OLSSON ASSOCIATES

PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

CNPPID Reregulating Reservoir Elwood and J-2 Alternatives Analysis Project Report

BLACK & VEATCH

PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

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EXECUTIVE SUMMARY

Alternative Goals:

The Platte River Recovery Implementation Program (PRRIP) has identified the need to create Short Duration High Flows (SDHF) to aid in the restoration of critical habitat in the Platte River of four endangered species; the interior least tern (*Sternula antillarum*), piping plover (*Charadrius melodus*), whooping crane (*Grus americana*), and pallid sturgeon (*Scaphirhynchus albus*) within the Platte River corridor. The Water Management Study (WMS) concluded that capacity constraints in the Platte River, and in the irrigation/hydropower districts' systems, prohibit a SDHF of the magnitude desired without additional new infrastructure.

The proposed use of the Elwood Reservoir, and/or a new J-2 reregulating reservoir, by the PRRIP may potentially aid in the development of a SDHF event and help reduce shortages to target flows in the Platte River (WMS Phase II, 2008) as a secondary benefit. Olsson Associates was contracted to analyze and screen alternatives for the potential development and operation of Elwood Reservoir and/or a J-2 reregulating reservoir to provide SDHF and reduce shortages to PRRIP target flows.

The primary goal in evaluating reservoirs was to augment flows for three days towards a total SDHF of 6,000 to 8,000 cfs as measured at the Overton gage (Platte River Recovery Program Implementation Program (PRRIP) Document (PRRIP), 2006 and Adaptive Management Plan (AMP), 2006). For purposes of meeting a SDHF it was assumed that reservoirs could be filled with either Environmental Account (EA) water released from Lake McConaughy and excesses to target flows (excess flows), if available, using CNPPID's full system capacity.

Each alternative was also evaluated for its ability to reduce shortages to target flows by storing excesses to target flows, and then make releases during times of shortages. This evaluation was done for illustrative wet, normal, and dry years. The results of this evaluation, as well as the SDHF evaluation, are shown in Table ES-1.

Potential hydropower flow cycling mitigation benefits were also evaluated for the potentialJ-2 reservoir sites. Because of its location, Elwood Reservoir cannot effectively mitigate hydropower flow cycling impacts.

Evaluated Alternatives:

A preliminary review of possible alternatives for the use of Elwood Reservoir, and potential J-2 reservoir sites for PRRIP use resulted in the identification and scoring[1] of the following alternatives as authorized by PRRIP:

J-2 Return Reservoir Alternatives (see section 4 and Appendix B for more information):

- J-2 Alt 1 – A series of four new dams in the South Channel of the Platte River below Central Nebraska Public Power and Irrigation District's (CNPPID) J-2 Return Canal (J-2 Return), upstream of and above Overton.
- J-2 Alt 2, Areas 1, 2, 3, and 4 – New, excavated reservoirs between the south bank of the South Channel of the Platte River and the CNPPID Phelps canal. Four separate potential reservoir areas and a combination of two reservoirs were investigated within this option.
- J-2 Alt 2, Areas 1 & 2 – J-2 alternative combination including both Areas 1 and 2.
- J-2 Alt 3 – Construction of an embankment across an unnamed creek immediately upstream of CNPPID Phelps Canal siphon (Phelps 9.7 reservoir).

[1] The "scoring" of alternatives in this study should not be confused with official PRRIP score that will be assigned to Water Action Plan (WAP) projects. Scoring in this study is a ranking solely to compare alternatives with the purpose of selecting alternatives for advancement. As described in Section 2 "Development of Alternatives Analysis Criteria", alternatives were first "scored" and then weighted to develop the final "score" or alternative rank.

Elwood Reservoir Alternatives

Alternatives E-1 through E-6 all would rely on Plum Creek, upgraded and armored, to convey flow to the Platte River.

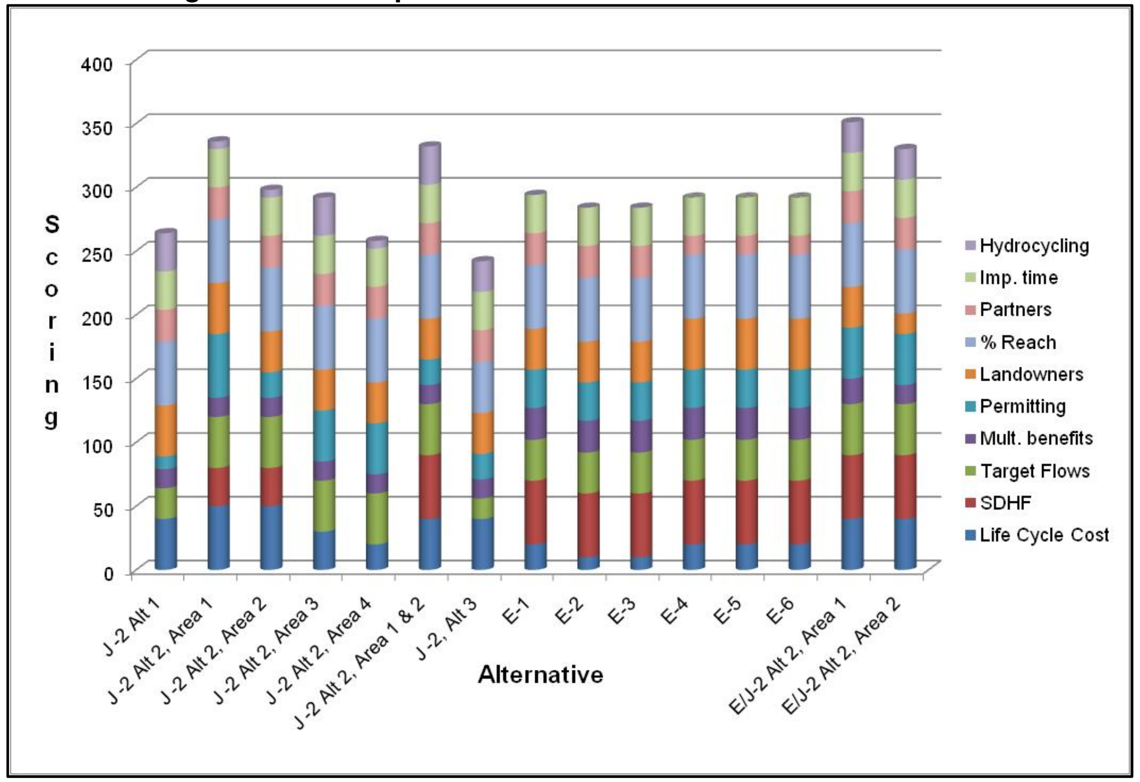
- E-1 – Elwood Reservoir, stabilized with a buttress, fed through a new gravity canal, using tunnels for the outlet.
- E-2 – Elwood Reservoir embankments removed and replaced, fed through a new gravity canal, using a new outlet in the embankment.
- E-3 – Elwood Reservoir embankments removed and replaced, upstream only, fed through a new gravity canal, using a tunnel outlet.
- E-4 – Elwood Reservoir embankment stabilized with a buttress, fed through the existing E-65 Canal, using tunnels for the outlet.
- E-5 – Elwood Reservoir embankments removed and replaced, fed through the existing E-65 Canal, using a new outlet in the embankment.
- E-6 – Elwood Reservoir embankments removed and replaced, upstream only, fed through the existing E-65 Canal, using a tunnel outlet.

Combination Alternatives

- E/J-2 Alt 2, Area 1 – Combined use of Elwood and J-2 (Alternative 2, Area 1) reservoirs. Elwood and Plum Creek modified to provide only 1,000 cfs of SDHF augmentation flow over three days.
- E/J-2 Alt 2, Area 2 – Combined use of Elwood and J-2 (Alternative 2, Area 2) reservoirs. Elwood and Plum Creek was modified to provide only 1,000 cfs of SDHF augmentation flow over three days.
  Table ES-1 summarizes the results from the analysis.

After screening and scoring, the results of the alternative analyses are shown in the chart below. The chart is a stacked chart showing the scoring for each of the scoring criteria. (See Screening and Scoring in Section 6).

Figure ES-1 Composite Score for all Evaluated Alternatives



Based on these results, it is recommended the J-2 Alternative 2, Areas 1 and/or 2 be advanced to feasibility stage of analysis. In addition, Elwood Reservoir appears to have an attractive use when used at a low release rate into Plum Creek. Although not a specific goal or objective of this study, modeling of reductions to shortages to target flows indicates Elwood Reservoir is typically at minimum stage over the winter months, which is also when the reliability of excess flows are high. More analysis is needed, but it appears using Elwood Reservoir to store winter excess flows would not interfere with CNPPID current use. A low release rate into Plum Creek of around 100 to 500 cfs would minimize Plum Creek stabilization costs and minimize roadway crossing upgrades. With a potential high volume yield and minimal capital costs, this alternative should be further investigated.

It became clear during the analysis and investigation that the J-2 Alternative 2 location is the preferred location for a reservoir to augment the SDHF, with the combination of areas 1 and 2 scoring the highest for the alternative. As the scoring has also pointed out, the option of using Elwood to reduce shortages to target flows, in conjunction with the J-2 Alternative 2 reservoir, is advantageous and should be included going forward.

Table ES-1 Reregulating Reservoirs Alternative Analysis Summary

Alternative	Reservoir[1]	Storage ac-ft	Inlet	Outlet	Conveyance to Platte River	Capital Costs[2] ($000)	1-yr Operating Costs ($000)	SDHF Augmentation[3] ac-ft / yr	Reductions to Shortages to Target Flows[4],[6]		
									Wet Yr ac-ft	Normal Yr ac-ft	Dry Yr ac-ft
J -2 Alt 1	J-2 south channel option	3,380	J-2 Return	Radial Gates	n/a	$17,460	$218	1,825	19,715	14,660	12,357
J -2 Alt 2, Area 1	J-2 excavation Area 1	9,716	Phelps Canal	Radial Gates	n/a	$24,206	$182	8,860	44,119	33,668	25,029
J -2 Alt 2, Area 2	J-2 excavation Area 2	6,580	Phelps Canal	Radial Gates	n/a	$17,483	$152	6,580	33,677	24,974	18,757
J -2 Alt 2, Area 3	J-2 excavation Area 3	4,516	J-2 Return	Radial Gates	n/a	$40,541	$331	4,516	25,952	20,341	16,331
J -2 Alt 2, Area 4	J-2 excavation Area 4	6,137	J-2 Return	Radial Gates	n/a	$83,877	$681	5,387	32,139	24,268	18,508
J -2 Alt 2, Areas 1 & 2[5]	J-2 excavation Areas 1&2	14,320	Phelps Canal	Radial Gates	n/a	$40,039	$321	11,901	57,931	47,480	34,237
J -2, Alt 3	Phelps 9.7 reservoir	1,659	Phelps Canal	Sluice Gates	Unnamed creek	$6,059	$106	1,659	10,569	8,298	7,078
E-1	Elwood, buttress	26,899	Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$42,942	$690	11,901	21,736	19,408	19,154
E-2	Elwood, remove & replace embankment	26,899	Gravity Canal	New Outlet (2 pipes)	Plum Creek, 2,400 cfs	$45,444	$721	11,901	21,736	19,408	19,154
E-3	Elwood, remove & replace upstream shell	26,899	Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$45,522	$722	11,901	21,736	19,408	19,154
E-4	Elwood, buttress	26,899	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$36,677	$449	11,901	21,330	17,788	19,162
E-5	Elwood, remove & replace embankment	26,899	Existing E-65 Canal	New Outlet (2 pipes)	Plum Creek, 2,400 cfs	$39,179	$468	11,901	21,330	17,788	19,162
E-6	Elwood, remove & replace upstream shell	26,899	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$39,257	$459	11,901	21,330	17,788	19,162
E/J-2 Alt 2, Area 1	Elwood, buttress & J-2 excavation, Area 1 modified	36,615	Existing E-65 and J-2 Return	Tunnel (1 only)	Plum Creek, 1,200 cfs	$51,626	$457	11,901	44,119	33,668	25,029
E/J-2 Alt 2, Area 2	Elwood, buttress & J-2 excavation, Area 2	33,479	Existing E-65 and J-2 Return	Tunnel (1 only)	Plum Creek, 1,200 cfs	$46,861	$422	11,901	33,677	24,974	18,757

Notes:
(1) Base cost of reservoir (total estimated project cost without inlet, outlet, and conveyance costs). For Elwood, the cost represents improvements to the embankment.
(2) Total estimated project cost including base reservoir cost, inlet, outlet, and conveyance costs
(3) Water to augment SDHF could be either environmental account (EA) water routed down from Lake McConaughy, and staged in the reservoir, or stored excess flows captured and stored in reservoirs immediately before a SDHF if available. Though units are ac-ft/yr, the values presented are the total volume of SDHF augmentation flows provided by the alternative over three days.
(4) Water to reduce shortages to target flows is excess flows in CNPPID's system that could between stored during times of excess, and released during periods of shortage.
(5) Assumes only gravity fill for Areas 1 and 2.
(6) Assumes J-2 storage site(s) are full at beginning of water year (October) for consistency of scoring all alternatives. Results shown are for the illustrative years only. Long-term yield averages will vary.

INTRODUCTION

The primary goal of the Platte River Restoration Implementation Program (PRRIP or Program) is to support the recovery of four threatened or endangered species: the interior least tern (*Sternula antillarum*), piping plover (*Charadrius melodus*), whooping crane (*Grus americana*), and pallid sturgeon (*Scaphirhynchus albus*) within the Platte River corridor. Several studies and documents have been completed that discuss various methods and options to support the recovery (Water Action Plan (WAP), 2000).

The PRRIP Water Advisory Committee (WAC) has compiled previous studies and directed the production of Water Management Study (WMS) Phase I and Phase II reports for the evaluation of augmenting short duration high flows (SDHF) and target flows. The goal of SDHF events (resulting in 6,000 to 8,000 cfs in the habitat reach for a duration of three days on an annual or near-annual basis) are to create a wider, shallow, braided river channel with seasonal sand bars for habitat recovery (PRRIP 2006 and Adaptive Management Plan AMP, 2006). The Phase I report (WMS Phase I, 2008) concluded that additional storage is needed near the associated habitat to help achieve SDHF objectives. The Phase I report also evaluated 13 projects identified in the Water Action Plan (WAP) for their potential contribution to the PRRIP flow targets (Table 1.5-1). Under target flow operations, flows in excess of PRRIP target flows (excess flows) are stored and then released when flows are below the target flows (shortage). Each WAP project was evaluated for the ability to reduce shortages to target flows. These 13 projects are depicted in Figure 3-1 of the Phase II Report (WMS Phase II, 2008). These three figures are also included in Appendix A of this report for reference purposes. The Phase I report found that in order to be effective in meeting SDHF goals, the water from these projects will need to be managed either in Lake McConaughy, or with other modified, existing, or new storage facilities near the associated habitats.

The WMS Phase II Report screened and evaluated three project concepts, including: re-operation of the existing Elwood Reservoir, creation of a Plum Creek Reservoir, and creation of reregulating reservoirs. The recommendations of the Phase II Report are as follows:

Recommendations from Phase II of the WMS (note the WMS used the term “pulse flow” rather than “SDHF”):

- Review effects of recent high flows on channel morphology and maintenance and determine what conclusions can be extrapolated to the potential for managed pulse flows to accomplish the desired effects in the Platte River corridor. The results of these analyses should be considered in future refinements to the reservoir feasibility and implementation in relation to the timing and location of the reservoir pulse flow releases.
- Perform a pilot pulse flow in the spring of 2009. Results from this pilot study will provide additional insight into channel maintenance, capacities, and flow attenuation.
- Re-regulation of Johnson Lake will be a key component of a full pulse flow. CNPPID agreed to test re-regulation of 6,000 ac-ft. If results of a test operation are positive, this volume of re-regulation might be increased. In addition to the physical results of a test, the process would involve re-positioning PRRIP water and making releases timed to supplement natural events, and will help define procedures for annual implementation.
- Further investigate: 1) next steps to achieve the 2011 pulse flow goals, 2) benefits of J-2 Return Reservoir on hydro-cycling and 3) procedures for implementing a test release from Johnson Lake.
- Define the 2009 Program activities related to the WMS Flow objectives:
    - Additional operations modeling of individual and combined projects.
    - Select preliminary design activities for specific storage sites, including:
        - Development of field exploration program(s) and data collection
        - Refinement of project facility types and capacities
        - Refinement and development of project cost ranges based on feasibility-level design.
    - Review the need to update assessments of previously defined WAP alternatives and/or quantification of availability of flows in excess to targets
    - Review the need to enhance the existing Routing Tool and Loss Model to potentially include:
        - Multiple-year operations for the Routing Tool
        - Ability to evaluate multiple targets in combination
        - Multiple project operations
        - Link daily time step Routing Tool with monthly time step Loss Model
        - Enhanced user output
- Investigate the reaction of land owners in the project areas to participate in the development of a project.
- Continue work on expanding the safe-conveyance capacity of the North Platte River at North Platte (choke point) and other channel restrictions that may be identified in the future. Continued improvements to restore a capacity of 3,000 cfs or greater at North Platte are important in achieving flow targets, minimizing the need and size of additional structural solutions to the extent possible.

The WMS Phase II study recommended storage near the associated habitat. Although a somewhat similar storage project had been included in the Reconnaissance-Level WAP, the design of a storage facility utilized to augment a SDHF would require considerably larger storage and outlet works capacity. The need to augment SDHFs in support of the Adaptive Management Plan experiments elevated the priority for investigating feasibility of these concepts above other potential WAP projects. The WMS Phase II Report also took the need for hydropower flow cycling mitigation into consideration based on CNPPID FERC license updates (Federal Energy Regulatory Commission (FERC), 2007).

Based upon the WMS Phase II, the WAC recommended two potential concepts from the Phase II Report be investigated further. These two concepts are: a re-operation of the existing Elwood Reservoir, and/or creation of a J-2 reregulating reservoir. Olsson Associates was selected in July of 2009 to analyze these concepts for the augmentation of SDHFs, target flows, and hydropower flow cycling to the Platte River. The goal of this analysis was to develop and evaluate CNPPID reregulating reservoir alternatives for the existing Elwood Reservoir and potential new reservoirs in the vicinity of CNPPID’s J-2 Return. Olsson was tasked with:

- Developing alternative locations that would best meet project objectives
- Interpreting the existing uses of CNPPID system components for use in analysis and identifying alternative-specific improvements to existing system components
- Calculating storage of the proposed alternatives
- Examining and summarizing technical and construction considerations
- Performing preliminary sizing of inlets and outlets for alternatives
- Developing estimates of probable costs
- Working with the ED Office and project workgroup to develop a method to score alternatives
- Working with the ED Office and project workgroup to develop procedures for evaluating each alternative’s ability to augment SDHFs and reduce shortages to target flows
- Developing and interpreting SDHFs, target flow operations, and hydropower flow cycling models for each alternative

Priority was placed on the alternatives being able to augment SDHFs. Separate analyses evaluated the ability of all alternatives to reduce shortages to target flows and mitigate hydropower flow cycling, using reservoir designs determined in SDHF analysis. For SDHF augmentation, water from the Environmental Account (EA) is routed from Lake McConaughy and stored in the reregulating reservoir immediately prior to a SDHF. Additionally for SDHF augmentation and target flow operations, excess flows that CNPPID has diverted from the Platte River and routed through their hydropower stations can be delivered to the reregulating reservoir, rather than released immediately back to the Platte River. Reregulating reservoirs may then release water when needed either to augment SDHFs or to reduce shortages to target flows. Excess flows routed to Elwood must bypass the Johnson Lake Hydropower Station 2 (J-2 hydro) so there are power bypass costs to the Program associated with any excess flows stored in Elwood. J-2 reservoir storage areas would be located below the J-2 hydro and adjacent to the J-2 Return Canal near the Platte River. As a result there are no power bypass costs associated with J-2 alternatives.

1.1 Elwood Reservoir Background

Elwood Reservoir (Elwood) is an existing reservoir located in north central Gosper County, Nebraska. This reservoir is depicted in Figure 4-2 of the WMS Phase II Report, which is included in Appendix A of this report. It is owned and operated by CNPPID. Elwood Reservoir is supplied water via the E-65 Canal, which diverts water from the CNPPID Supply Canal upstream of Johnson Lake. Water is stored in Elwood Reservoir prior to the irrigation season for use during the irrigation season. E-65 siphons, located upstream of Elwood Reservoir, do not have capacity to allow full irrigation deliveries, and Elwood is used to supplement the flow during peak irrigation season. Pumps are required to fill Elwood Reservoir. The reservoir is operated based on a target operating curve (TOC), which is discussed later. The available active storage between the minimum and maximum elevations of the TOC, commonly called the “beneficial use pool”, is approximately 26,900 ac-ft.

Elwood Reservoir was investigated for feasibility in storing and delivering flow to augment the SDHF defined in the operating criteria as discussed in Section 1.4. As a separate analysis, Elwood was also evaluated for the ability to reduce shortages to target flow, which is discussed in detail in Section 1.5. All Elwood alternatives assumed that Program use of Elwood would be outside of the irrigation season and the time period when CNPPID needs the E-65 Canal for operations. Elwood and the E-65 canal were assumed to be available from September 1 through March 7 or 15, at which time CNPPID starts to fill Elwood. As a result, the TOC was not negatively affected. As discussed in detail in Section 3.6.2, the analysis allowed available excess water to accumulate within this time period above and beyond the TOC, but only the releases prior to the start of the irrigation season were included in the shortage reduction volumes. The potential alternatives using the Elwood reservoir for this purpose involve modifications or revisions to the major components consisting of the dam, emergency spillway, outlet works, upstream siphons, inlet, and the outlet channel/conveyance system to the Platte River. After initial analyses were conducted, six alternatives for modifying Elwood Reservoir advanced to a more detailed preliminary investigation. All of the alternatives involved enlarging and armoring the Plum Creek channel to convey water to the Platte River. The alternatives were as follows:

- Alternative E-1 – Elwood Reservoir, stabilized with a buttress, fed through a new gravity canal, using two 8-foot diameter tunnels for the outlet.
- Alternative E-2 – Elwood Reservoir embankments removed and replaced, fed through a new gravity canal, using two 8-foot diameter pipes for the outlet, constructed using an open excavation during replacement of the embankments.
- Alternative E-3 – Elwood Reservoir embankment upstream shell removed and replaced, fed through a new gravity Canal, using two 8-foot diameter tunnels for the outlet.
- Alternative E-4 – Elwood Reservoir embankment stabilized with a buttress, fed through the existing E-65 Canal, using two 8-foot diameter tunnels for the outlet.
- Alternative E-5 – Elwood Reservoir embankments removed and replaced, fed through the existing E-65 Canal, using two 8-foot diameter pipes for the outlet, constructed using an open excavation during replacement of the embankments.
- Alternative E-6 – Elwood Reservoir embankment upstream shell is removed and replaced, fed through the existing E-65 Canal, using two 8-foot diameter tunnels for an outlet.

Each of these alternatives is discussed in Section 3.

1.2 J-2 Return Reregulating Reservoir Background

Three alternatives were investigated for a new J-2 Return reregulating reservoir. These alternatives are in Gosper, Phelps, and Dawson counties in central Nebraska. The original concepts for these reservoirs are depicted in the J-2 Return Pool Reservoir figures in the WMS Phase II Report and are provided for reference in Appendix A of this report. These alternatives were investigated to size and locate a reservoir capable of augmenting a SDHF as defined in the operating criteria. In addition, the alternatives were evaluated for their ability to reduce shortages to target flows and mitigate hydropower flow cycling based on the reservoir configurations developed during the SDHF analysis. The three alternatives investigated were:

- Alternative 1 – Reservoirs in the South Channel of the Platte River adjacent to Jeffrey Island
- Alternative 2 – Reservoirs excavated near the Platte River and the Phelps or J-2 Return Canals
- Alternative 3 – Reservoir at the exit of Phelps Canal Station 9.7

Each of the alternatives investigated is further discussed in Section 4.

1.3 Objectives

SDHF and target flows analysis goals were developed at an operational assessment meeting held on July 30, 2009 with ED Office staff and a project workgroup of WAC members.

1.4 Short Duration High Flow (SDHF)

The Program’s Adaptive Management Plan (AMP) identifies short-duration high flows (SDHFs) as a management action to be taken under the Flow-Sediment-Mechanical management strategy (AMP, 2006). SDHFs will be generated in the associated habitats in the spring or other times outside the main irrigation season with a goal of implementing these flows on an annual or near-annual basis (likely two out of every three years). The maximum magnitude of SDHFs will be roughly 8,000 cfs, based on natural flow in the river, the Program’s ability to deliver 5,000 cfs of water at Overton, local flood stage, and the Program’s “Good Neighbor Policy” which prevents Program water releases from exceeding flood stage.

The timeframe for a SDHF was discussed at the July 30, 2009 meeting and it was decided that for this analysis SDHFs would occur in late February or in March. This avoids icing concerns, and is prior to nesting and irrigation seasons (irrigation season was assumed to be April 1st through August 31st). For the purposes of this study, a mid-March release was selected (starting March 15th). CNPPID usually starts filling the reservoir between March 7 and March 15. During actual operations, all Program water would need to be released from the reservoir by then. For this analysis it was assumed that the three day SDHF would begin on March 15 and continue through March 18, after which time CNPPID would begin filling Elwood for irrigation purposes. Because CNPPID could begin filling Elwood Reservoir March 7, future analysis should consider a SDHF that is completed prior to March 7. The US Fish and Wildlife Service (USFWS) flow recommendations call for annual pulse and/or peak flows to be timed either in the February through March or May through June periods, so a SDHF during irrigation season is possible. If a different timeframe were used, results could be significantly different.

ED Office staff, with input from the WAC developed a conceptual diagram of Central Platte system components above Overton. The conceptual Platte River Components diagram, Figure 1.4-1, shows the estimated flows from various system components that could contribute to a SDHF. This resulted in approximately 4,703 cfs at Overton prior to flow augmentation flows from a reregulating reservoir. The values were estimated based on the system component capacities and an improved North Platte choke point capacity of 3,000 cfs.

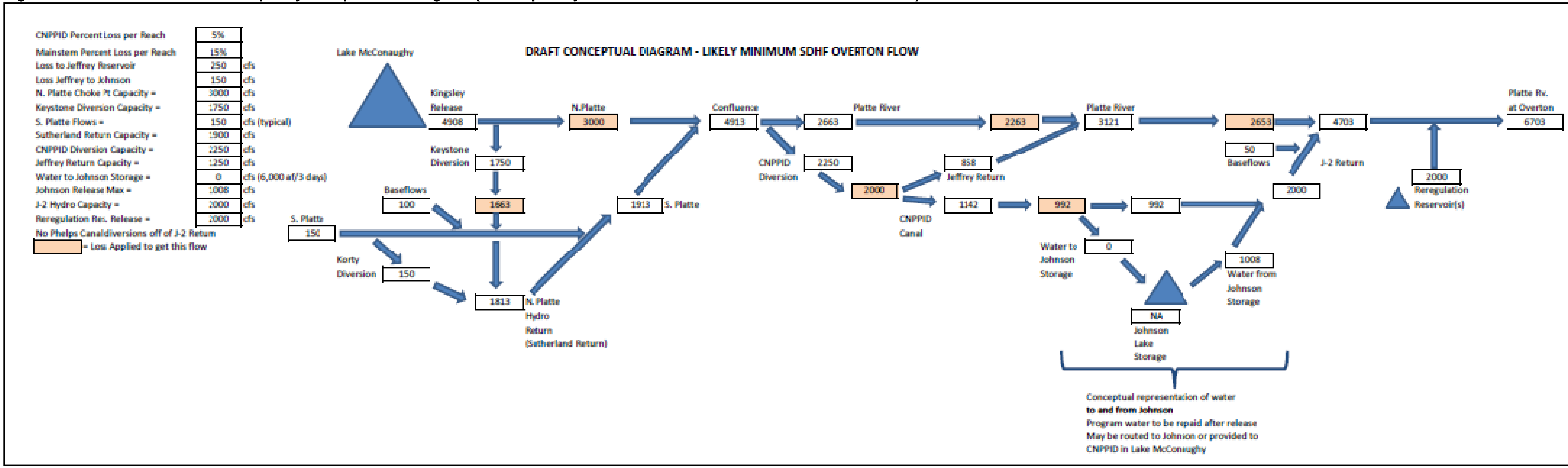
Examination of the known inflows from the Platte River at Cozad, J-2 Return Canal and Plum Creek, compared to the Platte River Overton gage recorded flows, revealed that there is a spring flow increase of 50 cfs that can be attributed to groundwater inflow. CNPPID also estimates that 50 cfs of baseflow accrues to the J-2 Return below the J-2 hydro. The Phelps Canal flume gage is located downstream of the J-2 Return. Flows are not directly measured at the J-2 wasting station but rather calculated as the difference between the J-2 Hydropower flow and the Phelps flow. Based on this it is not clear if the 50 cfs baseflow emerges entirely or partially through the J-2 Return. This amount was assumed to enter the system upstream of Overton either through the J-2 Canal outlet works or by direct groundwater inflows near the J-2 outlet works. It should also be noted that based on the hourly 2001 gage flow data from these same gage locations, the Platte River in this reach alternates from a losing stream in the winter to a gaining stream in the summer. It is likely that the irrigation canals surcharge the groundwater table in the summer and shift the Platte River to a gaining system during canal operation. The baseflow of 50 cfs is therefore relevant only in the spring and the amount of flow increase (or decrease) will vary during other parts of the year. The baseflow of 50 cfs is shown on the Platte River Components diagram, Figure 1.4-1.

A range of flows are possible depending on South Platte inflows, base flows below the Korty diversion, losses/gains for various reaches below North Platte, and success in timing releases to coincide with higher flows. If the system is operated optimally, assuming a reregulating reservoir can provide 2,000 cfs of supplemental flow at Overton would result in a total Overton flow of approximately 6,700 cfs, which would meet the SDHF goal of 6,000 to 8,000 cfs. As a result, the reregulating reservoir workgroup agreed that for analysis purposes during this study, attempts should be made to design reservoir alternatives to release 2,000 cfs over three days for SDHF augmentation.

To provide a SDHF, potential reregulating reservoirs analyzed during this study were assumed to be filled with EA water released from Lake McConaughy and excess flows, if available during the filling period. During the study analysis, reservoirs were filled as quickly as possible prior to the SDHF event, limited only by system and reservoir capacity. Analyses assumed that the PRRIP would use the CNPPID's and NPPD's full system capacities to route water in preparation for and during a SDHF. Because supply was limited only by system and reservoir capacities, rather than water supply availability, only one year was modeled for SDHF analysis.

Modeling was performed in two steps, a fill sequence and then an emptying sequence. The fill sequence involved routing EA water from Lake McConaughy and staging it in the reservoir or capturing excess flows, if available, immediately prior to the SDHF to fill the reservoir. The emptying sequence involved a controlled opening of the release gates to generate a three-day peak outflow.

Figure 1.4-1 – Platte River Peak Capacity Components Diagram (Developed by ED Office staff in coordination with the WAC).



1.5 Target Flow Analysis

The alternatives analysis included operational scenarios to reduce shortages to target flows for representative wet, normal, and dry years. The alternatives were scored based on their ability to reduce shortages to target flows.

1.5.1 Excess Flows

Excess flows were the supply for each reregulating reservoir alternative in evaluating the alternative's ability to reduce shortages to target flows. To this end, available excess flows in CNPPID's system were stored during periods of flow in the river that were above the targeted minimum flows and released during periods of less flow.

New information and ongoing discussions with the WAC, the project workgroup, and the USFWS led to changes in how excess flows are currently calculated for pre-feasibility analyses as compared to the Reconnaissance-Level WAP analysis done in 2000. For this study, PRRIP target flows were the daily values presented in Appendix A-5 of the Program Document Attachment 5 Water Plan, Section 11 Water Plan Reference Material (PRRIP, 2006) and are shown in Table 1.5-1. Excess flows in the Platte River were evaluated at the Overton gage. An evaluation completed by the ED Office found that using stream flow at the Overton gage rather than the Grand Island gage typically decreases the estimate of total annual excess flows available at the associated habitat. Relying on the Overton gage to evaluate shortages to target flows also leads to greater shortages as compared to using the Grand Island gage. This results in a decrease in potential project water supply from excess flows and an increase in shortages to target flows. Considering this information, the WAC determined that this conservative approach is appropriate for initial feasibility level project evaluation. However, it may be appropriate to modify excess flow analyses in subsequent phases of feasibility investigations to more thoroughly consider project complexities, interactions between projects, or upon further USFWS policy clarification.

Excess flows at Overton were calculated as flows greater than the maximum of PRRIP target flows, which vary depending on the hydrologic year type, and Nebraska Game and Parks Commission (NGPC) and Central Platte Natural Resource District (CPNRD) instream flows (target/instream flows) (Table 1.5-2). In normal and wet years, PRRIP target flows are always higher than instream flows. In dry years, there are periods when instream flows are higher than PRRIP target flow requirements. NGPC/CPNRD Grand Island instream flows (which are always the same or higher than Overton instream flows) were used to be conservative. The resulting targets for use in determination of excess flows are shown in Table 1.5-2. Excess flows at alternative reservoir locations were then calculated as the minimum of either water in CNPPID's returning to the Platte River through the J-2 Return or excess flows at Overton. Additional constraints based upon alternatives physical capacities (canal, inlet, storage, pumping, etc) were also applied to the analysis. Shortages to target flows were calculated as the difference between PRRIP target flows (Table 1-5.1) and Overton flows when target flows were greater than Overton flows. Excess flows and shortages to target flows were calculated on a daily basis which allows for days of excess flows and days of shortages in the same month.

To evaluate if excess flows were available at the potential reservoir locations, no lag times or gains and losses between the reservoir and gage locations and Overton were considered. Target flow analysis was evaluated over the course of the entire year using historic gage data. The Platte River target flows vary over the course of a year and will vary based on the yearly precipitation.

Table 1.5-1 Daily PRRIP Target Flows from PRRIP Program Document (PRRIP, 2006), Appendix A-5.

Time Period	PRRIP Target Flows		
	Wet	Normal	Dry
Jan 1 – Jan 31	1,000	1,000	600
Feb 1 – Feb 14	1,800	1,800	1,200
Feb 15 – Mar 15	3,350	3,350	2,250
Mar 16 – Mar 22	1,800	1,800	1,200
Mar 23 – May 10	2,400	2,400	1,700
May 11 – May 19	1,200	1,200	800
May 20 - May 26	4,900	3,400	800
May 27 – June 20	3,400	3,400	800
June 21 – Sept 15	1,200	1,200	800
Sept 16 – Sept 30	1,000	1,000	600
Oct 1 – Nov 15	2,400	1,800	1,300
Nov 16 – Dec 31	1,000	1,000	600

Table 1.5-2 – Maximum of PRRIP and NGPC/CPNRD Target/Instream Flows

Period	Condition		
	Wet	Normal	Dry
Jan 1 – Jan 31	1,000	1,000	600
Feb 1 – Feb 14	1,800	1,800	1,200
Feb 15 – Mar 15	3,350	3,350	2,250
Mar 16 – Mar 22	1,800	1,800	1,200
Mar 23 – May 10	2,400	2,400	1,700
May 11 – May 19	1,200	1,200	800
May 20 - May 26	4,900	3,400	800
May 27 - May 31	3,400	3,400	800
June 1 – June 20	3,400	3,400	1,000
June 21 - July 31	1,200	1,200	1,000
Aug 1 – Sept 15	1,200	1,200	800
Sept 16 – Sept 30	1,000	1,000	600
Oct 1 – Oct 11	2,400	1,800	1,350
Oct 12 - Nov 10	2,400	1,800	1,500
Nov 11 - Nov 15	2,400	1,800	1,300
Nov 16 – Dec 31	1,000	1,000	600

1.5.2 Representative Years Selection for Target Flow Analysis

A key challenge in modeling this system is that excess flows vary substantially from year to year, based on many factors such as snow melt timing and thunderstorm-related precipitation events. The project workgroup opted to select one representative dry, normal and wet year for this pre-feasibility level analysis. The group felt that it would be useful, and more intuitive when presenting results, to look at historical data for specific years rather than to evaluate alternatives relying on averaged data. Additionally, for engineering design of gates and embankments, averaged data sets tend to underestimate necessary sizes. Three illustrative years representing a typical wet, normal, and dry year were selected by the ED Office for screening analysis. Wet, normal and dry year classifications for the 1947 through 2006 period from the WMS Phase II Report (Boyle, 2008) were used. These were prepared according to a methodology prepared by the USFWS based on Grand Island flows. Though the USFWS methodology classified years by calendar year, water years were used for this analysis with October through December data of a previous year included with January through September data from the following year to arrive at data for a water year.

The normal "representative" year was selected by comparing the average monthly total flow at Overton for all "normal" years to each individual year's total monthly flow. This process was repeated for years classified as wet and those classified as dry. Water year 1975 was selected as the representative normal year, 1964 as the dry year, and 1986 as the wet year. ED Office staff then calculated excess flows and shortages to target flows for each representative year. This information was provided to Olsson Associates for use in the study analyses. Measured daily flows and target flows for the three representative years are graphically presented in Figures 1.5-1 thru 1.5-3. Note that these figures show excess flows in the river at Overton. Excess flows that could be stored in a reregulating reservoir were further limited by flows in CNPPID's J-2 Return (the supply for the reservoirs), and canal, inlet and reservoir storage capacities.

Figure 1.5-1 – 1964 Illustrative Dry Year – Target Flow and Measured Flow at Overton gage on the Platte River.

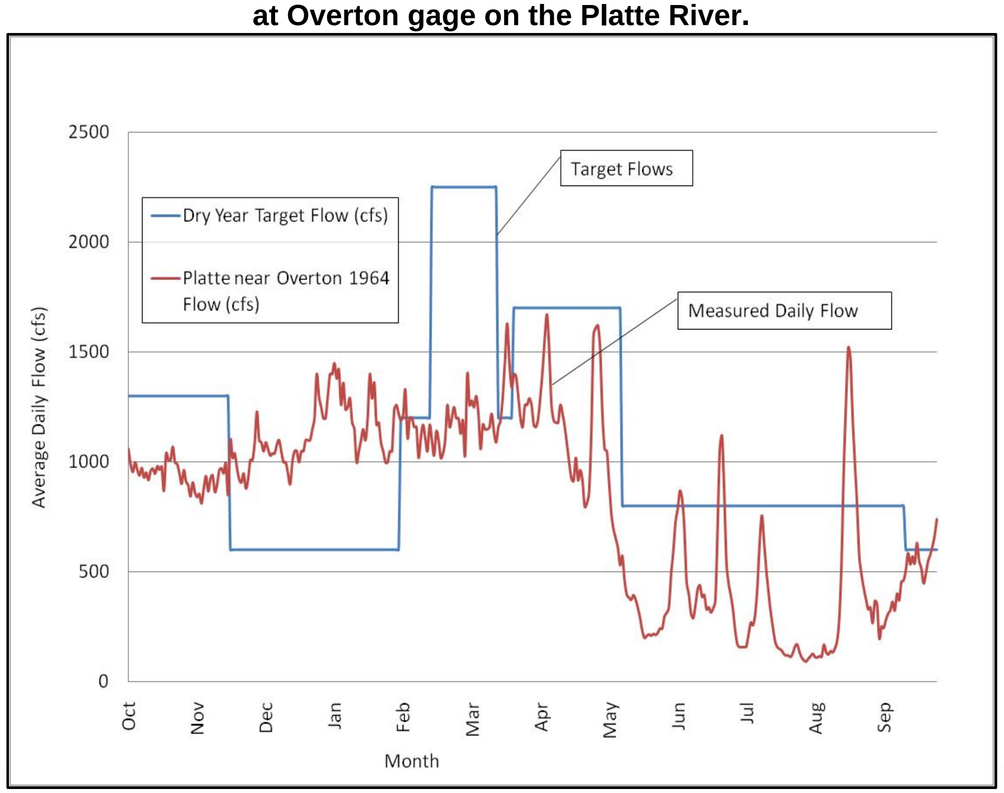


Figure 1.5-2 – 1975 Illustrative Normal Year – Target Flow and Measured Flow at Overton gage on the Platte River.

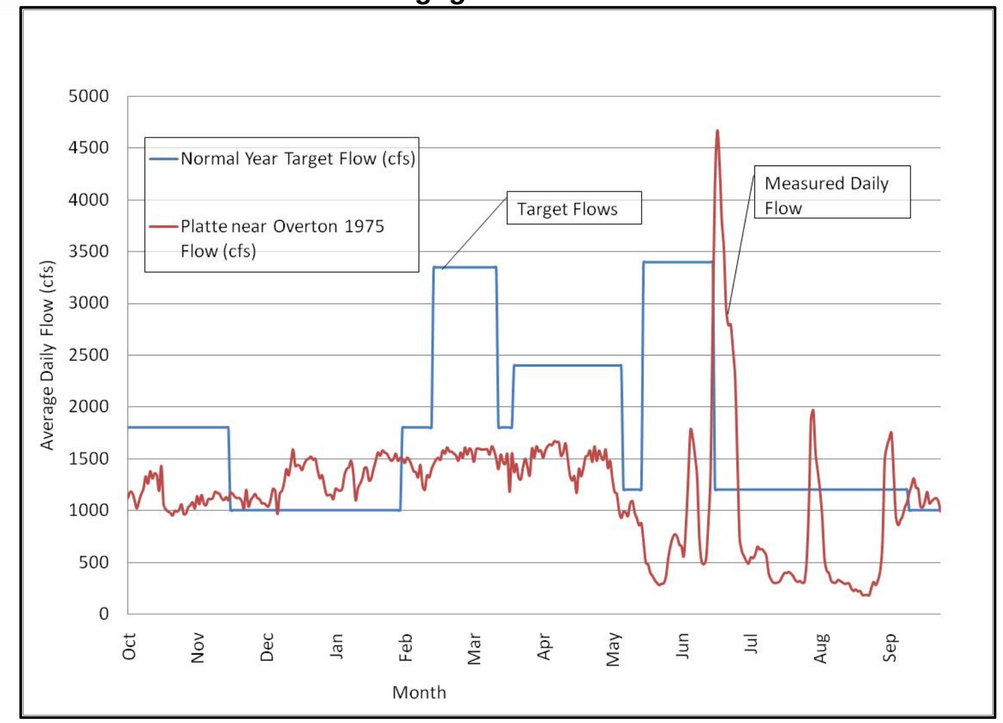
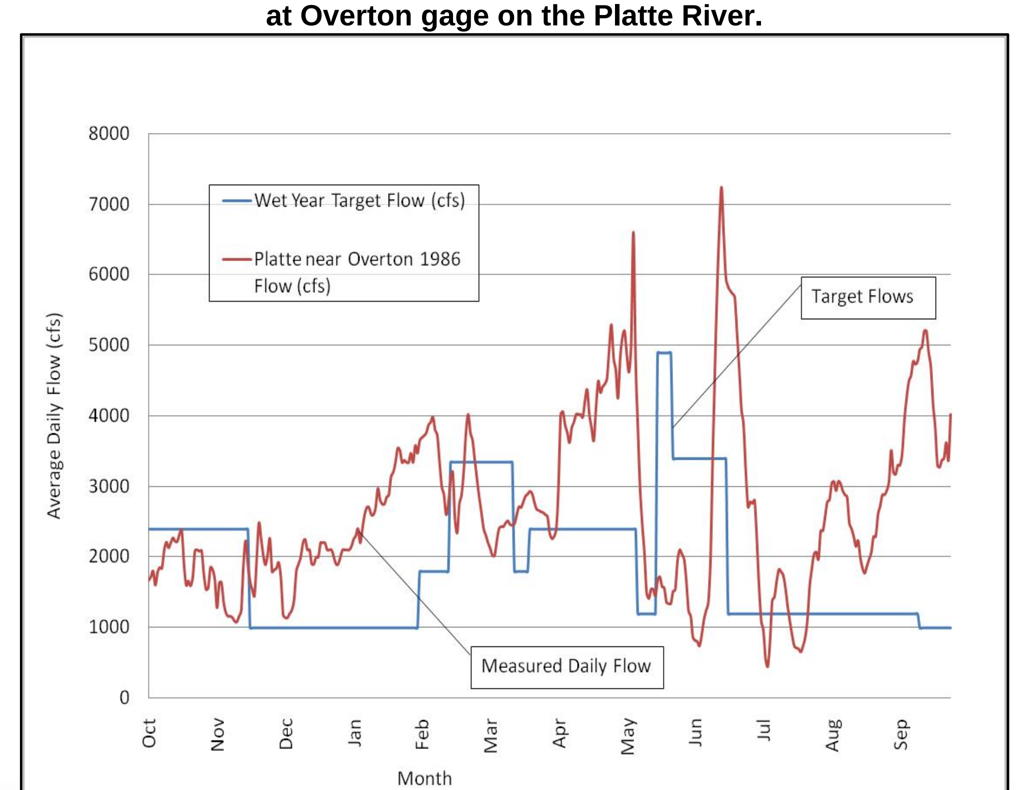


Figure 1.5-3 – 1986 Illustrative Wet Year – Target Flow and Measured Flow at Overton gage on the Platte River.



In all three representative years, excess flows were available in the December to February time frame, which gives an optimistic outlook on the availability of using excess flows for a SDHF event in mid-March. Exclusive of this time window however, there were numerous periodic shortages spaced throughout rest of the year. Even during a “wet” year there were shortages to target flows for portions of March and May/June. The graphs also highlight the quick response of the system to precipitation events. The graphs are based on averaged daily values which tend to dampen variation produced from hydropower cycling and storm events. Hourly records were not available for these years, but based on the 2001 data reviewed by Olsson and CNPPID’s J-2 hydro operations, it is anticipated the hourly records would show even more variability in flow rates.

Target-flow models for each of the J-2 alternatives were initially developed using HEC-HMS. Due to a modeling constraint in HMS that does not allow a continuous simulation to advance if the reservoir becomes dry, the models for Elwood and J-2 were re-developed in Excel. The models assume that water, up to the volume of excess flows in storage, is released to reduce the shortage to target flows up to the maximum outlet rate the alternative can support. Also, if the reservoir is full, any excess flows in CNPPID’s system are not stored but rather return to the Platte River. This assumption requires a controlled inlet into the CNPPID system for capturing only excess flows and a controlled outlet for each reservoir to output no more than the required flow for each day.

The J-2 alternatives are on or adjacent to the J-2 Return (capacity 2,000 cfs) and are located close to the Platte River. As a result they tend to fill and release quickly. Under these scenarios seepage rates are not critical and seepage losses were not calculated. Elwood Reservoir however, is located distant from the Platte River and seepage losses would not immediately flow into the Platte River. Therefore, Elwood target flow analysis includes the effects of seepage losses, in addition canal conveyance losses to the Platte River.

1.6 Hydropower Flow Cycling Impacts

Dampening of hydropower flow cycling from the J-2 Return is a desired characteristic for the alternatives. This is not a priority objective of the feasibility study assessment but was evaluated as a potential secondary benefit of the alternatives. Hydropower flow cycling is a concern of the USFWS (FERC, 2007). CNPPID may want to use a selected alternative to help mitigate cycling throughout the year and could potential provide funding assistance. CNPPID advised that the typical hydropower flow cycling portion of the modeling run analysis should be determined from the average operations. Olsson found the daily average volume from the J-2 Return of all the available years of data for the month of March (1947 – 2006) was 2,300 ac-ft. CNPPID was consulted and indicated the peak operating efficiency of the J-2 hydropower turbine is at 1,675 cfs. Further, the generated electricity can be sold at the highest rate during late evening hours. Using 1,675 cfs as the most efficient operating flow and the timeframe for highest value, the typical generation cycle runs from 7a.m. to midnight – approximately 70 percent of the day. The diurnal flow swing from 1,675 cfs to 0 cfs representing a cumulative volume of 2,300 ac-ft was used in all J-2 alternatives. Complete dampening would result in the maximum and minimum daily flow being equal to the calculated average daily flow, (zero departure from average). Elwood Reservoir is not located downstream of the J-1 or J-2 hydropower stations and hence does not have the ability to mitigate hydropower flow cycling impacts, so only the J-2 alternatives were evaluated for this operational mode. Figure 4.7-2 in the J-2 Alternatives analysis graphically portrays the swing.

2. DEVELOPMENT OF ALTERNATIVES ANALYSIS CRITERIA

In developing and screening the variety of alternatives a scoring/ranking system was needed to evaluate items such as ac-ft yields as well as non-numerical items such as anticipated ability to obtain permits. Further, each measured item did not carry the same importance. Olsson and ED Office staff, along with the subcommittee, discussed the potential ranking methods on August 11, 2009. It was decided the alternative screening would be accomplished using a standard Kepner Tregoe approach, which weighs and scores each alternative for how well it meets different project needs. Each resulting criterion has a relative weight for use in the scoring process. Additionally, some alternatives were screened out as being not feasible for cost or operational reasons. The "score" assigned to alternatives in this study is a ranking solely to compare alternatives with the purpose of selecting alternatives for advancement. The "scoring" of alternatives in this study should not be confused with official PRRIP score that will be assigned to Water Action Plan (WAP) projects.

2.1 Criteria

The following list of criteria were used in the scoring analysis of alternatives for Elwood and J-2, which are further clarified in subsequent sections:

1. Life cycle cost for the alternative, divided by the normal year delivered water
2. SDHF augmentation
3. Reduction of shortages to target flows
4. Operational flexibility and multiple benefits
5. Ability to obtain necessary federal, state, and local permits
6. Impacts to landowners, other facilities, and installations
7. Portion of the habitat reach that is positively affected by water delivery
8. Opportunities for partnering
9. Implementation time
10. Hydropower flow cycling mitigation

2.2 Scoring

All alternatives were scored from zero to five for their relative ability for achievement with respect to each specific criterion. The scoring for each criterion was based upon the scoring factors below.

2.2.1 Criterion No. 1 – Life Cycle Cost per Acre-Foot

The capital costs for each alternative were added to the operating costs for that alternative over a 50-year life span. This cost figure was then divided by the total volume of water to augment the SDHF, including EA water from Lake McConaughy staged in the reservoir prior to the event, plus the volume of reductions to shortages to target flows (ac-ft) of water that the alternative delivered in a normal year over the same 50-year time span. The lower the life cycle cost per ac-ft of water delivered, the higher the score. Consequently, the following range was used in the scoring:

5 - Less than $20 per ac-ft
4 - $21 to $40 per ac-ft
3 - $42 to $60 per ac-ft
2 - $61 to $80 per ac-ft
1 - $81 to $100 per ac-ft
0 - More than $100 per ac-ft

2.2.2 Criterion No. 2 –SDHF Augmentation

The reservoir alternatives were designed to store, discharge, and convey water to the associated habitat reach to augment SDHF. Conveyance of SDHF discharges were achieved by canal or by existing stream channel. Delivery from the system to the Platte River gage near Overton of 2,000 cfs average for three days was assigned a score of '5'. The scoring scale used was:

5 - 2,000 cfs average or more for three days
4 - 1,750 cfs to 2,000 cfs average for three days
3 - 1,500 cfs to 1,750 cfs average for three days
2 - 1,250 cfs to 1,500 cfs average for three days
1 - 1,000 cfs to 1,250 cfs average for three days
0 - less than 1,000 cfs average for three days

2.2.3 Criterion No. 3 – Reduction of Shortages to Target Flows

Using the normal illustrative year (1975), the ability of the alternative to reduce shortages to target flows in the Platte River at Overton was analyzed. The alternative configuration was then scored based upon the annual ac-ft of reductions to shortages the configuration could provide. In the normal year gage records, historical flows resulted in 540,662 ac-ft of shortages to target flows (PRRIP, 2006). However, there were periodic times of excess flows throughout the year. The reregulating reservoirs would capture the excess flows and release the water when the flow at Overton drops below target flows (PRRIP, 2006). Scoring the reduction of shortages to target flows was accomplished using the rating scale below.

5 - Greater than 20,000 ac-ft per year
4 - 15,000 to 20,000 ac-ft per year
3 - 10,000 to 15,000 ac-ft per year
2 - 5,000 to 10,000 ac-ft per year
1 - 2,500 to 5,000 ac-ft per year
0 - Less than 2,500 ac-ft per year

It should be noted the ac-ft reductions according to the study analyses was based on the illustrative years and was used as a screening tool. A complete analysis of all the gage records to date should be performed in future study and design phases to integrate with WAP ac-ft goals.

2.2.4 Criterion No. 4 – Flexibility and Multiple Benefits

Operational flexibility and multiple benefits were established as criteria because a strong alternative also would provide beneficial sedimentation delivery, benefits to CNPPID, and allow for alterations in the operation as the Adaptive Management Plan study findings became available. While this ranking is somewhat subjective, the following scoring was used:

5 - All three benefits – (sedimentation, benefits for CNPPID and incorporation of fisheries) – are identified
3 - Two out of three benefits are identified
1 - One out of three benefits is identified

2.2.5 Criterion No. 5 – Ability to Obtain Federal, State, and Local Permits

The screening process included a very preliminary estimate of the ease or difficulty of obtaining necessary permits. A ranking of '5' would mean that the alternative and configuration would result in minimal time, difficulty and mitigation cost for obtaining necessary permits. A ranking of '0' would mean that there would be a fatal flaw prohibiting permitting for that alternative. Intermediate rankings were provided based upon a subjective assessment by Olsson environmental specialists.

Due to the conceptual nature of the alternatives and the early stage of coordination with the USACE, existing databases, such as the National Wetland Inventory maps, were used to estimate potential impacts to waters of the U.S. Difficulty of permitting each alternative was then based on extent of impacts.

A re-regulating reservoir for the PRRIP is likely to require four environmental permits:

- Clean Water Act Section 404 Permit from the USACE
- Impoundment Permit from Nebraska Department of Natural Resources (NDNR)
- Floodplain Development permit from local governmental agency
- National Pollution Discharge Elimination System (NPDES) permit from local Natural Resources District (NRD) and/or Nebraska Department of Environmental Quality (NDEQ)

In addition, to meet the requirements of the National Environmental Policy Act (NEPA) the project would have to comply with the environmental commitments made in the 2006 Final Environmental Impact Statement (EIS) and Record of Decision (ROD) that established the PRRIP.

Due to the conceptual nature of the alternatives at this stage in project development, the scoring process for environmental permitting and NEPA included a search for "red flags" that would eliminate an alternative from being able to be implemented, as well as a very preliminary estimate of the ease or difficulty of obtaining appropriate permits for each alternative.

Section 404 Permit
In general, alternatives that had relatively large fill impacts to the Platte River (historic or existing channel) were ranked as most difficult to permit, and alternatives that impacted Plum Creek, smaller tributaries, or floodplain wetlands, were ranked as easier to permit depending on the extent of impacts that were likely to occur for each alternative. Alternatives that avoided fill within any stream scored higher.

NDNR Impoundment Permit
Concerns had been expressed that changes to the operation of Elwood could alter the amount of seepage that enters the Republican Basin instead of the Platte Basin. Discussions with Cory Steinke of Central Nebraska Public Power and Irrigation District (CNPPID) indicated that this is not a "red flag", but will require additional efforts. A new impoundment on or near the Platte River might be equally easy or difficult to permit as modifications to the existing Elwood Reservoir, and thus at this stage all alternatives were ranked equally for this permit.

Floodplain Development Permit
No detailed study of the impacts to floodplains was conducted as a part of this early screening process. As such, it is assumed that alternatives that dam the Platte River would be more difficult to permit.

NPDES Permit
This permit would be needed for any alternative that disturbs more than 1.0 acre of ground, and the effort to secure this permit would be similar for all alternatives.

NEPA Issues
The scoring process included categories for other EIS commitments such as not condemning property for PRRIP projects. However, the EIS also included other Program goals such as not contributing to additional listing of threatened and endangered species. At the moment, the conceptual alternatives are too general to determine impacts to other environmental resources such as rare species or cultural sites. Some of the resources that may need to be investigated further as alternatives are developed, include impacts to sloughs or backwater areas within or near the Platte Channel, such as the Platte River caddisfly that may be proposed for listing by the USFWS. Similarly, some alternatives near Elwood could also impact habitat for the American burying beetle. As a result of the speculative nature of these potential impacts and the likelihood that alternatives will change as they are developed further, these species were not a major consideration in screening alternatives at this time.

2.2.6 Criterion No. 6 – Impacts to Landowners, Other Facilities and Installations

The impact to landowners was considered a key issue for the PRRIP. The greater the number of landowners affected by the alternatives, the more challenging the development would become. This criterion included impacts to public roads and private facilities such as the Canaday Steam Plant located next to the J-2 Return Canal. A criterion to score this particular parameter was therefore included. It was scored as:

5 - 0 landowners to 1 landowner affected
4 - 2 landowners to 3 landowners affected
3 - 4 landowners to 5 landowners affected
2 - 6 landowners to 7 landowners affected
1 - 8 landowners to 9 landowners affected
0 - 10 landowners or more landowners affected

2.2.7 Criterion No. 7 – Portion of the Reach Positively Affected by Water Delivery

The ability of the alternative to deliver water to the entire associated habitat reach of the Platte River was of particular interest to the PRRIP. It was assumed if the water was delivered at Overton, the entire habitat reach would benefit and no attenuation of flows or other losses within the habitat reach were calculated. It was scored as:

5 - Delivering water to Overton
4 - Delivering water to 80% of the reach, between Overton and Chapman
3 - Delivering water to 60% of the reach, between Overton and Chapman
2 - Delivering water to 40% of the reach, between Overton and Chapman
1 - Delivering water to 20% of the reach, between Overton and Chapman
0 - Delivering water below Chapman

2.2.8 Criterion No. 8 – Opportunity for Partnering

The opportunity to partner with other entities such as USFW, CNPPID, Nebraska Natural Resources Districts, and others, for mutual beneficial use was considered to be valuable. Therefore scoring was performed using the following approach:

5 - If there were opportunities to partner with two other entities
3 - If there was an opportunity to partner with one other entity
0 - If there were no opportunities to partner

2.2.9 Criterion No. 9 – Implementation Time

The ability of each alternative to be designed, permitted, and constructed in a short time frame is an important factor to the PRRIP to meet timeline goals (PRRIP, 2006). Consequently, implementation time was identified as an important criterion, and scored as:

5 - Implementation by spring of 2011
3 - Implementation by 2014
0 - Implementation by 2019

2.2.10 Criterion No. 10 – Hydropower Flow Cycling Mitigation

The impact on the Platte River due to flow cycling resulting from hydropower generation has been an issue of concern (FERC, 2007). Complete dampening (100%) indicates the instantaneous peak flow maximum and minimum equal the average daily flow from the hydropower facility. This criterion was scored as:

5 - Dampening of 90% to 100% of the surge
4 - Dampening of 80% to 90% of the surge
3 - Dampening of 70% to 80% of the surge
2 - Dampening of 60% to 70% of the surge
1 - Dampening of 50% to 60% of the surge
0 - Dampening of less than 50% of the surge

2.3 Weighting

In order to be able to properly score the various alternatives, weighting factors were applied to the scoring criteria, to indicate the relative importance of the criterion. These weighting factors were applied as multipliers to the score of each parameter for each alternative. The weightings were developed with the ED Office, Olsson staff, and the project workgroup and are shown in Table 2.3-1.

Table 2.3-1 – Screening Criteria Weighting

Screening Criteria	Description	Weighting
1	Life Cycle Cost per ac-ft delivered to Reach	10
2	SDHF Augmentation	10
3	Reduction of Shortages to Target Flows	8
4	Flexibility and Multiple Benefits	5
5	Ability to Permit/NEPA	10
6	Impacts to Landowners/Others	8
7	Portion of the Reach Affected	10
8	Opportunities for Partnering	5
9	Implementation Time	10
10	Hydropower Flow Cycling Mitigation	6

3. ELWOOD RESERVOIR ALTERNATIVES

The WMS Phase II study (Boyle, 2008) determined that Elwood Reservoir and the Plum Creek channel, with modifications, could provide SDHF augmentation and potentially reduce as well in shortages to target flows to the Platte River as part of the PRRIP. The WMS recommended enlarging the E-65 Canal siphons to 650 cfs capacity to meet irrigation needs downstream of Elwood Reservoir without using the reservoir for supplemental irrigation storage. The existing E-65 Canal upstream of the reservoir currently cannot provide the full irrigation capacity required. Outlet works modifications were recommended to provide the needed outflow capacity for SDHF augmentation. Dam improvements to mitigate against the effects of rapid drawdown were noted to be potentially necessary. A new, unlined return canal between Elwood Reservoir and the Platte River was also recommended. A suggested alignment was provided in the report.

The estimated costs of the proposed improvements totaled $76 million. The first objective of the Elwood Reservoir Alternatives Screening Analysis performed for this study was to refine the WMS Phase II Study concepts by developing cost-feasible alternatives that are constructible, and to increase storage if possible. The second objective was to update modeling of the SDHF and reduce shortages to target flows. The third and final objective was to develop a scoring and ranking process to evaluate the developed alternatives.

The potential alternatives for using the Elwood reservoir for SDHF augmentation involve modifications or revisions to the following major components:

- The dam embankment
- The outlet works
- The siphons/inlet
- The outlet channel/conveyance system to the Platte River

3.1 Potential Dam Embankment Modifications

Providing a 2,400 cfs release rate for three days would be a significant change in the operation of Elwood Reservoir and would require modifications to the outlet works, the shoreline and surface below the current permanent pool, and possibly to the dam embankment. The evaluations of these key features are described in greater detail below. In addition, CNPPID has operational agreements with the National Wildlife Federation that will complicate making changes to the operating curve. Further, it has been estimated by others (CH2M Hill, 1993) that approximately 53% of the seepage losses in Elwood leave the Platte River basin and migrate to the Republican River basin. This study focused on potential engineering aspects of changes but did not evaluate regulatory or contractual issues that would likely be involved in changing the operation of Elwood.

3.1.1 Embankment Stability

As originally designed, Elwood Dam would experience maximum drawdown rates of 350 cfs. Based on available data it appears that average drawdown rates have been in the range of 150 cfs to 200 cfs. The stability of an embankment dam during reservoir drawdown is a critical component of a dam's overall safety and is a key element of most stability evaluations and analyses. During a reservoir drawdown, the stability of the upstream shell is dependent on the strength characteristics of the embankment material and the ability of the embankment to effectively dissipate pore pressures. Pore pressures are caused by the seepage of water through an embankment. Under steady state conditions, these pressures are a function of the reservoir elevation, local geology and the geotechnical characteristics of the embankment. Elwood Dam has blanket and toe drains to safely collect and convey seepage through the downstream portion of the embankment. If the pore pressures are not sufficiently reduced during a reservoir drawdown event, instability may occur in the upstream soil cement shell.

A review of the existing information and reports did not uncover any previous rapid drawdown stability analyses for the Elwood Dam. The geotechnical data from the original investigation was reviewed; however, John Livingston -- the CH2M Hill field engineer during construction -- has indicated that the fill material is a mixture of different materials. The geotechnical parameters are in general agreement with the finer-grained materials encountered during the initial geotechnical investigation. Based on experience and assumptions regarding soil conditions and discussions with members of the Olsson team who have worked with the soils in this area, the likely rapid drawdown loading condition that would occur due to the potential modifications was identified and evaluated. Several drawdown rates, ranging from 5,000 cfs to 500 cfs, were preliminarily evaluated to provide upper bounds and lower bounds for our analysis.

It also should be noted that the reservoir and corresponding phreatic surface – that is, the surface that defines the internal groundwater elevation within the embankment – were assumed to be at their maximum levels. This assumption means the reservoir would be filled to its highest level and maintained at this level until the pore pressures within the embankment stabilized. This condition is the most conservative loading condition for the drawdown analyses.

This is not how Elwood Reservoir has been operated in the past. The reservoir has been quickly filled. Water is then released before internal pore pressures are allowed to stabilize. This operation scheme limits the maximum pore pressure within the embankment, which benefits the stability of the dam. However, the future operation of the reservoir in this manner may not be workable if the reservoir were modified to provide SDHF augmentation. Consequently, it was prudent to evaluate a more conservative loading condition that may be necessary and to analyze the stability of the dam when this loading condition is subjected to the drawdown rates currently being considered. The following describes the results of this analysis.

The detailed embankment stability analysis is included in Appendix I. In general terms, the analysis showed that if steady state storage conditions are allowed to develop, rapid drawdown will result in unacceptable safety factors with regard to embankment stability.

3.1.2 Embankment Upgrade Alternatives

Five embankment upgrade alternatives were evaluated to produce acceptable factors of safety with regard to stability. Three of these alternatives advanced to scoring as part of the Elwood alternatives.
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3.1.2.1 Embankment Upgrade Alternative No. 1 - Do Nothing

The do nothing alternative would not use Elwood Reservoir for the SDHF augmentation purposes so there wouldn't need to be changes made in the way the Elwood Dam is currently operated. This alternative would continue limitation of releases to 350 cfs, and there would be no additional costs related to use by the Program. This alternative was not included in any of the Elwood alternatives.

3.1.2.2 Embankment Upgrade Alternative No. 2 - Remove and Replace Dam

The removal and replacement of the dam at Elwood – with embankments designed to perform satisfactorily during rapid drawdown – would address the rapid drawdown stability issue. There are potentially multiple configurations for this alternative, and in-depth evaluations of each are beyond the current scope. However, it is envisioned that these dams would be constructed as true zoned embankments, with the permeability of the core several orders of magnitude less than the permeability of the upstream and downstream shells. Additionally, the use of extensive internal filter and drainage zones would be required to ensure the adequate performance of the dam in a wide range of loading conditions. As with the current dam, a facing system – most likely soil cement – would be required to prevent erosion of the upstream face. This embankment alternative was included in Elwood alternatives E-2 and E-5, which are summarized in Section 1.1 and Table 3.6-1.

3.1.2.3 Embankment Alternative No. 3 - Upstream Buttress/Upstream Embankment Slope Flattening

The installation of an upstream buttress, or flattening the angle of the upstream slope, could provide the embankment additional strength to resist potential slope failure caused by rapid drawdown loading. The configuration of such a buttress could be accomplished by either constructing a new buttress at the toe of the upstream embankment or by flattening angle of the upstream slope. The current slope angle is 3 horizontal (H):1 vertical (V) and flattening this slope to between 4H:1V and 4.5H:1V would likely provide sufficient strength to remain stable during rapid drawdown loading. A buttress could be constructed with on-site materials – materials that are similar to those used for the original embankment construction. More detailed geotechnical work would be necessary for evaluation and design of a buttress. This embankment alternative was included in Elwood alternatives E-1 and E-4, as well as both combined Elwood/J-2 alternatives E/J-2 Alt 2, Area 1 and E/J-2 Alt 2, Area 2, as discussed later in the report.

3.1.2.4 Embankment Alternative No. 4 - Install Improved Internal Drainage System

Installing an improved internal drainage system within the embankment has the potential to improve the drainage characteristics and stability of the embankment, during rapid drawdown. However, there is considerable difficulty associated with this alternative. Also, without extensive geotechnical analysis, there is no assurance that a new drainage system would sufficiently stabilize the embankment during rapid drawdown. Further, the extent of the excavation required to improve the internal drainage would be so large that it may be comparable to the removal and replacement alternative previously discussed. This embankment alternative was not considered further.

3.1.2.5 Embankment Alternative No. 5 - Remove and Replace Upstream Shell Only

Removing and replacing the upstream shell as a means of stabilizing the Elwood Dam during rapid drawdown loading can be accomplished. The shell would be removed and replaced with known, suitable materials, faced with soil cement, and designed such that slope failure would not take place during rapid drawdown. Considerable questions remain about this embankment alternative. It is unclear if suitable material for this alternative is available on the site. It also is likely far less expensive to simply leave the upstream slope in place and install a new buttress or flatten the angle of the upstream slope, as previously discussed. This embankment alternative was included in Elwood alternatives E-3 and E-6.

3.2 Outlet Works

The existing outlet works cannot deliver 2,400 cfs for the SDHF augmentation. The existing capacity of the outlet works has been identified as having a maximum rate of 350 cfs (WMS Phase II, 2008). With this limitation, the construction of a new outlet works or the significant modification of the existing outlet works would be necessary if Elwood Reservoir were to be used for SDHF augmentation.

3.2.1 New Outlet Works Alternatives Evaluation

The existing outlet works could not meet the required releases of 2,400 cfs and a new outlet works would be required. The following criteria were established for this new outlet works:

a. Release velocities below 20 feet per second (ft/sec) within the discharge pipe to minimize potential damage to the outlet pipe

b. Invert elevation of 2,530 feet to maximize heads on the outlet works and provide a low-level outlet

c. Location on either the north or south abutment of the main dam, for easy access to either a new canal or Plum Creek

No major fatal flaws associated with the installation of new outlet works were identified during this study. The biggest question regarding its configuration are the limits to the size of the pipe and its regulating features. There are literally dozens of pipe and valve configurations that can accomplish the target release 2,400 cfs. For the purposes of this study, a basic hydraulic analysis determined that two 8-foot-diameter, steel-lined pipes would be capable of providing these flows. This configuration also would incorporate the use of the existing outlet works, which has a maximum discharge rate in the range of 300 to 350 cfs. A single, 12-foot diameter pipe also could also convey the desired flows; however, the costs of manufacturing and maintaining the gates for this size of pipe may be prohibitive. Twin conduits may provide benefits in flexibility of operation and maintenance.

Flows through the new outlet works could most easily be controlled at the downstream end of the conduits by using either hooded fixed-cone valves or radial gates that would discharge into a reinforced concrete stilling basin before entering the canal downstream.

An upstream control should also be provided to prepare for the unlikely event of problems with the pipe conduits, or the need to maintain or replace downstream valves or gates. This could be accomplished with hydraulically actuated vertical slide gates. They would normally be opened or closed under balanced head conditions, but also would be designed to close under their own weight in the case of an emergency at a time of concurrent power failure. Trash racks to exclude large debris, which could damage or jam the gates and valves, should be provided at the upstream end of the conduits.

3.2.1.1 Outlet Alternative No. 1 – Open Cut through Existing Dam

Making a cut through the existing dam and constructing a new outlet structure would be possible, but would give rise to a number of concerns in connection with replacing the dam fill. Specific concerns include achieving similar compaction characteristics to avoid differential settlement, as well as providing good watertight connections with the outside of the conduits and the body of the dam. Cutting into an existing embankment typically is regarded as something to avoid, as approval from dam safety regulators can be difficult to obtain. Therefore, this alternative should not be considered unless the alternative for complete replacement of the embankment is adopted. This alternative would require the complete draining of Elwood Reservoir for a minimum of two irrigation seasons.

3.2.1.2 Outlet Alternative No. 2 – Open Cut with Dam Replacement

In the circumstances where the whole dam was to be removed and replaced, it would be appropriate to incorporate a new high-capacity, low-level outlet excavated into the side slope of the dam foundation. Twin steel pipes, encased in concrete, would be an appropriate form of construction.

An upstream slide gate and trash racks would be incorporated in an outlet tower constructed in the reservoir at the upstream toe of the embankment. An access bridge from the embankment crest to the top of the tower would be required.

3.2.1.3 Outlet Alternative No. 3 – Tunneling

Tunneling through one of the abutments of the dam would appear to be a feasible option for providing a new outlet of sufficient capacity. The most appropriate tunneling method would be using pipe-jacking techniques, where a Tunnel Boring Machine (TBM) – consisting of a cutting head and a shield – is used to bore through the earth. As the shield advances, excavating material in its path, sections of pipe are hydraulically jacked into place directly behind it.

Usually, the pipes would be butt jointed concrete which would have an internal pressure rating up to approximately 100 pounds per square inch (psi). As the head on the tunnel would be about 33 psi --77 feet of water-- it would be necessary either to line the tunnel with a steel liner or use steel pipe sections for the jacking, which would require welding as each section is inserted. In either case, the annulus around the outside of the pipe would be grouted to ensure full support of the ground and avoid a potential seepage path along the outside of the pipe.

To provide the necessary thrust, reaction boring is normally started from a pit or shaft with a thrust wall constructed on the back face of the shaft. If this proves inappropriate at this site, a suitable thrust block/wall would need to be constructed aboveground.

The tunnel would be driven from the downstream end with a slight upslope to daylight at an upstream portal excavated into the slope and supported on each side by wing walls. The upstream slide gate and trash racks would be incorporated in an outlet tower constructed in the reservoir at the upstream end of the tunnel. An access bridge, from the abutment to the top of the tower, would be required.

3.2.1.4 Outlet Works Alternative Conclusions

A new outlet works would likely consist of a bored tunnel located on one of the abutments of the main dam. Modification of the existing outlet works would require the installation of two 8-foot diameter conduits or one 12-foot diameter conduit, as well as the installation of one or more additional pumps and upstream guard gates with trash racks. Modifications to the canal downstream of Elwood Reservoir would also be required

3.2.2 Geotechnical Recommendations for Further Embankment Analysis

A thorough geotechnical investigation would be needed to establish actual geotechnical characteristics of the dam and surrounding soils. The geotechnical investigation should include a review of the area geology, subsurface borings at the dam and anticipated location(s) for the outlet works, a laboratory testing program, and more extensive engineering analysis of the embankment during the planned rapid drawdown -- incorporating the results of the subsurface investigation and laboratory testing. An updated alternatives evaluation would also be required once the results of the updated engineering analysis are known. This geotechnical evaluation would provide a better understanding of the embankment, as well as provide valuable information for the determination of a feasible construction method for installation of new outlet works.

Concepts for the improved outlet works have been discussed and evaluated in general terms as part of this report, however additional analyses are required to determine the most feasible alternative. Specifically, a more thorough hydraulics evaluation of the planned operation and a study of specific gates and valves are recommended. Gates and valves of this nature are a specialty construction item.

3.3 E-65 Canal and Siphons to Elwood Reservoir

3.3.1 Existing E-65 Canal and Operation

The E-65 Irrigation Canal was built in the 1930s and 1940s to supply local farmers with irrigation water. The canal is currently owned and operated by CNPPID. The E-65 Canal system has a current capacity of 350 cfs between the Canal's origin at the Tri County Supply Canal and the Elwood Reservoir intake works. The primary flow capacity restrictions are due to the approximately 7,500 feet of siphons along the E-65 Canal upstream of Elwood Dam. Beyond Elwood Reservoir, the E-65 Canal system has a capacity of 650 cfs. The existing E-65 Canal alignment is shown in Figure 3.3-4.

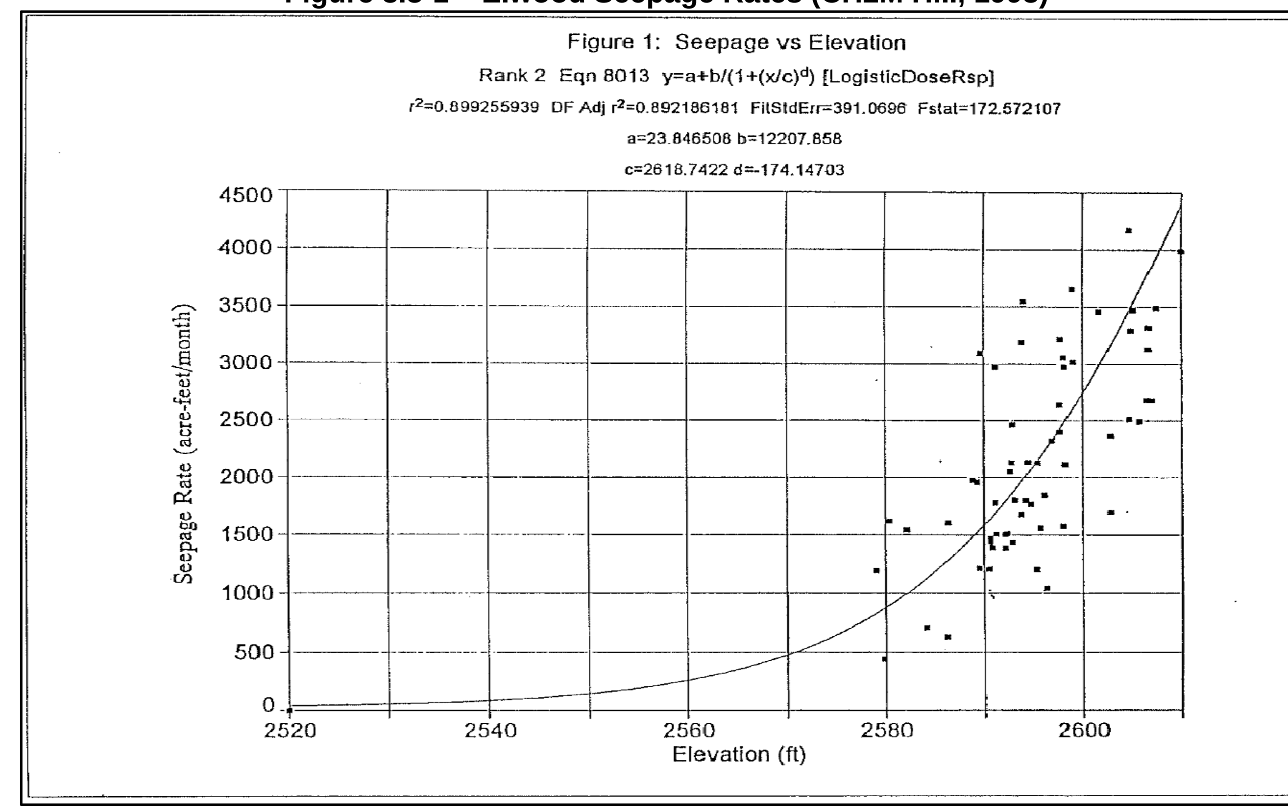
For the purposes of this evaluation, CNPPID stated that the E-65 canal would be available to deliver Program water to Elwood Reservoir for the period of September 1 through March 7 or 15 at the full capacity. CNPPID starts to fill Elwood on March 7 or 15. Starting in April, approximately 150 cfs is needed for flushing, surcharge, and weed control but the remainder of the canal capacity is conveyed to Elwood. The canal is available during the winter months, however, likely at a reduced capacity. In the past, CNPPID has allowed the canal to ice over and has run water under the ice.

3.3.2 Elwood Target Operating Curve Modifications

Elwood Reservoir was constructed in the late 1970s to provide additional irrigation water to the E-65 Canal. Elwood Reservoir is currently owned and operated by CNPPID. As stated above, the E-65 Canal has a capacity of 650 cfs downstream of the Elwood Dam and Reservoir. The E-65 Canal has a capacity of 350 cfs upstream of the Elwood Dam and Reservoir. The Elwood Dam and Reservoir is used to supplement 300 cfs during peak irrigation demand.

In 1993, a study of seepage rates was conducted, and opportunities for optimum operation of Elwood Reservoir were investigated. The seepage rates at Elwood are an exponential function in relation to the pool level of the reservoir. The seepage rate of Elwood Reservoir is shown in Figure 3.3-1, taken from the CH2M Hill memorandum.

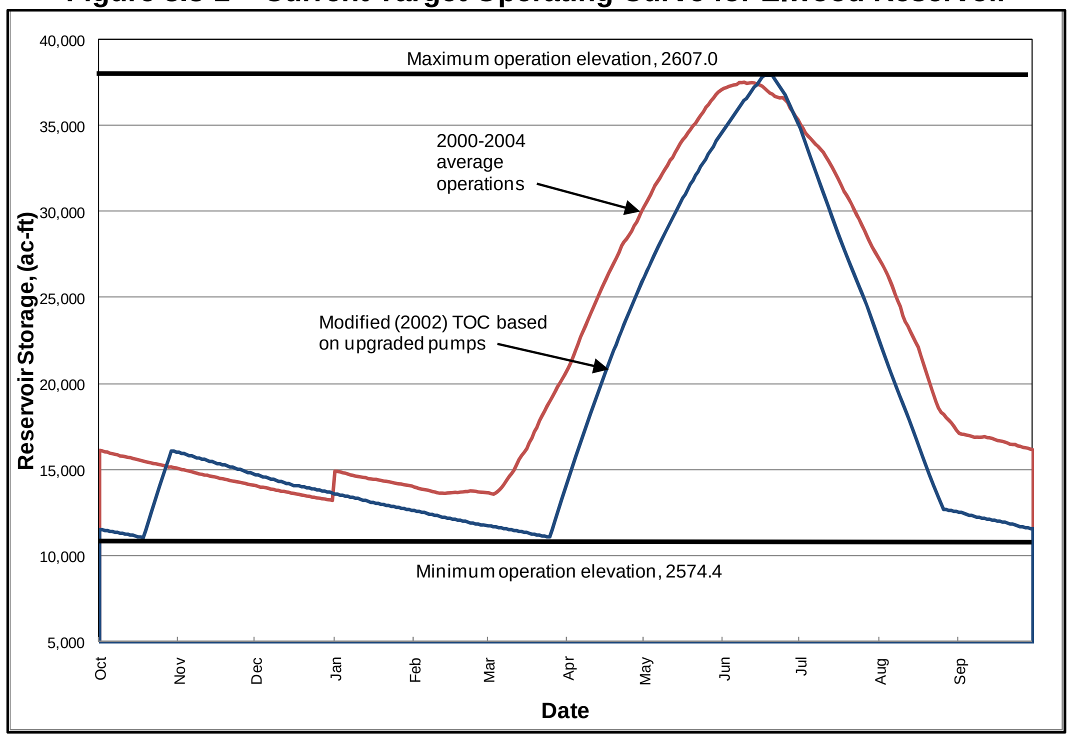
Figure 3.3-1 – Elwood Seepage Rates (CH2M Hill, 1993)



Seepage from Elwood Reservoir flows in to both the Republican River basin, and the Platte River Basin. CH2M Hill's 1993 memorandum defined the seepage rates from Elwood into both of these basins. The seepage leaving Elwood towards the Republican Basin includes the seepage south (24%), and the seepage west (29%). The total seepage from Elwood that would be expected to leave the Platte River basin, and flow towards the Republican River basin is 53% of the expected seepage from the reservoir.

A target operating curve (TOC) was developed to reduce the amount of seepage losses from the reservoir (CH2M Hill, 1993). Based on the standard TOC, Elwood Reservoir begins to fill around March 15 and continues to fill through June 15. Recently, the pump motors were replaced, which allows for a quicker fill rate than shown in the 1993 TOC study. The TOC as modified after replacement of the pump motors is shown in Figure 3.3-2. It entails starting filling operations on March 25. The pumps are able to move 190 cfs to 270 cfs. Irrigation water is released from mid-June to September. Elwood Reservoir is partially filled again each fall to account for winter seepage if needed. Based on the recent drought conditions and the unavailability of water supplied to CNPPID over the past five years, Elwood has not recently been used to supplement irrigation water. Figure 3.3-2 shows a comparison of the average operation for 2000-2004 and the current TOC.

Figure 3.3-2 – Current Target Operating Curve for Elwood Reservoir



3.3.3 Inlet Supply Alternatives

Three supply alternatives for conveying water into Elwood Reservoir were evaluated. The three alternatives include utilization of the existing supply system, a new gravity supply canal, and a twin E-65 Canal.

3.3.3.1 Inlet Supply Alternative No. 1 – Use Existing System

As previously stated, E-65 has a capacity of 350 cfs from the Tri County Supply Canal to Elwood Reservoir. The three existing vertical turbine pumps can deliver up to 270 cfs combined. In order to use the existing system without any modifications, filling of Elwood would have to begin in early February. It would take 37 days to fill Elwood with the approximately 14,300 acre-feet of water needed to release 2,400 cfs for three days. The filling rate takes into account an average seepage loss of 70 cfs. Based on comparisons of the fill and release to the TOC, it appears that independent operation of Elwood by the PRRIP and CNPPID is possible. In other words, the reservoir could be filled and emptied for use by the PRRIP prior to its use for irrigation purposes by CNPPID (this would still require construction at conveyance facilities).

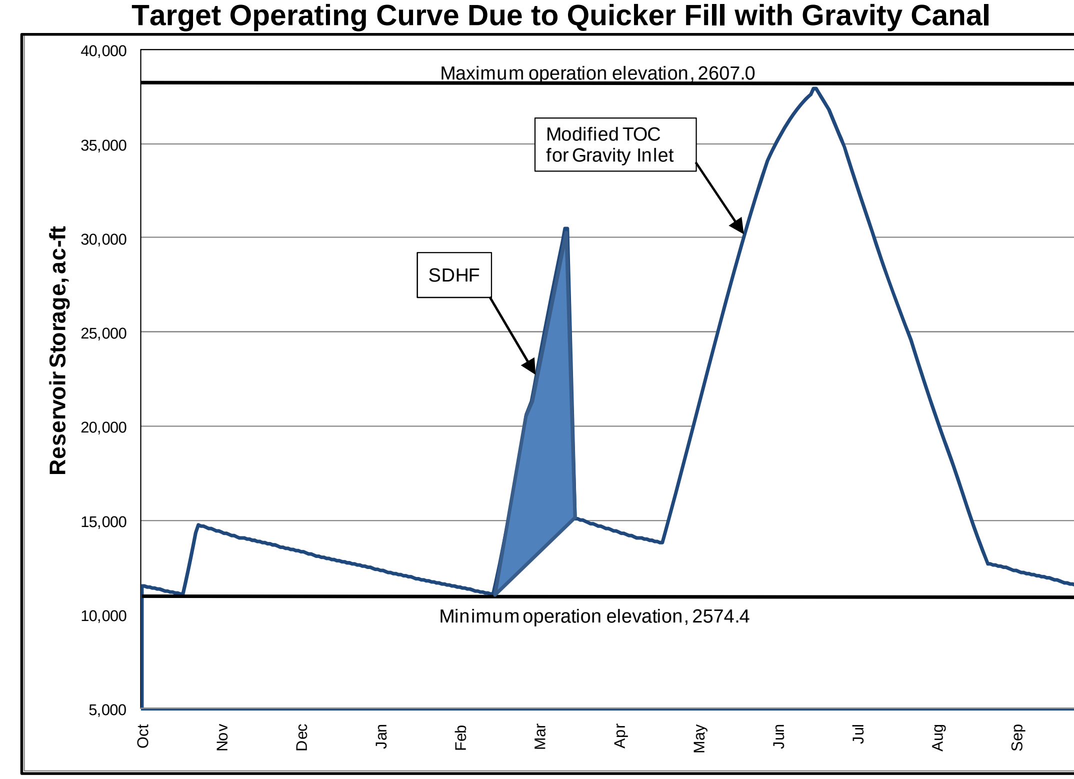
3.3.3.2 Inlet Supply Alternative No. 2 – Gravity Supply Canal

Due to the existing uses and limitations of the E-65 Canal and Elwood pump station, a new gravity canal between Johnson Lake and Elwood Reservoir was investigated. A potential horizontal alignment for a gravity supply canal is shown in Figure 3.3-4.

A 350 cfs gravity canal could provide the capacity to fill Elwood beginning on February 15 for a March 15 release. The Elwood gravity supply canal would include an 8-foot diameter 5,000 feet long siphon structure across Plum Creek. A plan and profile view of the Elwood gravity supply canal is shown in Figure 3.3-5, and typical cross sections are shown in Figure 3.3-6.

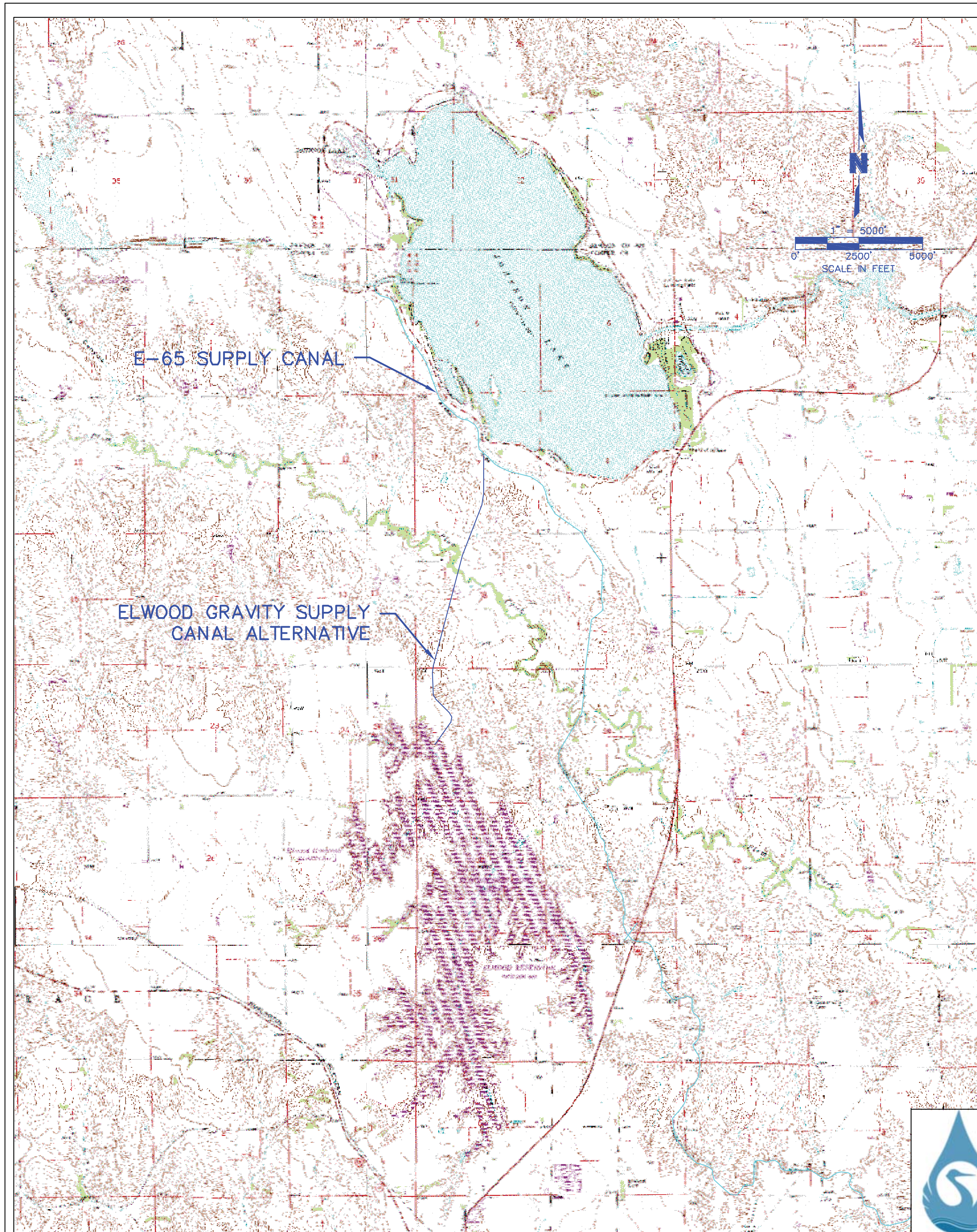
The Elwood gravity supply canal would have an invert elevation of 2,607 feet at Elwood Reservoir. This would allow the reservoir to store 37,000 ac-ft with no pumping cost. The Elwood gravity supply canal could be designed to deliver 350 cfs, which exceed s the existing supply system capacity of 270 cfs. With the increase in capacity, Elwood Reservoir could be filled more quickly than it can using the existing pumps 26 days rather than 37 days. A gravity canal capacity of 350 cfs was chosen to replace the capacity provided by the E-65 Canal. Figure 3.3-3 shows the SDHF superimposed on the TOC, which has been modified to reflect the faster filling due to the gravity canal. Potentially modifying the TOC, however, is not an endeavor to be underestimated.

Figure 3.3-3 – Suggested Elwood SDHF Operations Superimposed on Proposed Modifications to the Target Operating Curve Due to Quicker Fill with Gravity Canal



3.3.3.3 Inlet Supply Alternative No. 3 – Twin E-65 Canal

A twin E-65 Canal was evaluated from the Tri County Supply Canal to the Elwood Dam and Reservoir. In this alternative, E-65 Canal would have the capacity to deliver 650 cfs for irrigation usage. This would remove the CNPPID need for using the Elwood Dam and Reservoir to supplement irrigation water. Elwood Reservoir would then become available to the PRRIP year-round. This alternative opens the door for multiple uses of the Elwood Dam and Reservoir for delivering SDHF and target flows.



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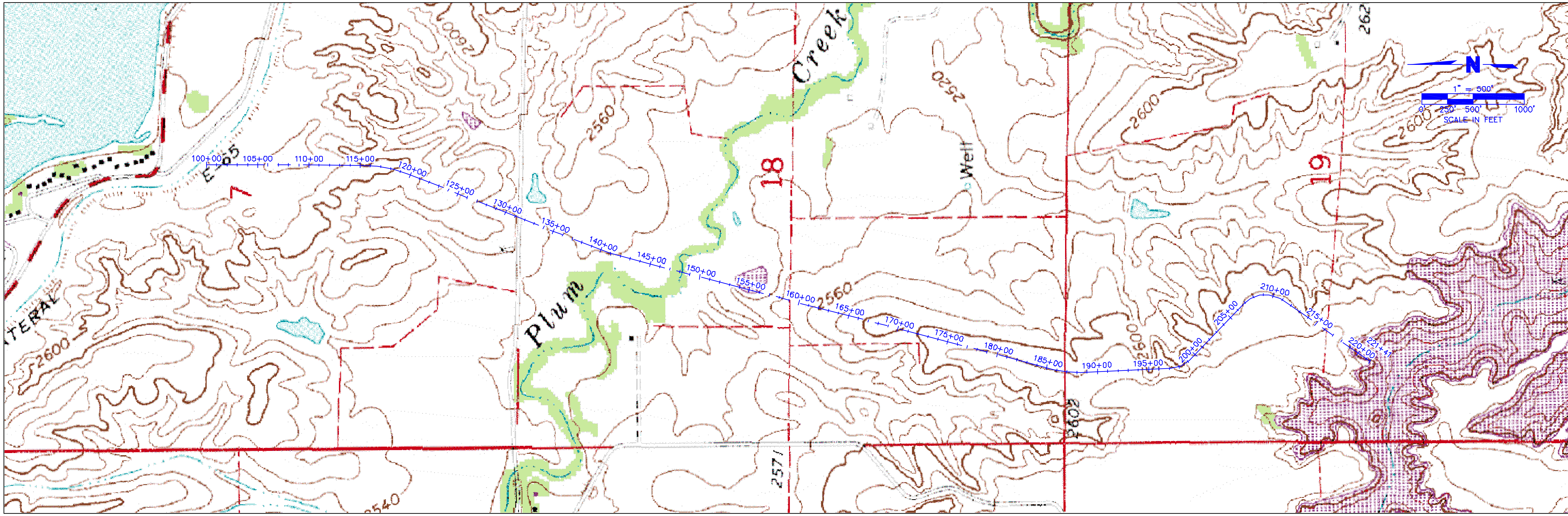
DATE: 10/01/09

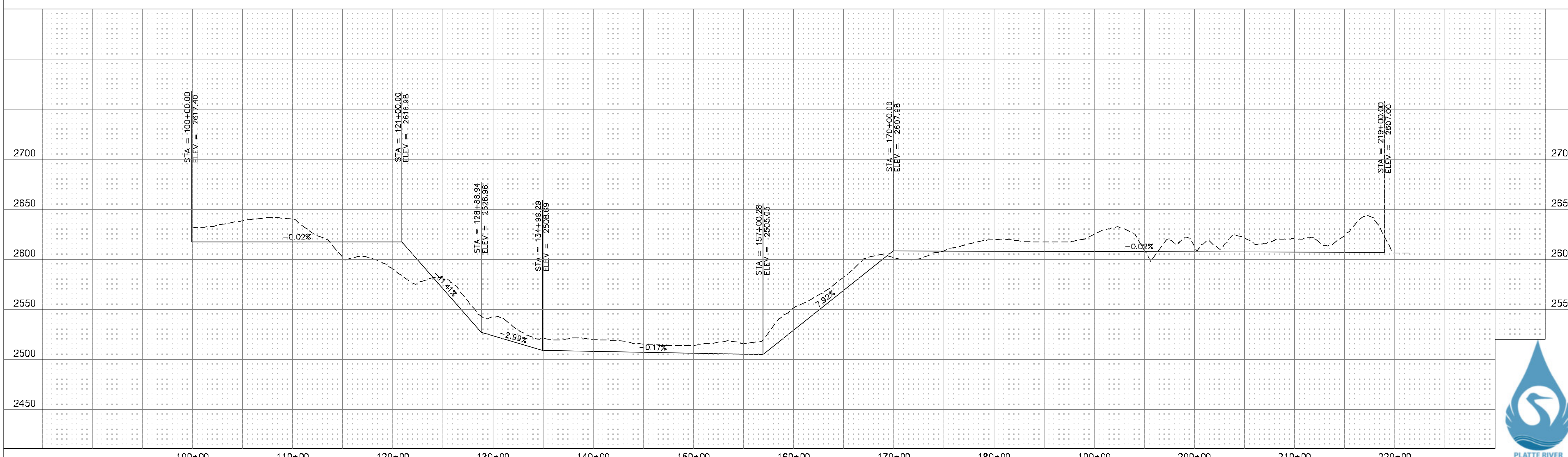
ELWOOD GRAVITY SUPPLY &
E-65 CANAL ALIGNMENT

OLSSON ASSOCIATES

FIGURE
3.3-4

USER: tgolka
TTLBLK-Elwood-22x34
F:\Projects\009-1466_MUNI\Exhibits\PRRIP-Elwood-Canal.dwg
XREFS: Johnson_Lake_QUAD
DWG:
DATE: Dec 21, 2009 9:56am





PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

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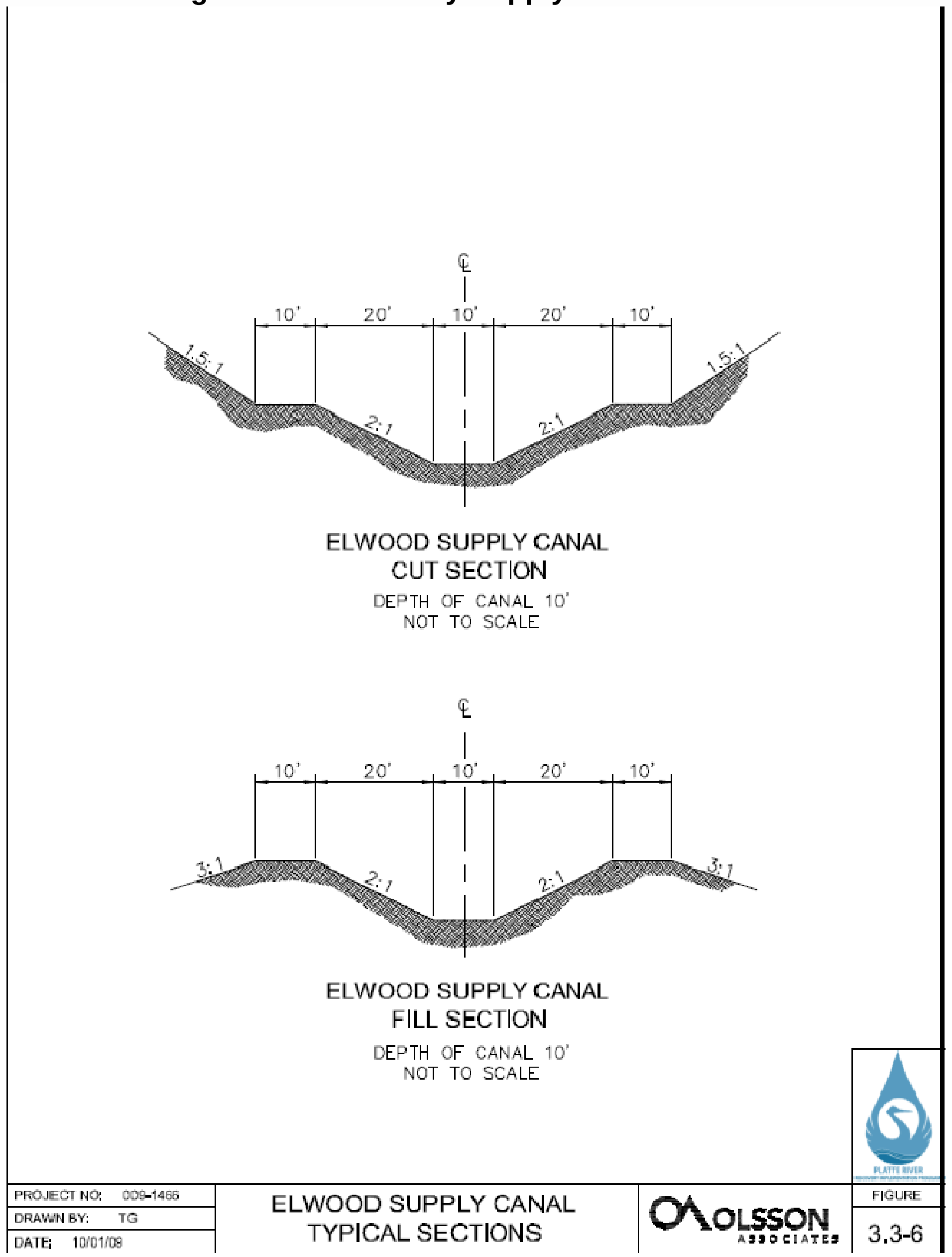
DATE: 10/01/09	

ELWOOD GRAVITY SUPPLY CANAL
PLAN AND PROFILE

OLSSON ASSOCIATES

FIGURE
3.3-5

Figure 3.3-6 – Gravity Supply Canal Sections



3.4 Conveyance to the Platte River

Three alternatives were evaluated for the conveyance of water from Elwood Reservoir to the Platte River for SDHF augmentation flows and target flow operations. These alternatives were:

- Use of Plum Creek
- Construction of a New Canal – WMS Phase II Alignment
- Construction of a New Canal – New Alignment

3.4.1 Conveyance Alternative No. 1 – Use of Plum Creek

Historic stream flow data is available for select years from the U.S. Geological Survey and Nebraska Department of Natural Resources (NDNR) for Plum Creek near Smithfield, Nebraska. The gage is located nearly halfway between Elwood Reservoir and the Platte River. The drainage area at the gage is 209 square miles. The drainage area near the confluence of Plum Creek with the Platte River is approximately 234 square miles, according to construction plans for Bridge RS-1550 (4) over County Road 749. Instantaneous peak flow information is available from the USGS for water years 1946-1978 and 1996-1999. Instantaneous peak flow data was obtained from the NDNR for water years 1981-1991 and 2003-2008.

Instantaneous peak flow data showed that the highest peak flow of 2,800 cfs was recorded on June 23, 1947. Six of the years had a peak flow higher than 1,000 cfs. The instantaneous peak flow average of the remaining years is 332 cfs. Table 3.4-1 lists the peak flows.

Table 3.4-1 – Historical Instantaneous Peak Flows for Plum Creek Gage at Smithfield, Nebraska, USGS Gage 06767500, Latitude 40°38'29" and Longitude 99°42'38"

Date	Peak Flow, cfs	Date	Peak Flow, cfs
6/23/1947	2,800	6/12/1974	15
6/23/1948	2,230	6/22/1975	462
6/6/1949	1,220	4/9/1976	143
5/30/1950	404	5/22/1977	323
6/10/1951	588	3/11/1978	270
5/27/1952	90	7/28/1981	130
5/10/1953	18	8/14/1982	44
5/16/1954	220	5/18/1983	26
6/16/1955	196	7/5/1984	427
6/5/1956	116	9/6/1985	549
6/16/1957	844	5/10/1986	280
2/27/1958	259	6/11/1987	186
3/26/1959	175	7/19/1988	222
3/22/1960	620	6/25/1989	905
8/17/1961	470	8/12/1990	218
6/7/1962	562	9/7/1991	437
6/15/1963	558	5/27/1996	242
4/20/1964	156	8/13/1997	34
5/24/1965	985	7/30/1998	264
10/18/1965	865	6/28/1999	346
6/13/1967	1,320	5/24/2003	175
8/10/1968	938	7/10/2004	89
9/18/1969	1,140	6/3/2005	335
6/12/1970	355	9/11/2006	184
3/25/1971	17	8/23/2007	306
6/24/1972	242	5/24/2008	1,440
9/1/1973	332	---	---

The WMS Phase II report stated that a high flow event occurred on Plum Creek in May 2008, which resulted in significant damage and flooding. According to the data provided by NDNR, the peak flow was 1,440 cfs. It is estimated that this peak flow represents the approximate 20-year return period peak flow based on an analysis of gaged data and regression equations. CNPPID reported that this flood event caused approximately $100,000 in damage to one of its siphons under Plum Creek.

3.4.1.1 Hydraulic Analysis of Plum Creek Channel

A Hydrologic Engineering Center-River Analysis System (HEC-RAS) model of Plum Creek was developed to serve three main purposes – to evaluate the flow attenuation between Elwood Reservoir and the Platte River, to evaluate the effect of a 2,000-cfs flow on the existing Plum Creek channel, and to estimate a flow rate that can be conveyed by the Plum Creek channel without causing damage. Cross-sections were developed using the USGS topographic quadrangle maps, which have contour intervals of 10 feet and 20 feet in the project area. Cross-sections were developed at an interval of approximately 1 mile. Only open channel cross-sections were entered, not bridges or culverts, due to the lack of survey data. Due to the lack of reliable topographic data, the model itself and results should be considered with an appropriate level of uncertainty.

3.4.1.2 Flow Attenuation

An unsteady HEC-RAS model was developed to evaluate flow attenuation. The model showed that it is necessary to release approximately 2,010 cfs at the upstream end of Plum Creek to achieve a flow of 2,000 cfs at the downstream end of the reach. Based on past experience, this result does not seem realistic and is probably due to the paucity of topographic information. Operational losses of 10% to 20% are considered reasonable assumptions, so a release of 2,400 cfs from Elwood Reservoir was considered to be desirable at this level of study.

3.4.1.3 Effect of SDHF on Existing Plum Creek

The effect of flows on the order of 2,400 cfs flow through Plum Creek was evaluated with a steady flow HEC-RAS model, along with flows of 2,000 cfs, 1,200 cfs, and 400 cfs. The resulting top widths, compared to the channel on the USGS maps, indicate that the flow would stay in the main channel in most cases. Average channel velocities and maximum channel velocities based on a velocity distribution within the cross-sections were checked. For the flows exceeding 2,000 cfs, slightly more than half of the cross-sections showed velocities exceeding 5 ft/sec. For the 1,200 cfs and 400 cfs flows, the cross-sections showing velocities exceeding 5 ft/sec decreased from 44% to 15%, respectively. Froude numbers indicate subcritical (stable) conditions. A summary of the modeling results is contained in the appendix. Given the inaccuracies in the topographic information from which the cross-sections were developed, the lack of representation of the channel meanders, and reported damage during the May 2008 storm event of 1,440 cfs, it is evident that flows of 2,000 cfs or more cannot be conveyed without scour or channel degradation.

The hydraulic design function of HEC-RAS was used to develop estimated stable channel cross-sections for flows of 2,400 cfs and 1,200 cfs. The average longitudinal slope of Plum Creek is approximately 0.1%. Three soil samples were collected from the banks of Plum Creek upstream of County Road 746, County Road 749, and State Highway 283. Particle size distributions were computed from the samples. Particle size parameters used in the stable cross section analysis were d_{84} of 1.7 mm, d_{50} of 0.25 mm, and d_{16} of 0.01 mm.

The computed stable cross-sections were determined to have bottom widths of 12 feet and 6 feet for 2,400 cfs and 1,200 cfs, respectively, assuming side slopes of 10H:1V. The relatively flat side slopes were selected to fit better with the available cross-section information. If this alternative were advanced to more detailed design, a composite channel with the existing low-flow section with steeper side slopes would be designed to reduce the overall channel width and assist with permitting. The fully excavated channel used for this analysis due to rough topographic data would have difficulty being permitted. Most of the cross-sections showed that some enlargement was needed, partly due to the fact that the cross-sections developed from the topographic mapping had one point for the thalweg, as opposed to having an actual bottom width. Excavation quantities to construct a stable channel were estimated to be 675,000 cubic yards and 325,000 cubic yards for the 2,400 cfs flows and 1,200 cfs flows, respectively.

In addition to channel enlargement, armoring of the outside bank meanders would be required to protect the side slopes from erosion and headcutting. The entire channel between Elwood Reservoir and the Platte River is approximately 27 miles long. It is estimated that 30% to 40% of the channel is made up of meanders that would require armoring. If 35% of the channel needed armoring on only one side, the outer bend, almost 10 linear miles of armoring would be needed for the 2,400 cfs flow.

Degradation of Plum Creek may provide beneficial sediment to the Platte River. The upper reaches are comprised of silty loess material that would not be beneficial for habitat creation. The lower reaches, however, are comprised of silty sandy material that might provide suitable material. With improved model accuracy, estimates of sediment yield to the downstream system could be developed. Degradation also may result in undercutting and scour of hydraulic structures. In some cases, the sediment yield from channel degradation may create deposition that clogs culverts or hydraulic structures at roadway crossings of the Plum Creek channel.

3.4.1.4 Plum Creek Existing Crossing Structure Capacities

Between Elwood Reservoir and the Platte River, Plum Creek is crossed by roads at 10 locations. The crossings range from twin 60-inch diameter corrugated metal pipe (CMP) culverts to a three-span bridge, which is 119 feet long. Construction plans were obtained for all seven of the bridges. The plans for four of the structures included design information, such as the design discharge or the 100-year discharge. The discharges ranged from approximately 3,000 cfs to 5,500 cfs. The “special plan” sheets that include the hydraulic design information were not included for the other three structures. The unit discharge was estimated for the four bridges with known design capacities on the basis of discharge per foot of bridge length. The unit discharges ranged from 43 cfs to 76 cfs per foot. To conservatively estimate the capacities of the three bridges, a unit discharge of 40 cfs per foot of bridge length was used. The resulting discharges ranged from 2,600 cfs to 3,600 cfs. Based on this information, the bridges appear to have capacity to convey 2,400 cfs. The backwater effects of the bridges were not evaluated due to the lack of accuracy in the cross-sections.

The remaining three structures, one bridge and two culvert crossings, were inventoried by Olsson staff. Estimates of the culvert capacities were developed using the Federal Highway Administration’s HY-8 program. The capacities were determined at a point of imminent overtopping of the road and were determined to be 608 cfs for the County Road 430 culvert and 316 cfs for the County Road 437 culvert. The culverts would need to be upgraded to bridges or box culverts to convey 2,400 cfs. The County Road 746 Bridge was analyzed using Bentley Systems’ FlowMaster program. Based on the open area and low chord of the bridge, the capacity was estimated to be 2,400 cfs. Photos of all of the structures and additional areas of interest are included in Appendix B.

Table 3.4-2 shows a summary of all the structures and their capacities.

Table 3.4-2 - Plum Creek Bridge and Crossing Capacities

Structure #	Structure ID	Location	Description	Capacity	Source/Comment
1	STP-BR-BH-283-1(106)	SH 283	1-48' and 2-35'-9" spans (119' total)	5500 cfs (Q_{100})	Construction Plans
2	BR-7037 (14)	CR 429	80' span	3200 cfs (approximate)	Estimated, missing Special Plan (Sheet 8)
3	---	CR 430	1-8' CMP	608 cfs	HY8, at top of road - imminent overtopping
4	BR-7037 (17)	CR 432	1-36' and 2-27' spans (90' total)	3600 cfs (approximate)	Estimated, missing Special Plan (Sheets 14-18)
5	RS-BRS-1695(3)	CR 433	59'-2" span	4300 cfs (design flood)	Construction Plans
6	C003731005	CR 746	29' span, average 12.05' height	2400 cfs	Flowmaster, at low chord
7	BR-7037 (15)	CR 435	1-26' and 2-19'-6" spans (65' total)	2600 cfs (approximate)	Estimated, missing Special Plan (Sheets 9-13)
8	BR-7037 (7)	CR 436	110' span	4700 cfs (Q_{100})	Construction Plans
9	---	CR 437	2-60" CMPs	316 cfs	overtopping
10	RS-1550 (4)	CR 749	1-28' and 1-21' span (49' total)	<3000 cfs	Construction Plans, Q_{100} = 5200 cfs, 3000 is overtopping

Crossings with less than 2000 cfs capacity. Upgrade required to convey 2,400 cfs.
Upgrade or replacement expected to convey 2,400 cfs

In addition to replacing two culvert crossings, some of the bridges will require enlargement or replacement to convey a 2,400 cfs flow without significant backwater effects. Although the capacity of the County Road 749 Bridge was documented to be almost 3,000 cfs on the as-built drawings, the May 2008 flow of 1,440 cfs caused scour and undermining of the bridge, as shown in Photo 3.4-1.

Photo 3.4-1 - County Road 749 Bridge over Plum Creek

Scour under the wingwall/ abutment apparent after May 2008 flood event

Riprap placed following the May 2008 flood event

County Road Bridge 749 should be replaced or enlarged, along with the County Road 746 Bridge, which was estimated to have a capacity of 2,400 cfs. Estimates of cost for replacement were developed on a square-foot basis for a concrete slab girder bridge.

Estimating the flows that could be conveyed in Plum Creek, under existing conditions without causing significant erosion and flooding cannot be adequately determined with the available topographic information. Developing a HEC-RAS model using the Light Detection and Ranging (LIDAR) mapping or select field survey information as base mapping is recommended to determine the appropriate range of flows. The model should incorporate the crossing structures to assess backwater conditions. Two significant flow rates worthy of mention, are the 2008 flooding at a rate of 1,440 cfs, which caused significant damage, and the average flow of 332 cfs, which is known to cause no damage. This information together with the capacities of existing structures gives us a suggested maximum capacity of 600 cfs, which would require replacement of the County Road 437 culvert and the County Road 430 culvert. When considering the combined Elwood/J-2 option discussed below, there likely will be an optimal flow rate trade-off with capital costs. If the combined case goes forward to feasibility, more work will be needed to determine this optimal flow rate. For use with the combined alternatives, a conveyance capacity in Plum Creek of 1,200 cfs was assumed.

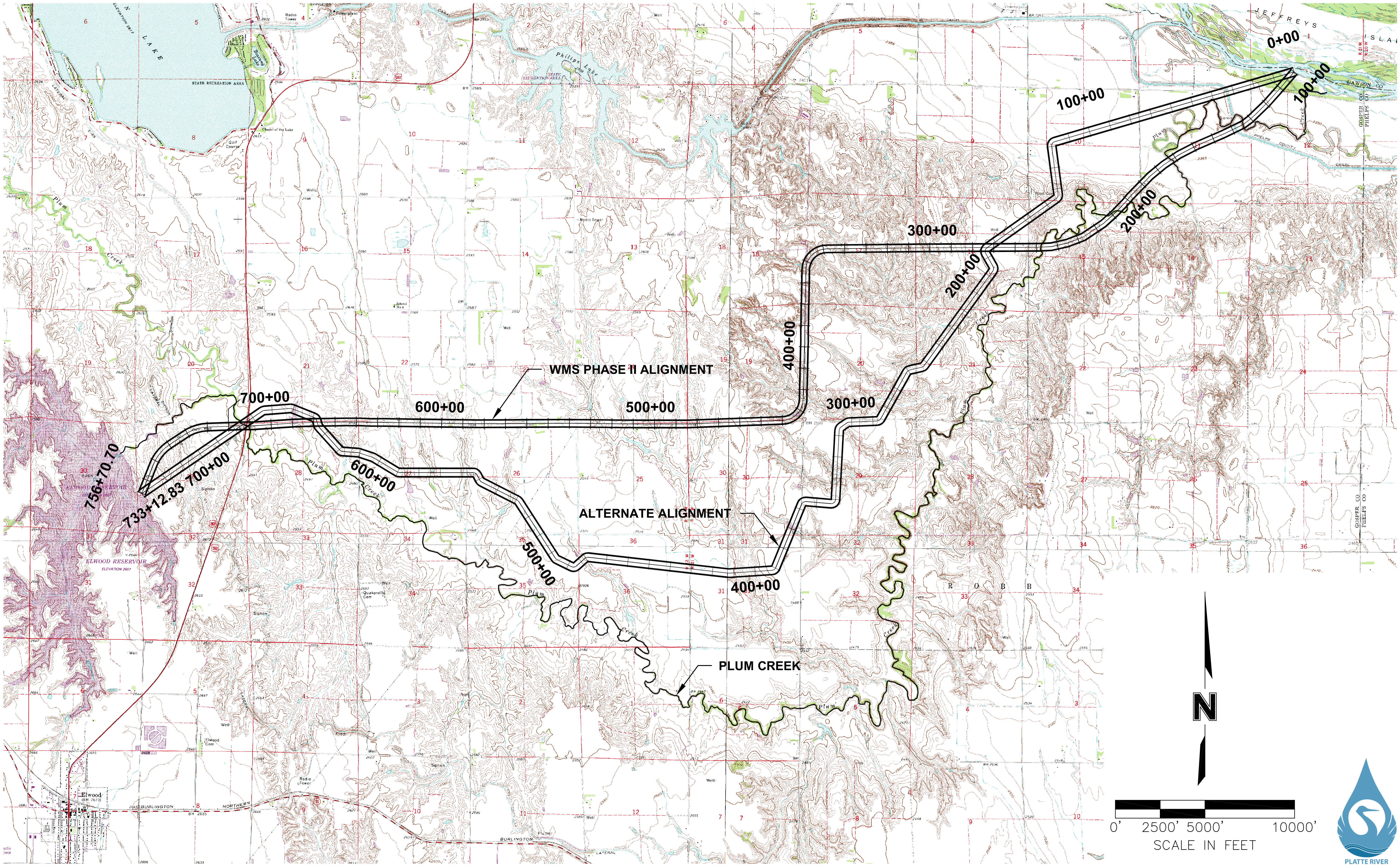
3.4.2 Conveyance Alternatives No. 2 and No. 3 - New Return Canals

Two potential alignments for a return canal to deliver a SDHF augmentation of 2,000 cfs between Elwood Reservoir and the Platte River were evaluated. To reduce the channel width, the cross-sectional geometry consisted of side slopes of 2H:1V and a maximum depth of 10 feet. The bottom width and longitudinal slope of the canal was developed to result in a velocity less than 5 ft/sec, while minimizing the width of the canal. The resulting bottom and top widths were 24 feet and 64 feet, respectively, not including freeboard.

Using USGS topographic maps for base mapping, the alignment identified in the WMS Phase II Report (Boyle, 2008) with the cross-section developed for the 2,000 cfs SDHF was evaluated. The alignment is shown in Figure 3.4-1. Construction along the alignment would require a significant volume of excavation. The alignment, which is 12.4 miles long, crosses many tributaries of Plum Creek and Plum Creek itself, necessitating a combination of siphons under tributaries and Plum Creek and flumes to carry smaller tributary flows over the canal. Information developed by the Bureau of Reclamation indicated that two 12-foot diameter siphons would be needed to convey 2,000 cfs with acceptable head requirements (Bureau of Reclamation, 1978).

A conceptual profile was developed based on the WMS Phase II alignment and is shown in Figure 3.4-1. Approximately 7 million cubic yards of excavation and nearly 17,000 total feet of siphons would be required, resulting in an estimated cost of $50 million for this alternative, before contingencies. The cost does not include the flumes for tributaries, inlet/outlet transitions for the siphons, drop structures, land acquisition, or considerations for spoil areas. If this alternative advanced to more detailed design, a balance of siphons, flumes, excavations and fills to reduce costs would be needed. Adding contingencies of 50% to account for some of these items and the uncertainty of the topographic mapping brings the estimated cost of the alternative to $75 million. Based on the costs of the excavation and siphons, the economic feasibility of this conveyance alternative was considered to be highly unlikely and a canal along this alignment was not evaluated further.

An alternate alignment was developed in an attempt to decrease the cost by reducing the excavation quantity and siphon crossings. The alternate alignment and profile are shown in Figures 3.4-2 and 3.4-3, respectively. For this alignment, which is 13.9 miles long, the estimated excavation was approximately 7 million cubic yards, with roughly 8,400 linear feet of siphons. The cost of the excavation and siphons was estimated at $42 million, without inlet/outlet transitions for the siphons, flumes for tributaries, drop structures, land acquisition, or considerations for spoil areas, and before contingencies. Incorporating contingencies of 50% to account for some of these items and the uncertainty of the topographic mapping bring the estimated cost of the alternative to $63 million. Based on these costs, the economic feasibility of this conveyance alternative was considered to be highly unlikely and a canal along this alignment was not evaluated further.



PROJECT:	09-1466
DRAWN BY:	CAU
DATE:	10.01.09

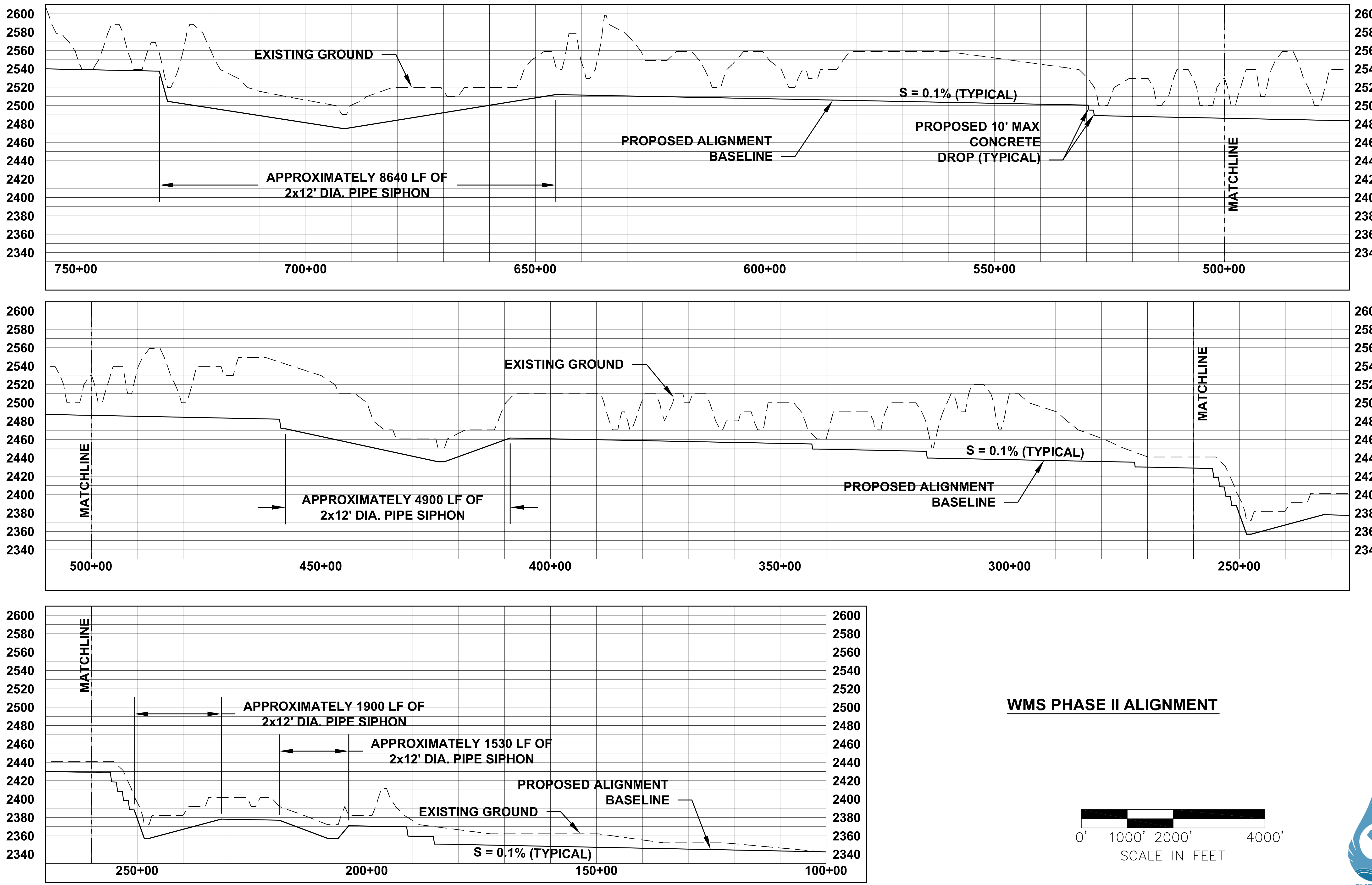
ELWOOD RESERVOIR SCREENING ALTERNATIVES
GOSPER COUNTY, NEBRASKA

OLSSON ASSOCIATES

CANAL ALIGNMENT ALTERNATIVES

FIGURE 3.4-1

DWG: F:\Projects\009-1466_WTRS\091466-DB\dwg\Exhibits\Ditch Alignment Exhibit.DWG USER: cuhrich
DATE: Dec 21, 2009 10:36am XREFS: Topo Title Block



PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

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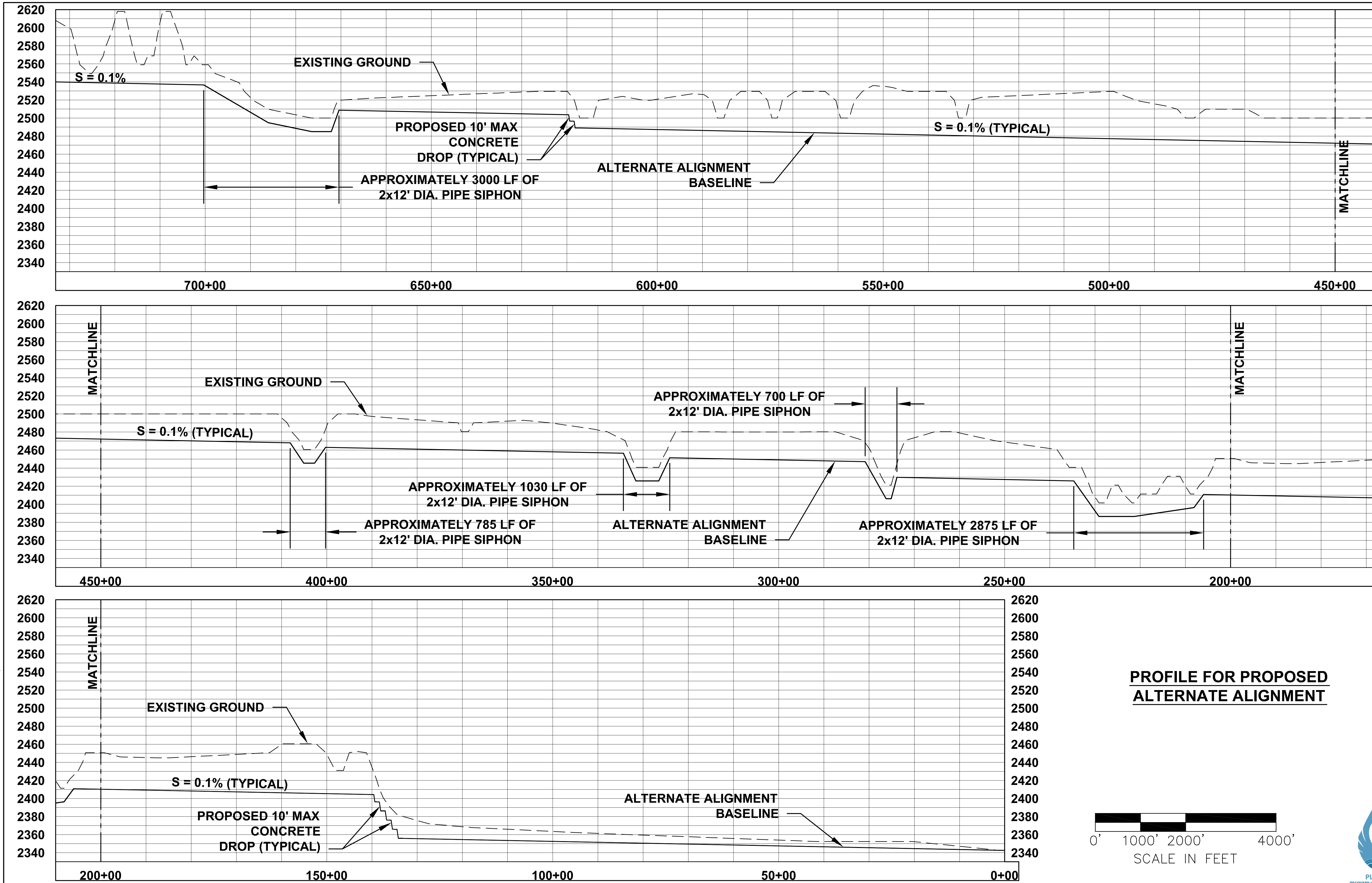
PROJECT: 09-1466	ELWOOD RESERVOIR SCREENING ALTERNATIVES GOSPER COUNTY, NEBRASKA
DRAWN BY: CAU	
DATE: 10.01.09	

OLSSON ASSOCIATES

WMS PHASE II CANAL PROFILE

FIGURE 3.4-2



PROFILE FOR PROPOSED ALTERNATE ALIGNMENT

0' 1000' 2000' 4000'
SCALE IN FEET

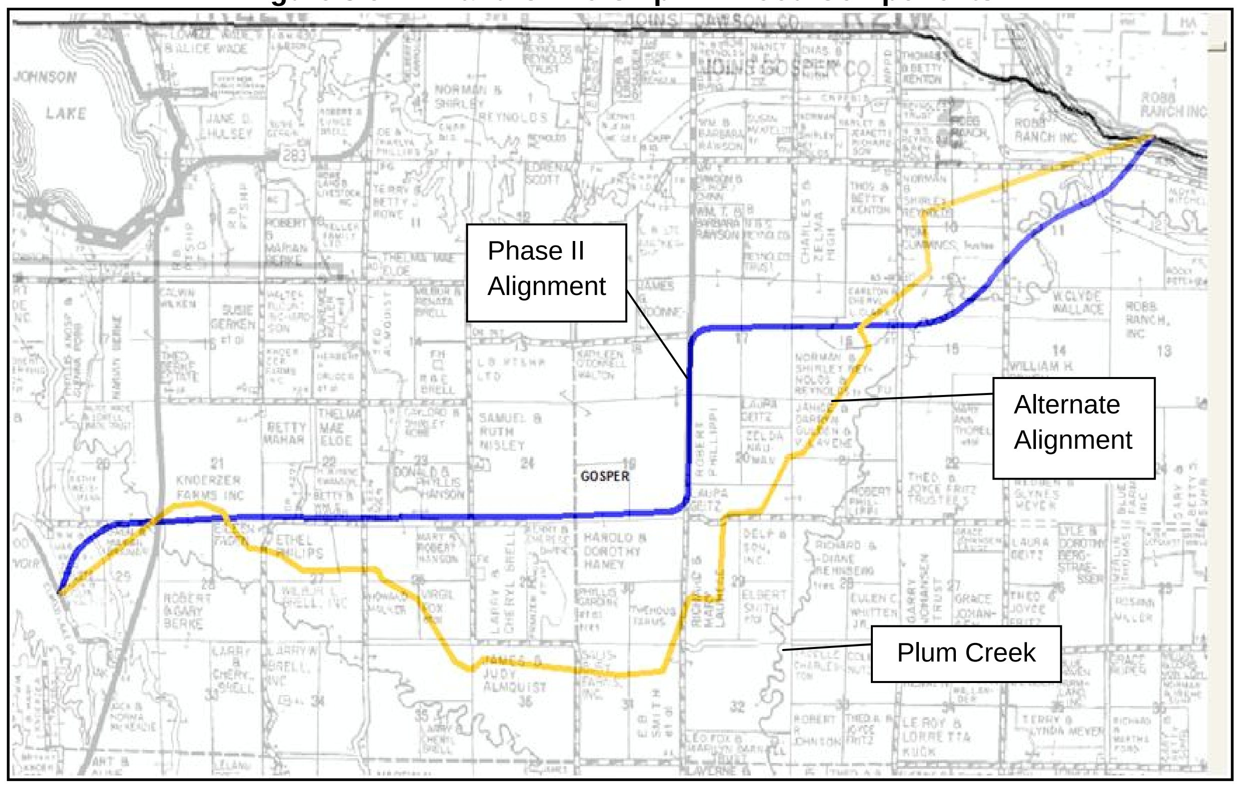
PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

PROJECT: 09-1466	ELWOOD RESERVOIR SCREENING ALTERNATIVES GOSPER COUNTY, NEBRASKA	OLSSON ASSOCIATES	ALT. ALIGN. CANAL PROFILE
DRAWN BY: CAU			FIGURE 3.4-3
DATE: 10.01.09			

3.5 Land Ownership

An investigation was done to determine the ownership of lands where proposed configuration elements for Elwood Reservoir would potentially be placed. Properties in Gosper and Phelps counties would be involved, including landowners along the potential gravity supply canal and Plum Creek, as well as along the return canals to the Platte River that were screened out of the analysis. The figure below shows the ownership along these proposed components based on landowner maps obtained from CNPPID. This figure also is included in larger format in Appendix B.

Figure 3.5-1 – Land Ownership – Elwood Components



3.6 Elwood Reservoir Operation

The use of Elwood Reservoir to store and deliver water to augment a SDHF and for target flow operations was evaluated for six different combinations of improvement components. Upgrades to the dam embankment to provide stability include buttressing of the upstream face, removing and replacing the upstream shell, including soil cement, and removing and replacing the entire embankment. If the entire embankment was removed, a new outlet structure could be constructed using open excavation techniques. Otherwise tunneling through the existing embankment may be required. Both construction of a new gravity canal to supply water to Elwood Reservoir and use of the existing E-65 Canal were evaluated. In all six alternatives, plus upgrading and armoring Plum Creek to convey a SDHF of 2,400 cfs, were included. Table 3.6-1 summarizes the six configurations of the Elwood upgrades that were advanced to the scoring evaluation.

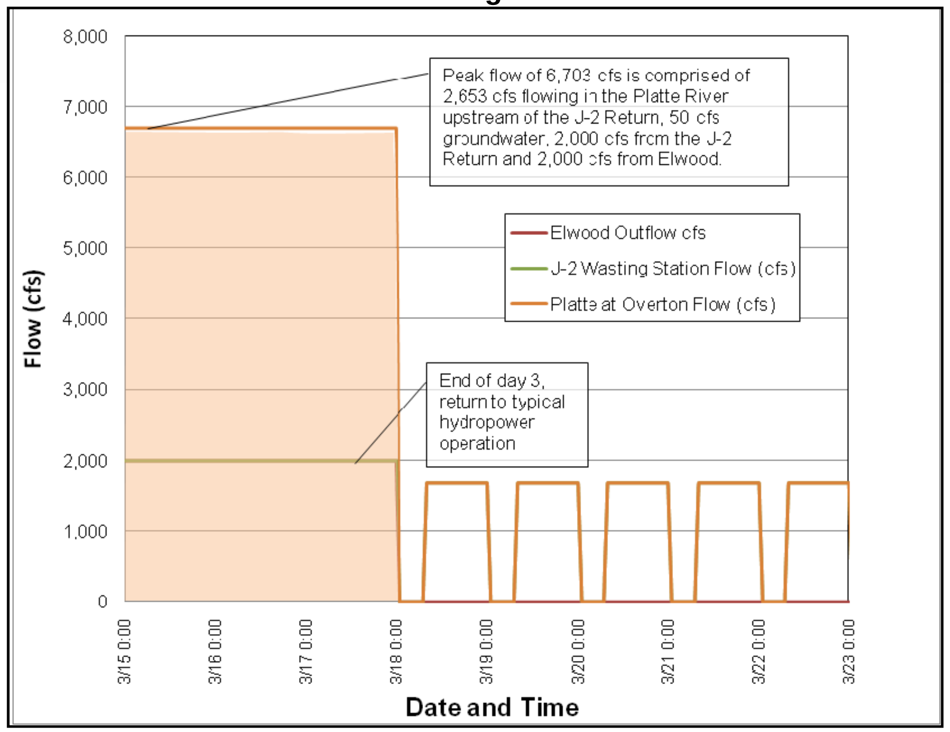
Table 3.6-1 – Elwood Reservoir Alternatives

Alternative	Embankment	Inlet	Outlet	Conveyance
E-1	Buttress upstream face of embankment	New Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs
E-2	Remove & replace embankment	New Gravity Canal	Open cut 2-8' pipes	Plum Creek, 2,400 cfs
E-3	Remove & replace upstream shell	New Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs
E-4	Buttress upstream face of embankment	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs
E-5	Remove & replace embankment	Existing E-65 Canal	Open cut 2-8' pipes	Plum Creek, 2,400 cfs
E-6	Remove & replace upstream shell	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs

3.6.1 Elwood SDHF Augmentation Results

As described in Section 1, Program actions will include implementation of SDHF events with flows of 6,000 cfs to 8,000 cfs for three days at Overton. The objective for this analysis was to create a SDHF of 2,000 cfs flow at Overton from Elwood Reservoir which would reach the lower end of the 6,000 to 8,000 cfs range. Based on losses along Plum Creek, it is expected that 2,400 cfs would need to be released from Elwood Reservoir for three days. Including an offset for seepage, a total of 14,280 ac-ft of water will be needed to be stored in Elwood Reservoir for this release, but the expected total volume of SDHF augmentation is 11,901 ac-ft. Figure 3.6-1 shows the inflow and outflow for the SDHF using Elwood Reservoir. The volume of SDHF augmentation delivered to the Overton gage is based on twin 8-foot diameter outlet pipes installed through the Elwood embankment and upgrading of Plum Creek to convey the flow. Additional flows from the Platte River, J-2 Return, and groundwater inflow are indicated to result in a 3-day SDHF Overton flow of 6,703 cfs.

Figure 3.6-1 – Elwood Reservoir Short Duration High Flow Outflow for Alternatives E-1 through E-6



The Operations criteria states: "For purposes of meeting a SDHF, reregulating reservoirs can be filled with either excess flows or EA water released from Lake McConaughy. Reservoirs will be filled as quickly as possible prior to the SDHF event, limited only by system and reservoir capacity constraints." Assuming the filling of Elwood is accomplished with EA water released from Lake McConaughy, the operations curve on which the SDHF operation has been superimposed, Figure 3.3-3 in Section 3.3, can be achieved using the existing E-65 Canal or a new gravity inlet canal.

It would take 37 days to fill Elwood for the SDHF, using the E-65 Canal, and during this time, approximately 1,863 ac-ft will seep from the reservoir. Using the CH2M Hill reported figure of 53%, the amount of water seeping into the Republican River basin would be 987 ac-ft of water for each year. Seepage that travels from Elwood Reservoir to the Republican River basin would not decrease; it would likely increase.

3.6.2 Elwood Target Flow Analysis Results

Table 3.6-2 shows a summary of how shortages to target flows could be reduced with the use of Elwood Reservoir alternatives E-1 through E-6. These are best case numbers based on many assumptions. More detailed analysis may reduce the volume of reductions to shortages to target flows since additional items such as conveyance losses along the E-65 Canal and evaporation from Elwood Reservoir were not considered in the analysis. A maximum available storage volume of 26,899 acre-feet was determined from the volume available in Elwood Reservoir between the minimum and maximum elevations shown on the TOC, 2574.4 and 2607.0, respectively (see Figure 3.3-2). The availability of the E-65 Canal was based on information provided by CNPPID and described in Section 3.3. Full capacity of the E-65 Canal was assumed for September 1 through March 15, with no available capacity for Program water the remainder of the year. The March 15 date was agreed upon by the workgroup, however, further discussions with CNPPID revealed that filling of Elwood Reservoir could begin either March 7 or March 15. For the target flow analysis for the illustrative years, filling of Elwood Reservoir via the E-65 Canal was completed prior to the beginning of March. Future analyses should assume no E-65 Canal capacity for the Program starting March 7. The 350 cfs capacity is the maximum potential capacity. It was assumed that the E-65 Canal could be run during the winter, but at a lesser capacity due to potential icing conditions. Because the capacity into Elwood Reservoir is limited by the existing pumps, the capacity in the canal was not decreased during the target flow operations. If further studies are conducted, assumptions using lesser capacities may be desirable. It should be noted that any excess flows routed to Elwood Reservoir have power bypass costs to the Program associated with them as water routed to Elwood does not flow through theJ-1 or J-2 hydropower stations. Power bypass costs for Elwood alternatives are included in total Elwood costs and are reflected in scoring.

Table 3.6-2 Elwood Reductions to Shortages to Target Flows Summary

	Dry Year 1964	Normal Year 1975	Wet Year 1986
Target Flow Shortages, ac-ft	266,711	540,654	227,917
Elwood Alternative	Reductions to Shortages to Target Flows, ac-ft		
E-1	19,162	17,788	21,330
E-2	19,162	17,788	21,330
E-3	19,162	17,788	21,330
E-4	19,154	19,408	21,736
E-5	19,154	19,408	21,736
E-6	19,154	19,408	21,736

For Elwood alternatives E-1 through E-3, inflows into Elwood Reservoir were limited to the new gravity inlet canal capacity of 350 cfs. For Elwood alternatives E-4 through E-6, which use the existing E-65 Canal, the inlet pump capacities limited the potential inflow into Elwood Reservoir. The maximum pump capacity is 270 cfs, however discussions with CNPPID at the beginning of the project indicated that for the last 10,000 acre-feet of storage, the pump capacity is 190 cfs. A relationship between full pump capacity of 270 cfs at the minimum TOC operating elevation/storage volume and the reduced pump capacity of 190 cfs at the elevation/storage volume at 10,000 acre-feet less than maximum TOC operating elevation/storage volume was developed. As the storage volume changed, so did the pump capacity. Pump capacity of 190 cfs was used for the highest 10,000 acre-feet.

Outflows were limited to the minimum of either the 2,400 cfs SDHF value since the alternatives assumed full upgrade of Plum Creek to convey the SDHF or the target flow shortage increased by a factor of 20% to account for transmission losses occurring in Plum Creek so that the full shortage would be delivered to the Platte River. A daily water balance was calculated and included seepage losses that were based on the seepage equation shown in Figure 3.3-1 and documented in the 1993 CH2M Hill memorandum (CH2M Hill, 1993). Evaporation from Elwood was not included in the target flow analysis, and will reduce the overall volumes. Target flow analysis was completed independently of the SDHF analysis. A transmission loss factor of 20% was applied to the released water as the last step in the analysis to reflect that not all water released will be conveyed to the Platte River.

Figures 3.6-2 through 3.6-4 show the filling and release of water for target flow operations based on the assumptions described. It was assumed that only storage above the TOC was available for target flow operations. Only the release of the main excess storage that takes place over the winter months was included as potential reduction to shortages to target flows. The potential exists for storage and release in other months, but these volumes were not included in the reported reduction volumes. In all illustrative years, the excess flows filled the reservoir over the winter. How quickly the reservoir filled and how long it stayed full until releasing the water varied, as well as the ending date at which the releases intersected with the TOC and ended the releases. In the wet year and perhaps the normal year, flows could be captured in September to supply reductions to shortages to target flows in October. Due to the fact that multiple years were not run, this potential volume was not included in the reported reduction volumes.

Comparison of Figures 3.6-2 and 3.6-3 indicate that less water is delivered during the illustrative normal year than during the illustrative dry year, which is not intuitive. Because the normal year has higher target flows, there are fewer excesses with which to fill Elwood Reservoir. It is noted that Elwood Reservoir does not fill to maximum during the normal illustrative year. The shortages to target flows are also greater than in the dry year, so the water is released more quickly. In the wet years, the shortages are smaller and the excesses more plentiful.

Figure 3.6-2 – Elwood Reservoir PPRIP/CNPPID Joint Operations 1964 Illustrative Dry Year

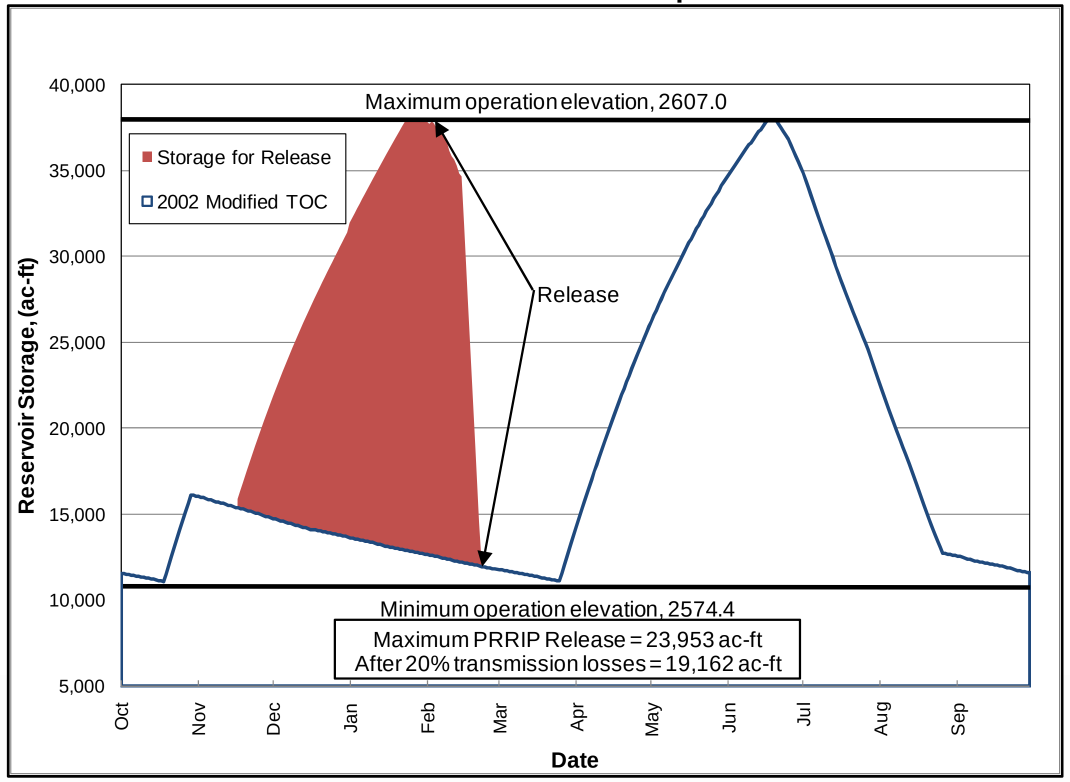


Figure 3.6-3 – Elwood Reservoir PRRIP/CNPPID Joint Operations 1975 Illustrative Normal Year

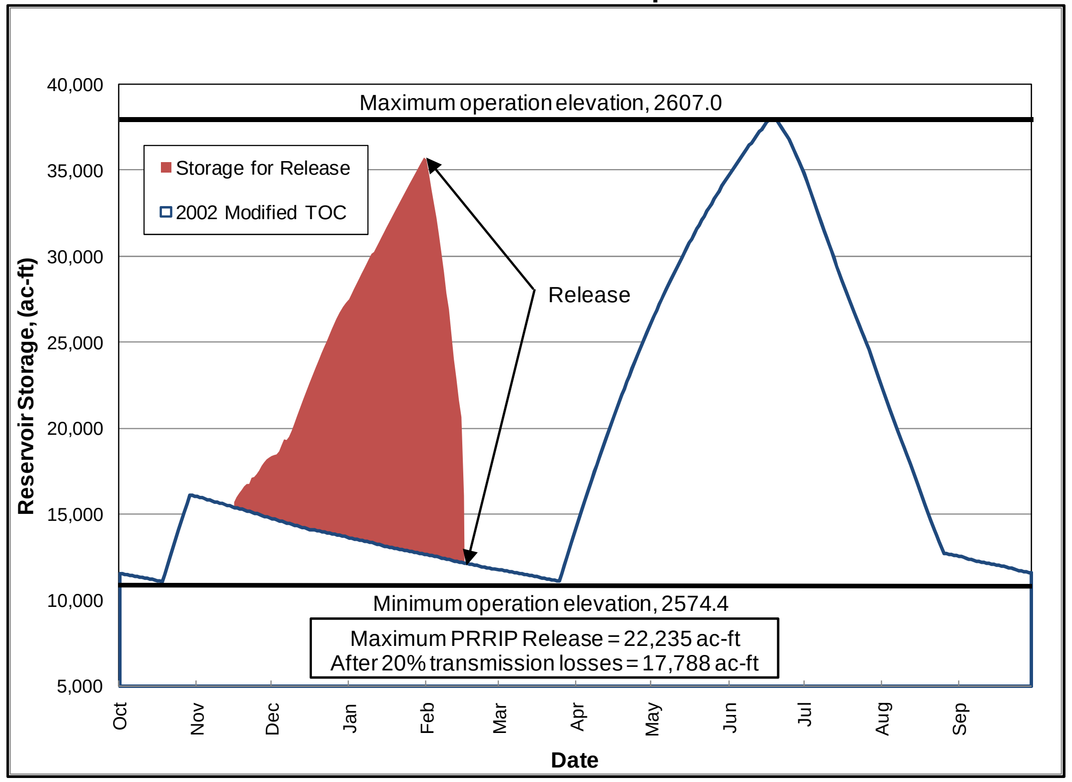
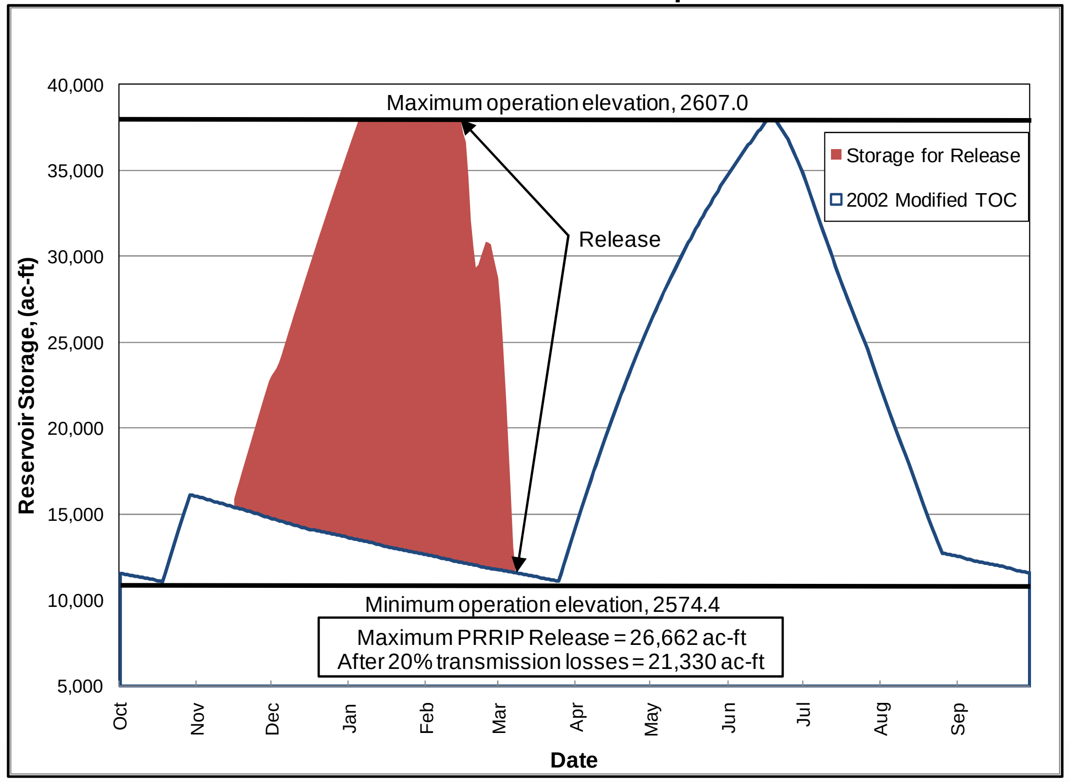


Figure 3.6-4 – Elwood Reservoir PRRIP/CNPPID Joint Operations 1986 Illustrative Wet Year



Figures 3.6-5 through 3.6-7 depict, for each illustrative year, the total flow at Overton, excess flows in the CNPPID system, excess flows in the E-65 Canal, and excess flows available for Elwood Reservoir based on the existing E-65 Canal and pump capacities. “Excess flows in E-65 Canal” are excess flows in CNPPID’s system which were not historically present in the E-65 Canal but which could have been routed down the canal rather than through the J-2 hydro and J-2 Return back to the Platte River. These excess flows were constrained based on the E-65 Canal capacity of 350 cfs. “Excess Captured in Elwood” represents the E-65 Canal flows further constrained by the pump capacities into Elwood as described in Section 3.6.2.

Figure 3.6-5 - Elwood 1964 Illustrative Dry Year Excess Flows with E-65 Canal Inlet and 190-270 cfs Pump Capacity into Elwood

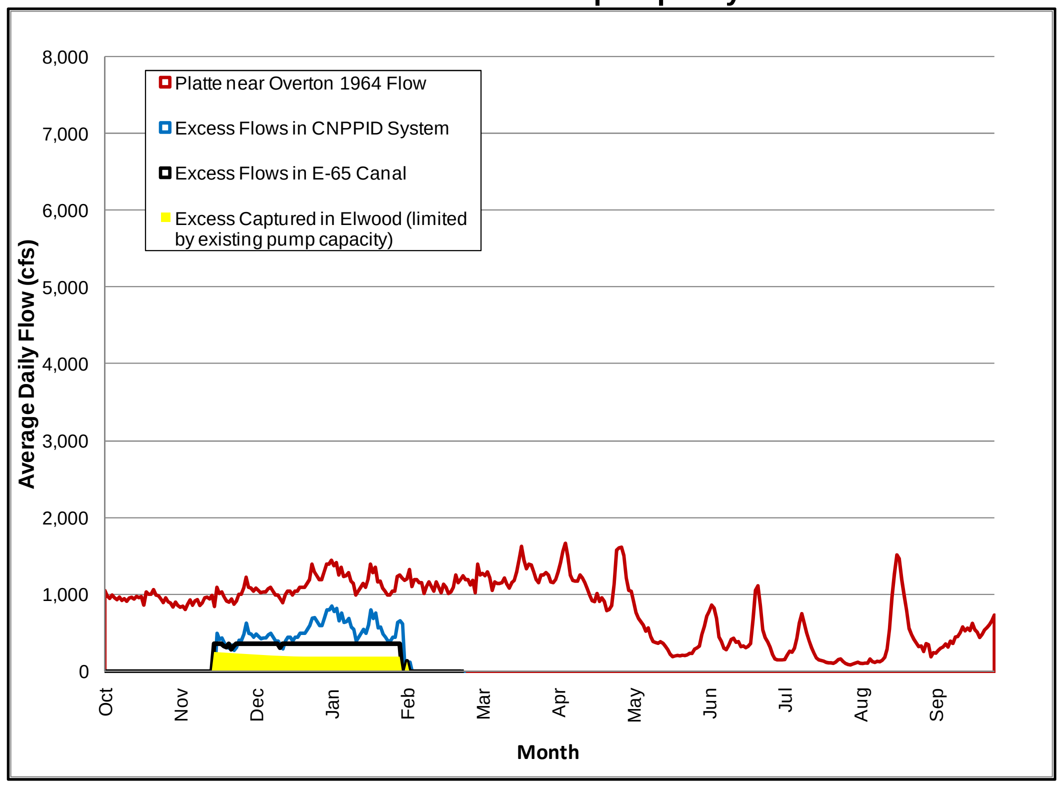


Figure 3.6-6 Elwood 1975 Illustrative Normal Year Excess Flows with E-65 Canal Inlet and 190-270 cfs Pump Capacity into Elwood

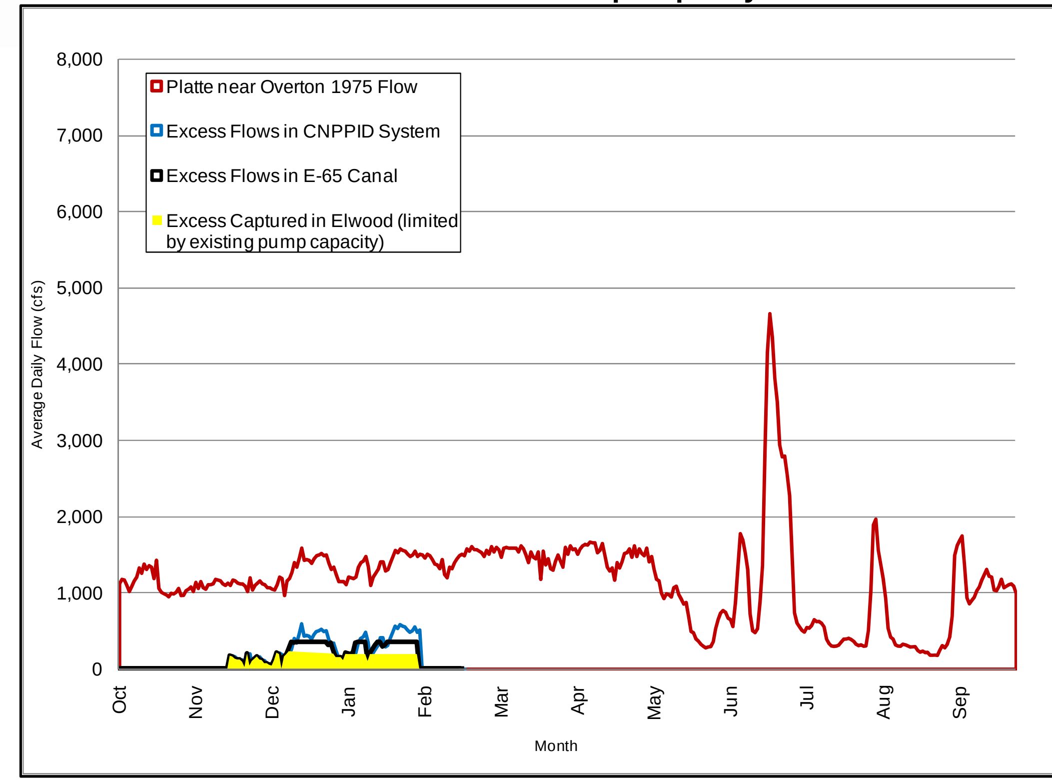
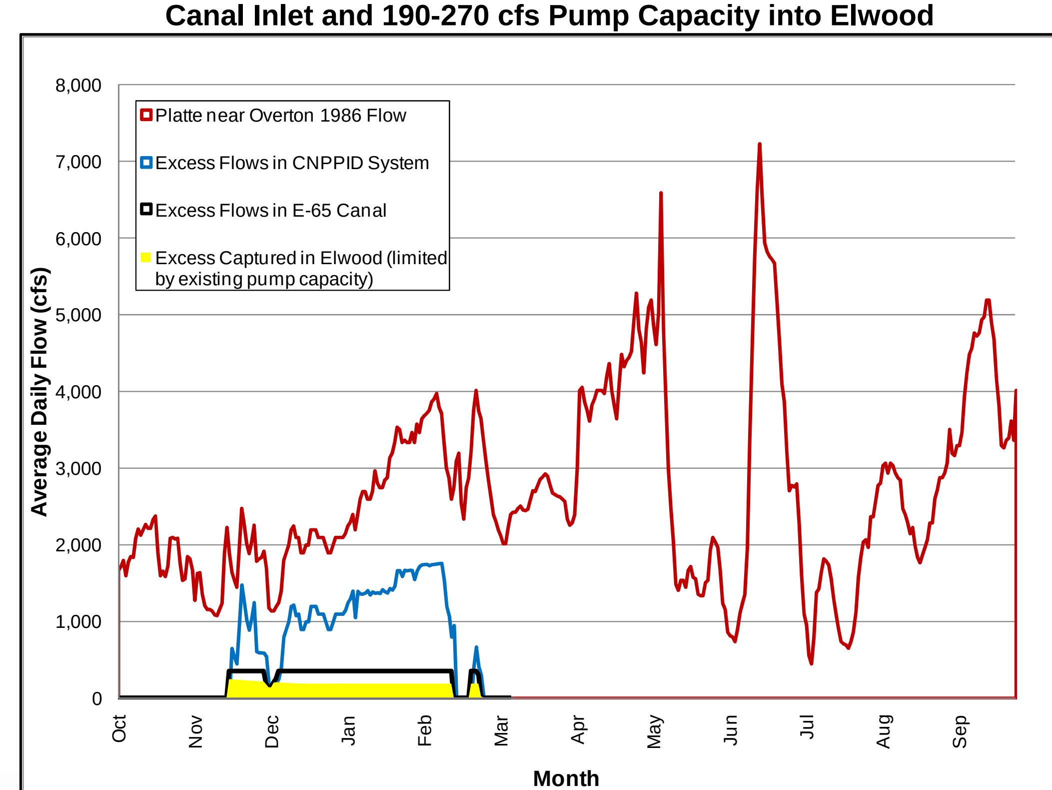


Figure 3.6-7 Elwood 1986 Illustrative Wet Year Excess Flows with E-65 Canal Inlet and 190-270 cfs Pump Capacity into Elwood



Figures 3.6-8 through 3.6-10 show the relationships between excess flows at Overton, excess flows captured by Elwood Reservoir and the flows released to reduce shortages to target flows and how the timing of the flows occur. It should be noted that travel times were not included as part of the analysis, which will affect the actual reductions to shortages to target flows depending on what flow is released on a particular day as compared to when that water actually arrives at Overton. As discussed in Section 3.6.2, releases for reductions in shortages to target flows were only made at one time, primarily since only one year was analyzed at a time, not a continuous simulation. There is potential, however, to capture and release additional water after excess flows have been captured and shortages are occurring outside of the time period when CNPPID is using the E-65 Canal. Releases were made later into March for the wet illustrative year since there was still excess water available in storage and the TOC storage/elevation level was lower at that time, as illustrated in Figure 3.6-4. For the dry and normal years, storage had been depleted by this time so there was no water left to release above the TOC storage/elevation.

Figure 3.6-8 Elwood Target Flow Analysis, 1964 Illustrative Dry Year Elwood with 190-270 cfs Inflow Capacity

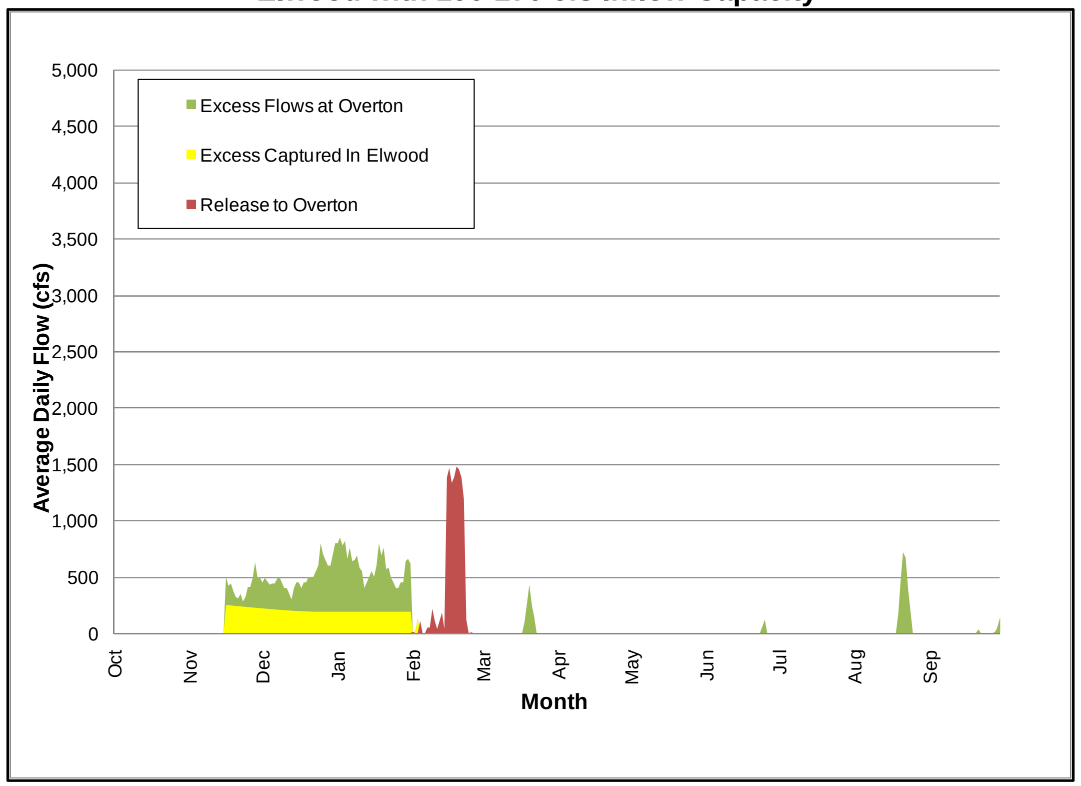


Figure 3.6-9 Elwood Target Flow Analysis, 1975 Illustrative Normal Year Elwood with 190-270 cfs Inflow Capacity

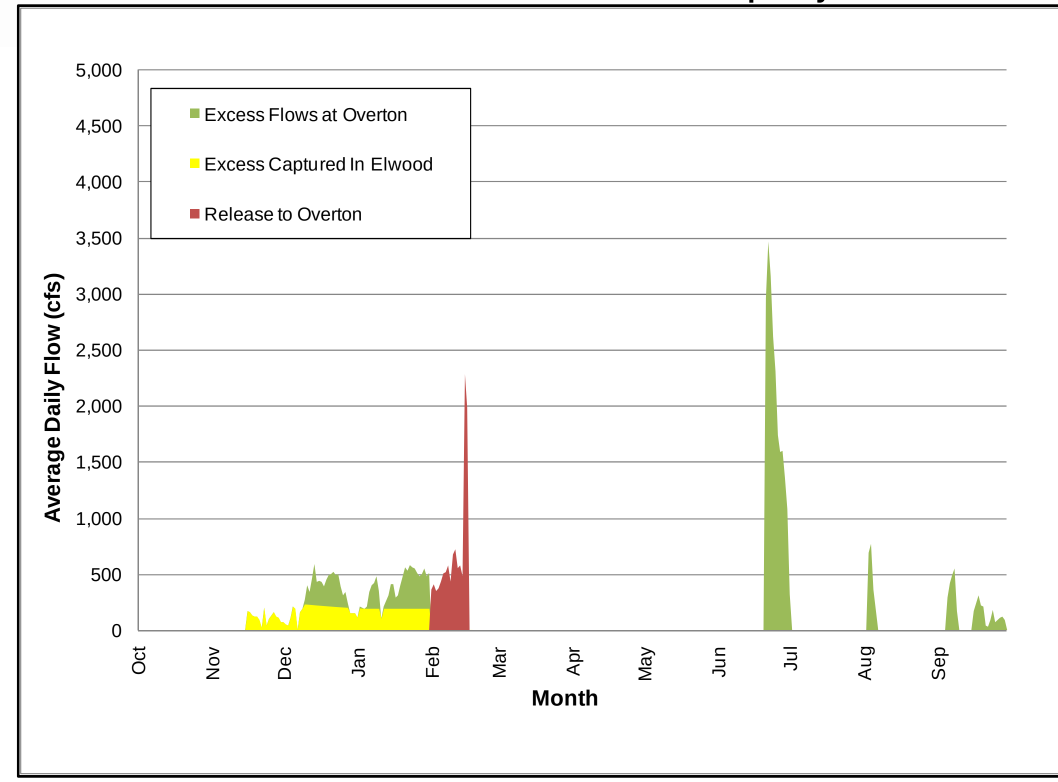
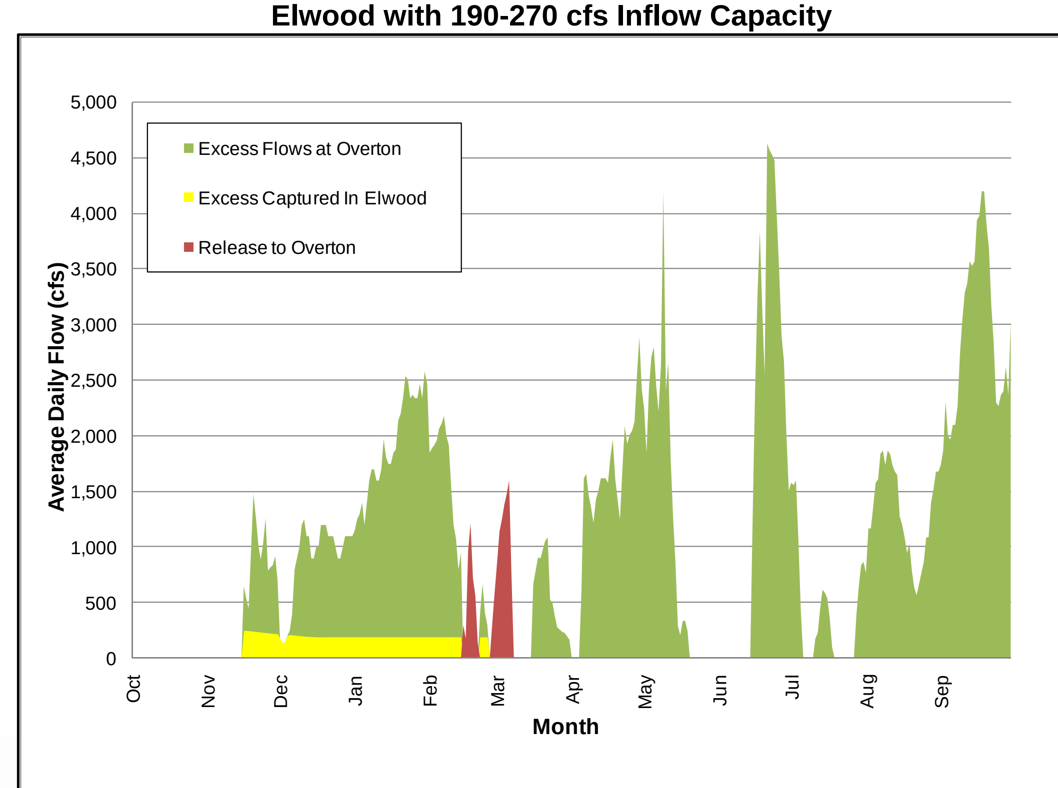


Figure 3.6-10 Elwood Target Flow Analysis, 1986 Illustrative Wet Year Elwood with 190-270 cfs Inflow Capacity



Although not a specific goal or objective of this study, modeling of target flow operations indicates Elwood Reservoir is typically at minimum stage over the winter months which is also when the reliability of excess flows are high. More analysis is needed, but it appears using Elwood Reservoir to store winter excess flows would not interfere with CNPPID current use. A low release rate into Plum Creek of around 100 to 500 cfs would minimize Plum Creek stabilization costs and minimize roadway crossing upgrades. With a potential high volume yield and minimal capital costs, this alternative should be further investigated.

3.6.3 Hydropower Flow Cycling Mitigation

Dampening of hydropower flow cycling from the J-2 Return is a desired characteristic for the alternatives. Elwood Reservoir is not located downstream of the J-1 or J-2 hydropower stations and hence does not have the ability to mitigate hydropower flow cycling impacts.

3.7 Elwood Opinion of Probable Construction Costs

Capital costs needed for the use of Elwood to provide SDHFs are composed of several components. The new or upgraded components needed to put Elwood Reservoir into operation for PRRIP use are:

- Gravity inlet canal
- Dam embankment stabilization
- Outlet works
- Plum Creek upgrades

The following table shows the combinations of components for the six Elwood alternatives and their associated capital costs. Appendix C provides a detailed summary of all alternatives, capital costs, operating costs, and life cycle costs. Each component is discussed in the following sections and detailed opinions of probable construction costs are included in Appendix C.

Table 3.7-1 Elwood Alternative Components and Capital Costs (in thousands of dollars)

Alternative	Embankment Stabilization Method	Inlet Works	Outlet Works Through Embankment	Conveyance to Platte River	Total Capital Costs
E-1	Elwood buttress $2,797	Gravity Canal $6,265	2-8' Tunnels $12,507	Plum Creek, 2,400 cfs $21,373	$42,942
E-2	Elwood remove & replace embankment $9,453	Gravity Canal $6,265	New Outlet (2 pipes) $8,353	Plum Creek, 2,400 cfs $21,373	$45,444
E-3	Elwood remove & replace upstream shell $5,377	Gravity Canal $6,265	2-8' Tunnels $12,507	Plum Creek, 2,400 cfs $21,373	$45,522
E-4	Elwood buttress $2,797	Existing E-65 Canal $0	2-8' Tunnels $12,507	Plum Creek, 2,400 cfs $21,373	$36,677
E-5	Elwood remove & replace embankment $9,453	Existing E-65 Canal $0	New Outlet (2 pipes) $8,353	Plum Creek, 2,400 cfs $21,373	$39,179
E-6	Elwood remove & replace upstream shell $5,377	Existing E-65 Canal $0	2-8' Tunnels $12,507	Plum Creek, 2,400 cfs $21,373	$39,257

3.7.1 Gravity Inlet Canal Capital Costs

The existing E-65 Canal could be used for the operation of Elwood for SDHF and target flows. However, the inclusion of a gravity canal feeding Elwood Reservoir would provide more flow capacity for the PRRIP operations. The existing E-65 Canal siphon is starting to show signs of erosion, needing increased maintenance, and, at some point, will need to be replaced, according to CNPPID. The cost of replacement (see twin E-65 discussion above) would be much higher than the cost of a gravity canal into Elwood, which was estimated to be approximately $6.3 million. Appendix C shows the cost breakdown.

3.7.2 Embankment Stabilization Capital Costs

Dam stabilization may be required if the reservoir will be full for a much longer portion of the year than it is now. The use that appears favorable is to stage the reservoir for SDHF and/or target flow operations over the winter and early spring months. It is unclear if this change in operation would allow the embankment to become fully saturated, which would result in stability problems on the upstream face when the water level is lowered quickly.

3.7.2.1 Remove and Replace Dam

Removal and replacement of the existing embankment would involve construction of zoned embankments, extensive internal filter and drainage zones, and a facing system – most likely soil cement. The estimated cost for replacing the dam would be approximately $9.5 million, as detailed in Appendix C.

3.7.2.2 Upstream Buttress/Upstream Embankment Flattening

The installation of an upstream buttress, which would flatten the angle of the upstream slope, would provide the embankment additional strength to resist slope failure caused by rapid drawdown loading. This alternative would be the least expensive upgrade of the embankment, with an approximate cost of $2.8 million. Appendix C shows a detailed cost breakdown.

3.7.2.3 Remove and Replace Upstream Shell Only

Removing and replacing the upstream shell with known, suitable materials and soil cement facing as a means of stabilizing the Elwood Dam during rapid loading can be accomplished. The estimated cost of this alternative is $5.4 million. A detailed cost breakdown is included in Appendix C.

3.7.3 Outlet Works Capital Costs

3.7.3.1 New Outlet as Part of Embankment Removal and Replacement

In the circumstances where the entire dam was to be removed and replaced, a new high-capacity, low-level outlet could be excavated into the side slope of the dam foundation. Twin steel pipes, encased in concrete, an upstream slide gate, and trash racks incorporated into outlet tower, as well as an access bridge would be included in the construction. The cost of this alternative would be approximately $8.4 million, but must be completed in conjunction with removal and replacement of the embankment. A cost breakdown is provided in Appendix C.

3.7.3.2 Tunneling of New Outlet Pipes

The estimated cost for tunneling two 8-foot diameter pipes through the existing embankment would be approximately $12.5 million, as detailed in Appendix C.

3.7.4 Conveyance to the Platte River – Capital Costs for Plum Creek Upgrades

Plum Creek upgrades will involve bank stabilization/armoring, as well as some bridge and culvert upgrades. The estimated costs for Plum Creek improvements required to convey 2,400 cfs would be approximately $21.4 million. As described in the Section 3.4.1.3, a significant amount of uncertainty exists in the data used to develop these costs, which are detailed in Appendix C.

Upgrading Plum Creek to handle SDHF augmentation of 1,200 cfs was considered for the case of combined operation with the J-2 Alternative 2 options. The costs were estimated to be $15.3 million. Appendix C shows a breakdown of costs.

3.8 Operating Costs

For purposes of scoring the alternatives, operating costs were calculated using the NRCS suggested rates for operating and maintenance costs. The resulting operating costs can be found in Appendix C. Elwood's operating costs were increased to include power generation offset costs due to water bypassing the J-1 and J-2 hydropower stations, as described in Section 3.6.2. The value calculated in the WMS Phase II (WMS, 2008), Appendix 4 of $7.89 per ac-ft was used.

4. J-2 RETURN RESERVOIR

Three alternatives were investigated for the J-2 Return Reservoir (J-2 Reservoir). These alternatives were investigated to size and locate a reservoir capable of augmenting the SDHFs defined in the operating criteria, and to reduce shortages to target flows in the Platte River. The three alternatives investigated were:

- Alternative 1 – Pond in the South Channel of the Platte River
- Alternative 2 - Excavate reservoirs near the Platte River
- Alternative 3 – Build a reservoir at Phelps Canal station 9.7

Some of the assumptions and characteristics of the J-2 alternatives are listed below in Table 4.1. These are discussed in detail in each alternative's respective section.

Table 4-1 – J-2 Alternative characteristics

Alternative	Gravity Fill Storage Capacity (ac-ft)	Total Storage Capacity (ac-ft)	Maximum Gravity Inflow (cfs)	Pumped Inflow (cfs)
Alt 1	3,380	3,380	2,000	n/a
Alt 2, Area 1	9,716	9,716	1,000	n/a
Alt 2, Area 2	4,604	6,580	1,000	240
Alt 2, Area 3	3,085	4,516	1,675	200-300
Alt 2, Area 4	1,217	6,137	1,675	200-300
Alt 2, Areas 1 & 2	14,320	14,320	1,000	n/a
Alt 3	1,659	1,659	1,000	n/a

4.1 Development Background

The WMS Platte River Phase II Evaluation of SDHF events (WMS, 2008) identified a potential 3,300 ac-ft storage area and estimated the construction costs at $31.3 million. The J-2 potential storage area identified in the WMS study was not able to meet the SDHF event and the cost per ac-ft stored was higher than many other storage options.

The J-2 Return Reservoir Alternatives Screening Analysis performed in this study sought to identify up to three alternative storage concepts, which potentially could offer more storage and/or better cost efficiency. The alternatives would be conceptually developed to support SDHF events. Reservoir configurations were then analyzed for their ability to reduce shortages to target flows and potentially minimize hydropower cycling.

4.2 Groundwater and Surface Water Interactions

Based on the proposed locations of the J-2 Return Alternatives, an important aspect of the engineering analysis included an assessment of the interaction between groundwater and surface water. The following sections provide a synopsis of the general geology, hydrogeology, and groundwater/surface water interactions. It should be noted that no fieldwork was conducted and no samples were collected or analyzed as part of this evaluation. The estimated seepage rates from the proposed structures are based on published values of hydraulic conductivities and standard engineering equations. The water level information was gathered from published USGS reports, Central Platte Natural Resources District (CPNRD) water level measurements and USGS water level monitoring data. As described below, due to the variability of the sand and gravel deposits along the Platte River, the seepage estimates and groundwater elevations will require field verification prior to design. http://groundwaterwatch.usgs.gov/StateMaps/NE.html)

The J-2 alternatives are in the floodplain of the Platte River. Boring logs from wells registered at the NDNR and University of Nebraska Conservation Survey Division's test holes in the immediate area of the proposed J-2 alternatives indicate the area is underlain by a sequence of sand and gravel deposits that are up to 300 feet thick. The surficial alluvial deposits of medium-to-coarse sand and gravel are approximately 47 feet to 50 feet thick. The sand and gravel deposits are underlain by the weakly cemented sandstone units of the Ogallala group. The Ogallala is approximately 250 thick in this area (UNL-CSD, Base of the principal aquifer map, http://snr.unl.edu/data).

Table 4.2-1 illustrates the nearest USGS water level monitoring sites in the project area. There are three monitoring well sites with consistent water level recordings from the early 1990s (USGS Active Groundwater Level Network, http://groundwaterwatch.usgs.gov/StateMaps/NE.html). Based on the three monitoring sites; the depth to water ranges from 2 feet to 9 feet below ground surface (bgs). The seasonal variations are dependent on precipitation and river stage levels. Average groundwater elevations at the three USGS water level measurements sites were calculated using the median value of depth to water subtracted from the reported ground surface elevation. The depth to water (DTW) measurements are provided below along with the ground surface elevations reported on the USGS Web site. The ground surface elevations are based on the USGS Differential Global Position System (GPS) survey which is accurate to one-tenth of 1 foot. Annual groundwater elevation changes along with more information on the monitoring sites are included in Appendix I.

Table 4.2-1 – Depth to Water and Water Level Elevation From USGS

USGS Site ID	Township (North)	Range (West)	Section	Ground Surface Elevation (ft msl)	Median Depth to Water (ft)	Median Water Level Elevation (ft msl)
404255099434201	9	21	28	2,379.1	6.5	2,372.6
404245099435501	9	21	32	2,374.3	4.3	2,370.0
404203099415901	9	21	34	2,365.6	6.2	2,359.5

msl = mean sea level

Streams interact with groundwater in several different ways: Streams gain water from inflow of groundwater through the stream bed, they lose water to groundwater by outflow through the streambed, and, some streams do both by gaining in some reaches and losing in other reaches (Winter and others 1998). The Platte River is a stream that does both, gaining in some stretches and losing in others.

During the spring of 1999, the USGS completed a detailed analysis of the Platte River between Gothenburg and Silver City to better assess the gaining and losing stretches along the central portion of the river. Based on the groundwater level measurements collected at the time, the Platte River from Gothenburg to Kearney was identified as a gaining stream (Stanton, 1999). The gaining and losing nature of a stream can change with variations in seasonal flow and pumping. However, as noted on the UNL-CSD Nebraska Generalized Gaining/Losing Streams map, this section of the Platte is illustrated as a gaining stretch. Additionally, the Central Nebraska Public Power and Irrigation District has quantified that the J-2 Return Canal also is gaining groundwater at a rate of approximately 50 cfs along this stretch of the canal.

The ramification of these analyses is that any proposed storage sites could fill with groundwater, instead of surface water, if they are incorrectly designed. Specifically, the base of the proposed storage site will need to be above the high groundwater elevation, unless the base of the reservoir is appropriately designed to inhibit groundwater movement.

4.3 J-2 Reservoir Alternatives

4.3.1 J-2 Alternative 1 - Pond in the South Channel of the Platte River

This concept was investigated as a means to minimize excavation costs by storing water in the degraded southern channel of the Platte River. The site location is shown in Photo 3.4-2. An embankment was previously constructed by a landowner across the southern channel of the Platte River upstream of the J-2 Return in order to access Jeffrey Island from the south bank of the Platte River. It is believed this upstream sand dam has isolated the southern channel from Platte River minor flood events and sediment supply. The southern channel is currently deeply incised and ED Office staff was not able to identify any suitable threatened and endangered species habitat. This concept would use this reach, which is roughly seven miles long, to store water from the J-2 Return.

Photo 4.3-2 – South Channel of the Platte River, looking north. The trees in distance are on Jeffrey Island

In order to maximize storage potential, four embankments would be needed to pond water over the entire reach up to the existing sand dam. Each of the four embankments uses Jeffrey Island, as the north abutment and the south bank of the Platte River, as the south abutment. The dams range from 8 feet to 17 feet in height, and each dam ponds some water on the upstream dam embankment toe. The plan view and stage-storage for this alternative is shown in Figures 4.3-1 and figures in Appendix B. The alternative would contribute a total storage of 3,175 ac-ft. The largest dam, Dam D, is the most upstream dam and will store 1,600 ac-ft. The smallest dam, Dam A, is the last one in the series and will only store 270 ac-ft. The south channel of the Platte River becomes more incised as you move upstream; hence Dam D is the deepest and has the most storage. The table below summarizes the surface areas, embankment lengths and storage volumes. Figures 1.1 through 1.9 in Appendix B summarize the storage locations, peak ponding limits, and storage volumes.

Table 4.3-1 J-2 Alternative 1 Dam Parameters

Embankment	Peak Ponding Elevation, ft msl	Peak Surface Area, ac	Peak Storage volume, ac-ft	Embankment Length, ft
A	2318	86	268	2,100
B	2327	176	657	2,200
C	2337	147	642	1,800
D	2348	282	1,608	3,800

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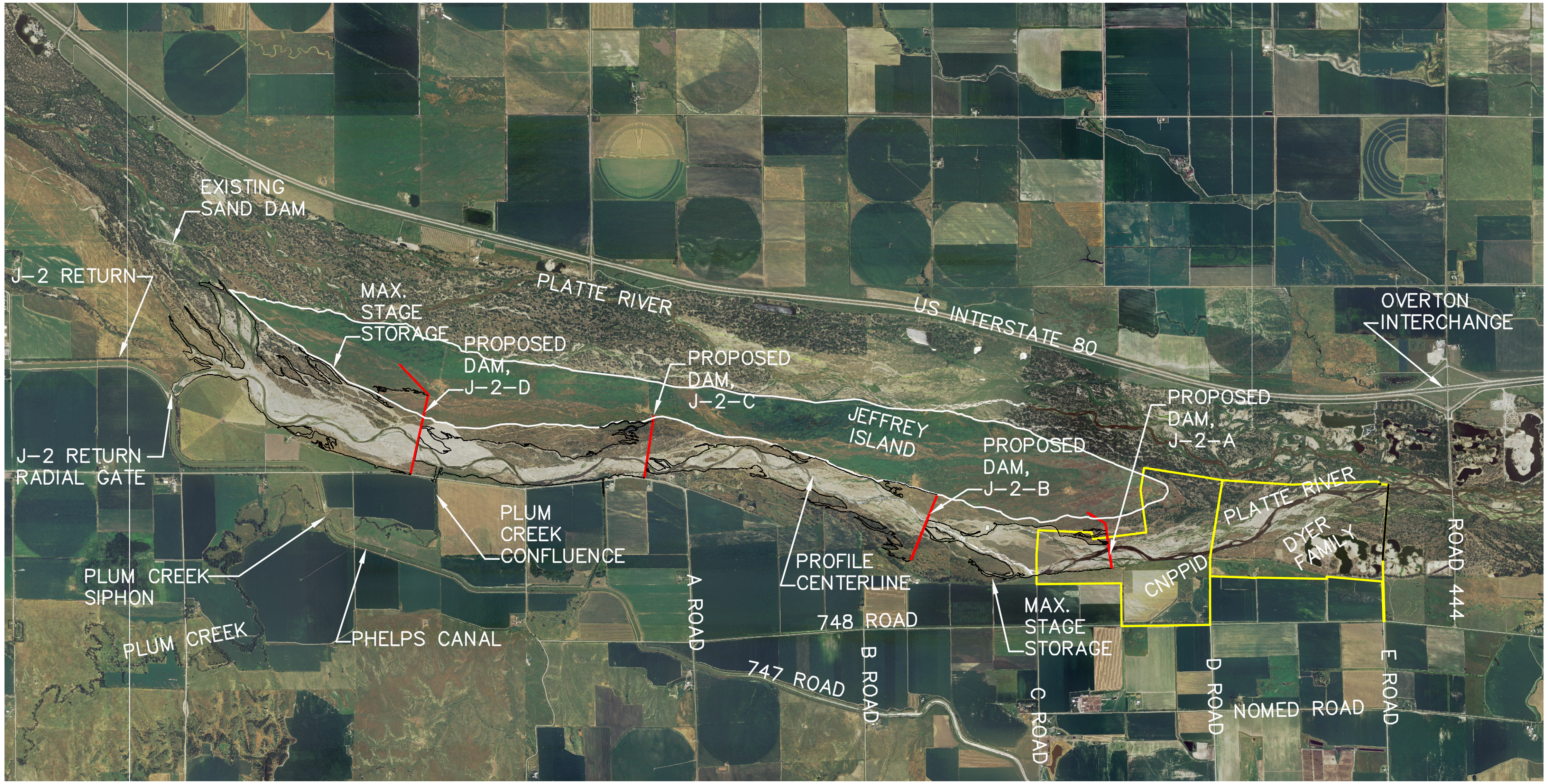
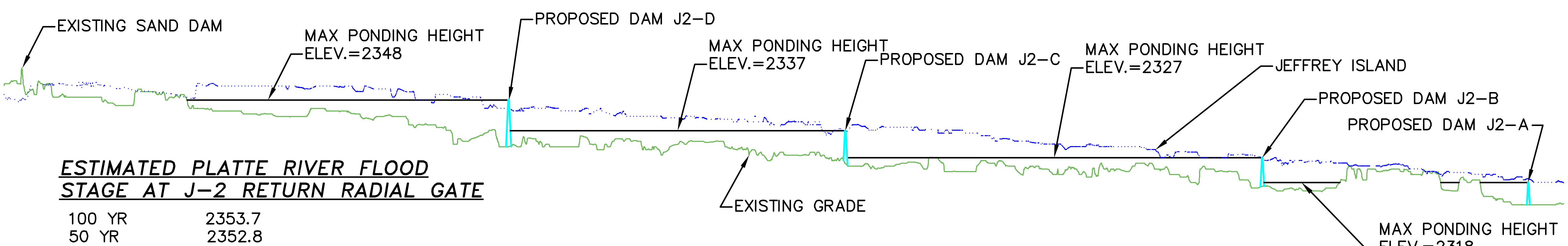


N

0' 2000' 4000'
SCALE IN FEET

LEGEND

- PROGRAM LAND BOUNDARY
- DAM CENTERLINE
- MAX STAGE STORAGE
- EXISTING GRADE
- PROPOSED DAM
- EXISTING JEFFREY ISLAND



ESTIMATED PLATTE RIVER FLOOD STAGE AT J-2 RETURN RADIAL GATE

100 YR	2353.7
50 YR	2352.8
25 YR	2347.6

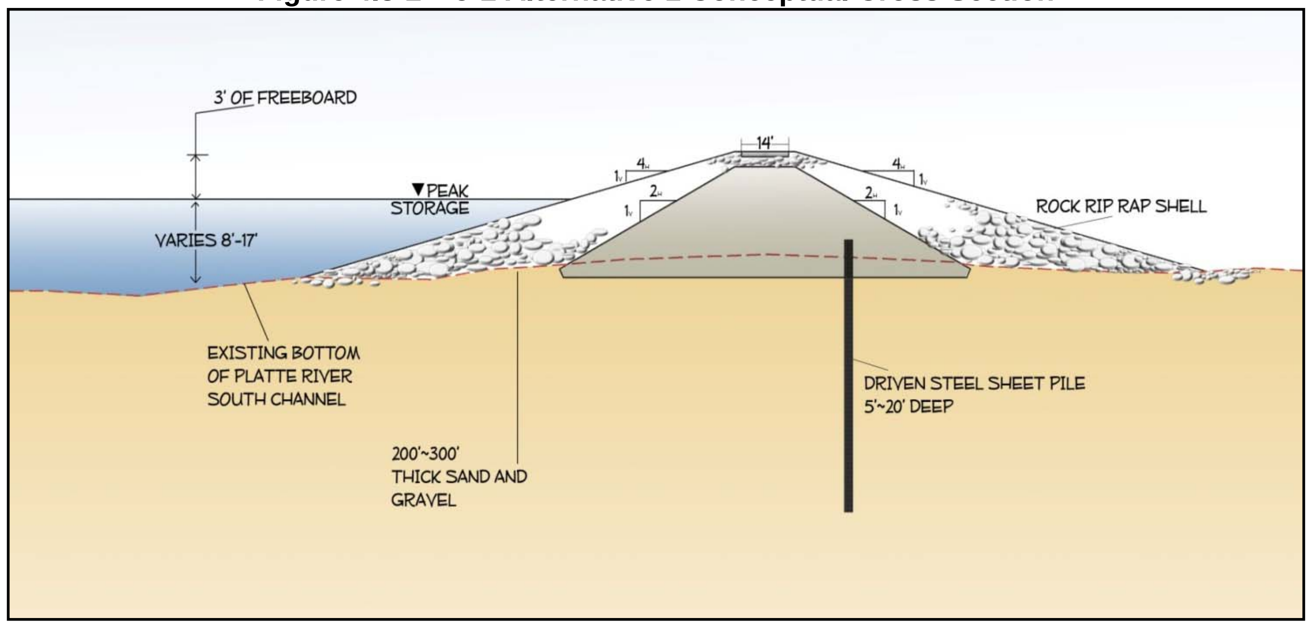
CROSS SECTION
NOT TO SCALE

PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

PROJECT: 09-1466	J-2 RETURN ALTERNATIVE 1	OLSSON ASSOCIATES	PROPOSED DIKE LOCATIONS	FIGURE
DRAWN BY: CRL	DAWSON, GOSPER, AND PHELPS COUNTY, NEBRASKA			4.3-1
DATE: 1.27.10				

All of the embankments will experience overtopping during a major flood on the Platte River. With this in mind, it is recommended the embankments are armored with rock riprap or built with erosion- resistant materials. As an alternative to rock riprap, concrete surfacing should be evaluated. Concrete surfacing has the advantage of being more durable during a flood on the Platte River and is likely cost neutral to the use of a rock riprap shell. Another alternative might be the use of sheet pile rather than soil to create the embankment. The sheet pile option appeared comparable for installation costs to concrete surfacing but would have long-term maintenance concerns.

Figure 4.3-2 – J-2 Alternative 1 Conceptual Cross Section

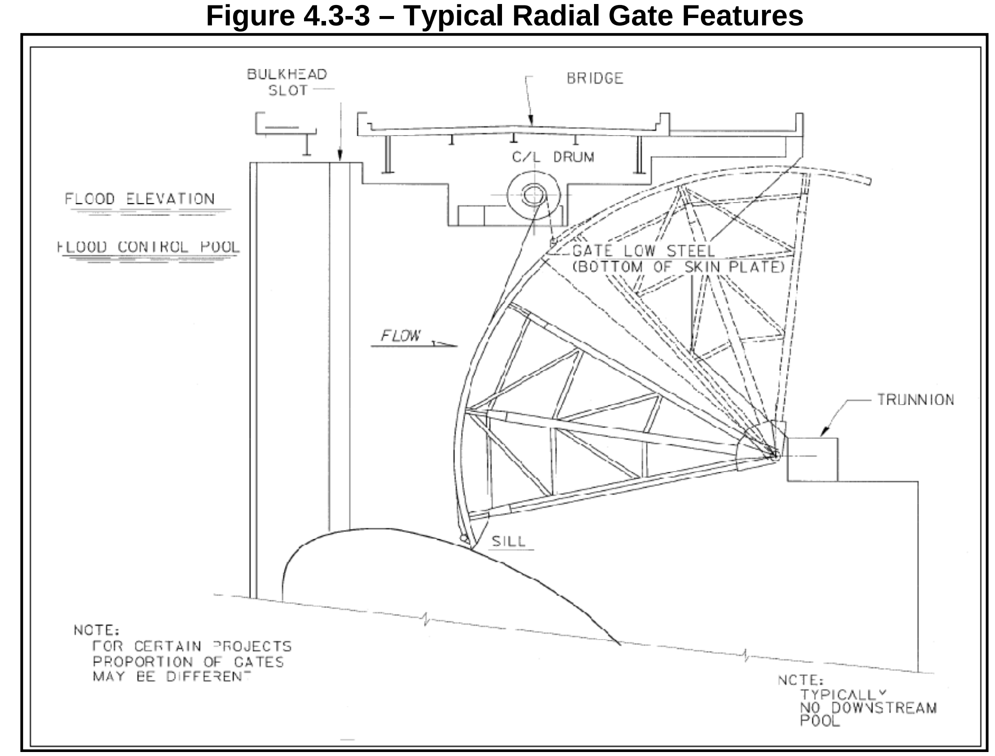


Using Jeffrey Island and the existing southern bank for the abutments is marginal, at best. Both locations have stratified sand/gravel/silt deposits and generally are non-cohesive. High seepage rates could develop at each abutment causing geotechnical stability concerns. The slope stability is covered more fully in the Appendix J.

The gate width is a critical design feature. It is desired to drain the stored water very quickly over a three-day period for the SDHF event. As the water leaves the storage area the driving head on the gate will decrease, hence decreasing the potential outflow rate. The final few feet of stored water will likely not be able to be drained out within the three days unless a significantly wider gate is used. The existing upstream J-2 Return radial gate is 30 feet wide and the flow from the J-2 Return will need to pass through this proposed dam. Gate sizing is discussed in more detail in the modeling section. The proposed outlet for Dam A is a 50-foot-wide radial gate. Though this reservoir has limited storage volume, it also has the shortest ponding depth -- and the outlet has to convey flows from both the J-2 Return and the upstream reservoirs – as a result, it was the largest gate of the four dams. A 48-foot-wide radial gate was chosen for the outlet of Dam D. Sluice gates with a labyrinth weir were evaluated against the costs and the flow characteristics of a radial gate. The radial gate was found to be superior for all options in Alternative 1. The proposed radial gate is a cable-lift, as depicted in the following page from the USACE's document EM-1110-2-2702.

Photo 4.3-1 – J-2 Return Radial Gate, 30 feet wide by 13 feet, 6 inches tall

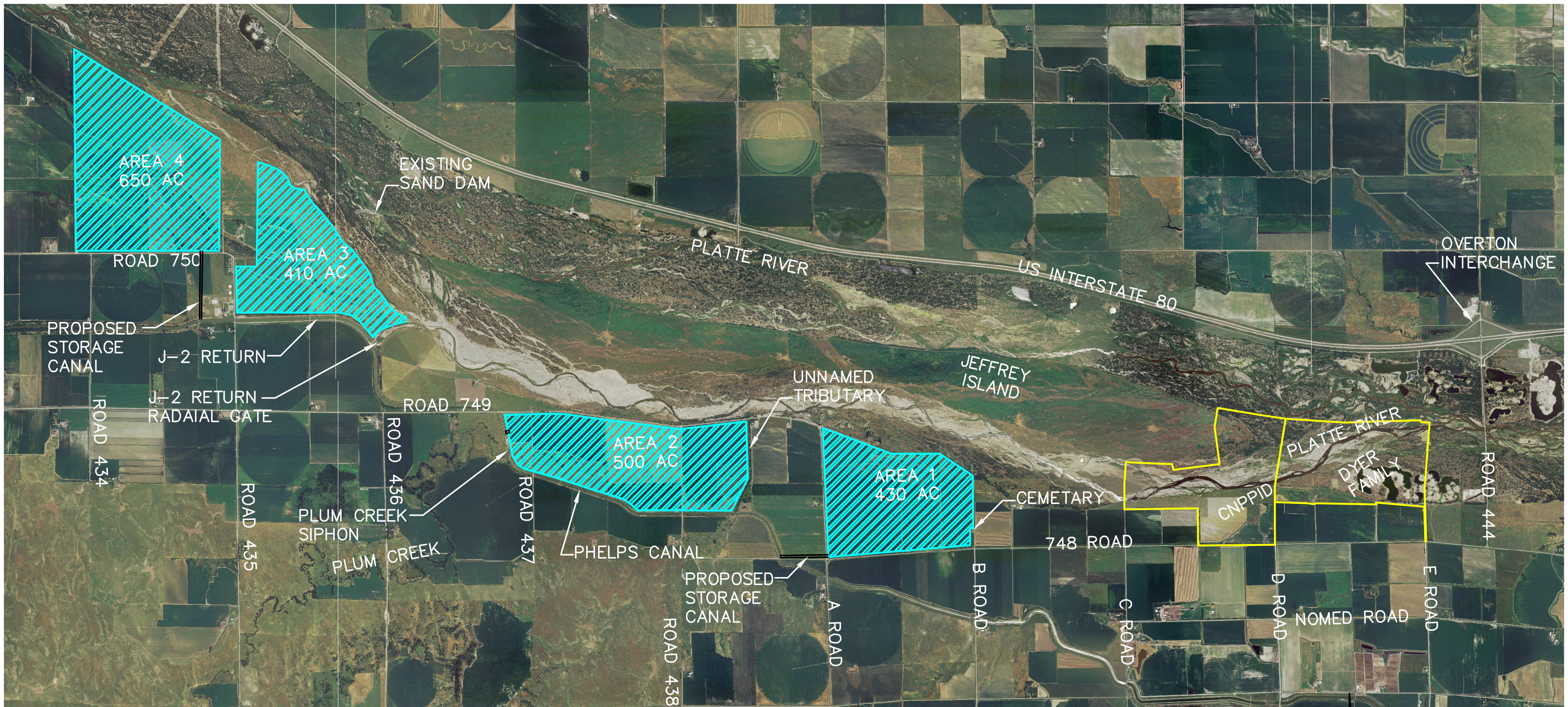
Figure 4.3-3 – Typical Radial Gate Features



Operationally, the inline series of embankments are problematic. The dams become lower as you move downstream. For the gates to maintain a uniform release rate, the width of the gates would need to increase in the downstream direction to compensate for the reduction in driving head. During the SDHF, 2,000 cfs is planned on being released from the J-2 Return. This water will need to pass through all four embankments. Plum Creek enters upstream of Dam C. The 100- year flood on Plum Creek is estimated at about 4,000 cfs. The outlet works -- for embankments C, B and A -- was designed with this overtopping in mind. As an alternative, a canal could be dug across Jeffrey Island to route J-2 Return and Plum Creek flows to the northern channel of the Platte River, rather than through dams C, B and A. This option was not further evaluated due to the desire to leave Jeffrey Island whole and to avoid impacts to the threatened and endangered species habitat on the north side of the island.

4.3.2 J-2 Alternative 2 - Excavate near the Platte River

This concept was based on the WMS Phase II idea of excavating a storage area adjacent to the J-2 Return Canal. In addition, the PRRIP may desire an excavation area to mine fine sand for sediment augmentation purposes. Reviewing the quadrangle, the existing ground elevation between the canals and the Platte River is lower than the storage elevation identified in the WMS Phase II study. Lower ground elevations relative to the canal, would result in reduced excavation volumes and cost savings. Two areas were identified – Area 1 and Area 2 – which could be filled from Phelps Canal.



N

0' 2000' 4000'
SCALE IN FEET

LEGEND

PROGRAM LAND BOUNDARY
EXCAVATION AREA BOUNDARY

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DATE: Feb 17, 2010 7:53am
USER: cluttrell
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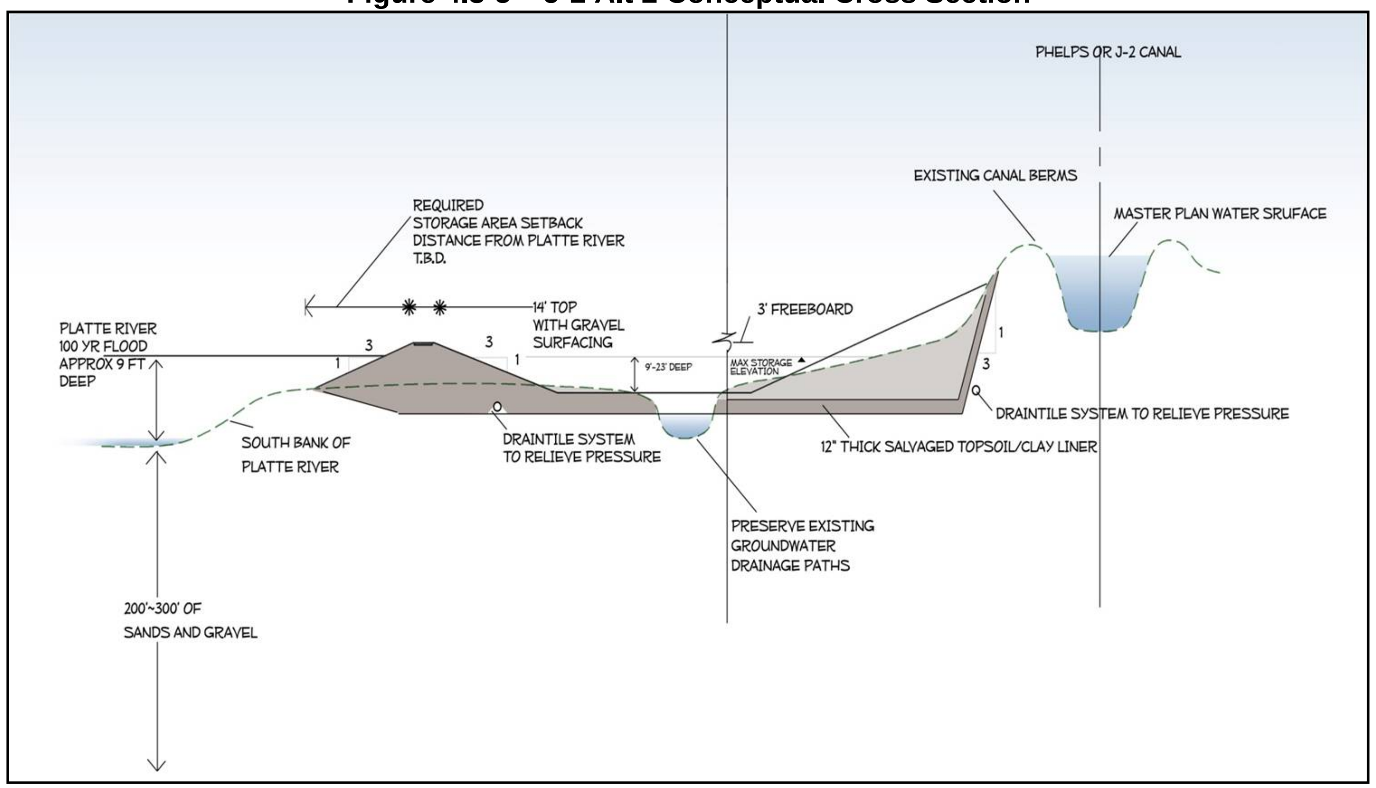
PROJECT: 09-1466 DRAWN BY: CRL DATE: 1.27.10	J-2 RETURN ALTERNATIVE 2 GOSPER AND PHELPS COUNTY, NEBRASKA	OLSSON ASSOCIATES	PROPOSED EXCAVATION	FIGURE 4.3-4

PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

Moving upstream, there are two other areas that could be filled from the J-2 Return (Area 3 and Area 4). The ground surface is higher relative to the canal for these two areas and pumps would be required to fill to the full elevation. Alternatively, significantly more excavation could be performed to eliminate the pumps.

The upward vertical limit for a gravity fill storage area is based on the Master Plan Water Surface Elevation (MPWS) referenced on the J-2 and Phelps Canals construction drawings. The elevations on the construction plans were converted to an elevation of North America Vertical Datum (NAVD) 88, by the addition 0.91 feet. (The NAVD is a system for measuring elevation).The lower vertical limit is based on the flow line of the Platte River, Plum Creek, and the abandoned southern canal of the Platte River on the south side of Danielson Island. As discussed previously in the Groundwater and Surface Water section, groundwater is close to the surface. By locating these storage areas near and adjacent to existing seepage drains, streams and the Platte River, the groundwater elevation is naturally reduced to near the existing flow lines. The storage area bottoms were held higher than these existing groundwater controls. A conceptual cross section is shown in Figure 4.3-5 that highlights some of the key considerations for Alternative 2.

Figure 4.3-5 – J-2 Alt 2 Conceptual Cross Section



In plan view, the storage areas generally followed section lines, avoiding homes, roads, and cemeteries when possible. In general, the outside toe of the embankment was set 33 feet from the roadway centerlines or section lines.

Seepage out of the reservoir was limited by the salvaging of the top 12 inches of material. In general, the geology of the area is silty loam over sand and gravel. Salvaging this top layer and reapplying once the excavation is complete will reduce the seepage losses and lower the construction cost compared to importing a clay or membrane liner. The permeability of the resulting liner is discussed in Appendix J.

The gate width is a critical design feature. It is desired to drain the stored water very quickly over a three-day period for the SDHF event. As the water leaves the storage area, the driving head on the gate will decrease, hence decreasing the potential outflow rate. The final few feet of stored water may not be able to be drained out within the three days unless a significantly wider gate is used or the outlet gate is depressed below the bottom of the storage site.

4.3.2.1 Key Features of Area 1

Location

Area 1 is bounded by Road 749 on the north, Road 748 on the south, Road B on the east, and Road A on the west. It is anticipated all county roads would remain open for this location. A major consideration is the family cemetery on the west side of Road B. The cost estimate includes installing groundwater controls around the cemetery to limit a localized groundwater rise that might be associated with SDHF peak storage elevation. It is assumed any upstream surface water runoff will be directed around the storage area and that the contributing drainage area is zero. The total usable storage is 9,700 ac-ft, and additional details are shown in Figure 4.3-7.

Earthwork

The available LIDAR data did not extend beyond the banks of the Platte River. The National Elevation Data set (NED) was obtained and added to the available LIDAR points. The combined points were used to create a terrain model and then used to estimate earthwork volumes. The average ground elevation through this area is approximately 2,332 feet. The lowest elevation is the northeast corner with an existing elevation of 2,328 feet. The highest elevation is the southwest corner at approximately elevation 2,346 feet. There are remnants of Plum Creek through this area. The bottom of excavation was set at 2,330 feet and resulted in approximately 2.5 million cubic yards of excavation over a 400-acre bottom, which is 3 feet to 4 feet of excavation, on average. As currently configured, there is approximately 0.7 million cubic yards of excess material. The majority of the excavation is in the southwest corner and this area could be raised to better balance earthwork without sacrificing much volume. The accuracy of the elevation data is a key consideration when looking at these large areas. For instance if the elevation was incorrect by an average of 1 foot, this may increase or decrease excavation by 0.7 million cubic yards.

Embankment

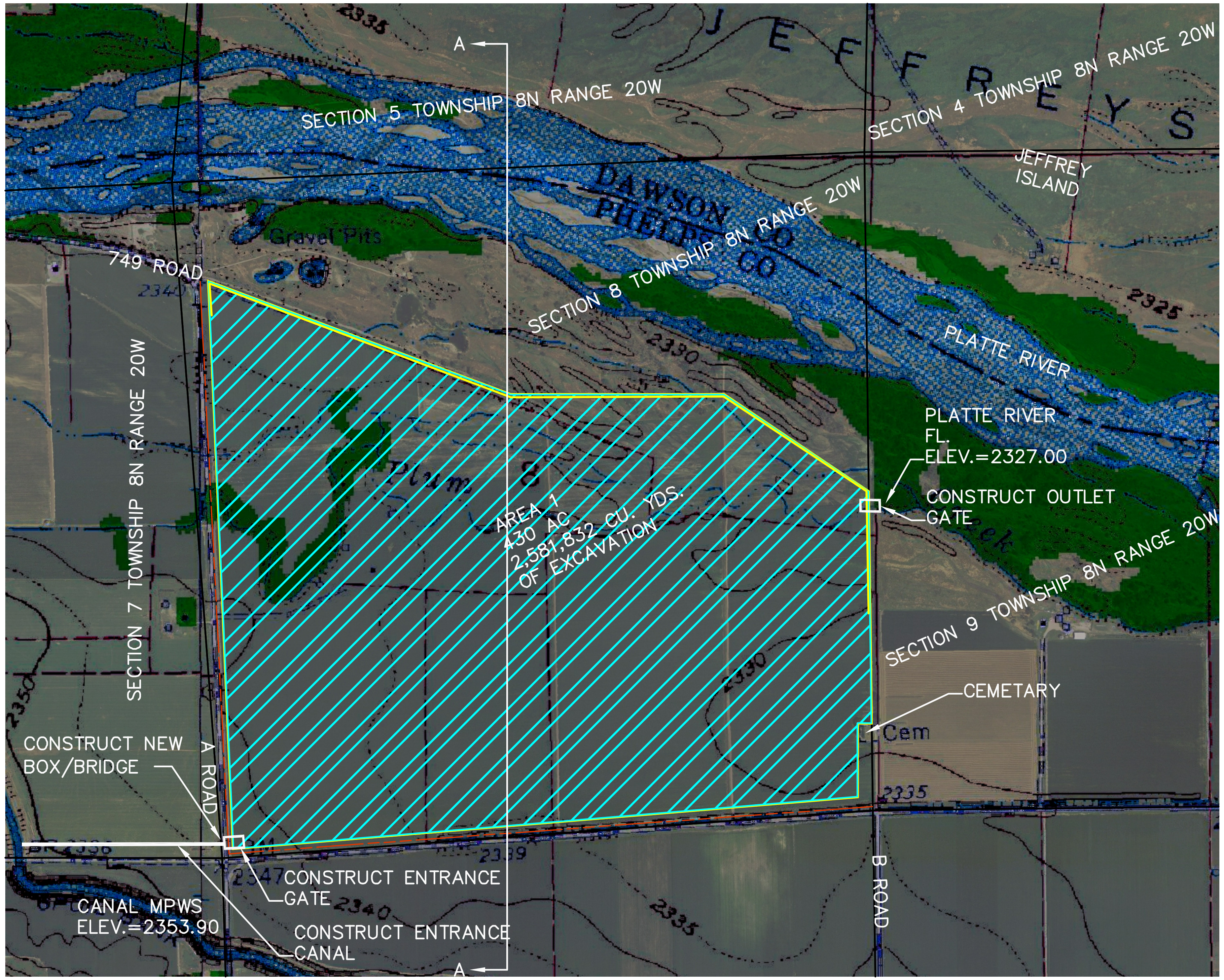
The embankment includes 3 vertical feet of freeboard, a 14-foot top width and 3H:1V side slopes. The height of the embankment above existing grade in the northwest corner is 28 feet and it is10 feet in the southwest corner. The on-site material appears to be suitable for construction of the embankments. The structure would be considered a regulatory dam by the State of Nebraska. The preliminary hazard classification is low hazard (class A3). It appears a potential breach would flow directly into the Platte River and would not inundate any residential structures or critical infrastructure. A hazard class analysis would need to be performed to verify this assumption. Approximately 1.5 million cubic yards of fill are required to construct the embankments.

Inlet and Outlet Works

A new inlet canal and gate would be required from Phelps Canal. The canal would need to cross either A Road or 749 Road and a new culvert or bridge would be required and there are several feasible combinations of bridges, gates, and canals. Figure 4.3-4 shows the preliminary canal, gate, and crossing location. It appears integrating the canal crossing with the gate would be the most cost-efficient arrangement. The capacity of Phelps Canal is 1,410 cfs in this area based on construction plans and the inlet works are designed to match the canal design capacity. Operationally, however, CNPPID limits the canal to 1,000 cfs based on the current condition of the system. For this analysis, the lower 1,000 cfs capacity was used to be conservative. The J-2 hydropower station has a peak output of 2,000 cfs so filling Area 1 from Phelps Canal results is a potential loss of up to 1,000 cfs of excess flows but this did not appear to be a fatal flaw. Additional investigation of the canal capacity limitations in this area might identify low cost improvements could be performed to return the canal up to the original design capacity.

The outlet works consists of a radial gate, 40 feet wide and 25 feet high, that will discharge directly into the south channel of the Platte River. This is a relatively high radial gate but the costs appeared much lower than an equivalently sized multiple sluice gates structure.

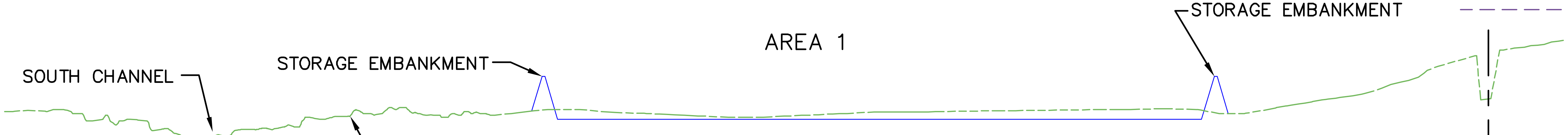
Both the inlet works and outlet works will require energy dissipation. A concrete energy dissipation followed by rock riprap will be required. The associated costs of the energy dissipation are included in the construction cost estimate.



J-2 Return Alternative 2 Stage Storage - Area 1				
Elevation	Area (sf)	Area (acre)	Incremental Storage (acre-ft)	Total Storage (acre-ft)
2328	14,787	0	0	0
2329	148,867	3	2	2
2330	17,409,781	400	202	203
2331	17,461,971	401	400	604
2332	17,514,236	402	401	1,005
2333	17,566,575	403	403	1,408
2334	17,618,989	404	404	1,812
2335	17,671,477	406	405	2,217
2336	17,724,039	407	406	2,623
2337	17,776,676	408	407	3,031
2338	17,829,387	409	409	3,439
2339	17,882,172	411	410	3,849
2340	17,935,032	412	411	4,260
2341	17,987,966	413	412	4,673
2342	18,040,975	414	414	5,086
2343	18,094,058	415	415	5,501
2344	18,147,215	417	416	5,917
2345	18,200,446	418	417	6,334
2346	18,253,752	419	418	6,753
2347	18,307,132	420	420	7,172
2348	18,360,587	422	421	7,593
2349	18,414,116	423	422	8,015
2350	18,467,719	424	423	8,439
2351	18,521,397	425	425	8,863
2352	18,575,149	426	426	9,289
2353	18,596,671	427	427	9,716

LEGEND

- STORAGE AREA BOUNDARY
- EXISTING GRADE
- PROPOSED STORAGE
- TOP OF EMBANKMENT
- EXISTING MAJOR CONTOUR



CROSS SECTION A-A
NOT TO SCALE

NOTE:
CONTOURS DEVELOPED FROM THE COMBINATION OF LIDAR POINTS WITHIN THE PLATTE RIVER AND NED POINTS SOUTH OF THE PLATTE RIVER.

PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

PROJECT: 09-1466 DRAWN BY: CRL DATE: 1.27.10	J-2 RETURN ALTERNATIVE 2 PHELPS COUNTY, NEBRASKA	OLSSON ASSOCIATES	EXCAVATION AREA 1	FIGURE 4.3-6

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USER: cluttrell

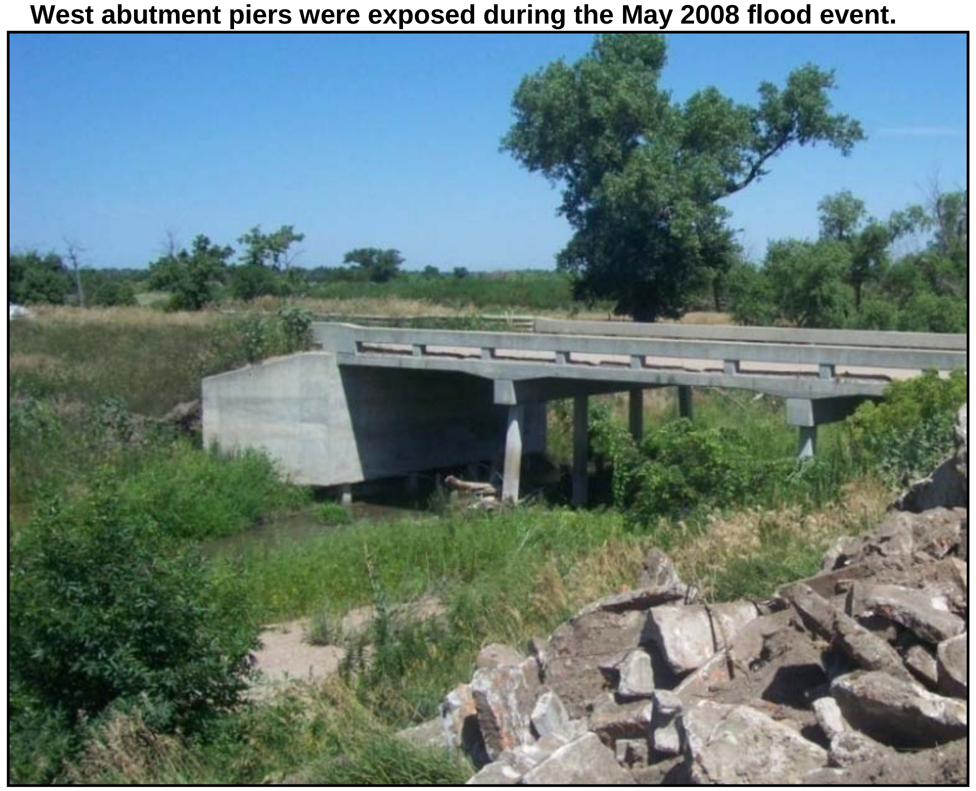
4.3.2.2 Key Features of Area 2

Location

Area 2 is bordered by Road 749 on the north, an unnamed stream on the east, and Phelps Canal on the south and west. It is built over the current and historic Plum Creek alignment. There are two roads that cross the potential storage area, Road 438 and Road 437. It appears Road 437 could be elevated or realigned and could remain in service without losing much storage volume. It appears possible to use the existing Road 437 Bridge over Plum Creek and not require a new crossing. Likewise Road 438 potentially could be elevated and the existing bridges utilized, but this roadway would have a larger reduction in storage. The potential overtopping frequency and temporary road closures will need to be discussed with Gosper County during the feasibility study. The total usable storage is 6,800 ac-ft.

Area 2 will receive flows from Plum Creek, which raises multiple design considerations. The USGS stream gage (No. 06767500) is located near Smithfield upstream of the storage area. The approximate drainage area is 209 square miles at the gage and the estimated 100-year flow is 3,800 cfs to 6,900 cfs. The flood of record is 2,800 cfs recorded, on June 23, 1947. On May 24, 2008, an estimated flow of 1,440 cfs was reported. This flow damaged CNPPID, Plum Creek siphon, caused a large amount of stream bank scour, and bridge scour at 749 Road. The scour related impacts to the pile cap and bottom of wing wall are still visible, as shown in figure 5.3.6. In addition to flood waters, Plum Creek is incising upstream and will be delivering sediment into the storage area, which will decrease the storage volume over time, unless it is mechanically removed. On the positive side, Plum Creek generated 11,590 ac-ft of water in water year 2007. A portion of the flows could be captured during times of excess and could be released when flows drop below target flows.

Photo 4.3-2 – County Road 749 Bridge over Plum Creek.
West abutment piers were exposed during the May 2008 flood event.



Earthwork

The available LIDAR (Light Detection and Ranging) data, which is a remote sensing system used to gather topographic information, did not extend beyond the banks of the Platte River. The NED (National Elevation Dataset) set was obtained and added to the available LIDAR points. The combined points were used to create a terrain model and then used to estimate earthwork volumes. The average ground elevation through this area is approximately 2,350 feet. The lowest elevation is the Plum Creek outlet at the north side with an existing elevation of 2,340 feet, plus or minus. There is a large jump in elevation at this location between the LIDAR data set and the NED. The Platte River flow line in this area is 2,333 feet which is based on the more reliable LIDAR data. The highest existing elevation within the storage area is between Plum Creek and Phelps Canal along the southern border and is above 2,355 feet. The bottom of excavation was set at 2,346 feet and resulted in approximately 0.84 million cubic yards of excavation over a 490-acre bottom (1 to 2 feet of excavation on average).

The top of the Phelps Canal in this area is 2,364 feet (+/-) and master plan water surface is 2,357.1 feet, based on the construction plans. With the bottom of storage set at 2,346 feet, this would only allow for 4,600 ac-ft of storage based on a gravity fill up to the 2,357.1 feet elevation. To balance the earthwork at the site and to further increase the storage, a pump station was investigated to increase the storage height. Leaving 3 feet of freeboard from the top of Phelps Canal berms sets the maximum fill elevation of 2,361 feet. There might need to be an additional one- to two-foot-high embankment next to portions of Phelps Canal or, alternatively, the Phelps Canal embankment can be raised. The pump station is discussed in more detail in the Inlet and Outlet Works discussion

As currently configured with the top of the storage embankments set to 2,364 feet and the bottom of excavation at 2,346 feet, the site earthwork is balanced and results in the lowest potential construction costs. The accuracy of the elevation data is a key consideration when looking at these large areas. For instance, if the elevation was incorrect by an average of 1 foot, this may increase or decrease excavation by 0.8 million cubic yards.

Embankment

The embankment includes 3 vertical feet of freeboard, a 14-foot top width and 3H: 1V side slopes. The height of the embankment above existing grade on the north side is 20 feet. No embankment would be required on the south side. In addition to the primary embankments, a berm is needed on Plum Creek to prevent the backwater from impacting the adjacent residence. The on-site material appears to be suitable for construction of the embankments.

The structure would be considered a regulatory dam by State of Nebraska due to the storage volume. The preliminary hazard classification is low hazard (class A3). It appears a potential breach would flow directly into the Platte River and would not inundate any residential structures or critical infrastructure. The potential groundwater and breach impacts to the homestead on the south side of 749 Road will need to be closely evaluated. A hazard class analysis would need to be performed to verify the low hazard assumption. Approximately 0.8 million cubic yards of fill are required to construct the embankments.

Inlet and Outlet Works

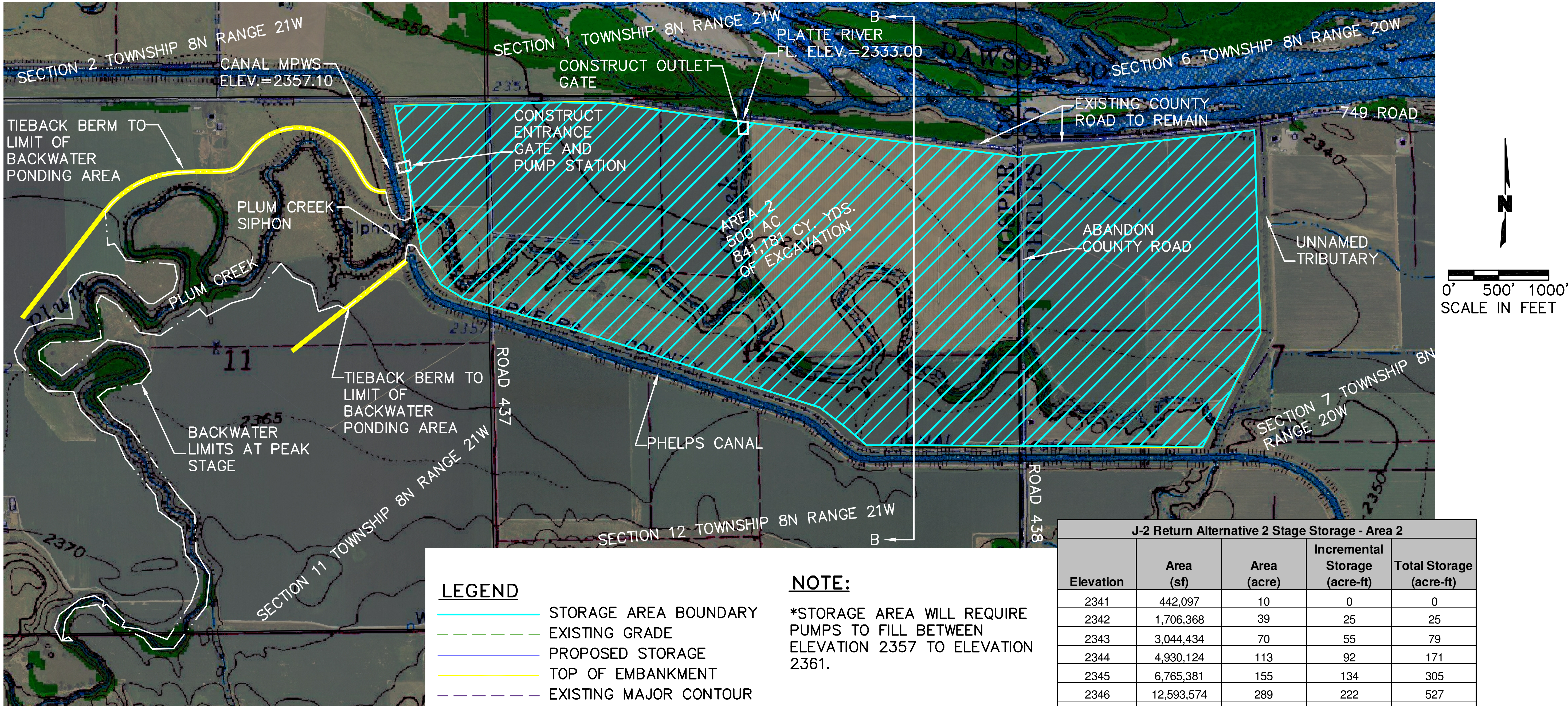
A new inlet gate and pump station would be required from Phelps Canal upstream of the Plum Creek siphon. Figure 4.3.7 shows the suggested gate and pump station location. The capacity of Phelps Canal based on the construction drawings is 1,410 cfs in this area and the inlet works are designed to match the canal design capacity. As discussed previously, the SDHF and target flow analysis maximum inflow rates were set to 1,000 cfs due to the current conditions of Phelps Canals. The J-2 hydropower station has a peak output of 2,000 cfs so filling Area 2 from Phelps Canal results in a potential loss of up to 1,000 cfs of excess flows, but this did not appear to be a fatal flaw.

Operationally, the sluice gates would be open and water would gravity flow from the Phelps Canal into the storage area up to the canal's maximum water surface elevation of 2,357.4 feet. Once the storage area has reached the maximum gravity fill elevation, the sluice gates would close and the pump station would draw water from the Phelps Canal until the peak storage elevation of 2,361 feet is reached. A radial gate is generally more cost effective than sluice gates, but radial gates are not able to operate with the high water surface elevation alternating from the canal side to the storage side. When the storage area is not in use, approximately 11 feet of water will need to be prevented from flowing into the storage area. When the storage is at peak stage, approximately 4 feet of water will need to be prevented from flowing into the canal. This preliminary gate analysis identified that a sluice gate would be more suitable than a radial gate with this alternating high water side. The pump station will need to operate at a rate to exceed seepage losses while filling the storage area in a reasonably short amount of time prior to the SDHF event. It is estimated a flow rate of 240 cfs would fill the storage area from 2,357.4 feet to 2,361 feet over a 10-day period.

Photo 4.3-3 – Plum Creek upstream of the Phelps Canal Siphon

The outlet works consists of a radial gate, 30 feet wide and 20 feet high, that will discharge directly into the south channel of the Platte River. The radial gate costs appeared much lower than an equivalently sized multiple sluice gate structure. The design will need to dissipate the energy prior to the bridge, or to armor the opening under the bridge to prevent additional scour.

In addition to the radial gate outlet, a labyrinth weir is needed to handle the flood flows from Plum Creek. 4,000 cfs was used as the Plum Creek 100-yr design flow for the weir. The labyrinth weir consists of 469 feet of sheet pile and a concrete outlet. The cost for this weir is listed in Appendix C.

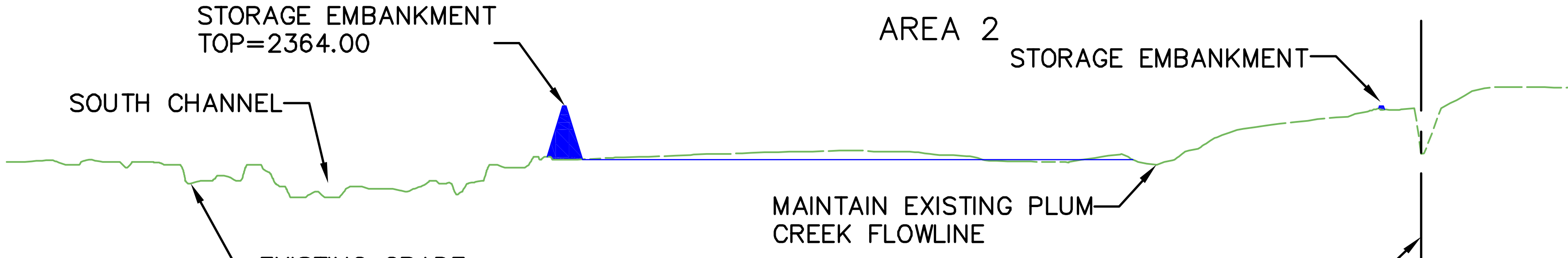


LEGEND

- STORAGE AREA BOUNDARY
- EXISTING GRADE
- PROPOSED STORAGE
- TOP OF EMBANKMENT
- EXISTING MAJOR CONTOUR
- BACKWATER LIMITS AT PEAK STAGE

NOTE:

*STORAGE AREA WILL REQUIRE PUMPS TO FILL BETWEEN ELEVATION 2357 TO ELEVATION 2361.



CROSS SECTION B-B
NOT TO SCALE

J-2 Return Alternative 2 Stage Storage - Area 2				
Elevation	Area (sf)	Area (acre)	Incremental Storage (acre-ft)	Total Storage (acre-ft)
2341	442,097	10	0	0
2342	1,706,368	39	25	25
2343	3,044,434	70	55	79
2344	4,930,124	113	92	171
2345	6,765,381	155	134	305
2346	12,593,574	289	222	527
2347	12,854,484	295	292	819
2348	13,341,838	306	301	1,120
2349	13,959,225	320	313	1,433
2350	14,597,327	335	328	1,761
2351	15,450,584	355	345	2,106
2352	16,531,221	380	367	2,473
2353	17,471,798	401	390	2,863
2354	18,061,498	415	408	3,271
2355	18,743,860	430	422	3,694
2356	19,769,331	454	442	4,136
2357	20,972,974	481	468	4,604
2358*	21,253,566	488	485	5,088
2359*	21,519,828	494	491	5,579
2360*	21,795,508	500	497	6,076
2361*	22,061,056	506	503	6,580

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PROJECT: 09-1466	J-2 RETURN ALTERNATIVE 2
DRAWN BY: CRL	GOSPER AND PHELPS COUNTY, NEBRASKA
DATE: 1.27.10	

OLSSON ASSOCIATES

EXCAVATION AREA 2

FIGURE 4.3-7

PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

4.3.2.3 Key Features of Area 3

Location

Area 3 is bounded by the Platte River on the north, J-2 Return Canal on the south, J-2 Return on the east and Road 435 on the west. It was decided not to build the structure downstream of J-2 Return gate in order not to impede the 2,000 cfs that could be delivered through the J-2 Return gate during a SDHF event. A remnant of one of the southern channels of the Platte River runs through the site and the northern half is identified as Danielson Island on a USGS quadrangle. It was assumed surface water flowing down the abandoned channel would be routed north and into the Platte River. Under these conditions, the contributing drainage area is zero.

Earthwork

The available LIDAR data did not extend beyond the banks of the Platte River. The NED set was obtained and added to the available LIDAR points. The combined points were used to create a terrain model and then used to estimate earthwork volumes. The average ground elevation through this area is approximately 2,360 feet. The lowest elevations are along the remnant of the Platte River. In the northeast corner, this elevation is approximately 2,355 feet. There is a large jump in elevation at this location between the LIDAR data set and the NED. The Platte River flow line in this area is 2,345 feet which is based on the more reliable LIDAR data. The highest existing elevation within the storage area is in the southwest corner near Road 435 and is approximately 2,362 feet.

The top of the J-2 Return Canal in this area is 2,362.4 feet. Leaving 3 feet of freeboard sets the maximum gravity fill elevation at 2359.4 feet. With the bottom of storage set at 2,355 feet, this would only allow for 1,749 ac-ft of storage based on gravity fill. To reduce the amount of haul from the site and to further increase the storage, a pump station was investigated to increase the storage height to 2,366 feet. The pump station is discussed in more detail in the Inlet and Outlet works discussion. Elevation 2,366 feet was selected as a reasonable estimate of the allowable ponding depth based on adjacent infrastructure, which includes a residence and the Canaday Steam Plant. It might be possible to increase the ponding depth with only a marginal increase in cost, but the adjacent developed conditions will need to be investigated further.

With the bottom of excavation was set at 2,355 feet, it resulted in approximately 3.4 million cubic yards of excavation over a 385-acre bottom (5 feet to 6 feet of excavation on average). As currently configured, there is approximately 3.2 million cubic yards of excess material. Excluding the Platte River channel remnant, the excavation is fairly uniform and increases slightly in the southwest corner. The southwest corner could be raised to reduce haul but a fully balanced earthwork site likely is not possible. The cost estimate assumes the soil-wasting area would be adjacent to the site and would be accessible with scraper equipment. The accuracy of the elevation data is a key consideration when looking at these large areas. For instance if the elevation was incorrect by an average of 1 foot, this may increase or decrease excavation by 0.6 million cubic yards.

Embankment

The embankment includes 3 vertical feet of freeboard, a 14-foot top width and 3H: 1V side slopes. The maximum height the embankment is above existing grade is on the east side and is 24 feet. The embankment would be about 6 feet high along the south, which is approximately 3 feet higher than the J-2 Return dikes. The on-site material appears to be suitable for construction of the embankments.

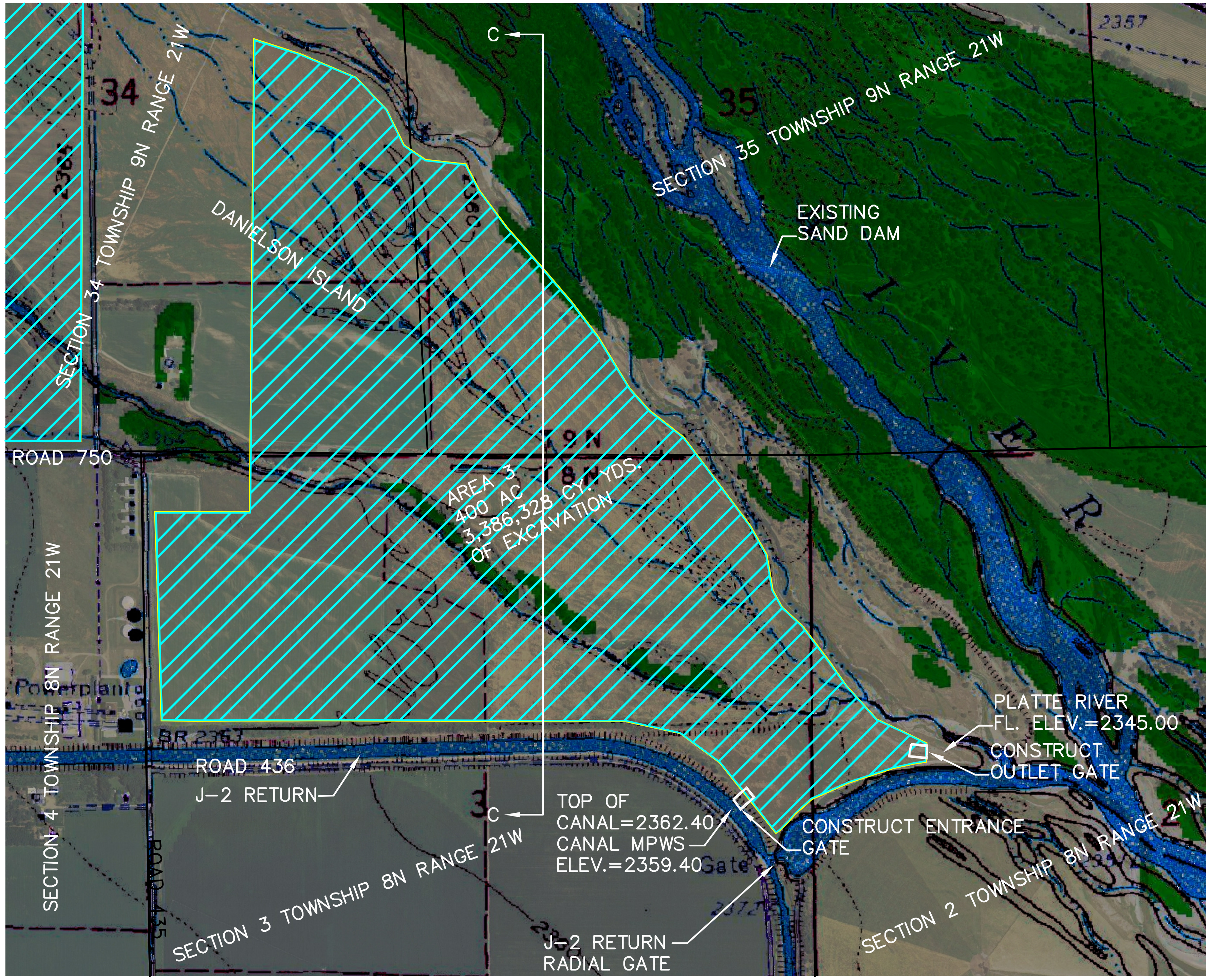
The structure would be considered a regulatory dam by State of Nebraska due to the storage volume. The preliminary hazard classification is significant hazard, class B. There is a residential structure near the intersection of Road 750 and Road 435 that would be 3 feet to 4 feet below the top of the embankment. In addition, Canaday Steam Plant sits across Road 435 to the west. The steam plant is approximately 3 feet below the top of embankment. It appears a potential breach would flow directly into the Platte River and would not inundate these structures. A hazard class analysis would need to be performed along with alternate considerations for peak ponding depths to verify these structures would not be inundated and hence necessitate a high hazard classification. Approximately 0.4 million cubic yards of fill are required to construct the embankments.

Inlet and Outlet Works

A new inlet gate and pump station would be required from J-2 Return Canal upstream of the J-2 Return radial gate. Figure 4.3-6 shows the suggested pump and sluice gate location. Operationally, the sluice gates would be open and water would gravity flow from the J-2 Return into the storage area up to the canals maximum water surface elevation of 2,359.4 feet. The canal flow line at this location is approximately 2,344.3 feet; the bottom of the proposed storage area is 2,355 feet. Once the storage area has reached the maximum gravity fill elevation, the sluice gates would close and the pump station would draw water from the J-2 Return until the peak storage elevation of 2,366 feet is reached. A radial gate is generally more cost-effective than sluice gates; however, radial gates are not able to operate with the high water surface elevation alternating from the canal side to the storage side. When the storage area is not in use, the approximately 12 feet of water will need to be prevented from flowing into the storage area. When the storage is at peak stage, approximately 8 feet of water will need to be prevented from flowing into the canal. This preliminary gate analysis identified that a sluice gate would be more suitable than a gate with this alternating high water side.

The pump station will need to operate at a rate to exceed seepage losses, while filling the storage area in a reasonably short amount of time prior to the SDHF event. It is estimated a flow rate of 300 cfs would fill the storage area from 2,359.4 feet to 2,366 feet over an eight-day period.

The outlet works consists of a radial gate, 30 feet wide by 20 feet high, which will discharge directly into the south channel of the Platte River. The radial gates appear to be much less expensive than an equivalently sized multiple sluice gates structure. The design will need to dissipate the energy on the downstream side to prevent a scour hole in the Platte River.



J-2 Return Alternative 2 Stage Storage - Area 3				
Elevation	Area (sf)	Area (acre)	Incremental Storage (acre-ft)	Total Storage (acre-ft)
2346	4,016	0	0	0
2347	16,558	0	0	0
2348	28,845	1	1	1
2349	32,001	1	1	1
2350	34,814	1	1	2
2351	37,539	1	1	3
2352	40,343	1	1	4
2353	43,376	1	1	5
2354	47,320	1	1	6
2355	16,759,270	385	193	199
2356	16,820,489	386	385	584
2357	16,881,784	388	387	971
2358	16,943,155	389	388	1,359
2359 *	17,004,601	390	390	1,749
2360 *	17,066,122	392	391	2,140
2361 *	17,127,719	393	392	2,533
2362 *	17,189,390	395	394	2,927
2363 *	17,251,137	396	395	3,322
2364 *	17,312,959	397	397	3,719
2365 *	17,374,857	399	398	4,117
2366 *	17,436,830	400	400	4,516

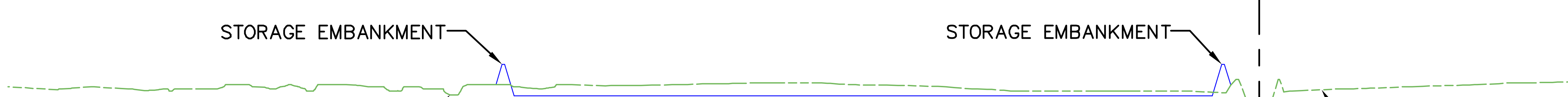
N

0' 500' 1000'
SCALE IN FEET

LEGEND

- STORAGE AREA BOUNDARY
- EXISTING GRADE
- PROPOSED STORAGE
- TOP OF EMBANKMENT
- EXISTING MAJOR CONTOUR

AREA 3



CROSS SECTION C–C
NOT TO SCALE

NOTE:

*STORAGE AREA WILL REQUIRE PUMPS TO FILL BETWEEN ELEVATION 2359 TO ELEVATION 2366.

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PROJECT: 09-1466	J-2 RETURN ALTERNATIVE 2 GOSPER COUNTY, NEBRASKA	OLSSON ASSOCIATES	EXCAVATION AREA 3	FIGURE 4.3-8
DRAWN BY: CRL				
DATE: 1.27.10				

4.3.2.4 Key Features of Area 4

Area 4 is bounded by the Platte River on the north and Road 750 the south. Half-section lines for sections 33 and 34 were followed for the eastern and western boundary. A remnant of one of the southern channels of the Platte River runs through the site. The site's northern half is identified as Danielson Island on the USGS quadrangle. It was assumed surface water flowing down the abandoned channel would be routed north and into the Platte River. Under these conditions, the contributing drainage area is zero.

There are two major electric transmission lines that run through the storage area and are assumed to originate from the Canaday Steam Plant. It might be possible to reduce the size of the storage area in the southwest corner to avoid one set of transmission lines. For the lines that transect the northwest corner, it might be possible to excavate around the towers and leave the lines in place. An access and maintenance agreement with the utility would need to be negotiated regarding impacts to the transmission lines.

Earthwork

The available LIDAR data did not extend beyond the banks of the Platte River. The NED set was obtained and added to the available LIDAR points. The combined points were used to create a terrain model and then used to estimate earthwork volumes. The average ground elevation through this area is approximately 2,367feet. The lowest elevations are along the remnant of the Platte River. In the southwest corner, this elevation is approximately 2,360 feet. The highest existing elevations within the storage area are in the northwest and southwest corners at approximately 2,371 feet.

A new inlet canal would be needed for delivery of water from the J-2 Return Canal to the storage area. The 2,800 foot long canal was tentatively located on the west side of the Canaday Steam Plant and has an average depth of 14 feet. The maximum water surface for J-2 Return in this area is 2362.4 feet and with the bottom of storage set at 2,361 feet, would allow only for 960 ac-ft of storage, based on gravity fill. To reduce the amount of haul from the site and to increase storage, a pump station was investigated to increase the storage height to 2,370 feet. The pump station is discussed in more detail in the Inlet and Outlet works discussion. An elevation of 2,370 feet was selected as an estimate of the allowable ponding depth based on adjacent infrastructure, residence and the Canaday Steam Plant. This ponding elevation is approximately 5 feet higher than existing grade near the developed areas. Additional storage may not be possible due to the adjacent development, but the adjacent developed conditions will need additional investigation if this location is selected for further analysis.

The bottom of excavation was set at 2,361 feet, which resulted in approximately 8 million cubic yards of excavation over a 640-acre bottom (7 feet to 8 feet of excavation on average). As currently configured, there is approximately 7.8 million cubic yards of excess material. Excluding the Platte River channel remnant, the excavation is fairly uniform and slightly increases in the southwest and northeast corner. The southwest and northeast corners could be raised to reduce haul but a fully balanced earthwork site is not likely possible. The cost estimate assumes the soil-wasting area would be adjacent to the site and would be accessible with scraper equipment. The accuracy of the elevation data is a key consideration when looking at these large areas. For instance, if the elevation was incorrect by an average of 1 foot, this may increase or decrease excavation by 1 million cubic yards.

Embankment

The embankment includes 3 vertical feet of freeboard, a 14-foot top width and 3H: 1V side slopes. The maximum height the embankment is above existing grade is 13 feet, and it is on the southeast side. An embankment would not be needed on the western side. The on-site material appears to be suitable for construction of the embankments.

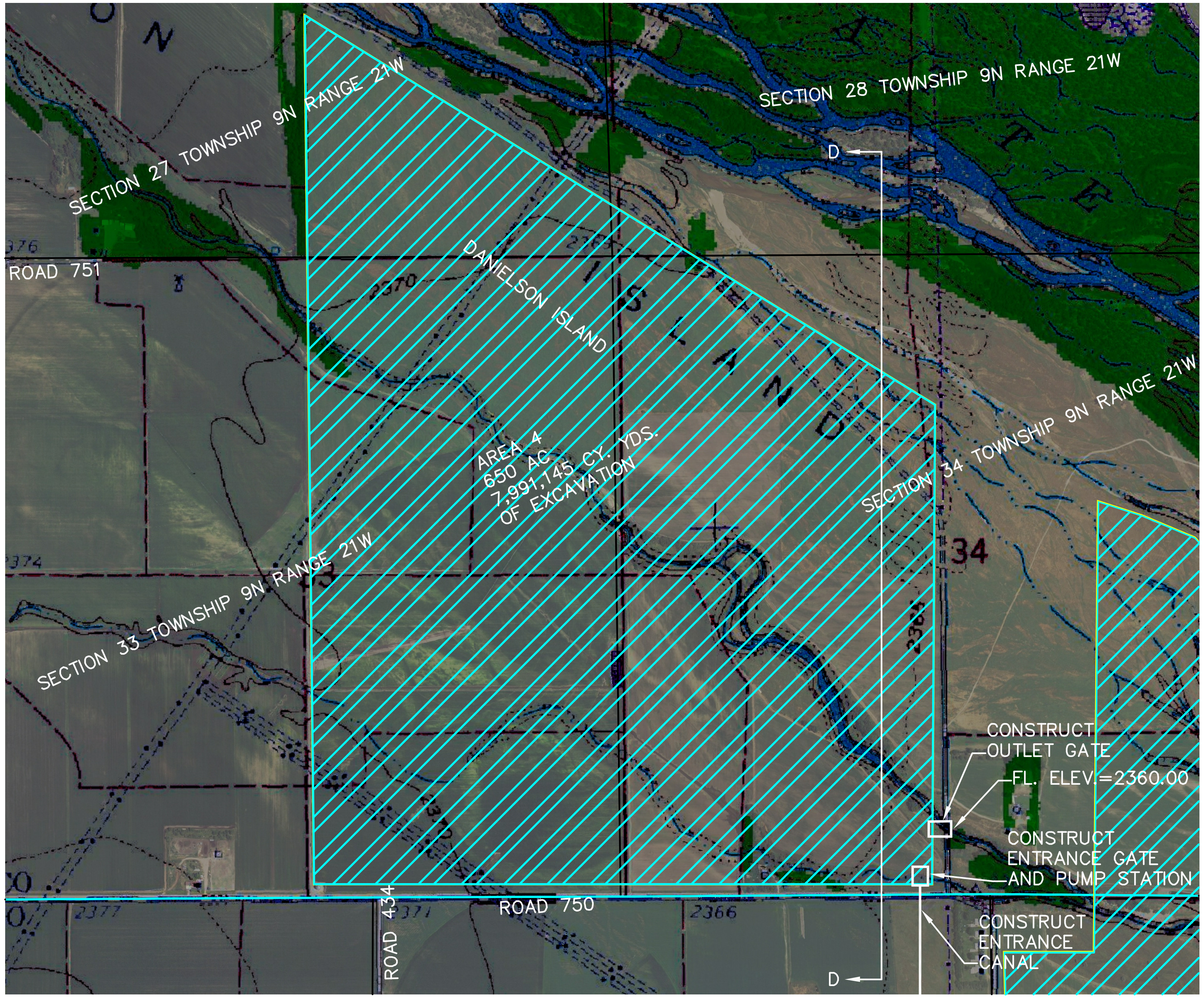
The structure would be considered a regulatory dam by the State of Nebraska due to the storage volume. The preliminary hazard classification is high hazard (class C). There is a residential structure near the intersection of Road 750 and Road 435 that would be 3 feet to 4 feet below the top of the embankment. In addition, Canaday Steam Plant is adjacent to the fill canal. The steam plant is approximately 3 feet below the top of embankment. It is not clear if a potential breach would flow directly into the Platte River without inundating these structures. A hazard class analysis would need to be performed along with alternate considerations for peak ponding depths to verify these structures would not be inundated; if the structures aren't inundated, a high hazard classification would not be needed. Approximately 0.4 million cubic yards of fill is required to construct the embankments.

Inlet and Outlet Works

A new inlet canal, gate and pump station would be required from J-2 Return Canal upstream of the J-2 Return radial gate. Figure 4.3-7 shows the inlet canal location shows the suggested pump and sluice gate location. Operationally, the sluice gates would be open and water would gravity flow from the J-2 Return into the storage area up to the canal's maximum water surface elevation of 2,362.4 feet. The canal flow line at this location is approximately 2,342 feet and the bottom of the proposed storage area is 2,361 feet. Once the storage area has reached the maximum gravity fill elevation, the sluice gates would close and the pump station would draw water from the J-2 Return until the peak storage elevation of 2,366 feet is reached. A radial gate is generally more cost-effective than sluice gates; however, radial gates are not able to operate with the high water surface elevation alternating from the canal side to the storage side. When the storage area is not in use, approximately 12 feet of water must be prevented from flowing into the storage area. When the storage is at peak stage, approximately 4 feet of water must be prevented from flowing into the canal. The preliminary gate analysis identified a sluice gate would be more suitable than a gate with this alternating high water side.

The pump station will need to operate at a rate exceeding seepage losses to fill the storage area in a reasonably short amount of time prior to the SDHF event. It is estimated a flow rate of 300 cfs would fill the storage area from 2,362 feet to 2,366 feet over a 10-day period.

The outlet works consists of a radial gate, 30 feet wide by 20 feet high, which will discharge directly into the south channel of the Platte River. The radial gate appears to be much less expensive than an equivalently sized multiple sluice gates structure. The design will need to dissipate the energy on the downstream side to prevent a scour hole in the Platte River.



J-2 Return Alternative 2 Stage Storage - Area 4

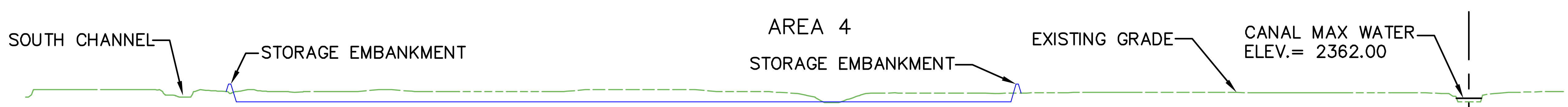
Elevation	Area (sf)	Area (acre)	Incremental Storage (acre-ft)	Total Storage (acre-ft)
2360.5	27,826,729	639	0	0
2361	27,859,787	640	320	320
2362*	27,925,961	641	640	960
2363*	27,992,213	643	642	1,602
2364*	28,058,544	644	643	2,245
2365*	28,124,954	646	645	2,890
2366*	28,191,442	647	646	3,536
2367*	28,258,008	649	648	4,184
2368*	28,324,653	650	649	4,834
2369*	28,391,376	652	651	5,485
2370*	28,458,178	653	653	6,137

NOTE:

*STORAGE AREA WILL REQUIRE PUMPS TO FILL BETWEEN ELEVATION 2362 TO ELEVATION 2370.

LEGEND

- STORAGE AREA BOUNDARY
- EXISTING GRADE
- PROPOSED STORAGE
- TOP OF EMBANKMENT
- EXISTING MAJOR CONTOUR



CROSS SECTION D–D
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PROJECT: 09-1466 DRAWN BY: CRL DATE: 1.27.10	J-2 RETURN ALTERNATIVE 2 GOSPER COUNTY, NEBRASKA	EXCAVATION AREA 4	FIGURE 4.3-9

OLSSON ASSOCIATES
PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

4.3.2.5 Key Features of Area 1 and 2 Combination

As the independent J-2 Alternatives did not completely satisfy the stated goal of a 3-day, 2,000 cfs release for SDHF augmentation a cursory look was taken at a combination option. The two most attractive sites based on the cost per ac-ft are Areas 1 and 2 within the J-2 Alternative 2.

Area 1 has the storage volume to supply 2,000 cfs over 1.5 days and then flow tapers off due to a lack of storage and available head. Area 2 is currently sized to deliver 2,000 cfs for approximately 1 day and then flows taper off due to lack of storage and available head.

The SDHF evaluation of this combined option was modeled so that Area 2 would supplement the flow as needed to reach 2,000 cfs after day 1.5 when the flow from Area 1 tapered off. The SDHF and Target Flow analyses for this combination option were performed with the gravity fill storage volumes of both reservoirs because this analysis was performed prior to adding the volume above gravity fill for Area 2.

For the combination option of Alternative 2, Area 1 and 2, the capital cost for the alternatives were not evaluated or changed for the scoring purposes. However, the Area 2 gate is sized to release 2,000 cfs whereas only 1,750 cfs would be needed. It is believed the amount of excavation within either reservoir could be slightly reduced if the remaining flow was delivered from the other reservoir.

4.3.3 J-2 Alternative 3, Phelps Canal 9.7

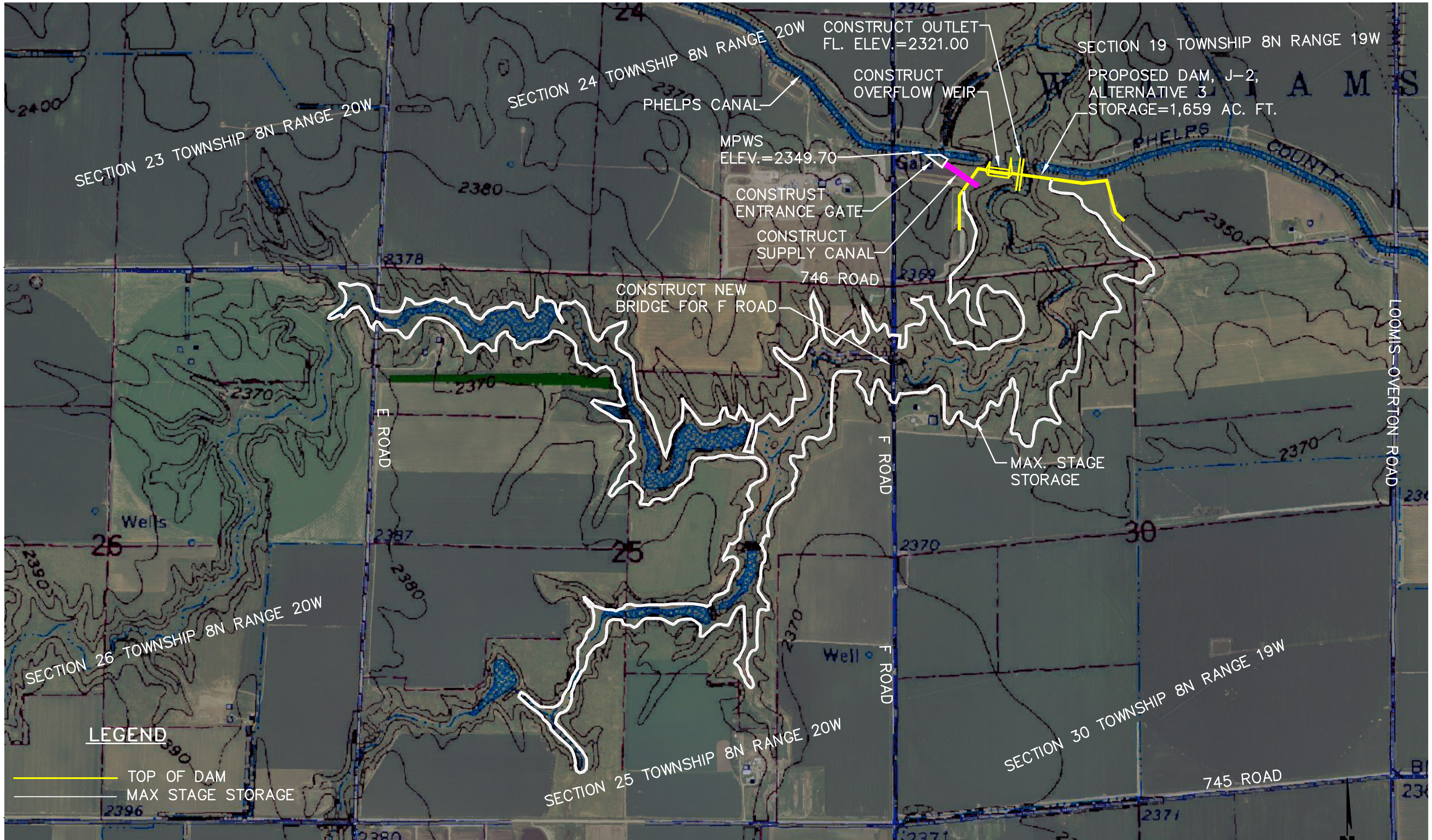
Alternative 3 involves constructing an embankment across the unnamed creek, immediately upstream of the Phelps Canal siphon at canal mile station 9.7. The dam would receive inflow from Phelps Canal and from the unnamed creek, which is a tributary to the Platte River. Plan view location and stage-storage for this alternative is shown in Figure 4.3-10. The reservoir is limited by the small amount of storage available in this location, before it ponds water over the residence to the west. The downstream flow delivery will reach the Platte River downstream of Overton, which is the evaluation point for the alternatives.

The contributing drainage area for this alternative is 17 square miles. Due to the storage and height of embankment, this will be a regulatory dam by State of Nebraska statutes. The downstream land use was reviewed utilizing aerial photographs and USGS topographic mapping. There are two residences and five roadway crossings that could be at risk in the event of a dam breach. For this reason, the preliminary hazard classification is high hazard, Class C. A hazard class analysis would need to be performed to verify this assumption. The watershed yield from times of excess flow could possibly be stored and used later to reduce shortages to target flows.

Based on the hazard classification, the embankment must be designed to contain the probable maximum precipitation (PMP) event without overtopping the structure. The 24-hour all season PMP for this area is 22.2 inches based on the state-adopted PMP study. The drainage area, curve number, time of concentration, and storage information was input into the dam sizing software SITES, produced by the U.S. Department of Agriculture (USDA). The proposed outlet works will consist of a twin 15-foot-wide sluice gate; a 30-foot-wide by 15-foot-tall box culvert; and a 100-foot-wide to 200-foot-wide auxiliary spillway vegetated chute. It is anticipated that turf reinforcement mat will be needed on the auxiliary spillway. The auxiliary spillway stability analysis should be evaluated in the next phase if this option is selected for further analysis. Based on this outlet works configuration, the top of embankment will be approximately 2,360 feet. The top of useable storage is 2,351 feet, which would store 1,660 ac-ft of water.

New inlet works would be needed to convey water from the upstream side of the Phelps Canal wasting station into the storage site. It is anticipated that due to the contributing drainage area the water surface in the storage area will occasionally exceed the water surface in the canal. For this reason a sluice gate is preferred over a radial gate. The inlet canal would affect a feedlot lagoon. It is assumed the existing lagoon liner would be salvaged and the volume lost due to the inlet canal would be replaced with a new lagoon configuration that would extend further south.

Placing the embankment on the downstream side of the Phelps Canal wasting station was briefly considered. A downstream location would have the advantage of increased storage and utilization of the existing wasting gate to fill the storage site. The disadvantages would include a requirement for extensive canal work to prevent surcharging the canals during a high flow event on the creek. The storage area could easily back water over the wasting gates. In addition, due to the 17-square-mile drainage area, the surface water runoff could potentially pond more than 30 feet of water on top of the existing siphon and potentially threaten its structural stability.



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SCALE IN FEET

PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM

PROJECT: 09-1466 DRAWN BY: CRL DATE: 1.27.10	J-2 RETURN ALTERNATIVE 3 PHELPS COUNTY, NEBRASKA	OLSSON ASSOCIATES	PLAN VIEW	FIGURE 4.3-10

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DATE: Jan 27, 2010 3:42pm USER: cluttrell
XREFS: 09_1466_LowResolutionAerials 009-1466_SectionTownship 09_1466_Pbase J2_TOPO TitleBlock

4.4 Seepage Analysis

A calculation of the seepage rate from J-2 Alternative 1 and Alternative 2 was an important consideration for the feasibility of these alternatives. Without field data on the seepage rates, estimates were based on published hydraulic conductivity data on Platte River alluvial deposits, seepage data from other reservoirs and canals and calculations of seepage using standard engineering equations. Because the rate of seepage from each of the J-2 alternatives is a critical aspect of the engineering feasibility analysis, field testing should be performed to provide site-specific data for the next stage of this analysis. The method described in Chen, et al, 2009 describes a new method for mapping vertical seepage flux in streambeds that may be applicable to this site.

For the alluvial deposits that overlie the Ogallala group, the horizontal hydraulic conductivity (Kh) was estimated at 125 feet to 240 feet per day (Cannia, Woodward and Cast, 2006). The vertical conductivity (Kv) values were estimated at 10% of the horizontal. East of the J-2 alternative sites (from Kearney to Columbus) Kv values were measured using electrical conductivity logs and permeability tests on sediment cores (Chen, Burbach and Cheng, 2008; and Chen, 2005). Kv values in the samples collected between Kearney and Gibbon had the highest Kv values in the shallow cores with an average Kv of 125 feet per day.

Using the published Kv values and seepage rate equations from Chow (1969), seepage rates for Alternative 1 and Alternative 2 were calculated and then compared to seepage data from existing reservoirs. As described in Section 5-b-ii, for Alternative 1, the Dupuit-Forchheimer equation was used to estimate seepage. For Alternative 2, the Chow method was used to estimate seepage. The Chow method incorporates an organic layer with a low hydraulic conductivity into the equation, which represents placement of topsoil in the base of the reservoir during construction. Table 4.4-1 lists the peak range of seepage rates calculated using the two methods in comparison with seepage data from reservoirs in Nebraska.

Table 4.4-1 Estimated and Published Seepage Rates

Reservoir Name	*Sediment Type*	*Published Seepage Rate in ft/mo per surface acre*	*Notes with References Identified*
Elwood Reservoir	*Loess*	*0.335*	*Existing reservoir*[1]
Big Sandy Dam	*Sand and Gravel*	*0.56*	*Reservoir has an organic layer*[2]
Clay County Dam	*Sand and Gravel*	*0.44*	*Reservoir has an organic layer*[3]
York County Dam	*Sand and Gravel*	*0.45*	*Reservoir has an organic layer*[3]
Reservoir Name	***Sediment type***	***Estimated Peak Seepage rate in ft/mo per surface acre***[4]	***Notes with references identified***
Alternative 1	*Sand and Gravel*	*309*	*Estimated without low Kv layer*
Alternative 2	*Sand and Gravel*	*18*	*Estimated with low Kv layer*
Alternative 3		*3*	*Estimated without low Kv layer*

[1] CH2M Hill TOC Analysis
[2] Eisenhauer, Potential for Groundwater Recharge with Seepage from Flood-Retarding Reservoirs in South Central Nebraska (1982).
[3] Little Blue Natural Resources District, Big Sandy Creek Watershed NRC Application.
[4] Dupui-Forcheimer calculation method (Chow, 1969)

The difference in seepage rates between the two alternatives is based on placement of a low hydraulic conductivity layer during construction of the reservoir. Without the low permeability material, the seepage rates are more comparable to seepage from unlined sandy canals which can range from one-tenth foot to 4 feet a day (personal communication D. Woodward, 2009).

Reservoir seepage for Alternative 1 was developed using the Dupuit-Forchheimer method described in “Advances in Hydroscience” (page 139) by Ven Te Chow. The seepage rate will vary based on the ponding depth. The infiltrated water will reappear immediately into the downstream storage area and is assumed to be surface water upstream of Overton. Due to the very high permeability rates and the short distance of flow, no lag time was assumed between infiltration and re-emergence.

Reservoir seepage for Alternative 2 and Alternative 3 was developed using the method described in “Advances in Hydroscience” (page 139) by Ven Te Chow for reservoirs with an organic layer at their perimeter. This method assumes that topsoil will be replaced in the reservoir after construction, providing a thin layer of low hydraulic conductivity. A rate of 0.028-foot per day was chosen as the seepage rate of topsoil. Similar to Alternative 1, it was assumed the seepage water will combine with the high water table in the Platte River Valley and will reappear as surface water upstream of Overton. Due to the very high permeability rates of the Platte River gravels and the short distance of flow, it was assumed there is no lag between infiltration and re-emergence.

4.5 Short Duration High Flow (SDHF) Evaluation

An inflow hydrograph was developed for each reservoir. The inflow hydrograph has three distinct parts: initial flow to fill the reservoirs, flow during the SDHF event and typical flows following the SDHF event. The inflow hydrograph was developed with input from CNPPID and ED Office staff. The peak operating efficiency for the J-2 hydropower plant is 1,675 cfs. It was assumed this rate would be run continuously during the filling process for the sites filled from the J-2 Return. Olsson found the average daily volume from the J-2 Return, of all the available years of data in the month of March (1947 – 2006) to be 2,300 ac-ft. Running the hydropower plant at 1,675 cfs yields 3,300 ac-ft per day, which would require that some water be routed down to the reregulating reservoir from the EA in Lake McConoughy prior to a SDHF. The maximum conveyance capacity for Phelps Canal is 1,410 cfs based on the construction plans, however the inlet flow was limited to 1,000 cfs for this analysis based on conversations with CNPPID.

The maximum outflow capacity of the J-2 hydropower plant is 2,000 cfs. It was assumed this flow rate would be generated during the three-day SDHF event and would be released to the Platte River through the J-2 Return. Alternatives in line with the J-2 Return would need to pass this water in addition to stored water. CNPPID staff indicated the J-2 hydropower plant can be ramped up or down very quickly, so a long ramp up or down curve is not needed.

Because all the J-2 reservoir storage areas draw water from the J-2 Return or Phelps Canal, the Canaday Steam plant could be affected by the fluctuating water surface in the canal. The steam plant utilizes cooling water from J-2 canal. The stream plant needs a nearly constant water surface with variable discharge, which causes the canal to fill and empty. Coordination between PRRIP and the steam plant will be necessary prior to and during the SDHF event.

Following the three-day SDHF event, CNPPID advised that the typical hydropower flow cycling portion of the run should be determined from the average operations. Using 1,675 cfs as the most efficient operating flow, the typical generation cycle runs from 7 a.m. to midnight for the 2,300 ac-ft typically available at this time of year. The resulting reregulation reservoir inflow hydrographs are depicted in the following figures.

Typical hydropower cycling operations are shown after the SDHF event of Alternative 1. Note that all flows from the J-2 Return, including hydropower cycling flow and three days of 2,000 cfs for the SDHF, must be routed through the J-2 Alternative 1 because it is in line with the J-2 Return (Figure 4.5-1). However, due to modeling constraints, the SDHF analysis was done with the reservoir gates completely open after the SDHF event. Therefore, the hydropower cycling mitigation shown is not the same as that evaluated in the hydropower cycling mitigation analysis, which is discussed in Section 4.7.

Because hydropower cycling flows are currently routed through the J-2 Return to the South Channel of the Platte River, hydropower cycling flows are not shown on the inflow hydrographs for either Alternative 2 or 3. The hydropower cycling flows are shown at Overton for all alternatives, but note that in this SDHF evaluation, the hydropower cycling flows do not travel through the Alternative 2 or 3 reservoirs.

Figure 4.5-1 – J-2 Return Short Duration High Flow Analysis – Inflow Hydrograph for Alternative 1

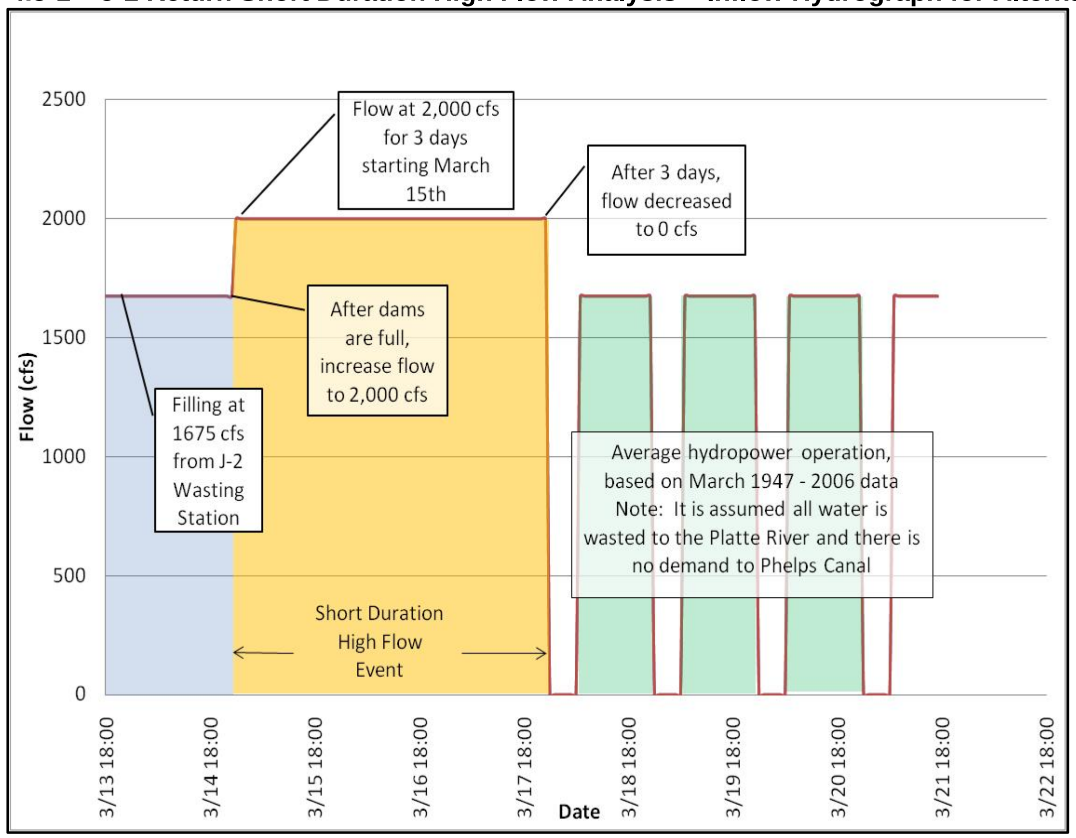


Figure 4.5-2 – J-2 Return Short Duration High Flow Analysis – Inflow Hydrograph for Alternative 2, Area 1

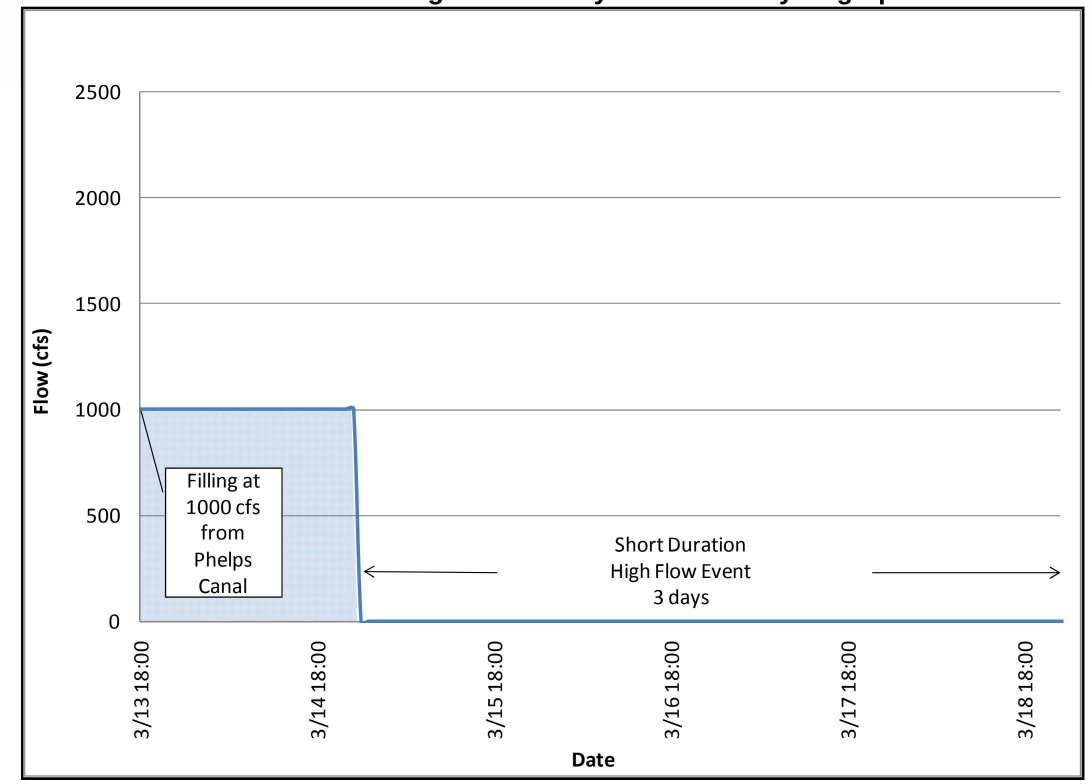


Figure 4.5-3 – J-2 Return Short Duration High Flow Analysis – Inflow Hydrograph for Alternative 2, Area 2

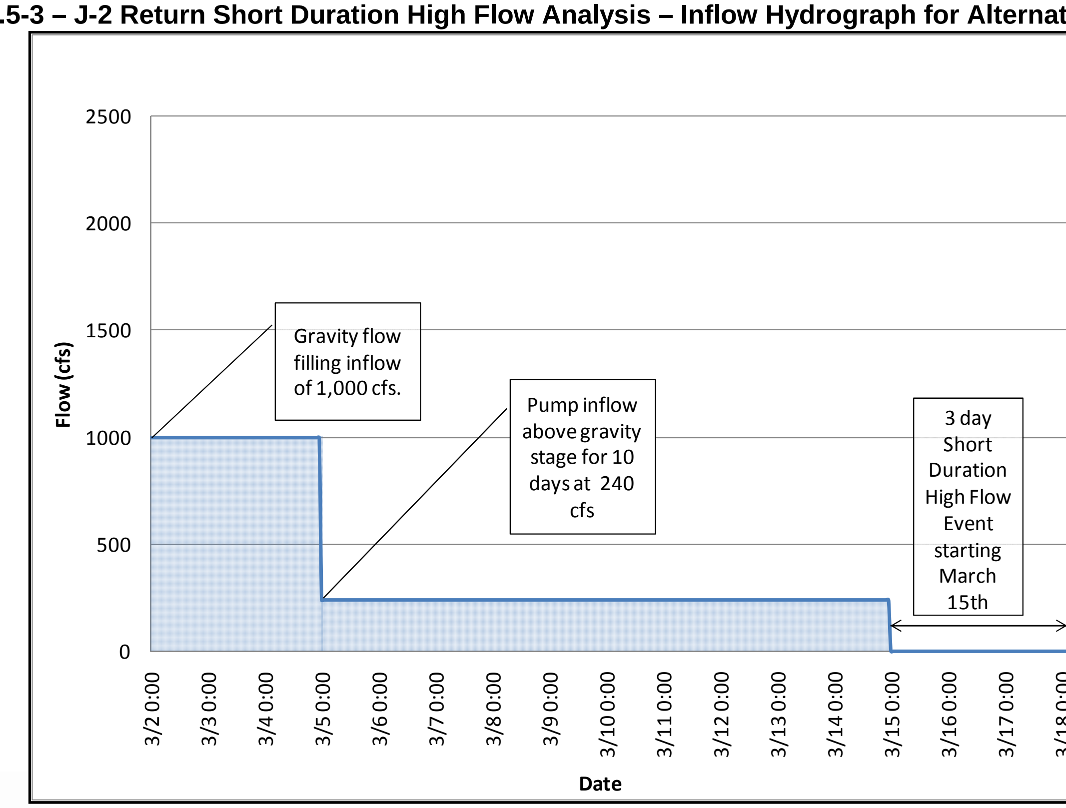


Figure 4.5-4 – J-2 Return Short Duration High Flow Analysis – Inflow Hydrograph for Alternative 2, Areas 3 & 4

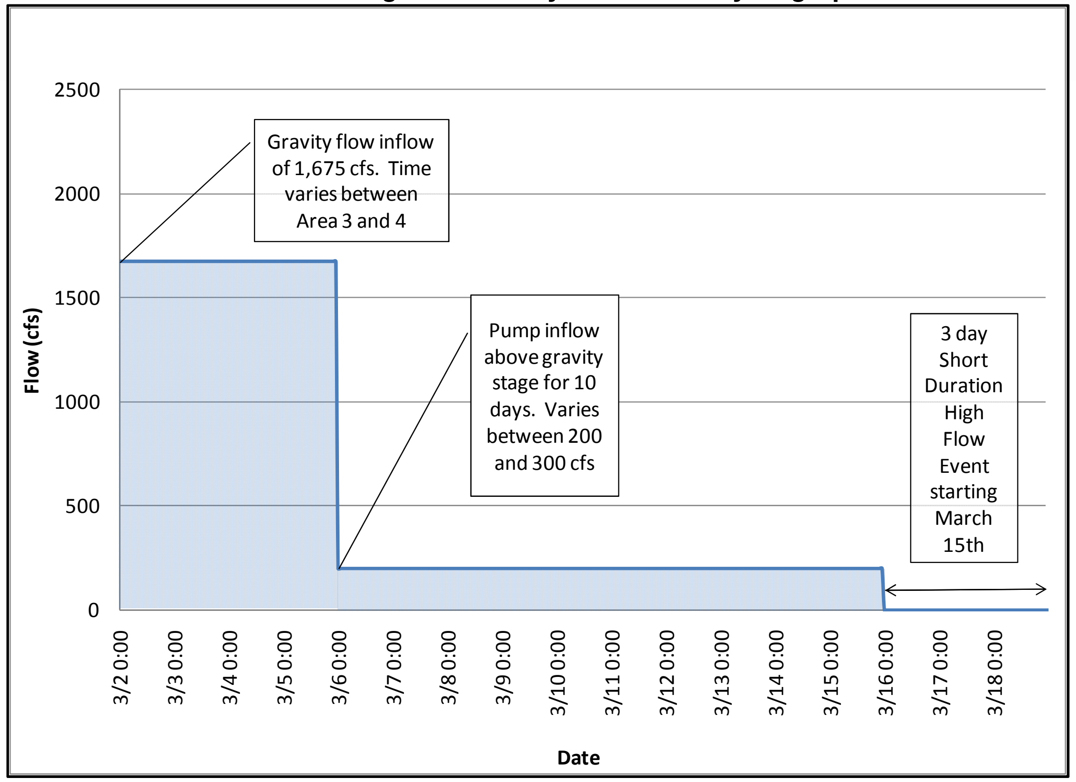


Figure 4.5-5 – J-2 Return Short Duration High Flow Analysis – Alternative 1, Dam D (48-ft wide radial gate)

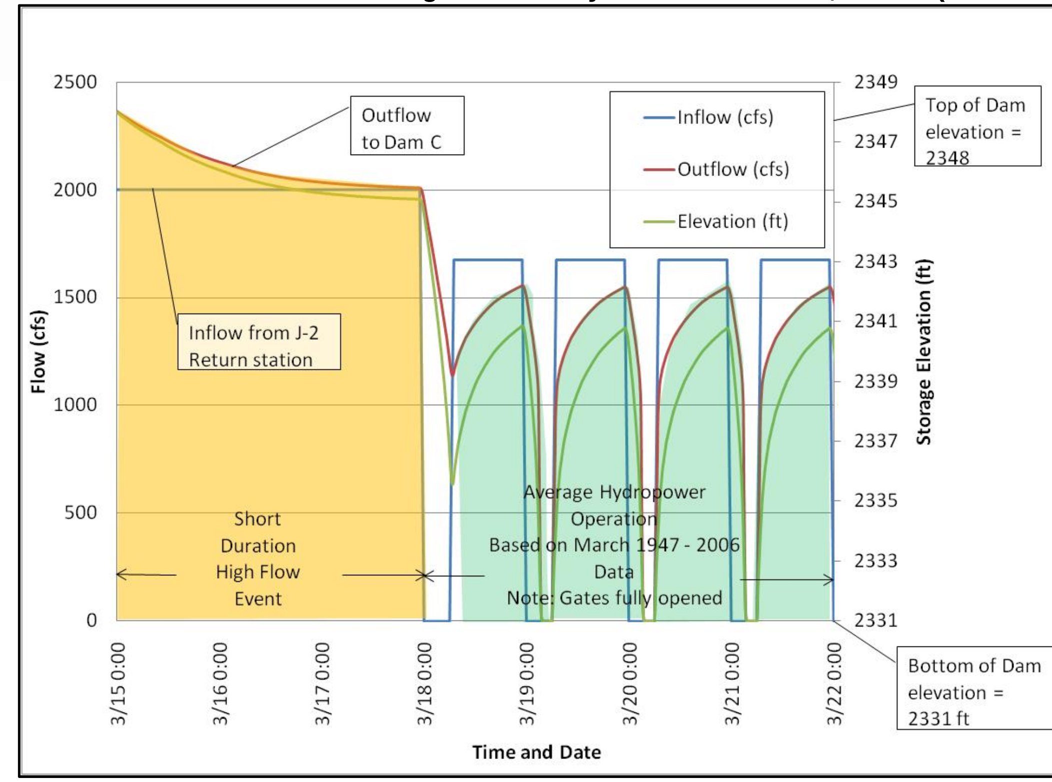
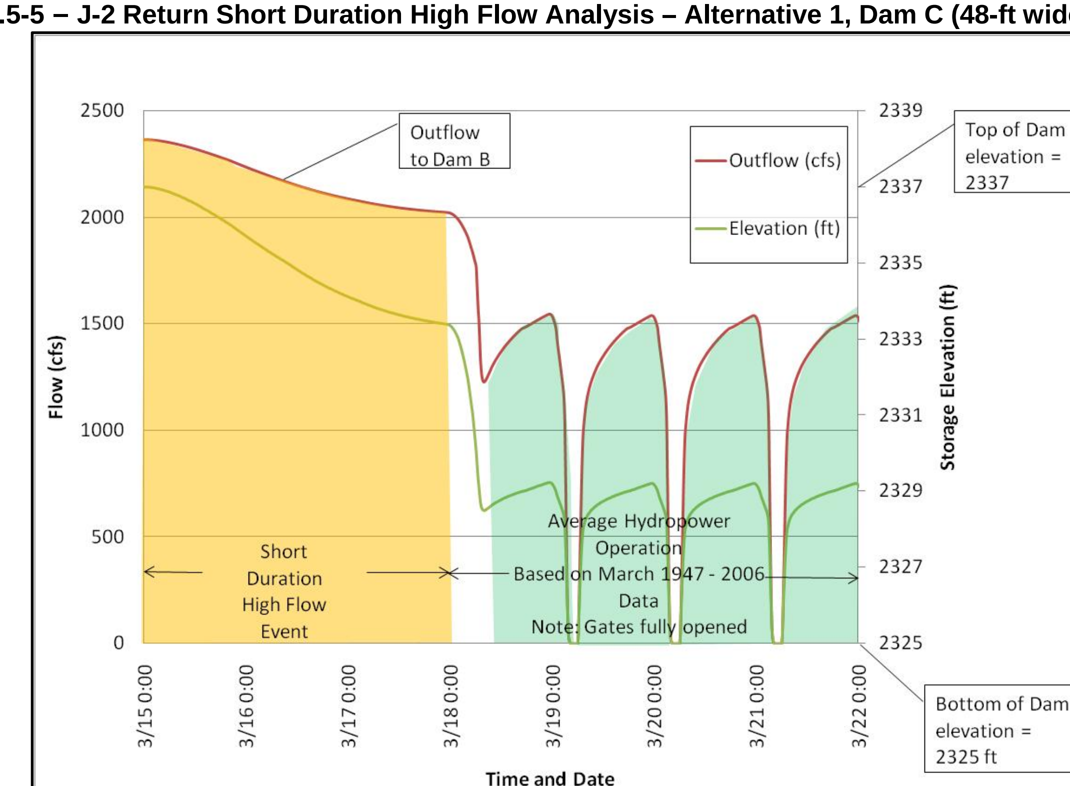


Figure 4.5-5 – J-2 Return Short Duration High Flow Analysis – Alternative 1, Dam C (48-ft wide radial gate)



For each alternative a variety of gate sizes were evaluated to optimize the three-day release rate versus the available storage. The shallower the storage site, the wider the gate needed to evacuate the water. The following figures (such as 4.5-5) charts show the drain down time for the final foot or two feet of water outside of the 3 day SDHF event. The storage at the bottom foot or two feet of the reservoir in general is not advantageous for the SDHF event. The bottom storage does have some benefit for target flows and mitigating hydropower cycling. If these are intended for just SDHF, the bottoms of the storage areas should be sloped to minimize excavation, enhance the head on the gates and minimize the retained water.

As a result of the iterative modeling process, the reservoirs for Alternative 1 are recommended to use 48-foot to 50-foot wide gates. The reservoirs for Alternative 2 are recommended to use 30-foot to 60-foot-wide radial gates, depending on the storage area being modeled. The Alternative 3 reservoir was modeled using a 30-foot-wide sluice gate. In all alternatives, the gates were partially opened and then slowly raised to generate a steady release rate as the storage areas drained. Once the water level decreased to the point the gates were no longer under pressure, the weir crest of the gate controlled the remainder of the outflow. The modeling results showing the storage and outflow rates are presented in the following figures. For Alternative 1, water flows first through Dam D (Figure 4.5-5), then C (Figure 4.5-6), B (Figure 4.5-7) and finally Dam A (Figure 4.5-8). Note that only the figure for Dam A, the most downstream dam, includes resulting Overton flows.

Figure 4.5-7 – J-2 Return Short Duration High Flow Analysis – Alternative 1, Dam B (48-ft wide radial gate)

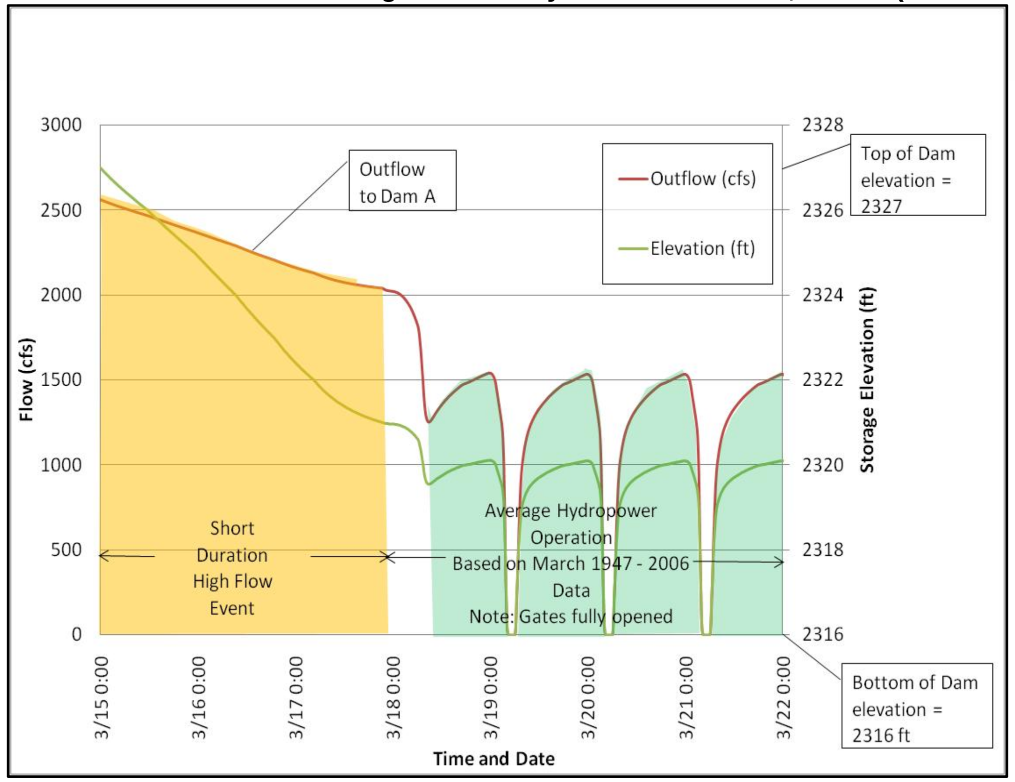
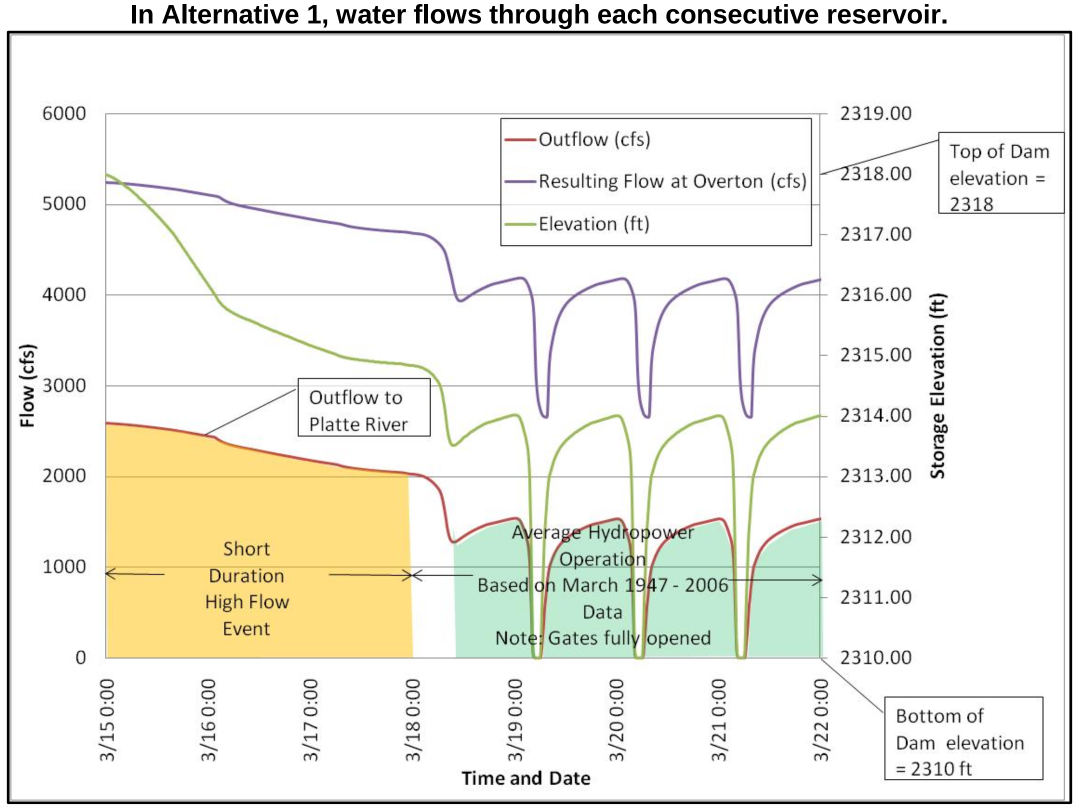


Figure 4.5-8 – J-2 Return Short Duration High Flow Analysis – Alternative 1, Dam A (50-ft wide radial gate). Note: In Alternative 1, water flows through each consecutive reservoir.



SDHF results for Alternative 2 are shown in Figures 4.5-9 through 4.5-12. Areas 1 and 2 receive flow from Phelps Canal, which was modeled with an operational capacity of 1,000 cfs. Therefore they have a modified inflow hydrograph as compared to Alternative 1. Areas 2, 3, and 4 have storage above the top of canal elevation and therefore cannot be completely gravity filled. Pumps would be needed to utilize the remaining storage. The results from Alternative 2, Area 1 and 2, show that neither area could provide 2,000 cfs for three days from the reservoir alone. Therefore, a chart (Figure 4.5-14) showing a combination of Area 1 and 2 is included, which does provide at least 2,000 cfs for three days from the two reservoirs. This combined option requires a longer fill time, which was not specifically modeled. Also note that normal hydropower flow cycling operations entering the river from the J-2 Return and at Overton are shown on these figures, but this water is not routed through J-2 Alternatives 2 and 3. A separate analysis of hydropower cycling mitigation is provided below in section 4.7.

Figure 4.5-9 – J-2 Return Short Duration High Flow Analysis – Alternative 2, Area 1 (40-ft wide radial gate)

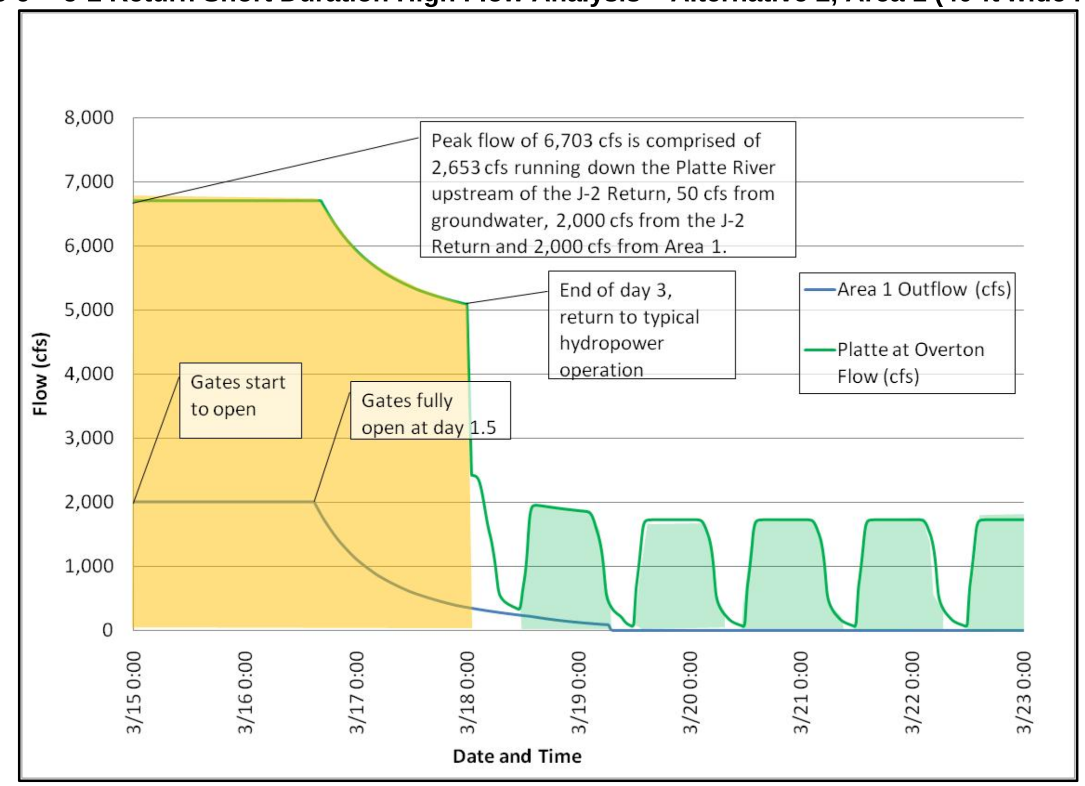


Figure 4.5-10 – J-2 Return Short Duration High Flow Analysis – Alternative 2, Area 2 (30-ft wide radial gate)

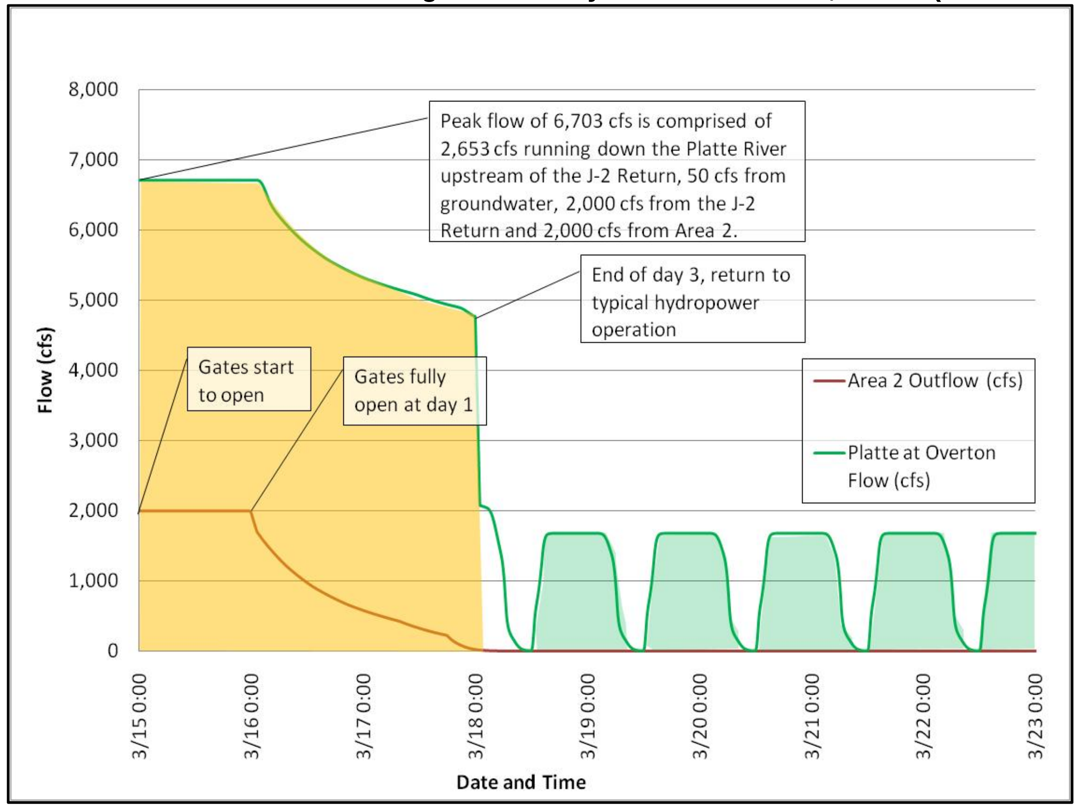


Figure 4.5-12 – J-2 Return Short Duration High Flow Analysis – Alternative 2, Area 4 (60-ft wide radial gate)

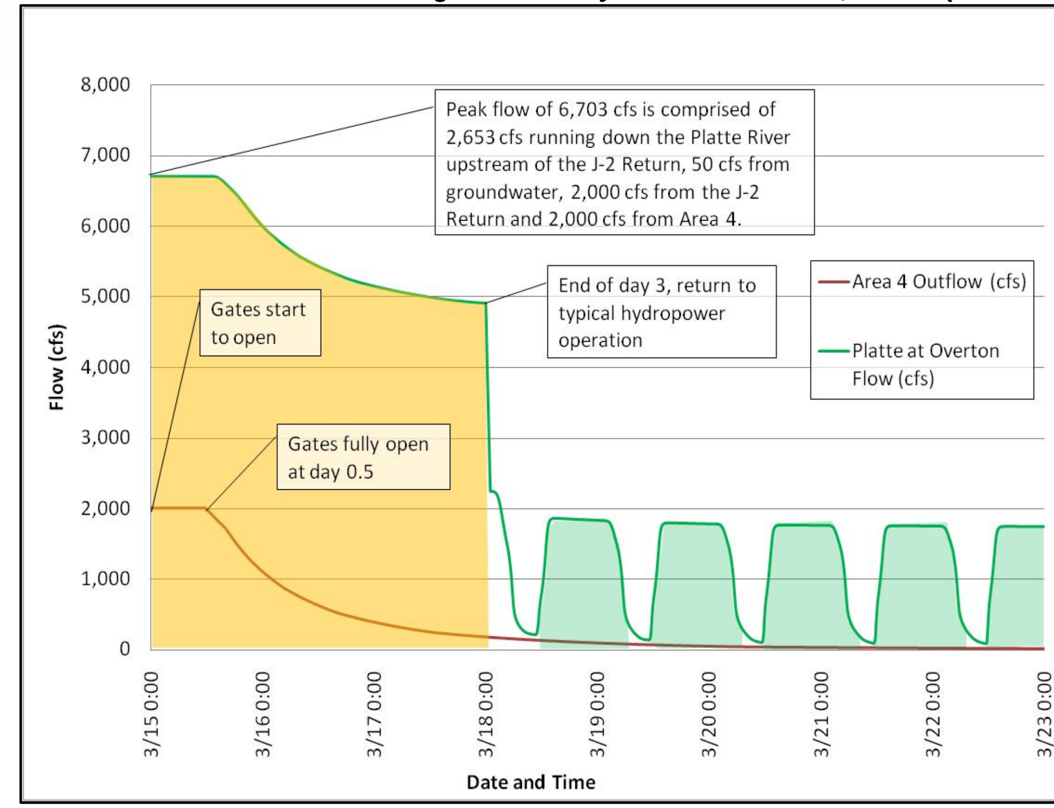


Figure 4.5-11 – J-2 Return Short Duration High Flow Analysis – Alternative 2, Area 3 (30-ft wide radial gate)

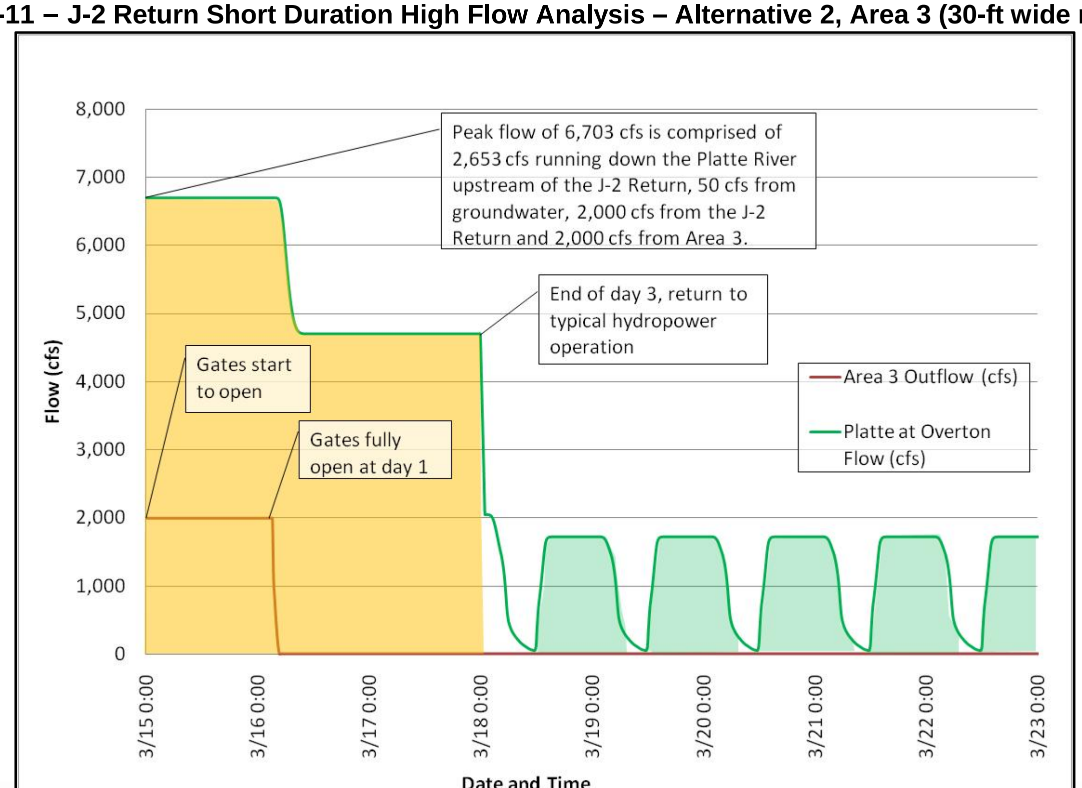


Figure 4.5-13 J-2 Return Short Duration High Flow Analysis – Alternative 3

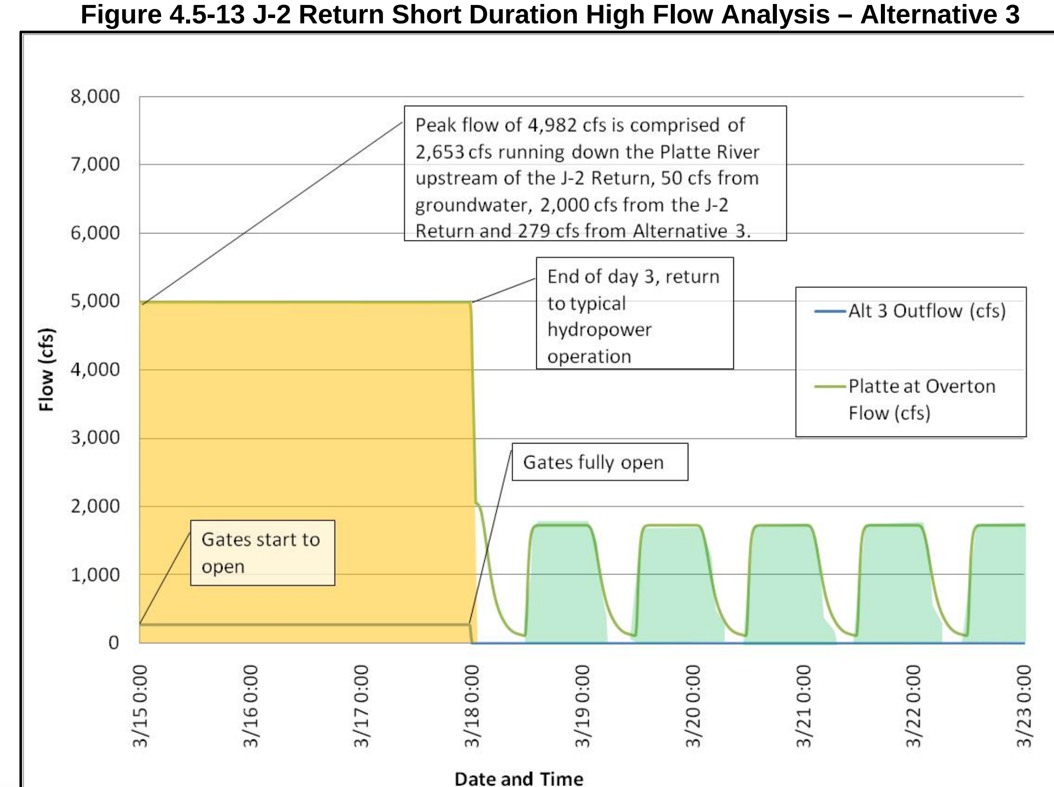


Figure 4.5-14 – J-2 Return Short Duration High Flow Analysis – Alternative 2, Areas 1 & 2

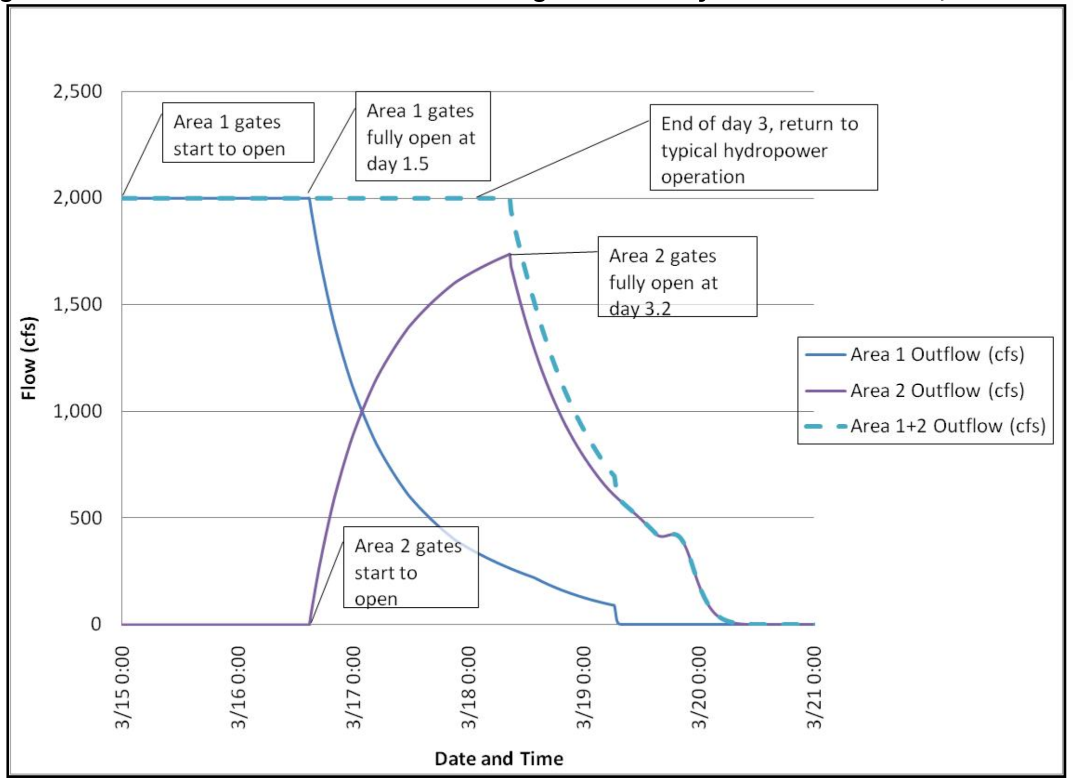
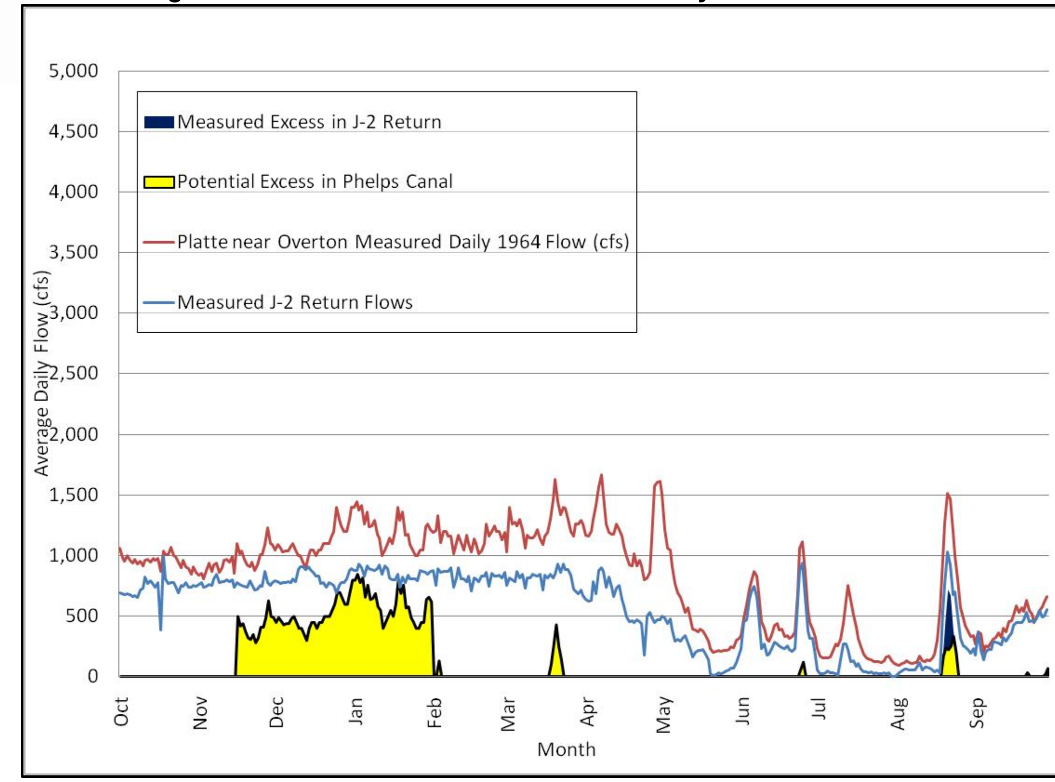


Figure 4.6.1 – J-2 Return 1964 Illustrative Dry Year Excess Flows



4.6 Target Flow Analysis

The storage areas were evaluated for their ability to reduce shortages to target flows in the Platte River. Results are presented in Table 4.6-1 below. These are best case numbers assuming all excess flows could be captured and stored up to the maximum storage potential of each alternative.

The inflow of each J-2 alternative is limited by canal and reservoir inflow capacities. The graphs below depict the total flow at Overton, J-2 Return flows (which return to the Platte River above Overton), excess flows in the J-2 Return, and how much excess flow could potentially be delivered to a reregulating reservoir via the Phelps Canal. "Potential Excess in Phelps Canal" are excess flows in CNPPID's system which were not historically present in Phelps Canal but which could have been routed down the canal rather than back to the Platte River. These excess flows were constrained based upon the canal capacity and historical diversions. Residual capacity was taken into account for J-2 Alt 2, Areas 1 and 2 as well as Alt 3 by assuming that all irrigation water during irrigation season was not able to be used to fill the reservoirs. Per the recommendations of ED Office staff and WAC members during the July 30th, 2009 meeting, icing was not considered to be a concern for filling these reservoirs.

Figure 4.6.2 – J-2 Return 1975 Illustrative Normal Year Excess Flows

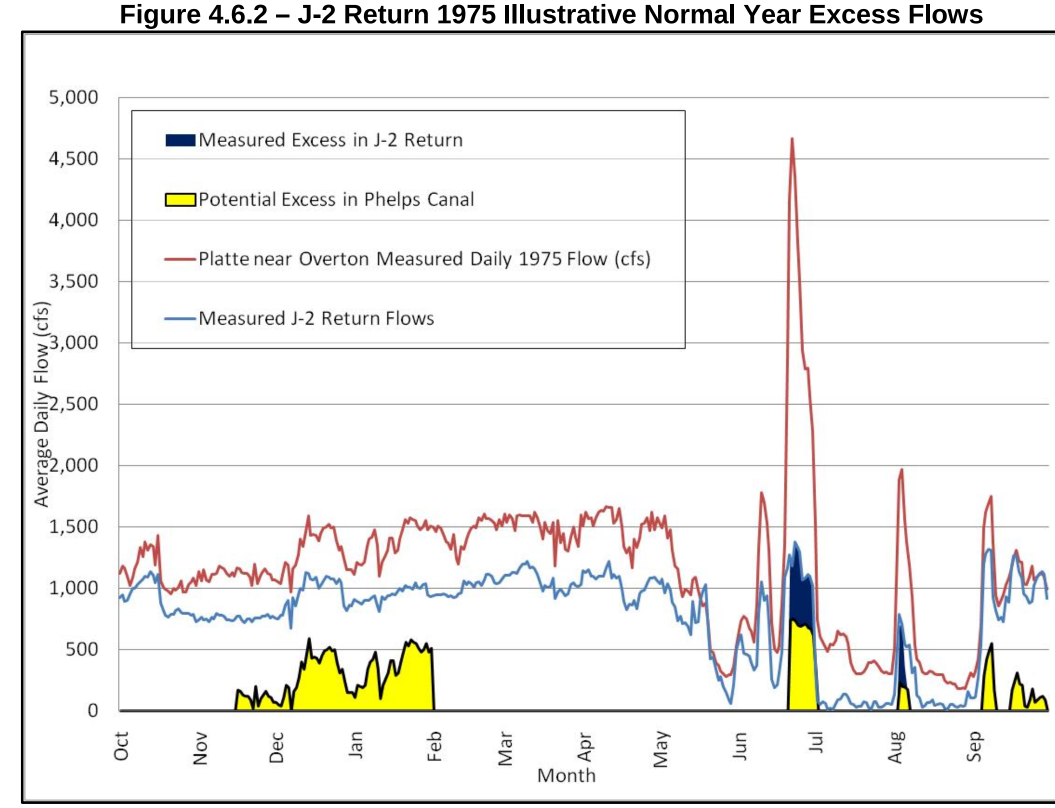


Figure 4.6.3 – J-2 Return 1986 Illustrative Wet Year Excess Flows

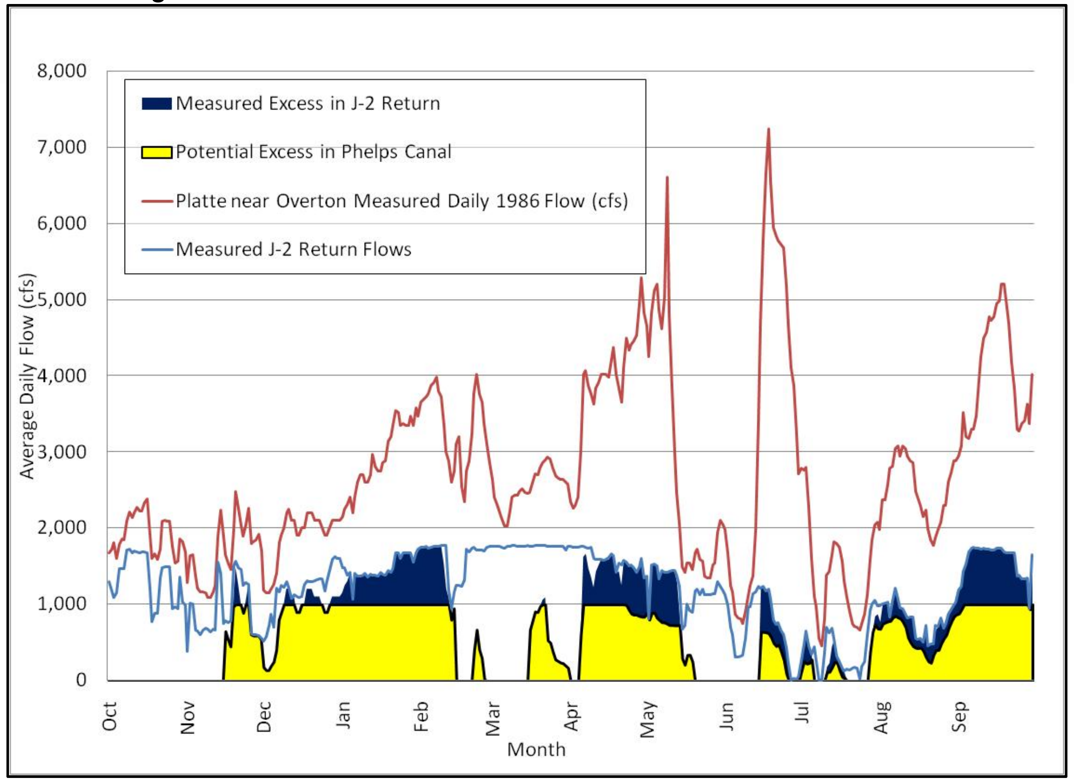
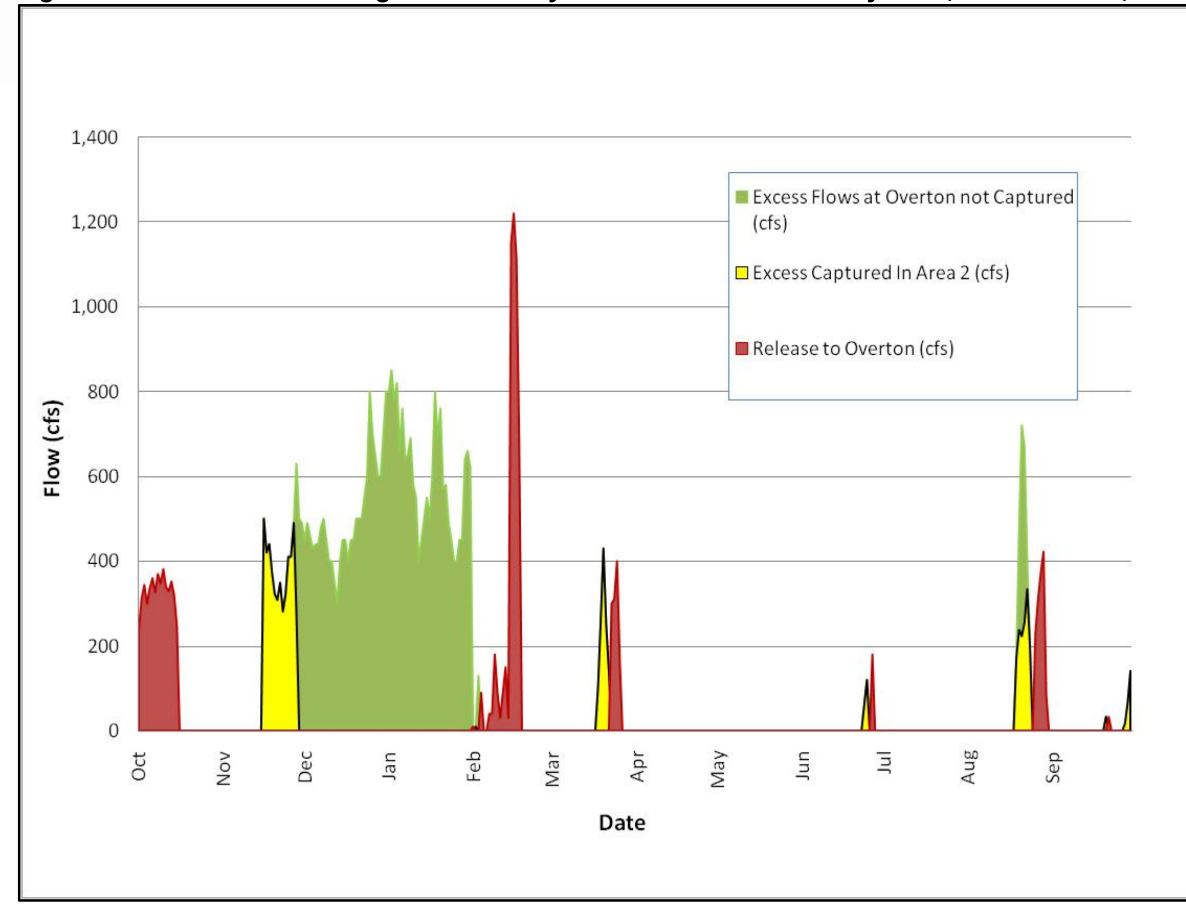


Figure 4.6-4 – J-2 Return Target Flow Analysis – 1964 Illustrative Dry Year, Alternative 2, Area 2



Each J-2 alternative has varying rates of seepage which have not been factored into this analysis. The worst case condition from a seepage loss standpoint would be to store water from the winter excesses and release it in late summer. This, however, does not appear to the typical operation scenario. Based upon the year analyzed, and illustrated here with the normal year, it appears the winter stored water will be consumed quickly in the spring. Figure 4.6-4 shows excess flows being stored in December and held over until a period of shortage in February. In real operations, given that excess flows occur throughout much of the winter, excess flows would likely be stored later in the winter season to decrease holding time in the reservoir. The summer pattern is that reservoirs fill during a period of excess flows and then release to reduce shortages to target flows a few days following the filling event. Under this short residence time condition, a modest seepage loss, like the range proposed for Alternatives 2 and 3, would be appropriate. A graphical sample of the analysis is shown in Figure 4.6-4 for one of the alternative reservoirs. Note that all J-2 alternatives were assumed to start the water year at capacity. For all alternatives this water was immediately released to reduce October shortages to target flows.

Table 4.6-1 – Potential Annual Reductions to Shortages to Target Flows

		Dry Year 1964	Normal Year 1975	Wet Year 1986
Target Flow Shortages, ac-ft		266,715	540,662	227,920
J-2 Alternative	Storage Capacity, ac-ft	Reductions to Shortages to Target Flows, ac-ft		
Alt 1	3,380	12,357	14,660	19,715
Alt 2, Area 1	9,716	25,029	33,668	44,119
Alt 2, Area 2	6,580	18,757	24,974	33,677
Alt 2, Area 3	4,516	16,331	20,341	25,952
Alt 2, Area 4	6,137	18,508	24,268	32,139
Alt 2, Areas 1 & 2	14,320	34,237	47,480	57,931
Alt 3	1,659	7,078	8,298	10,569

As discussed in previous sections, some uncertainty is inherent in the calculation of storage area due to uncertain topographic information. The areas that used USGS contour data rather than LIDAR data include J-2 Alt 2 and Alt 3 reservoirs. When considering the effects of topographic uncertainty, the standard is to use ½ the contour interval. The USGS contour data at Alt 2 and Alt 3 reservoirs has a 5-foot contour interval. Therefore, the uncertainty is +/- 2.5'. Each Alt 2 and Alt 3 reservoir storage area was calculated 2.5 feet higher and lower. These areas were then used to recalculate the potential reductions to shortages to target flows. The percent difference from the results presented in Table 4.6-1 above are presented in Table 4.6-2.

In addition, for consistency of scoring all areas were assumed to start full. If the preceding year had a wet fall, this assumption is likely true for the areas with modest storage volumes. If the preceding fall is dry this would be a non-conservative assumption. A continuous simulation model using all years of record should be developed so that a firm yield could be developed for WAP purposes.

Table 4.6-2 – Potential Percent Difference in Reductions in Shortages to Target Flows when a Topographic Uncertainty of +/- 2.5 feet is Included.

J-2 Alternative	Percent Difference in Reductions in Shortages to Target Flows		
	Dry Year 1964	Normal Year 1975	Wet Year 1986
Alt 2 Area 1	9% / -3%	10% / -4%	4% / -3%
Alt 2 Area 2	14% / -1%	13% / -4%	12% / -4%
Alt 2 Area 3	11% / -1%	14% / -1%	14% / -0.02%
Alt 2 Area 4	18% / -21%	20% / -20%	16% / -22%
Alt 2 Areas 1 & 2	12% / -1%	12% / -4%	8% / -4%
Alt 3	22% / -0.5%	19% / -0.4%	22% / -0.5%

4.7 Hydropower Flow Cycling Dampening

A separate analysis was completed for each alternative to evaluate their usefulness in dampening the large swing in hydropower influenced flows in the Platte River. The hydropower flow cycling portion of the inflow hydrograph developed for March SDHF runs was used in the dampening analysis. It is anticipated that late spring, summer and fall would produce less of a hydropower flow cycling volume and hence would not require as much storage. The goal of the model was to produce as close as possible a uniform release rate from the reservoir, while it filled and drained each day. It should be noted that if the reservoirs were operated as modeled for hydropower flow cycling mitigation, this would significantly decrease ability to use reservoirs for target flow operations.

The following charts summarize the usefulness of using Alternative 1 to dampen the swing. Flows progress downstream starting at Dam Site D and moving finally through Dam Site A. All analysis was conducted in HEC-HMS and involved adjusting the gate opening through the day to maximize storage while achieving as nearly of a constant flow as possible. No new gates or changes were made to the alternatives, as computed from the SDHF analysis. As you would expect, as the hydropower flow pulse moved downstream, each storage area dampened the swing until the final out flow was nearly constant.

Figure 4.7-1 – J-2 Return Hydropower Cycling Analysis – Alternative 1 – Dam D (48-ft wide radial gate)

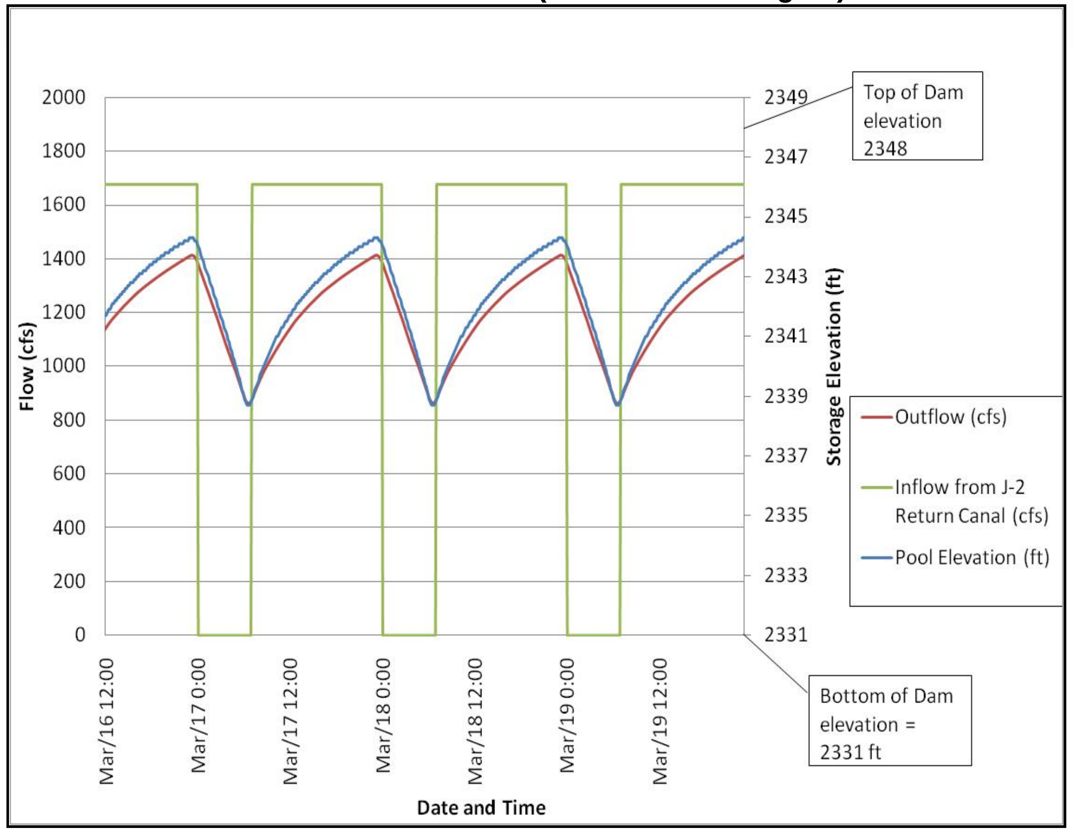


Figure 4.7-2 – J-2 Return Hydropower Cycling Analysis – Alternative 1 – Dam C (48-ft wide radial gate)

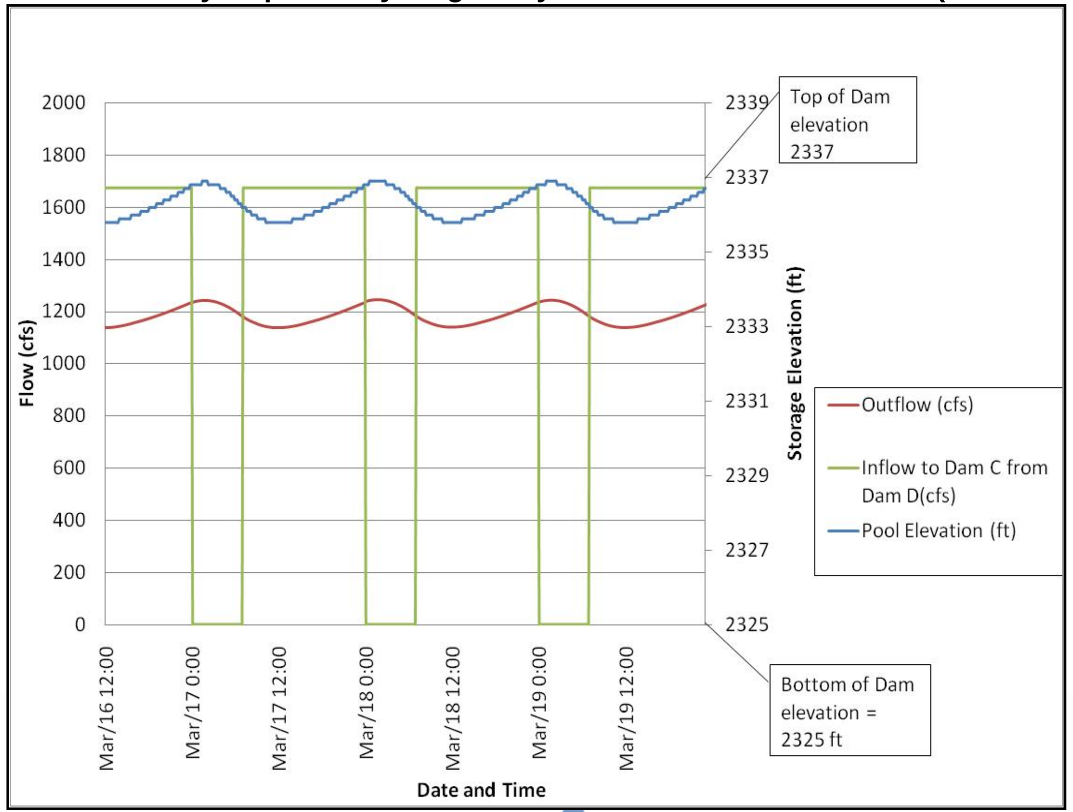


Figure 4.7-3 – J-2 Return Hydropower Cycling Analysis – Alternative 1 – Dam B (48-ft wide radial gate)

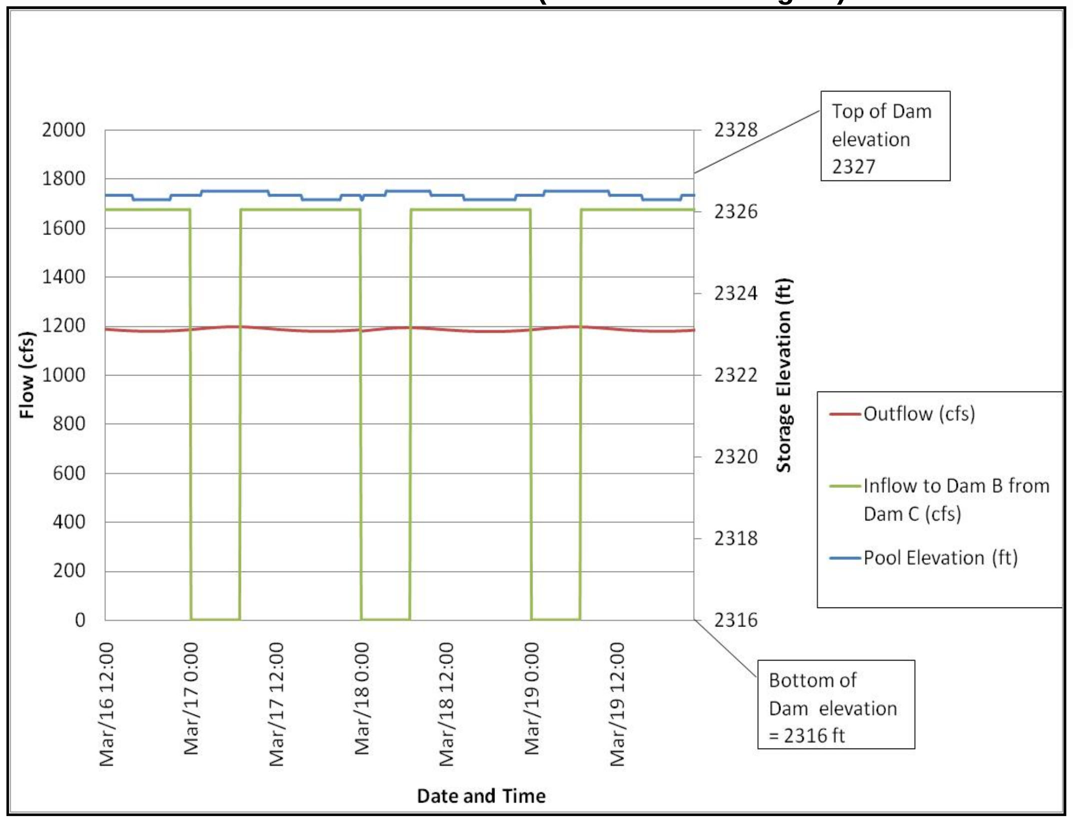
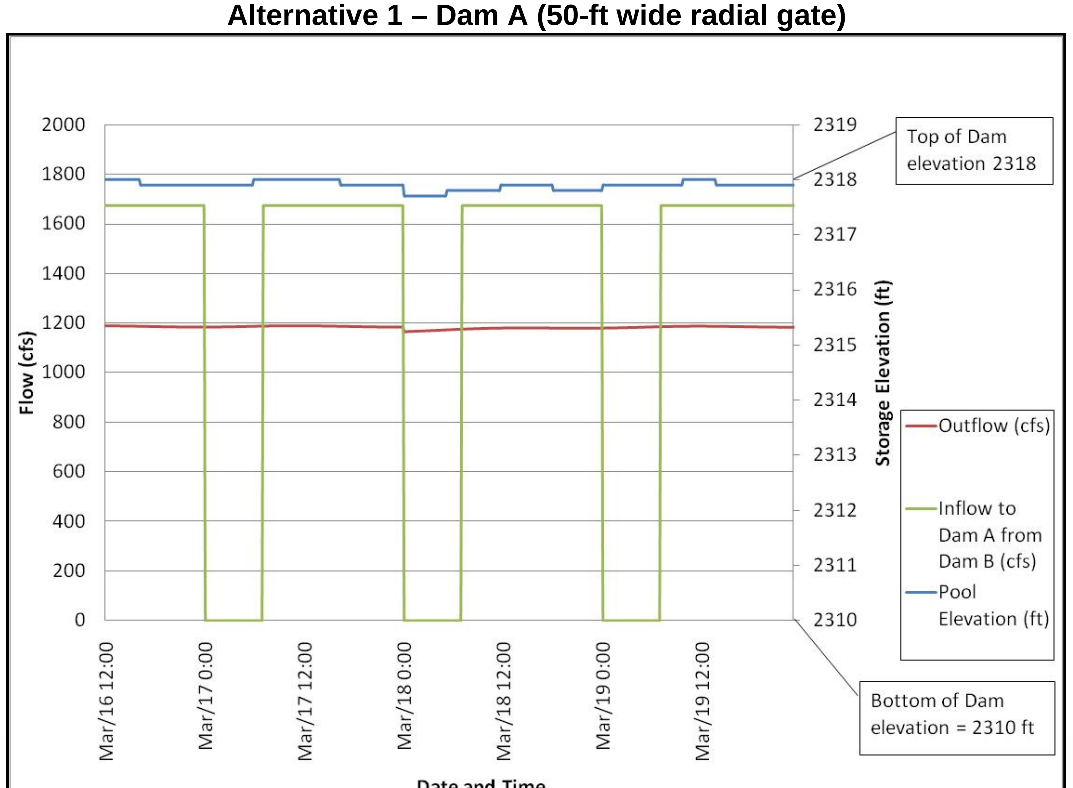


Figure 4.7-4 – J-2 Return Hydropower Cycling Analysis – Alternative 1 – Dam A (50-ft wide radial gate)



The next charts summarize the usefulness of using each of the areas in Alternative 2 to dampen the swing and were calculated in the same manner as Alternate 1. Areas 1 and 2 were restricted due to filling off of Phelps Canal. The peak outflow rate from the J-2 Return is 1,675 cfs but only 1,000 cfs could be delivered to the storage areas via the Phelps Canal. The difference was sent down the J-2 Return and resulted in a daily surge in flows at Overton. Areas 3 and 4 performed better and could nearly deliver a uniform outlet rate. In general, all areas needed to develop a fairly high head to pass the average daily flow value. If there is no limitation on water availability, the average daily outflow value will approach 1,675 for areas 3 and 4. If there is no limitation on water availability, the outflow for areas 1 and 2 would approach 1000 cfs due to the limited supply rate of Phelps Canal. Through the summer months as water becomes scarcer, the average daily value declines but the peak inflow rate does not. This indicates a fairly large inflow gate is needed to accept these high inflow rates. A wide outflow gate is needed to evacuate the water with minimal head. Similar to the SDHF analysis, the ideal use of the system is trending toward very wide gates and storage areas with a sloping bottom that minimizes the volume stored at low elevations. The target flow analysis however benefits from storage at these low heads and elevations.

Figure 4.7-5 – J-2 Return Hydropower Cycling Analysis – Alternative 2, Area 1 (40-ft wide radial gate)

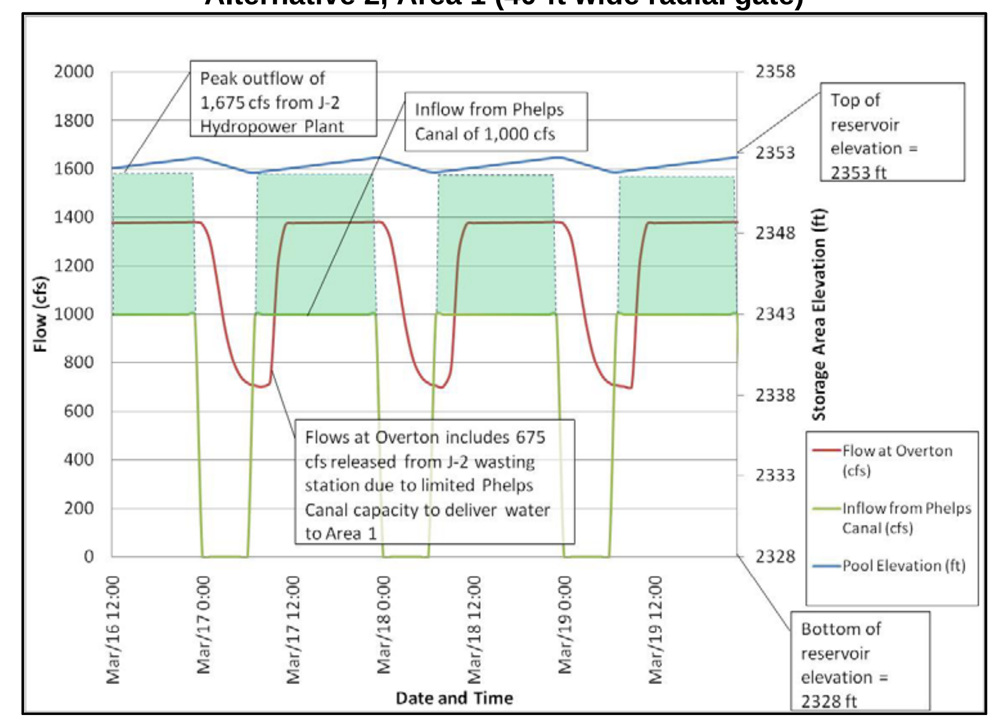


Figure 4.7-6 – J-2 Return Hydropower Cycling Analysis – Alternative 2, Area 2 (30-ft wide radial gate)

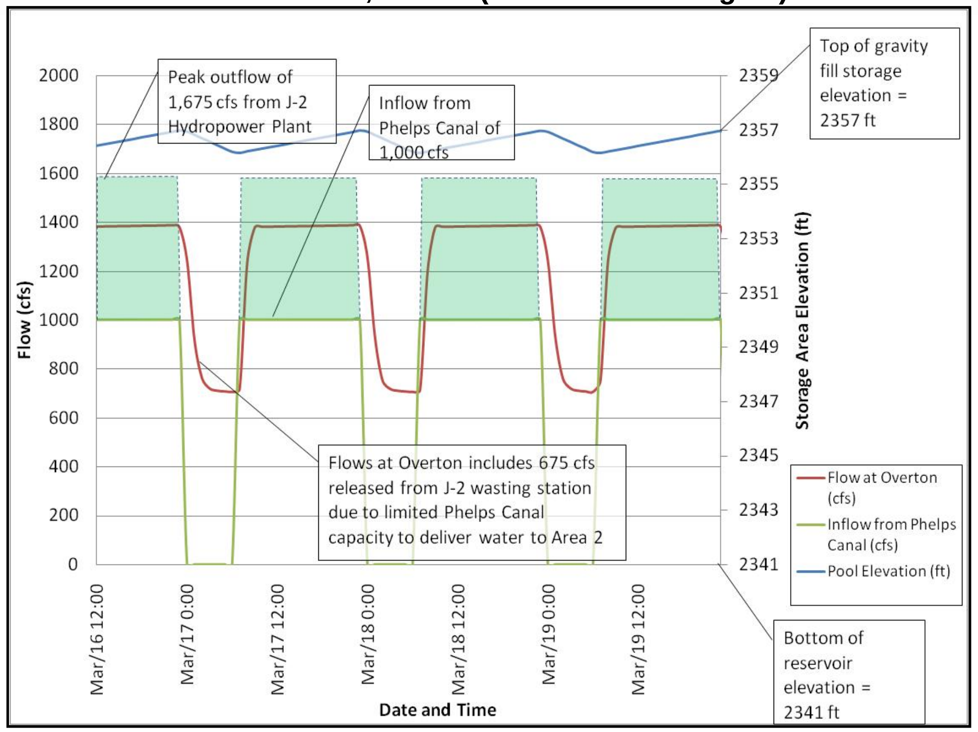


Figure 4.7-7 – J-2 Return Hydropower Cycling Analysis – Alternative 2, Area 3 (30-ft wide radial gate)

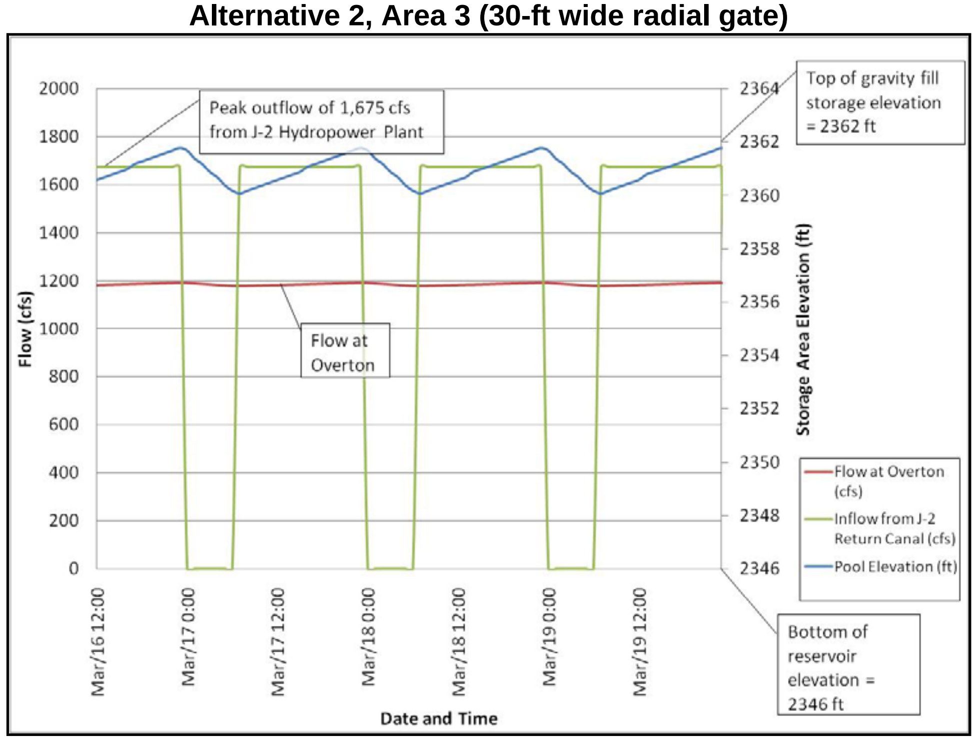
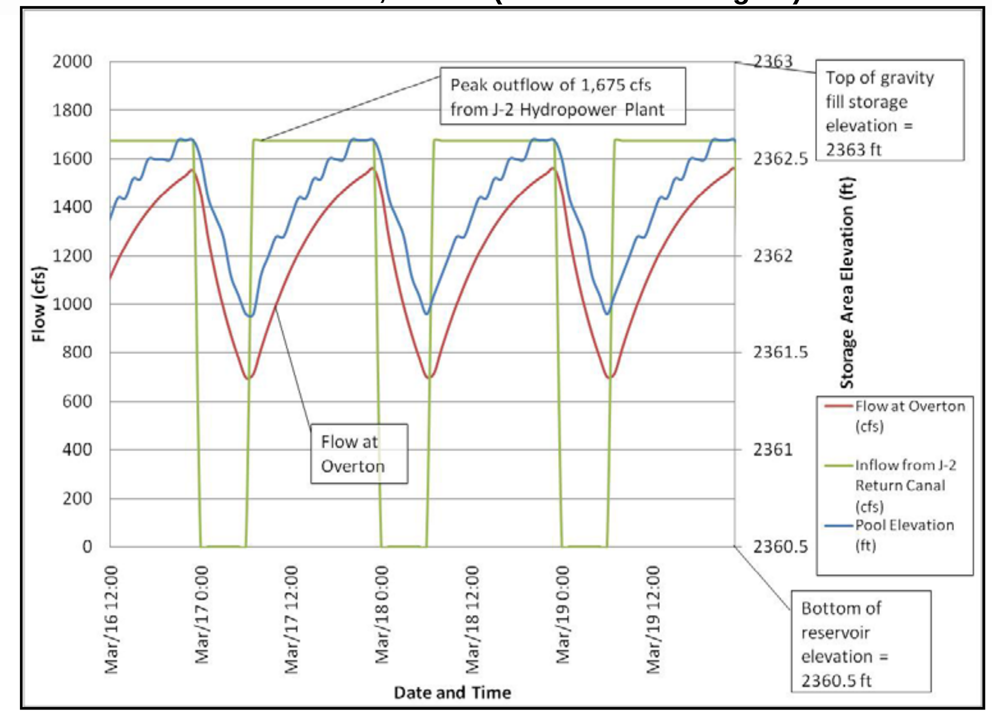
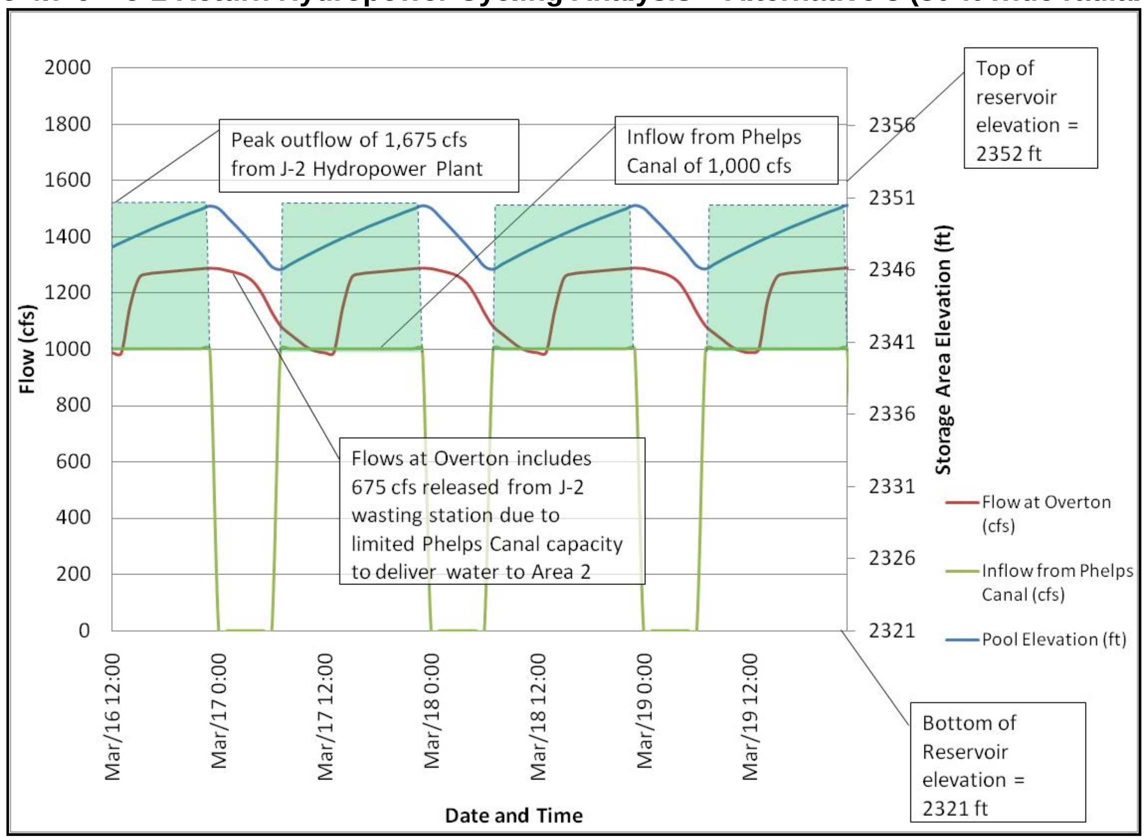


Figure 4.7-8 – J-2 Return Hydropower Cycling Analysis – Alternative 2, Area 4 (60-ft wide radial gate)



The following charts summarize the usefulness of using Alternative 3 to dampen the hydropower cycle outflow swing. Alternative 3 was restricted due to filling off of Phelps Canal. The peak outflow rate from J-2 is 1,675 but only 1,000 cfs could be delivered to the storage areas via the Phelps Canal. The difference was sent down the J-2 Return and resulted in a pulse flow at Overton. This alternative needed to develop a fairly high head to pass the average daily flow value. If there is no limitation on water availability, the average daily value will approach 1,000 cfs. Through the summer months, as water becomes scarcer, the average daily value declines, but the peak inflow rate does not. This indicates a fairly large inflow gate is needed to accept these high inflow rates. A wide outflow gate is needed to evacuate the water with minimal head. Similar to the SDHF analysis, the ideal use of the system is trending toward very wide gates and storage areas with sloping bottom that minimize the volume stored at low elevations. The target flow analysis however benefits from storage at these low heads and elevations.

Figure 4.7-9 – J-2 Return Hydropower Cycling Analysis – Alternative 3 (30-ft wide radial gate)



Summary of Hydropower Cycling

All three alternatives were then evaluated for uniformity in flow at Overton. This was calculated by the percent difference in the hourly minimum to maximum flow compared to the average daily flow. In other words, if the minimum hourly flow equaled the maximum hourly flow then the hydropower cycle was 100% dampened. The results of the hydro cycling evaluations are presented in Table 4.7-1.

Table 4.7-1 – Results of Hydropower Flow Cycling Mitigation Runs

	Average Daily Flow at Overton (cfs)	Minimum Daily Flow (cfs)	Maximum Daily Flow (cfs)	% Dampening (goal is 100%)
Alternative 1, Dam D (48' gate)	1,187	864	1,413	73%
Alternative 1, Dam C (48' gate)	1,187	1,138	1,243	95.9%
Alternative 1, Dam B (48' gate)	1,187	1,178	1,197	99.3%
Alternative 1, Dam A (50' gate)	1,187	1,184	1,189	99.8%
Alternative 2, Area 1 (40' gate)	1,180	703	1,384	59.5%
Alternative 2, Area 2 (30' gate)	1,189	706	1,388	59.4%
Alternative 2, Area 3 (30' gate)	1,186	1,179	1,192	99.5%
Alternative 2, Area 4 (60' gate)	1,187	701	1,558	59%
Alternative 3 (30' wide gate)	1,188	990	1,290	83%

4.8 Capital Costs

Preliminary estimates of probable cost were developed for the alternatives and are itemized in Appendix C. The following costs presented for all alternatives are based on a 2009 cost index. Whenever possible, line-item costs were derived from bid tabs of previous projects in the region. If no comparison was available for a line item, the RS Means, “Heavy Construction Cost Data, 23rd Annual Addition, 2009” was used. These costs are only order of magnitude projections. Project costs include construction costs, permitting and design, and land acquisition costs. Costs represent the major cost items associated with each project as this is a preliminary estimate. A more detailed estimate of probable cost would be available in later phases for the selected alternatives.

Pump stations in the range required for this project are very site-specific and involve a large number of components. A detailed design and construction estimate was not prepared for this level of study but rather the total costs were estimated from recently constructed projects. The Harvey Street pump station in New Orleans was designed by USACE and installed in 2007 by D & D Machine and Hydraulics, Inc, of Fort Myers, Florida.

Photo 4.8-1 – Harvey Street Pump Station in New Orleans

Staff with D&D indicated the station was sized for 660 cfs and a total dynamic head of 21.5 feet. Seven 42-inch diameter 500 horsepower diesel driven axial flow pumps were utilized. The total bid price in 2007 was $6.8 million however this included a FEMA storm shelter, emergency power, boats and other accessories. Based on discussions with D&D, they indicated the pump station and all critical structural and control items consisted of $4.5 million of the $6.8 million bid. The $4.5 million cost for a 660 cfs pump station was used to prorate the costs associated with a 300 cfs to 400 cfs total capacity. The total dynamic head for J-2 alternatives will likely be in the same range as the Harvey Street system. If the options utilizing a pump station are selected for further analysis a more refined method should be used to develop the anticipated cost of construction.

One of the questions raised at the beginning of the study dealt with the cost efficiency of excavating and disposing material off-site versus ponding to a higher elevation by the use of pumps. In general, the cost for excavating and disposing soil off-site results in a cost of $8,000 per acre-foot. The associated costs to install a 300 cfs pump station and the electricity to operate it will cost approximately $1,000 per ac-ft. Both assume the same footprint would be needed and the pump costs did not include the cost of additional embankment and seepage controls. The preliminary costs indicate that if the site cannot be balanced to minimize haul, then pumps might be a viable consideration. The pumps, however, will have a much higher life cycle cost.

The costs for the radial gates and sluice gates are based on the manufactured delivered price for all key components and controls plus a 155% installation cost. The volume of structural concrete was estimated based on the anticipated wall and floor dimensions.

Contractor mobilization and demobilization are equal to 2.5% of the construction line items, not including contingency. A 20% contingency factor also was added. In addition, a 20% topographic uncertainty contingency factor was added for Alternative 2 and Alternative 3. This was added because LIDAR data was not currently available in these areas and USGS topography was used. Permitting and design costs are approximated as 8% of the construction subtotal.

Table 4.8-1 – J-2 Alternatives Cost Summary Table

Alternative	Total Project Costs ($000)	Life Cycle Costs per ac-ft
Alt 1	$28,373	$34
Alt 2, Area 1	$33,283	$16
Alt 2, Area 2	$25,089	$16
Alt 2, Area 3	$57,091	$46
Alt 2, Area 4	$117,917	$80
Alt 2, Areas 1 & 2	$40,039	$19
Alt 3	$11,361	$23

4.9 Land Ownership

The ownership of land for proposed locations for Alternatives 1, 2, and 3 for the J-2 reservoirs was investigated, and shown in the following figures:

Figure 4.9-1 – Alternative 1, Land Ownership

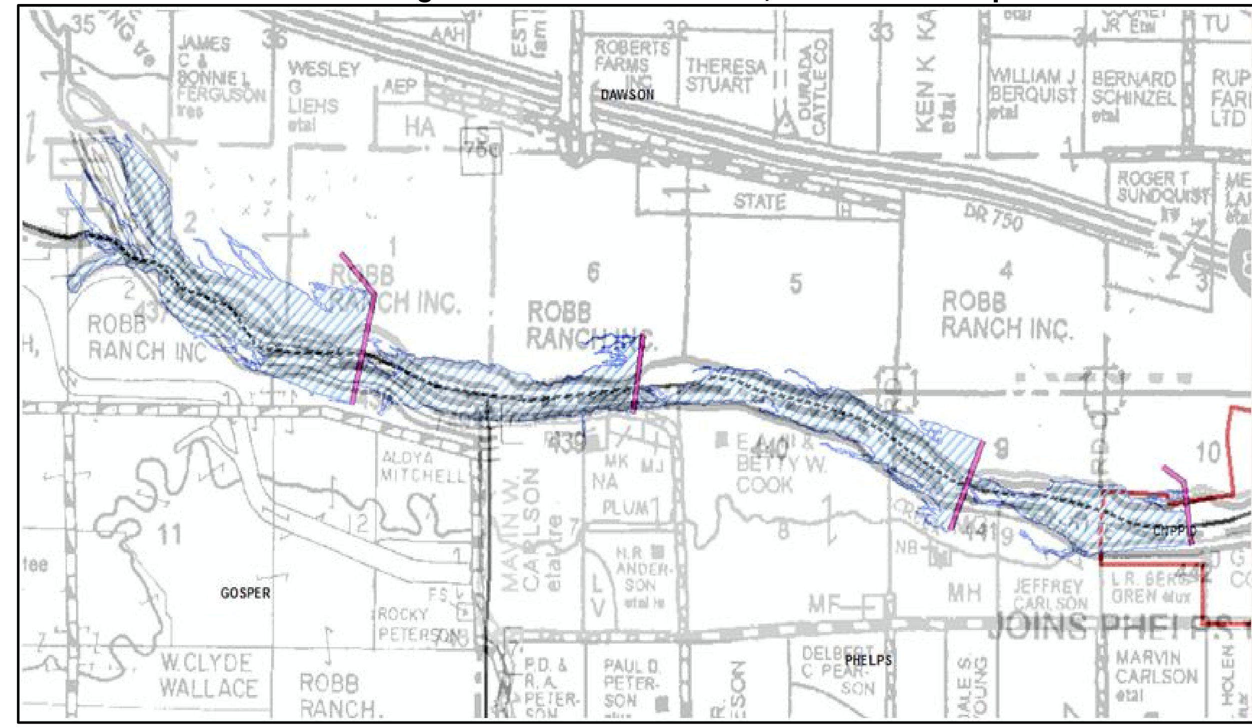


Figure 4.9-2 – Alternative 2, Area 1 Land Ownership

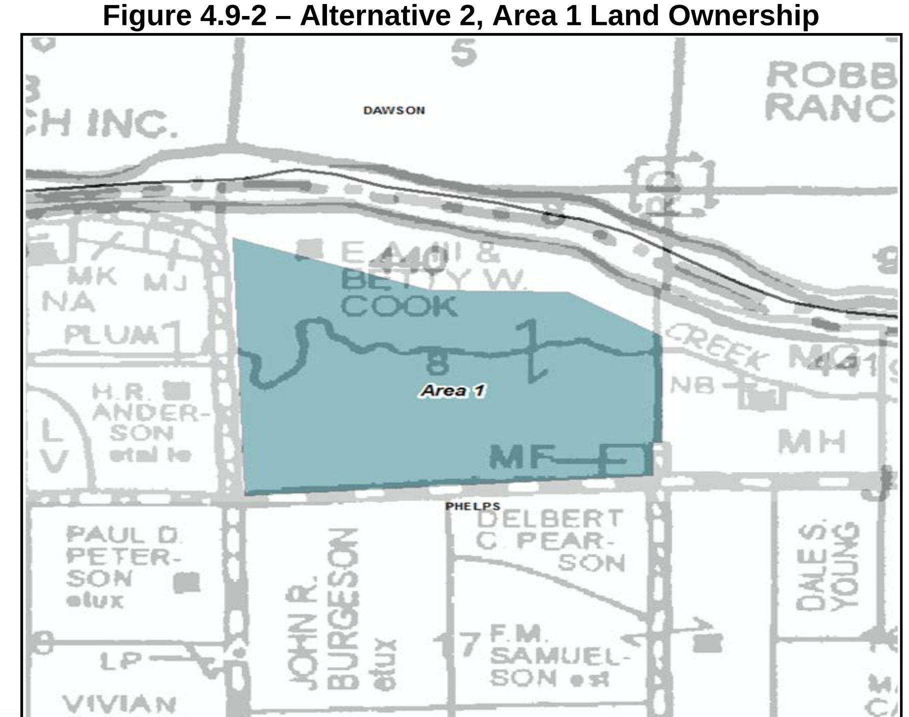


Figure 4.9-3 – Alternative 2, Area 2 Land Ownership

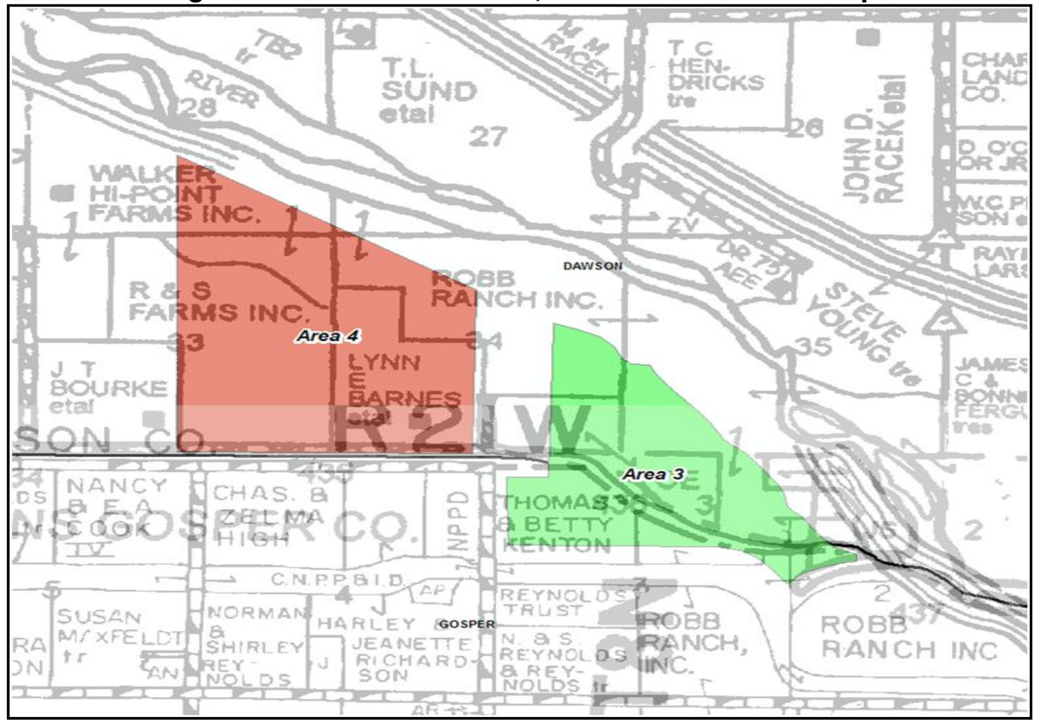
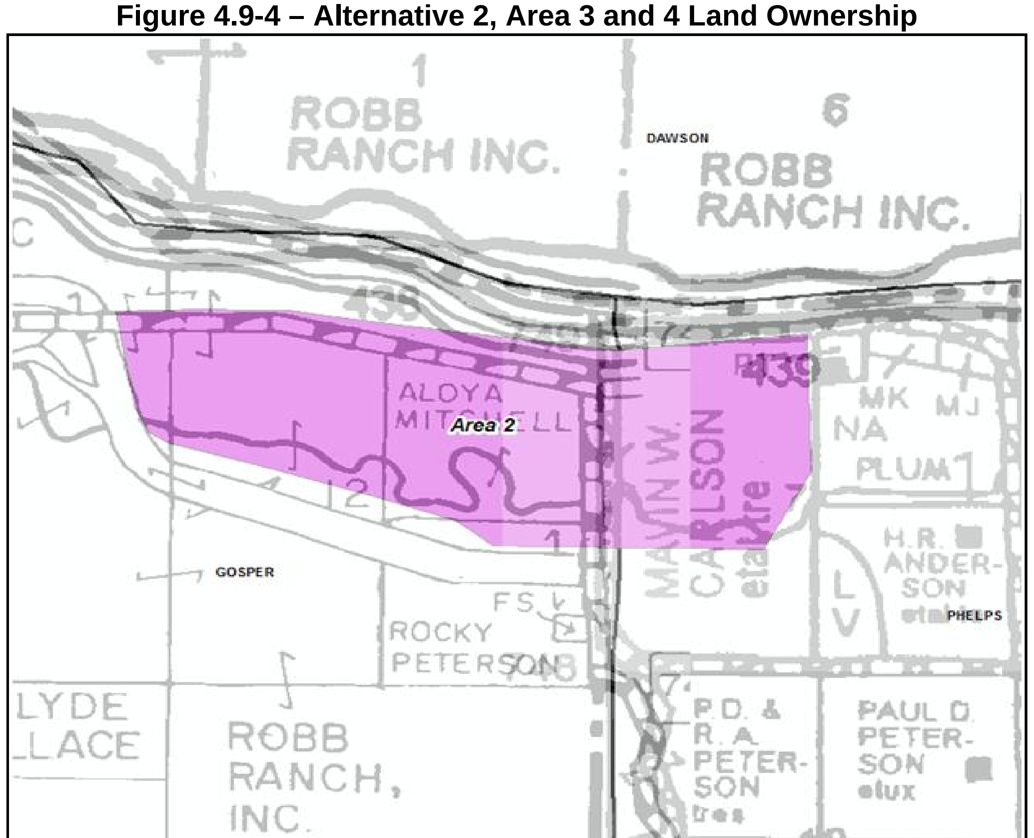


Figure 4.9-4 – Alternative 2, Area 3 and 4 Land Ownership



5. J-2/ELWOOD COMBINATION ALTERNATIVES

Two alternatives combining the operation of Elwood and J-2 were investigated and scored. The two alternatives are:

Alternative E/J-2, Area 1 – The use of J-2 Alternative 2, Area 1 modified, and the use of Elwood, with a buttressed dam, gravity inlet canal, one tunnel outlet, and Plum Creek to convey water to the Platte River.

Alternative E/J-2, Area 2 – The use of J-2 Alternative 2, Area 2, together with the use of Elwood, with a buttressed dam, gravity inlet canal, one tunnel outlet, and Plum Creek to convey water to the Platte River.

The independent J-2 Alternatives did not adequately satisfy a 3-day, 2,000 cfs release for SDHF augmentation unless multiple areas were built, such as Areas 1 and 2. The two most attractive sites based on the cost per ac-ft are Areas 1 and 2 within the J-2 Alternative 2. Consideration was also given for a J-2 Alternative 2 and Elwood combination alternative that would yield the desired SDHF augmentation at a lower cost than constructing both J-2 Areas 1 and 2. The Elwood release of 1,200 cfs to Plum Creek also appeared to be a viable alternative, therefore, these alternatives were selected for a joint operational analysis.

The J-2/Elwood combinations alternatives were evaluated for target flow operations as well as for their ability to mitigate hydropower flow cycling impacts. The results of these evaluations are presented in Table 5-1 below.

Table 5-1 – J-2/Elwood Combination Alternatives Yield Summary Table

Alternative	Reservoir	SDHF Augmentation[1] ac-ft / yr	Reductions to Shortages to Target Flows[2],[3]		
			Wet Yr ac-ft	Normal Yr ac-ft	Dry Yr ac-ft
E/J-2 Alt 2, Area 1	Elwood, buttress & J-2 excavation, Area 1 modified	11,901	44,119	33,668	25,029
E/J-2 Alt 2, Area 2	Elwood, buttress & J-2 excavation, Area 2 modified	11,901	33,677	24,974	18,757

Notes: [1]Water to augment SDHF could be either environmental account (EA) water routed down from Lake McConaughy, and staged in the reservoir, or stored excess flows captured and stored in reservoirs immediately before a SDHF if available. Though units are ac-ft/yr, the values presented are the total volume of SDHF augmentation flows provided by the alternative over three days.
[2]Water to reduce shortages to target flows is excess flows in CNPPID's system that could between stored and released during periods of shortage.
[3] Assumes J-2 storage site(s) are full at beginning of water year (October) for consistency of scoring all alternatives. Results shown are for the illustrative years only. Long-term yield averages will vary.

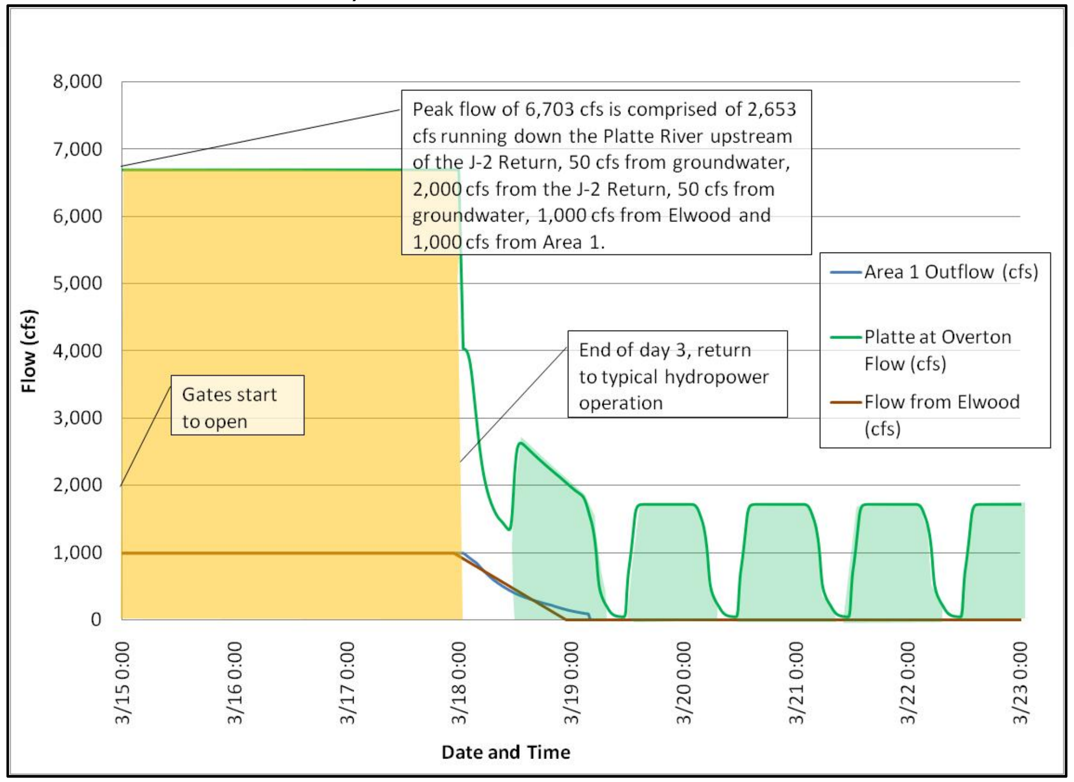
Table 5-2 – J-2/Elwood Combination Alternatives Cost Summary Table

Alternative	Reservoir	Capital Costs ($000)	1-yr Operating Costs ($000)
E/J-2 Alt 2, Area 1	Elwood, buttress & J-2 excavation, Area 1 modified	$51,626	$470
E/J-2 Alt 2, Area 2	Elwood, buttress & J-2 excavation, Area 2	$46,861	$434

5.1 Elwood plus J-2 Alt 2, Area 1 (E/J-2 Alt2, Area 1)

Under this scenario Elwood would deliver 1,000 cfs to the Overton gage by routing 1,200 cfs flows down Plum Creek and into the south channel of the Platte River. J-2 Alternative 2, Area 1 would then be required to deliver 1,000 cfs for the 3 day period. A figure of the outflow from each element and total flows at Overton are presented in Figure 5.1-1. Area 1 has the potential storage of 9,700 ac-ft of water which is greater than the volume needed to supply 1,000 cfs over the 3 days. Likewise the gate is sized to release 2,000 cfs where as only 1,000 cfs would be needed. It is believed the amount of excavation within Area 1 could be reduced if 1,000 cfs were delivered from Elwood. Although a detailed analysis has not been performed, it is estimated the construction costs for Area 1 could be reduced approximately $2 million. The armoring/upgrades to Plum Creek could be reduced by $4 million; the outlet tunnel from Elwood could be reduced by $4.6 million.

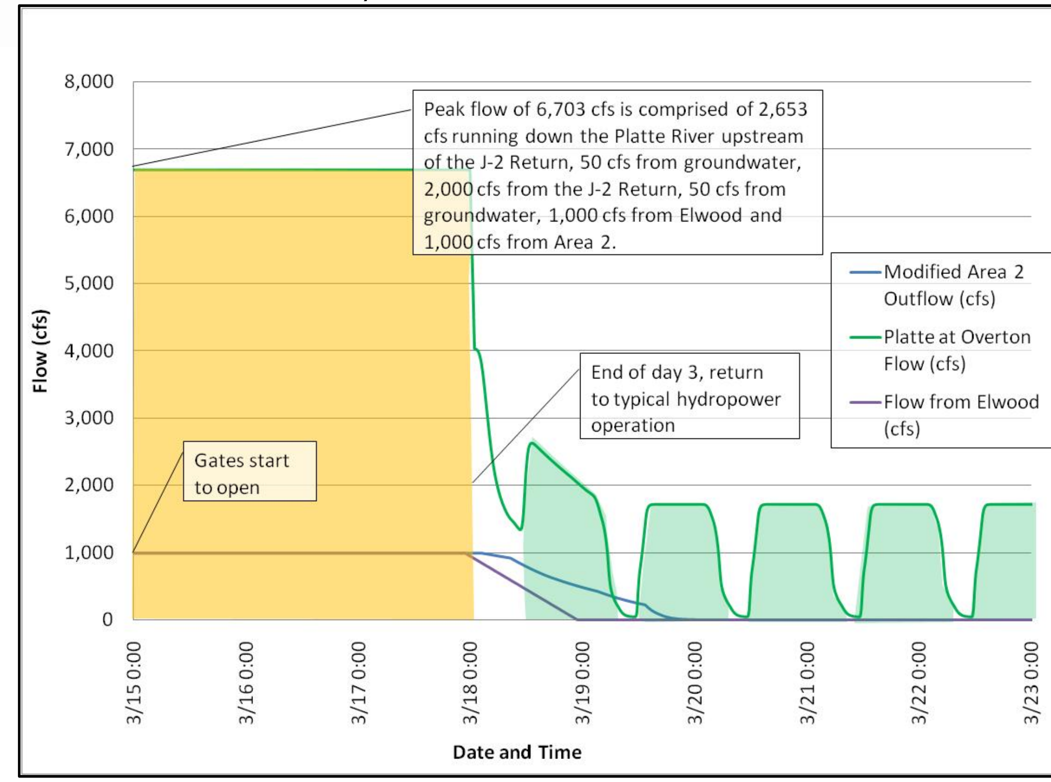
Figure 5.1-1 – J-2 Return Short Duration High Flow Analysis – Combination of 1,000 cfs from Elwood with a modified Area 1



5.2 Elwood plus J-2 Alt 2 Area 2 (E/J-2 Alt 2, Area 2)

Under this scenario Elwood would deliver 1,000 cfs to Area 2 by routing 1,200 cfs flows down Plum Creek. Area 2 would then route the stored water plus the water from Elwood into the south channel of the Platte River for a total release contribution at Overton of 2,000 cfs. Area 2 is currently sized to deliver 2,000 cfs for approximately 1 day and then flows taper off due to lack of storage and available head. Under the combined scenario, the full 2,000 cfs gate capacity would be needed to route the Elwood flows and the stored flows. A labyrinth weir would also still be required for Plum Creek flood flows. There would likely be some attenuation of the flows from Elwood as it moved through Area 2 storage site before the flows reached the outlet gate. This combined option would likely not be able to deliver the full 2,000 cfs under the current configuration. It is likely that with additional storage and wider gates, this option could potential deliver the full 2,000 cfs for the 3 days. The cost increase for additional excavation and wider outlet gates has not been calculated but would likely be on the order of $10 million.

Figure 5.2-1 – J-2 Return Short Duration High Flow Analysis – Combination of 1,000 cfs from Elwood with a modified Area 2



For E/J-2 Alternative 2, Area 2, the capital cost for Alternative 2, Area 2 was not evaluated or changed for the scoring purposes, since Plum Creek would have to pass through the J-2 reservoir with no attenuation. As discussed above, there would be increased capital costs for this combined alternative in the Area 2 gates and increased excavation. This alternative was not seen to be viable, and therefore no additional cost estimating was completed. The armoring/upgrades to Plum Creek were reduced by $4 million; the outlet tunnel from Elwood was reduced by $4.6 million.

6. SCORING AND SCREENING

6.1 Elwood Reservoir

The alternatives that survived screening, and were carried forward into scoring were:

Alternative	Embankment	Inlet	Outlet	Conveyance
E-1	Buttress upstream face of embankment	New Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs
E-2	Remove & replace embankment	New Gravity Canal	Open cut 2-8' pipes	Plum Creek, 2,400 cfs
E-3	Remove & replace upstream shell	New Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs
E-4	Buttress upstream face of embankment	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs
E-5	Remove & replace embankment	Existing E-65 Canal	Open cut 2-8' pipes	Plum Creek, 2,400 cfs
E-6	Remove & replace upstream shell	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs

6.2 J-2 Return Reservoir Alternatives

6.2.1 New Reservoir Location Alternatives

Alternatives for the J-2 Return Reservoir that made it past screening and were scored were:

Alternative	Reservoir	Inlet	Outlet	Conveyance
J -2 Alt 1	J-2 south channel option	J-2 Return	Radial Gates	n/a
J -2 Alt 2, Area 1	J-2 Excavation Area 1	Phelps	Radial Gates	n/a
J -2 Alt 2, Area 2	J-2 Excavation Area 2	Phelps	Radial Gates	n/a
J -2 Alt 2, Area 3	J-2 Excavation Area 3	J-2 Return	Radial Gates	n/a
J -2 Alt 2, Area 4	J-2 excavation Area 4	J-2 Return	Radial Gates	n/a
J -2 Alt 2, Area 1 & 2	J-2 excavation Areas 1&2	Phelps	Radial Gates	n/a
J -2, Alt 3	Phelps 9.7 reservoir	Phelps	Sluice Gates	Unnamed Creek

6.3 Combined Configuration Alternatives

The combined alternatives, which were scored as viable alternatives involving both the operation of the Elwood Reservoir and the J-2 Return Reservoir, were:

Alternative	Reservoir	Inlet	Outlet	Conveyance
E/J-2 Alt 2, Area 1	Elwood buttress, J-2 excavation, Area 1 modified	Existing E-65 Canal and J-2 Return	Tunnel (1 only)	Plum Creek, 1,200 cfs
E/J-2 Alt 2, Area 2	Elwood buttress, J-2 excavation, Area 2	Existing E-65 Canal and J-2 Return	Tunnel (1 only)	Plum Creek, 1,200 cfs

6.4 Summary of Alternatives Analysis

Table 6.2-1 presents a summary of the alternatives analyzed, along with operating costs, SDHF augmentation, and reductions to shortages to target flows. Detailed scoring information for each alternative is provided is Appendix C.

Table 6-1 Reregulating Reservoirs Alternative Analysis Summary

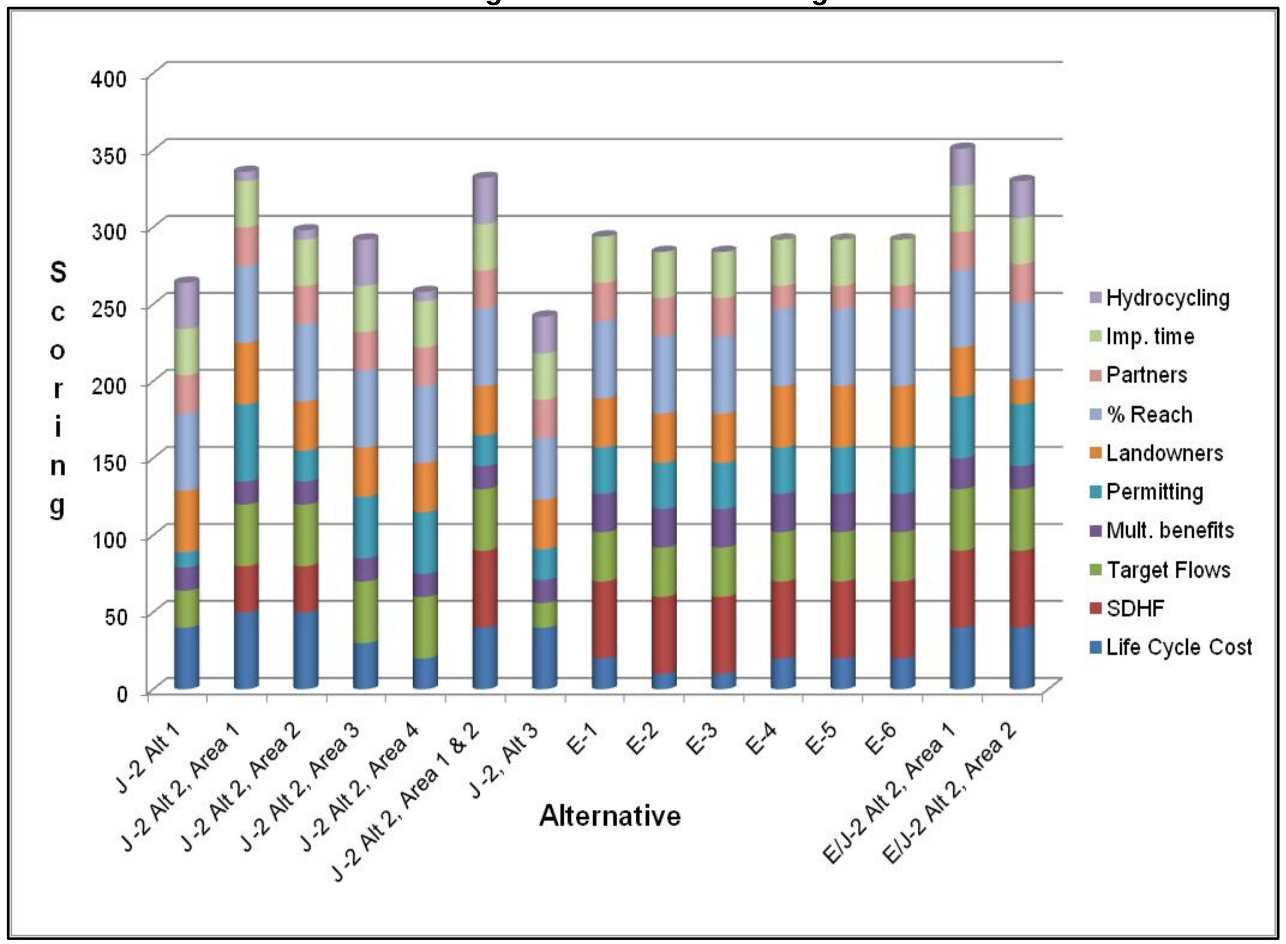
Alternative	Capital Costs ($000)	1-yr Operating Costs ($000)	SDHF Augmentation[1] ac-ft / yr	Reductions to Shortages to Target Flows[2],[4]		
				Wet Yr ac-ft	Normal Yr ac-ft	Dry Yr ac-ft
J -2 Alt 1	$17,460	$218	1,825	19,715	14,660	12,357
J -2 Alt 2, Area 1	$24,206	$182	8,860	44,119	33,668	25,029
J -2 Alt 2, Area 2	$17,483	$152	6,580	33,677	24,974	18,757
J -2 Alt 2, Area 3	$40,541	$331	4,516	25,952	20,341	16,331
J -2 Alt 2, Area 4	$83,877	$681	5,387	32,139	24,268	18,508
J -2 Alt 2, Area 1 & 2[3]	$40,039	$321	11,901	57,931	47,480	34,237
J -2, Alt 3	$6,059	$106	1,659	10,569	8,298	7,078
E-1	$42,942	$690	11,901	21,736	19,408	19,154
E-2	$45,444	$721	11,901	21,736	19,408	19,154
E-3	$45,522	$722	11,901	21,736	19,408	19,154
E-4	$36,677	$449	11,901	21,330	17,788	19,162
E-5	$39,179	$468	11,901	21,330	17,788	19,162
E-6	$39,257	$459	11,901	21,330	17,788	19,162
E/J-2 Alt 2, Area 1	$51,626	$457	11,901	44,119	33,668	25,029
E/J-2 Alt 2, Area 2	$46,861	$422	11,901	33,677	24,974	18,757

Notes: [1]Water to augment SDHF can be either environmental account (EA) water routed down Lake McConaughy and staged in the reservoir or excess flows captured and stored in reservoirs immediately before a SDHF if available. Though units are ac-ft/yr, the values presented are the total volume of SDHF augmentation flows provided by the alternative over three days.
[2]Water to reduce shortages to target flows is excess flows in CNPPID's system that were stored during times of excess and released during periods of shortage.
[3] Assumes only gravity fill for Areas 1 and 2
[4] Assumes J-2 storage sites are full at beginning of water year (October) for consistency of scoring all alternatives. Results shown are for the illustrative years only. Long-term yield averages will vary.

6.5 Scoring Results

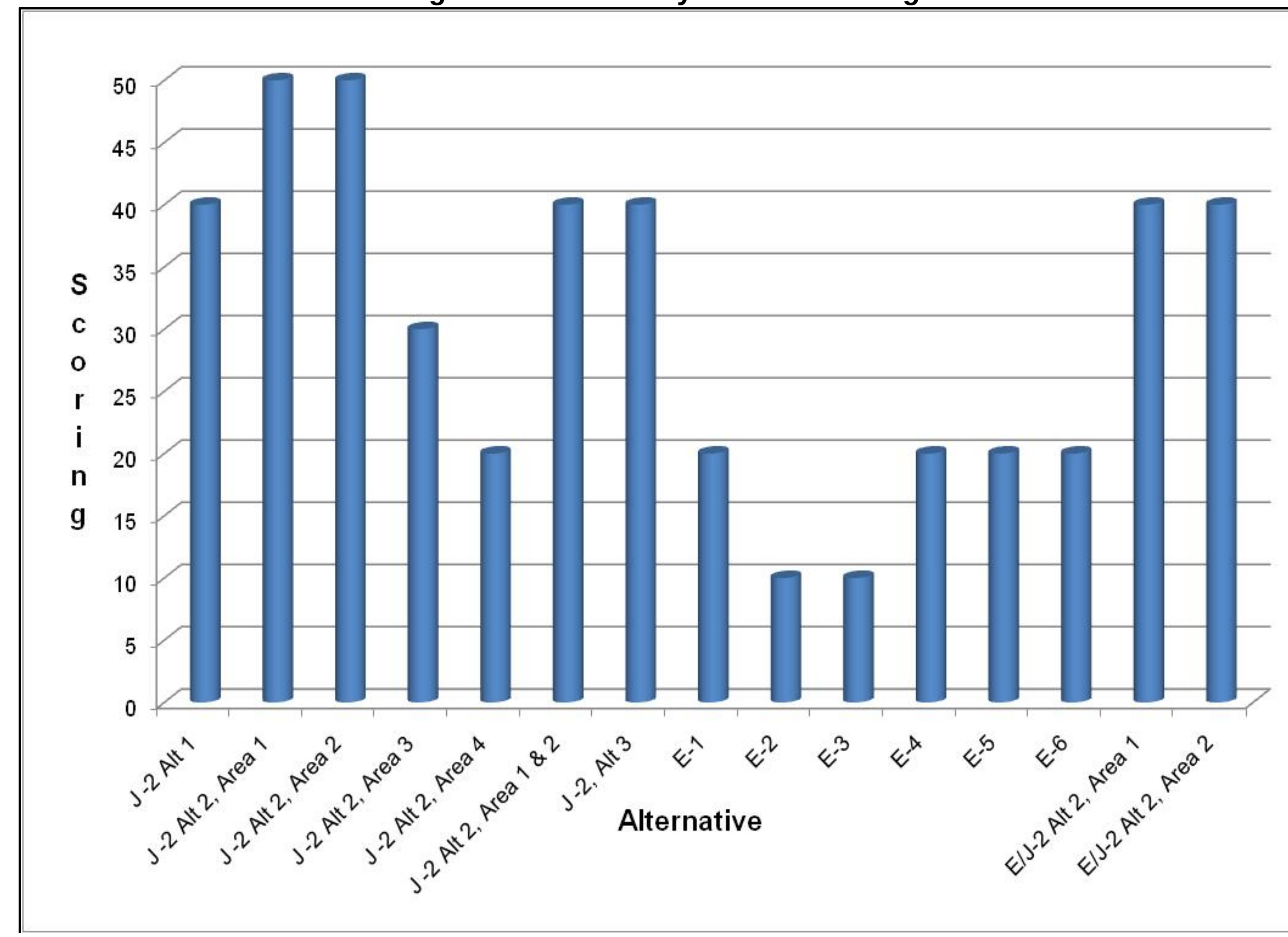
The result of the screening and scoring are displayed in the following graphs for total scoring and each scoring category. These results are also included in detail in Appendix D. General conclusions from the scoring are noted with the relevant category graph.

Figure 6.5-1- Total Scoring



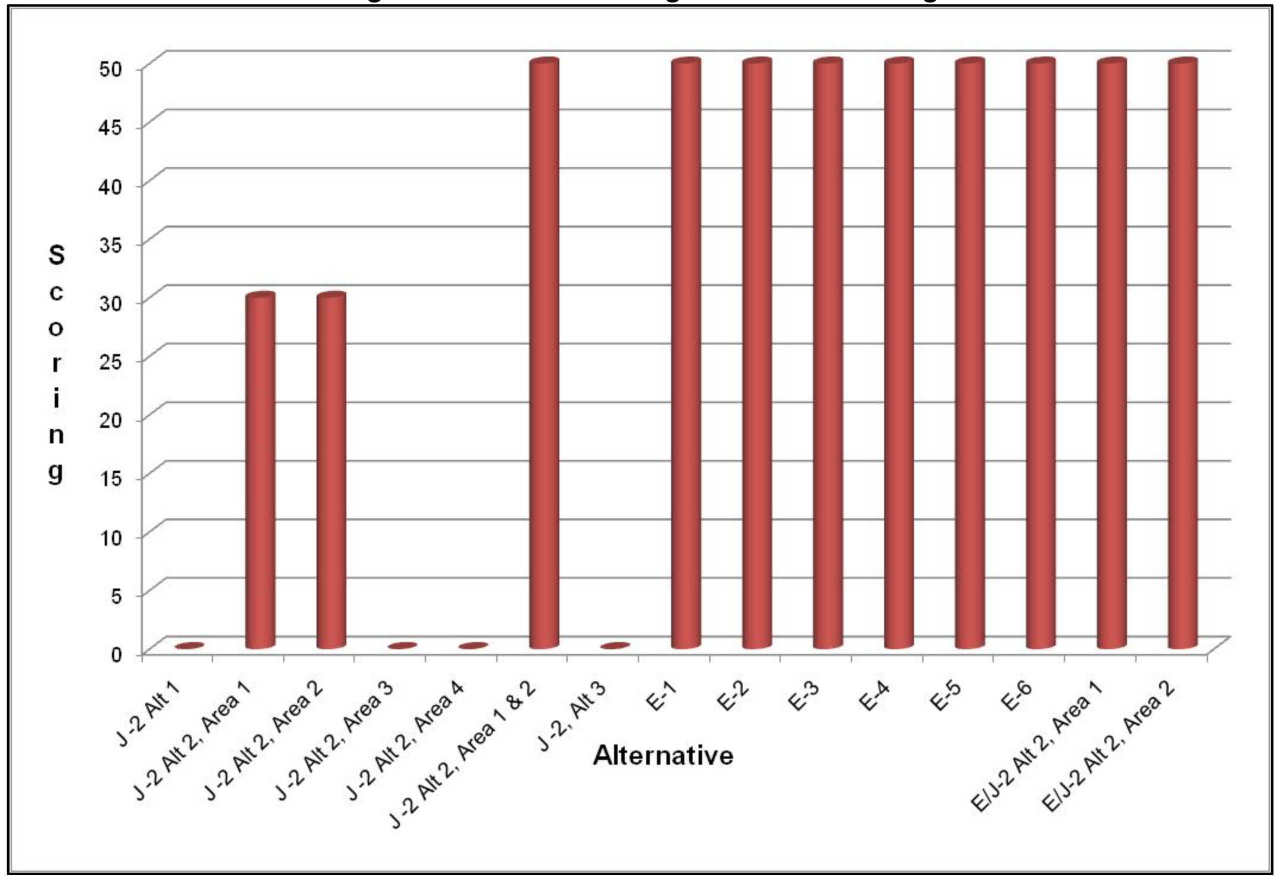
It became clear during the analysis and investigation that a J-2 Alternative 2 location is the preferred location for a reservoir to augment the SDHF and to reduce shortages to target flows, with the combination of areas 1 and 2 scoring the highest for the alternative. As the scoring has also pointed out, the option of using Elwood to support the SDHF, in conjunction with a J-2 Alternative 2 reservoir, is advantageous and should be included going forward.

Figure 6.5-2 – Life Cycle Cost Scoring



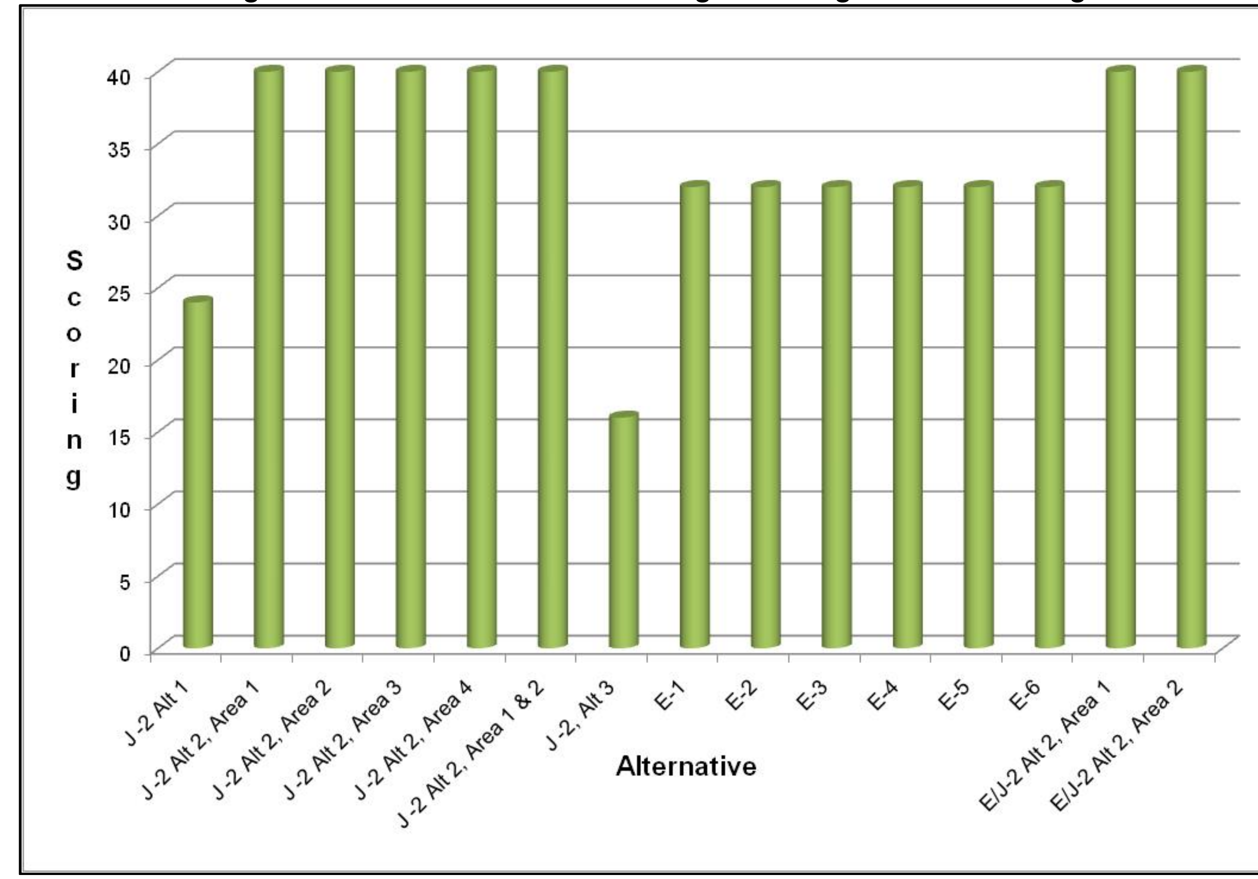
Life cycle costs were calculated as total capital and operating costs over 50 years divided by the delivered acre-feet of water over that same 50 years including both SDHF and normal year target flow releases. The costs varied from a low of $16 per acre-foot to a high of $84 per acre-foot. The Elwood options have higher life cycle costs per acre-foot in general and include power bypass costs associated with bypassing the J-1 and J-2 Hydropower stations. The alternatives were scored from 0 to 5, with a criterion weight of 10.

Figure 6.5-3 – SDHF Augmentation Scoring



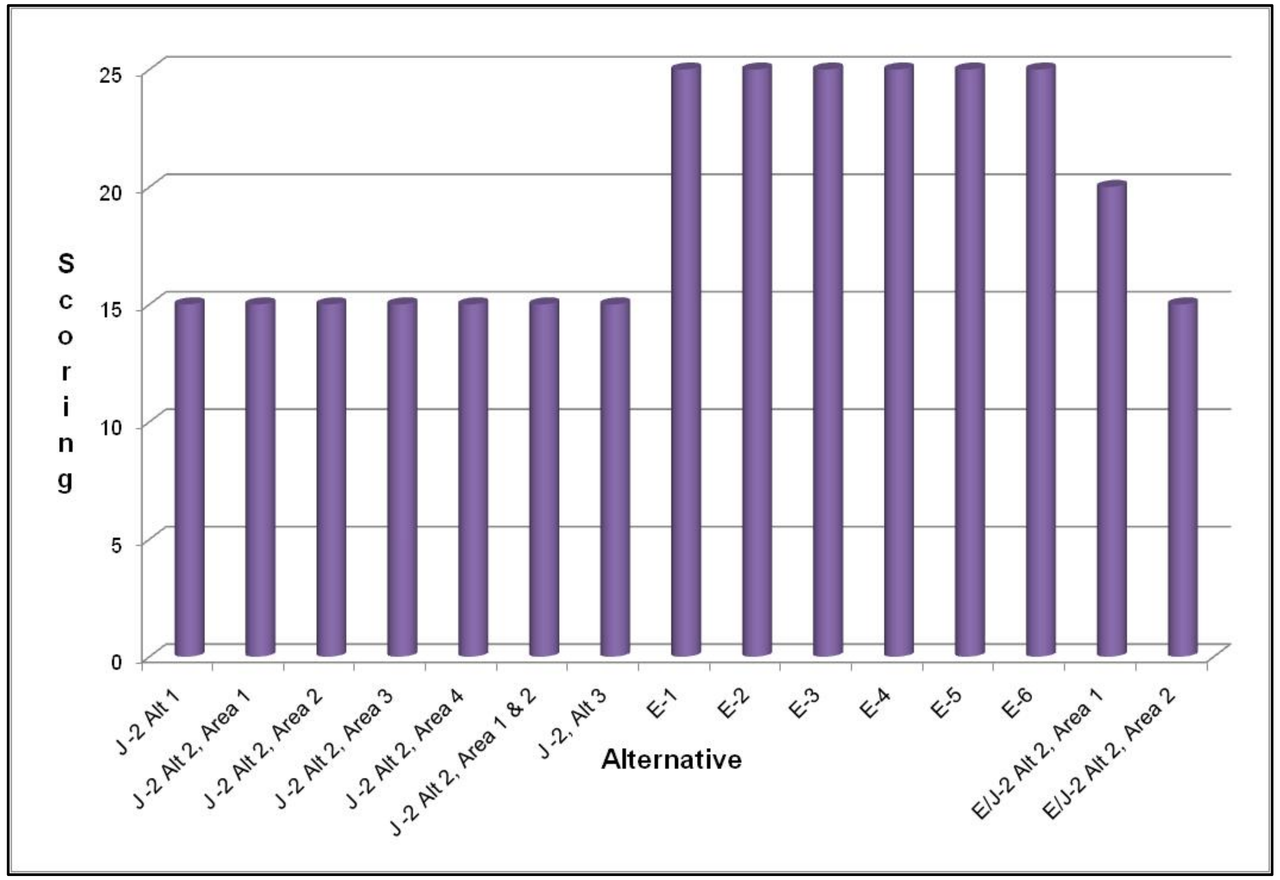
The alternatives varied from delivering 667 cfs over three days for the SDHF for the J-2 Alternative 3 to up to 2,000 cfs for three days for the J-2 Alternative 2 Areas 1 & 2, Elwood and combined J-2/Elwood alternatives. The alternatives were scored from 0 to 5, with a criterion weight of 10.

Figure 6.5-4 – Reduction to Shortages to Target Flows Scoring



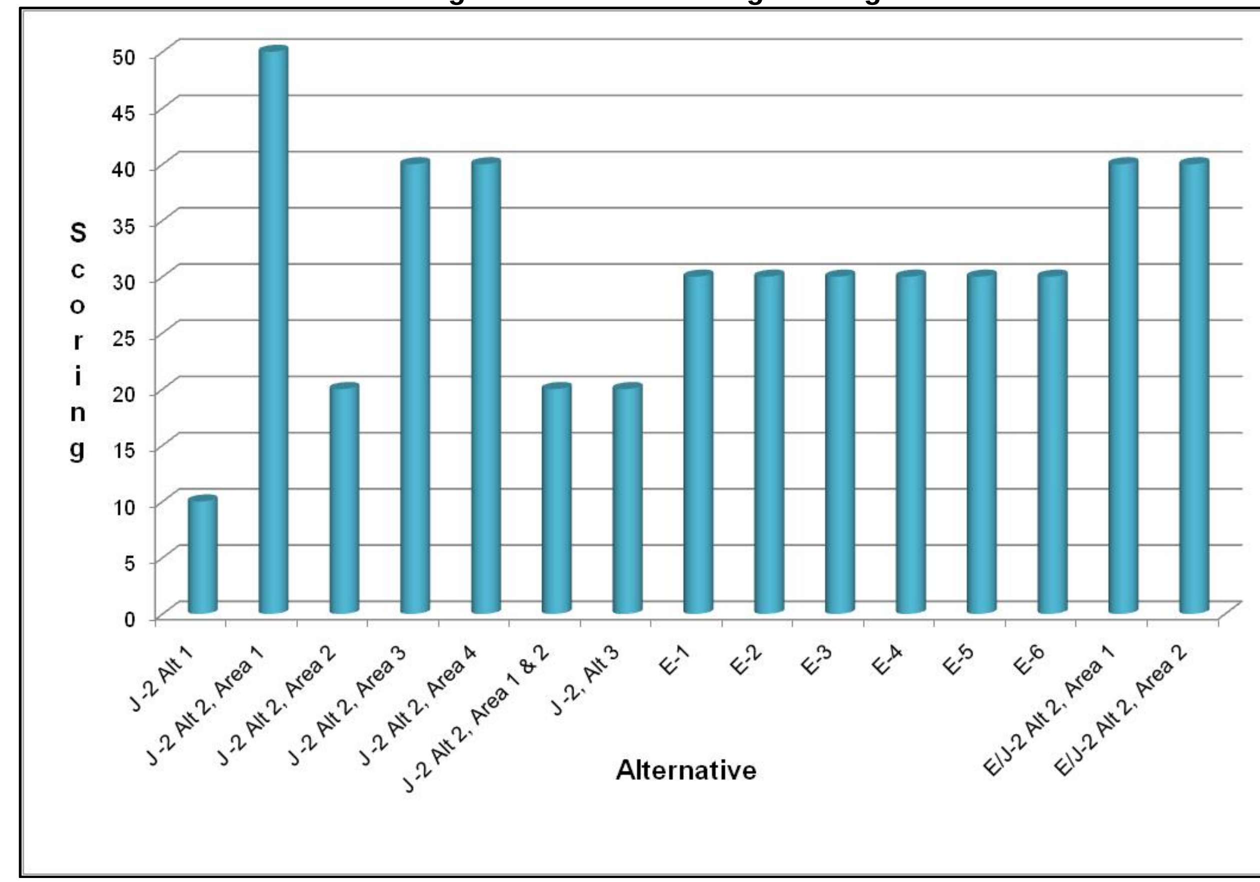
Reductions to shortages to target flows for the normal illustrative year varied between the alternatives from a low of approximately 18,000 acre-feet per year for the Elwood alternatives up to a high of 33,000 ac-ft per year for the Elwood /J-2 Alternative 2, Area 2 combination. The alternatives were scored from 0 to 5, with a criterion weight of 8.

Figure 6.5-5 – Flexibility Scoring



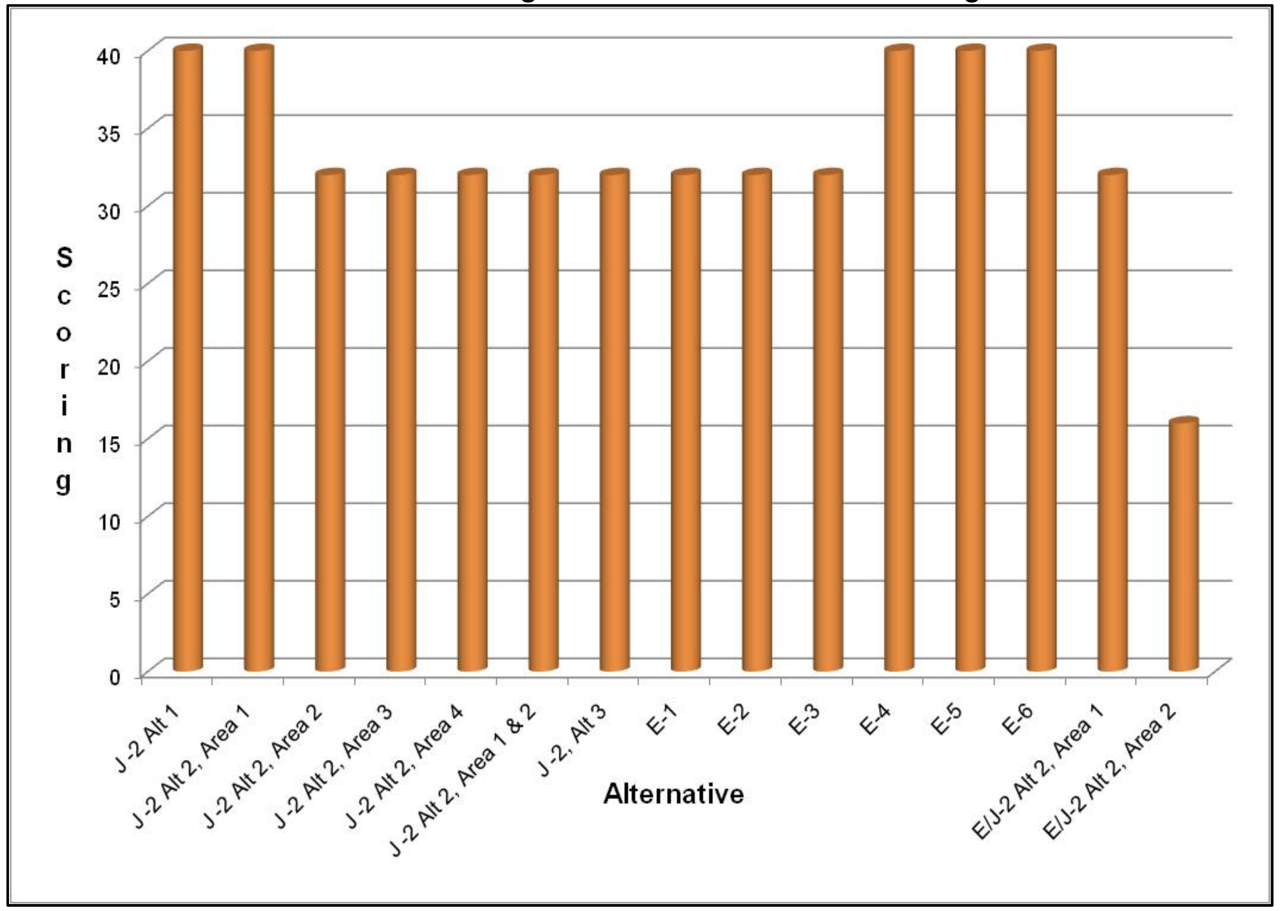
Flexibility and multiple benefits were scored based on the alternative's ability to provide sediment delivery, benefit fisheries, and benefit CNPPID. The J-2 alternatives scored a 3 (15 when weighted) because sedimentation delivery is considered to be minimal, whereas the Elwood alternatives scored higher due to their ability to deliver Plum Creek sediment during SDHF events. The alternatives were scored a 1 if one of the three benefits were achieved, a 3 if two of the benefits were achieved, and a 5 if all three of the benefits were achieved. The weight of this criterion was 5.

Figure 6.5-6 – Permitting Scoring



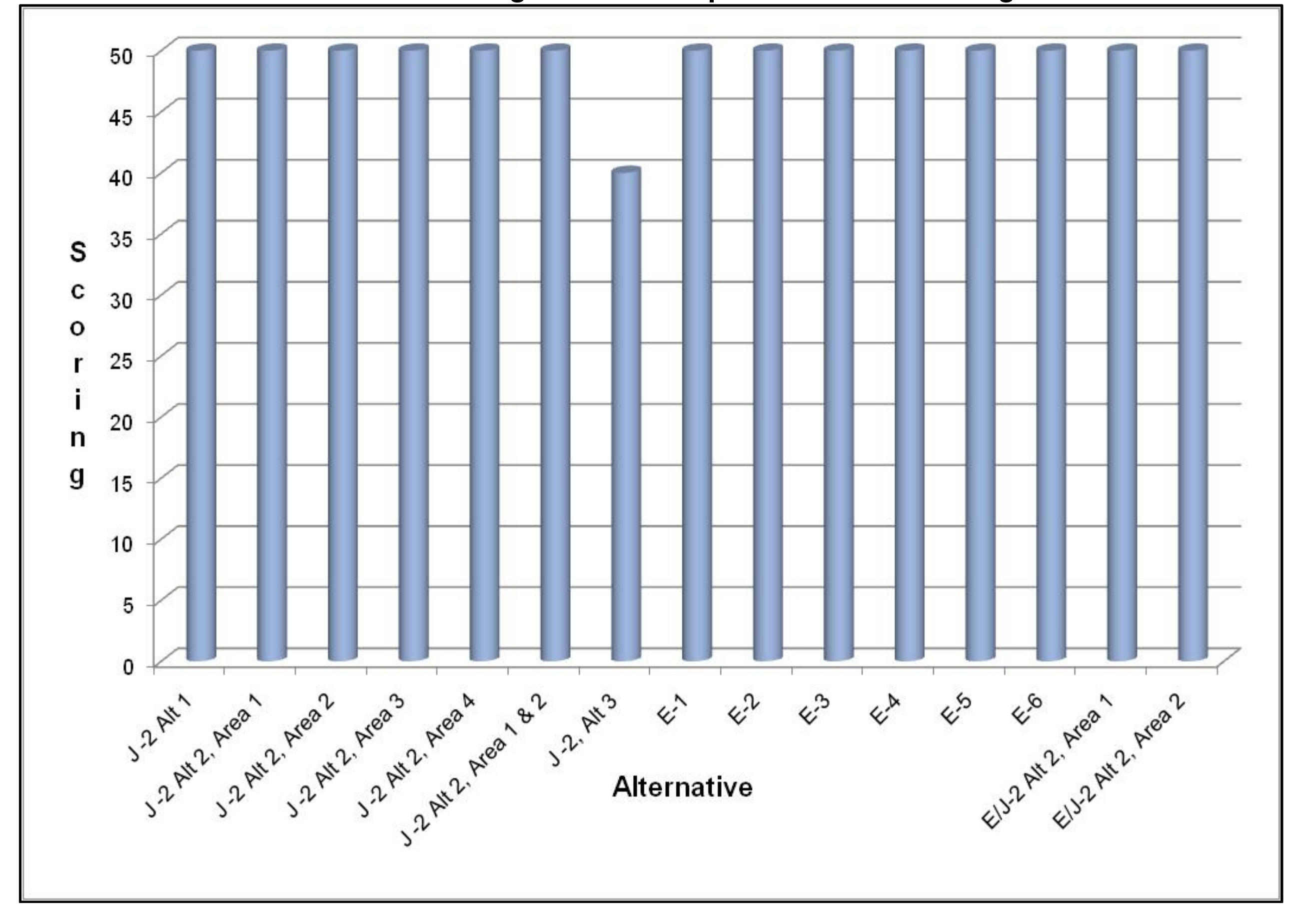
Regarding permitting, alternatives that had relatively large fill impacts to the Platte River (historic or existing channel) were generally ranked as most difficult to permit, and alternatives that impacted Plum Creek, smaller tributaries, or floodplain wetlands were ranked as easier to permit depending on the extent of impacts that were likely to occur for each alternative. J-2 Alternative 2, Area 1, did not place fill in a stream and hence scored the highest. The alternatives were scored from 0 to 5, with a criterion weight of 10.

Figure 6.5-7 – Landowners Scoring



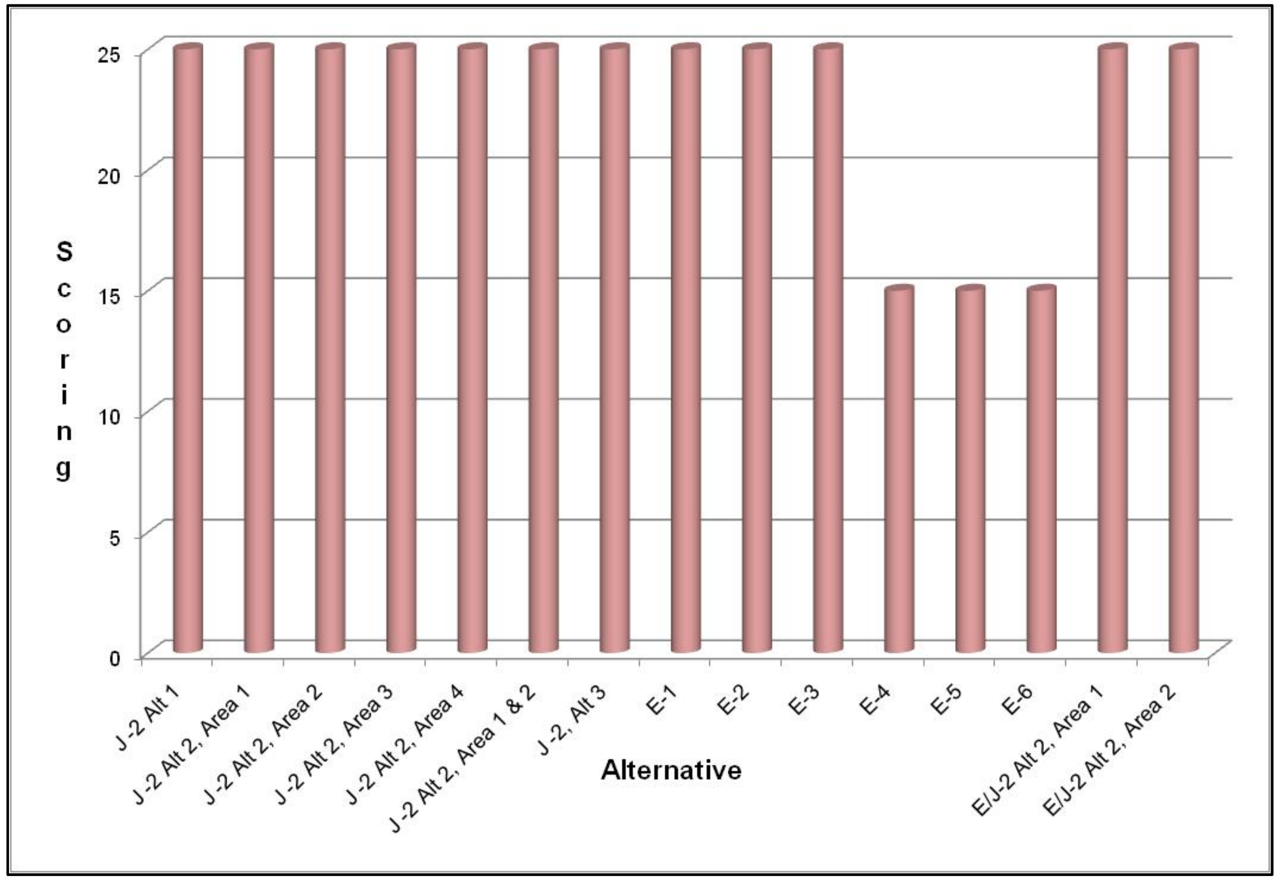
Several of the alternatives result in landowner impacts, including up to six landowners for the case of the combined J-2/Elwood alternatives. The J-2 alternatives had the fewest landowner impacts. The Elwood conveyance to the Platte River could potentially impact a large number of landowners, but at this stage it was assumed Plum Creek enlargement could be accomplished with easements. The alternatives were scored from 0 to 5, with a criterion weight of 8.

Figure 6.5-8 - Impacted Reach Scoring



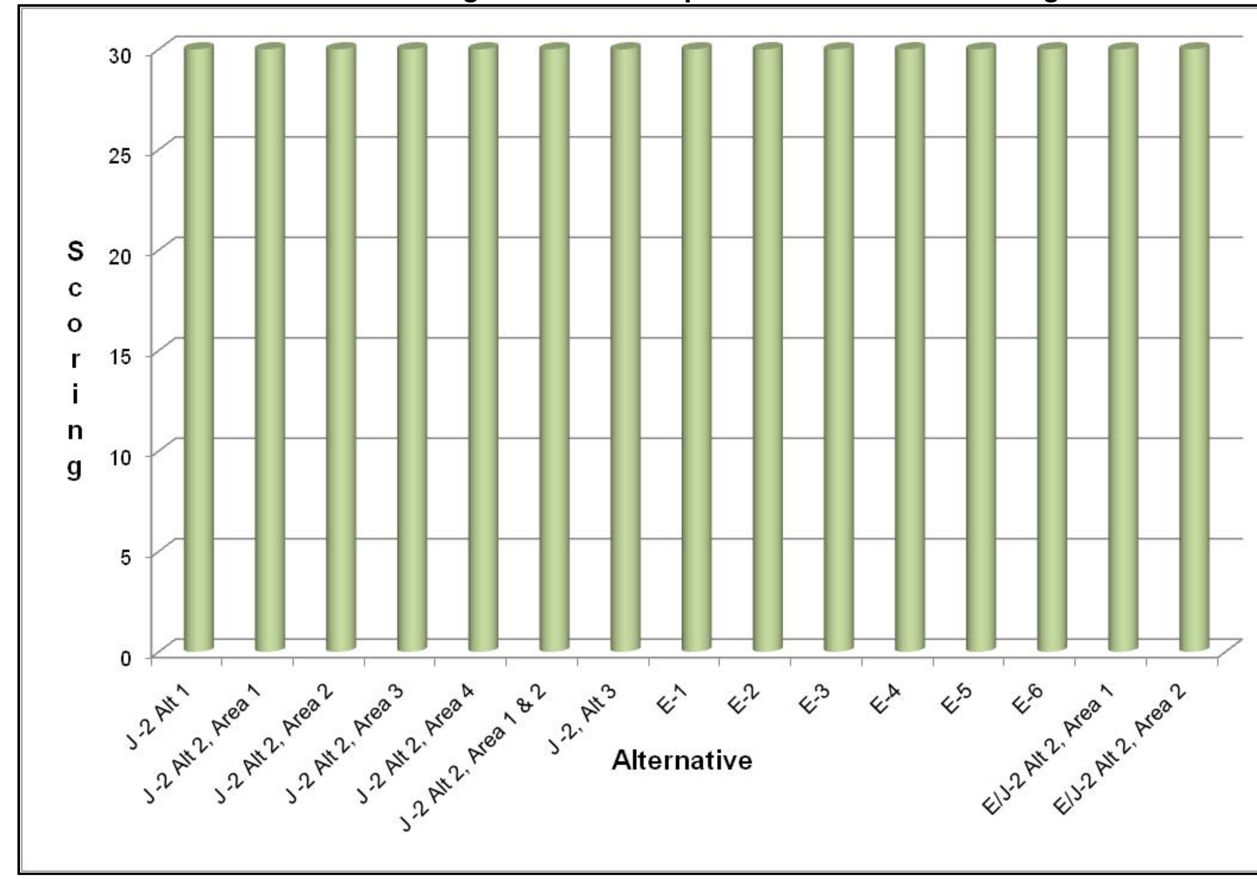
All alternatives other than J-2, Alternative 3 provided water for the entire reach. J-2 Alternative 3 (9.7 Canal Reservoir) could only provide water to 80% of the reach and was therefore was scored a 4. The alternatives were scored from 0 to 5, with a criterion weight of 10.

Figure 6.5-9 – Partnering Opportunity Scoring



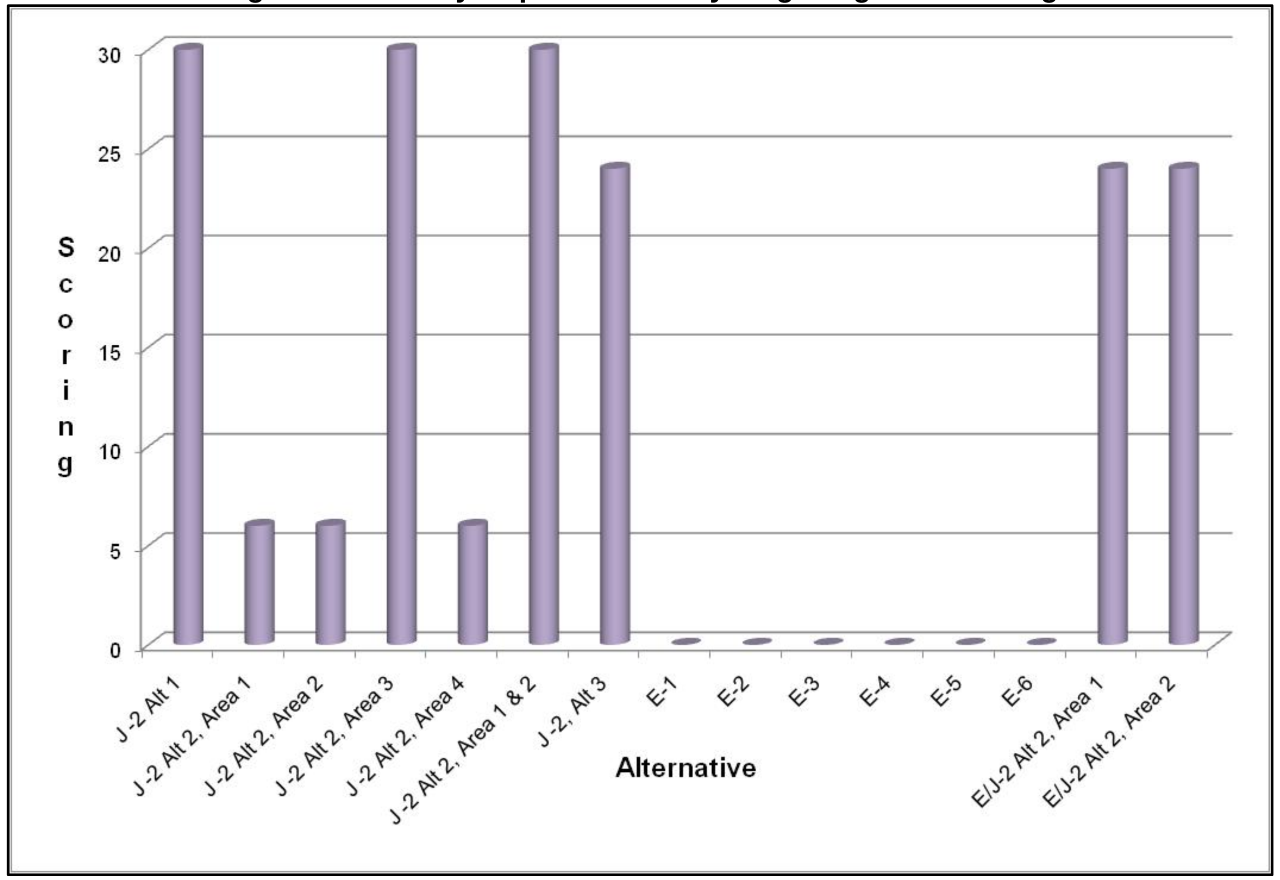
Opportunities for partnering with CNPPID and the USFWS were deemed high for all alternatives except the Elwood alternatives that did not involve the building of a gravity channel for inlet to Elwood. If potential for partnering with both entities exists, the alternative was scored a 5. Elwood alternatives E-4 through E-6 were scored a 3 since only the potential to partner with USFWS exists. The weight for this criterion was 5.

Figure 6.5-10 – Implementation Time Scoring



None of the alternatives would likely be permitted and built for the 2011 SDHF, and therefore none of the alternatives scored a 5. It is very likely that all of the alternatives could be built by 2014, and therefore they all scored a 3. The weight for this criterion was 10.

Figure 6.5-11 – Hydropower Flow Cycling Mitigation Scoring

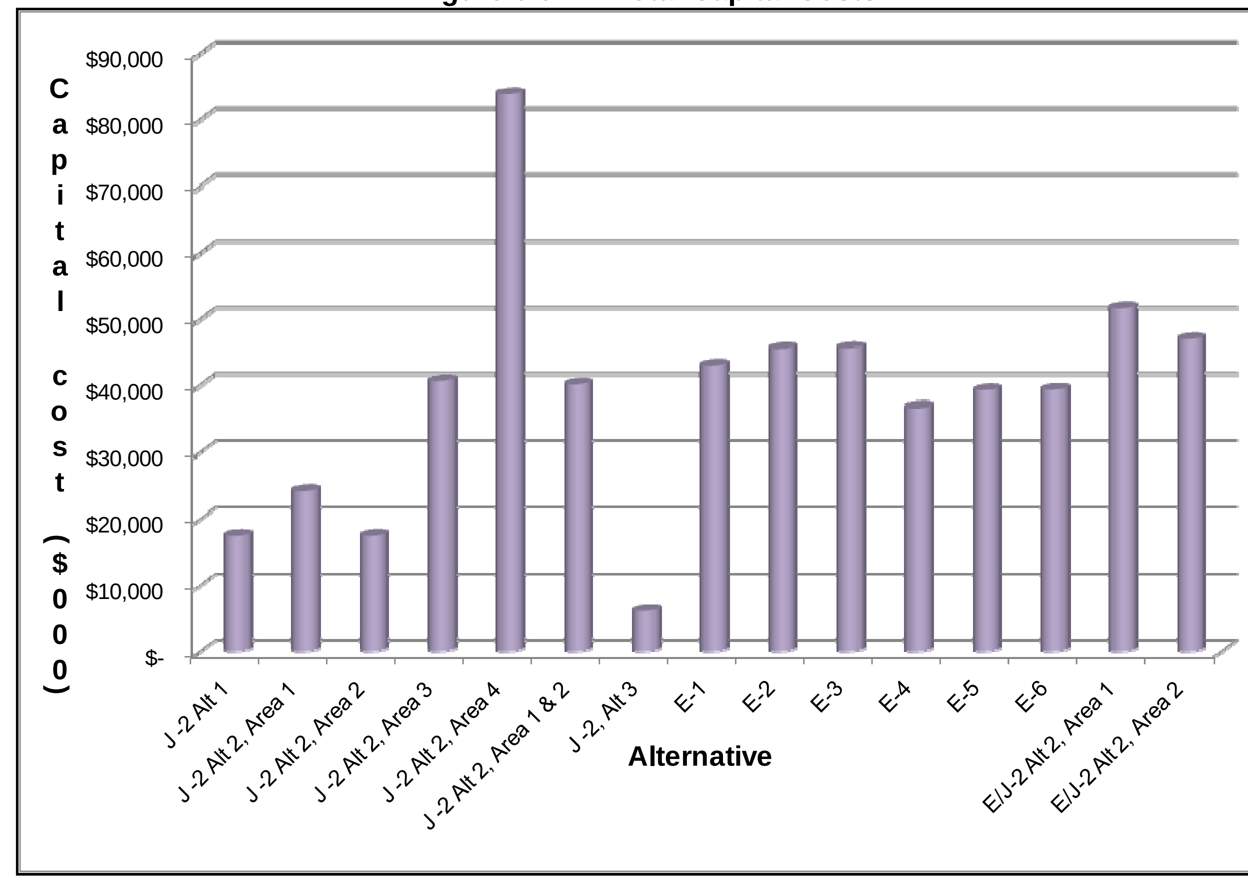


Hydropower flow cycling mitigation can be accomplished with all of the J-2 alternatives but none of the Elwood alternatives since it is not located downstream of the J-1 or J-2 hydropower stations. The alternatives were scored accordingly. The weight for this criterion was 6.

6.6 Capital Cost Comparisons

Capital costs for each of the alternatives vary dramatically with the location, size, and type of construction. The charts below show the overall capital cost per acre-foot of SDHF augmentation (regardless of whether the water is EA water routed down from Lake McConaughy and staged in the reservoir or captured excess flows) and cost per acre-foot of reduction to shortages to target flows for the alternatives investigated. Overall total capital costs for each of the alternatives investigated are shown in Figure 6.6-1.

Figure 6.6-1 – Total Capital Costs



The following charts illustrate the capital costs per delivered SDHF augmentation (regardless of whether the water is EA water routed down from Lake McConaughy and staged in the reservoir or captured excess flows), target flows, and total flows.

Figure 6.6-2 – Capital Costs per SDHF Augmentation Acre-feet

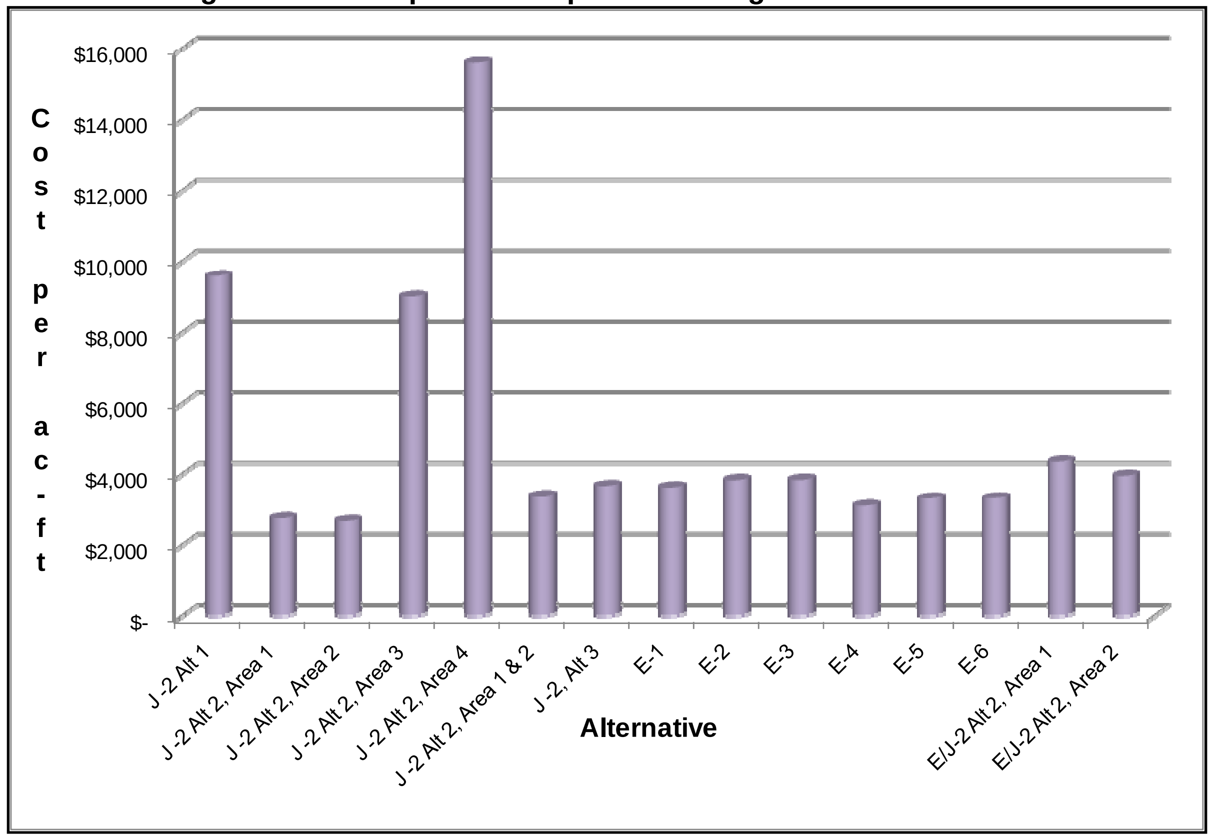


Figure 6.6-3 – Capital Costs per Reduction to Shortages to Target Flows for Normal Illustrative Year (1975)

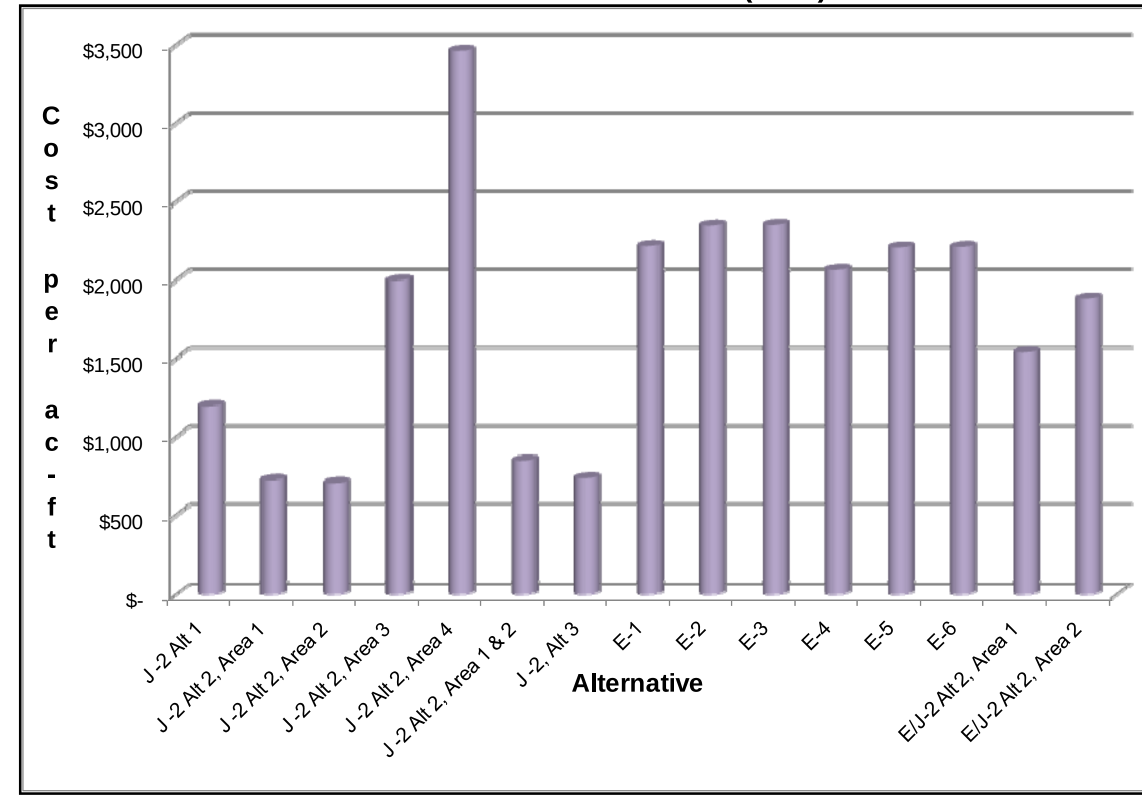
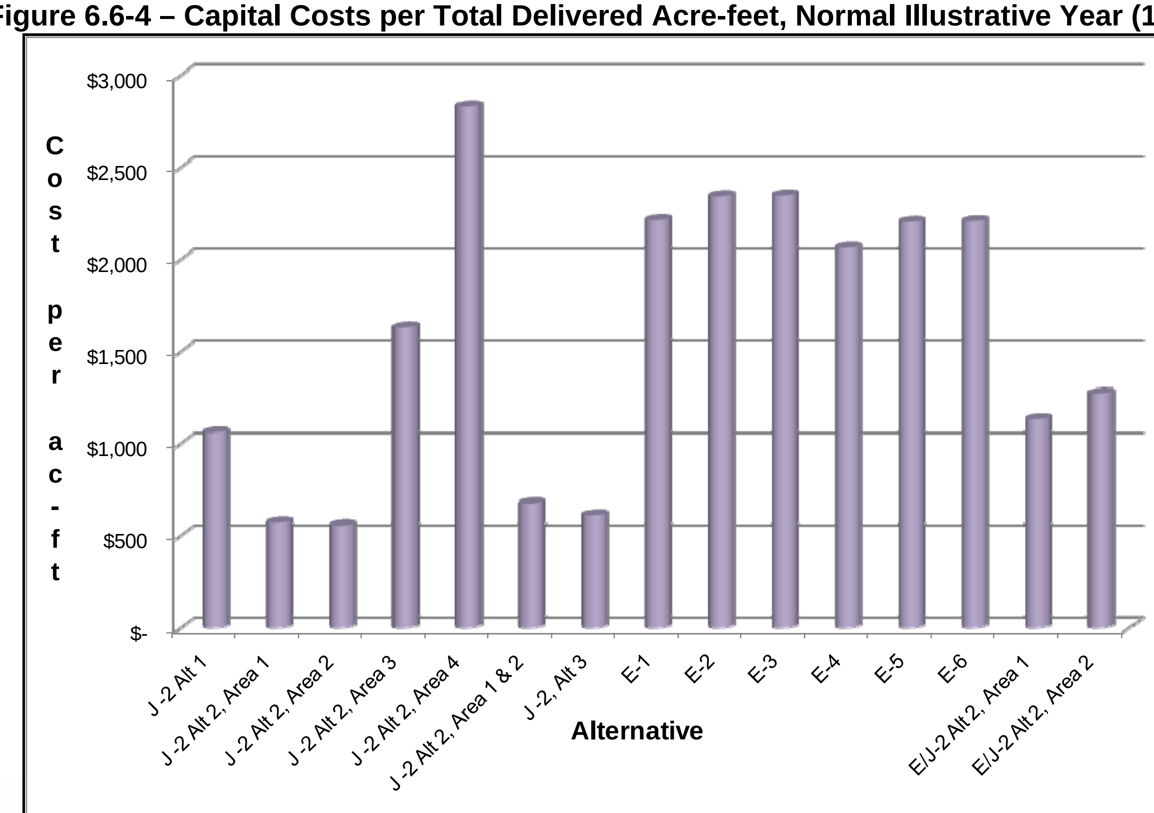


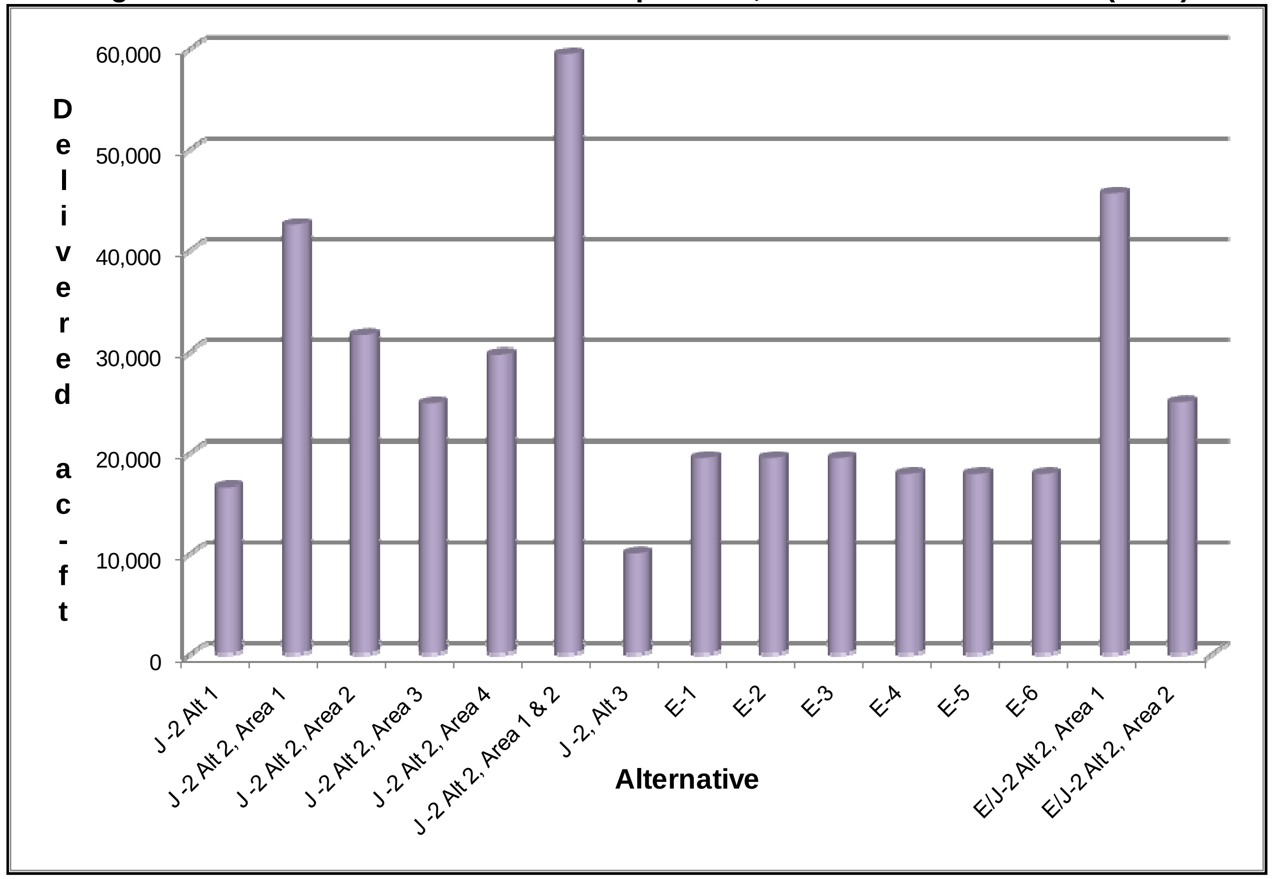
Figure 6.6-4 – Capital Costs per Total Delivered Acre-feet, Normal Illustrative Year (1975)



6.7 Comparison of Delivered Water

The following charts illustrate the quantity of water delivered for each alternative for SDHF augmentation, reductions to shortages to target flows, and total water.

Figure 6.7-1 – Total Delivered Acre-feet per Year, Normal Illustrative Year (1975)



For the J-2 alternatives, water for SDHF augmentation and reductions to shortages to target flows can both be accomplished. For the Elwood alternatives, the total water captured and delivered using by Elwood Reservoir can either be used for SDHF or reductions to shortages to target flows, but not both.

Figure 6.7-2 – Reduction to Shortages to Target Flows Acre-feet per Year for the Normal Illustrative Year (1975)

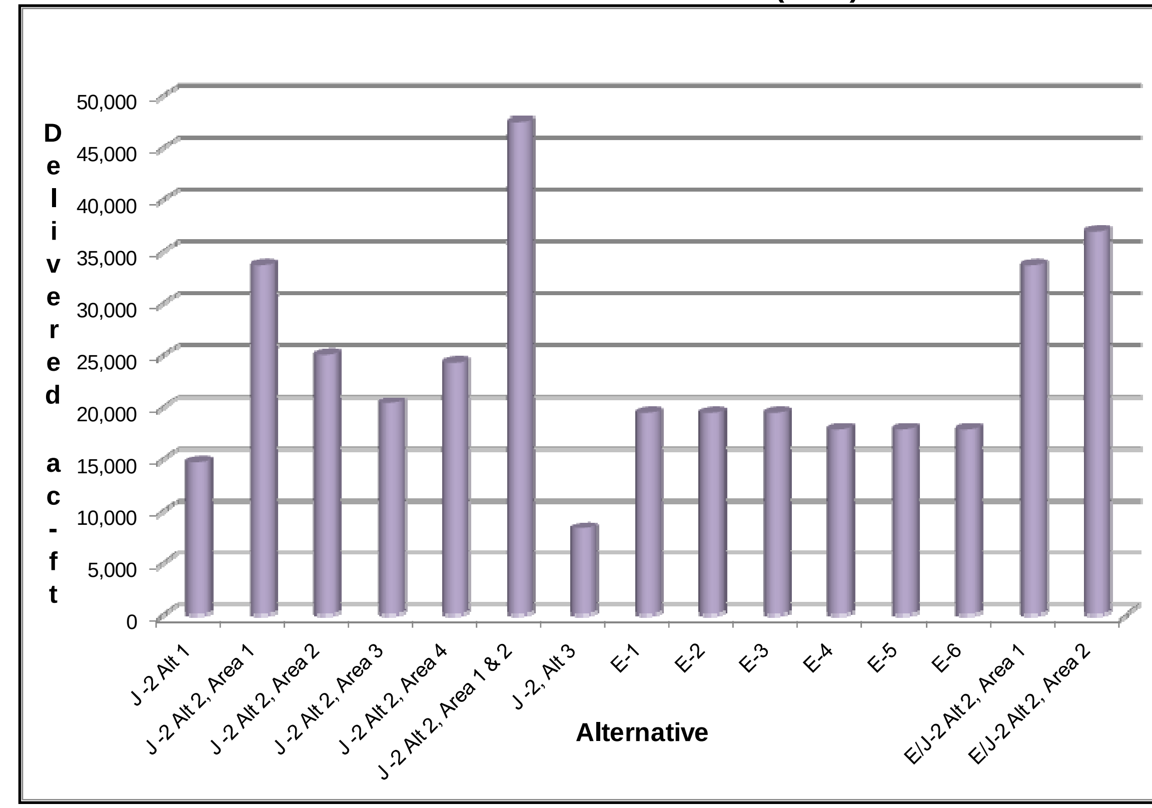
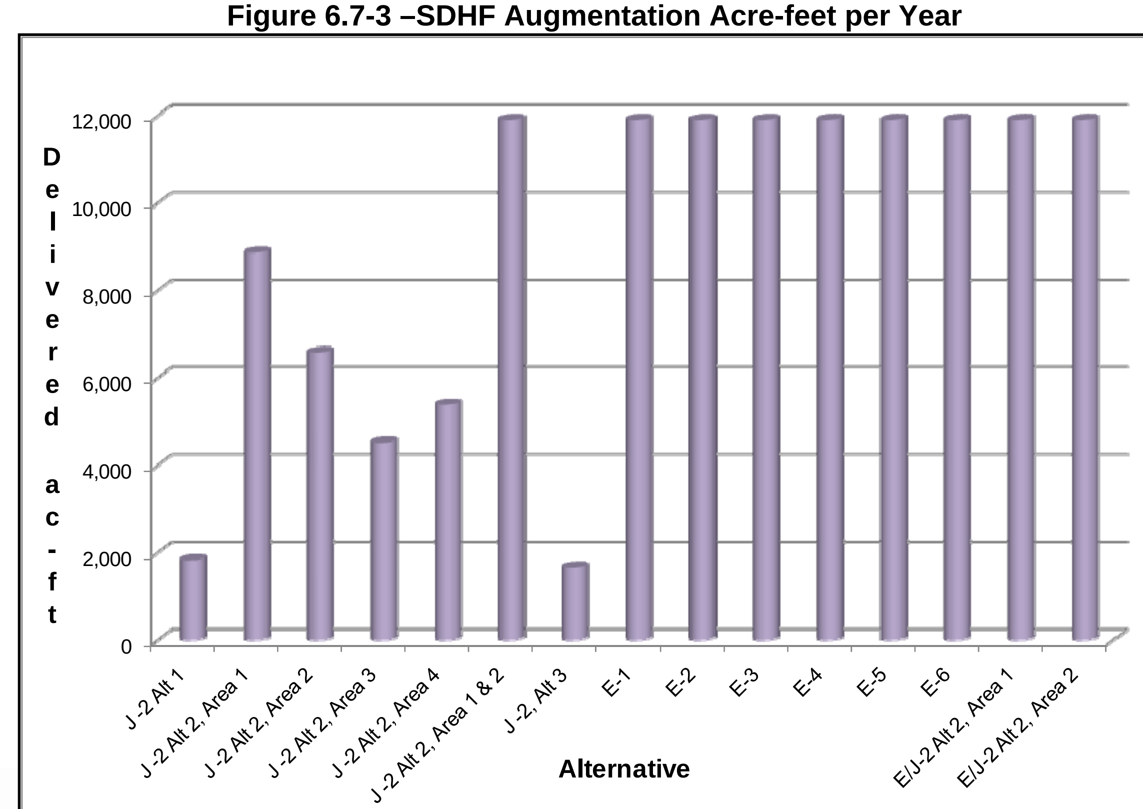


Figure 6.7-3 –SDHF Augmentation Acre-feet per Year



7. CONCLUSIONS AND RECOMMENDATIONS

The Elwood and J-2 Alternative Analysis project was able to identify several alternatives that meet the goal of providing 2,000 cfs for 3 days at Overton. The alternatives that met this goal are shown in Table 7-1.

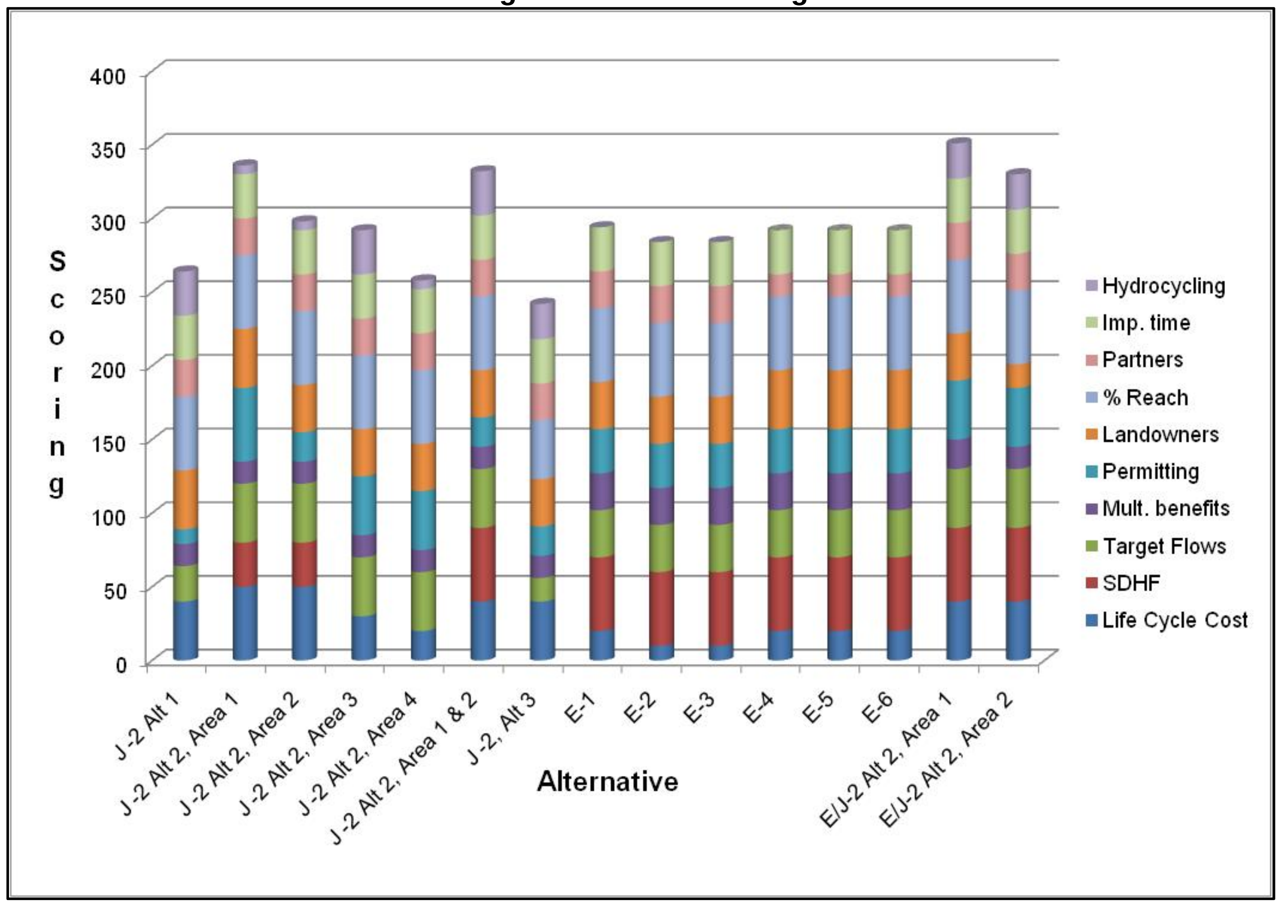
Table 7-1 Summary of Alternatives that Meet SDHF Goals

Alternative	Reservoir[1]	Storage ac-ft	Inlet	Outlet	Conveyance to Platte River	Capital Costs[2] ($000)	Annual Operating Costs ($000)	SDHF Augmentation[3] ac-ft / yr	Reductions to Shortages to Target Flows[4],[6]		
									Wet Yr ac-ft	Normal Yr ac-ft	Dry Yr ac-ft
J -2 Alt 2, Area 1 & 2[5]	J-2 excavation Areas 1&2	14,320	Phelps Canal	Radial Gates	n/a	$40,039	$321	11,901	57,931	47,480	34,237
E-1	Elwood buttress	26,899	Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$42,942	$690	11,901	21,736	19,408	19,154
E-2	Elwood remove & replace embankment	26,899	Gravity Canal	New Outlet (2 pipes) Tunnels	Plum Creek, 2,400 cfs	$45,444	$721	11,901	21,736	19,408	19,154
E-3	Elwood remove & replace upstream shell	26,899	Gravity Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$45,522	$722	11,901	21,736	19,408	19,154
E-4	Elwood buttress	26,899	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$36,677	$449	11,901	21,330	17,788	19.162
E-5	Elwood remove & replace embankment	26,899	Existing E-65 Canal	New Outlet (2 pipes)	Plum Creek, 2,400 cfs	$39,179	$468	11,901	21,330	17,788	19.162
E-6	Elwood remove & replace upstream shell	26,899	Existing E-65 Canal	2-8' Tunnels	Plum Creek, 2,400 cfs	$39,257	$459	11,901	21,330	17,788	19.162
E/J-2 Alt 2, Area 1	Elwood buttress, J-2 excavation, Area 1 modified	36,615	Existing E-65 and J-2 Return	Tunnel (1 only)	Plum Creek, 1,200 cfs	$51,626	$457	11,901	44,119	33,668	25,029
E/J-2 Alt 2, Area 2	Elwood buttress, J-2 excavation, Area 2	33,479	Existing E-65 and J-2 Return	Tunnel (1 only)	Plum Creek, 1,200 cfs	$46,861	$422	11,901	33,677	24,974	18,757

Notes:
(1) Base cost of reservoir (total estimated project cost without inlet, outlet, and conveyance costs). For Elwood, the cost represents improvements to the embankment.
(2) Total estimated project cost including base reservoir cost, inlet, outlet, and conveyance costs
(3) Water to augment SDHF could be either environmental account (EA) water routed down from Lake McConaughy, and staged in the reservoir, or stored excess flows captured and stored in reservoirs immediately before a SDHF if available. Though units are ac-ft/yr, the values presented are the total volume of SDHF augmentation flows provided by the alternative over three days.
(4) Water to reduce shortages to target flows is excess flows in CNPPID's system that could between stored during times of excess, and released during periods of shortage.
(5) Assumes only gravity fill for Areas 1 and 2.
(6) Assumes J-2 storage site(s) are full at beginning of water year (October) for consistency of scoring all alternatives. Results shown are for the illustrative years only. Long-term yield averages will vary.

The alternatives that incorporate a high rate of release from Elwood Reservoir are capital cost prohibitive and are logistically difficult to implement. A Kepner Tregoe scoring matrix was developed to more fully evaluate the alternatives. The following graph summarizes the composite scoring results that indicate the J-2 Alternative 2, Areas 1 and/or 2 ranked the highest.

Figure 7-1 Total Scoring



Based on these results, it is recommended the J-2 Alternative 2, Areas 1 and/or 2 be advanced to feasibility stage of analysis. In addition, Elwood Reservoir appears to have an attractive use when used at a low release rate into Plum Creek. Although not a specific goal or objective of this study, modeling of target flow operations indicates Elwood Reservoir is typically at minimum stage over the winter months which is also when the reliability of excess flows are high. More analysis is needed, but it appears using Elwood Reservoir to store winter excess flows would not interfere with CNPPID current use. A low release rate into Plum Creek of around 100 to 500 cfs would minimize Plum Creek stabilization costs, Elwood upgrade costs, and minimize roadway crossing upgrades. With a potential high volume yield and minimal capital costs, this alternative should be further investigated.

Scope of Work for Feasibility Analysis of Preferred Alternative
The next step for implementation of the preferred option should be a feasibility analysis to refine the design, costs, constraints and schedule of the project. The following is a brief description of the major subjects to be analyzed.

Topographic Information
Before any further analysis is performed, much better and more accurate topographic information is required. Therefore, the aforementioned LIDAR must become available, or the area should be mapped with conventional aerial photography or land-based topographic survey methods.

Geotechnical Analysis
Assuming that the alternative that moves forward for further analysis is a combination of a new J-2 reservoir and the Elwood Reservoir, further geotechnical analysis, including soil borings, must be conducted. Even if Elwood is not used for the bulk of SDHFs, any change to its operation must be analyzed in more detail than it has been so far with the additional geotechnical information. Likewise, further analysis of any J-2 reservoir alternative must be with the benefit of additional geotechnical information. Seepage is a major concern and lining options must be evaluated.

Permitting Information
An in-depth evaluation of environmental permitting requirements, with an emphasis on timeframes, must be conducted during the feasibility analysis.

Conceptual Design and Conceptual Design Level Opinions of Construction Costs
The cost estimates in the screening analysis must be further refined with the benefit of better topography, more complete geotechnical information and more developed design. Therefore, the following components of design should undergo conceptual-level design:

- Outlet works from Elwood reservoir
- Conveyance from the outlet works to Plum Creek
- Enlarging and armoring of Plum Creek
- Earthwork for the proposed J-2 reservoir
- Outlet gate for the proposed J-2 reservoir
- Pump station for the proposed J-2 reservoir

Model Operations and Refine the Impact on Operational Costs
The entire operations must be modeled with refinements, which will allow operational costs and yields to be more accurately estimated. Continuous simulation modeling for multiple years is necessary to accurately predict alternatives' ability to reduce shortages to target flows. The next phase of project analysis will include refining target flow operations analysis and actual WAP scoring.

Land Acquisition Requirements and Costs
A detailed assessment of land acquisition needs and the associated costs must be performed.

Schedule
A complete upgraded schedule is important at this time. Implementation by the year 2014 is still achievable, but a detailed schedule including critical path elements will be a useful tool for moving forward. The major components to be scheduled include:

- Preliminary design
- Environmental permitting
- Land acquisition
- Final Design and construction documents
- Construction
- Operational start up