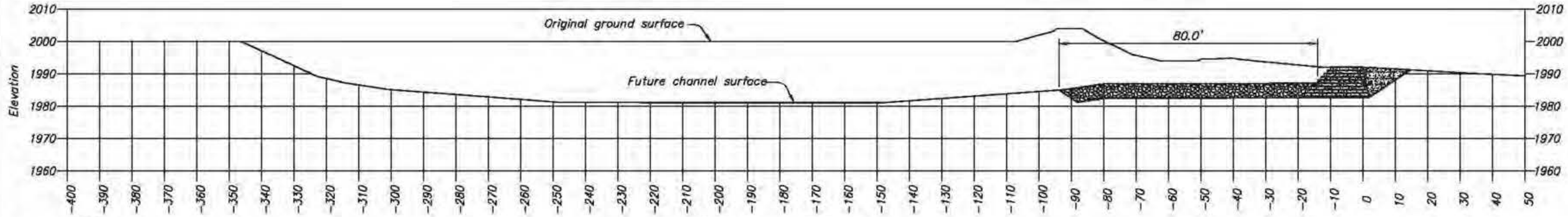

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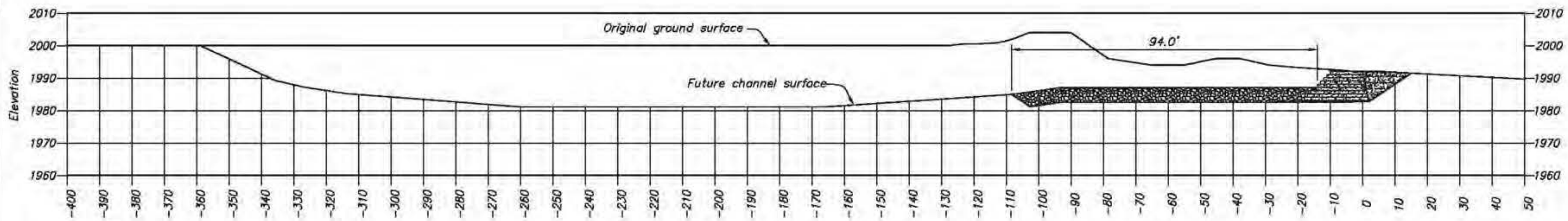

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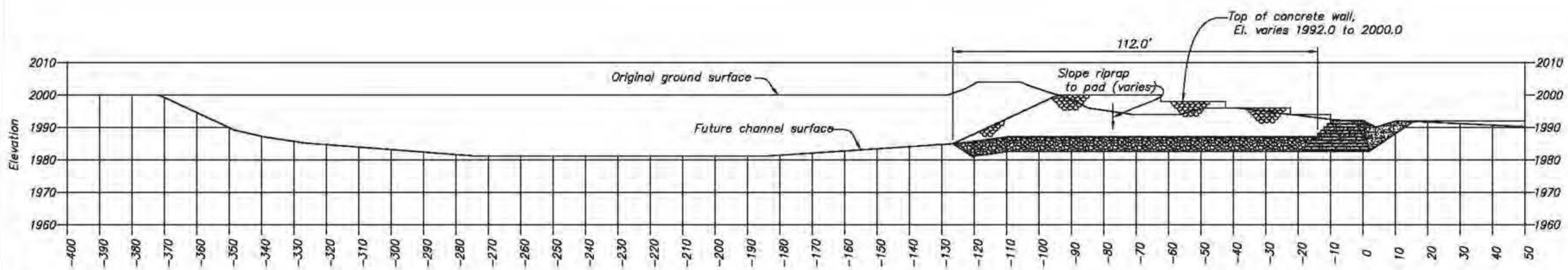

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Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Decision Document
April 2011
Updated JULY2012

Attachment 7

Concrete Structures

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Concrete Structures
April 2011

1. Introduction.

Concrete structures consist of the Dam, the Upstream Control Structure, and the Flow Augmentation Structure. The Dam is required for either the Rock Ramp scenario or the Bypass scenario to ensure adequate head at the new headworks structure to provide the irrigation district's full water rights. The Upstream Control Structure is required for the Bypass scenario only. The Flow Augmentation Structure is an option for the Bypass scenario only.

2. Dam (Rock Ramp & Bypass Scenarios)

2.1 Existing Operation and 404 Concerns.

In the past, the irrigation district has replaced rock riprap on the dam as needed to increase head at the headworks to allow the irrigation district to obtain it's full water right during the low flow months. Head requirements at the new headworks are approximately 1 ft greater than that for the old structure due to head loss through the new screens.

After implementation of a rock ramp alternative, addition of rock riprap to the dam to maintain the required head would negatively impact performance of the ramp as ice and water flow relocated rock from the dam onto the ramp necessitating the need for a concrete structure.

Continued placement of rock on the dam crest as part of implementation of a bypass channel option would necessitate construction of an access bridge or low water crossing to facilitate stockpiling of rock adjacent to the dam crest. Neither structure is desirable from a fish passage standpoint and the bridge option would likely prove to be expensive. The irrigation district has also indicated that the trolley system used to place rock is nearing the end of it's functional life. In addition, Movement of rock from the crest over time could result in impacts to any attractive flow structures that may be required at the outlet of the chute or result in partial plugging and increased O&M of the bypass.

For these reasons, replacing the existing rock dam with a concrete dam is proposed. Two options for concrete dams are described below, one of which requires a cofferdam for construction. Construction of a cofferdam significantly adds to the cost of the structure.

2.2 Dam Constructed Behind a Coffer Dam.

A concrete dam as shown in Figure 1 requires use of a cofferdam to dewater the site. The front face is sloped 1 vertical to 3 horizontal to facilitate ice to pass over the dam. The large size of the dam, and the weight required for stability make cast-in-place concrete more feasible than precast concrete, although precast hollow sections that are filled with concrete after placement may be possible. Floating in precast sections and sinking them may not be feasible because the tolerance limits for top elevation require placement on a firm, level foundation. Risks associated with construction of a concrete dam are delays due to overtopping of the cofferdam during

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construction, ice damage to the cofferdam, and undetected areas of soft subgrade. Long term risks are settlement due to soft subgrade, or displacement due to ice forces much larger than predicted.

2.2.1 Coffer Dam for Construction of Dam (Rock Ramp Scenario)

The cofferdam used for construction of the existing dam consisted of timber sheet piles with timber bracing. Because that cofferdam was placed before the dam and rock, it was not as high as a cofferdam would need to be for construction of a new dam, and in fact construction was delayed at least once due to overtopping of the cofferdam. A timber cofferdam is not practical for the new cofferdam. Several options were evaluated for a cofferdam:

- Option 1 - A coffer dam and bypass using the existing chute with a levee to provide flow conveyance
- Option 2 - A coffer dam plus a short excavated bypass channel with a levee
- Option 3 - A coffer dam with sheet pile separation in channel to dewater half of the channel at a time

The cofferdam is required to remain in place during winter. A sheet pile cofferdam cannot resist ice floes in the river. Therefore, an earthen (various size stone/gravel) cofferdam with sheet pile cutoff wall is the most practical type of cofferdam. If a temporary bypass is excavated, cofferdams could be used upstream and downstream of the dam for construction in one season. Or, cofferdams could be used to dewater one-half of the river at a time. Soil borings do not indicate boulders at the proposed dam site upstream from the existing dam. Rock downstream from the existing dam would have to be removed at the location of the sheet pile cutoff dividing the river to enable pile driving. There remains some risk of encountering deeper boulders that would impede driving of the sheet pile cutoff wall. Other construction risks are damage from ice, and overtopping.

2.2.2 Coffer Dam for Construction of Dam (Bypass Scenario)

The cofferdam cannot be installed until low flow after the fish migration season when in-water work is prohibited. This does not allow time for construction of the dam in one year, therefore the cofferdam is required to remain in place during winter. A sheet pile cofferdam cannot resist ice floes in the river. Therefore, an earthen (various size stone/gravel) cofferdam with sheet pile cutoff wall is the most practical type of cofferdam . If a temporary bypass is excavated, cofferdams could be used upstream and downstream of the dam for construction in one season. Or, cofferdams could be used to dewater one-half of the river at a time. Soil borings do not indicate boulders at the proposed dam site upstream from the existing dam. There remains some risk of encountering deeper boulders that would impede driving of the sheet pile cutoff wall. Other construction risks are damage from ice, and overtopping.

2.3 Dam with Integral Sheetpile:

If use of permanent steel sheet pile is acceptable, two lines of sheet pile could be placed without dewatering, thus eliminating the need for a cofferdam. Concrete would be tremied underwater in the 24 foot space between the two lines of sheet pile to form a mass of concrete capable of resisting ice forces as shown in Figure 2. Rock fill would be placed to form a sloping face upstream of the concrete block, and the rock ramp placed downstream of the dam. This option also has the risk of encountering boulders that would impede driving sheet pile. Long term risks

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are displacement from ice forces much larger than predicted. There is also concern that the Pallid Sturgeon may be electroreceptive which could adversely impact pallid passage over the structure. The sheetpile could be coated with form release agent or otherwise modified to facilitate pulling the sheetpile after the concrete has cured. If pulling the pile failed, it could be cut off a few feet below the top of the rock layer - that may make this option infeasible for use either with a rock ramp passage option. Potential impacts to passage from steel sheet pile have not been fully evaluated. It is possible that sheet pile cut off at a sufficient depth below the passage zone may allow for successful passage if if electoreceptivity is an issue.

The current plan would be to would have the sheet pile form walls spaced at 24 feet extending from the existing headworks and to the right bank. The alignment would be upstream of the existing diversion dam but as close as possible to it will allowing for the maintenance of current rock pile slope so as to avoid making passage over the existing rock field for other native fish that may utilize it worse than it already is . The construction process would involve driving the sheet pile in a paired parallel wall configuration to create a 50 feet long segment of the dam utilizing T-connection sheet pile installed at the ends with other sheets to complete the cell. The cell would not be dewatered by pumping The concrete will be placed with pumper hose below water in the cell. As the concrete fills it would displace the water and the water would flow over the sheet pile cell. The sheet pile lengths used would be 40 feet. The top elevation of the sheet pile would be approximately 4.5’ above the final crest elevation of 1990.5. After the concrete has set, the sheet piles would be pulled for reuse or cut-off below the final crest elevation. The next cell would be started and the process repeated. A triangular riprap section on the upstream side would be placed from the crest to the river bottom, this would allow for ice to slide up and over the crest. The area between the new crest and the old diversion dam would be filled with coarse material and capped with riprap layer. The sequencing of the cells in 50-foot lengths would allow for river flows to pass without coffering or diverting water. This work would involve both barge (where draft is available) and conventional construction equipment (at river banks) for creating ramps for access and construction platforms. Risks during construction are delays caused by floods, encountering deeper boulders that would impede driving of the sheet pile, or inability to pull sheet pile if presence of buried steel is unacceptable. Long term risks are settlement due to soft subgrade, or displacement due to ice forces much larger than predicted.

2.4 Dam VE Alternative (not recommended at this time)

An alternative to construct a precast concrete dam placed on driven steel piles as shown in Figure 3 is being evaluated. At this time the feasibility of this alternative is not 57
certain because it is not know if the piles or the precast to pile connection would have the capacity to resist the force of ice floes in the river. Other risks associated with this alternative are encountering boulders that would impede driving steel pile. Long term risks are displacement from ice forces much larger than predicted.

3. Upstream Control Structure Concrete Slab (Bypass Scenario)

The Upstream Control Structure requires a concrete slab 30 feet long (upstream to downstream) to provide stable grade control that will not be moved by ice floes. A 6-foot thick concrete slab is required for stability against the expected ice forces. The slab can be constructed of roller compacted concrete place in a dewatered excavation as shown in Figure 4. Risks associated with this construction method are increased dewatering cost due to groundwater seepage much larger than predicted,. Long term risks are displacement due to ice forces much larger than predicted.

See Attachment 6 *Bypass Channel*, Appendix C-*30% Design Features* for details pertaining to the upstream control structure, including the concrete slab and associated riprap tieback features."

4. Flow Augmentation Structure (Bypass Scenario)

The Flow Augmentation Structure is an option that could be constructed near in downstream end of the bypass to provide additional attraction flow. The structure can be constructed of roller compacted concrete place in a dewatered excavation as shown in Figure 5. Risks associated with this construction method are increased dewatering cost due to groundwater seepage much larger than predicted. Long term risks are settlement due to soft subgrade, or displacement due to ice forces much larger than predicted.

5. Ice Forces

The ice loads used for structural design are as recommended by the Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL) as summarized in Table 1.

Table 1. Summary of Ice Loads on Structures

			Whichever produces the greater force	
Structure	Material	Reference	Impact acting as horizontal line load	Shear acting as horizontal friction on top surface. (Need not exceed the impact line load.)
Existng Dam Crest	Riprap	a. Para 6.4	15 klf	2 ksf
Upstream Control Structure	Concrete	b. Para 6.a.	10 klf	NA
Channel Plug	Riprap	b. Para 6.b.	NA	NA
Armoring at Channel Bends	Riprap	b. Para 6.c.	NA	NA
Vertical Control Structures in Bypass Channel and Outlet	Riprap	b. Para 6.d.	NA	NA
Downstream Lateral Stability Structure	Riprap	b. Para 6.f.	NA	NA
New Dam Crest	Concrete	b. Para 6.g.	15 klf	2 ksf
Flow Augmentation Structure	Concrete	b. Para 6.h.	10 klf	2 ksf

References

a. Ice Forces on Intake Dam, Lower Yellowstone River: 30 Percent Design, Andrew M. Tuthill, CRREL

b. Evaluation of Ice Impacts on Fish Bypass Channel at Intake Dam, Lower Yellowstone River, Andrew M. Tuthill, Meredith L. Carr, CRREL, Feb 12, 2012

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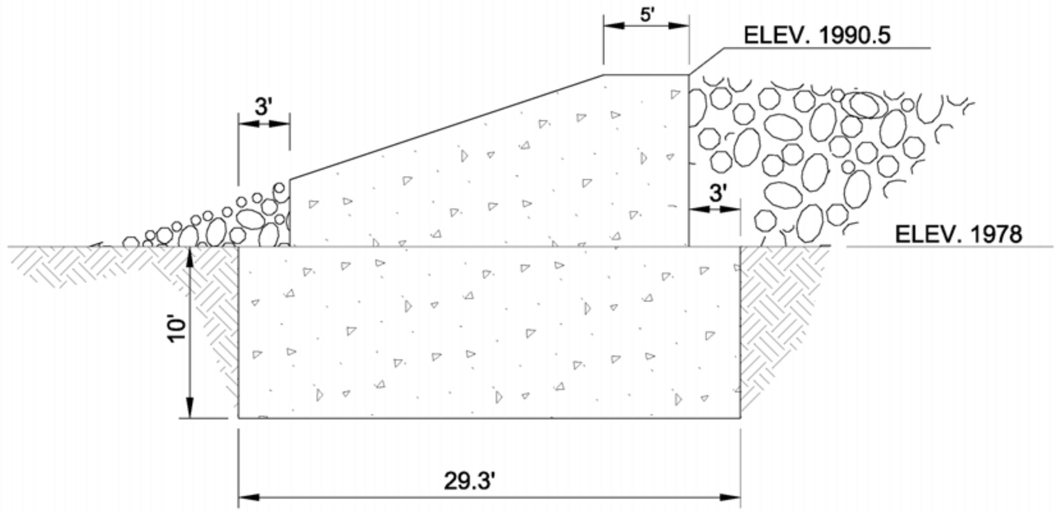

Figure 1 Cast-in-Place Reinforced Concrete Dam

(Reinforcement not shown, a cofferdam is required for construction)

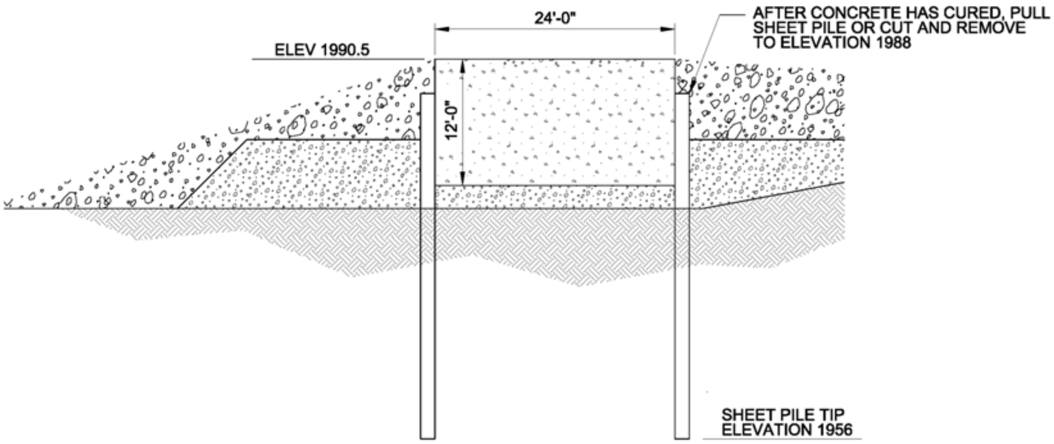

Figure 2 Concrete Dam
(Concrete Placed Underwater without Cofferdam)

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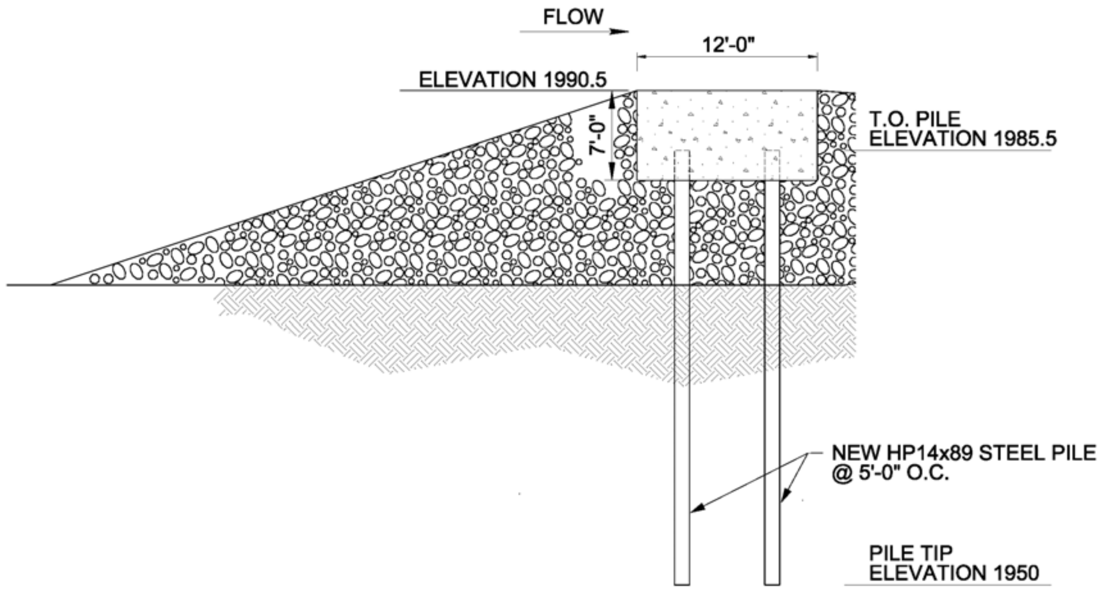

Figure 3 Precast Dam on Steel Piles
(VE Alternative not yet fully evaluated)

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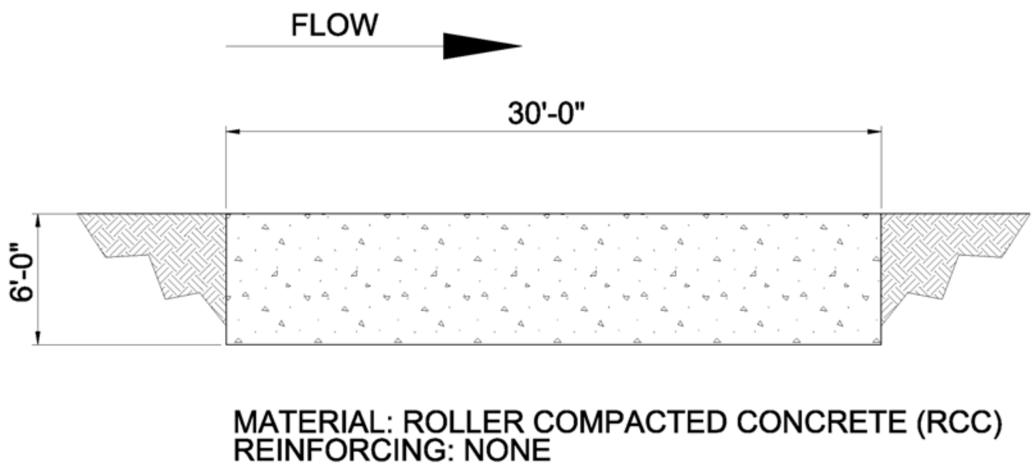

Figure 4 Slab at Upstream Control Structure

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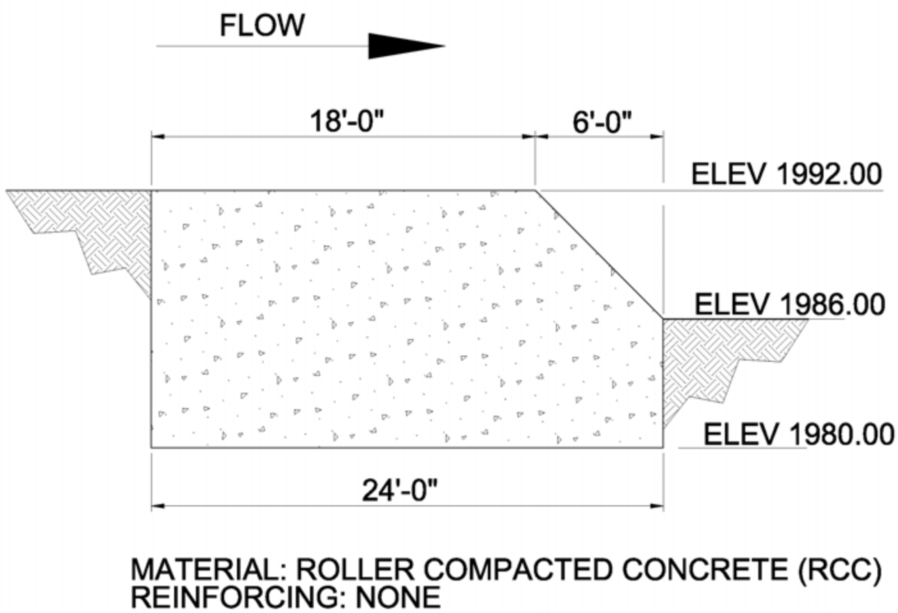

Figure 5 Flow Augmentation Structure

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Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Intake Fish Bypass Option Evaluation Summary
May 2012

Attachment 8

Material Management and Logistics Risks and Uncertainties

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Material Management and Logistics Risks and Uncertainties
May 2012

Introduction

A brief review was performed to identify and evaluate the significant risks and uncertainties associated with material management and logistics for both the ramp and chute options.

Rock Source.

The rock ramp alternative cost estimate developed for ATR certification and submission to the ASA(CW) was based on the assumption that stone from a nearby quarry would be used for construction of the project. This stone has been used historically for regular maintenance of the existing dam crest height following degradation of the crest by ice effects. Given the size requirements for the rip rap in the ramp design it was determined that the local rock would not be suitable due to availability, identification of fracturing of in-place rock on the ramp, and assumptions about likely durability made prior to testing of the stone for compliance with COE specifications.

The local stone source was inspected 6 April 2011 and samples were collected for submission for geotechnical analysis. The land is privately owned and rock used for the irrigation weir is collected from stone which has fallen naturally from the cliff face. The property owner has indicated a willingness to discuss operation of a quarry on that property to the COE construction representative on the Headworks project. The stone occurs as a cap rock in a shale formation with several feet of soil overburden. The cap rock is approximately 25-30 ft above the toe of the slope with good access to the area. Haul roads and working areas would need to be prepared to facilitate sizing, sorting and hauling of rock at a scale required to make use of the quarry feasible.

The cap rock unit comprises a fine grained sand stone with distinct bedding. Bedding failure and cross bedding fractures appear to control the maximum size of the boulders available. Inspection of the slope at the toe of the unit reveals an average maximum rock size of approximately 2 ft in diameter. Reclamation has selectively harvested larger rock up to 3 ft in diameter for use on the existing dam.

The sandstone contains a mixture of quartz, feldspar, amphibole and some mica. There is evidence of significant mineral alteration in hand samples and along bedding surfaces. There appears to be at least moderate porosity/permeability in the exposed hand samples which could contribute to adverse freeze thaw response. The exposed rock surface shows significant fracturing which would preclude obtaining the larger rock sizes. Some of the larger stones stockpiled near the tramway by Reclamation show low permeability and more competence suggesting that active quarrying away from the weathering surface could encounter better rock. Large rock placed along the river banks downstream from the dam show some sign of breakage.

This local stone, has recently undergone ASTM testing for durability and freeze thaw characteristics and, for the size of rock naturally accumulating at the base of the outcrop, was found to meet USACE specifications for rip rap. Given the prevalence of vertical fractures and bedding plane fractures seen in the outcrop, and the evidence of breakage seen in the larger stones placed in the existing crest structure, the assumption that the quarry local source would not be feasible for material for a rock ramp is probably still valid. Testing was performed on a very

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limited sample and additional investigation would be required to confirm the viability of this outcrop as a rip-rap quarry.

In addition to questionable rock quality for larger rocks, the limited available volume and limited maximum rock size may preclude use of the local stone for major components of the ramp. Use of smaller stone gradations could be justifiable for toe protection in the outer bends of a bypass chute option or in scour holes if use of the rock was also supported by the economics of quarry development. If full quarry development is not found to be cost effective, collection of naturally fallen rock would still likely provide some quantity of material at a lower cost than would be the case with imported rock. High quality stone is required for the inlet and outlet grade controls on the bypass chute as well as for the plug between the bypass chute and the existing high flow channel. If a local quarry is developed and if local rock quantity limitations are identified which require importation of additional rock the imported rock would be utilized for the more critical structures. Local rock should be used for all O&M and repair of any structures.

Prior to confirming the potential for utilization of the local rock additional investigation would be required into quarry permitting requirements for the site. Negotiations with the landowner as to compensation and site restoration requirements would need to be undertaken prior to determining actual cost for the rock. Additional NEPA investigations into potential cultural resource impacts in the area of the potential quarry will be required and operation and restoration plans will need to be developed.

Bypass Channel Excavation Spoils

Excavation of a bypass channel capable of passing 15% of the Yellowstone River flow would result in approximately 1.2M cubic yards of spoils. Test pits excavated in the proposed bypass alignment during the fall of 2011 revealed that the stratigraphy comprises an upper zone of 3-8 ft of silts to fine sand in lens and bedded geometries. This fine material overlays a deeper unit of rounded, often platy, river cobble with a maximum diameter of approximately 4 inches. The matrix varies from silt to well graded sand and gravel. The planned total depth of the test pits was encountered only in one excavation due to sideslope failure in all of the others.

Some of the cobbles could be screened on site to generate an engineered armor layer for the bottom of the channel, some could help fill the area between the old and new weirs and some could potentially be used as a concrete aggregate if an on-site batch plant was deemed cost effective. All other material will need to be disposed of. A preliminary call to Fisher Sand and Gravel in Glendive Mt revealed that if imported rip rap is required and obtained through Fisher they would be willing to backhaul some of the gravel and cobble as a marketable material at no charge with the potential open to offset some of the rip rap haul cost. This arrangement, provided trucks were direct loaded at the excavation, would reduce the cost of rip rap by the amount it would take to move and place the material onsite. The aggregate would require additional testing to determine suitability.

A potential spoil area has been identified immediately to the west of the entrance to Joes Island off the pave surface road along the bluff. Recent investigation into ownership of the property shows it to be held by BOR. All generated spoils could be placed in that area leading to approximately 12 ft of fill placed over 60 acres though there are concerns with potential infilling of existing drainage ways that may require specific traffic and fill plans designed to avoid those areas.

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Material Transport Logistics.

Use of the local rock source would facilitate direct transportation from the quarry to the construction site from the south of the construction site. The 30% design cost estimate is built on use of imported stone obtained from quarries in Montana and Wyoming to use the most conservative and certain sources. Use of imported stone would require transportation by rail or truck to Glendive Mt and then by truck to the site. The riprap for the 15% flow diversion channel structures and toe protection would required approximately 50,000 tons of rock plus bedding material. Pending complete feasibility analysis of quarrying local stone it must be assumed that imported rock will be required for a majority of the rock requirement

Railroad Siding Concerns.

To facilitate railroad transport of rock for a ramp alternative an eight to ten car rail siding exists on the north side of the project site which potentially could be expanded depending on site ownership and available space. The limits of expansion and the costs for expansion have not been captured at this time. Based on communications with construction personnel who have worked with offloading rock from rail cars approximately 30 cars could be unloaded in a work day dependent on site conditions. The limitations on the rail siding appears to prevent efficient use of unloading equipment at the site unless a way was found to provide a continuous supply of rail cars. Preliminary communication with rail representatives suggests that trains would be expected to be cycled through the area on approximately5 day intervals. This turnaround time could possible improve once negotiations for a contract were actually initiated. Use of this siding is only feasible for the rock ramp option Provision of riprap for a bypass channel would necessitate trucking the rock to Joe’s Island either from a railroad siding in Glendive or direct from the quarry.

Rock Delivery and Staging for Construction.

Rock Ramp

Several options were evaluated for delivery of rock and possible rock staging methods at the construction site.

Due to the need to work in non-flow conditions behind a partial cofferdam in the wet construction option, or to the presence of a center channel sheet pile wall in the half channel diversion method, delivery of rock to the construction area from one side of the site only is only feasible with total stream diversion and dry construction. To facilitate placement from both sides under the first two construction options requires material to be either trucked from the site siding or the quarry to the opposite side of the channel, trucked from the siding at Glendive (if available), or trucked directly from a quarry.

Trucking material into the south side of the project will require traversing existing unimproved roads across “Joes Island”. Use of unimproved haul roads by trucks, especially over the road trucks will require significant upgrading and regular maintenance of the road as well as dust control for visibility and safety concerns. Dust control and maintenance could be a significant cost potentially not fully captured in the cost estimate as it is weather dependent to a degree.

If over the road haul trucks are used for any component of rock transportation for ramp construction it is likely that multiple handling of the rock will become necessary as it is unlikely that construction of haul roads sufficient to allow those kinds of trucks to transverse the ramp would be possible except maybe for dry construction. Multiple handling of rock could have adverse impacts on gradations as well as add cost.

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Fish Bypass Channel

For construction of rock structures in a bypass channel option haul roads created to support the channel excavation would be utilized to truck rock to the structure locations. It is anticipated that the roads can be maintained sufficiently to allow for direct haul to the area of placement without needing to double handle rock. Haul road maintenance to allow over the road truck traffic would be at a higher degree than would be required for off road trucks. To reduce maintenance costs over time it may be cost effective to stage construction so that rock placement occurs during focused time windows. The county road accessing the site from the highway is also a gravel surface road which would require maintenance and potentially some post project restoration to address damage from the relatively large number of haul trucks importing stone.

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Intake Diversion Dam Modification
Lower Yellowstone Project, Montana

Intake Fish Bypass Option Evaluation Summary
May 2012

Attachment 9

Real Estate

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Real Estate
May 2012

1. Introduction

Real estate property acquisition and/or easements will be required for any construction features on the south bank of the Lower Yellowstone River or on Joe's Island. The Bureau of Reclamation is responsible for the real estate actions for this project to move into the construction phase.

2. Property Ownerships

A search of the Montana Cadastral Survey resulted in four known ownerships and one unknown ownership in the Joe's Island footprint. The ownerships are the Burlington Northern Santa Fe Railway (BNSF RR), the Bureau of Land Management (BLM), the State of Montana, Gentry Land & Livestock Inc., and a potential unknown owner of the Turtle Island property on the south bank of the river (see Figure 1.). The Bureau of Reclamation or the irrigation district may have maintenance easements on some of these properties, but construction acquisition is a certainty with some of the project components analyzed.

3. Impacted Properties

The bypass of water, and the Bypass Channel for fish components, will require some type of real estate action on the Bureau of Land Management and State of Montana ownerships and a possible unknown owner of Turtle Island. A small portion of the BNSF RR property will be affected by the bypass of water by channel option. Construction access from the south for the rock ramp options will be on the State of Montana and BLM properties, and possibly the BNSF RR property. It is likely that all of these properties may have leases to farmers and ranchers for cattle grazing that will be affected.

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Figure 1

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