



GEOTECHNICAL AND
WATER RESOURCES ENGINEERING

CONCEPTUAL DESIGN REPORT
J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES, NEBRASKA

Submitted to
Platte River Recovery Implementation Program
Executive Directors Office
2727 Bryant Street
Denver, Colorado 80211

Submitted by
RJH Consultants, Inc.
9800 Mt. Pyramid Court, Suite 330
Englewood, Colorado 80112
303-225-4611
www.rjh-consultants.com

April 2013
Project 12116




Robert J. Huzjak, P.E.
Project Manager

TABLE OF CONTENTS

TABLE OF CONTENTS	I
EXECUTIVE SUMMARY	V
SECTION 1 - INTRODUCTION	1
1.1 PURPOSE OF REPORT	1
1.2 OBJECTIVES OF WORK.....	1
1.3 SCOPE OF WORK	1
1.4 AUTHORIZATION	2
1.5 PERSONNEL	2
SECTION 2 - GENERAL PROJECT DESCRIPTION	4
2.1 PROJECT GOALS, BACKGROUND, AND OBJECTIVES	4
2.2 OVERVIEW OF PROJECT CONCEPT	4
2.3 OPERATIONAL REQUIREMENTS	5
SECTION 3 - RESERVOIR SITING	7
3.1 GENERAL	7
3.2 OPERATIONS AND WATER AVAILABILITY.....	7
3.3 GEOLOGY	7
3.4 HYDROLOGY	7
3.5 TOPOGRAPHY	8
3.6 INFRASTRUCTURE	8
3.7 ENVIRONMENTAL AND CULTURAL	8
3.8 LAND AVAILABILITY.....	8
3.9 CONCLUSIONS FOR SITE SELECTION	8
SECTION 4 - EXISTING CONDITIONS.....	10
4.1 GENERAL	10
4.2 EXISTING LAND USE	10
4.3 FLOODPLAIN AND HYDROLOGIC CONDITIONS	10
4.3.1 Platte River.....	11

4.3.2	Plum Creek	12
4.3.3	Unnamed Tributary	13
4.4	SUBSURFACE AND GROUNDWATER CONDITIONS.....	13
4.4.1	Site Geology.....	13
4.4.2	Geotechnical Conditions	14
4.4.3	Groundwater Data	14
SECTION 5 - PRIMARY TECHNICAL ISSUES AND DESIGN OVERVIEW.....		16
5.1	IDENTIFIED DESIGN ISSUES	16
5.2	OVERVIEW OF DESIGN CONCEPTS.....	17
5.2.1	Seepage Management System	17
5.2.2	Sloped Reservoir Bottom	18
5.2.3	Shallow Borrow Areas	18
5.2.4	Zoned Embankment	19
5.2.5	Embankment Slope Protection	19
5.2.6	Hydraulic Facilities.....	19
5.2.7	Exterior Embankment Protection	19
5.2.8	Phelps Canal Modifications.....	20
5.2.9	Two-Reservoir Concept	20
SECTION 6 - EMBANKMENT AND RESERVOIR CONCEPTS		21
6.1	GENERAL	21
6.2	RESERVOIR FOOTPRINTS	21
6.3	EMBANKMENT CONFIGURATION	22
6.3.1	Zoned Embankment Concept.....	22
6.3.2	Material Properties	22
6.3.3	External Slopes	25
6.4	RESERVOIR LINER.....	26
6.4.1	Liner Concept	26
6.4.2	Seepage Analysis	26
6.4.3	Conclusions from Seepage Analyses.....	27
6.5	RESERVOIR GRADING	30

6.6	RESERVOIR STORAGE	31
6.7	UPSTREAM SLOPE PROTECTION AND DAM CREST ELEVATION	31
SECTION 7 - RESERVOIR HYDRAULIC STRUCTURES		34
7.1	GENERAL	34
7.2	AREA 2 INLET/OUTLET STRUCTURE	36
7.3	AREA 2 OUTLET STRUCTURE	37
7.4	AREA 1 INLET STRUCTURE	37
7.5	AREA 1 OUTLET STRUCTURE	38
SECTION 8 - CANAL AND CREEK MODIFICATIONS		40
8.1	PHELPS CANAL MODIFICATIONS.....	40
8.2	EROSION PROTECTION ALONG EMBANKMENTS	41
8.2.1	Unnamed Tributary	41
8.2.2	Plum Creek	41
SECTION 9 - OPINION OF PROBABLE PROJECT COST		43
9.1	CAPITAL COSTS	43
9.2	OPERATION AND MAINTENANCE COSTS	46
SECTION 10 - SEEPAGE COLLECTION AND PUMP SYSTEM ALTERNATIVE.....		48
10.1	GENERAL	48
10.2	EVALUATION OF TECHNICAL FEASIBILITY AND RISKS	48
10.3	DESCRIPTION OF THE RJH CONCEPT.....	50
10.4	OPINION OF PROBABLE PROJECT COSTS FOR ALTERNATIVE	50
SECTION 11 - LIMITATIONS		53
SECTION 12 - REFERENCES		54
LIST OF TABLES		
Table 6.1	Summary of Geotechnical Material Properties	
Table 6.2	Summary of Recommended and Computed Slope Stability	
Table 7.1	Summary of Selected Gate Types and Sizes	
Table 7.2	Summary of Hydraulic Design Requirements	
Table 9.1	RJH Opinion of Probable Project Costs	
Table 9.2	Opinion of Annual O&M Costs	

Table 10.1 RJH Opinion of Probable Project Costs for SCPS Alternative

LIST OF FIGURES

Figure 2.1	Site Vicinity Map
Figure 2.2	Location of Proposed J-2 Reservoirs
Figure 2.3	Operational Flow Criteria
Figure 4.1	Existing Land Use
Figure 4.2	FEMA-Mapped Floodplains Near Site
Figure 4.3	Thickness of Clayey Soils at Site
Figure 4.4	Groundwater Conditions
Figure 6.1	J-2 Reservoir Overall Plan
Figure 6.2	J-2 Reservoir Embankment Sections, Sheet 1 of 2
Figure 6.3	J-2 Reservoir Embankment Sections, Sheet 2 of 2
Figure 6.4	Elevation-Area-Capacity Curves
Figure 7.1	Plan of Reservoir Hydraulic Structures
Figure 7.2	Area 2 Inlet/Outlet Structure
Figure 7.3	Area 2 Outlet Structure
Figure 7.4	Area 1 Inlet Structure
Figure 7.5	Area 1 Outlet Structure
Figure 8.1	Phelps Canal Improvements
Figure 10.1	Seepage Collection and Pump System (SCPS) Schematic
Figure 10.2	SCPS Alternative Conceptual Plan
Figure 10.3	Typical Seepage Collection Pump System Section
Figure 10.4	Typical Pump Station

EXECUTIVE SUMMARY

This report presents a conceptual design and a corresponding opinion of probable project cost (OPPC) for the J-2 Regulating Reservoir Project (Project) located in Gosper and Phelps Counties, Nebraska.

RJH identified the following primary issues that significantly impacted the development of the concept design for the Project:

- The lack of a continuous and reliably thick low-permeability soil unit in the foundation, the presence of variable and interlayered sandy materials in the foundation, the presence of relatively shallow groundwater below much of the site, and the need to construct the embankment and slope protection using on-site materials to manage Project costs.
- The planned operation of the reservoir, which will require the reservoir level to routinely fluctuate from full to empty. For some operational conditions the reservoir will be lowered from full to near empty in about 3 days.
- The requirement for hydraulic facilities to convey both small flows, which will be less than 500 cfs and large flows, which will be on the order of 2,000 cubic feet per second (cfs) with reasonable control and accuracy. The need to operate the upper 3 feet of the Area 2 Reservoir to convey water back into Phelps Canal.
- The exterior slopes of the embankments will be subject to flows from the Platte River, Plum Creek, and the unnamed tributary.
- Phelps Canal downstream of the J-2 turnout needs to convey 1,675 cfs to the Area 2 Reservoir without inundating crossing bridges or creating a significant risk of breakout and 1,000 cfs downstream of the Area 2 Reservoir.

The primary Project components needed to meet Project objectives and address these primary issues are illustrated on Figures 6.1 through 6.3 and include:

- **Two-reservoir concept.** Both reservoirs would have a normal maximum pool at Elevation (El.) 2356.0. The total active reservoir storage for the Project would be about 15,400 acre-feet (ac-ft) when both reservoirs are at El. 2356.0. The active storage in Area 1 and Area 2 reservoirs would be about 12,135 ac-ft and about 3,265 ac-ft, respectively. Approximately 840 ac-ft of storage in the upper 3 feet of Area 2 Reservoir could be conveyed into Phelps Canal for regulation of irrigation flows.
- **A seepage management system.** RJH selected a clay-lined reservoir to manage seepage. The reservoir liner concept includes a 1.5-foot-thick liner constructed

using the clayey soils at the site protected with 3 feet of cover soil. The cover soil is required to protect the liner from the long-term impacts from the environment such as freeze-thaw and wet-dry cycles, burrowing animals, vegetation, etc. The bottom of the liner would be generally above the average estimated groundwater elevation to reduce dewatering required for construction and potential uplift pressures associated with fluctuations in the groundwater elevation.

- **Sloped reservoir bottoms.** A sloped reservoir bottom provides two primary benefits: 1) it reduces the potential for uplift of the reservoir liner because the sloped reservoir bottom maintains the liner above the groundwater levels, and 2) it maintains reservoir head on the Platte River outlet gates for a significant portion of the total storage.
- **Relatively shallow borrow excavations.** If significant borrow below the groundwater level was required, the dewatering costs would likely become a significant factor in the overall Project costs. Except for localized borrow for sandy soils that will be needed for the soil-cement slope protection and select parts of the embankment, the liner fill, cover fill, and much of the embankment fill materials are available above the anticipated groundwater levels.
- **Zoned earthen embankments for the Project dams.** The zoned earthen embankment dams would extend a total length of about 5.7 miles (along the centerline) around most of the reservoir perimeters and abut the natural ground surface near Phelps Canal. The maximum height of the dams would be about 32 feet and 22 feet for Area 1 and Area 2 reservoirs, respectively. The embankment would consist of: an upstream zone of primarily sandy soils to address concerns related to frequent rapid fluctuations in the reservoir pool, a central zone comprised of clayey soils connected to the liner to reduce seepage losses, a filter sand/chimney drain downstream of the central clayey zone to safely manage seepage, and a downstream zone of random fill that would be obtained from the on-site materials.
- **Complete upstream slope protection for the dams.** The entire upstream slope would be covered with soil-cement to provide protection from wave erosion. Soil-cement was selected based on performance history and economics. A gravel layer below the soil-cement was included in the concept to prevent uplift pressures on the soil-cement and to mitigate removal of embankment soils through cracks during reservoir drawdown.
- **Multi-gate hydraulic structures.** The concept includes inlet and outlet structures equipped with at least two different size gates to provide accurate

hydraulic regulation of flow. Armored discharge channels were also included to safely convey discharges to the Platte River channel.

- **Protection against erosion for portions of the downstream embankments subject to stream flows.** RJH included an engineered grass-lined channel with a small concrete low-flow channel to safely convey routine flows from the unnamed tributary between the two reservoirs. The potential for undermining and eroding the embankment where Plum Creek would turn and flow parallel the west side of the Area 2 dam would be mitigated with a buried sheetpile wall and soil-cement armoring on the lower part of the exterior embankment slope. The computed maximum velocity in the Platte River, for the most conservative condition during the 100-year event is less than 2 feet per second (fps). Therefore, erosion potential along the Platte River can be mitigated with a grass-covered embankment.
- **Improvements to Phelps Canal.** Modifications to parts of Phelps Canal between the J-2 Return and the Area 1 intake include: minor placement of fill to provide 1 foot of freeboard in the canal, raising three existing bridges over the canal, adding a new siphon parallel to the existing siphon below Plum Creek; raising the walls of the existing flume over the unnamed tributary, and adding a check structure in the canal as part of the Area 1 intake structure.

RJH's OPPC is about \$62.6 million (2013 dollars). This includes contingencies, direct construction costs, and allowances for engineering, permitting, etc. This cost does not include land costs. RJH's opinion of Annual Operations and Maintenance costs is \$140,000.00.

SECTION 1 - INTRODUCTION

1.1 Purpose of Report

This report presents a conceptual design and a corresponding opinion of probable project cost (OPPC) for the J-2 Regulating Reservoir Project (Project) located in Gosper and Phelps Counties, Nebraska.

1.2 Objectives of Work

The Platte River Recovery and Implementation Program (Program) retained RJH Consultants, Inc. (RJH) to:

- Provide an independent engineering assessment of design concepts and Project costs as proposed by Olsson and Associates (Olsson) (2012).
- Develop an updated conceptual design for the Project based on existing data and stated operational requirements.
- Develop an independent OPPC based on the updated conceptual design.
- Identify primary differences between the previous design concepts and Project costs developed by Olsson, and the updated design concepts and costs developed by RJH. The primary differences between the RJH concept and the Olsson concept are presented in a separate document.

1.3 Scope of Work

To accomplish the objectives stated in Section 1.2, RJH performed the following tasks:

- Participated in Project meetings to identify primary operational requirements, provide interim updates of findings and Project progress, and discuss design issues and owner preferences. Meeting notes were prepared and distributed after the meetings. Meeting notes are also included in the Project Notebook¹.

¹ The Project Notebook is a separate document that contains project information, engineering analyses calculations, and other supporting documents that were used to develop the conceptual design and cost opinion.

- Performed a site visit to observe the proposed Project area and the existing infrastructure owned by Central Nebraska Public Power and Irrigation District (CNPPID).
- Reviewed topographic, hydrologic, geotechnical, and hydrogeologic (i.e., groundwater) data available from Olsson, the Program, CNPPID, or from readily available publications and evaluated the reliability of that data.
- Based on the available data and operational requirements, identified key issues that needed to be addressed to develop a safe and reliable concept for the Project.
- Reviewed the site selected previously by others and evaluated if the selected reservoir site is appropriate for this evaluation.
- Reviewed the conceptual design and supporting engineering as presented in Olsson 2012.
- Performed preliminary engineering analyses to support development of a conceptual design that addressed key safety and operational issues identified for the Project.
- Developed an OPPC for the RJH conceptual design.
- Prepared this report.
- Prepared a separate memorandum that compares differences between the RJH and Olsson 2012 conceptual designs and cost opinions.
- Prepared a Project Notebook (refer to Footnote 1, page 1).

1.4 Authorization

The scope of work performed by RJH was authorized in a contract between RJH and the Nebraska Community Foundation, Inc. (representing all signatories to the Program) dated August 10, 2012.

1.5 Personnel

The primary personnel responsible for performing the scope of work stated above are:

RJH Consultants:

Robert J. Huzjak, P.E.	Project Manager
A. Tom MacDougall, P.E.	Project Engineer



Daniel J. Brauer, P.E.	Hydraulic Structures Engineer
Tracy E. Owen, E.I.	Hydrologic/Hydraulic Engineer
James A. Olsen, P.E.	Geotechnical Engineer
Adam B. Prochaska, Ph.D., P.E.	Geotechnical/Geological Engineer

Wenck:

Jack Meena, P.E.	Hydrologic Engineer
------------------	---------------------

Additional RJH staff that provided technical review and consulting on special elements of the Project include:

Danny K. McCook, P.E.	Geotechnical Engineer
BTA Sagar, Ph.D.	Gates

SECTION 2 - GENERAL PROJECT DESCRIPTION

2.1 Project Goals, Background, and Objectives

The Project is part of an overall strategy to implement certain aspects of the U.S. Fish and Wildlife Service's (FWS) recovery plans for four species that are listed as threatened or endangered. The four species are:

- Interior Least Tern
- Whooping Crane
- Piping Plover
- Pallid Sturgeon

Program water activities will be designed to provide water capable of improving the occurrence of Platte River flows in the Central Platte associated habitats.

The first two phases of an on-going Water Management Study (WMS) were completed in 2008 and evaluated numerous scenarios to improve the occurrence of Platte River flows. Based on study recommendations, the Program selected to perform a feasibility study for the J-2 Regulating Reservoir (Areas 1 and 2). Olsson performed a feasibility study and developed a concept for the J-2 Reservoirs, which is presented in Olsson, 2012. The J-2 Project is intended to achieve the following objectives:

- Provide routine supplemental flows to the Platte River.
- Provide periodic short-duration high flows (SDHF) to the Platte River.
- Mitigate hydrocycling impacts and allow for more regulated discharge to the Platte River and the irrigators.
- Improve CNPPID's hydropower generation by enabling generation at optimal flows and during periods of high electricity demand.

The RJH concept updated the Olsson concept to address key safety, maintenance, and operational issues. The RJH concept is presented in this report.

2.2 Overview of Project Concept

The Project would be located in Gosper and Phelps Counties, Nebraska as shown on Figure 2.1. The RJH concept, which is similar to the previous concept, is a two-reservoir

system situated adjacent to, and north of Phelps Canal, south of the Platte River, east of Plum Creek, and west of Road B, which is near the county line between Gosper and Phelps County as shown on Figure 2.2. A discussion of RJH's consideration of the reservoir site is in Section 3.

The Area 1 Reservoir would occupy about 630 acres and the Area 2 Reservoir would occupy about 300 acres. The two reservoirs would be filled primarily by discharge water from CNPPID's J-2 hydropower plant and conveyed to the reservoirs through the Supply Canal and Phelps Canal. The Area 2 Reservoir would regulate water to be released to either the Platte River or back to Phelps Canal for downstream irrigators. The Area 1 Reservoir would store water to be released to the Platte River. A description of the reservoir concept is in Section 6.

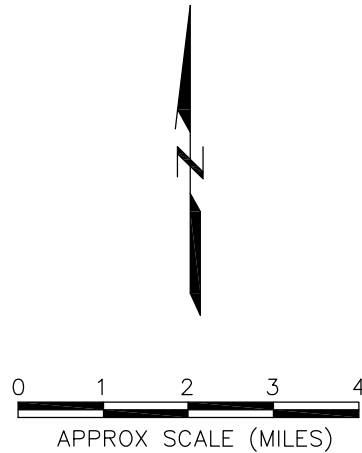
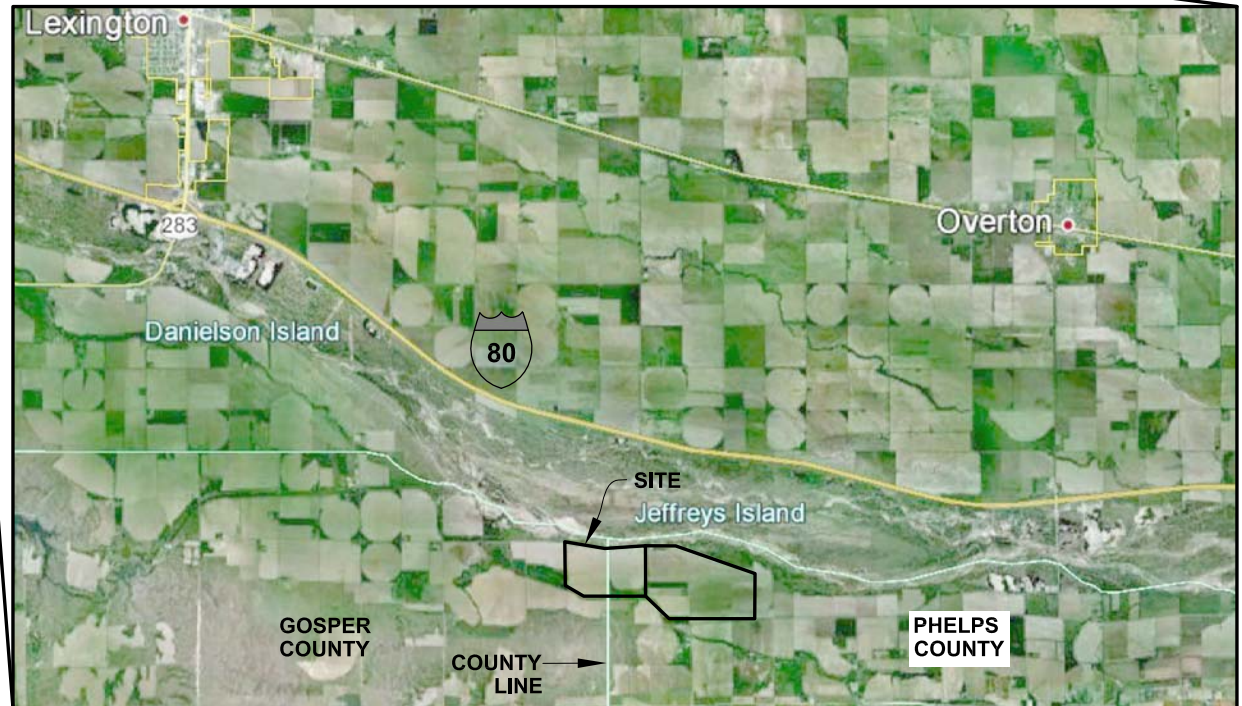
Currently, and based on discussions with CNPPID, the existing Supply Canal can safely convey at least 1,675 cubic feet per second (cfs) from the J-2 hydropower plant to the existing J-2 Return with about 2 feet of freeboard (refer to Figure 2.2). Downstream of the J-2 Return, Phelps Canal is sized to convey about 1,350 cfs with 2 feet of freeboard. The proposed intakes to the Area 2 and Area 1 reservoirs would be located about 3.0 and 3.2 miles downstream of the J-2 Return. Some improvements are needed along Phelps Canal downstream of the J-2 Return to enable conveyance of 1,675 cfs to the reservoirs. A description of the concepts to improve the canal is in Section 8.

2.3 Operational Requirements

To meet Project objectives, the concept for the modified canal and new reservoirs need to provide the following:

- Provide a minimum of 12,000 acre-feet (ac-ft) of storage; more storage would be preferred.
- Release routine flows of 0 to 500 cfs from either of the proposed reservoirs to the Platte River (target flows).
- Release 2,000 cfs to the Platte River for 3 days, a total of 11,900 ac-ft, with no inflow from Phelps Canal (SDHF). This could be a combined release from both reservoirs.
- Convey up to 1,675 cfs into the Area 1 or Area 2 Reservoir from Phelps Canal with 2 feet of differential head between Phelps Canal and the reservoirs.
- Convey at least 1,000 cfs to Phelps Canal from the Area 2 Reservoir with 1 foot of differential head.

These operational criteria are depicted on Figure 2.3.



THIS DRAWING MUST BE REPRODUCED IN COLOR.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

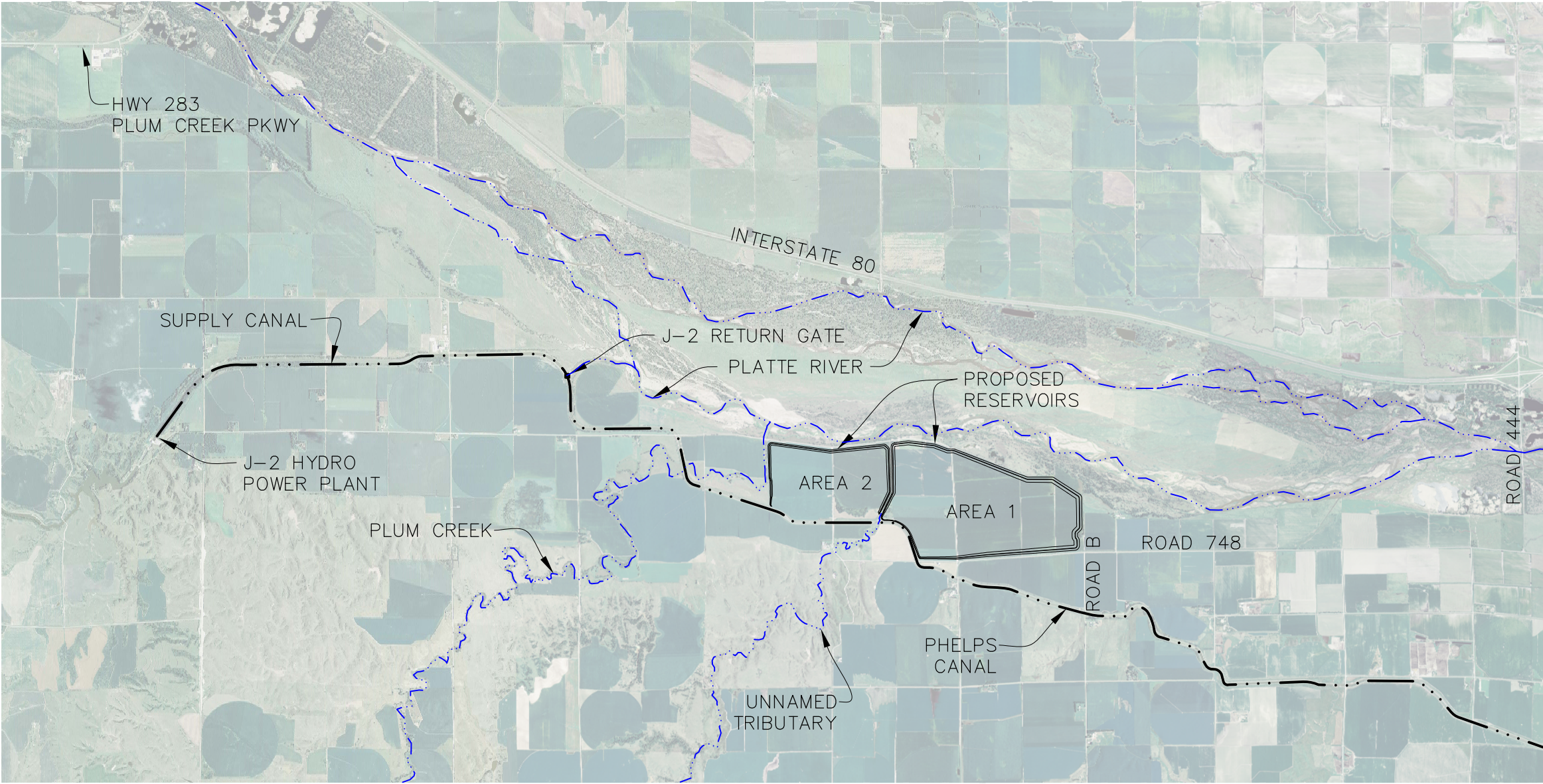
SITE VICINITY MAP

PROJECT NO. 12116

April 2013

Figure 2.1

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_2-2-LOCATION_MAP.DWG 4/10/2013 1:29 PM



THIS DRAWING MUST BE REPRODUCED IN COLOR.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

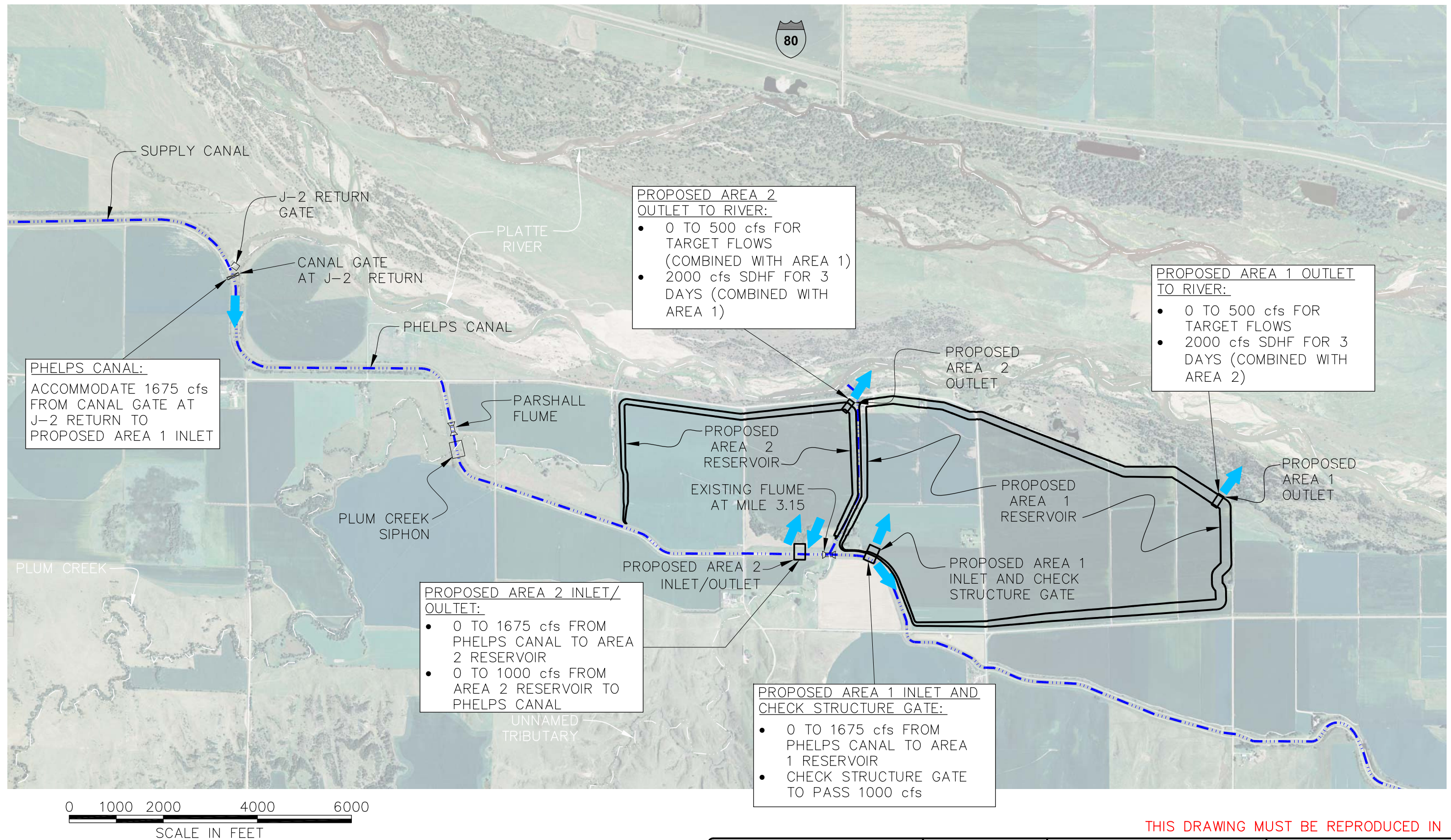
LOCATION OF PROPOSED
J-2 RESERVOIRS

PROJECT NO. 12116

April 2013

Figure 2.2

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_2-3-FLOW_DIAGRAM.DWG 4/10/2013 9:36 AM



THIS DRAWING MUST BE REPRODUCED IN COLOR.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

OPERATIONAL FLOW
CRITERIA

PROJECT NO. 12116

April 2013

Figure 2.3

SECTION 3 - RESERVOIR SITING

3.1 General

RJH evaluated the Project location selected previously to identify potential issues and to select an appropriate location for this concept design. RJH based our evaluation on the considerations presented in the following sections. Additional information is presented in a memorandum in the Project Notebook.

3.2 Operations and Water Availability

Phelps Canal provides a relatively low-cost and abundant source for water that could be conveyed to the reservoirs by gravity. To convey water into the reservoirs and to the Platte River by gravity, the reservoirs would need to be north (downhill) of Phelps Canal and south of the Platte River. If Platte River water or groundwater were the source for a storage project, significant pumping and/or diversions would be needed and would likely result in a considerably more expensive Project concept.

3.3 Geology

The regional geology consists of alluvium or eolian soils overlying the Ogallala Formation. The alluvial and eolian soils are predominantly sandy, but also contain gravels, clays, and silts. For a reservoir project, low-permeable clayey soils are typically preferred to reduce seepage losses. The general geologic conditions are relatively consistent for potential reservoir sites between the Platte River and Phelps Canal and any site is expected to have similar issues to address in design. Therefore, site geology and subsurface conditions were not considered a significant factor in site selection.

3.4 Hydrology

The identified site for the reservoirs is within the Federal Emergency Management Agency's (FEMA) mapped 100-year floodplain for Plum Creek, the unnamed tributary, and the Platte River. It may be preferable to locate the reservoirs in an area with fewer intersecting drainages; however, it appears that most sites within a few miles to the east would have similar issues.

3.5 Topography

The topography is relatively flat and similar between potential Project sites. Therefore, topography, like geology, was not considered a significant factor in site selection. A consequence of the relatively flat site is that any reservoir would require a long ring dam.

3.6 Infrastructure

RJH considered that locating the site as close to the existing J-2 Return gate as feasible would reduce the length of required canal improvements downstream of the existing J-2 Return. Downstream of the J-2 Return, Phelps Canal would need to be improved to convey 1,675 cfs. Additionally, locating the reservoirs near the Platte River would reduce the need for large discharge conveyances that would require land acquisition or easements (see land availability section). The selected site is relatively favorable considering both the distance from the existing J-2 Return and the distance between the proposed reservoirs and the Platte River.

3.7 Environmental and Cultural

Most potential areas between Phelps Canal and the Platte River have been and are currently being cultivated. RJH expected that most locations between the Platte River and Phelps Canal would potentially disturb similar cultural or environmental resources. Additional evaluation of these parameters is needed, but because most potential reservoir locations include farmed land, cultural or environmental issues are not expected to be a significant factor in site selection.

3.8 Land Availability

Given the relatively flat topography, the sizes of proposed reservoirs would be similar and require a similar footprint of land if located between Phelps Canal and the Platte River. It is RJH's general experience that when fewer parcels are needed and fewer structures are impacted, the likelihood of successfully obtaining needed property increases and land acquisition costs decrease. The parcels are larger and fewer structures would be impacted at the identified site relative to other possible sites to the east.

3.9 Conclusions for Site Selection

Based on our evaluation, the primary issues in selection of a reservoir site for this Project are land availability and infrastructure. The current location of the reservoirs is favorable

relative to other sites to the east because there are relatively few parcels that would need to be acquired and fewer structures impacted. The selected site is both close to the existing J-2 Return and the Platte River and would therefore require less infrastructure than other sites to achieve Project goals.

Based on the factors above, RJH did not identify any other sites that appear to be preferred to the existing site and concluded that the current site should be used for this conceptual design.

SECTION 4 - EXISTING CONDITIONS

4.1 General

RJH compiled existing data and performed analyses to better understand and define the existing site conditions. Existing land uses, hydrologic conditions, and geotechnical conditions are expected to have the most significant impact on developing the Project. RJH's understanding of each key condition is presented in this section. Additional information and calculations relied upon to understand the existing conditions are included in the Project Notebook.

4.2 Existing Land Use

The site proposed for the Project is primarily used for growing crops. The land also contains a few residential structures, gravel-paved public and private roads, a marshy area, and a small cemetery. Phelps Canal borders the site to the south and Road 749 borders the site to the north. North of Road 749, there are a few residences that would remain following Project development. The Area 1 Reservoir would be between an unnamed tributary on the west and Road B on the east. At the southeast corner of the proposed Area 1 Reservoir there is a small cemetery that RJH assumed would remain following construction of the Project. Area 2 Reservoir would be between the unnamed tributary on the east and Plum Creek on the west. Portions of Road A and Road 438 would be within the proposed reservoir footprints. An aerial photo of the proposed site is shown on Figure 4.1.

4.3 Floodplain and Hydrologic Conditions

RJH evaluated the hydrologic conditions anticipated for the site, which is located in the mapped floodplain of three large drainages: the Platte River, Plum Creek, and the unnamed tributary. The Project site and the approximate limits of the 100-year floodplain of these three drainages based on FEMA maps are shown on Figure 4.2. The proposed reservoirs would not impound stream flows, but flow from storm run-off within the adjacent drainages would be diverted along the downstream toe of the dams and this flow could potentially erode the dams during high flow events. A summary of our hydrologic evaluation for each drainage is provided in the following sections.

4.3.1 Platte River

RJH evaluated the 100-year flow in the Platte River. Based on current FEMA maps, the proposed reservoirs would be within a “Zone A” floodplain. Zone A means that the limits of the mapped floodplain are not based on detailed analyses and are approximate. RJH used the following methods to evaluate possible floodplain impacts to Project development.

- Regression analysis using USGS gage data for the Platte River (USGS, 2012a).
- RJH HEC-RAS analysis estimating a 100-year discharge value from inundation limits shown on Zone A (approximate) FEMA floodplain maps (FEMA, 2008 and 2011a).
- USGS peak-flow frequency estimates from USGS gage data for the current, regulated condition of the Platte River in Nebraska (USGS, 1999a).
- FEMA Flood Insurance Study (FIS) reports for Dawson and Kearney Counties (FEMA, 1984 and 2011b).
- The Program’s flow exceedance probability curve that is based on USGS gage data for the Platte River near Overton, Nebraska (PRRIP, 2009).

The computed 100-year flow, based on the regression analysis of the Platte River USGS gage data since 1915, was estimated to be about 43,000 cfs (USGS, 2012b). Based on the figure developed by the USGS, the computed 100-year discharge for the current regulated condition of the Platte River, which is based on USGS flow data after Kingsley Dam was constructed in 1941, is estimated to be about 30,000 cfs (USGS, 1999a). The 100-year flow from an existing FEMA FIS, upstream and downstream of the J-2 site was about 34,000 and 32,200 cfs, respectively (FEMA, 1984 and 2011b). According to RJH’s HEC-RAS model, the Platte River flow would need to be about 120,000 cfs for the flow limits to match the current approximate FEMA 100-year floodplain limits (FEMA, 2008 and 2011a). The Program computed a 100-year flow of approximately 42,000 cfs (PRRIP, 2009). It is probable that the actual 100-year flow is in the range of 32,000 to 42,000 cfs. RJH used the HEC-RAS model and a flow of 42,000 cfs and concluded that for this condition the proposed reservoirs would not be within the floodplain of the Platte River.

To be conservative at this stage of design development, RJH used the higher flows that were developed based on the FEMA 100-year floodplain maps (120,000 cfs) to evaluate possible impacts to the 100-year floodplain limits from construction of the Project. Based on this conservative model, we concluded that changes to the FEMA floodplain

limits would be insignificant. The theoretical rise in flood water elevation would be less than 0.3 foot and only impact small areas of undeveloped property. Interstate 80 would not be impacted.

Independent of the actual limits of the 100-year floodplain, the proposed reservoirs are within the mapped FEMA floodplain for these drainages, and therefore a comprehensive study would be needed in subsequent phases of the Project to develop this Project within the mapped floodplain.

For conceptual design, RJH considered that the 100-year flood in the Platte River would be an appropriate storm event to consider as the basis of design instead of the probable maximum flood (PMF) because the PMF flood would likely be so large that if the reservoirs failed, the increase in downstream damage caused by the failed reservoirs would be negligible. However, this preliminary conclusion and potential Project risks inherent in selecting design criteria will need to be further evaluated. Costs to protect the embankments from failure during extremely remote events such as the PMF would likely be significant. Prior to, or early in the next phase of design, a consideration of risk and an appropriate design storm event needs to be confirmed.

4.3.2 Plum Creek

RJH performed a preliminary evaluation to estimate the 100-year flow in Plum Creek, which has about a 200 square mile drainage basin. Based on a regression equation developed by USGS (USGS, 1999b, the 100-year discharge of Plum Creek at the J-2 Project site was estimated to be about 7,000 cfs. However, this estimate may be unreliable because the USGS regression equation may not appropriately address this large drainage basin area.

RJH also performed a Log Pearson Type III Distribution analysis using Plum Creek gage data from a gage located on Plum Creek near Smithfield, Nebraska (USGS, 2012a). The 100-year discharge of Plum Creek at the J-2 site was estimated to be about 2,700 cfs.

Based the results of these two analysis methods, it is probable that the 100-year discharge for Plum Creek at the J-2 site would be between about 2,500 and 7,000 cfs. This uncertainty in the possible flow did not have a significant impact on the current concept, which is to protect the dam from scour and erosion. However, it will ultimately be important to have a reliable estimate of the 100-year flow in Plum Creek during future stages of design development.

4.3.3 Unnamed Tributary

RJH performed a preliminary evaluation to estimate the 100-year flow and PMF flow in the unnamed tributary, which has about a 6.9 square mile drainage basin. RJH used a regression equation developed by the University of Nebraska, Lincoln, for the Nebraska Department of Roads (NDOR) to estimate the 100-year flow (NDOR, 2005). Using drainage basin characteristics and the regression equation developed for NDOR, the 100-year discharge was calculated to be about 2,500 cfs. RJH evaluated the PMF flow in the unnamed tributary using the HEC-HMS program and precipitation data obtained from the Site-Specific Probable Maximum Precipitation (PMP) Study for Nebraska (Applied Weather Associates, 2008). The resulting PMF peak discharge was estimated to be about 4,000 cfs. RJH selected to design for the PMF flow of 4,000 cfs in the unnamed tributary because:

- The geometry of the channel between the reservoirs will be controlled by civil layout requirements.
- It is more probable to have a PMF on a small drainage basin.
- This flow will not noticeably impact the overall cost of the Project.
- To be conservative at this stage of Project development.

4.4 Subsurface and Groundwater Conditions

4.4.1 Site Geology

Based on published geologic maps for Project areas, the geology at the location of the reservoirs consists of Quaternary-age (less than 2 million years old) soil overlying Tertiary-age (2 to 65 million years old) bedrock. The soil at the site generally consists of eolian loess and sand overlying alluvial clay, silt, sand, and gravel (Dreeszen et.al., 1973). Bedrock at the site consists of the Ogallala Formation, which is mostly fine- to medium-grained sandstone and clayey or sandy siltstone with scattered lenses of coarse sand and gravel (Richmond, 1994). The top of the Ogallala Formation is estimated to be approximately 30 to 40 feet below the ground surface on the north side of the site and the depth to bedrock increases to about 100 feet south of the site. The Ogallala Formation averages about 300 feet thick and is a regional aquifer with a relatively high permeability. (Schreurs and Rainwater, 1956).

4.4.2 Geotechnical Conditions

Based on Olsson's geotechnical data, the subsurface soils at the site consist of interbedded layers of sand, gravelly sands, clayey and silty sands, and sandy clays. The surficial soils in the upper 3 to 15 feet consist predominately of low-permeability soils. RJH considered soils with more than about 30 percent fines (fines are soil particles that pass the No. 200 sieve in a laboratory gradation test) to be low-permeability. RJH identified these soils as clayey foundation soils. Below the surficial low-permeability layer, a relatively clean sand or gravelly sand (less than about 10 percent fines) layer extends for tens of feet. The clean sand layer is much more permeable than the overlying clayey materials. RJH identified these soils as sandy foundation soils.

Figure 4.3 shows estimated thicknesses of the surface layer of clayey soils based on the Olsson 2012 borehole data. As shown on Figure 4.3, the layer of clayey soils is generally thinner (less than 4 feet) at the north and northeast parts of the site and thicker (up to 18 feet) at the south part of the site. The portion of Area 1 west of Road A was not explored. RJH extrapolated the geotechnical conditions identified in Area 2 and Area 1 to prepare Figure 4.3 and for use in this conceptual design. However, there is significant uncertainty in the geotechnical conditions in this portion of the Area 1 Reservoir.

RJH evaluated collapse potential of clayey foundation soils using available on-site data and two qualitative U.S. Bureau of Reclamation (USBR) procedures. Based on the results of our analysis, the overwhelming majority of clayey foundation soils are not predicted to have collapse potential. In addition, there does not appear to be a spatial pattern to the areas with collapse potential. In our opinion, the magnitude of potential collapse should not be considered to be a significant design issue. However, additional data is required to evaluate collapse and settlement potential in future stages of design to support design of facilities that can accommodate this movement.

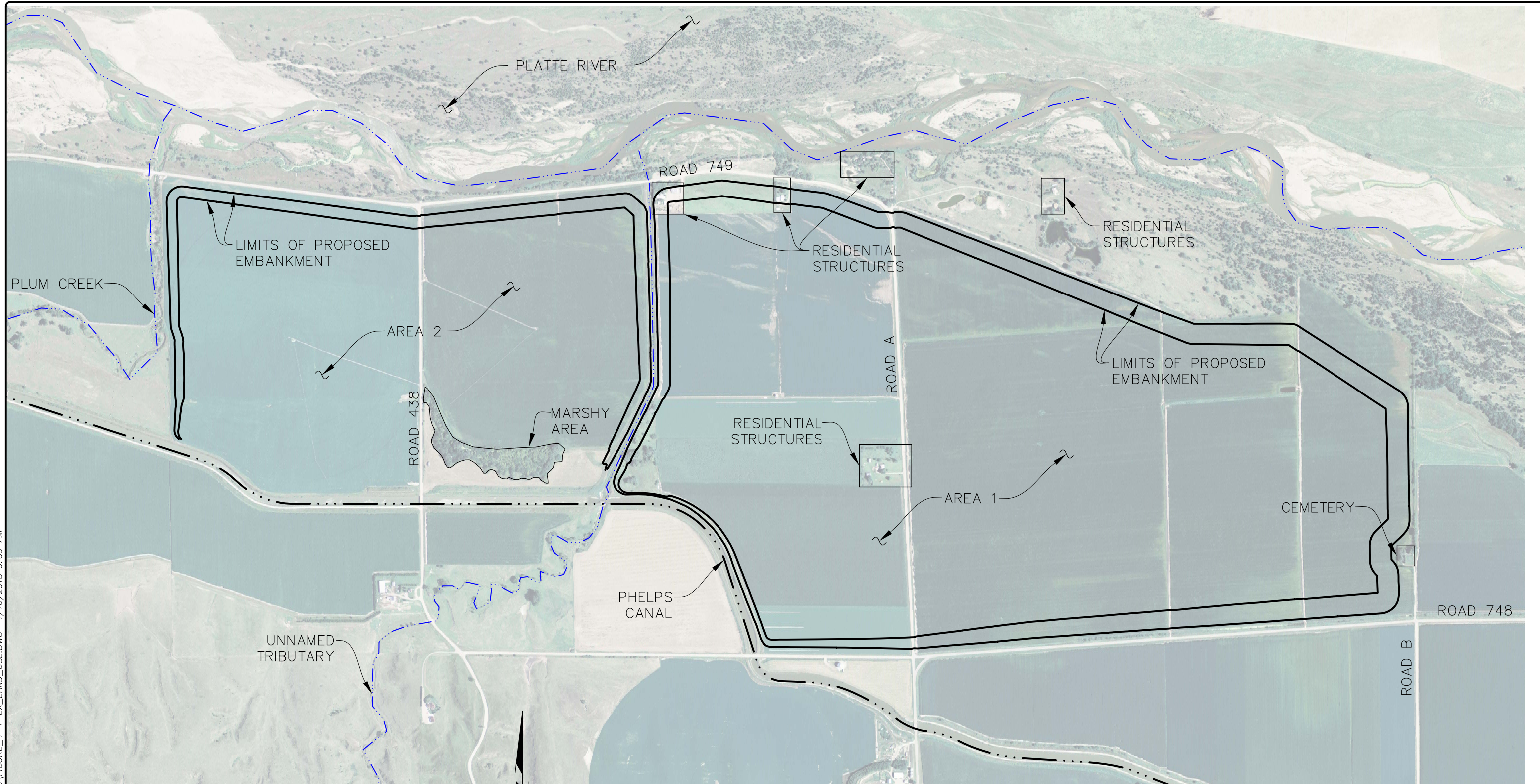
4.4.3 Groundwater Data

RJH developed a contour map of the likely groundwater surface based on groundwater data collected by USGS and Olsson. Groundwater at the site fluctuates over time and the elevation contours shown on Figure 4.4 depict the approximate groundwater conditions in the spring of 2010. RJH selected this time because both USGS and Olsson data was available from various locations across the site. According to the groundwater map, the elevation of the top of the groundwater decreases from the west-southwest to the east-northeast at an approximate gradient of about 0.2 percent. Along the west and southwest sides of the site, the groundwater is typically 10 feet or more below the ground surface.

In the east and northeast of each reservoir area, the groundwater is about 4 feet below the ground surface. Based on historical records of groundwater in the area between 1998 and 2012, the regional groundwater surface appears to fluctuate by about 4 to 7 feet. RJH identified one well that fluctuated 16 feet (Well P-110²), but it is located adjacent to a residence and is likely being influenced by a domestic well. A portion of the available time history of groundwater data is presented in a report of a pilot-scale recharge study (EA, 2012) and additional groundwater data is available at www.nwis.waterdata.usgu.gov/nwis.

² USGS Well No. 404040099383501.

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_4-1-EX_LAND_USE.DWG 4/10/2013 9:59 AM



THIS DRAWING MUST BE REPRODUCED IN COLOR.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

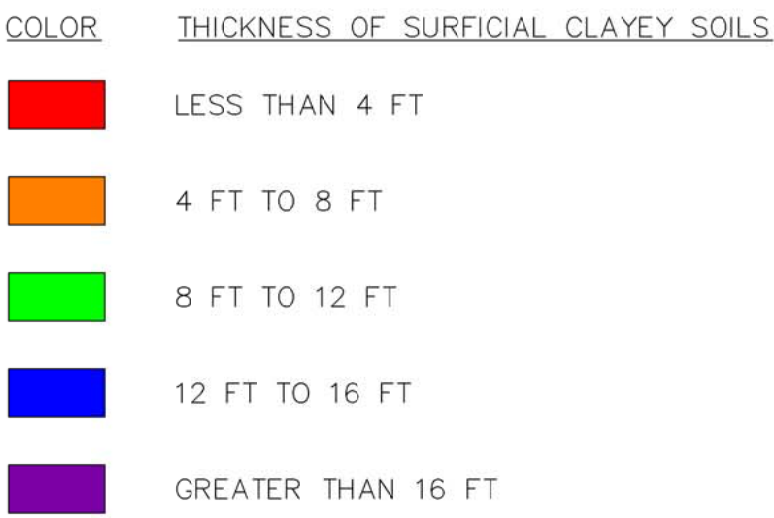
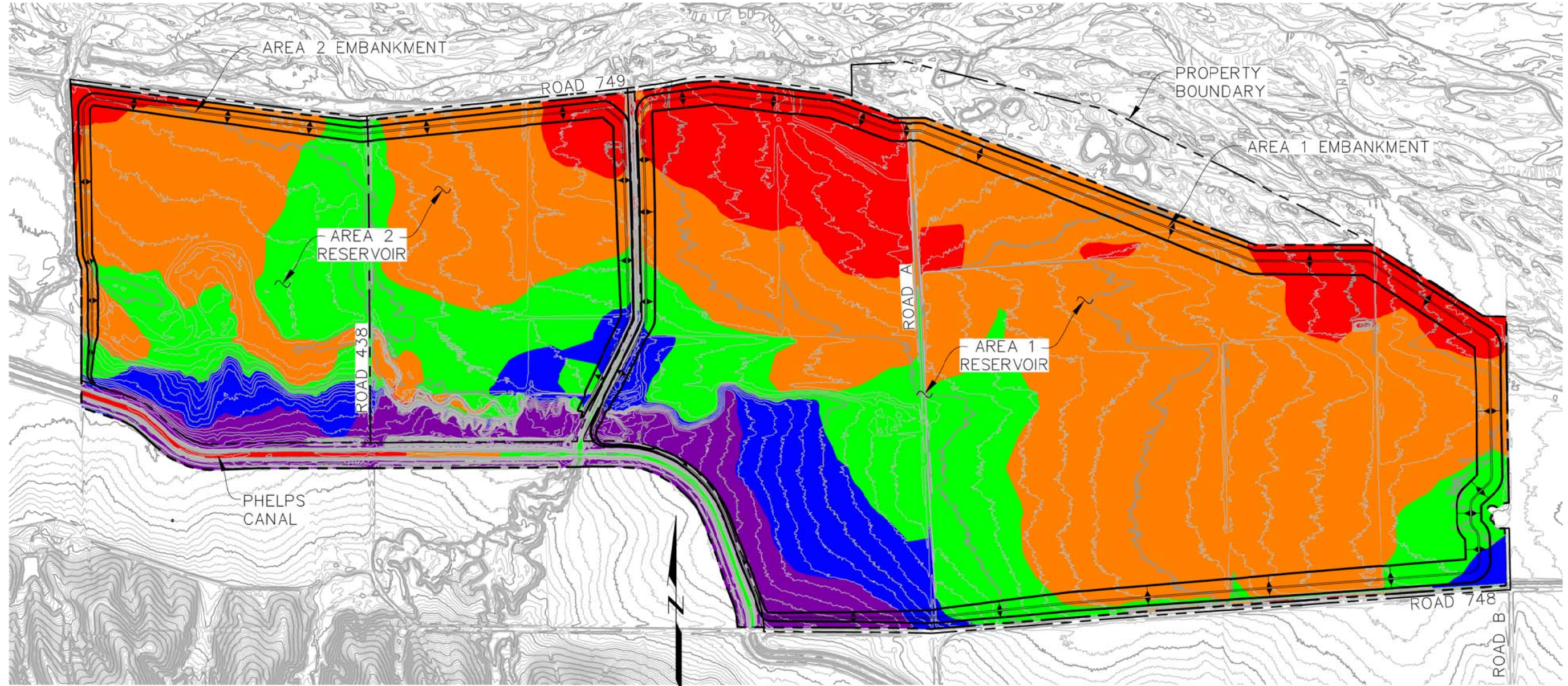
PROJECT NO. 12116

EXISTING LAND USE



April 2013

Figure 4.1

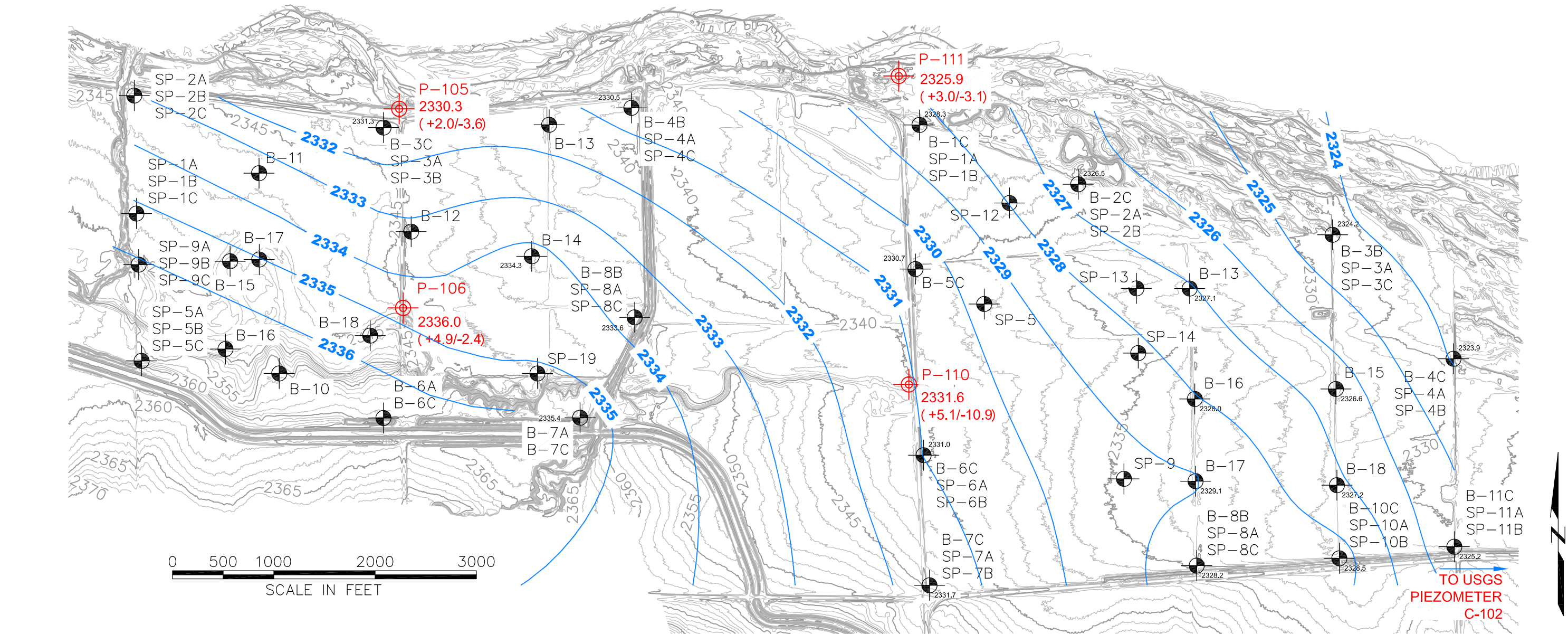
P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_4-3-CLAY_THICKNESS.DWG 4/10/2013 10:24 AM



THIS DRAWING MUST BE REPRODUCED IN COLOR.

		J-2 REGULATING RESERVOIR PROJECT GOSPER AND PHELPS COUNTIES, NEBRASKA	THICKNESS OF CLAYEY SOILS AT SITE	
		PROJECT NO. 12116	April 2013	Figure 4.3

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_4-4-GROUNDWATER_PLAN.DWG 4/10/2013 10:32 AM



EXPLANATION

- B-11C, SP-11A, SP-11B: LOCATIONS OF EXPLORATION BOREHOLE BY OLSSON (2012).
- OLSSON BOREHOLE LABELS
- OLSSON GROUNDWATER ELEVATION (NOTE 5)
- P-111: LOCATION OF USGS MONITORING WELL (NOTE 1)
- 2325.9: USGS GROUNDWATER ELEVATION (NOTE 3)
- (+3.0/-3.1): VARIATION IN GROUNDWATER ELEVATION (NOTE 4)
- 2330: APPROXIMATE GROUNDWATER CONTOUR (NOTE 2)
- 2340: GROUND SURFACE ELEVATION CONTOUR

NOTES:

- USGS MONITORING WELL DATA IS PRESENTED ON SEMI-ANNUAL FREQUENCY.
- GROUNDWATER ELEVATION CONTOURS ARE APPROXIMATE AND REPRESENT THE GENERALIZED GROUNDWATER CONDITIONS DURING SPRING 2010.
- USGS GROUNDWATER ELEVATIONS FROM APRIL 16, 2010.
- THE (+/-) DATA REPRESENTS THE HIGHEST AND LOWEST RECORDED ELEVATIONS REFERENCED TO THE APRIL 16, 2010 ELEVATION DESCRIBED IN NOTE 3. SEE TIME HISTORY DATA PLOTS IN APPENDIX F OF THE PILOT-SCALE RECHARGE REPORT (EA ENGINEERING, 2012)
- GROUNDWATER ELEVATION COMPUTED FROM OLSSON DATA (TYP)(APPROX). OLSSON GROUNDWATER DEPTHS WERE RECORDED BETWEEN 3-27-2010 AND 3-30-2010. COMPUTED GROUNDWATER ELEVATIONS ARE BASED ON ESTIMATED GROUND SURFACE ELEVATIONS AND ARE APPROXIMATE.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

GROUNDWATER
CONDITIONS

PROJECT NO. 12116

April 2013

Figure 4.4

THIS DRAWING MUST BE REPRODUCED IN COLOR.

SECTION 5 - PRIMARY TECHNICAL ISSUES AND DESIGN OVERVIEW

5.1 Identified Design Issues

Given the Project goals and operational requirements (as presented in Section 2); the site location (as presented in Section 3); and the existing hydrologic, subsurface, and other conditions (as presented in Section 4), RJH identified primary issues that significantly impacted the development of the concept design for the Project. The primary issues included:

- The lack of a continuous and reliably thick low-permeability soil unit in the foundation. This will impact the ability to retain water and safely manage seepage losses.
- The presence of variable and interlayered sandy materials in the foundation. Sandy seams and layers could allow for reservoir seepage to exit uncontrolled to the ground surface (including to the sides and bottoms of the Platte River and other drainage features). If uncontrolled seepage could exit through sandy foundation soils, the foundation could become unstable and allow for erosion. This phenomenon is typically referred to as “piping.” Provisions to mitigate the potential for piping of the foundation will be required.
- The relatively shallow groundwater below much of the site. The shallow groundwater will impact the location of the bottom of the reservoir and excavation for borrow materials.
- The planned operation of the reservoir will require the reservoir level to routinely fluctuate from full to empty and for some operational conditions the reservoir will be lowered from full to near empty in about 3 days. This will expose the entire upstream slope to the effects of wave erosion and the embankment to routine extreme drawdown rates.
- The need to construct the embankment and slope protection using on-site materials to manage Project costs.
- Hydraulic facilities will be required to convey both small flows, which will be on the order of 0 to 500 cfs, and large flows, which will be on the order of 2,000 cfs, with reasonable control and accuracy.
- The exterior slopes of the embankments will be subject to flows from the Platte River, Plum Creek, and the unnamed tributary. The flows from Plum Creek

would impact the west embankment for Area 2 at about a 90 degree angle and the flows from the unnamed tributary would flow between the two reservoirs.

- The upper 3 feet of the Area 2 Reservoir needs to be operated to convey water back into Phelps Canal.
- Phelps Canal downstream of the J-2 turnout needs to convey 1,675 cfs without inundating crossing bridges or creating a significant risk of breakout.

5.2 Overview of Design Concepts

The primary Project components needed to address these primary issues are illustrated on Figures 6.1 and 6.2 and include:

- A seepage management system. RJH selected a clay-lined reservoir to manage seepage.
- Sloped reservoir bottoms.
- Relatively shallow borrow excavations.
- Zoned earthen embankments for the Project dams.
- Complete upstream slope protection for the dams.
- Multi-gate hydraulic structures.
- Protection against erosion for portions of the downstream embankments subject to stream flows.
- Modifications to Phelps Canal.
- A two-reservoir concept to achieve the needed storage and facilitate operation and maintenance.

A general description of each primary component is described in the following sections. More detailed discussions regarding the design considerations and supporting analyses are presented in Sections 6 (Embankment and Reservoir Concepts), 7 (Reservoir Hydraulic Structures), and 8 (Canal and Creek Modifications).

5.2.1 Seepage Management System

Generally three primary methods are available to manage seepage from a reservoir:

- Line the reservoir to reduce seepage losses.

- Construct a deep cutoff around the perimeter to a consistent low permeable layer.
- Allow seepage to exit the reservoir and collect and manage the seepage with a downstream seepage collection system.

RJH evaluated both a reservoir liner and a downstream seepage collection system. A deep cutoff wall was not evaluated because, based on published geology, a consistent low-permeable unit is not present below the site within the upper several hundred feet. RJH selected the reservoir liner concept based on cost and technical reliability. Additional information on the downstream seepage management system is provided in Section 10.

The reservoir liner concept includes a 1.5-foot-thick liner constructed using the clayey soils at the site protected with 3 feet of cover soil. The cover soil is required to protect the liner from the long-term impacts from the environment such as freeze-thaw and wet-dry cycles, burrowing animals, vegetation, etc. The liner would be generally above the average estimated groundwater elevation to reduce dewatering required for construction and uplift pressure associated with fluctuations in groundwater elevation.

5.2.2 Sloped Reservoir Bottom

A sloped reservoir bottom provides two primary benefits:

- It reduces the potential for uplift of the reservoir liner because the sloped reservoir bottom maintains the liner above groundwater levels.
- It maintains reservoir head on the river outlet gates for a significant portion of the total storage, which results in smaller-sized gates and outlet structures than if the reservoirs had flat bottoms.

5.2.3 Shallow Borrow Areas

If significant borrow below the groundwater level was required, the dewatering costs would likely become a significant factor in overall Project costs. Except for localized borrow for sandy soils that will be needed for soil-cement slope protection and select parts of the embankment, the liner fill, cover fill, and much of the embankment fill materials are available above the anticipated groundwater levels.

5.2.4 Zoned Embankment

The concept includes zoned embankment dams that would be primarily constructed from the soils available from within the reservoir basin. The embankments would include:

- An upstream zone of primarily sandy soils to address concerns related to frequent rapid fluctuations in the reservoir pool.
- A central zone comprised of clayey soils connected to the liner to reduce seepage losses.
- A filter sand/chimney drain located downstream of the clayey central zone to safely manage seepage comprised of specifically-graded sand obtained by processing the on-site materials.
- A downstream zone of random fill that would be obtained from the on-site materials.

5.2.5 Embankment Slope Protection

The entire upstream slope would be covered with soil-cement to provide protection from wave erosion. Soil-cement was selected based on performance history and economics. Soil-cement has over 50 years of successful performance for reservoir slope protection and it can be manufactured using on-site sandy soils. Other possible alternatives such as riprap, cellular concrete mats, etc. were dismissed based on economic considerations.

5.2.6 Hydraulic Facilities

The concept includes inlet and outlet structures equipped with at least two gates. At least two gates were used to provide accurate hydraulic regulation of flow. Armored discharge channels will also be used to safely convey discharges to the Platte River channel.

5.2.7 Exterior Embankment Protection

An engineered grass-lined channel with a small concrete low-flow channel will be used to safely convey routine flows from the unnamed tributary between the two reservoirs. Drop structures would be included in the unnamed tributary channel to maintain the flow velocity during the PMF to be compatible with grass-covered embankment slopes. The potential for undermining and eroding the embankment where Plum Creek would turn and flow parallel the west side of the Area 2 dam would be mitigated with a buried sheetpile wall and soil-cement armoring on the lower part of the exterior embankment

slope. This will provide protection for at least the 100-year event. The computed maximum velocity in the Platte River, for the most conservative condition, 120,000 cfs, (refer to Section 4.3.1) during the 100-year event is less than 2 feet per second (fps). Therefore, erosion potential can be mitigated with a grass-covered embankment.

5.2.8 Phelps Canal Modifications

Modifications to parts of Phelps Canal between the J-2 Return and the Area 1 intake include:

- Minor placement of fill to provide 1 foot of freeboard in the canal.
- Raising three existing bridges over the canal.
- Adding a new siphon parallel to the existing siphon below Plum Creek.
- Raising the walls of the existing flume over the unnamed tributary.
- Adding a check structure in the canal as part of the Area 1 intake structure. This will enable a consistent MNWS in the Area 1 and Area 2 reservoirs at El. 2356.0. The dam crest would be at El. 2360.0 to provide 4 feet of freeboard for wave action.

5.2.9 Two-Reservoir Concept

A two-reservoir concept would achieve storage requirements and facilitate operations and maintenance. Approximately 840 ac-ft of storage in the upper 3 feet of the Area 2 Reservoir could be used for regulation of irrigation flows in the canal. The total active reservoir storage for the Project would be about 15,400 ac-ft when both reservoirs are at El. 2356.0.

SECTION 6 - EMBANKMENT AND RESERVOIR CONCEPTS

6.1 General

RJH developed the dam and reservoir concepts based on preliminary analyses and on engineering experience and judgment. Analyses performed are documented in the Project Notebook. In Section 6, we present descriptions and supporting information regarding the dam and reservoir concepts.

In general, the dam and reservoir concepts include two earthen dams that impound two reservoirs as shown on Figure 6.1. The total storage in the two reservoirs would be about 15,400 ac-ft. The reservoirs would be lined with compacted clayey soils to manage seepage safely, be graded to slope toward the outlets (i.e., to fully drain), and would cover a total of about 930 acres of land.

The reservoirs would be impounded by zoned earthen embankment dams that would extend a total length of about 5.7 miles (along the centerline) around most of the reservoir perimeters and abut the natural ground surface near Phelps Canal. The maximum height of the dams would be about 32 feet and 22 feet for Area 1 and Area 2 reservoirs, respectively. The dams would have a zone of low permeability clayey fill that is connected to the clayey reservoir liner and a zone of filter sand downstream of the clayey zone to mitigate internal erosion. The entire upstream face of the dams would consist of soil-cement to protect the embankment from wave erosion. For conceptual design, the upstream slopes would be sloped at a ratio of 4 horizontal (H) to 1 vertical (V) and the downstream slopes would be 3H:1V.

6.2. Reservoir Footprints

The combined reservoir footprints for Area 1 and Area 2 reservoirs would be about 930 acres. This includes about 300 acres for the Area 2 Reservoir and about 630 acres for the Area 1 Reservoir. The footprint of the total site (the limits of acquired property) would be about 1,100 acres.

RJH established the exterior limits for the dams based on assumed property boundaries estimated from available property tax parcels. Where actual property parcel information was not readily available, RJH assumed the property boundary was either the edge of cultivation, or 40 feet from the centerlines of the existing roads, whichever resulted in a smaller reservoir. Once RJH developed approximate property boundaries, RJH created a

“Project Boundary” to represent the area on the property available for possible Project development. Next, RJH established the downstream edge (toes) of the proposed embankments 50 feet inside of the Project boundary to allow space of future facilities such as access roads, monitoring wells, or other infrastructure needed for safe operation and maintenance of the dams. A plan of the two reservoirs is shown on Figure 6.1.

6.3 Embankment Configuration

6.3.1 Zoned Embankment Concept

RJH developed a concept for a zoned embankment based on the anticipated materials available at the site, typical dam design, dam safety practices, and our experience. The embankment concept is illustrated on Figures 6.2 and 6.3 and includes the following primary zones:

- Soil-cement upstream slope protection.
- Gravel drainage layer.
- An upstream sandy zone. This was selected to provide stability during rapid drawdown conditions.
- A central clay core. This will be used to reduce and control seepage through the embankment.
- A filter sand zone (chimney) downstream of the clay core. This will mitigate the potential for piping or internal erosion in the embankment and manage seepage flow through the embankment. The filter sand will extend through the upper clayey foundation soils and terminate in the sandy foundation soils. This will enable any collected seepage to be conveyed into the regional groundwater.
- A downstream random fill zone. This will provide protection to the core and filter zones, and provide stability to the downstream side of the embankment.

The underlying sandy foundation soils are significantly more permeable than the liner and will effectively serve as a drain to collect seepage that passes through the liner. The natural sandy foundation soils are also filter compatible with the proposed liner material.

6.3.2 Material Properties

RJH developed material properties for the existing foundation soils and embankment materials for use in geotechnical analyses performed to support development of the

concept design. Material properties were developed based on the Olsson geotechnical data, published correlations, and experience. Typically, material properties were selected with the intent to be slightly conservative. This approach was used because there are significant data gaps and concerns with the reliability of some of Olsson's geotechnical data. However, RJH concluded that the on-site data when combined with published correlations and experience was sufficient to develop appropriate conceptual-level material properties. The material properties selected for use in analyses are summarized in Table 6.1.

TABLE 6.1
SUMMARY OF GEOTECHNICAL MATERIAL PROPERTIES

Material	Moist Unit Weight, γ (pcf)	Saturated Unit Weight, γ_{sat} (pcf)	Drained Shear Strength		Undrained Shear Strength		Saturated Vertical Hydraulic Conductivity, k_v (cm/sec)	Anisotropic Ratio, k_h/k_v	Saturated Volumetric Water Content, θ_{sat}	Liquid Limit
			Friction Angle, Φ' (degrees)	Cohesive Intercept, c' (psf)	Friction Angle, Φ_T (degrees)	Cohesive Intercept, c_T (psf)				
Sandy Foundation	128	132	32	0	--	--	1×10^{-3}	10	0.35	<16
Clayey Foundation	115	121	29	0	0	700	1×10^{-6}	5	0.45	35
Sandy Embankment Fill	131	135	35	0	--	--	1×10^{-3}	5	0.32	<16
Clayey Embankment Fill	120	125	29	0	0	1,000	5×10^{-7}	9	0.41	35
Filter Sand	131	135	35	0	--	--	1×10^{-2}	2	0.32	<16

6.3.3 External Slopes

RJH performed preliminary slope stability analyses to support selection of the exterior slopes of the reservoir. We developed one representative section that generally represented the maximum height of the embankment, which is about 32 feet (distance between the existing ground and the dam crest). The maximum embankment section would be generally located at the northeast corner of the Area 1 Reservoir. Although slightly different than the soil stratigraphy illustrated on Figure 4.3, we considered that the foundation soils below the embankment consisted of about 8.5 feet of clayey foundation soils to be conservative at this stage of Project development and to account for data gaps. The strength of the clayey foundation soils is critical to the stability of the embankment and additional data is required in future stages of design to confirm the strength of these soils.

Analyses were performed for steady state, end of construction, and rapid drawdown loading conditions. The computed slope stability factors of safety and the recommended minimum factors of safety are presented in Table 6.2.

TABLE 6.2
SUMMARY OF RECOMMENDED AND COMPUTED SLOPE STABILITY

Loading Condition	4H:1V Upstream Slope FS	3H:1V Downstream Slope FS	Recommended Minimum FS
End of Construction	1.71	1.44	1.3
Steady State Seepage, Full Reservoir	2.60	1.69	1.5
Rapid Drawdown	1.42	--	1.4

Except for the rapid drawdown condition, the recommended factors of safety are based on the values typically used for embankment dams. RJH selected a factor of safety of 1.4 for the rapid drawdown condition, which is higher than the commonly-used factor of safety of 1.2. This was selected because rapid drawdown is expected to be a frequent loading condition associated with the SDHF releases that could occur as frequent as once per year.

As illustrated in Table 6.2, downstream slopes would be stable at a ratio of 3H:1V and the upstream slopes would be stable at 4H:1V.

6.4 Reservoir Liner

6.4.1 Liner Concept

The reservoir concept includes a low-permeable clay liner connected to the central clayey core in the embankment to mitigate seepage losses and high-energy seepage from undermining and failing the dam (usually referred to as a “piping” failure). The clay liner overlying sandy foundation conditions provides head loss (or energy loss) as the seepage travels from the reservoir to a downstream exit face. An 18-inch-thick clay liner was selected based on the results of our analysis, constructability, and experience. The liner would be constructed in two lifts. Placing the liner in two lifts would significantly decrease the probability that there would be a defect in the liner because two defective zones would have to be constructed on top of each other. With careful observation of fill during construction, there would be a very low probability of constructing a defective liner. RJH based the design concept on 2-dimensional seepage analyses. RJH used the seepage analyses as a tool to evaluate the need for seepage management facilities in the embankment or foundation.

6.4.2 Seepage Analysis

RJH performed preliminary 2-dimensional seepage analyses to support the concept design of the liner and embankment. Four generalized cross sections were developed to represent the general variation in the topographic and site conditions. The locations of the modeled cross sections are shown on Figure 6.1. The cross section locations are generally described as follows:

- **Area 2 Profile** – This section extended from south of Phelps Canal to the Platte River. The primary purposes of this section were to calibrate material properties and the boundary conditions to observed groundwater levels, evaluate the impact of leakage from Phelps Canal on the liner, and to evaluate changes in predicted groundwater levels below the liner and between the embankment and the Platte River.
- **Area 1 Max Embankment** – This section was located near the maximum embankment height, which is near the northeast corner of Area 1. The primary purposes of this section were to evaluate seepage through the embankment and model potential changes in groundwater levels between the embankment and the Platte River.

- **Area 1 South** – This section was located near the east end of the south side of Area 1. The primary purpose of this section was to evaluate changes in the groundwater levels at the up-gradient side of the reservoir.
- **Unnamed Tributary** – This section was located across the northern part of the unnamed tributary and extended from the Area 1 Reservoir into the Area 2 Reservoir. The primary purposes of this section were to evaluate embankment seepage and changes in the elevation of groundwater between the two reservoirs.

Analyses were performed at each section to evaluate two primary conditions: seepage through an intact liner and seepage through a liner defect. The liner defect was modeled to be 5 feet wide and located at the interior toe of the embankment. The defect was conservatively assumed to extend through the liner and the underlying clayey foundation soils to the sandy foundation soils. Inclusion of a hole in a 2-dimensional model is very conservative because the hole is modeled as a continuous strip (into and out of the cross section) rather than a point. In reality, the effects of seepage from an isolated hole would be less severe than the model results because the seepage would spread in three dimensions. Modeling a liner hole is also conservative because, for a liner constructed in two lifts, it is highly unlikely that such a defect (high permeability fill) would be placed at the same location in both lifts.

Material properties used in the analyses are presented in Table 6.1.

6.4.3 Conclusions from Seepage Analyses

6.4.3.1 General

We concluded the following based on our analyses:

- Uplift of the liner would generally not be a concern.
- Construction of lined reservoirs would likely mound the groundwater south of the Area 1 Reservoir.
- The risk of a piping failure is low for the clay liner concept.
- Construction of the reservoirs would raise the groundwater level along the unnamed tributary.
- The volume of seepage losses from Phelps Canal and the reservoirs was not well defined in this conceptual study. Additional data and evaluation would be needed in subsequent phases of design.

- Based on a very conservative analysis, annual seepage losses could be on the order of 6,000 ac-ft per year if both reservoirs were maintained full for the entire year. The seepage would generally flow in the direction of the regional groundwater gradient, which is northeast. Additional data and more rigorous analysis is needed in subsequent phases of design to refine this estimate.

Additional information related to each of these conclusions is provided in the following sections.

6.4.3.2 Uplift Potential of Reservoir Liner

Uplift of the liner as currently configured would generally not be a concern. Based on recorded groundwater levels, groundwater would need to rise about 5 feet above the high historical levels before uplift would be a significant concern. Uplift could be a concern for the proposed concept if:

- Phelps Canal has significant seepage losses near the reservoirs. This would likely occur if the materials below Phelps Canal are sandy for a significant distance along the canal. This should only be a concern over a localized area and could be mitigated by lining the canal or including a drainage system between the canal and the liner. However, based on our analyses and information provided by CNPPID, we do not consider this to be a significant issue at this time and have not included provisions to mitigate significant canal leakage. Additional data would be required in later stages of design to resolve this item.
- The reservoirs are empty and the Platte River is in flood stage at about 2 feet above the invert of the outlet gates. This potential risk for liner uplift could be mitigated by opening the reservoir outlet gates and allowing the reservoirs to fill with water from the Platte River. This could easily be included in the Standing Operating Procedures for the facility.

6.4.3.3 Groundwater Mounding South of Site

Construction of the reservoir would likely result in raising the regional groundwater level south of the Area 1 Reservoir several feet (often referred to as “mounding”). This would create ponded water in the fields south of the Area 1 Reservoir and adversely impact farming. This would also allow seepage to exit unprotected to an existing drainage ditch, which could allow piping to initiate.

To address these issues, we included a groundwater drain south of Area 1. We assumed that the existing drainage ditch north of Road 748 could be disturbed to construct the groundwater drain. The groundwater drain would consist of a slotted plastic pipe encased in gravel filter material. The invert of the drain would be set to maintain the groundwater at pre-Project levels. The drain would be sloped very gently to daylight to the Platte River north of the site by gravity. Consequently, groundwater could temporarily back up and the drain could be ineffective at lowering the groundwater during periods of high flows in the Platte River. During future stages of design, other alternatives could be evaluated in place of the gravity discharge pipe to the Platte River.

6.4.3.4 Piping Potential

For a concept that includes a continuous and well-constructed liner, the risk for a piping failure to develop would be low. However, if there were a defect in the liner, based on a 2-dimensional model, the groundwater level would increase downstream of the embankment and the risk for piping to develop would increase. Although a defect would not be expected to result in a rapid and catastrophic failure (as previously stated a defect is not likely), provisions need to be included to mitigate this possible failure mode. Therefore, RJH included a series of remotely monitored vibrating wire piezometers between the dam and the Platte River as a safety measure. Data would be collected continuously and conveyed using telemetry. The piezometer trends would need to be monitored to identify if there were changes in seepage pressures (energy) along the downstream side of the dams. If a leak in the liner were to develop, it would likely be detected by an anomalous rise in the piezometer levels and could be addressed well in advance of initiation of piping.

6.4.3.5 Groundwater Mounding in the Unnamed Tributary

Construction of the reservoirs is expected to increase the elevation of groundwater between the Area 1 and Area 2 Reservoirs. A drain would be installed in the unnamed tributary below the invert of an engineered channel to maintain the level of groundwater below the channel, prevent uplift pressures, and mitigate seepage from exiting to the ground surface. The drain would consist of a slotted pipe encased in gravel filter material.

6.4.3.6 Seepage Losses in the Canal

The volume of seepage losses in Phelps Canal and the potential impact from this loss on the Project was not fully resolved at this stage of design. The material along the sides and bottom of the canal is unknown and probably varies at different locations from

sandy to clayey soils. Based on current data and results of our preliminary analyses, there are unresolved conflicts in calibrating our seepage models. Although seepage from Phelps Canal is not expected to cause uplift concerns on the liner, additional data and evaluation is needed to confirm the materials below the canal. The additional data would be used to better evaluate the permeability of the material below the canal and to calibrate seepage models. Better defined material properties and better calibrated seepage models would enable a more reliable evaluation of the post-construction groundwater conditions below and downstream of the canal than is currently feasible based on existing data.

When the vertical permeability presented in Table 6.1 (1×10^{-6} cm/sec) was used for the material below the canal, the zone of saturation from the canal did not extend to the baseline regional groundwater elevation and the groundwater levels in the model generally matched regional levels developed from the 2010 data (see Figure 4.4). The vertical permeability of the materials below the canal needed to be about 1×10^{-4} cm/sec to match the infiltration rates from the pilot recharge test (EA, 2012) and recorded changes to groundwater levels in P-106 of 2 to 4 feet.

6.5 Reservoir Grading

The concept includes grading the bottom of each reservoir to generally slope from the southwest down to the northeast. The concept for reservoir grading is shown on Figure 6.1. RJH selected to slope the reservoir bottoms primarily based on the need to have the reservoir liner above the anticipated groundwater elevation. The reservoir liner needs to be above the groundwater to mitigate the potential to damage the liner from uplift pressures (as discussed in Section 6.4.3.2) and to reduce construction costs.

In addition, the reservoir grading concept was developed considering:

- The preference to borrow construction materials (i.e., fill soils) from within the reservoir area.
- The preference to maintain most of the borrow areas above typical groundwater levels.
- The preference to roughly balance excavation and fill quantities.
- The preference to avoid creating areas of dead storage.

According to preliminary calculations, RJH estimated that for this grading plan there is about 20 percent more soil available than needed to construct the embankment, liner, and liner cover. This excess quantity is less than desired for most large earth dam projects,

which is commonly 50 percent. This estimate should be confirmed following more extensive geotechnical data collection. Balancing earthwork was considered important to avoid expensive import or off-haul of construction materials. As designs are refined, it is likely that slight changes will be needed to maintain a balanced site. These slight changes could impact the storage volume of the reservoirs by a few percent.

Although most of the borrow excavation can be performed above groundwater levels, some excavation in Areas 1 and 2 would likely need to extend below the groundwater level to obtain sufficient granular materials to process for filter sand drain gravel, and for soil-cement (discussed in Section 6.7). Excavations below the groundwater level would require dewatering. Based on available gradation data, RJH anticipates that all granular materials needed for embankment construction could be obtained from on-site borrow excavations.

Almost the entire reservoir could be drained and the entire reservoir storage could be considered active storage (beneficial use). Additionally, the sloped reservoir bottom would maintain a reservoir head at the outlet gates, even during the final hours of the SDHF. By maintaining reservoir head at the outlets, smaller gates would be required to deliver the flows needed for the SDHF release.

6.6 Reservoir Storage

To maximize reservoir storage volume, RJH considered the need to maintain a high water surface elevation in Phelps Canal, which feeds the reservoirs by gravity flow. Refer to Section 8 for information on canal modifications and analysis.

Based on an updated HEC-RAS model of the canal and maintaining a minimum of 1 foot of freeboard in the canal, the maximum normal water surface elevation (NWSE) in both the reservoirs could be at about El. 2356.0. Based on the reservoir grading and embankment concepts presented on Figure 6.1, RJH developed elevation-area-capacity curves for both reservoirs. The elevation-area-capacity curve for each reservoir and for the combined facility is presented on Figure 6.4. The estimated active storage at the NWSE of 2356.0 would be about 12,135 ac-ft for Area 1 and about 3,265 ac-ft for Area 2, resulting in a total combined storage capacity of about 15,400 ac-ft.

6.7 Upstream Slope Protection and Dam Crest Elevation

Based on the embankment configuration shown on Figure 6.1, the minimum crest elevation of both dams would be El. 2360.0. To select the crest elevation RJH calculated

the wave run-up within the reservoirs to select the freeboard (i.e., distance between the NWSE and dam crest). The calculated freeboard was between 3 and 5 feet. The freeboard calculations were performed using historical wind speeds, fetch lengths, and standard U.S. Army Corps of Engineers (USACE) and U.S. Bureau of Reclamation (USBR) procedures for reservoirs. For a rough and uneven slope (i.e., stair-stepped armoring or uneven riprap), the anticipated wave run-up was about 3 feet. For a smooth-paved slope (i.e., plated soil-cement), the calculated wave run-up was about 5 feet. Given the potential wave heights and likelihood that similar sized waves could develop at most water levels in the reservoir, RJH concluded that upstream slope protection should be provided from the bottom of the upstream slope to the dam crest. Slope protection will also mitigate erosion on the upstream slope caused by surface runoff.

RJH selected soil-cement for slope protection. Soil-cement was selected primarily because the materials (except for the cement) are available on-site and the cost would be much less than riprap or other types of slope protection. RJH considered that commonly used concrete rubble broken into the specific sizes needed to avoid multiple costly bedding layers could not be relied upon to be readily available at the time of construction. Based on preliminary research, it appears that stone riprap is not locally available and would require significant cost to transport by rail and by truck to the site. RJH searched for potential sources for stone riprap by reviewing published geologic maps and performing internet searches for nearby rock quarries.

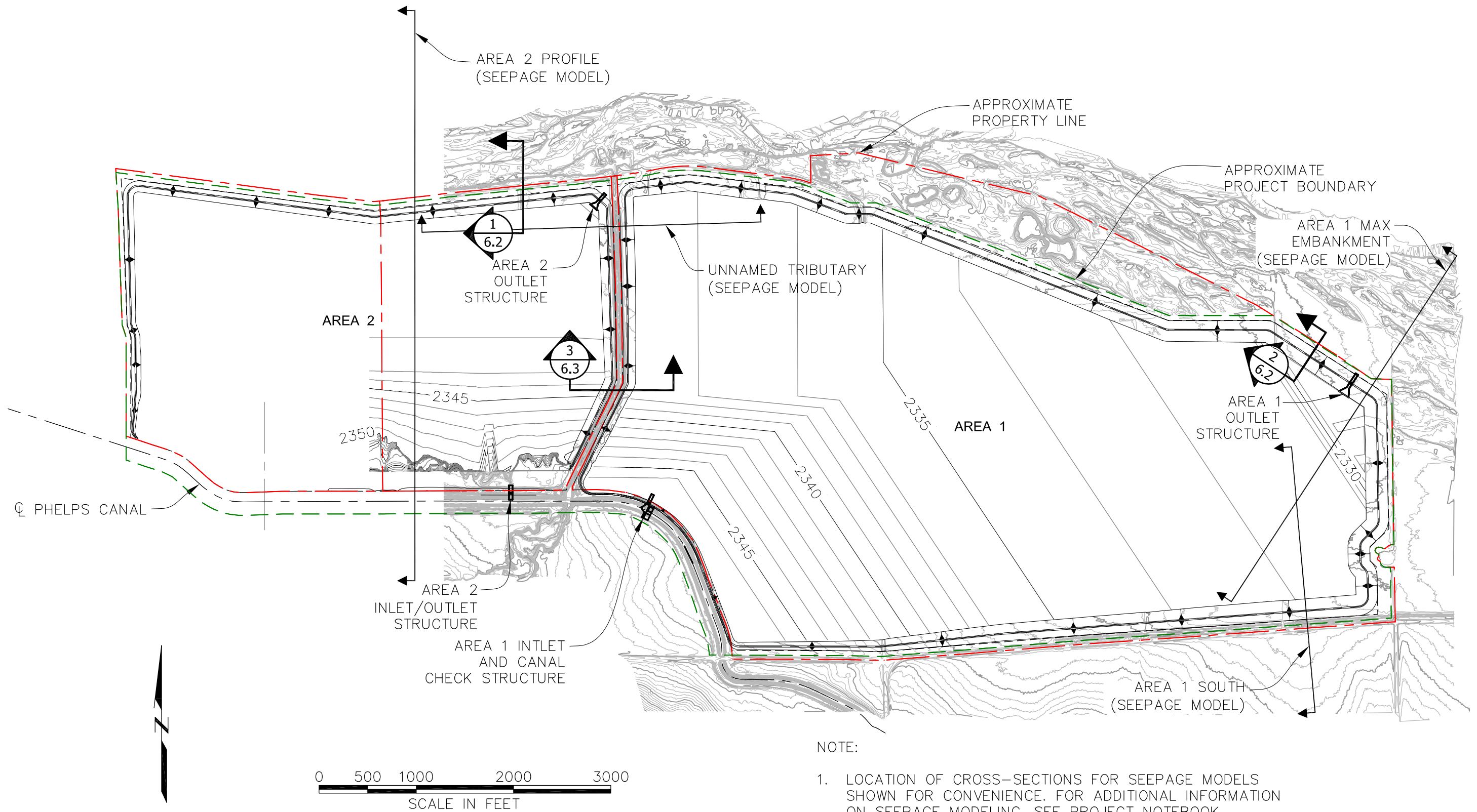
The overall upstream slope protection consists of a hybrid of plated and stair-stepped soil-cement as shown on Figure 6.2. RJH selected to use plated soil-cement from the toe of the dam to El. 2354.0 (2 feet below normal maximum pool elevation) because this would require less soil-cement per vertical foot than the stair-step method of construction and consequently would result in a lower cost. RJH selected the plated soil-cement to be 16 inches thick based on experience. Additional evaluation is needed in later stages of design to optimize the actual thickness needed, but 16 inches is a reliable thickness for use in this conceptual evaluation.

RJH selected stair-stepped soil-cement from El. 2354.0 to El. 2360.0 (the crest of both dams) because the stair-stepped armoring decreased the wave run-up height relative to a slope protected with plated soil-cement. Although the stair-step method requires more soil-cement material than the plating method per vertical foot of dam, the overall cost for the upper part of the embankment would be less for stair-stepped slope protection because the dam crest would be at a lower elevation. The lower crest elevation results in less volume of embankment fill and fewer vertical feet of slope to protect. RJH selected each “stair” to be 1 foot thick and 8 feet wide. The width of 8 feet was based on the

minimum size of construction equipment that could effectively build each lift of the stepped slope protection. The final lift of soil-cement at the dam crest would be 14 feet wide to provide a uniform surface on the dam crest.

A 12-inch-thick gravel layer was included immediately below the soil-cement to prevent uplift pressure below the soil-cement and to mitigate removal of embankment soils through cracks during reservoir drawdown or wave action. The soil-cement will crack and gravel particles would be large enough to not be plucked out through the cracks anticipated to develop in the soil-cement. This gravel material could be obtained by processing on-site soils.

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_6-1-OVERALL_PLAN.DWG 4/10/2013 10:43 AM



THIS DRAWING MUST BE REPRODUCED IN COLOR.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

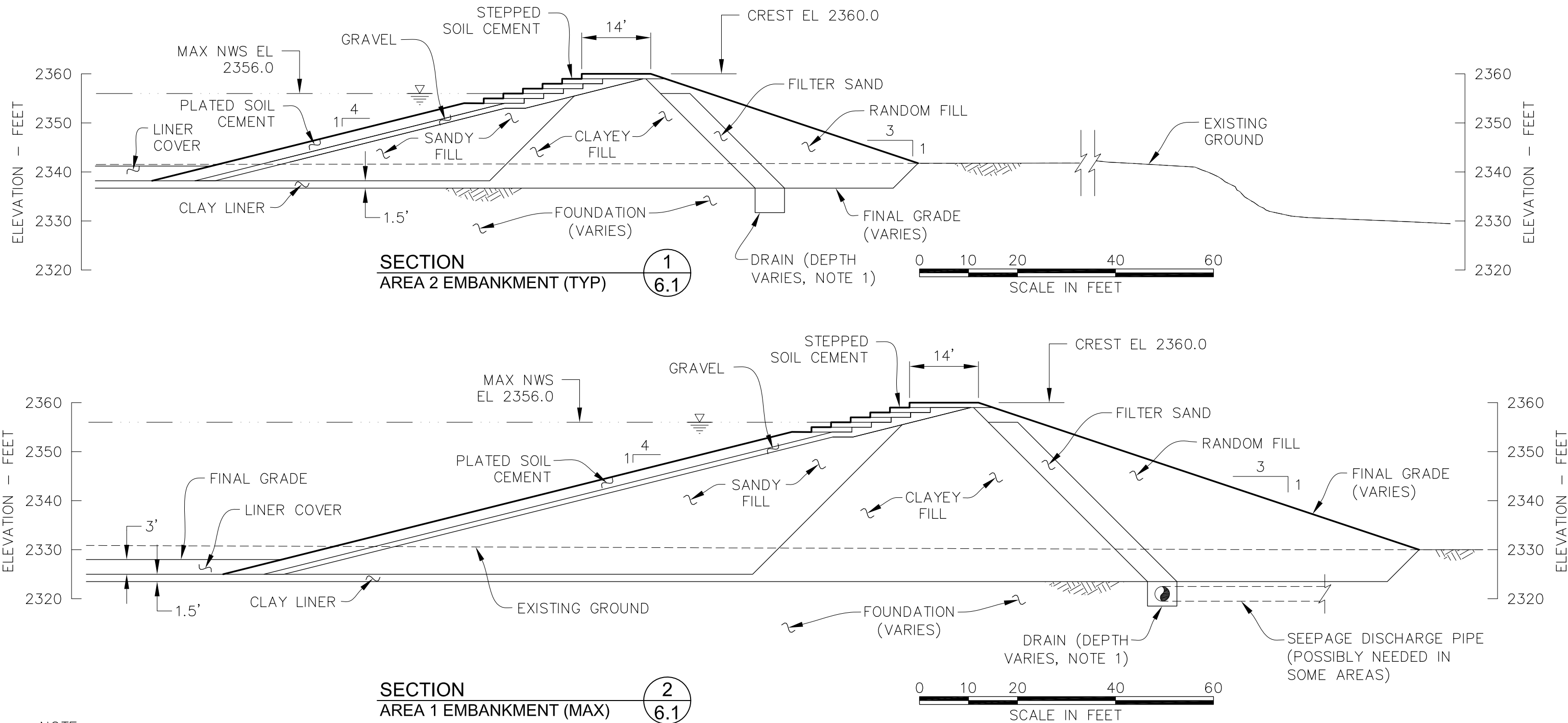
J-2 RESERVOIR
OVERALL PLAN

PROJECT NO. 12116

April 2013

Figure 6.1

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_6-2-SECTIONS.DWG 4/10/2013 10:50 AM



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

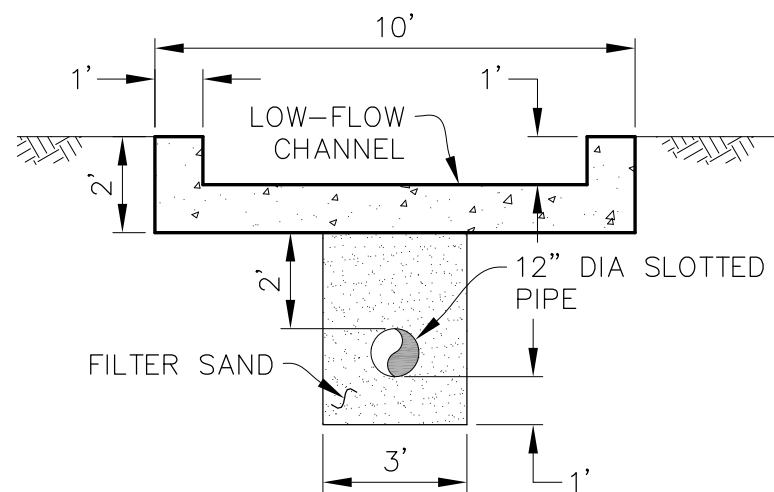
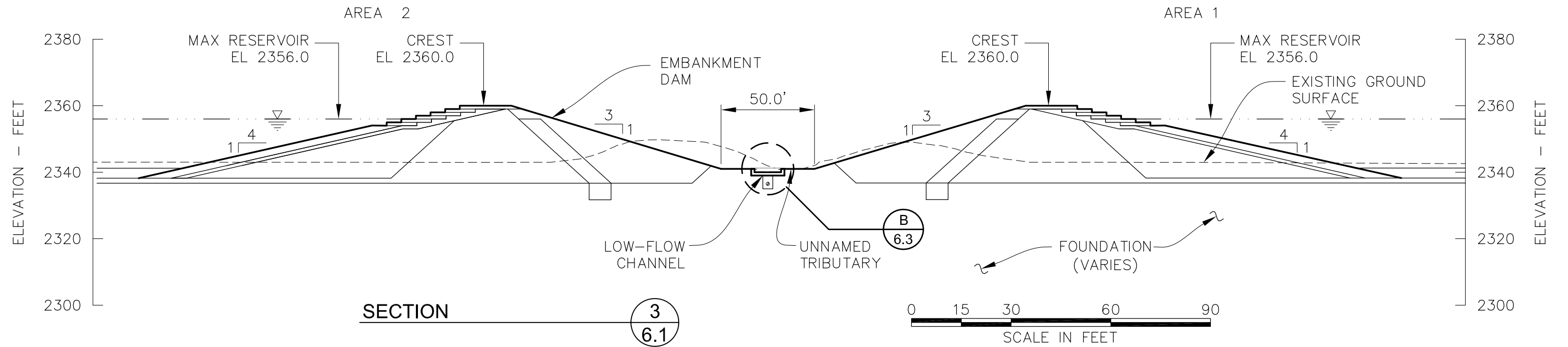
PROJECT NO. 12116

J-2 RESERVOIR
EMBANKMENT
SECTIONS 1 OF 2

April 2013

Figure 6.2

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_6-3-SECTIONS.DWG 4/10/2013 10:58 AM



NOTE:

1. EMBANKMENT ZONING IS ANNOTATED ON FIGURE 6.2.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

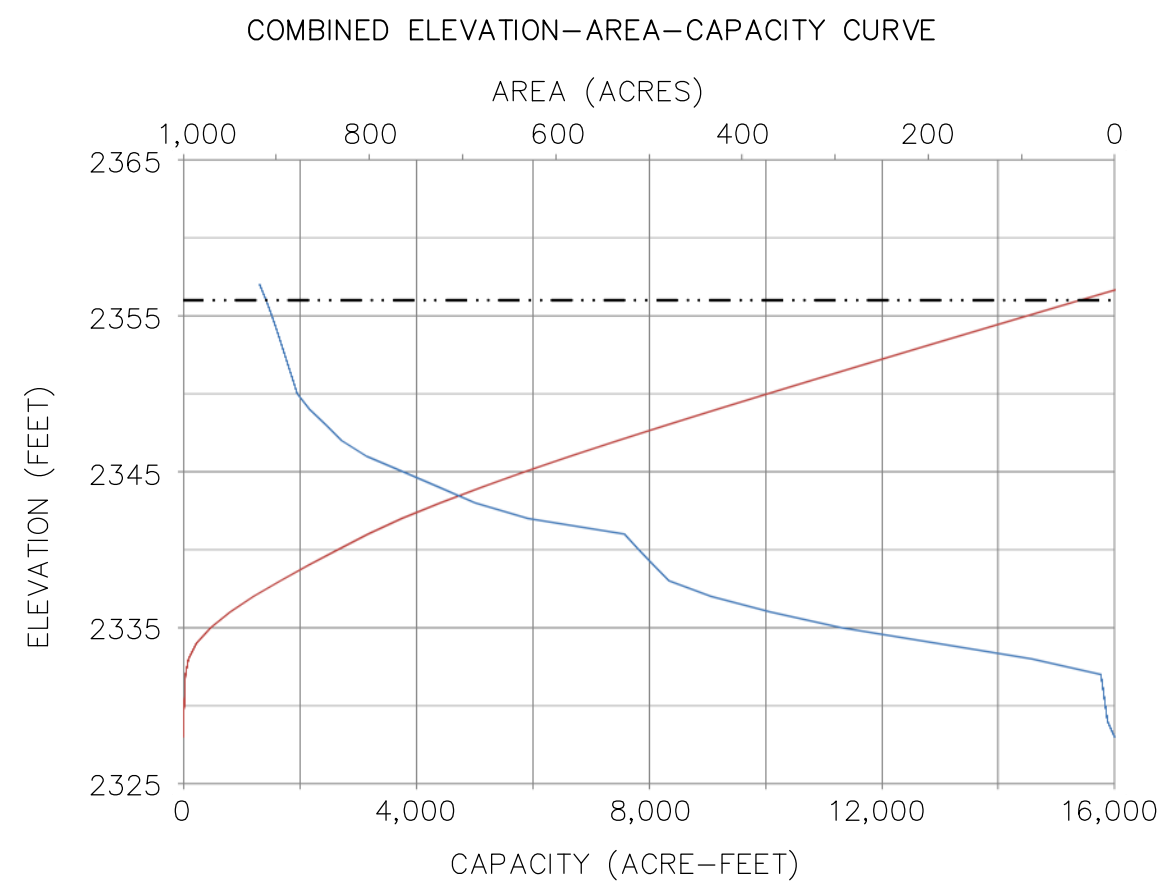
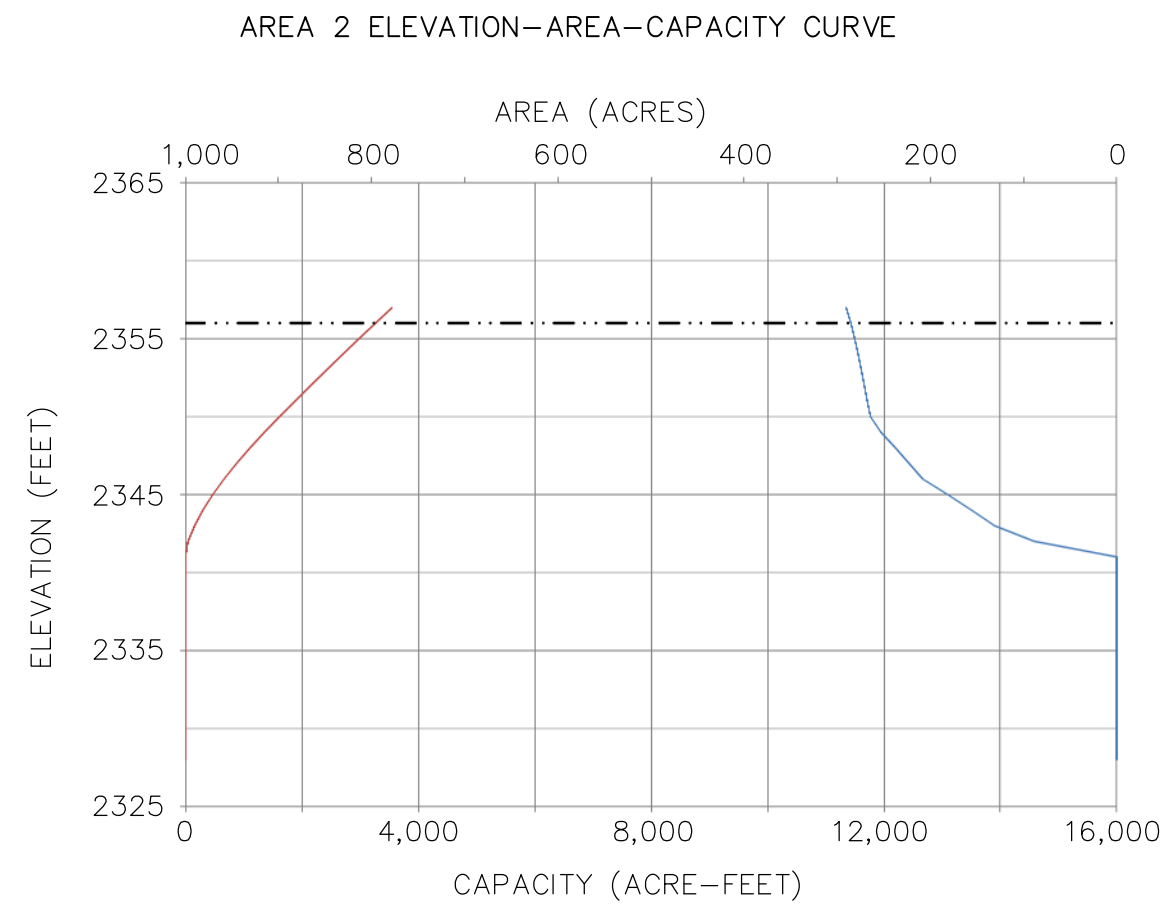
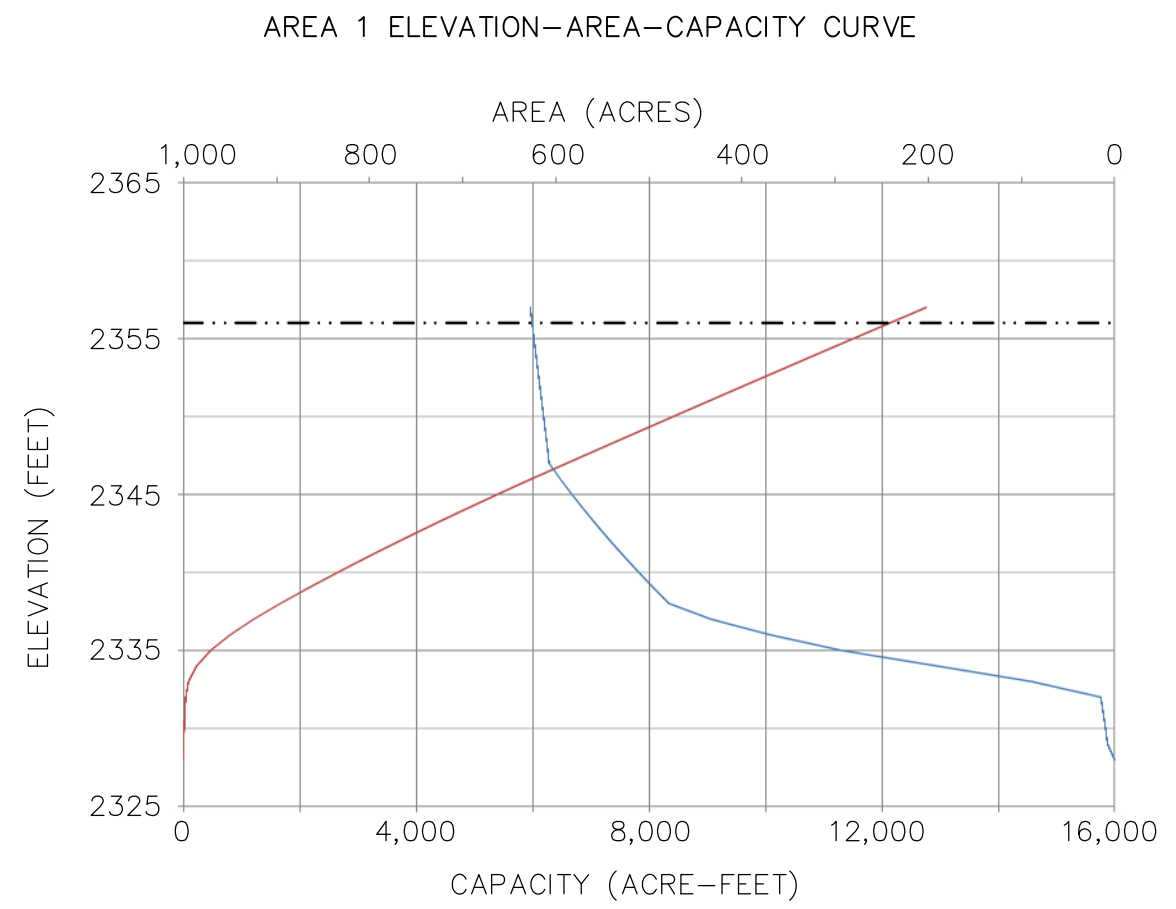
PROJECT NO. 12116

J-2 RESERVOIR
EMBANKMENT
SECTIONS 2 OF 2

April 2013

Figure 6.3

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_6-4-AREA_CAP_CURVES.DWG 4/10/2013 11:12 AM



EXPLANATION

- MAX NWS
- ELEVATION-CAPACITY
- ELEVATION-AREA

THIS DRAWING MUST BE REPRODUCED IN COLOR.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

ELEVATION-AREA-
CAPACITY CURVES

PROJECT NO. 12116

April 2013

Figure 6.4

SECTION 7 - RESERVOIR HYDRAULIC STRUCTURES

7.1 General

The RJH concept includes four hydraulic structures:

- Area 2 Inlet/Outlet Structure.
- Area 2 Outlet Structure.
- Area 1 Inlet Structure (including a check gate in the canal).
- Area 1 Outlet Structure.

Each hydraulic structure would control flows into and out of the reservoirs with a combination of two or more gates. The selected gates and sizes at each hydraulic structure are summarized in Table 7.1.

TABLE 7.1
SUMMARY OF SELECTED GATE TYPES AND SIZES

Control Gate Location	Control Gate Type ⁽¹⁾⁽²⁾	No. of Gates	Gate Size (Width x Height)	Gate Bottom Sill Elevation
AREA 2 INLET/OUTLET	BI-DIRECTIONAL SLIDE	3	10' x 10'	2342.0
AREA 2 OUTLET	RADIAL	1	15' x 15'	2340.0
	SLIDE	1	10' x 10'	2338.0
AREA 1 INLET	RADIAL	1	20' x 15'	2341.0
	SLIDE	1	10' x 10'	2341.0
CANAL CHECK	RADIAL	1	30' x 15'	2341.0
AREA 1 OUTLET	RADIAL	1	25' x 15'	2330.0
	SLIDE	1	10' x 10'	2328.0

Notes:

1. All radial gates would have cable drum hoists.
2. All slide gates would have electric actuators.

The slide gates at the inlet and outlet structures were sized to provide control for flows less than 500 cfs. The radial gates were sized and selected, and the inlet and outlet structures were sized to control flows greater than about 500 cfs. The Area 2 Reservoir inlet/outlet gates would all be slide gates designed for bi-directional flow (i.e., for differential head in either direction). The canal check gate that would be downstream of

the Area 1 inlet was sized based on the existing canal geometry and to be similarly-sized to the existing J-2 canal gate at the J-2 Return.

The structures were sized considering the hydraulic requirements summarized in Table 7.2.

TABLE 7.2
SUMMARY OF HYDRAULIC DESIGN REQUIREMENTS

Structure	Flow Condition	Minimum Flow (cfs)	Minimum Differential Head (ft)	Elevation Constraints ⁽⁵⁾ (ft)
AREA 2 INLET/OUTLET	Canal to Reservoir	1,675	1.7 ⁽⁴⁾	Area 2 Reservoir WSE must be below about 2354.3
	Reservoir to Canal	1,000	0.7 ⁽²⁾⁽⁴⁾	Area 2 Reservoir WSE must be above about 2349.0
AREA 2 OUTLET	Reservoir to Platte River	900	N/A	Area 2 Reservoir WSE must be above about 2347.7 ⁽¹⁾
AREA 1 INLET	Canal to Reservoir	1,675	1.8 ⁽³⁾⁽⁴⁾	Area 1 Reservoir WSE must be below about 2354.2
CANAL CHECK	Upstream Canal to Downstream Canal	1,000	N/A	N/A
AREA 1 OUTLET	Reservoir to Platte River	1,150	N/A	Area 1 Reservoir WSE must be above about 2338.0 ⁽¹⁾

Notes:

N/A represents that the value was not considered a design constraint and therefore not evaluated.

1. Flow and reservoir elevation based on flow only through the radial gate. Combined flow rate through both gates will be higher. Also, reservoir elevation at minimum flow will be lower once both gates are considered.
2. Future refinements to the number and size of the gates will likely be required to optimize operational flexibility and costs. This may result in an increase in the cost of this structure.
3. Future refinements to the number and size of the gates will likely be required to optimize operational flexibility and costs. It is possible that the slide gate could be eliminated because accurate regulation of low flows may not be required. This may result in a decrease in the cost of this structure.
4. The flow rate will be reduced as the differential head is reduced.
5. The elevation constraints noted refer to constraints in achieving the minimum flow through the hydraulic structure.

The radial gates at the Area 1 inlet, the canal check structure, Area 1 outlet, and Area 2 outlet would all be designed with a top elevation at El. 2356.0 to safely allow for overflow. RJH considered that these gates could be used as spillways to control the water level in the canal and the reservoirs. Spillways are a critical dam safety component used to avoid overtopping even if reservoir inflows are controlled because of the possibility of equipment malfunctions or operator errors. By using the gates as overflow spillways, a separate spillway structure would not be required.

Although the overflow radial gates could be supplied by several manufacturers, the following issues need to be considered at later stages of design to protect the gates:

- In addition to water, items such as trash, debris, or ice blocks could be carried over the top of the radial gates.
- Typical paints may be damaged by the impact and abrasion by trash and ice on the gate members.
- The radial gate trunnions could be periodically submerged under water during high flows.

The locations of the hydraulic structures are shown on Figure 7.1 and each structure is discussed in the following sections.

7.2 Area 2 Inlet/Outlet Structure

The Area 2 inlet/outlet structure is shown on Figure 7.2 and would consist of a reinforced concrete structure about 35 feet wide and 140 feet long with three, 10-foot by 10-foot bi-directional slide gates. The bi-directional slide gates would be upward opening gates with the invert of the gates located at the bottom elevation of Phelps Canal.

The structure and gates were sized to convey 1,000 cfs from the Area 2 Reservoir to the canal with 1 foot of differential head when the Area 2 Reservoir WSE is above about El. 2349. The Area 2 inlet structure could also convey 1,675 cfs from the canal to the Area 2 Reservoir with a minimum of 1.7 feet of differential head and about 700 cfs with a differential head of 0.3 foot.

The structure would include a bridge to allow for maintenance access to the gate stems and to allow unimpeded access along the canal. The bottom and reservoir-side of Phelps Canal would be lined with soil-cement for a distance of 50 feet upstream and downstream of the inlet channel to protect against erosion from changing flow directions of the water. Inside the reservoir, an earthen channel would be excavated so that flow could be

conveyed between the canal and reservoir without causing erosion of the protective cover over the liner.

7.3 Area 2 Outlet Structure

The Area 2 outlet structure is shown on Figure 7.3 and would consist of a reinforced concrete structure about 27 feet wide by 183 feet long with one 10-foot by 10-foot slide gate and one 15-foot-wide by 15-foot-high radial gate. The slide gate would be used to release smaller target flows (less than 500 cfs). The radial gate would be used to release larger flows (e.g., the SDHF) and would act as an overflow spillway. The gates were sized to convey 900 cfs when the reservoir WSE was at El. 2347.7. An outflow of 900 cfs from Area 2 combined with outflow from Area 1 would meet the requirements of the SDHF flow (about 2,000 cfs total flow) for 3 days when both reservoirs are full at the start of the SDHF. The flow capacity through the radial gate would be significantly greater when the reservoir is above El. 2347.7.

The outlet structure would be located in the northeast corner of the Area 2 Reservoir and was aligned to discharge into the unnamed tributary. A discharge channel with baffle blocks was included to dissipate energy and enable the release of low velocity flows (sub-critical). The discharge channel downstream of the outlet structure would be armored with soil-cement, would include a soil-cement drop structure for energy dissipation, and would include sheetpiles for erosion protection at the end of the channel.

RJH included a bridge across the unnamed tributary downstream of the outlet structure to allow for access to the private residences that would remain north of the site. RJH assumed a small, one-lane bridge would be suitable for this private access and to provide access for maintenance and inspections to CNPPID personnel.

7.4 Area 1 Inlet Structure

The Area 1 inlet structure is combined with a check structure within Phelps Canal. The combined Area 1 inlet/check structure is shown on Figure 7.4. This inlet structure would consist of a reinforced concrete structure about 37 feet wide and 115 feet long with one 10-foot by 10-foot slide gate and one 20-foot-wide by 15-foot-high radial gate to convey water from Phelps Canal into the Area 1 Reservoir. The radial gate at the Area 1 inlet would be designed to overflow at El. 2356.0. The inlet gates for Area 1 Reservoir were sized to convey 1,675 cfs from the canal to the Area 1 Reservoir with a minimum of 1.8 feet of differential head. The flow capacity would be about 1,100 cfs with a minimum of 1.0 foot of differential head.

The check structure in the canal would consist of reinforced concrete walls, channel lining, and a 30-foot-wide by 15-foot-high radial gate. The top of the gate and check structure in the canal would be at El. 2356.0. This structure would provide significant flexibility for water control to and from the reservoirs. Water levels in the canal could be maintained high enough to enable storage at El. 2356.0 and also be lowered up to 3 feet to allow withdrawal from Area 2 Reservoir for downstream irrigators without discharge from the J-2 hydropower plant.

Based on HEC-RAS modeling when Phelps Canal was flowing at about 1,675 cfs, the top of the check structure only needed to be at about El. 2352.0 to back up flow to El. 2356.0 at the Area 1 and Area 2 inlet structures. However, the top elevation of the check structure was set at El. 2356.0 to provide significantly more flexibility for system operations and enable all flow to be conveyed into either reservoir.

Soil-cement lining would be included in Phelps Canal for about 50 feet upstream and downstream of the reinforced concrete lining, and for about 35 feet downstream of the concrete inlet channel.

The elevation difference from the end of the inlet structure (El. 2341) and the relative flat reservoir bottom (El. 2337) is about 4 feet. An engineered channel consisting of generally flat unlined sections with either soil-cement or sheetpile drop structures would be used to safely convey the flow to the reservoir to avoid eroding the protective cover over the liner. It is estimated that the unlined areas would slope at about 0.1 to 0.4 percent and the drop structures would be about 2 to 4 feet high, depending on the number of drop structures selected.

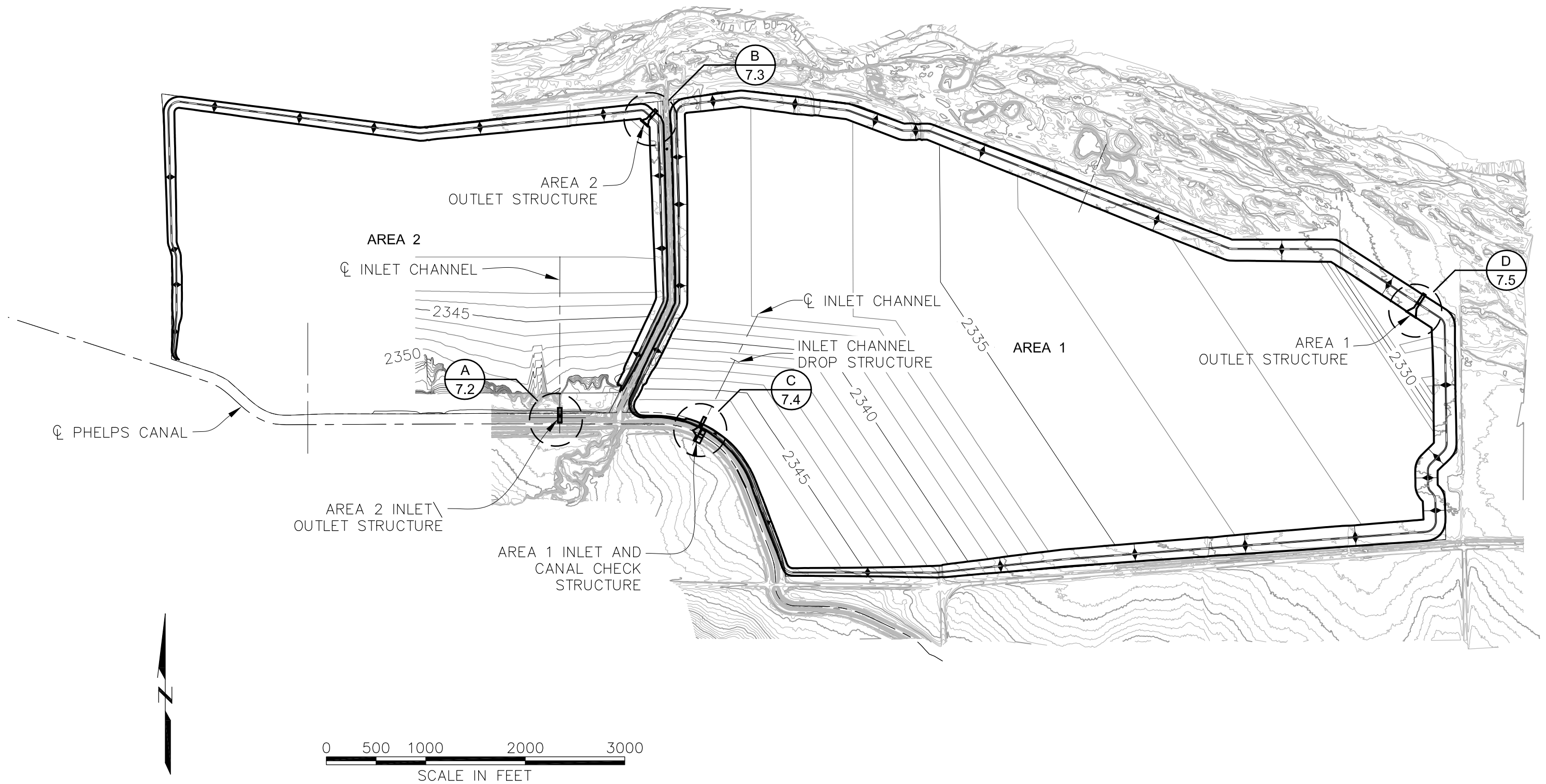
7.5 Area 1 Outlet Structure

The Area 1 outlet structure is shown on Figure 7.5 and would consist of a reinforced concrete structure about 27 feet by 250 feet with one 10-foot by 10-foot slide gate and one 25-foot-high by 15-foot-wide radial gate. The structure is located in the northeast corner of Area 1 Reservoir. The slide gate would be operated to release smaller target flows (less than 500 cfs). The radial gate would be used to release larger flows (e.g., SDHF). The structure was sized to convey 1,150 cfs when the reservoir water surface elevation is as low as El. 2338.8. This discharge flow, when combined with discharge from Area 2 Reservoir during the SDHF, would achieve the total outlet flow of 2,000 cfs for 3 days.

If both the radial gates and the slide gates are used to make the SDHF releases, both reservoirs could be at El. 2353.4 at the start of the SDHF. The water surface would be at El. 2336.0 and El. 2343.0 in Area 1 and Area 2, respectively, at the end of the SDHF event.

The long, flat reinforced discharge structure combined with the baffle blocks will provide energy dissipation so that low-velocity flow would exit from the structure. The flow needs to be safely conveyed to the Platte River. RJH assumed that grading a channel to the Platte River would not be feasible because of environmental and property constraints. Therefore, our concept includes a soil-cement apron that is set at the elevation of the normal thalweg of the Platte River (El. 2322.0) and sheetpiling to prevent headcutting and erosion. If the channel cannot be cut between the sheetpiling and normal flow channel of the Platte River because of land ownership issues, natural cutting by the flow will occur after flow is released through the outlet.

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_7-1-PLAN_OF_HYD_STRUCT.DWG 4/10/2013 11:16 AM



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

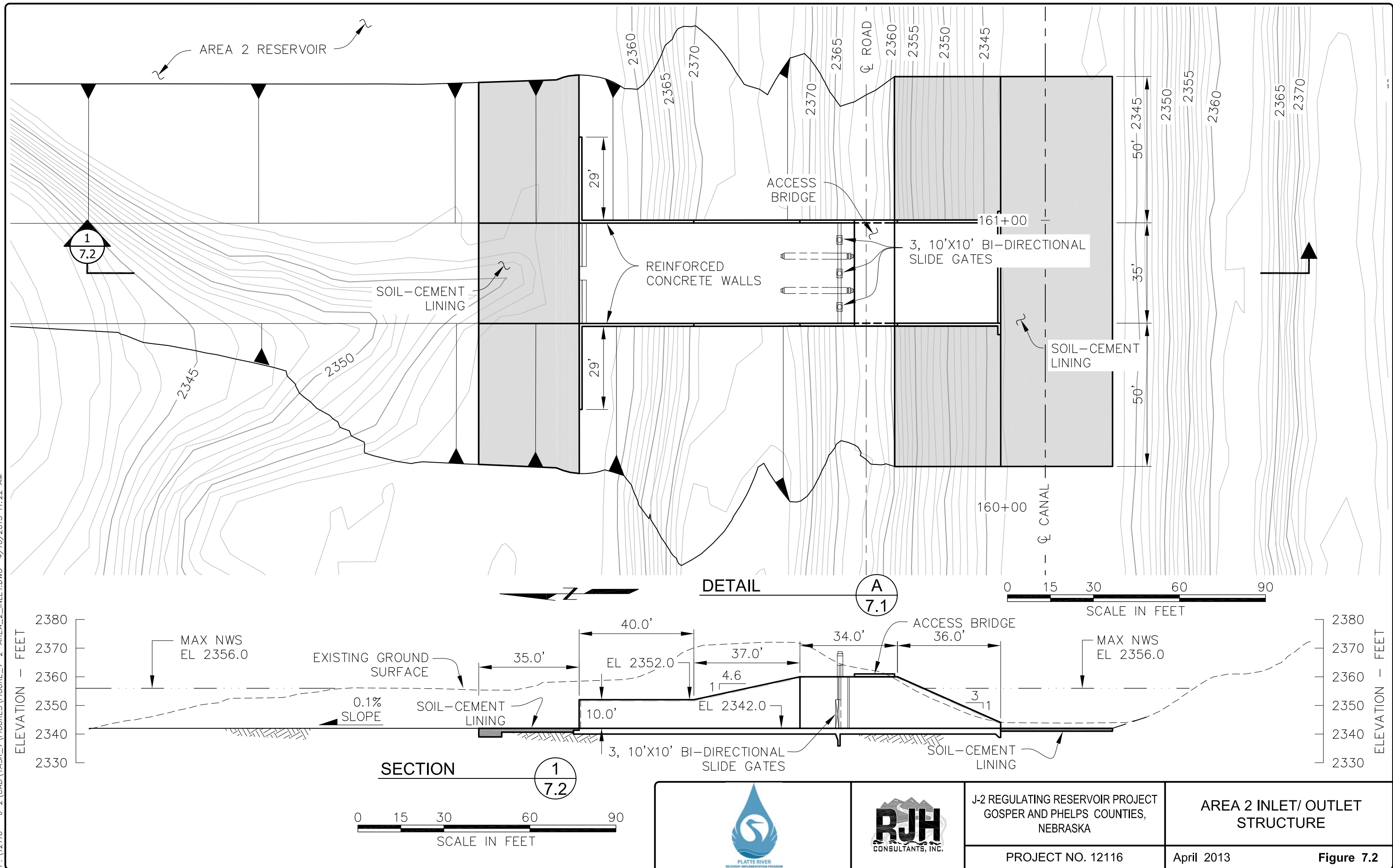
PLAN OF RESERVOIR
HYDRAULIC STRUCTURES

PROJECT NO. 12116

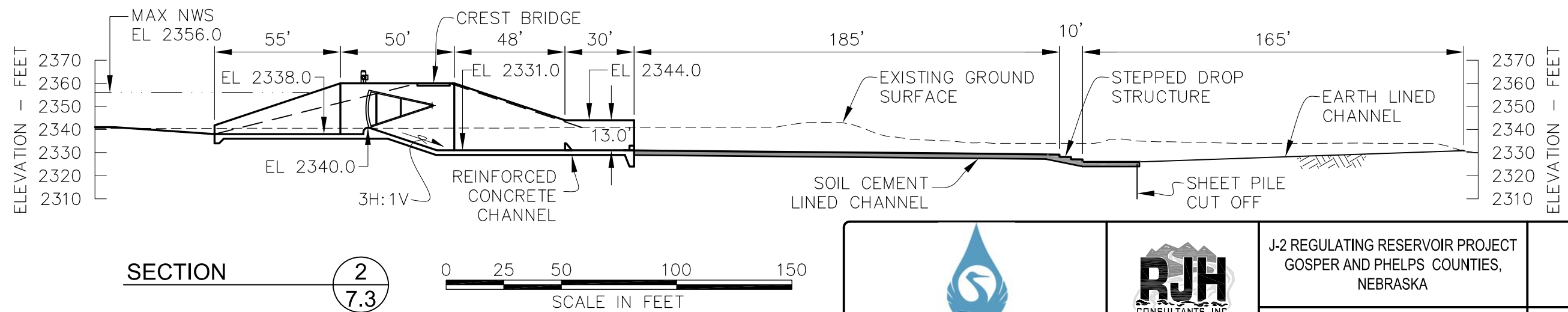
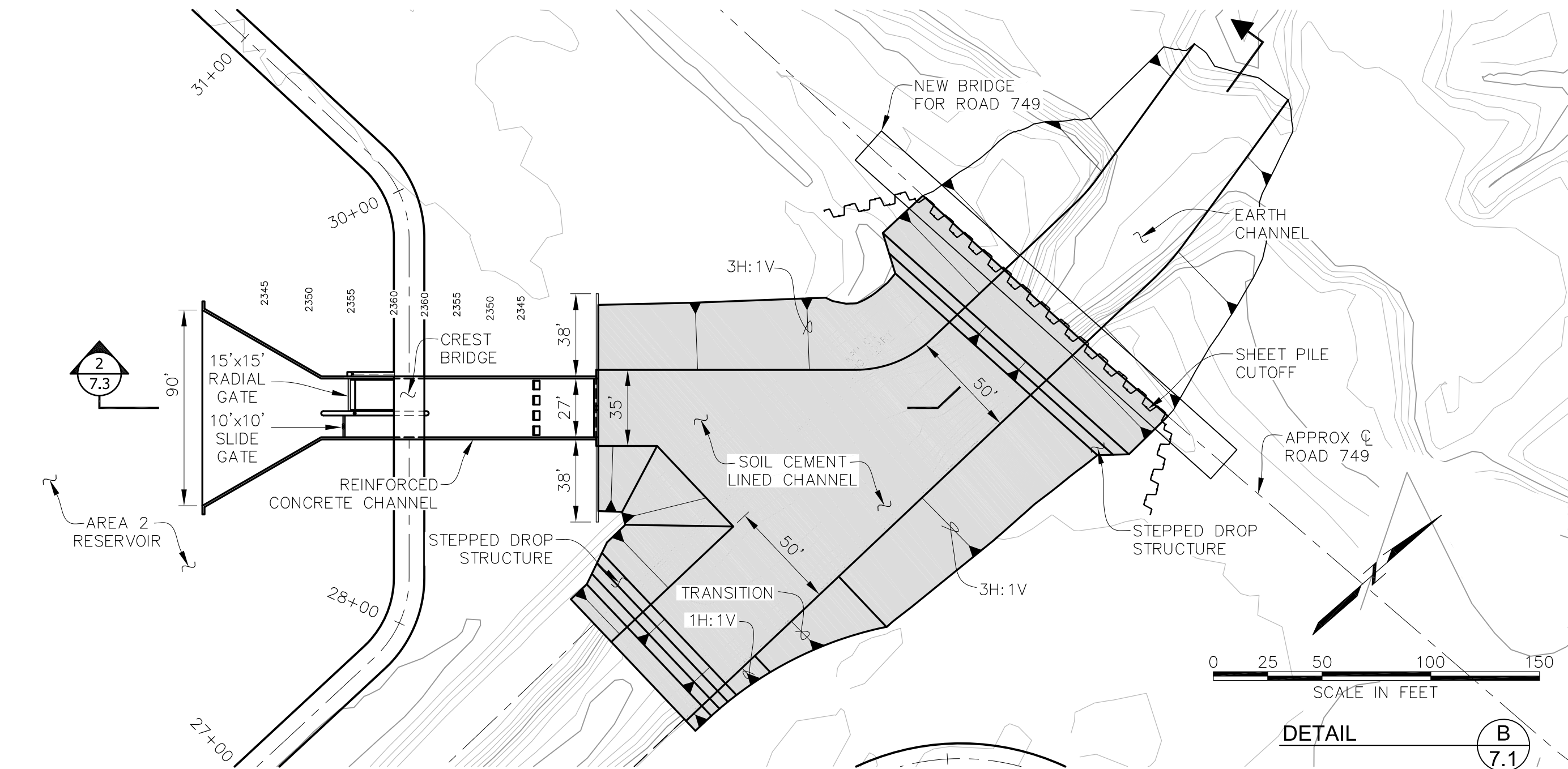
April 2013

Figure 7.1

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_7-2-AREA_2_INLET.DWG 4/10/2013 11:22 AM



P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_7-3-AREA_2_OUTLET.DWG 4/10/2013 11:27 AM



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

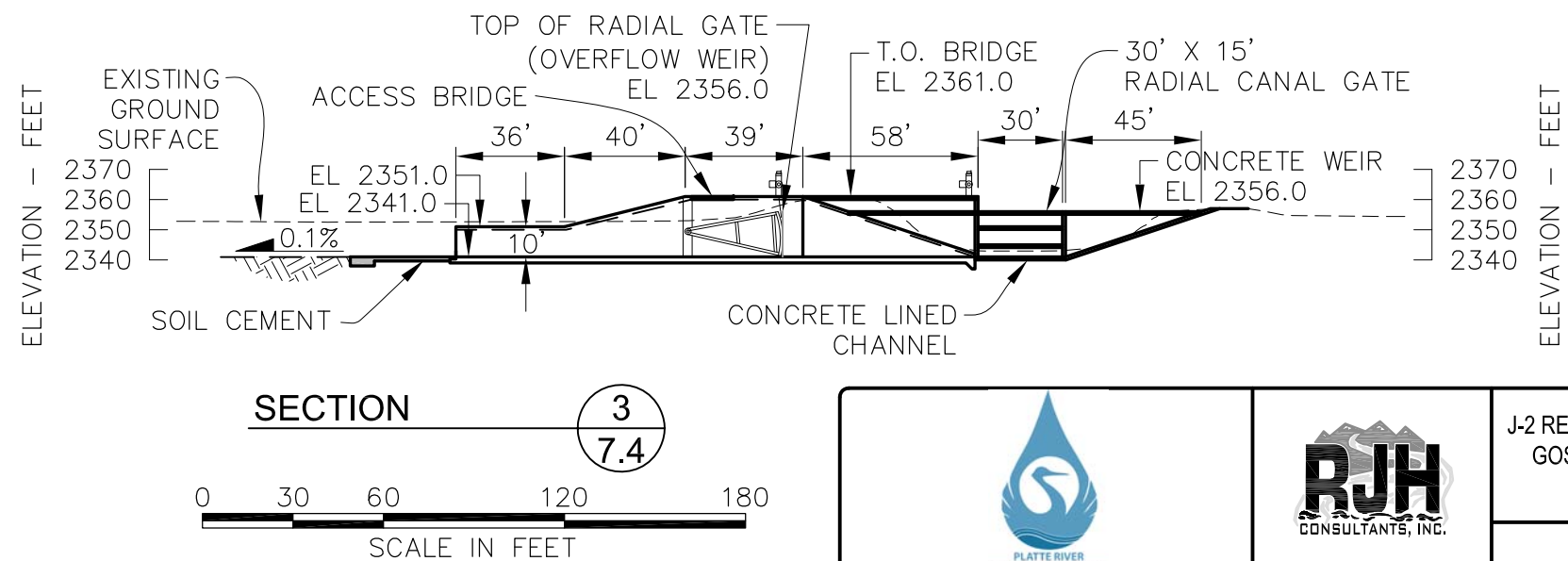
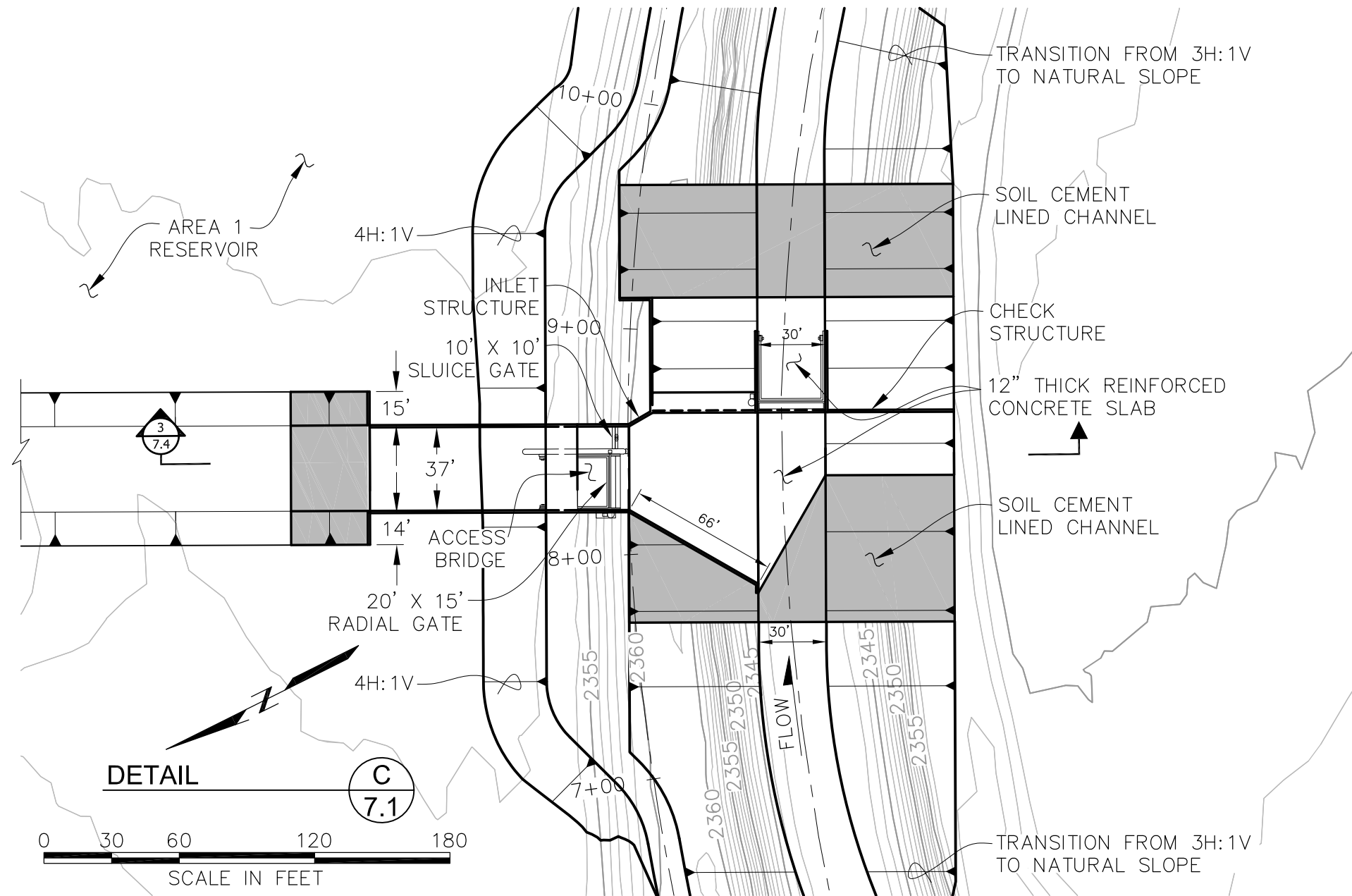
PROJECT NO. 12116

AREA 2 OUTLET
STRUCTURE

April 2013

Figure 7.3

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_7-4-AREA_1_INLET.DWG 4/10/2013 11:33 AM



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

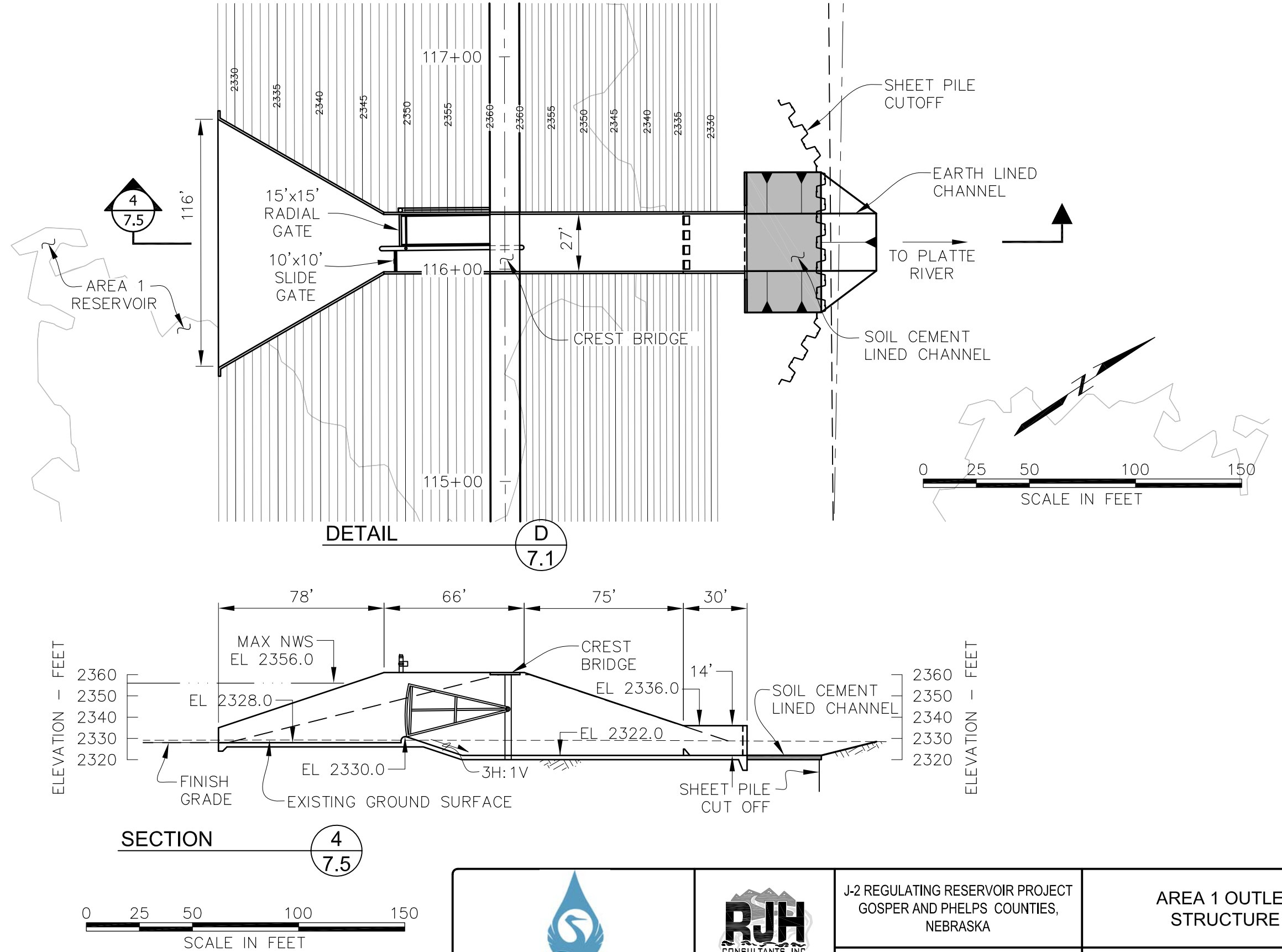
PROJECT NO. 12116

AREA 1 INLET
AND CANAL
CHECK STRUCTURE

April 2013

Figure 7.4

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_7-5-AREA_1_OUTLET.DWG 4/10/2013 11:43 AM



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

PROJECT NO. 12116

AREA 1 OUTLET
STRUCTURE

April 2013

Figure 7.5

SECTION 8 - CANAL AND CREEK MODIFICATIONS

8.1 Phelps Canal Modifications

Phelps Canal would need to be modified to accommodate the increased flow of 1,675 cfs downstream of the existing J-2 Return. HEC-RAS, a 1-dimensional open channel flow computer model, was used to model the existing canal with the addition of a downstream check structure and an additional siphon. The model computed the WSE in the canal at 1,675 cfs and RJH compared the WSEs to bank elevations to identify locations where the canal bank would need to be raised to maintain 1 foot of freeboard. The computed WSEs were also used to identify bridges whose low chords would be inundated with the increased flow. Velocities calculated by the model were considered to evaluate scour potential. Based on the HEC-RAS computations, the required modifications along the canal would include:

- Raising the banks about 1 foot at two locations on the north side of the canal for a total length of about 750 linear feet. The two locations are between about Station 32+50 and Station 35+50 and Station 90+92 and Station 95+42.
- Adding a second parallel siphon below Plum Creek. The concept would be to add a 13-foot-diameter siphon to the east of the existing siphon and re-construct the east walls of the existing siphon intake and outlet chute.
- Raising two bridges by 1.5 feet whose existing low-chords would be inundated by the raised water surface.
- Raising by 1.5 feet, or removing an existing wooden farm bridge at Station 101+61.
- Raising the wall of the existing flume (about Station 167+28) over the unnamed tributary by about 2.5 feet.

The canal modifications are shown in plan on Figure 8.1. In addition, modifications to the canal are needed to accommodate the new reservoir inlet structures. Discussion regarding the hydraulic structures is presented in Section 7 of this report.

Slope protection along the canal would not be required (except at the inlet structures) because the anticipated velocities at the design flow would be less than 2 fps. Velocities of 2 fps would be slow enough to not cause erosion of the canal slopes. In addition, based on the HEC-RAS models of the existing canal, the existing flow velocities are approximately 1.7 to 1.8 fps and the canal has performed well.

8.2 Erosion Protection along Embankments

8.2.1 *Unnamed Tributary*

RJH addressed potential issues with flow in the unnamed tributary eroding the reservoir embankments by modifying the grading in the existing unnamed tributary stream channel from the existing 0.3 percent slope to a 0.1 percent slope. This would reduce the steepness of the channel and therefore the flow velocity. Two soil-cement drop structures about 3 feet high would also be constructed. By re-grading the channel and adding the armored drop structures, the velocities in the unnamed tributary were estimated with HEC-RAS to be about 6.5 fps for the PMF flow of 4,000 cfs. Velocities of 6.5 fps are at the threshold of predicted erosion for a sod-covered slope. For extreme and rare events such as the PMF, RJH considered that erosion, if it occurred, would not breach the dam and could be addressed with repairs instead of expensive soil-cement armoring.

RJH also included a 10-foot-wide by 1-foot-deep concrete-lined low-flow channel to avoid excessive maintenance of the channel. The RJH concept for the concrete-lined low-flow channel could convey about 40 cfs. A groundwater drain (see Section 5) was included below the low-flow channel to manage uplift pressures. This concept is shown on Figure 6.3.

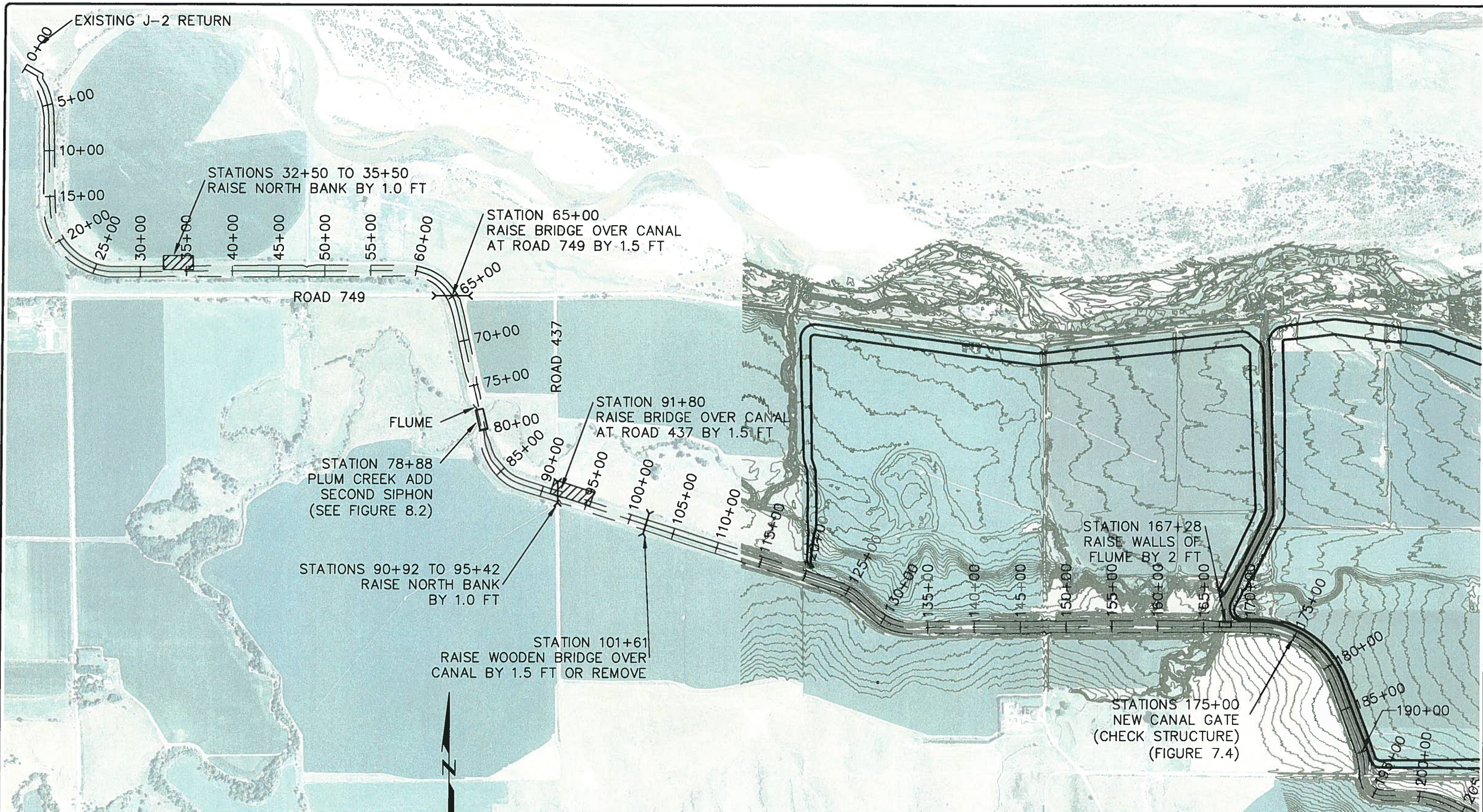
8.2.2 *Plum Creek*

RJH addressed potential issues with flow in Plum Creek by including slope protection on the west side of the Area 2 embankment. The protection would include sheetpiles below the embankment (foundation treatment) and soil-cement on part of the embankment slope. The foundation treatment would include about 1,000 linear feet of 30-foot-deep sheetpiles (where Plum Creek must be turned 90 degrees), and 1,500 linear feet of 15-foot-deep sheetpiling north of the 30-foot-deep piles when the flow would be parallel to the embankment. The sheetpiling is needed to mitigate potential for scour to undermine and fail the dam. Above the sheetpiles, RJH included 12-inch-thick soil-cement armor on the embankment up to about 8 feet above the existing ground surface.

This concept is reasonable for a conceptual-level study and to support a conceptual cost opinion, however the Plum Creek basin hydrology is not well defined at this stage (see Section 3) and significant additional evaluation of both the hydrology and the scour and erosion requirements are required to select a final concept. The preferred solution from a long-term reliability standpoint would be to re-route Plum Creek through the adjacent

property to the west so that the channel is maintained in a straighter alignment and far away from the dam.

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_8-1-PHELPS_CANAL_IMP.DWG 4/10/2013 11:48 AM



THIS DRAWING MUST BE REPRODUCED IN COLOR.



PLATTE RIVER
RECOVERY IMPLEMENTATION PROGRAM



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

PROJECT NO. 12116

PHELPS CANAL
IMPROVEMENTS

April 2013

Figure 8.1

SECTION 9 - OPINION OF PROBABLE PROJECT COST

9.1 Capital Costs

RJH developed a conceptual-level opinion of probable Project costs based on the conceptual design presented in this report. RJH based our opinion of costs on bid tabs from similar projects, estimates from RS Means cost data books, planning level quotes from gate and bridge suppliers, adjustments for location and inflation based on ENR index of construction prices, and general experience with large earthen dam projects. The intent of the cost opinion as stated in the scope of work was to develop the cost opinion to a Class 4 level estimate as defined by the Association for the Advancement of Cost Estimating (AACE). This level is appropriate for a study or feasibility phase where the design engineering is between 1 and 15 percent complete. The reliability of this level of estimate according to the AACE should be considered to be between about minus 15 to 30 percent and plus 20 to 50 percent. It is our opinion that the concepts for the primary items that represent over 70 percent of the Project costs are reasonably defined and in our opinion, the reliability of the opinion of probable costs presented in this report is likely between minus 15 to plus 20 percent. This means that the final Project cost is likely to be between 15 percent less to 20 percent more than the cost provided in this report, when all costs are compared to 2013 dollars.

RJH's Opinion of Probable Project Costs is about \$62.6 million (2013 dollars). This includes contingencies; direct construction costs; and allowances for engineering, permitting, etc. Our opinion of direct construction costs (DCC) is \$45.0 million. The DCC is RJH's estimate of what we would expect bid costs to be currently. Our OPPC for the primary Project elements are shown in Table 9.1.

RJH developed the allowances based on percentages of the DCC. Our estimate for design engineering is 7 percent of the DCC. This is a typical percentage based on published data and experience. We estimated owner administration as 2 percent, which is typical for planning projects but should be reviewed by the Program to evaluate if appropriate for this Project and anticipated administration. For construction engineering (including submittal reviews, resident engineering, and materials testing), we estimated the cost would be 8 percent of the DCC. This is a typical percentage based on published data and experience. We estimated permitting and environmental mitigation as 2 percent of the DCC. The Program recently has constructed wetland areas and credits from these areas could be dedicated to this Project to fulfill requirements to rebuild any disturbed wetlands as a result of the Project.

RJH anticipates that the permitting effort will be highly variable based on the regulatory agencies involved. At this stage and based on discussions with the Program, RJH assumed that regulatory coordination could involve the Federal Energy Regulatory Commission (FERC) and could be significant.

RJH also included a Design and Construction contingency of 20 percent of the DCC. In RJH's opinion, this contingency, although within accepted ranges for conceptual design, is slightly lower than typical conceptual design contingencies (i.e., 25 to 30 percent) because:

- The geotechnical conditions are generally reasonably understood.
- The design for the major cost items such as the liner, embankment, and slope protection are not anticipated to change significantly and represent a major portion of Project costs.
- The quantities for major items are generally well understood.
- A reasonable contingency at the end of final design for construction is 10 percent. For this level of concept development, 10 percent is appropriate to account for unlisted items and design elements that have not been resolved.

The key cost factors in this Project are all related to earthwork. The reservoir liner accounts for over 25 percent of the DCC. The slope protection accounts for just under 25 percent of the DCC. The embankment fill accounts for about 15 percent of the DCC. To significantly reduce the cost of this Project, a significantly different concept would be needed that did not require a reservoir liner.

This opinion of probable construction costs is based on professional opinion of the costs to construct the Project as described in this report. Actual costs would be affected by a number of factors beyond current control such as supply and demand for the types of construction required at the time of bidding and in the Project vicinity, changes in material supplier costs, changes in labor rates, the competitiveness of contractors and suppliers, changes in applicable regulatory requirements, and changes in design standards and concepts. Therefore, conditions and factors that arise as Project development proceeds through construction may result in construction costs that differ from the estimates documented in this report.

Much of the cost for this Project is earthwork and the cost for earthwork is highly sensitive to fuel costs. If fuel costs change significantly in the next few years the cost of

the Project could be directly impacted. RJH has not attempted to predict changes in future fuel prices to develop this OPPC.

TABLE 9.1
RJH OPINION OF PROBABLE PROJECT COSTS

Reservoir and Embankments	Quantity	Unit	Unit Cost (\$)	Subtotal (\$)
General Site Work				
Erosion Control	1	LS	100,000.00	100,000.00
Clearing and Grubbing	1,050	acre	750.00	787,500.00
Reclamation	102	acre	5,700.00	581,400.00
			Subtotal	1,468,900.00
Seepage Management/Liner				
Dewatering	1	LS	500,000.00	1,000,000.00
Foundation Preparation	4,874,500	SY	0.20	974,900.00
Reservoir Liner	2,148,000	CY	2.50	5,370,000.00
Cover material	4,3000,000	CY	1.50	6,450,000.00
			Subtotal	13,794,900.00
Embankment				
Zone 1 (low perm.) Embankment Fill	1,210,000	CY	2.50	3,025,000.00
Zone 2 Granular Fill	473,000	CY	2.00	946,000.00
Filter Sand	145,000	CY	22.00	3,190,000.00
Groundwater Drain	7,350	LF	67.00	492,450.00
Instrumentation	1	LS	350,000.00	350,000.00
			Subtotal	8,003,450.00
Slope Protection				
Drainage Gravel	100,000	CY	36.00	3,600,000.00
Soil-cement Plating	78,700	CY	47.00	3,698,900.00
Soil-cement Stair-step	67,000	CY	47.00	3,149,000.00
			Subtotal	10,447,900.00
Plum Creek/Unnamed Tributary				
Plum Creek Modifications	1	LS	2,075,000.00	2,075,000.00
Unnamed Tributary	1	LS	483,000.00	483,000.00
			Subtotal	2,558,000.00
Inlets and Outlets				
Area 1 Inlet	1	LS	915,657.00	915,657.00
Area 1 Outlet	1	LS	1,505,650.00	1,505,650.00
Area 2 Inlet	1	LS	839,435.00	839,435.00
Area 2 Outlet and Bridge	1	LS	1,626,150.00	1,626,150.00
New Check Structure	1	LS	250,000.00	250,000.00
			Subtotal	5,136,892.00
Phelps Canal				
Bridge Modifications	3	EA	350,000.00	1,050,000.00
Siphon	1	EA	1,324,000.00	1,324,000.00
Canal Freeboard	225	CY	27.00	6,075.00
Raise Flume	1	LS	160,000.00	160,000.00
			Subtotal	2,540,075.00
Base Construction Cost (BCC)				43,950,117.00
Mob/Demob (1.5% of BCC)				659,251.76
Bonds/Insurance (1% of BCC)				439,501.17
			Subtotal	1,098,752.93
Direct Construction Cost (DCC)				45,048,869.93
Allowances				
Construction Contingencies (20% of DCC)				9,009,773.99
Final Design and Engineering (7% of DCC)				3,153,420.89
Owner Administration (2% of DCC)				900,977.40
Construction Engineering (8% of DCC)				3,603,909.59
Permitting and Environmental Mitigation (2% of DCC)				900,977.40
			Subtotal	17,569,059.27
Grand Total				62,617,929.20

Note:

- Does not include any costs for land acquisition.

9.2 Operation and Maintenance Costs

RJH developed an opinion of probable operations and maintenance (O&M) costs based on consideration of the conceptual design and on our experience with other dam and reservoir projects in the United States. RJH considered that for planning purposes, O&M activities could generally be divided into three categories:

- **Routine O&M.** The routine activities would include regularly-scheduled inspections of the reservoir facilities; implementation of mowing, vegetation management, and animal control; and data collection and reduction from Project instrumentation.
- **Annual O&M.** The annual activities would include more detailed inspections, maintenance activities needed to fulfill requirements of dam safety regulators (i.e., FERC or State of Nebraska), and maintenance activities recommended from equipment manufacturers. These activities would likely include a walking inspection with regulators of the embankments, downstream toe, and hydraulic facilities; gate and gate hoist maintenance (i.e., painting, greasing, etc.); and minor earthwork and other repairs (i.e., patching of concrete structures, clean-out of buried drain pipes, instrumentation repairs, grading to repair small erosion rills or crest ruts, etc.).
- **Special/Unusual Event Inspections.** The special or unusual event inspections would be required following large storm events where high flows occurred in any of the three adjacent drainages, canal flows exceeded design flows, the first few SDHF releases, earthquakes, or other similarly unusual events. RJH only included a small cost for inspections, but did not include maintenance or repair costs for these unusual events.

Our cost opinion was developed considering the cost of labor, equipment, and supplies. We estimated labor costs by considering the percentage of labor needed from a full-time staff to accomplish the anticipated O&M tasks. We considered the annual cost for one full-time staff to be \$100,000, which is the total cost for employment, not the base salary of the employee. We also considered that the staff would be about 70 percent utilized in performing and coordinating routine and annual O&M tasks. We developed cost opinions for equipment and supplies considering the cost of the base equipment, the typical difficulty and frequency of the maintenance activities (i.e., mowing, minor grading, etc.), and the anticipated supplies or components expended in performing the maintenance. RJH assumed that the typical design life for the primary components of the reservoir system (i.e., slope protection, gates, concrete structures, channel lining, drain pipes, etc.) would be at least 50 years.

RJH's opinion of probable annual O&M costs is presented in Table 9.2.

TABLE 9.2
OPINION OF ANNUAL O&M COSTS

Item	Estimated Cost (\$)
Routine O&M	98,000
Annual O&M	40,000
Special Inspections	2,000
Total	140,000

SECTION 10 - SEEPAGE COLLECTION AND PUMP SYSTEM ALTERNATIVE

10.1 General

RJH considered that a key component of the total Project costs was the liner system selected to manage seepage. Therefore, RJH developed an alternative concept to manage seepage that included a seepage collection and pump system (SCPS). The objectives of RJH's work were to evaluate if an SCPS could safely manage seepage and if so, develop a cost opinion for the alternative to compare its costs with the clay liner concept. This section provides information about our evaluation, design concept, and cost opinion for the SCPS alternative.

The general concept of a SCPS includes the following;

- Constructing a perimeter trench on the downstream side of the dam embankments and collecting seepage in a filter-protected slotted pipe installed in the trench.
- Conveying collected seepage through the slotted pipes to nearby pump vaults.
- Pumping the collected seepage to a discharge location.

This concept allows seepage to exit the reservoir uncontrolled through the natural soil deposits but collects the seepage below the ground such that the groundwater level on the downstream side of the dam is maintained too low for seepage to exit to a free face with any significant energy that could result in development of piping.

A schematic diagram showing the concept of a SCPS is shown on Figure 10.1. The conclusions of our evaluation are that although the SCPS could be technically feasible, the concept is inherently more risky than the liner concept. Our OPPC are similar to the clay liner concept. Therefore, RJH recommends that a SCPS concept not be considered further.

10.2 Evaluation of Technical Feasibility and Risks

RJH relied on 2-dimensional seepage modeling and engineering experience to develop the concept of a SCPS. Our seepage models were developed using three of the cross sections developed to evaluate the clay liner concept: the Area 2 profile, the Area 1 south profile, and the Unnamed Tributary profile (refer to Figure 6.1). Analyses were

performed to evaluate the sensitivity of downstream seepage energy and volumes to modest variations in material properties. RJH modeled the following two scenarios:

- A condition where the clayey foundation soils have an average vertical permeability of 1.0×10^{-6} cm/sec (about one-half order of magnitude more than the compacted clay liner).
- A condition where the clayey foundation soils have an average vertical permeability of 1×10^{-5} cm/sec. This generally would represent a condition where about 1 percent of the reservoir bottom consisted of the sandy foundation soils near the surface.

Based on the model results, RJH concluded that the concept could be feasible from a technical standpoint. Based on consideration of the concept, RJH also concluded that there would be more inherent risk with this concept relative to the clay-lined reservoir concept in design, construction, and operation.

The risks inherent in the SCPS and methods to manage those risks include:

- The layout and size of the SCPS components are highly dependent on the subsurface conditions and could change significantly based on minor variations in subsurface materials. To partially address this risk during design, an extensive geotechnical exploration would be required to develop a thorough understanding of not only the general subsurface conditions, but also of the potential for minor discontinuities and relatively small pervious zones. Even with an extensive geotechnical exploration program, it is improbable that all discontinuities and seams of pervious materials would be identified. Consequently and to further manage the risk in design, large safety factors and conservative assumptions would be appropriate. Additionally, at the conceptual development stage, there is risk for significant changes to the trench depths, pipe sizes, or vault sizes that could result in large cost increases for the Project. Therefore, a larger conceptual-level contingency was used for this SCPS relative to other concepts.
- During construction, extensive dewatering would be needed to construct the trench and drain below the groundwater levels. Based on our experience, estimating the required work and costs to effectively manage groundwater is difficult and often results in contractor claims for changed conditions. Provisions in the specifications would be needed to help manage that risk; however, it is unlikely that specification provisions would completely eliminate the risk inherent in extensive dewatering programs.

- The SCPS relies on electrical power to pump collected seepage and maintain lowered groundwater levels. During operation if electrical power is lost or a pump stops working, the seepage conditions would immediately begin to progress toward being unsafe. This risk associated with stopped pumps could be managed with redundant pumps and backup power systems. Additionally, consistent maintenance and some repairs would be needed for an SCPS to remain functional for the life of the Project.

10.3 Description of the RJH Concept

RJH developed a concept for a SCPS based on the available subsurface information and preliminary seepage analysis. As discussed above, there is a relatively high risk that the sizes and layout of the components would need to change as more information becomes available. A plan of the SCPS concept is shown on Figure 10.2. A typical section through the embankment and foundation is shown on Figure 10.3. The concept includes a 4-foot-wide seepage collection trench that varies from about 16 to 20 feet deep and is generally located about 25 to 30 feet from the downstream toe of the dams. The seepage collection trench would be about 25,000 feet long and include a filter-gravel backfill and a 12-inch-diameter slotted pipe. The pipe would collect seepage and convey it to one of 12 pump vaults. Each pump vault would consist of a two-level, below-grade, reinforced concrete structure. The pump vaults would be about 10 feet wide by 20 feet long by 24 feet deep. This relatively large footprint was selected based on the consideration that the cost for additional dewatering, deeper excavations, and deeper vaults would exceed the cost of the relatively wide and shallow vault. The lower level would be a wet well and house a submersible pump. The upper level would facilitate pump maintenance. Figure 10.4 shows the concept of a typical pump vault.

The pumps could discharge the collected seepage either back to the reservoir or to an adjacent drainage. For cost estimating, RJH considered that a 16 horsepower pump could adequately manage the volume of anticipated seepage; however, significantly more data would be needed to refine this concept during design.

10.4 Opinion of Probable Project Costs for Alternative

RJH developed a conceptual-level OPPC for this alternative based on the conceptual design presented above and the OPPC presented in Section 9. RJH used the methods described in Section 9, except that an additional contingency of 10 percent, which resulted in a total of 30 percent, was applied to the SCPS components. The higher contingency was included to account for the higher degree of uncertainty for this

alternative and the high risk for cost increases associated with design development and construction.

RJH's OPPC when the SCPS is used instead of a clay liner to manage reservoir seepage is \$58.1 million. Although this is about \$4.5 million less than the clay liner concept it does not include the additional operation and maintenance costs. We estimate that the annual costs for operation and maintenance, which includes electric power, repair and replacement of pumps, inspection, etc. would be on the order of \$70,000 per year. This O&M cost would be in addition to the O&M costs for the dams and reservoirs as presented in Section 9.2. If you consider the present value of this annual cost over a 50-year period, the cost of this alternative would increase by approximately \$3.5 million (assuming an effective interest rate of 0 percent). Therefore, the combined operation and maintenance and capital costs are about \$1.0 million lower than the clay liner concept. Our OPPC for the primary Project elements are shown in Table 10.1.

The risks for cost increases for the SCPS alternative during later stages of design are greater than for the clay liner alternative. Also, the technical and performance risks are higher for the SCPS alternative because it relies on an active mechanical system to maintain safe performance. It is our opinion that this alternative should not be considered further.

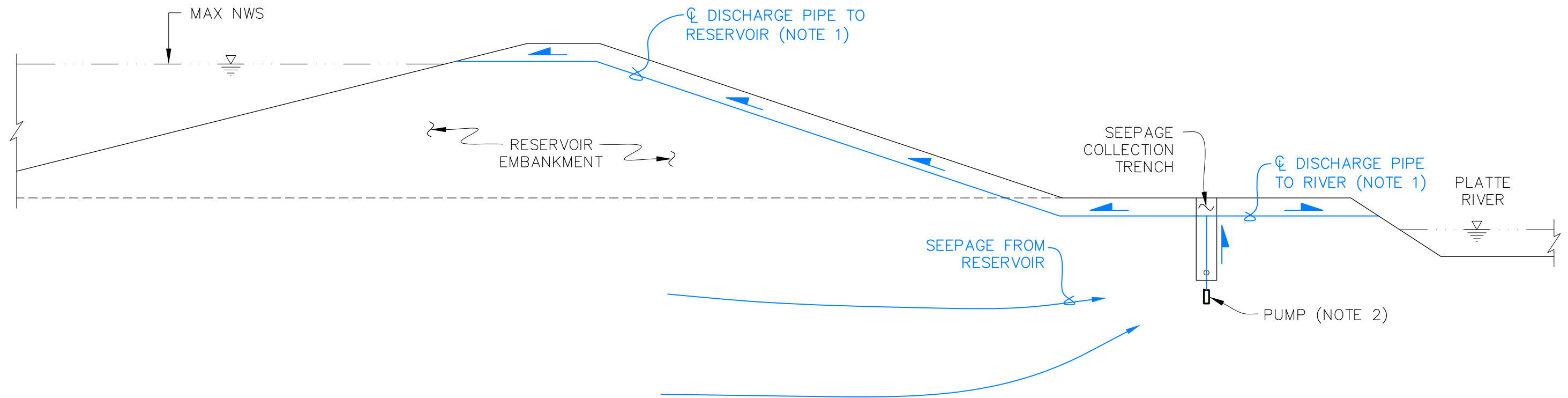
TABLE 10.1
RJH OPINION OF PROBABLE PROJECT COSTS FOR SCPS ALTERNATIVE

Reservoir and Embankments	Quantity	Unit	Unit Cost (\$)	Subtotal (\$)
General Site Work				
Erosion Control	1	LS	100,000.00	100,000.00
Clearing and Grubbing	1,050	Acre	750.00	787,500.00
Reclamation	102	Acre	5,700.00	581,400.00
			Subtotal	1,468,900.00
Seepage Management				
Pumpback				
Dewatering	1	LS	1,550,000.00	1,550,000.00
Foundation Preparation	4,874,500	SY	0.20	974,900.00
				2,524,900.00
Collection Trench				
Scraper Excavation/Backfill	217,700	CY	2.00	435,400.00
Trench Excavation	32,000	CY	4.00	128,000.00
Filter Material	5,800	CY	25.00	145,000.00
Low Permeable Cap	6,200	CY	2.50	15,500.00
Subdrainage Pipe	27,207	LF	25.00	680,175.00
				1,404,075.00
Pump Station				
Fill	42,670	CY	5.00	213,350.00
Concrete	1,450	CY	600.00	870,000.00
Pumps	24	EA	5,500.00	132,000.00
Electrical and Instrumentation	1	LS	4,500,000.00	4,500,000.00
				5,715,350.00
Pumpback System Subtotal				9,644,325.00
Pumpback System Additional Contingency (10% of Subtotal)				964,400.00
			Subtotal	10,608,725.00
Embankment				
Zone 1 (low perm.) Embankment Fill	1,210,000	CY	2.50	3,025,000.00
Zone 2 Granular Fill	473,000	CY	2.00	946,000.00
Filter Sand	145,000	CY	22.00	3,190,000.00
Groundwater Drain	7,350	LF	67.00	492,450.00
Instrumentation	1	LS	350,000.00	350,000.00
			Subtotal	8,003,450.00
Slope Protection				
Drainage Gravel	100,000	CY	36.00	3,600,000.00
Soil-cement Plating	78,700	CY	47.00	3,698,900.00
Soil-cement Stair-step	67,000	CY	47.00	3,149,000.00
			Subtotal	10,447,900.00
Plum Creek/Unnamed Tributary				
Plum Creek Modifications	1	LS	2,075,000.00	2,075,000.00
Unnamed Tributary	1	LS	483,000.00	483,000.00
			Subtotal	2,558,000.00
Inlets and Outlets				
Area 1 Inlet	1	LS	915,657.00	915,657.00
Area 1 Outlet	1	LS	1,505,650.00	1,505,650.00
Area 2 Inlet	1	LS	839,435.00	839,435.00
Area 2 Outlet and Bridge	1	LS	1,626,150.00	1,626,150.00
New Check Structure	1	LS	250,000.00	250,000.00
			Subtotal	5,136,892.00
Phelps Canal				
Bridge Modifications	3	EA	350,000.00	1,050,000.00
Siphon	1	EA	1,324,000.00	1,324,000.00
Canal Freeboard	225	CY	27.00	6,075.00
Raise Flume	1	LS	160,000.00	160,000.00
			Subtotal	2,540,075.00
Base Construction Cost (BCC)				40,763,942.00
Mob/Demob (1.5% of BCC)				611,459.13
Bonds/Insurance (1% of BCC)				407,639.42
			Subtotal	1,019,098.55
Direct Construction Cost (DCC)				41,783,040.55
Allowances				
Construction Contingencies (20% of DCC)				8,356,608.11
Final Design and Engineering (7% of DCC)				2,924,812.84
Owner Administration (2% of DCC)				835,660.81
Construction Engineering (8% of DCC)				3,342,643.24
Permitting and Environmental Mitigation (2% of DCC)				835,660.81
			Subtotal	16,295,385.81
Grand Total				58,078,426.36⁽¹⁾

Note:

1. Does not include any costs for land acquisition.

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_10-1-SCPS_SCHEMATIC.DWG 4/10/2013 12:03 PM



NOTES:

1. PUMP STATION WOULD DISCHARGE TO THE RESERVOIR OR THE RIVER.
2. PUMP STATION NOT SHOWN.

THIS DRAWING MUST BE REPRODUCED IN COLOR.



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

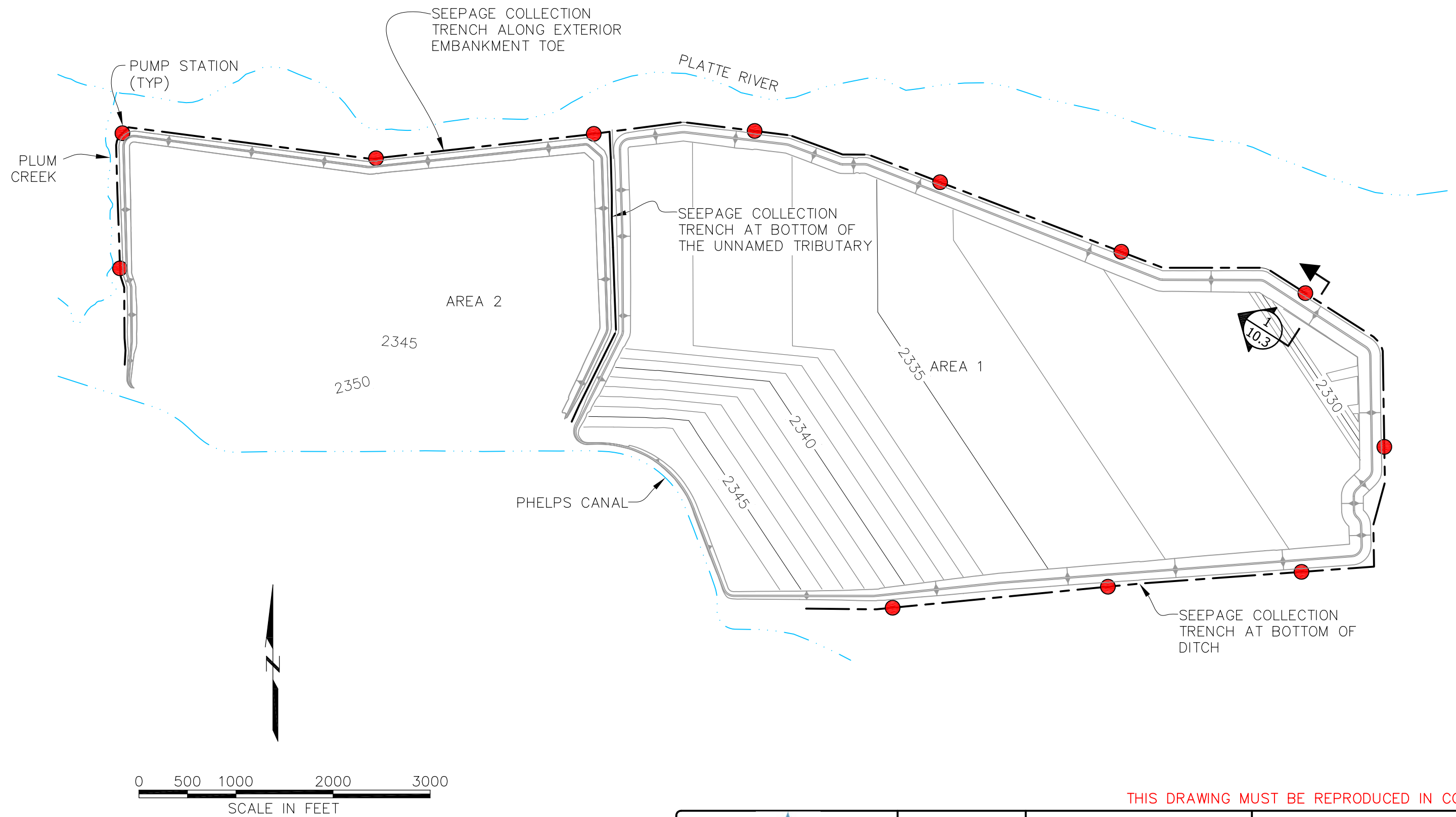
PROJECT NO. 12116

SEEPAGE COLLECTION AND
PUMP SYSTEM (SCPS)
SCHEMATIC

April 2013

Figure 10.1

P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_10-2-SCPS_ALT_PLAN.DWG 4/10/2013 12:05 PM



THIS DRAWING MUST BE REPRODUCED IN COLOR.



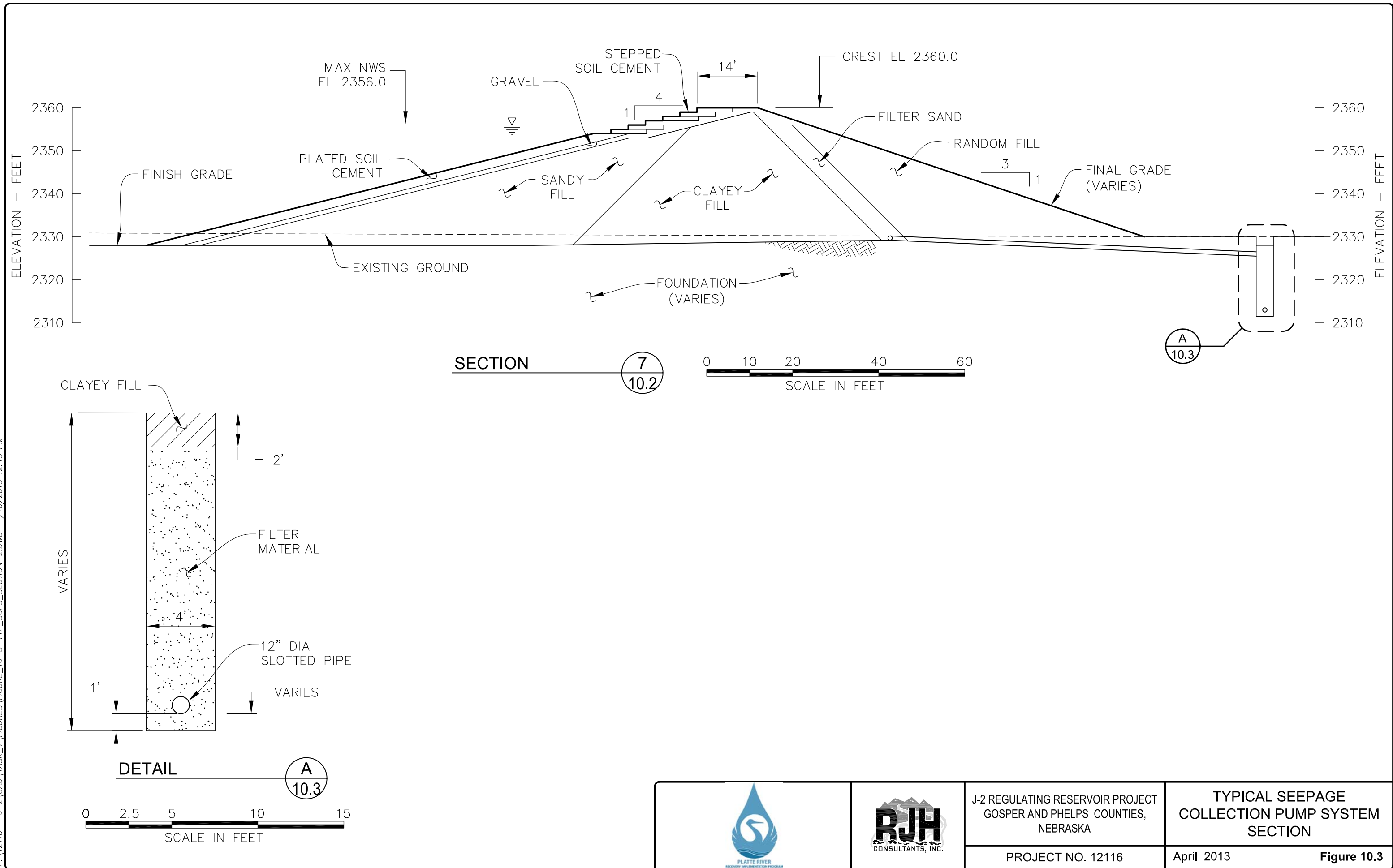
J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

PROJECT NO. 12116

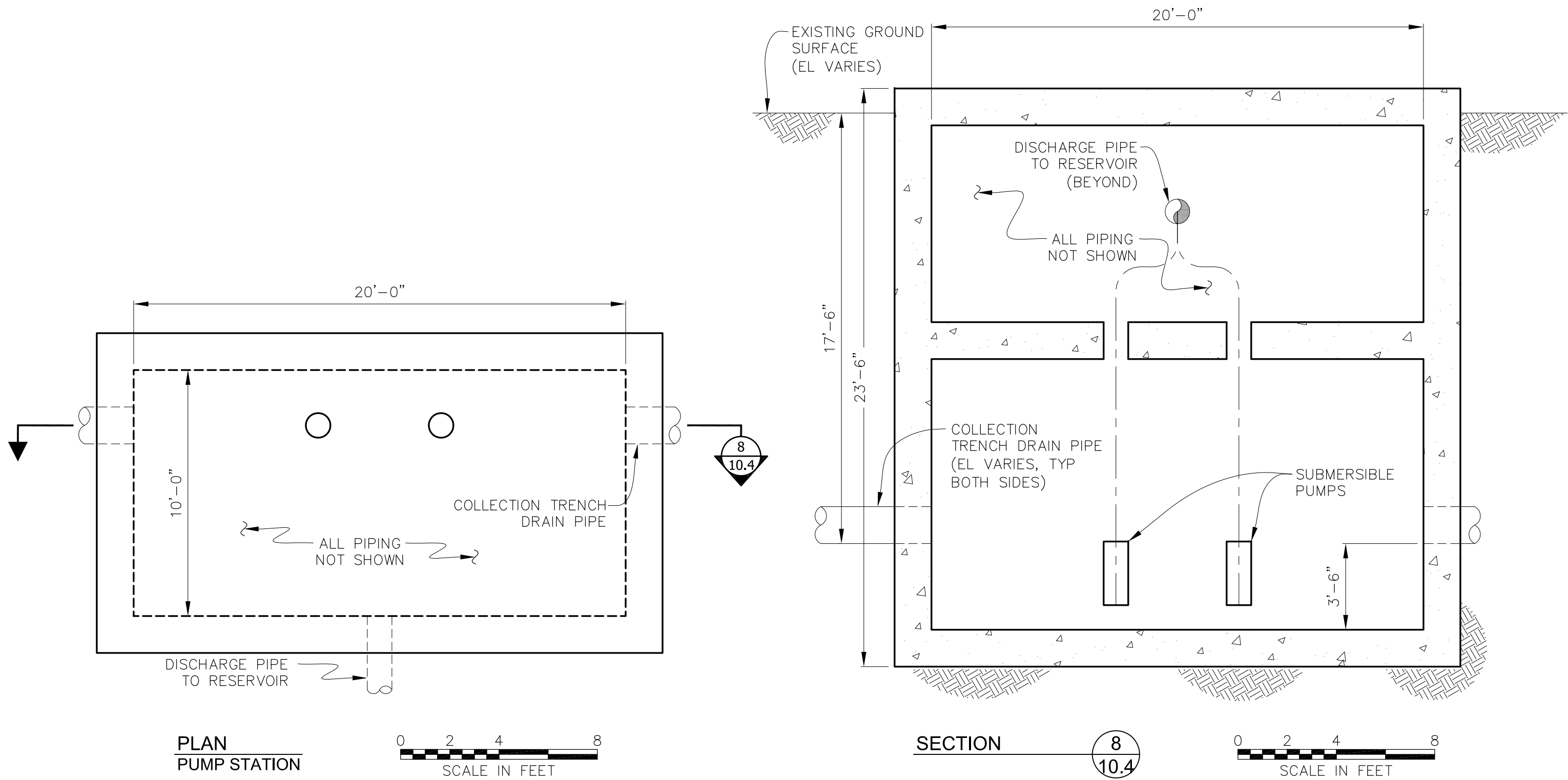
SCPS ALTERNATIVE
CONCEPTUAL PLAN

April 2013

Figure 10.2



P:\12116 - J-2\CAD\TASK_7\FIGURES\FIGURE_10-4-TYP_PUMP_STATION.DWG 4/10/2013 12:17 PM



J-2 REGULATING RESERVOIR PROJECT
GOSPER AND PHELPS COUNTIES,
NEBRASKA

PROJECT NO. 12116

TYPICAL PUMP STATION

April 2013

Figure 10.4

SECTION 11 - LIMITATIONS

The information presented in this report is suitable for conceptual design purposes only. The information in this report is based primarily on data obtained from review of existing documents, data, and studies for the subject site. Significant additional data is needed to refine the concepts in this report. Also, the nature and extent of variations between specific subsurface data may not become evident until future phases of exploration and construction. Timely and comprehensive observation and evaluation of actual subsurface conditions, supported by appropriate field and laboratory testing, will be critical during future design and construction phases. Variations in the subsurface profile described herein should be anticipated.

RJH has endeavored to conduct our professional services for this Project in a manner consistent with a level of care and skill ordinarily exercised by members of the engineering profession currently practicing in Nebraska under similar conditions as this project. RJH makes no other warranty, expressed or implied.

Opinions of probable Project costs presented in this report are based on our professional opinion of the cost to construct the Project as described in this report. The estimated costs are based on the sources of information described herein, and our knowledge of current construction cost conditions in the locality of the Project. Actual Project construction costs are affected by a number of factors beyond our control. Therefore, conditions and factors that arise as Project development proceeds through design and construction may result in construction costs that differ from the estimates documented in this report.

This report has been prepared for use by the Nebraska Community Foundation and the Program and for exclusive application to the J-2 Project.

SECTION 12 - REFERENCES

- Applied Weather Associates (2008). *Site-Specific Probable Maximum Precipitation (PMP) Study for Nebraska*. Submitted to Lower Platte North (NRD), December.
- Dreeszen, V.H., Reed, E.C., Burchett, R.R., and Prichard, G.E. (1973). *Bedrock Geologic Map Showing Thickness of Overlying Quaternary Deposits, Grand Island Quadrangle, Nebraska and Kansas*. U.S. Geological Survey Miscellaneous Geologic Investigations Map I-819.
- EA Engineering (EA) (2012). *Pilot Scale Recharge Report for Nebraska Groundwater Recharge Feasibility Study Platte River Recovery Implementation Program*, July.
- Federal Emergency Management Agency (FEMA) (1984). *Flood Insurance Study: City of Kearney, Nebraska*.
- Federal Emergency Management Agency (FEMA) (2008). *Flood Insurance Rate Map: Phelps County, Nebraska*.
- Federal Emergency Management Agency (FEMA) (2011a). *Flood Insurance Rate map: Dawson County, Nebraska*.
- Federal Emergency Management Agency (FEMA) (2011b). *Flood Insurance Study: Dawson County, Nebraska*.
- Nebraska Department of Roads (NDOR) (2005). Regression Equations, Transportation Research Studies, University of Nebraska, Lincoln for Nebraska Department of Roads, NDOR Research Project Number SPR-1(2) p 541, August.
- Olsson and Associates (Olsson) (2012). *CNPPID J-2 Reregulating Reservoir Feasibility Report*. Submitted to the Platte River Recovery Implementation Program, May.
- Platte River Recovery Implementation Program (PRRIP) (2009). *Data Analysis Summary Report for May 2008 Natural High Flow Event*. Submitted to the Governance Committee of the Platte River Recovery Implementation Program, January.

- Richmond, G.M. (1994). *Quaternary Geologic Map of the Platte River 4° x 6° Quadrangle, United States*. U.S. Geological Survey Miscellaneous Investigations Series Map I-1420.
- Schreurs, R.L. and Rainwater, F.H. (1956). *Geology and Ground-Water Resources of Buffalo County and Adjacent Areas, Nebraska, with a section on Chemical Quality of the Ground Water*. U.S. Geological Survey Water-Supply Paper 1358.
- U.S. Geologic Survey (USGS) (1999a). *Peak-Flow Frequency Relations and Evaluation of the Peak-Flow Gaging Network in Nebraska*.
- U.S. Geologic Survey (USGS) (1999b). *Water Resources Investigations Report 99-4032*.
- U.S. Geologic Survey (USGS) (2012a). *Stream Gage Data for the Platte River near Overton, Nebraska (Gage 0678000)*. Available online at <http://waterdata.usgs.gov>.
- U.S. Geologic Survey (USGS) (2012b). *National Water Information System – USGS Groundwater Data for Nebraska*. Available online at <http://nwis.waterdata.usgs.gov/nwis>.
- U.S. Geologic Survey (USGS) (2012). *Stream Gage Data for Plum Creek near Smithfield, Nebraska (Gage 06767500)*. Available online at <http://waterdata.usgs.gov>.